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The Role of Structural Design Decisions in Achieving Circular and Low-Carbon Buildings

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ABSTRACT

Achieving net-zero carbon targets in the built environment requires a fundamental shift towards low-carbon and circular design strategies. This paper introduces a comprehensive hierarchy that integrates core principles to support sustainable decision-making across both new and existing buildings. A key feature of this hierarchy is its clear distinction between circularity and carbon-reduction strategies, enabling transparent trade-offs and alignment with whole-of-life carbon goals. The hierarchy prioritises design strategies such as longevity, adaptability, and ease of disassembly, while also promoting minimal resource use, efficient material selection, and future-proofing of building systems. Digital innovations—including material passports and digital twins, aligned with Construction 4.0—are also embedded in the framework to enhance lifecycle performance and enable reuse or repurposing of components. A significant contribution of this work is its focus on quantifying the impact of key structural design decisions on whole-of-life carbon emissions. Through a detailed case study of a typical office building in Aotearoa New Zealand, the research demonstrates that implementing readily available, non-complex circular and low-carbon strategies can reduce the superstructure's whole-of-life carbon emissions by more than 50%, without requiring changes to structural systems or materials. The findings offer actionable insights for designers and engineers, highlighting the pivotal role of design choices in achieving carbon and circularity goals. This study provides a scalable, practical tool that supports the construction sector's response to climate change, resource scarcity, and the transition to a more sustainable built environment.

INTRODUCTION

Achieving a net-zero carbon future requires a transformative shift toward a circular economy (CE) that embraces regenerative and restorative practices by design (Andisheh, 2024). At the core of this transition is low-carbon circular design, which replaces the traditional linear model

by prioritising resource efficiency, waste minimisation, and reductions in both upfront and whole-of-life carbon emissions (Shahmohammadi et al., 2025a). This approach includes strategies such as refurbishment, repurposing, and reuse of existing buildings, along with designing new structures for longevity, adaptability, and ease of deconstruction, while regenerating materials at end-of-life (Baker-Brown, 2019; Cheshire, 2021).

Although circularity and carbon reduction often support one another, tensions can arise—e.g., when circular features increase upfront carbon to deliver long-term benefits. Such trade-offs necessitate careful evaluation, particularly given the urgent need to reduce emissions to address the climate crisis. These themes are explored further in a circular and low-carbon design framework developed by the authors of this study.

Design decisions in the built environment are further complicated by multiple, sometimes conflicting, objectives and uncertainties. For instance, replacing circular materials with non-circular but lower-carbon alternatives can significantly alter structural performance, especially under severe seismic or fire events. Simplified solutions are therefore critical to enable wide, sustainable adoption of CE and low-carbon principles without requiring complex analysis.

Transparency in communicating the rationale and carbon impacts of design choices—particularly when trade-offs exist between circularity, upfront carbon, and whole-of-life carbon—is essential (LETI, 2021). Prior research supports the importance of structural design choices: Helal et al. (2022) found that plan irregularity in tall buildings can increase column-related embodied GHG emissions by 23%, and overall structural emissions by 3%. Andisheh (2025) demonstrated that reusable, low-damage seismic solutions can reduce whole-life carbon emissions in steel buildings. Shahmohammadi et al. (2025a) showed that efficient structural choices substantially lower upfront carbon, and the use of high-strength steel reduces both member size and embodied carbon (Mela and Heinisuo, 2014). Meanwhile, studies by Moynihan and Allwood (2014), Dunant et al. (2018), and Orr et al. (2011) revealed significant material inefficiencies due to overdesign, with Dunant et al. (2018) emphasising the role of serviceability assumptions in resource overuse.

Despite these insights, few studies have quantified the impact of design decisions on structural carbon. This research addresses that gap by demonstrating how readily available and non-complex circular and low-carbon strategies can reduce a building's superstructure whole-of-life carbon emissions by over 50%, without requiring changes to structural materials or systems. Through a detailed building case study, this paper offers actionable insights into the influence of design choices on carbon outcomes—supporting the advancement of more sustainable and resilient building practices.

PROPOSED LOW-CARBON AND CIRCULAR DESIGN HIERARCHY

Based on two decades of research, the proposed Low-Carbon and Circular Design Hierarchy offers a structured, practical framework to guide building designers in implementing circular economy and low-carbon strategies. Further detail is provided in the HERA R4-164 framework (Shahmohammadi et al., 2025a).

The Hierarchy synthesises key strategies from international models and academic literature (see Figure 1), and is structured into four interconnected components:

1. Circular Design Strategies for Greenfield Sites ("4D's") Represented as the left leg of the hierarchy, these strategies prioritise circularity for new developments, using a synergistic, ranked approach known as the "4D's".

2. Reuse and Recycling Strategies for Brownfield Sites ("4R's") The right leg targets sites with existing buildings, promoting reuse, refurbishment, and recovery through the "4R's" to retain embodied carbon and reduce demolition waste.

3. Low-Carbon Strategies ("2L's") Centrally placed, the "2L's" address both upfront and whole-of-life carbon, focusing on material efficiency, low-carbon alternatives, and structural optimisation.

4. Information Management and Digital Integration Bridging the hierarchy, this component highlights the role of Construction 4.0 technologies—such as BIM, digital twins, and robotics—in capturing and managing lifecycle data. Material passports are key tools in this approach, enabling traceability, transparency, and informed decision-making.

Together, these components offer an actionable pathway for embedding circularity and carbon reduction into design processes, supported by emerging digital technologies.

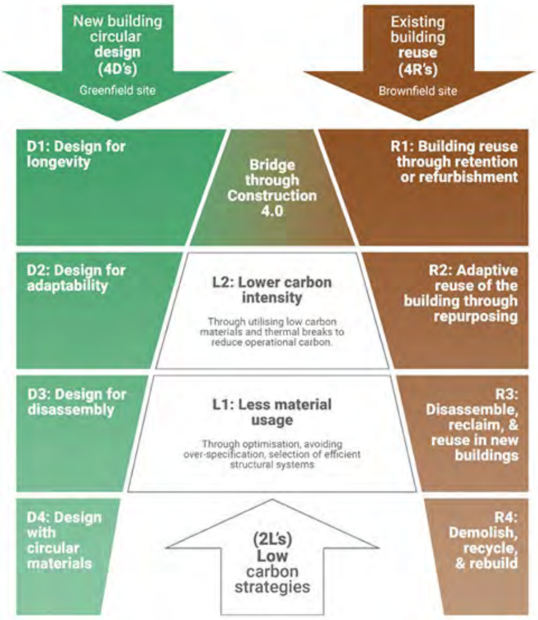

Figure 1: The HERA Low-carbon and Circular Design Hierarchy

BUILDINGS CASE STUDIES AND LIFE CYCLE ASSESSMENT

Building Case studies

To demonstrate the practical application of the proposed hierarchy for reducing carbon emissions and to highlight the impact of design decisions on whole-of-life carbon in structural building design, a real-world case study was selected. As illustrated in Figure 2, the case study involves a three-storey office building located in Christchurch, Aotearoa New Zealand. The building's lateral load-resisting systems comprise reinforced concrete shear walls in one direction and moment-resisting steel frames (MRSFs) in the perpendicular direction, designed for nominal ductility (ductility factor of 1.25). Steel–concrete composite floor systems are used throughout. Constructed in 2014, the building was designed in accordance with New Zealand Standards and reflects common engineering practices of that time.

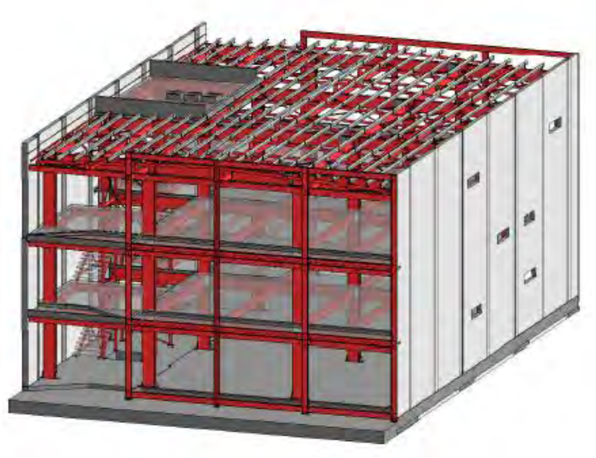

Figure 2: The low-rise building case study

Life Cycle Assessment

This study follows the guidelines outlined in ISO 14044 (ISO, 2006) and EN 15978 (CEN, 2011) for conducting carbon life cycle assessments (LCAs). The term *carbon* refers broadly to greenhouse gas (GHG) emissions—such as carbon dioxide, methane, and refrigerants—expressed in CO_2-equivalents (CO_2-eq) in accordance with EN 15978.

The LCA applies a Cradle-to-Cradle approach (Modules A–D) and evaluates the carbon footprint of building case studies across seventeen modules grouped into four life cycle stages. As the study focuses solely on superstructures, some life cycle modules are excluded from the analysis. Included stages are: material production (A1–A3), transport to site (A4), construction and onsite waste generation (A5), deconstruction and waste transport (C1–C2), waste

processing and disposal (C3–C4), and benefits from reuse or recycling at end of life (Module D). All assessed materials have service lives exceeding the 50-year period defined for the LCA. The analysis was carried out using a custom spreadsheet model developed by the author and validated across multiple scenarios against LCAQuick V3.6 (Dowdell et al., 2020). LCAQuick, developed by BRANZ, is a publicly accessible tool for estimating the environmental impacts of building designs.

The following end-of-life assumptions, consistent with current LCAQuick settings, were used for Modules C3, C4, and D:

- **Steel**: 85% of standard structural steel and 20% of reinforcing bar steel are assumed to be recycled. These recycling rates are lower than those in steel EPDs, likely leading to an underestimation of carbon benefits in Module D—especially for steel reinforcement in concrete.
- **Concrete**: End-of-life emissions are assumed to be zero, in line with current EPDs for concrete products in Aotearoa New Zealand, where Modules C and D have not yet been fully developed.

The functional unit for comparison is one square meter of Gross Floor Area (GFA) over a 50-year life span, applying the Cradle-to-Cradle approach. The GFA of 1,755 m^2 is based on the RICS (2015) definition of gross internal area, measured to the internal face of perimeter walls at each floor level. Further details regarding the Life Cycle Inventory (LCI) and Environmental Product Declarations (EPDs) used in this study can be found in HERA R4-166 (Shahmaohammadi et al., 2025b).

RESULTS AND DISCUSSIONS

Circular and Low-Carbon Strategies and Solutions for Achieving +50% Reduction in Whole-of-Life Carbon Emissions

Implementing solutions that alter structural systems, building systems, or structural materials can significantly affect structural performance. Therefore, advanced analysis is required to identify optimal solutions that balance cost, environmental impact, stakeholder requirements, and building needs. To facilitate practical implementation and avoid the complexities associated with such advanced evaluations, this study applied readily available circular and low-carbon strategies and solutions aimed at achieving a 50% reduction in whole-of-life carbon emissions. The criteria for selecting these strategies and solutions were:

- Retain the existing materials and structural systems for the building superstructure to achieve significant carbon reduction;
- Minimise the replacement of materials and systems to maintain progress towards net-zero goals;
- Ensure the solutions are applicable and implementable in the New Zealand context.

Accordingly, a combination of circular strategies (e.g., design for disassembly) and low-carbon strategies (e.g., use of low-carbon materials) was adopted to reduce the whole-of-life carbon emissions of the building's superstructure. Table 1 presents the life cycle assessment (LCA) results for the reference superstructure, including the effects of applying the selected strategies. The total non-biogenic carbon emissions of the reference building superstructure were 248 $kgCO_2e/m^2$. Using reversible connections in flooring systems to enable disassembly and reuse of steel beams and concrete slabs achieved a 5% reduction. Substituting conventional materials with low-carbon alternatives led to further reductions—8% for concrete, 16% for reinforcing bars, and 17% for structural steel. These cumulative measures resulted in a 51% total reduction in whole-of-life carbon emissions. Importantly, the implementation of these solutions does not compromise structural performance, operational carbon, fire resistance, or durability. Additionally, no negative impacts on other building systems or life cycle modules were observed when compared with the reference building.

Table 1: LCA results for the superstructure of the reference building with circular and low-carbon strategies for +50% carbon reduction

Strategy	Solution	Superstructure carbon emission (kgCO2eq/m2)						
		Life cycle modulus			Total (non-biogenic)	Biogenic	Carbon reduction %	Cumulative Carbon Reduction %
		A	C	D				
Reference Building		377	14	-143	248	0	N.A.	**N.A.**
Design for disassembly	Reversible connection in flooring systems	377	13	-153	237	0	5	**5**
Low Carbon Intensity	Specify Low-carbon concrete	356	14	-169	201	0	8	**13**
	Specify Low-carbon structural steel	166	17	-33	150	0	21	**34**
	Specify Low-carbon reinforcing rebs	80	17	9	106	0	17	**51**

Impact of Building Structural Systems on Whole-Life Carbon

Selecting an appropriate structural system is a fundamental design decision that significantly affects the whole-of-life carbon emissions of a building's superstructure. Table 2 compares different lateral and gravity resisting systems, while maintaining a consistent structural ductility factor across all options. The gravity and flooring systems were also held constant. Replacing the reference building's lateral resisting system with an eccentrically braced frame (EBF) resulted in a 35% carbon reduction. Similarly, replacing reinforced concrete (RC) shear walls (used in one direction) with a moment-resisting steel frame (MRSF) led to a 34% reduction. However, replacing both the steel gravity and RC shear wall systems with a full RC gravity and lateral system (with RC walls and MRSF) increased the whole-of-life carbon by 53%. It should be noted that such a change may also affect the building's seismic and fire performance, which warrants further analysis.

Given the strong influence of structural systems on carbon performance, it is essential for designers to enhance their capability to assess both structural and environmental

performance. Close collaboration with architects and the project team from early design stages is critical for selecting efficient systems.

Table 2: Whole-life carbon emissions for superstructures with different lateral resisting systems

Type of structural systems	Superstructure carbon emission (kgCO2eq/m2)					
	Life cycle modulus			Total (non-biogenic)	Biogenic	Carbon changes %
	A	C	D			
Reference Building	377	14	-143	248	0	**N.A.**
EBF Building: Gravity resisting: Steel, Steel-concrete composite flooring	306	10	-155	161	0	**-35**
MRSF Building: Gravity resisting: Steel, Steel-concrete composite flooring	313	10	-159	164	0	**-34**
Concrete Building: Gravity: RC, Lateral: RC wall + RC MRF, Double Tee flooring	408	22	-158	379	-13	**+53**

Impact of Different EBF Structural Solutions on Whole-Life Carbon Reduction

This section evaluates how various structural design choices—such as increasing efficiency, enhancing element utilisation, and enabling reuse—affect the whole-life carbon of the reference building case study.

As shown in Table 3, replacing the lateral resisting system of the reference building with an eccentrically braced frame (EBF) designed for ductility level 3 results in a 35% reduction in whole-life carbon. Further carbon reduction of 7% is achieved through an optimised design that increases the capacity-to-demand (utilisation) ratio of EBF structural elements. Incorporating reversible connections to ensure that all EBF components can be reused leads to an additional 12% carbon reduction compared to EBF systems with non-reversible connections. Altogether, the combination of a high-performance lateral system (EBF with ductility 3), optimised element utilisation, and reusable connections enables a total whole-life carbon reduction of 54% compared to the original case study building.

Table 3 LCA results for different EBF-based structural solutions

Type of structural systems	Superstructure carbon emission (kgCO2eq/m2)					
	Life cycle modulus			Total (non-biogenic)	Biogenic	Carbon Reduction %
	A	C	D			
Reference Building	377	14	-143	248	0	**N.A.**
EBF Building: (replaceable shear link, ductility: 3)	306	10	-155	161	0	**35**
EBF Building: (replaceable shear link, ductility: 3, Optimised design)	263	9	-130	143	0	**42**
Reusable EBF Building: (replaceable shear link and structural elements, ductility: 3, Optimised design)	260	9	-156	113	0	**54**

The Impacts of Structural Ductility Factors on the Whole-Life Carbon of EBF Buildings

Table 4 presents the life cycle assessment (LCA) results comparing the whole-life carbon emissions of the superstructure of EBF buildings with different ductility factors, benchmarked against the reference building. The results indicate that an elastic EBF design (ductility 1) achieves a 27% reduction in whole-life carbon. Increasing the design ductility improves carbon performance:

- EBF with ductility 2 achieves a 42% reduction,
- EBF with ductility 4 achieves a 44% reduction compared to the reference building.

Comparing the outcomes of Tables 3 and 5 highlights the advantage of combining ductile design with reusability. While an elastic EBF (ductility 1) provides a 27% reduction, the reusable EBF design with ductility 3 achieves a 54% reduction—twice as much. This comparison underscores the significant whole-life carbon savings possible with a reusable EBF design compared to a traditional elastic approach.

Table 4: Whole-life carbon emissions of the EBF building superstructure for different ductility factors

Building description and design ductility	$\emptyset_{oms}$	Superstructure carbon emission (kgCO2eq/m2)					
		Life cycle modulus			Total (non-biogenic)	Biogenic	Carbon Reduction %
		A	C	D			
Reference Building	N.A.	377	24	-143	248	**0**	**N.A.**
EBF, ductility 1	N.A.	353	12	-183	182	**0**	**27**
EBF ductility 2	1.3	267	9	-132	144	**0**	**42**
EBF ductility 4	1.4	256	8	-126	138	**0**	**44**

N.A.: Not Applicable

To further explore the effect of ductility on carbon emissions, Table 5 and Figure 3 present a direct comparison between different EBF ductility levels. EBF designs with ductility 2 and 4 emit 21% and 24% less carbon, respectively, compared to the elastic EBF design (ductility 1).Figure 3 shows that transitioning from an elastic to a limited ductility design (ductility 2) leads to a 21% reduction in superstructure carbon emissions. However, increasing the ductility further to ductility 4 results in only an additional 3% reduction—highlighting diminishing returns in carbon reduction at higher ductility levels when reuse is not considered.

Table 5: Whole-life carbon emissions of the EBF building superstructure relative to elastic EBF

Building description and design ductility	$\emptyset_{oms}$	Superstructure carbon emission (kgCO2eq/m2)					
		Life cycle modulus			Total (non-biogenic)	Biogenic	Carbon Reduction %
		A	C	D			
EBF, ductility 1	N.A.	353	12	-183	182	**0**	**N.A.**
EBF ductility 2	1.3	267	9	-132	144	**0**	**21**
EBF ductility 4	1.4	256	8	-126	138	**0**	**24**

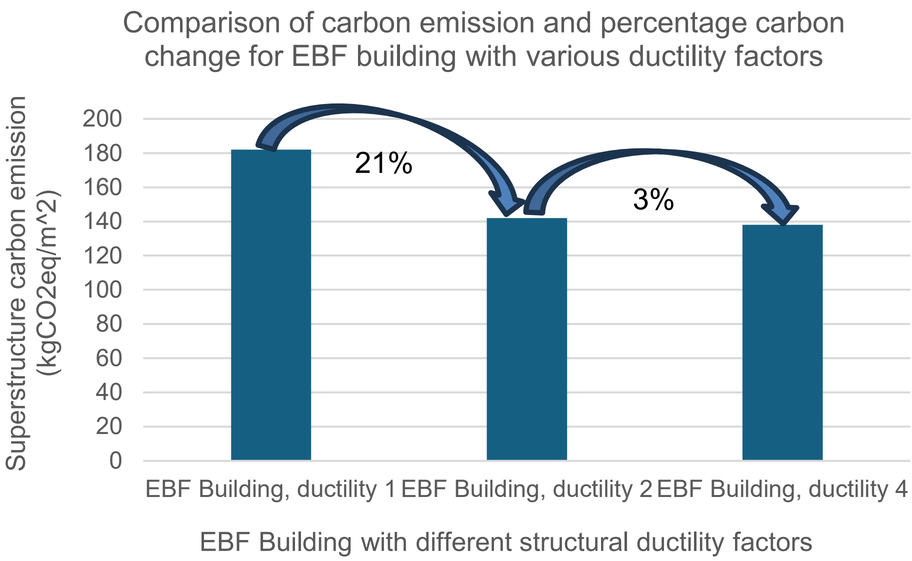

Figure 3: Whole-life carbon reduction of EBF buildings with varying ductility levels.

CONCLUSION

This study developed and evaluated a unified framework combining circular and low-carbon strategies for reducing whole-of-life carbon emissions in structural systems. The proposed approach emphasises non-complex and practical solutions applicable to New Zealand buildings without compromising performance. Key conclusions include:

- Implementing simple and readily available circular and low-carbon solutions can achieve a 51% reduction in whole-of-life carbon for conventional steel and concrete buildings—without altering structural systems or materials or compromising performance.
- Structural system selection is a critical design decision impacting carbon performance. However, when changing systems or materials, further evaluation is needed to assess the implications on structural, seismic, and fire performance. In complex cases, multi-objective or multi-criteria decision-making tools are essential to identify optimal solutions.
- Structural decisions such as the type of lateral system, utilisation ratios, and reusability significantly influence carbon outcomes. Combining efficient lateral systems, high utilisation ratios, and reusable components can reduce superstructure carbon emissions by over 50%.
- Structural ductility levels also affect carbon performance. Designs with limited or moderate ductility provide significant carbon savings over elastic systems. However, increasing ductility beyond a certain point yields diminishing carbon reduction benefits.

This work provides a pathway for structural engineers to play a more active role in decarbonising buildings through informed design decisions supported by LCA and circular design principles.

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Seismic Strengthening of the Heritage Category 1 Carillon Tower in Wellington, New Zealand

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ABSTRACT

The seismic strengthening of Wellington's Category 1 Heritage listed Carillon Tower to 100% New Building Standard (NBS) exemplifies how engineering innovation can safeguard cultural heritage for future generations. This paper presents the unique approach to strengthening the 1930s reinforced concrete structure, home to the world's third-largest carillon with 74 bronze bells, while maintaining its integral dual function as both a musical instrument and part of the National War Memorial site. The 50-meter tall structure's distinctive architectural feature - transitioning from a solid shear wall base to a more flexible cantilever wall system - while essential for sound propagation, posed a significant seismic vulnerability. The strengthening solution implemented viscous dampers within steel frames at the tower's apex, specifically engineered to match the building's deflection profile. This approach effectively reduced seismic forces while preserving the tower's crucial acoustic properties and heritage façade. This forward-thinking approach prioritizes sustainability through reduced embodied carbon and incorporates comprehensive corrosion protection strategies to ensure longevity in the maritime environment. The project demonstrates how careful consideration of a structure's historical, cultural, and functional significance can inform engineering decisions that will protect and preserve such monuments for future generations. The successful implementation of this strengthening strategy ensures this significant cultural landmark will continue to serve its commemorative and musical purposes for generations to come. Construction is well underway at the time of writing.

CARILLON HISTORY

Wellington’s Carillon was built in 1931 and opened on ANZAC day in 1932 as the first building on the National War Memorial site, in Mt Cook, Wellington. It was built as a national landmark and a memorial to New Zealand’s military sacrifices in the First World War. It was designed by

prominent architects Gummer and Ford who won a competition to design the Carillon Tower. Gummer and Ford also designed the Auckland Railway Station, Dunedin Cenotaph and the Bridge of Remembrance in Christchurch.

Figure 1: Carillon War Memorial opening, Wellington, Anzac Day 1932 (Crown Studios Ltd 1932).

A carillon is primarily a musical instrument. The Wellington carillon houses 74 bronze bells ranging from 12 tonnes to 4kg. The carillon was initially installed with 49 bells in 1932 and was extended over the years to have 74 bells – it is the third largest carillon in the world by weight (over 70 tonnes). The bells are housed in two bell frames, the upper and lower, and are played from a keyboard in the clavier chamber between the two frames (Figure 3). The bells are fixed in position into a headstock in the frame and the clapper for each bell is tied to a wire. The wire is connected to a keyboard in the clavier room and the instrument is played by fists and feet playing keys to pull the wires.

CARILLON STRUCTURE

The Carillon Tower is a 52m tall reinforced concrete tower. It is primarily solid concrete walls, approximately 380mm (15 inch) thick at the base. Part way up the tower, the solid concrete tubular wall construction transitions to cantilevered, perforated walls and columns at the top (Figure 2 & 3). This 'openness' is for optimal sound transmission throughout the Wellington central city area. The building has a footprint of approximately 131m^2 at the base (10.3m x 12.7m). The tower sits on shallow raft foundations and the original drawings for the building

show calculations for rocking behaviour. It appears that the foundations of the building were intended to rock on the weathered Greywacke soils below. There is a 100mm seismic gap between the tower and the adjacent Hall of Memories building. The carillon is an Importance Level 3 (IL3) structure to NZS1170:2002 due to it housing contents of high value to the community.

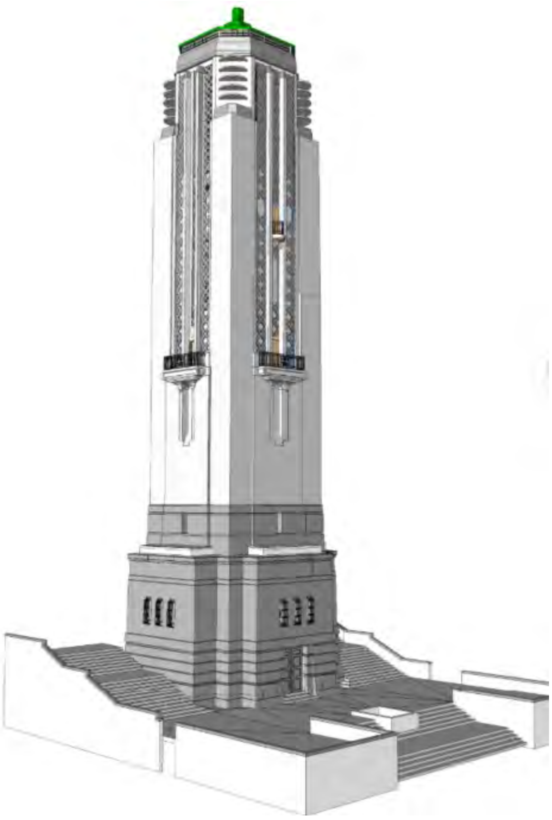

Figure 2: Structural model of the Carillon Tower

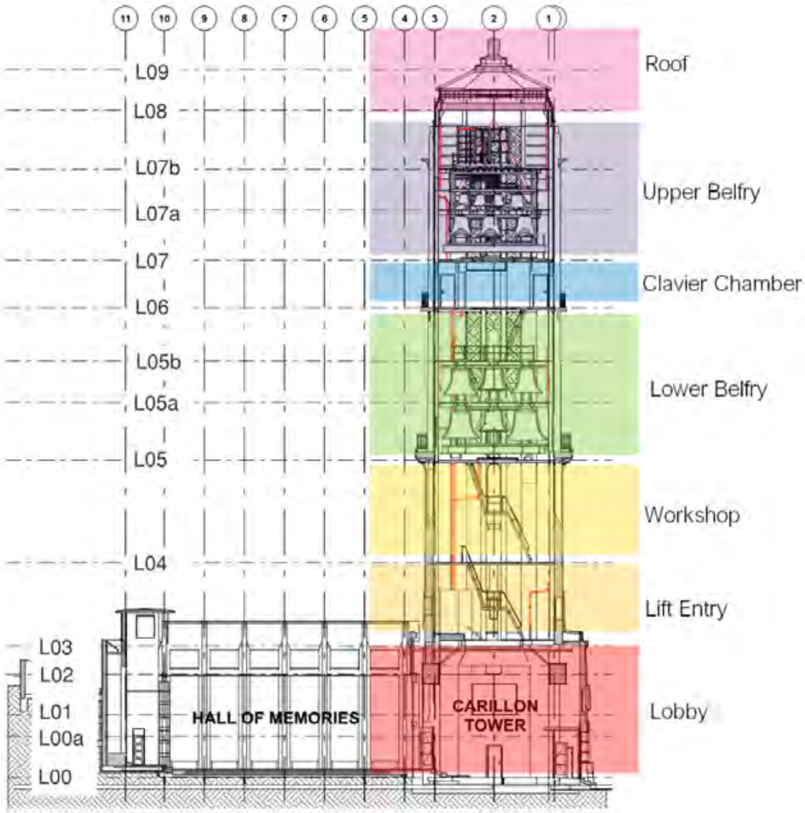

Figure 3: Section showing layout of the Carillon (Studio Pacific Architecture 2023).

The carillon has undergone various assessments and previous strengthening over the years. In the 1980's steel concentrically braced frames were installed at approximately the upper and lower bell frame levels to strengthen and stiffen the building. In 2012, strengthening and access upgrades to the upper levels of the tower were completed. A DSA was produced in 2020 which identified several elements with a capacity below 34%NBS. Holmes was asked to peer review this DSA, provide input and subsequently asked undertake the assessment and strengthening works for the building.

ASSESSMENT METHOD

The analysis model is built using ANSR, Holmes' in-house non-linear modelling software. The software comprises the non-linear analysis engine ANSR-II from (Mondkar & Powell, 1979) further developed with additional functionality and input and output processing. The program is based on a concentrated or lumped plasticity model.

Non-linear time history analysis gives us a detailed insight into the specific seismic vulnerabilities of the structure. This allows us to tailor the retrofit scope to the minimum required to achieve the earthquake strengthening target.

Material properties of reinforced concrete and steel bars in the model are based on the properties specified in the original structural specifications and supplemented by the NZSEE Assessment Guidelines (NZSEE 2017, NZSEE 2018). An eigen analysis of the modal was performed to determine the fundamental periods and mode shapes. The first modes for X and Z translation and Y rotation are shown graphically in Figure 4 below. Generally, the ground motion selection and scaling procedure is as per ASCE41-17. Near fault affects were also included in the scaling process.

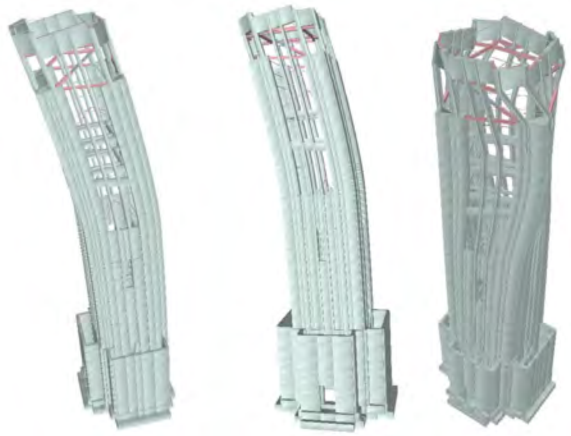

Figure 4: Fundamental modes of 0.85s X translation, 0.81s Z translation and 0.13s torsional translation (strengthened building).

Typically ASCE41-17 was used for defining the numerical acceptance criteria based on modelling parameters for the non-linear procedure for all the reinforced concrete walls and beams and NZ Assessment Guidelines Section C5 for concrete columns.

In general terms, we found that the building in its existing state had a capacity of approximately 34%NBS - beyond which significant structural failures cause the building to become structurally unstable. The main structural weaknesses with the building were found to be:

- Early onset of failure in the lower 1980's steel brace and existing concrete column system, with the steel braces driving large forces into the concrete elements.
- Relative displacement incompatibility between the upper steel braces and the concrete columns/walls at the top. The existing brace system stiffens the upper structure and forces the cantilever wall/column system into a stiff braced profile driving large forces into the concrete elements and overloading them.

CONCEPT STRENGTHENING SCHEMES

Using our detailed model, we were able to quickly iterate four strengthening schemes which were proposed to the client. These included a viscous damped (VD) solution in a moment resisting frame for 100%NBS IL3 performance, a space constrained BRB and post tensioned system, a lower performing moment resisting frame only and a more conventional, fully braced, steel internal frame. Advantages and disadvantages for all systems, along with cost estimates, informed the clients decision to proceed with the viscous damped with steel moment frame solution aiming for 100% NBS performance.

The tower had limited existing inherent damping, therefore additional damping in the form of viscous dampers was a great option especially given the large earthquake demands in Wellington. The viscous damped concept solution was costed at approximately half the cost of the conventional braced frame and post design analysis has determined that the viscous damped solution has approximately 30% of the embodied carbon when compared to the conventional fully braced solution. A 70% saving in embodied carbon is a major win for the project, along with the double advantage of reduced construction cost.

STRENGTHENING DESIGN

The viscous dampers are modelled in ANSR as simple velocity dependent only damper elements. Our ability to tune the damper design to meet the target performance level and then verify with the analysis model helped add to the eventual cost and embodied carbon savings.

The advantages of a viscous damped solution over a conventional braced frame solution are primarily the reduction of the forces transferred to the building below and that the viscous dampers do not significantly alter the natural displacement profile of the building meaning that they do not fight or stiffen the existing structure significantly. Overall, the building displacements, accelerations and forces are reduced when compared to the existing building and therefore the building performance increases. The moment resisting frame that the viscous dampers are housed in, also provides flexural strengthening to the concrete piers and the steel columns continue down below the point where the cantilever walls transition to solid walls

(Level 5). Figure 6 below shows the structural steel strengthening isolated from the structural model, Figure 7 shows an elevation of one face of the strengthening in the top half of the tower.

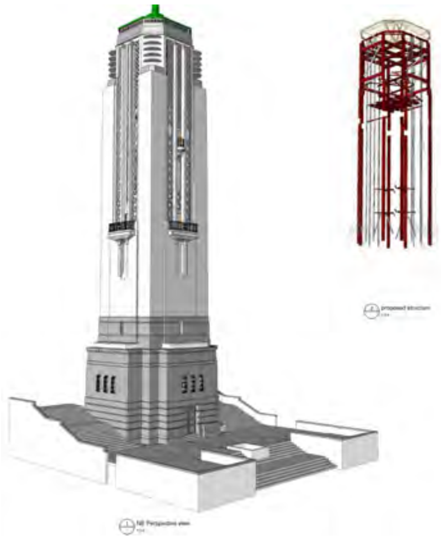

Figure 6: Structural steel added to building

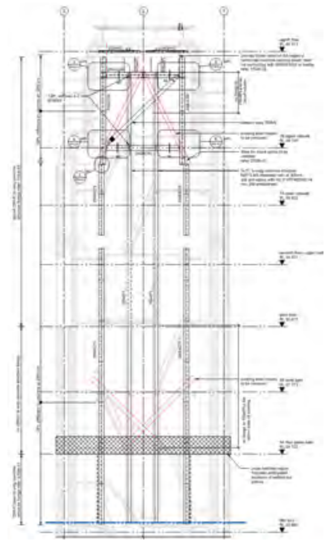

Figure 7: Top half of one face showing new VD moment frame (existing braces to be removed in red)

The NZ Building Code Verification Method B1/VM1 does not consider fluid viscous dampers. Consequently, the design was considered an alternative solution to the New Zealand Building Code, and as such required external peer review. The following compliance pathway was followed.

- Primary reference: ASCE7-16
- Supplementary references: ASCE41-17; NZSEE Assessment Guidelines (NZSEE, 2017) (NZSEE, 2018); and NZS 3404 (Standards New Zealand, 2007) for capacities of new steel elements
- Region-specific seismic hazard study (Bradley & Tarbali, 2017)
- Seismic demands on the tower have been computed through output from the nonlinear time history analysis. Parts & Components loading utilising modal accelerations and NZS 1170.5:2004 has been used for part elements of the structure, including the two bell frames

BELL FRAME STRENGTHENING AND CORROSION ISSUES

The Carillon Tower is located approximately 1km from the shore and the wind-blown salts deposited on the internal concrete and steel structure are unwashed by rain, creating a highly corrosive environment inside the tower. Steel members of the upper bell frame and parts of the upper steel braces were identified as suffering from widespread corrosion (Figure 8). These

frames have historically been recoated and members replaced, multiple times, with signs of corrosion reappearing much sooner than expected.

Concrete sections have also suffered from cracking and spalling across the height of the structure. A previous attempt to arrest corrosion of the reinforcing steel, utilising a cathodic protection system, had been unsuccessful when the titanium wires broke under internal wind loads. A specialist corrosion protection consultant was engaged to provide expert advice and specify steel coatings for all existing and new steelwork. Full sandblasting and recoating of the bell frames in a controlled environment will be employed. Additionally, concrete element corrosion will be mitigated with a maintenance regime and regular washdown of deposited salts.

The bell frames were also seismically strengthened using accelerations from the ANSR model at the height of the bell frame connections, and compared to loads using NZS1170.5 Section 8 for Parts. Due to the effects of the viscous dampers, the accelerations in the damped building were over 30% lower than those derived using NZS1170 Parts loads, which significantly reduced the strengthening required.

Figure 8: Upper bell frame - Locations of typical corrosion

Figure 9: Steel portal frame at Level 8 and double cleat for the viscous damper install.

CONSTRUCTION PROGRESS

The Ministry of Culture and Heritage awarded the construction contract to Maycroft Construction Limited in May 2024, and construction works are well underway.

The Carillon Tower as a construction site presents unique challenges for the installation of strengthening works. Internal access is provided by sets of access ladders, while a small, non-commercial lift exists, it only starts at Level 3 and reaches up to Level 7 and is not suitable for lifting any construction material. The only practical option of getting steel into the building is feeding the steelwork through small floor openings thus requiring small assemblies. Internal temporary works and rigging were installed by the Contractor, and the tower is fully scaffolded and wrapped externally.

The Contractor undertook 3-D scanning of the tower internally, which informed the construction methodology and proved valuable in the steel shop drawing process by identifying potential clashes and construction tolerances. Steelwork has been installed on site flawlessly to date (Figure 9).

CONCLUSIONS

The design of the strengthening for the Category 1 Heritage listed Carillon Tower in Wellington to 100%NBS (IL3) exemplifies how consideration of heritage and cultural value, and long term building function, can drive innovative solutions. Our non-linear analysis and design allowed a detailed insight into the buildings seismic vulnerabilities which enabled us to tailor the retrofit scope to the minimum required while achieving the clients performance target (100% NBS IL3), saving them money (over 50%) and providing a significant reduction in embodied carbon (70%) when compared to a more traditional, conventional strengthening scheme. The strengthening design for this building implemented viscous dampers into a steel moment frame at the top of the tower and steel column flexure strengthening to the upper level column/wall piers which reduced the design accelerations by 30% when compared to code Parts loads. The strengthening work means that this iconic building and musical instrument will be around for future generations to enjoy.

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Sensitivity: General

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A Structural Design Process for Low-Rise Buildings that incorporates seismic performance considerations for NSEs

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ABSTRACT

Historically, structural system options for low-rise buildings are considered alongside the architectural aesthetic and layout. During this structural optioneering phase there is usually little consideration given to the implications of the structural response on the components which provide building functionality. With most of our existing and future building stock being low-rise it is important that structural engineers understand that the structural response often dictates the seismic performance of non-structural elements. Structural engineers should work collaboratively with the design team to understand the extent and types of non-structural elements that will be included in the building and incorporate NSE response and performance limits into the design of the structure to achieve improved functionality and resilience for low-rise buildings.

The New Zealand Code of Practice for the Seismic Performance of Non-Structural Elements (NSE CoP) was first issued to industry in 2024 and provides guidance around the broader topic of seismic performance of structure and the impact of the structural response on the seismic performance of NSEs. This paper discusses various structural systems for low-rise buildings and how the response of those structural systems can impact the seismic resilience of the building. The paper explains how the NSE CoP can be used by structural engineers to support selection of the structural system to meet the client needs and the seismic performance and resilience expectations of building owners and tenants.

INTRODUCTION

Low-rise buildings dominate New Zealand's existing and future building stock, making their seismic resilience a critical area of focus for reducing earthquake-related risks. Improving the seismic performance of these structures is imperative not only to protect life safety but also to

ensure the continued functionality of buildings after seismic events. Several recent earthquakes in New Zealand – including the 2010–2011 Canterbury earthquakes, the 2013 Seddon earthquake, and the 2016 Kaikōura earthquake – resulted in a significant number of low-rise buildings being damaged and unable to be re-occupied due to failures and damage to non-structural elements (NSEs) (Dhakal, 2010; Baird, 2017). While this paper primarily focuses on new builds, many of the concepts discussed can also be applied when evaluating options to seismically strengthen existing low-rise buildings.

The structural design of both new buildings and the seismic strengthening of existing ones involve two key considerations:

1. Structural Response and NSE Performance: The response of the structure often dictates the seismic behaviour and performance of non-structural elements (NSEs), such as ceilings, partitions, and building services.
2. Structural Design for NSE Reactions: Structural systems must not only account for gravity and seismic actions imposed on the primary structure but also accommodate the reactions generated by the seismic response of NSEs.

Fortunately, structural engineers can provide significant enhancements to NSE performance with relatively minor shifts in their approach. By proactively accounting for NSE requirements and seismic response limits during the structural design phase, they can ensure better outcomes for overall building performance. This paper aims to assist structural engineers in achieving these improvements by drawing out key guidance from the 2024 New Zealand Code of Practice for the Seismic Performance of Non-Structural Elements (NSE CoP).

ROLE OF THE NSE COP

The New Zealand Code of Practice for the Seismic Performance of Non-Structural Elements (NSE CoP), released in 2024, offers structural engineers a comprehensive framework to integrate NSE considerations into building designs. In addressing the seismic performance of NSEs, the NSE CoP could have focused on numerous potential issues. After gathering industry feedback, key issues were identified as primary areas for focus, which align with the document's purpose above. These are explored further in the context of the NSE CoP's first version to industry, with future stages anticipated to broaden its scope. The main areas of focus are:

- **Roles, responsibilities and scope gap.** The NSE CoP encourages coordination among engineers, architects, and other stakeholders. Such collaboration ensures that structural designs are aligned with NSE constraints, promoting an integrated and efficient building process. It also defines key roles and responsibilities such that key design considerations are properly considered and coordinated.
- **Clarifying seismic performance objectives.** The NSE CoP provides guidance on seismic performance limits, such as acceptable movement, drift, and acceleration

levels for typical non-structural elements. This ensures that these components, which include mechanical systems and façade elements, maintain integrity and functionality during seismic events.

- **Provide a baseline for seismic performance.** The aim here is that by providing an industry baseline, not only will that assist in standardising approaches for better outcomes for NSEs, it will also foster innovation and research.

HISTORICAL STRUCTURAL SYSTEM OPTION DEVELOPMENT

Historically, structural system options for low-rise buildings have been evaluated primarily based on architectural aesthetics and layout, with minimal attention given to how the structural response impacts components critical to the building's functionality. This approach often overlooks the seismic demands placed on NSEs and their interactions with the structural system. Whilst NZS1170.0 does suggest drift limits for protection of some types of non-structural elements from seismic loading, they are optional and do not appear to be widely adopted in practice. Furthermore, as the design forces for restraint of parts and components is independent of the building characteristics according to NZS1170.5 (noting that loads can vary depending on building characteristics within TS1170.5), practitioners have been given little insight into the potential impact that structural design choices can have on the seismic performance of non-structural elements.

Compounding the problem is the limited early engagement of building services teams during the structural optioneering phase. When involved, building services often provide only basic placeholders, such as approximate locations for plant, equipment, and reticulation runs around floor plates, without detailed consideration of seismic demands. This disconnect makes it challenging to develop a structural system that incorporates the seismic response of the building holistically, whilst also ensuring that the structure is adequately designed for the seismic demands placed upon it by NSEs.

One common example is steel purlins in lightweight roof systems, which are typically designed to resist wind loads but often have limited additional capacity needed to support seismic loads from suspended services, such as HVAC units or fire sprinkler systems. This can lead to significant amounts of bridging being required to distribute loads from suspended services. These oversights can lead to significant challenges during construction and, more critically, compromise building performance during an earthquake.

To address these issues, structural engineers must adopt a more integrated approach during conceptual design, considering not just the primary structure but also how it will interact with NSEs under seismic loading. By engaging with architects and building services early and accounting for potential NSE loads and locations at the concept stage, engineers can create structural systems that enhance both the safety and functionality of low-rise buildings during and after seismic events.

HOW TO USE HOLISTIC PROJECT COORDINATION AND DESIGN TO IMPROVE SEISMIC PERFORMANCE

Structural engineers can work collaboratively with the design team to understand the extent and types of non-structural elements that will be included in the building. A good place to start is for all disciplines to complete the simple check box table provided in Appendix B-B2 of the NSE CoP, refer to Figure 1 below for a snip from that table. The purpose of this table is that at conceptual design phase the full design team (architect, building services, acoustic engineer, passive fire engineer, structural engineer, etc.) use this table to identify the key non-structural elements that will be included in the building. It is intended only to be a high-level identification of components and not design or confirmation of components.

Figure 1: Simple checkbox to complete to understand what NSEs are to be included in the building (excerpt from Table B-B2 of NSE CoP)

Discipline/Sub-Category	Component	Does this NSE exist in this building?	Is this NSE critical for operation or seismic performance of Blg?
ARCHITECTURAL COMPONENTS			
	Adhered veneer		
	Anchored veneer		
	Prefabricated concrete panels		
	Other cladding panels		
	Framed exterior wall systems		
	Glazed exterior wall system		
	Glass Blocks		
Interior Partitions	Heavy		
	Light		
	Glazed		
	Fire		
	Acoustic		
	Wet area - membraned		
Interior Veneers	Stone & tile		
Bathrooms	Toilets		
	Showers		
Ceilings	Suspended lay-in tile ceiling systems		
	Ceilings applied directly to structure		

Structural engineers can then use the outputs from this check-box exercise to understand what key non-structural components will be included in the building and where these are likely to be. They can then use the performance limits for varying performance criteria as provided in Part C of the NSE CoP for different non-structural elements to consider what type of structural system would best achieve the required drift and acceleration limits for the non-structural elements that will be incorporated into the building.

In addition to thinking about structural system lateral response, it is also important that the structural engineer knows where heavy plant is expected to be located and what type of non-structural components are likely to be suspended or braced from the roof structure. This information is critical to enable early adoption of a structural system that is fit for purpose which is cognisant of NSE response and performance limits. This will lead improved functionality and resilience for low-rise buildings and enable the building as a whole to achieve code compliance as well as any additional performance requirements provided by the client that are over and above the NZ Building Code performance requirements.

BUILDING MOVEMENT STRATEGY

The Building Movement Strategy is an important aspect of coordination and holistic design for every design project, whether a new build or existing building. Every building has movements that occur during the day-to-day functions of the building, as well as movements that will occur to the building and components during seismic shaking.

Work on the Building Movement Strategy should commence during the conceptual design phase. As described in the NSE CoP roles and responsibilities matrix during the design phase, the Building Movement Strategy should be led by the Design Manager, Architect and NSE Seismic Designer with secondary responsibility and coordination and input from the fire engineer, structural engineer, building services engineers. The Building Movement Strategy starts with discussion and coordination between all disciplines regarding expectations for building and component movements. In development of the Building Movement Strategy the design team should consider all aspects of building movement and document requirements for:

- Movement and gaps vs locations/zones where movement needs to be restricted/restrained
- Thermal movements
- Drift and seismic gaps

Currently, as the industry strives to satisfy multiple performance requirements, compromises may be required where thermal/acoustic/vibration/passive fire/structure drift requirements cannot all be met at given locations. In those locations the design team needs to work together to discuss and agree what aspects are to take precedence and which can be compromised and why the decision was made.

The Building Movement Strategy can be documented in a short 1-hour multi-discipline meeting. For larger projects the process can be expanded to a workshop or multiple meetings. The key is that the conversation is had and documented. The Building Movement Strategy should be discussed and updated at each phase of the project. Further information on the Building Movement Strategy can be found in the NSE CoP.

CONSIDERATIONS FOR SPECIFIC STRUCTURAL SYSTEMS

Table 1 below outlines key considerations for holistic design and coordination of non-structural elements (NSEs) for a number of specific structural systems. By addressing both the structural response and NSE interactions, this guide supports structural engineers in optimizing building performance to achieve improved seismic resilience and functionality.

In addition to the specific tabulated considerations, it should also be recognized that the following considerations could be relevant to the concept development of the structural system:

- Acceleration demands on mechanical equipment will tend to be lowest at the ground level. Consequently, mechanical and electrical equipment should be located at ground level where possible.
- Buildings with irregular primary structure configurations are likely to twist in earthquakes, amplifying demands on non-structural elements. Such amplification is not currently accounted for in the parts and components provisions for NZS1170.5 (nor TS1170.5) and thus, regular structural layouts that avoid or minimise torsional response should be encouraged.
- Foundation flexibility is often thought to be beneficial for structural design since it lengthens the building period, leading to small spectral acceleration demands. However, foundation flexibility leads to increased storey drift demands and thus, some allowance for foundation flexibility may be required when estimating SLS drift demands for structures on flexible foundations.
- For structures with flexible or semi-rigid roof diaphragms, the in-plane deformation of the diaphragm should be limited in order to limit the likelihood of loss of watertightness. This can be done by limiting the lateral displacement of the individual lateral load resisting systems. This consideration may be particularly relevant for mixed structural systems (e.g. structure formed with stiff RC end-walls together flexible steel portal frames resisting loads across the middle of the building).
- After a significant earthquake, inspection and repair of potential plastic hinge regions may be necessary. If many non-structural elements are positioned around potential plastic hinge locations, then the removal and reinstatement of such NSEs may add considerable repair cost and time (Arifin et al. 2021). Ideally, therefore, the location of the structural system and NSEs will be coordinated with other consultants to facilitate post-earthquake inspection and repairs.

Table 1: Key Considerations for Holistic Design of Structural System

Structural system	**Holistic design and NSE coordination considerations**
Steel frames	- Depending on the shape of the steel roof frame, large ceiling plenums can occur. This can result in long hanging distances for suspended services and the NSE diagonal bracing can be long. This reduces efficiency of the bracing, leads to more material and weight and also causes a higher risk of clashes with other components.

	- The lateral deformations of steel portal frames need to be compatible with the seismic performance of NSEs, in particular consideration of the cladding system and where there are full height partition walls. It is necessary to consider the drift requirements and related seismic performance limits for these systems. Partition walls with a sliding plane that is located directly above the ceiling level can be more expensive but can have the benefit that the portion of the partition above the ceiling is hung and restrained off the roof/floor above and therefore will move similarly to any services hung in the ceiling plenum, compared to full height partitions where the interstory drift can be challenging for services that penetrate the partitions which are suspended from above. The trade-off is that the hung portion of the partition (or bulkheads) can potentially need to hang long distances from the portal frames and this can add significant weight. - Deeper beams can lead to services reticulation restrictions and require early coordination via a services reticulation strategy. - To run services, close to the underside of the structural diaphragm, steel penetrations may be required, leading to additional coordination and sign-off. - Suspending services below beams to reduce steel penetrations might exclude the use of the NZS4219 “within short distance of structure” rule to reduce the seismic restraint requirements. - Braced bay locations need to consider service reticulation, since these can be a major restriction to where services can run. This is particularly true at internal braced bay locations. Locations of key lateral structural systems such as brace locations should be identified as early as possible for coordination with services engineers and architects to consider the implications of these. - Consideration of fixing NSEs to steel structures should consider whether the steel requires fire protection. Intumescent coatings typically have limits on the allowable fixing area to steel, while cementitious coatings need to be applied after fixings are made to the steel, which can be problematic when considering this is typically done when the structure does not have NSEs installed. This can result in the need to remove the coating and then reapply. - In areas with heavy suspended services, such as large pipework, large ductwork, or large cable trays, the seismic restraint load path should be considered early as the lightweight purlins may not be able to accommodate seismic loads from these elements without significant amounts of bridging secondary structure.
Reinforced concrete (RC) and RC	- Typically, this structure type results in a stiff lateral system which assists in minimising movement compatibility detailing requirements, i.e. simple acoustic detailing can typically be used where services cross full-height partitions in low-rise concrete wall buildings. Similarly, it is often not necessary to have a lowered deflection head in partitions to achieve

masonry walls	displacement compatibility between services running through partition walls. To calculate SLS drift demands, that should be passed on to architects and building services engineers, consideration of appropriate cracked stiffness properties for the walls should be made (refer NZS3101), as should the potential impact of foundation deformations on drift demands. - Wall locations need to consider service reticulation, since these can be a major restriction to where services can run. This is particularly true at internal wall locations. Locations of key lateral structural systems such as walls should be identified as early as possible for coordination and consideration of the implications of these by the services engineers and architect. - Noting that this paper is focussed on low-rise structures, structural engineers may need, at times, to consider if the increased rigidity of this type of structure leads to higher accelerations at height. In general ground level provides the lowest accelerations and it is always worth considering if acceleration sensitive equipment can be located lower within the structure.
Timber frames	- Timber frames can have similar challenges as steel frames due to their flexibility (refer Steel frames above). - There can be challenges when detailing connections for the seismic restraint of heavy services. - Restraint of heavy equipment to timber floors can be a challenge due to potentially high point loads and the relatively low strength of timber. It will often by necessary to use timber plinths or spreader beams along with nogs to adequately distribute the load to multiple members. - In areas with heavy suspended services, such as large pipework, large ductwork, or large cable trays, the seismic restraint load path should be considered early as the lightweight joists or roof members may not able to accommodate seismic loads from these elements without significant amounts of bridging.

HOLISTIC COORDINATION AT KEY INTERFACES

Some interfaces between design disciplines have a magnitude of facets that requires input and ownership from multiple disciplines. Those interfaces have traditionally led to conflict, such as commercial responsibility for installation or on engineering tasks. The NSE CoP has identified six key interfaces and outlined primary roles & responsibilities for certain design aspects of each interface. The interfaces are:

- Inter-story Drift
- Vibration Isolation
- Thermal Expansion
- Ceiling/partition
- Equipment/system selection

- Passive Fire

Figure 2 below shows the example of inter-story drift interfaces from the NSE CoP. The NSE COP has intentionally aligned roles with the New Zealand Construction Industry Council Guidelines (NZCIC, 2023).

Figure 2: Inter-story drift interface (Table A-7 NSE CoP)

Design element	Primary Responsibility
Amount of building movement during seismic events	Structural Designer
Confirmation on where movement occurs	NSE Seismic Designer
Identification of NSE that are impacted by the building movement	Building Services Designer, Architect, Fire Engineer
Amount of movement for movement joint selection	NSE Seismic Designer
Selection of movement joints (material, suitability for the relevant service, amount of movement)	Building Services Designer, Architect, Specialist Supplier
Illustration of movement & break away joints on drawings	Building Services Designer, Architect
Support/Restraint of movement & break away joints	NSE Seismic Designer – note timing of when this occurs should be agreed (design or construction phase)
Selection of suitable passive fire products that can accommodate movement	Passive Fire Designer, Specialist Supplier

For the inter-storey drift interface example above, the table shows that it is the Structural engineer's responsibility to confirm the amount of inter-story drift. But it is the Building Services engineers responsibility to understand which building services NSEs are impacted by this movement. The NSE Seismic Designer is responsible to confirm the amount of movement on each NSE and align the restraint methodology accordingly as the restraint will restrict movement.

CONCLUSIONS

Low rise construction makes up the bulk of New Zealand's building stock. This paper raises the importance of a coordinated approach to the design of structural and non-structural elements in these building types. Early consideration of the seismic response of the structure and the demands that this will place on the non-structural elements is an important part of the design process. Conversely, the gravity and seismic demands that structure must support require careful and early consideration. The use of the NSE CoP will assist in this process.

The NSE CoP is a document that can be used by all parties in the design and construction process to support selection of the structural system that best meets the seismic performance and resilience expectations of building owners and tenants. The guidance includes tools to

help identify important nonstructural elements that require coordination early in the design process when problems can be more cheaply and easily rectified. This is particularly important at key interfaces between engineering disciplines. It also identifies the importance of a multi-discipline building movement strategy whereby any necessary compromises can be identified and agreed early in the design process. Also, the NSE CoP provides information to allow seismic performance criteria to be set to ensure that resilience, business continuity and Building Code requirements are met.

Industry-wide adoption of the NSE CoP is identified as an important step in improving the seismic performance of our buildings through the lens of a holistic design process. The second phase of the project is now underway to increase the scope of the CoP to include a greater range of NSEs, provide coordination guidance in later stages of design and construction and to provide assistance with the seismic qualification of plant and equipment. Funded by BRANZ and the Building Innovation Partnership, phase 2 should be available early 2026.

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A Streamlined Approach to Soil-Structure Interaction (SSI): Reducing Iterations Between Structural and Geotechnical Engineers

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ABSTRACT

As part of a detailed seismic assessment and strengthening project, an alternative approach to soil-structure interaction (SSI) modelling was implemented for a building at 40 Queens Drive, Lower Hutt, using nonlinear time history analysis. Designed in 1988, the eight-storey building features an offset reinforced concrete core with C-shaped and coupled walls extending below the basement. The foundation consists of a 15m × 17m × 2m concrete raft, supported by 16 'franki piles', each 500mm in diameter with double bulbs. The original design assumed overturning moments were resisted through a combination of soil-bearing pressure and pile tension. The structural model initially included nonlinear vertical springs for the raft and nonlinear axial/lateral springs for the piles at designated depth intervals, as provided by the geotechnical engineer. However, this approach failed to capture rotational stiffness adequately, leading to an overestimation of pile compression and inconsistencies with separated geotechnical analyses. To address this, the geotechnical engineer analysed the foundation under five loading conditions (G, G + 33%EQ, G + 67%EQ, G + 100%EQ, G + 130%EQ), providing raft displacements and pile loads. Using these results, the structural engineer calibrated nonlinear springs to better match the observed foundation response, ensuring rotational compatibility and improved alignment with the geotechnical models. This refined approach shifted the focus from simply exchanging soil springs to coordinating foundation behaviour under expected loads. As a result, it enhanced the representation of pile-soil interaction by reducing discrepancies in pile-loading predictions and improved the accuracy of the SSI assessment. By refining input-output coordination, the process minimised iterations and strengthened confidence in the seismic performance evaluation.

INTRODUCTION

The building was designed in 1988 and is 8-stories high with a plant room on top of the building's roof. The lateral load resisting system is an offset reinforced concrete core, consisting of 'C' shaped and coupled walls. The building is founded on a combination of isolated pads, raft and of bulb-ended reinforced concrete piles, known as 'franki piles'. The soil type classification is Type D – soft ground.

Originally, the building was designed to be symmetrical with two “office floor pods” on each side of the lift and stair core. Due to only one of the two “pods” being constructed, the building has an offset core in its structure, hence creating a torsional response. This produces high demands on the floor diaphragms and perimeter frames in some areas, which is the main reason for the assessment and strengthening design undertaken by New Zealand Consulting Engineers Ltd. (NZCEL).

At an early stage of the assessment, a linear fixed-based model was developed. The results confirmed the initial expectations: a torsional first-mode, coupled translational modes and inter-storey drift exceeding the limit at the perimeter frames. It should be noted that the main reason for the high inter-storey drifts at the perimeter of the building -when using a modal response spectrum analysis (MRSA)- was the scaling factor k (NZS 1170.5 2004) used to match the base shear obtained by using the equivalent static method (ESM). In the author’s opinion, the important discrepancy of the base shear between both models indicated that an elastic analysis was not appropriate for the building, providing conservative outcomes. Some of the difficulties of the implementation of base shear scaling in complex buildings have been studied (Davidson 2008).

Due to the above, NZCEL performed some sensitivity checks to evaluate the effect of the base supports on the mode shapes. The results showed that the inclusion of soil springs of different stiffnesses bring the translational modes to the first two modes. In addition, a preliminary nonlinear time history analysis (NLTHA) using the previous soil springs showed that inter-storey drifts were comparable to the unscaled MRSA (i.e., base shear not scaled to the ESA base shear), confirming that the use of a linear method was not the correct tool to assess the building.

Accordingly, the building was evaluated using a NLTHA with consideration of soil-structure interaction (SSI). As noted in (MBIE 2017), *while the SSI modelling principles are generally applicable to the New Zealand context, the use of SSI to reduce the seismic demand using SSI damping and kinematic effects is not provided for in these guidelines*. They are considered as an alternative solution to the guidelines. Conservatively, to avoid overestimating the total damping on the structure, it was decided to ignore the SSI damping and kinematic effects.

The project was categorized as interactive according to section C4.2.4 of (MBIE 2017), where significant interaction between the structural and geotechnical engineer is expected.

FOUNDATION SYSTEM

The foundation of the gravity system consists of isolated pads at the centre of the building and isolated or interconnected pads supported on single-bulb ‘franki piles’ at the perimeter of the building. The piles were design to support compression-only forces.

The foundation of the lateral resisting system consists of a 2m deep raft which has a rectangular shape of 15m x 17m. Underneath the raft there are 16 double-bulb ‘franki piles’, which were designed to control the overturning of the raft (i.e. providing tensile capacity) when resisting the overstrength capacity of the core walls which are seated on the raft. The piles have a double-bulb in order to resist tensile capacities and have closely spaced hoops (HR16-100) in the top 750mm, which allow them to develop plastic hinges at the location of maximum moment demand.

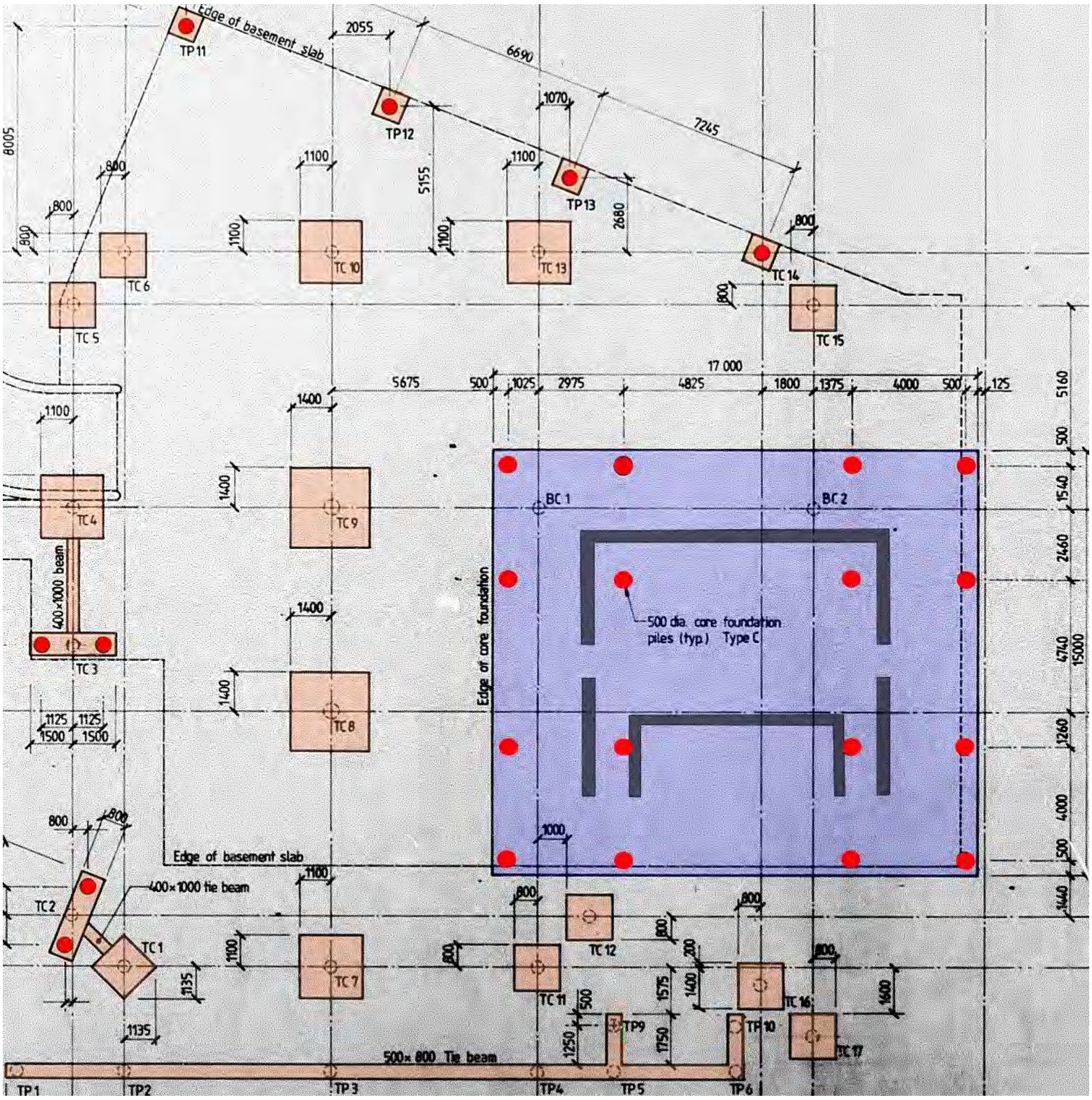

Figure 1: Foundation system

INITIAL APPROACH

Following the initial screening to assess sensitivity of behaviour, it was decided to use a flexible base model using explicit vertical and horizontal (i.e., p-y curves) compression-only nonlinear springs.

The ground model

A ground model was developed by Torlesse from the available site data which included a total of 5 No. Boreholes and 2 CPTs. Groundwater was taken at 2.0m below ground level.

Some lenses of liquefaction were identified from the site data but the lenses where not persistent across the site and of limited thickness. As such the liquefaction risk was considered low and liquefied conditions were not modelled in the assessment.

A summary of the developed ground model and geotechnical design parameters is presented in the table below:

Table 1: Ground Model and Parameters

Geology	Description & Strength	Top Depth (m)	SPT 'N' Range [Average]	γ (kN/m3)	φ' (°)	c' (kPa)	Su (kPa)	E' (MPa)
Fill	Medium dense, fine to coarse GRAVEL (basecourse)	0.0	-	19	32	0	-	60
Alluvium	Firm SILT	0.5	-	18	-	-	50	20
	Medium dense to dense sandy GRAVELS	1.4 to 2.2		20	34	0	-	60
Taita Alluvium/ Melling Peat	Firm SILT/Organic SILT	4.3	20 – 50+ [34]	18	28	2	40	8
	Medium dense to dense sandy GRAVELS	5.0		20	34	0	-	60
Petone Marine Beds	Medium Dense fine SAND	8.0	20 – 27 [24]	19	32	0	-	50
Waiwhetu Gravels	Dense to very dense GRAVELS	14.0 to 16.0	-	22	-	0	-	100

Geotechnical outputs

The following table show the ultimate bearing capacity (UBC) and subgrade modulus of the building's foundations calculated by Torlesse. All the properties were assessed for 3 cases: the 'best estimate' or probable, the upper bound, and lower bound.

Table 2: Shallow foundations – non-liquefied subgrade modulus - Probable

Foundations	Founding Unit	UBC (kPa)	Subgrade Modulus (kN/m3)
TC1, TC2, TC7, TC11, TC12, TC16, TC17	Firm SILT	400	20,000
TC3 - TC6, TC8 - TC10, TC13 - TC15, TP11 - TP13	Medium dense to dense sandy GRAVEL	900	35,000
TP1 - TP10 Tie Beam	Firm SILT	350	12,000
TC1, TC2 Tie Beam	Firm SILT	400	50,000
TC3, TC4 Tie Beam	Medium dense to dense sandy GRAVEL	900	65,000
Raft (Shear Core Foundation)	Medium dense to dense sandy GRAVE	450	4000

The table below show the properties of the piles located underneath the raft that were provided to the perform the structural model. These properties correspond to the probable case.

Table 3: Vertical Pile Displacement Curves Compression / Tension - Probable

Displacement (mm)	Compression Load* (kN)	Tension Load** (kN)
0	0	0
2.5***	1703	760
10	2310	760
75****	3859	760

*Compression values include both skin friction and end bearing.

**Tension values ignore the bulb due to poor curtailment at the base of the pile.

***Skin friction assumed to fully mobilise at 2.5mm displacement.

****Shear Core piles with 0.5m Shaft (B = 0.75m bulb) with ultimate displacement capacity at 10% of B.

Structural model results

After implementing nonlinear springs in the structural model using (ETABS 2023), analysis results revealed an unexpected issue: excessive compressive forces were being resisted by the piles. This contradicted the original design assumption, which intended overturning overstrength moments to be resisted through a combination of soil-bearing pressure and pile tension. The following explanation, sourced from the original design documents, provides additional clarity

The shear core foundation was designed to transfer the wall overstrength moments and shear forces and axial force to the ground. Moment is resisted by compression of soil and tension supplied by bulbed uplift piles. Shear is transferred to the ground via friction under the core foundation pad. Torsion on the core is transferred to gravity column foundation pads, and hence taken out to ground via friction, by the basement floor slab.

The excessive compressive load in the piles led to a reduction in rotational capacity, as the axial stress ratio ($N/A_g f'_c$) exceeded 0.3. Consequently, these critical elements began to govern the overall behaviour of the building. Following a technical discussion involving NZCEL and Torlesse, a potential cause for the observed behaviour was identified: while simplified vertical spring models provide good estimates of vertical displacements, they tend to underestimate the rotational stiffness of rigid foundations. Guidance for modifying the springs to achieve the correct behaviour is provided in various documents such as (NIST 2012) and (ASCE 2017). However, a key limitation of the referenced methods is that they are primarily formulated for 2D analyses, i.e., axial force with uniaxial bending. NZCEL extended the methods proposed to a 3D context, which improved the estimation of rotational stiffness of the raft in isolation. However, the piles continued to attract significant axial loads.

NZCEL began performing manual calculations (i.e., using force equilibrium) to assess whether the foundation system's capacity using the properties provided in Table 2 and Table 3 above could withstand the load demand resulting from the overstrength. Unexpectedly, the foundation system's capacity was insufficient to resist the moment demand, and this was communicated to Torlesse. After discussing the spring properties, it was agreed that Torlesse would develop a model to confirm whether the foundation system could resist the load.

Geotechnical model results

A preliminary 2D finite element analysis (FEA) model was developed by Torlesse (RS2 2023) to evaluate whether the foundation system could withstand the overstrength loads from the supporting walls. The geotechnical model is described in detail later. The results indicated that the foundation system could resist the loads in both directions of analysis, with the axial loads in the piles being lower than those predicted by the structural model. These results were more consistent with the assumptions made by the original designer.

OPPORTUNITY IN AN ALTERNATIVE APPROACH

According to the geotechnical model, the foundation was capable of resisting the overstrength loads in both principal directions of analysis. However, the structural model -utilizing nonlinear springs provided by Torlesse- did not accurately replicate this behaviour. Rather than deriving spring properties from the geotechnical model and incorporating them into the structural analysis, the chosen strategy was to align the structural model's response with the behaviour observed in the geotechnical model. Based on this approach, NZCEL proposed an alternative methodology to Torlesse: instead of providing nonlinear soil springs, the geotechnical engineer would supply foundation deformations and corresponding reactions under the expected loading conditions. The following information was requested from Torlesse:

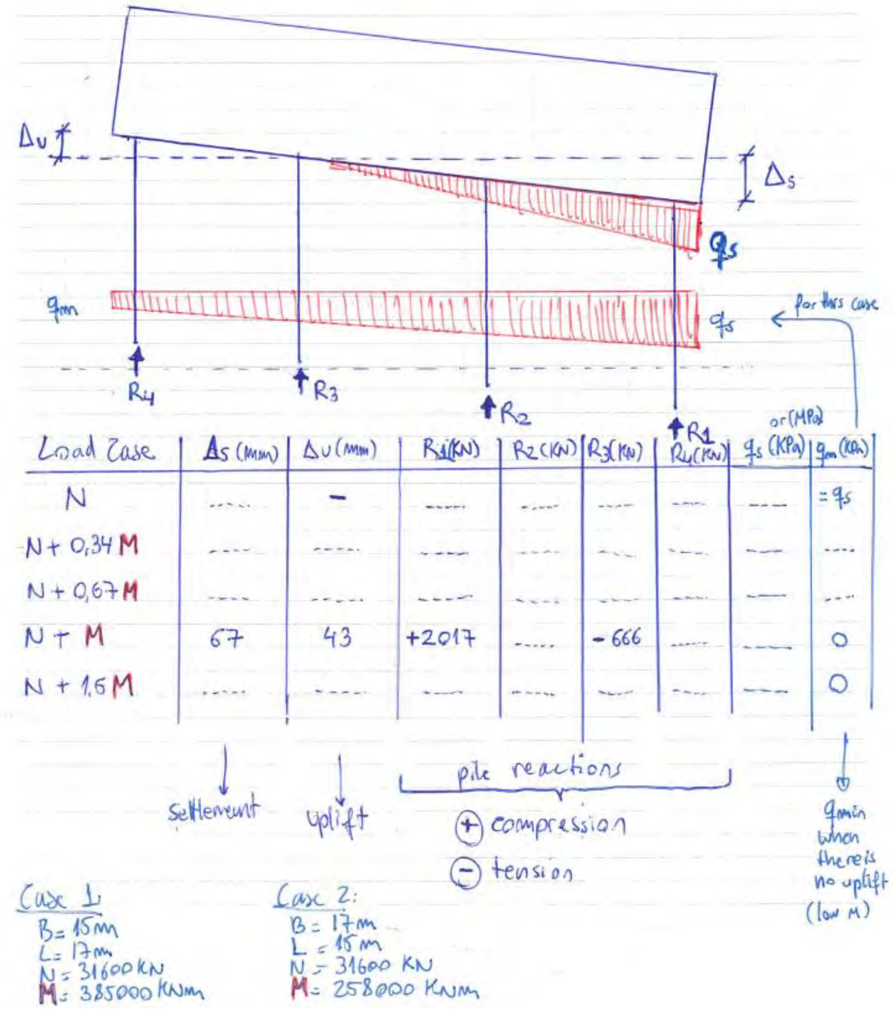

Figure 2: NZCEL's requested information

Geotechnical model

Following discussions between the structural and geotechnical engineer, a 2D finite element analysis (FEA) model was developed using (RS2 2023) to investigate the relationship between moment and displacement, with vertical load held constant. The model was validated against the two simplified methods (Bowles 1996) and (Poulos 2001), showing strong consistency in vertical settlements across a range of applied loads. The primary advantage of using FEA in this context was the ability to model the rotation of the raft and capture the varying demands on the piles, including both tension and compression forces.

For this specific building, the model was simplified by treating the large, very stiff raft as a rigid body, with the vertical load remaining constant as the moment demand increased. A snapshot of the FEA model is provided in Figure 3 below.

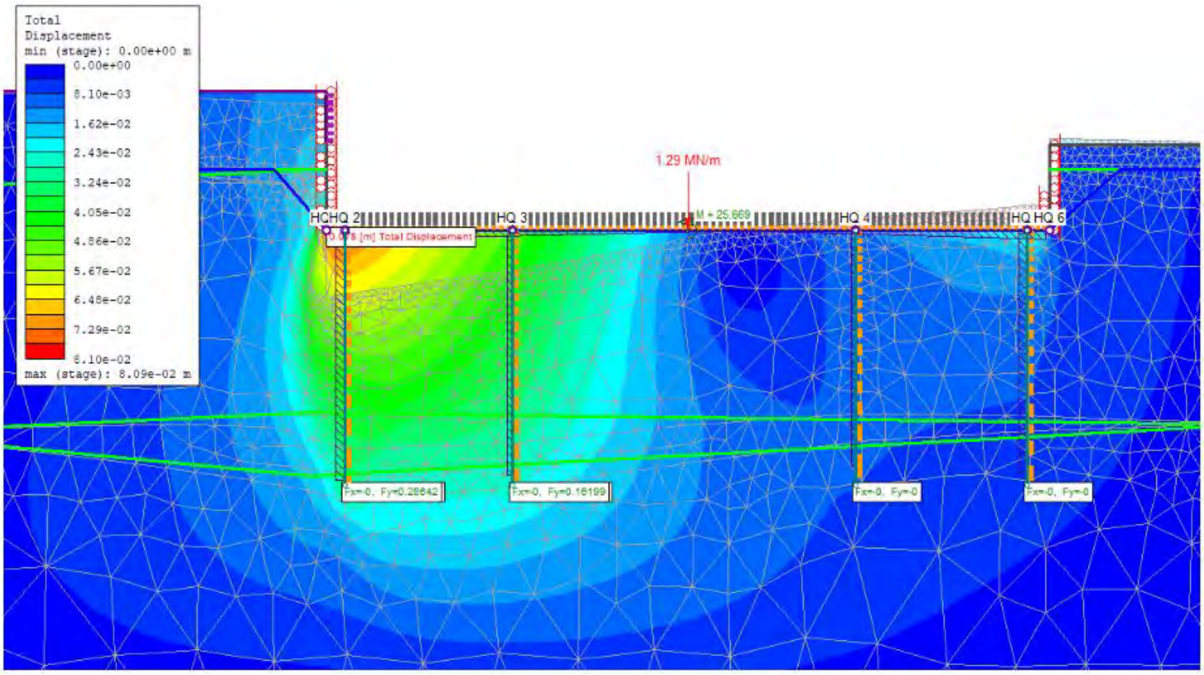

Figure 3: 100% Moment Demand – 17m Wide Raft - Best Estimate

In addition to evaluating the behaviour for the 'best estimate', upper and lower bounds were also investigated. A range of soil properties was assessed based on the variability of the geotechnical data and the model was re-run for each case. This approach provided a much narrower range of results compared to simply using half or double the best estimate.

Geotechnical Outputs

Table 4 below presents a typical output from the model for the structural engineer.

Table 4: Best Estimate Results – 15m wide raft case

Stage	Desc.	N (kN)	M (kNm)	Δs (mm)	Δu (mm)	R1 (kN)	R2 (kN)	R3 (kN)	R4 (kN)	Qs** (kPa)	Qm (kPa)
1	Calibration	5500	-	4	-	634	537	486	519	40	20
2	Calibration	19,360	-	16	-	1042	967	863	810	190	80
3	Calibration	5500	55,000	18	-3	1005	724	419	199	310	20
4	34%	19,360	87,720	30	-3	1297	1095	648	-164	350	10
5	67%	19,360	172,860	43	5	1573	1241	409	-595	450	0
5A	90%	19,360	232,200	54	11	1779	1345	-4	-617	540	0
6	100%	19,360	258,000	60	13	1878	1401	-356	-612	580	0
7	130%	19,360	335,400	95	41	2407	1438	-587	-574	810	0
8*	160%	19,360	412,800	-	-	-	-	-	-	-	-

*Model not able to converge – results unreliable.

**Some localised higher peak values located at edge of raft.

Calibration of structural model

The calibration of the structural model involves incorporating nonlinear springs to replicate the vertical deformation and rotational behaviour of the foundation system, as characterized by Torlesse, under varying levels of moment demand. The following springs have been created:

- A single vertical nonlinear spring, K_P, applied to all the piles beneath the raft
- A single vertical nonlinear spring, K_R, located at the centre of the raft
- A single bi-axial rotational spring, $K_{\theta R}$, also placed at the centre of the raft.

The process consists in calibrating the behaviour at each stage shown in Table 4, by adding an additional pair to the nonlinear springs modelled in (ETABS 2023), i.e., $\{\Delta\ ;\ F\}$ for vertical springs and $\{\theta\ ;M\}$ for rotational springs. The following flowcharts show the process to calibrate the springs for the gravity load cases (Figure 4) and the seismic load cases (Figure 5). It should be noted that for the seismic load cases, both directions of analysis are calibrated in parallel:

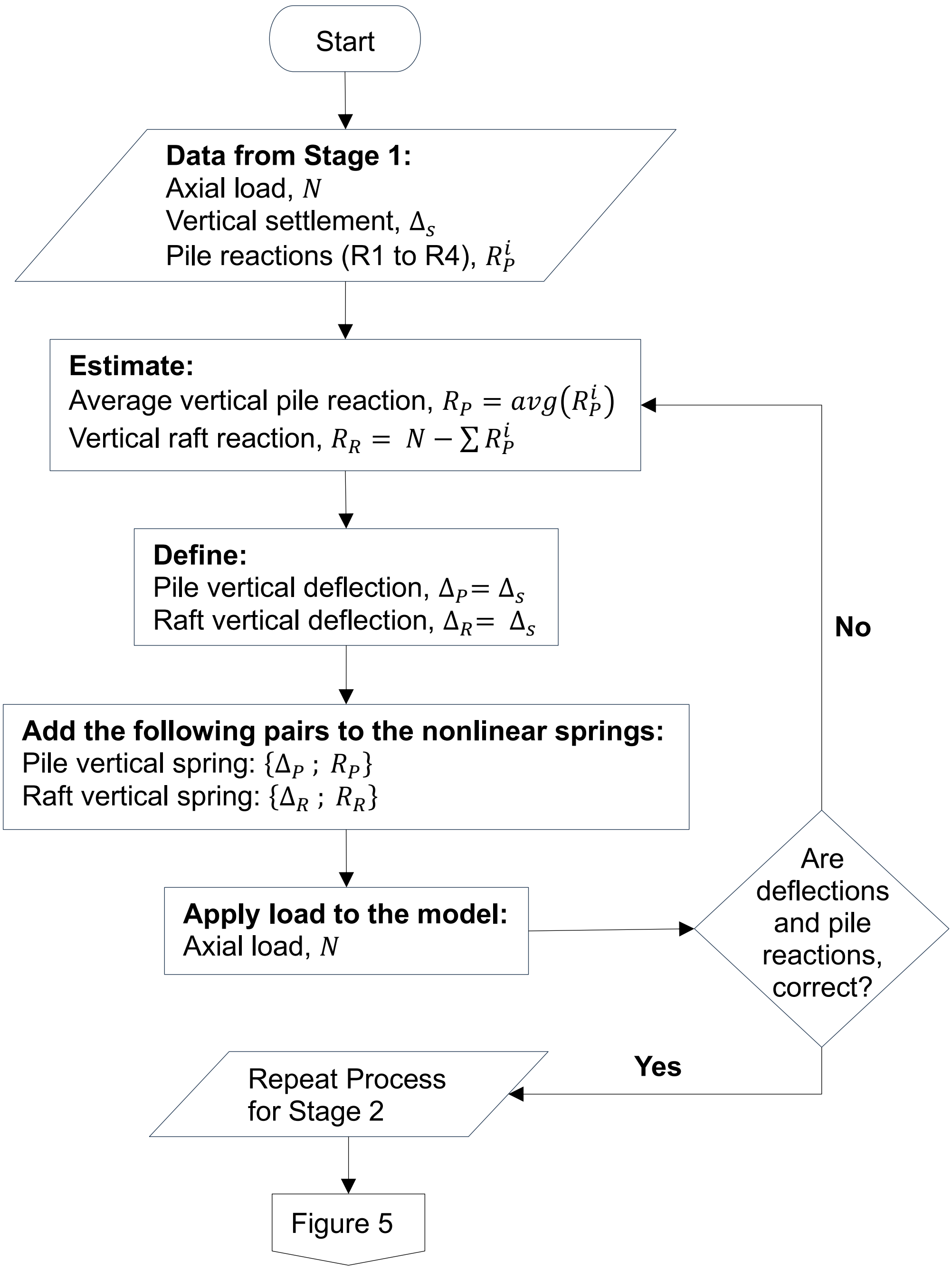

Figure 4: Calibration of gravity load cases (Stage 1 and 2 of Table 4)

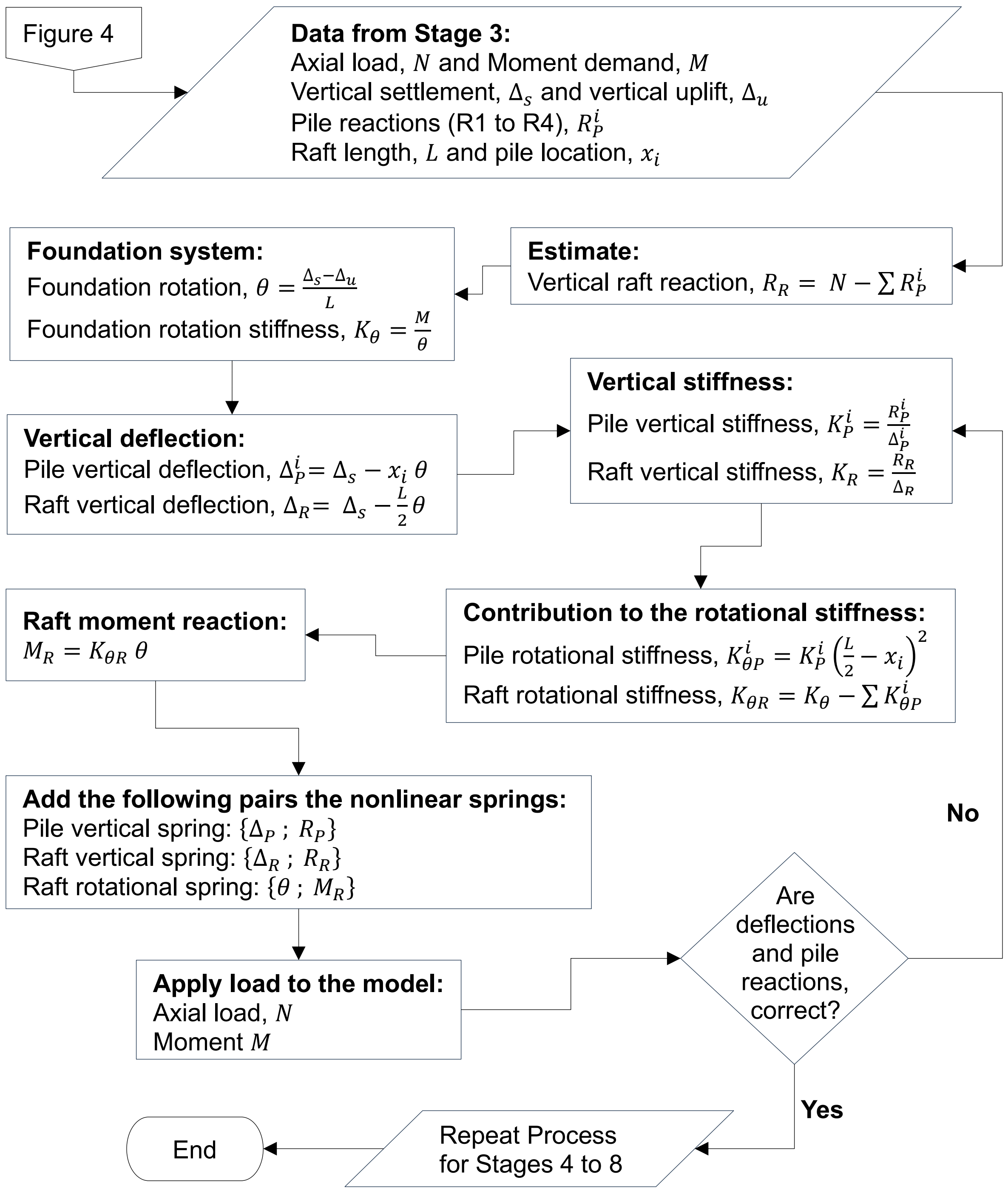

Figure 5: Calibration of seismic load cases (Stages 3 to 8 of Table 4)

RESULTS

Rotation of the foundation system

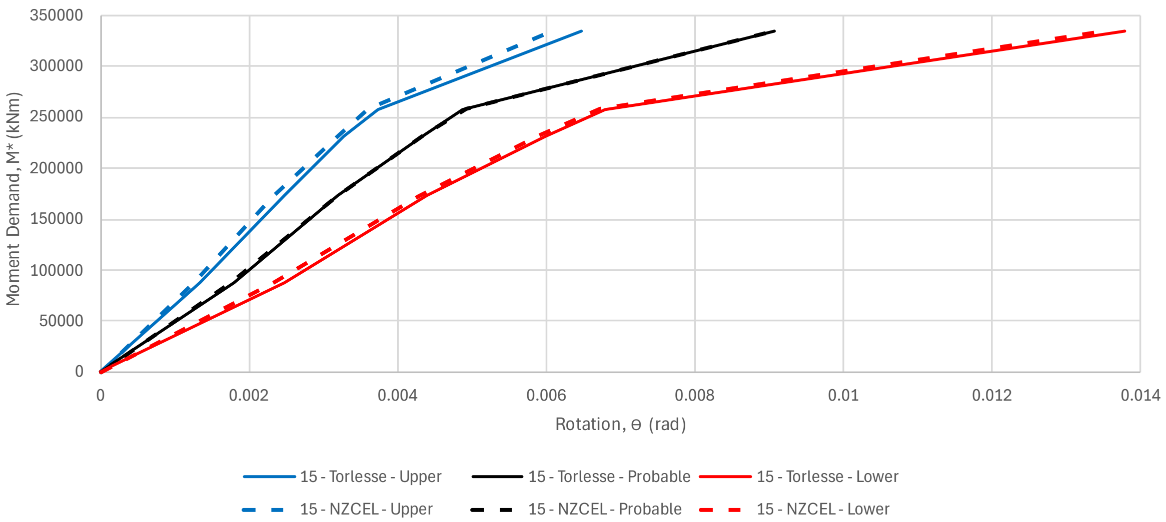

Figure 6: Moment-Rotation of the foundation system

Vertical displacement at the centre of the raft

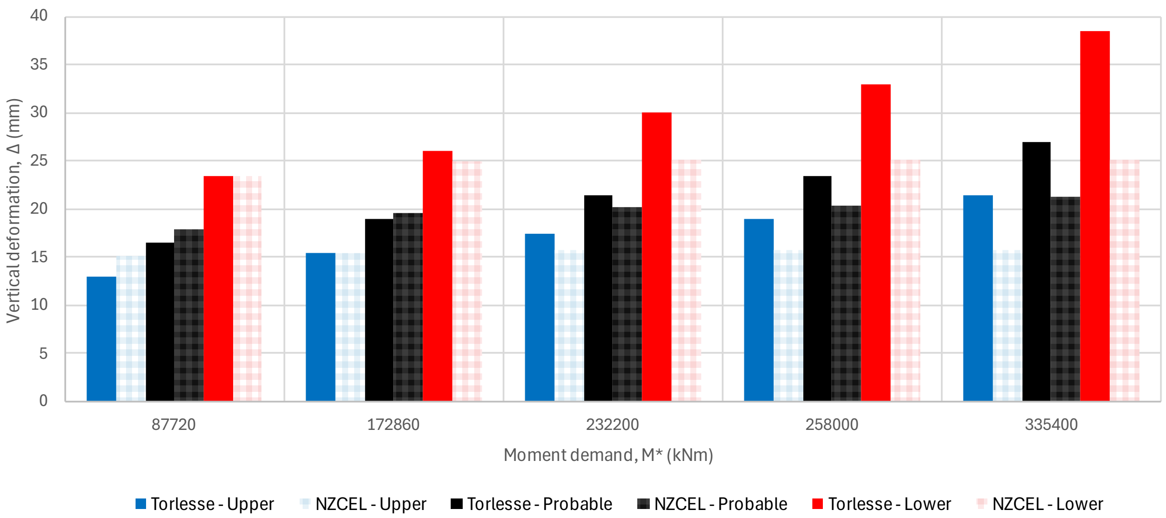

Figure 7: Vertical displacement at the centre of the raft

Pile reactions

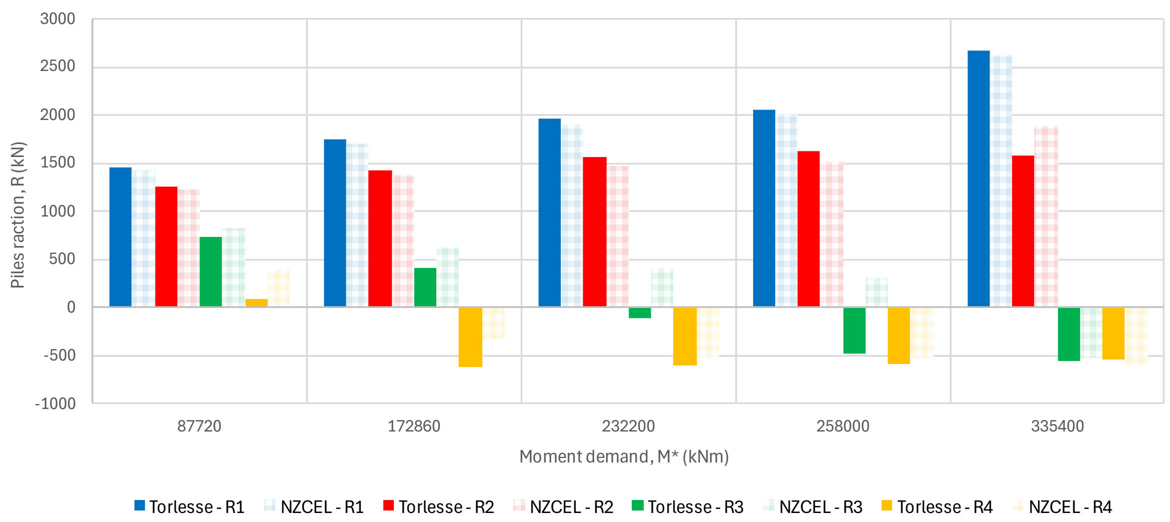

Figure 8: Pile reactions (R1 to R4) for increasing moment demand - Probable

DISCUSSION

Based on the results presented in Figure 6, it is evident that after the calibration process described in Figure 4 and Figure 5, the rotation of the foundation in the structural model closely aligns with that of the geotechnical model across varying levels of moment demand. This consistency holds true for different soil conditions evaluated by Torlesse -ranging from lower to upper bound soil properties- and in both directions of analysis.

In addition to foundation rotation -which was the primary parameter used for calibration due to its influence on building drift- the vertical deformation at the center of the raft was also assessed. As shown in Figure 7, the vertical displacement predicted by the structural model closely matches that obtained by Torlesse for moment demands up to 67%EQ. For higher load levels, the structural model slightly underestimates vertical deformation by approximately 5 mm to 10 mm.

Figure 8 presents the compression loads on the piles. Since all piles beneath the raft are similar, particular attention was given to those at the edges, where the highest compression (R1) and tension (R4) forces occur. The compression loads show excellent agreement with Torlesse's results for the different levels of seismic load. While the structural model underestimates tensile forces for lower moment demands (up to 67%EQ), it closely matches the tensile loads under maximum seismic demand. These latter results are particularly significant, as they confirm that the peak tensile forces on the piles in the structural model are consistent with those expected from the geotechnical model.

CONCLUSION

It is typically recommended that the geotechnical engineer provides spring parameters to the structural engineer, who incorporates them into the structural model. The structural engineer then shares the resulting forces, displacements, and soil reactions back with the geotechnical engineer. If the stresses and deformations do not align with the geotechnical model, the springs are adjusted and reapplied to the structural model. This process is repeated iteratively until both models produce consistent stresses and deformations. As highlighted by (MBIE 2017), "*significant interaction between the geotechnical and structural engineer during a DSA is considered essential*". However, several limitations exist with this approach:

- **High Number of Iterations:** multiple iterations may be required. Since the time spent by each party is variable and coordination is not always immediate, the process can become inefficient and time-consuming.
- **Unbalanced Control of Accuracy:** the number of iterations is usually determined by the geotechnical engineer, who judges whether the structural results are compatible with the geotechnical model. This gives the geotechnical engineer control over the final accuracy, even though outputs like settlement and rotation are often more critical to the structural engineer.
- **Shifted Focus in Structural Modelling:** Instead of verifying geotechnical parameters such as soil pressures and uplift, the structural engineer can concentrate on accurately replicating the foundation behaviour within the structural model. This approach relies on the assumption that the geotechnical engineer has already assessed and validated the soil response through the geotechnical model and communicated the expected behaviour to the structural engineer. As a result, the structural engineer's focus shifts toward ensuring the structural model aligns with the expected foundation performance, rather than re-evaluating geotechnical conditions.

This methodology presented offers several advantages and is particularly well-suited to specific scenarios such as raft-pile configurations. For torsionally sensitive buildings, accurate prediction of foundation deformations is essential, as these deformations contribute directly to the overall building drift and structural performance. The approach is most applicable to foundations subjected to constant axial loads -such as cantilever walls- and to rigid foundations, where behaviour can be effectively characterized by vertical displacement at the centre and rotational movement. Conceptually, the method resembles a discretized pushover analysis, where foundation behaviour is assessed at discrete load levels and linear interpolation is assumed between them. However, continuous load-deformation curves can be developed if needed, particularly in cases involving step-change behaviour.

In the author's opinion, the traditional practice of exchanging springs between geotechnical and structural engineers is not optimal. A more effective strategy is to exchange foundation displacements and rotations for given load combinations. This significantly reduces the number of iterations required and creates a clearer division of responsibilities between the geotechnical and structural disciplines.

ACKNOWLEDGEMENTS

The authors extend their gratitude to the building owner for granting permission to use the project as a case study for this paper. They also thank Spectrum Consulting Engineers Ltd. (peer reviewer) and Enlink Consulting Ltd. (site-specific seismic hazard assessment) for their valuable contributions to this ambitious project.

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SESOC Retaining Wall Design Guidance & Software

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Beca, Auckland, and SESOC ManComm

ABSTRACT

SESOC has spent considerable effort in the development of retaining wall design guidance, for Cantilever Timber Pole Walls (CTP) and more recently for Reinforced Concrete Walls (RCW). These have been prepared as a formal, robust – and transparent, basis for the respective design tools.

This paper will briefly overview the current status of each, including :

- The CTP design guide, first published Jun 2024
- The CTP app, launched December 2024
- The latest status on the RCW Design Guide and associated software.

INTRODUCTION

Upon taking responsibility for the SESOC Software portfolio circa 2010, it was the author's particular question and concern regarding the SESOC Soils program – "What is the Technical Basis ?". In the absence of a national standard, or any substantive, comprehensive, or generally agreed design guidance, SESOC – naively – commenced development of this.

This paper overviews the core concepts espoused in the Design Guides. Graphics have typically been extracted directly from the relevant Design Guide. Further detail can be found in the Design Guides themselves.

CREDITS

I wish to particularly highlight the hugely valued contributions of the following parties,

(i) my co-author and pen-holder, Allan McPherson – and his attention to detail,
(ii) John Wood, whose thoughtful insights, generous support and substantive verification efforts have richly enhanced these documents. And lastly,
(iii) the three NZGS member/reviewers of the CTP document, Andrew Langbein (T&T), Ian McPherson (Aurecon), and James Burr (Beca).

CANTILEVER TIMBER POLE WALL (CTP)

The following diagrams are intended to demonstrate some of the key aspects around the CTP methodology and design actions :

The design guide covers all common loading and geometric scenarios, including:

- Wall geometries
- Founding soil, cohesive or cohesionless
- Backfill soil assumed as cohesionless
- Surcharge loading, including DL & LL
- Seismic loading,
- water table options, none, @GL, & user defined above GL

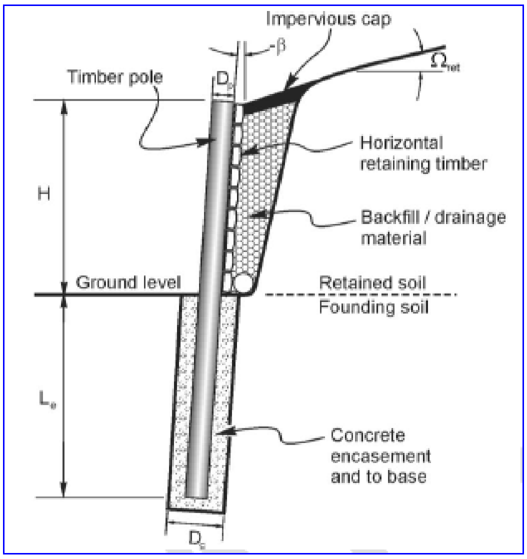

Figure 1 : Generalised CTP Wall Layout

The design guide assumes a continuous wall basis, with load spreading out to a maximum of 3.5 Dc (encased pole diameters).

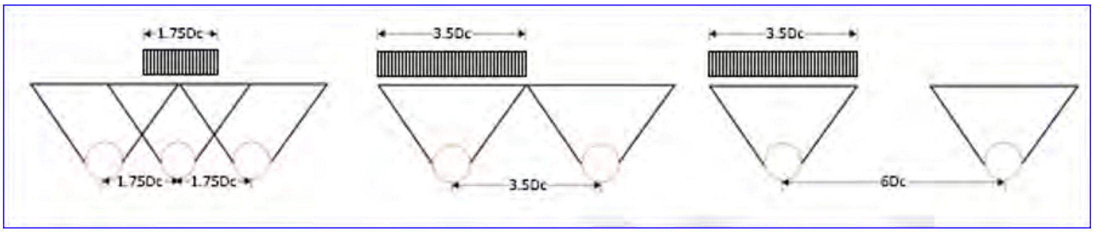

Figure 2: Passive pressure widths, i.e. below GL

As engineers, we're inclined to categorise soils as either cohesive (clays), or cohesionless (sand), though in reality most exhibit a combination of both. However, in terms of geotechnical design, an important parameter is the loading regime, i.e. static, or dynamic/seismic, as this significantly affects the response behaviour of the soil. For this reason, for design purposes, we categorise soil behaviour in to 'Drained', or 'Undrained', as shown below:

Founding Soil Type	Loading	Analysis Type	Strength Parameter
Cohesive	Static, long-term	Drained	φ', c'
Cohesive	Seismic, short-term	Undrained	s_u
Cohesionless	Static, long-term	Drained	φ'
Cohesionless	Seismic, short-term	Drained	φ'

Figure 3: Analysis type: relationship between soil type and loading scenario

The following diagram shows the 'structural mechanics' pressure blocks for both drained and undrained analysis types.

In the diagrams,

- active pressures - blocks 1, 2, 3, & 4
- passive pressures - blocks 5 & 6
- block 7 is water pressure

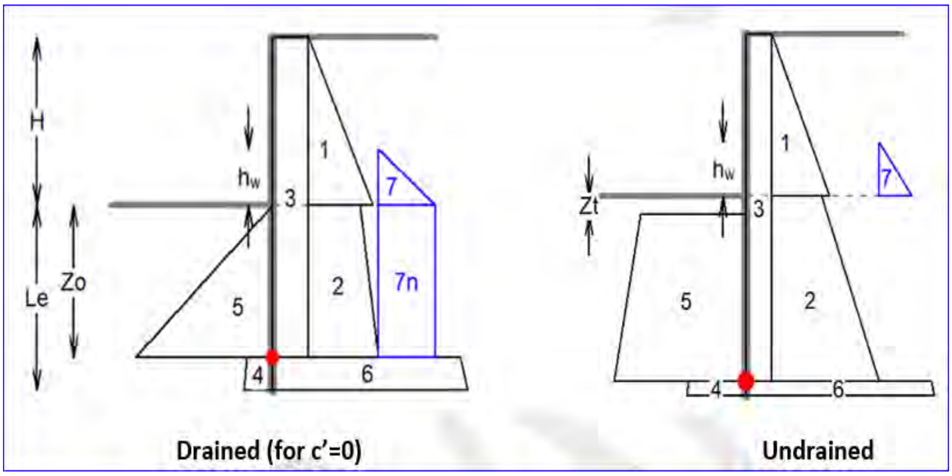

Figure 3: Typical pressure block diagrams

SESOC Software App, Retaining Wall

First launched Dec 2024, the SESOC App is now available on the SESOC website. This software espouses the methodology outlined in the above design guide, [1], a sample screen shot of which is shown below.

Timber member design aspects are currently disabled, to be made available with the next release, following verification

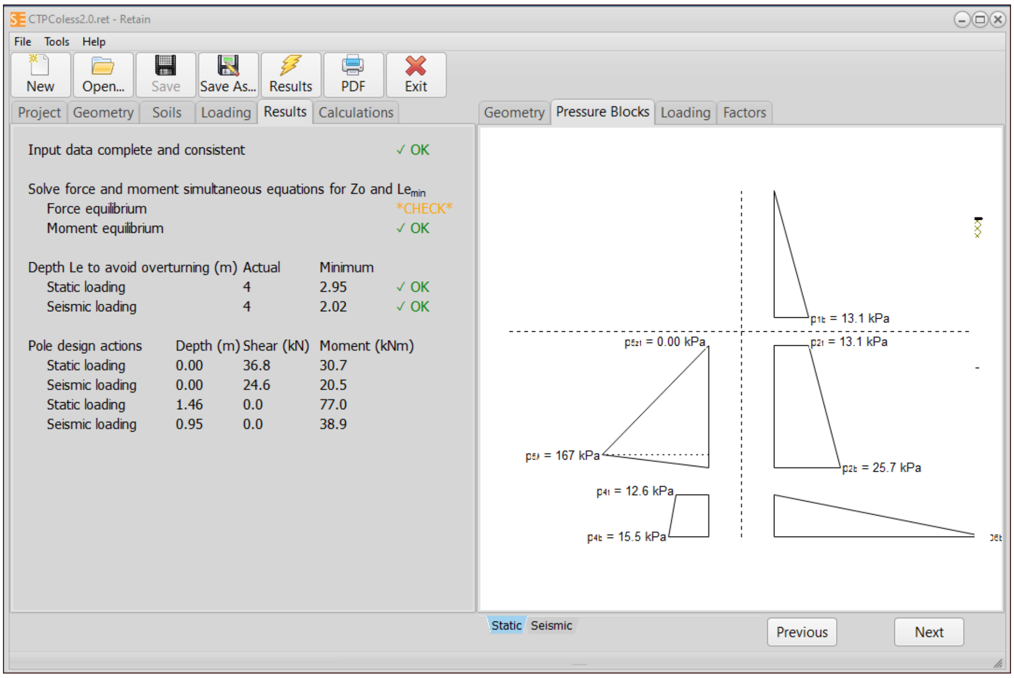

Figure 4: SESOC App : Sample results screen

REINFORCED CONCRETE CANTILEVER WALL (RCW)

A typical wall cross section is shown below. Also shown is the 'virtual back face' – a nominal vertical line up from the rear of the footing. This virtual back face delineates the 'integral wall + soil mass' versus the 'other', where the former is assumed to contribute to stability (for both overturning and sliding), as well as the soil-to-soil friction interface which also contributes to overturning stability.

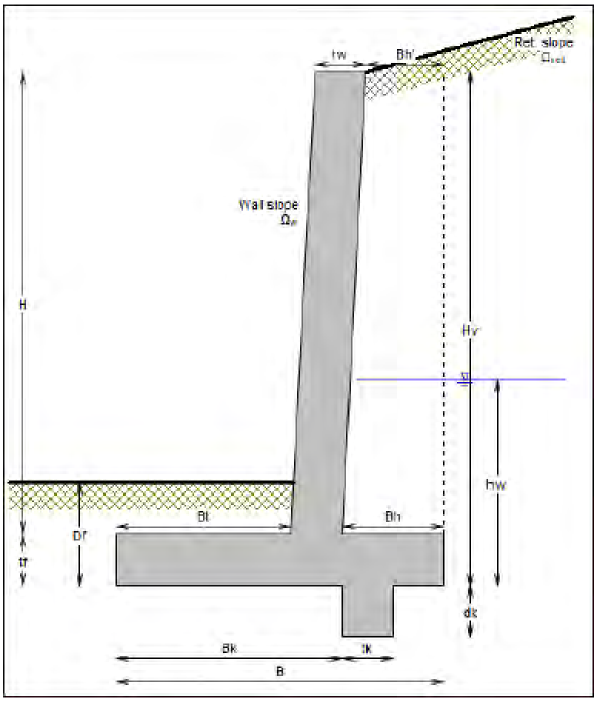

Figure 5: Generalised layout

The adjacent diagram shows the origin location, as well as the various components contributing to the overturning stability aspects of the wall behaviour.

In this diagram, V* includes wall self weight, captured soil mass above the heel, and the virtual back face interface friction.

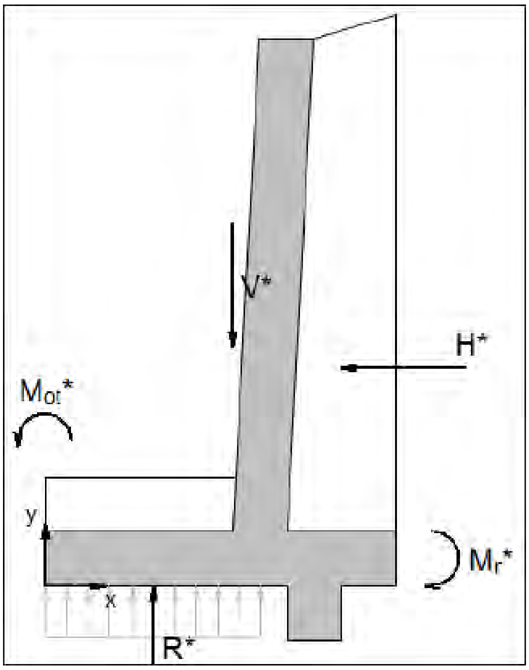

Figure 6: Overturning stability actions

The following shows both the driving and resistance components for the sliding limit state, for both key and non-key scenarios.

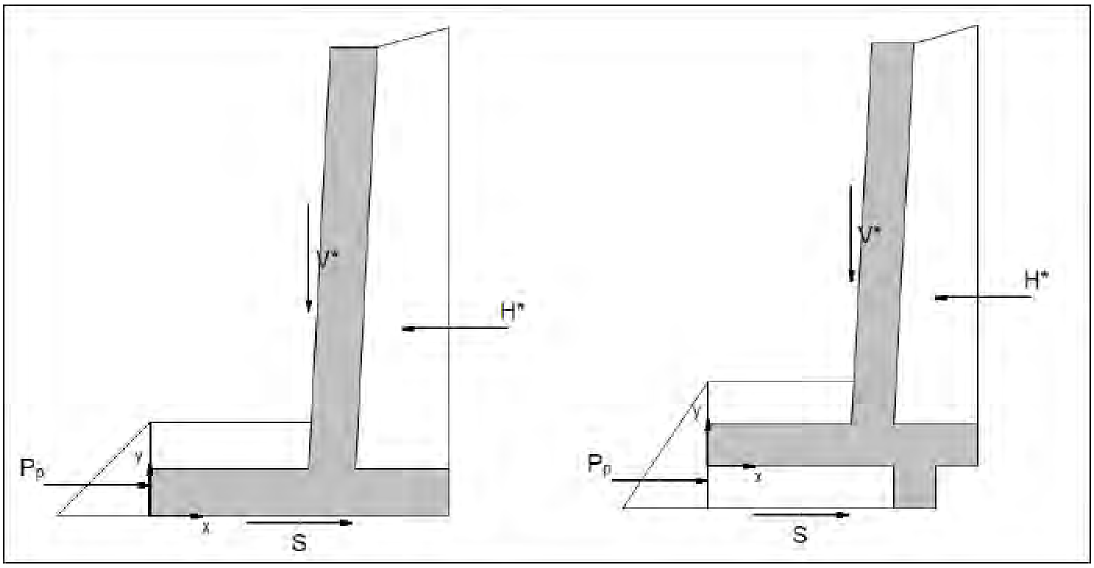

Figure 7: Sliding stability - Diagrammatic representation of actions

Other Matters

Soil pressures

As the reader may be aware, there are numerous methods to calculate active and passive pressures – along with significant diversity in the outcomes. We have sought to implement methods that are broadly accepted, ‘reasonable’, and ‘closed form’, i.e. can be formulated and calculated directly. The following is the basis used :

- Active pressures, static - Coulomb, plus Mayniel and Muller-Breslau modifications
- Active, seismic – Mononobe-Okabe
- Passive - Mylonakis

Load factors

Historically, geotechnical load factors have been subject to some considerable debate – and variation, as evidenced by the study undertaken during preparation of these guides, and tabulated in B.4 of [1].

SESOC has sought to balance the load multipliers (LMs) and strength reduction factors (SRFs), in order to achieve an outcome in line with NZ standards, current norms (where possible), as well as a suitable overall FoS. Refer B.4 of [1]

Validation & verification

Notwithstanding the long-standing and broadly accepted Geotech principles upon which this above content is based, it is recognised that the structural mechanics model is just that – a model, an approximation, of the real-world behaviour. Further, whatever the model, it must still align with best practice and generally accepted results by other tools or methods.

For this reason, since the outset, the authors have been very much aware of the need for validation and verification. Validation, of the methodology – including nuances, and verification, or (quantifiable) alignment with current best practice.

To this end, a comprehensive series of analysis models have been run, for both drained and undrained conditions, across multiple facets, including surcharge loading, seismic loading, water, retained slope angle, strong/weak soils, etc. These analyses have been independently undertaken using multiple software applications and approaches. An indicative graphical summary of verification of one of the soils types is shown below. Refer [2] for further detail.

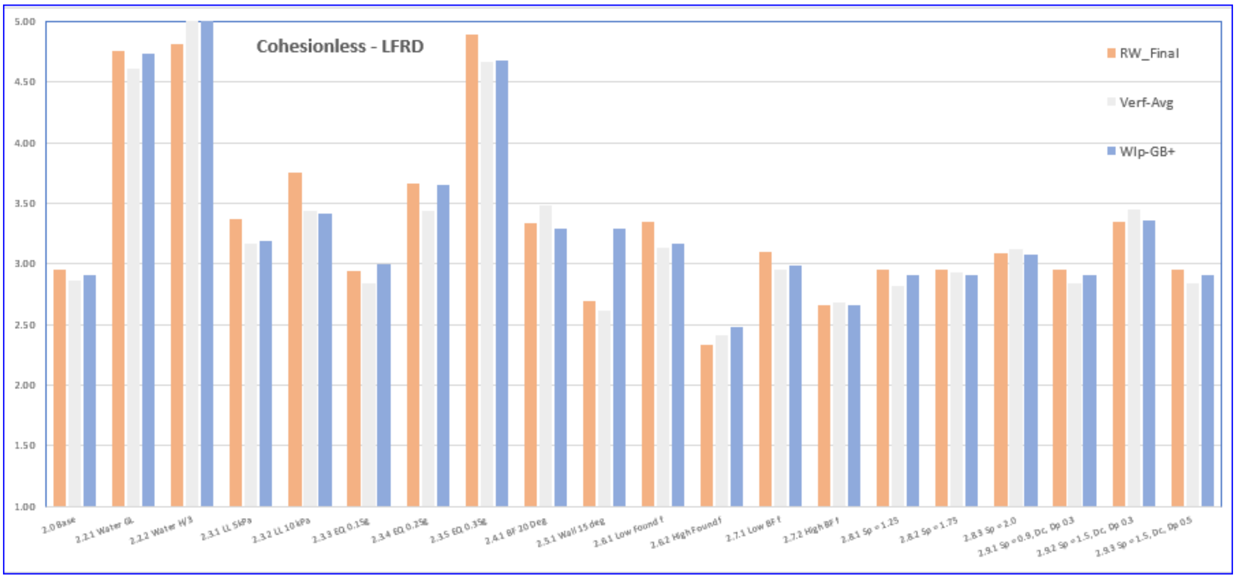

Figure 8: Drained (cohesionless model) results summary

CTP Adjustments Pressure Modifiers

During verification, we worked hard to get reasonable alignment across the different softwares - and the CTP methodology, for each verification test scenario. Overall, we achieved this pretty well for the recommended load factors. However, running some analyses with considerably reduced, or even unity, load factors, we found results considerably below that of Wallap – or 'software of record'. This potential un-conservatism bothered us.

Further investigation yielded evidence of inconsistency between our idealistic limit state stress block model, and 'real' soil pressures, especially at, and below, the rotation point. For this reason we introduced 'stress block multipliers' to both provide a 'tapered' behaviour around the rotation point, as well as to scale back the passive pressure at the foot of the pole.

These are shown graphically in the below snip, where we have adjusted the stress block model, right, to better align with the Wallap pressures, left.

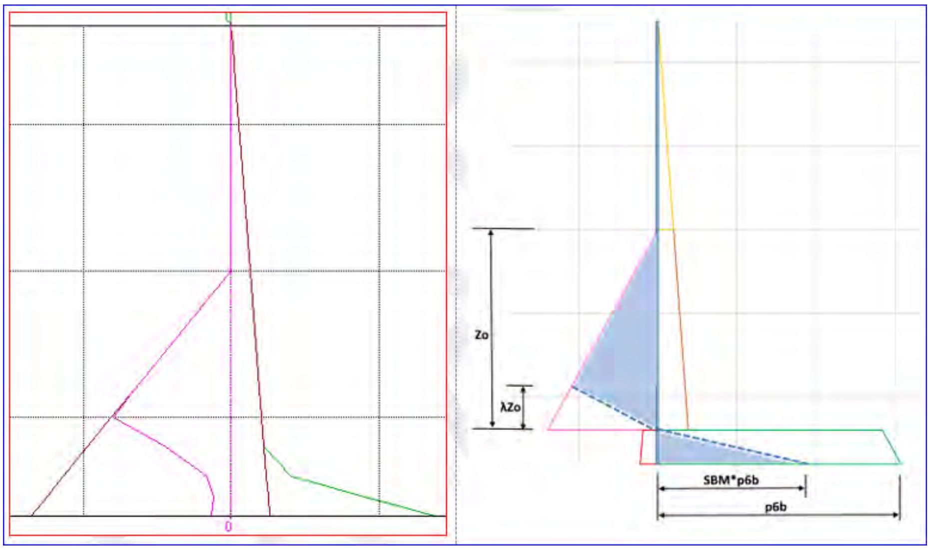

Figure 9: Pressure adjustment modifiers

Further detail is provided in [1].

Conclusions

This paper has presented some of the fundamental design aspects of the published CTP design guide, and the in-progress RCW design guide. There is, however, a huge amount of detail behind all this, and the reader is encouraged to read the design guides directly.

While the CTP design, - and app, have both been published, the RCW is yet a work-in-progress. We are currently undertaking a similar validation and verification process to that of the CTP, which is expected to take several months to complete.

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[1] Bird, G. D. & McPherson A.R 2024. Timber Pole Retaining Walls, SESOC Design Guide, *SESOC*

[2] Bird, G. Timber Pole Retaining Wall Design Guide (SESOC) – Methodology and Validation, *Presentation and paper to the NZSEE Conference 2024*

Advancing Non-Structural Building Element Engineering through Open Source Methods

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ABSTRACT

The calculation of non-structural building elements (NSE) varies significantly across the industry, as it is an emergent field without universally accepted methodologies. This presents an opportunity to develop best practice engineering methods for this field in a new and collaborative way, particularly through open-source sharing of computational tools and standardised libraries.

Using a plasterboard ceiling engineering method as a case study, this paper explores the development of open-source best practices. Key topics include integration within the GitHub environment, licensing considerations, and the benefits of shared function libraries and computational notebooks.

By fostering an open-source approach to NSE design, the industry can drive consistency, improve compliance, and enhance innovation, ultimately leading to more robust and efficient engineering solutions.

INTRODUCTION

Non-structural elements (NSEs), such as ceilings, partitions, and building services, have historically been peripheral in structural engineering. Despite their contribution to building functionality and their susceptibility to seismic events, NSEs remain under-emphasised in engineering education. The release of the ER95 Code of Practice (Stanway et al. 2024) marked a significant step toward standardisation, but consistent industry-wide implementation is still lacking.

This emergent status offers a unique opportunity. Instead of following the traditional path of slow codification through papers, organic dissemination, codes and conferences, the field can adopt agile, collaborative methods from the outset. Open-source development, particularly on platforms like GitHub, enables peer review and rapid refinement and dissemination of tools and methods, and is a proven model in software engineering. This model is well suited to the challenges and opportunities of NSE engineering method development.

BEST PRACTICE THROUGH OPEN-SOURCE COLLABORATION

Open-source development refers to the practice of publishing code and methods in public repositories, allowing anyone to inspect, use, and contribute to the work. For engineering, this enables transparent workflows, collective validation, and rapid iteration. Platforms such as GitHub support version control and peer review, while tools like Google Collab and Jupyter Notebooks provide a low-barrier interface for running and modifying Python-based calculations.

Table 1. Enabling Technologies for Open-Source Engineering

Tool / Platform	Functionality
GitHub	Version control, issue tracking, collaborative review, licensing
Google Colab	Cloud-hosted execution of Python notebooks; supports sharing and collaboration
Jupyter Notebook	Combines code, narrative, and visual outputs in a single interface
Python (NumPy, Pandas, Matplotlib)	Core numerical libraries for computation and data visualisation
Markdown / reStructuredText	Lightweight formatting for inline documentation and comments

Github repositories distribute code under a license. Highly permissive licences, such as MIT or Apache 2.0, maximise professional benefit by encouraging widespread use and reuse. More restrictive or hybrid licences, like GPL or commercial clauses, may suit those protecting proprietary implementations.

Table 2. Comparison of Common Open-Source Licences

Licence Type	Name	Key Permissions	Restrictions	Use Case
Permissive	MIT, Apache 2.0	Free use, modification, redistribution	Attribution required	Encourages widespread adoption and integration
Copyleft	GPL v3	Must share derivative works under same licence	No closed-source reuse	Ensures future work remains open

Restrictive	CC BY-NC	Free use with attribution	No commercial use	Academic or research-only applications
Custom Hybrid	e.g. Open Source + Commercial Clause	Open codebase, commercial use under agreement	Requires dual licensing	Enables collaboration while protecting IP

This balance is particularly relevant in emerging fields like NSE design, where shared standards support consistency, while advanced tools may retain commercial value. A simple framework can help map these licensing approaches against their professional and commercial impact.

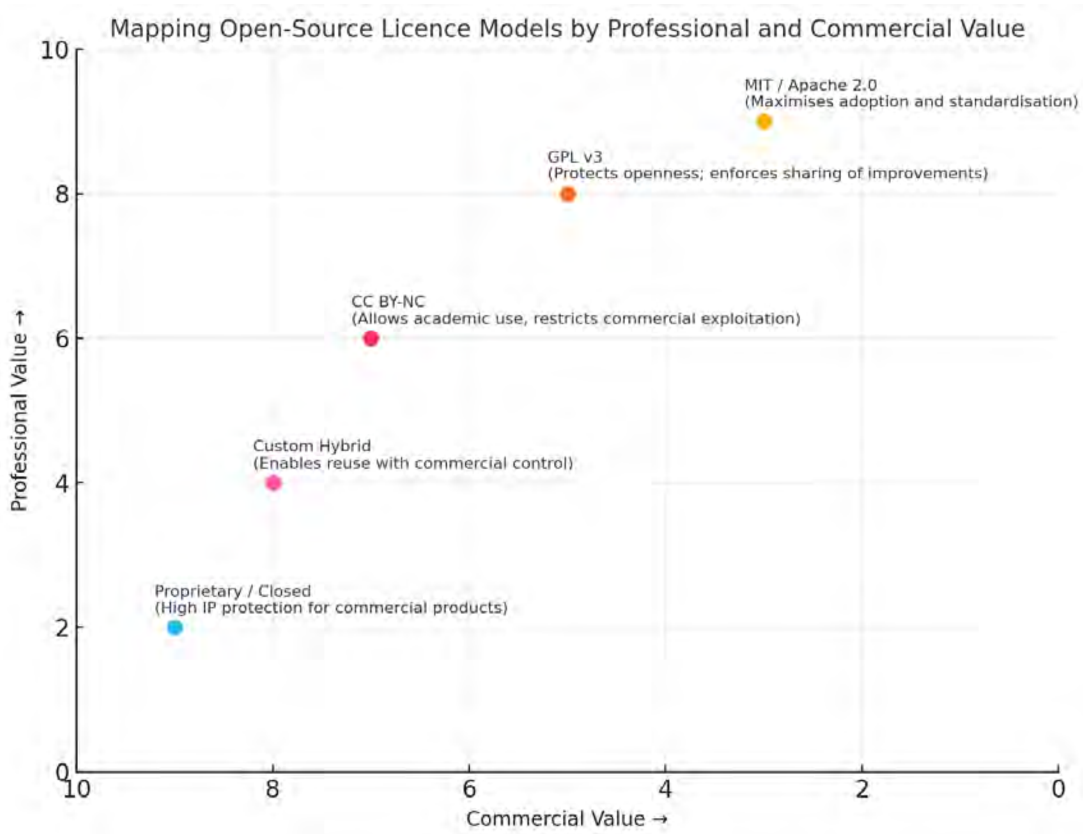

Figure 1: Graphing licensing options by commercial and professional value

Selecting an open-source licence is not merely a technical or commercial decision, it also reflects professional values around knowledge sharing. While engineering practice has traditionally favoured proprietary tools, emerging expectations for transparency and consistency, especially in fields like NSE design, increasingly support open access to foundational methods. This shift does not undermine commercial value but rather relocates it: as routine calculations become standardised and widely available, professional expertise will

focus more on interpretation, contextual adaptation, and complexity management. This transition positions engineering as a consultative discipline less defined by repetition, and more by insight and judgement.

CASE STUDY: SEISMIC RESTRAINT OF PLASTERBOARD CEILINGS

This case study presents a complete, executable engineering method for the seismic restraint of suspended plasterboard ceilings, structured to meet the NZBC Clause B1 pathway using AS/NZS 1170 and 4600 standards. The method is built in Python, delivered via Jupyter Notebook (.ipynb filetype), and available for peer review via Github. The method draws from modular, standards-aligned libraries hosted under https://github.com/Team-Brevity/standards. These include:

- `NZS_1170_5_2004.py` – Earthquake actions
- `AS_NZS_1170_0_2002.py` – General structural principles
- `AS_NZS_1170_2_2021_A2.py` – Wind actions
- `AS_NZS_4600_2018.py` – Cold-formed steel design provisions

Each library recreates the logic and data of the relevant standard in callable Python functions, separating project-specific logic from regulatory content. For example, this is the engineering method cell for the part spectral shape factor. Method specific assumptions are captured in the method (e.g. the rigid part assumption), however the calculation of part spectral shape factor is called back to the imported standard library:

```
# Set part period to 0 seconds (assumption valid for comparatively stiff parts)

Tp = 0

CiTp = NZS_1170_5_2004.part_spectral_shape_factor(Tp)
```

The above function calls the relevant imported library file:

```
# Example from the Python library (NZS_1170_5_2004.py)


def part_spectral_shape_factor(Tp):


    if Tp <= 0.75:
```

```
        CiTp = 2.0
    elif Tp >= 1.5:
        CiTp = 0.5
    else:
        CiTp = 2 * (1.75 - Tp)

    return CiTp
```

The difficulty of converting a standard into code is demonstrated here, the code is above and the standard clause from NZS 1170.5 is below. Standard clauses often rely on descriptive logic that doesn't translate directly into if/else code. To be machine-readable, they must be rewritten in code-compliant forms. However, using a high-level language like python results in code that is also human-friendly and can be read with minimal training.

8.4 PART OR COMPONENT SPECTRAL SHAPE COEFFICIENT | Amd 1 Sep '16

The part or component spectral shape coefficient, $C_i(T_p)$, is the ordinate of a tri-linear function depicting the shape of the horizontal acceleration of the part with the period of that part, T_p. The ordinates of the part spectral shape factor are given in Equations 8.4(1), 8.4(2) and 8.4(3).

$$C_i(T_p) = 2.0 \quad \text{for } T_p \leq 0.75 \text{ s} \qquad \ldots 8.4(1)$$
$$= 0.5 \quad \text{for } T_p \geq 1.5 \text{ s} \qquad \ldots 8.4(2)$$
$$= 2(1.75 - T_p) \quad \text{for } 0.75 < T_p < 1.5 \text{ s} \qquad \ldots 8.4(3)$$

Figure 2: Clause 8.4 from NZS 1170.5

Engineering Method Summary

Project Configuration: User defines location, importance level, and design life; these parameters determine seismic coefficients.

Geometry and Grid Setup: Inputs include ceiling dimensions and hanger layout, from which tributary areas and ceiling mass are derived.

Load Determination: Grid and lining masses are entered, and gravitational and incidental actions are calculated.

Seismic and Wind Actions: Calculations are made using library functions consistent with NZS 1170.5 and 1170.2. Wind loads are included as needed.

Bracing and Restraint Design: Seismic demands are converted into brace forces, checked against proprietary components or calculated capacities per AS/NZS 4600.

Minimum load allowances are retained to ensure serviceability. The full notebook is available under MIT licence at https://github.com/Team-Brevity/engineering-methods.

Key Advantages of open source engineering methods released on GitHub

Modularity: All standard logic is handled in external libraries, allowing independent maintenance, validation, and reuse. For example, when a constant such as the hazard factor (Z) is updated in the relevant NZS 1170.5 Python library, all downstream functions that depend on it, such as spectral shape calculations, acceleration coefficients, or bracing demand, automatically reflect the change. This eliminates the risk of inconsistent updates across project files and ensures compliance logic is centrally maintained.

Transparency: The notebook exposes every calculation step, enabling full peer review and easy auditing. Version-controlled platforms such as GitHub allow every change to be tracked, discussed, and reverted if necessary. Engineers can raise issues directly against the codebase, whether identifying errors, requesting clarification, or suggesting enhancements, ensuring that the method remains open to scrutiny and continuous improvement.

Extensibility: Engineers can extend or fork the method to adapt to new products, alternative grid configurations, or evolving code clauses. When published under open-source licences, these methods can be rapidly disseminated and improved by the wider professional community. Platforms like GitHub support branching and pull requests, allowing engineers to build upon a shared foundation while maintaining the integrity of the core method. Although the level of openness depends on the licence chosen, this structure supports a more collaborative and iterative evolution of best practice across the industry.

Standardisation: Adoption of shared libraries across the industry can reduce fragmentation and improve compliance confidence. In engineering, even small inconsistencies in method, such as misapplied factors, misinterpreted units, or outdated constants, can result in significant risk, particularly when safety-critical elements are involved. By consolidating these methods into shared, version-controlled codebases, engineers benefit from collective review and uniform application. For example, when the spectral_shape_factor() function is centrally maintained, all dependent methods across different firms and projects inherit the same logic and assumptions, reducing divergence. Mistakes can be quickly identified through GitHub issues, and corrections can be reviewed, tested, and merged into the main codebase with full traceability. This reduces both technical risk and the administrative burden of rechecking individual spreadsheets or custom implementations.

FUTURE WORK

The value of open-source methods extends beyond immediate engineering benefits. As artificial intelligence (AI) tools become more prevalent in professional practice, the need for grounded, verifiable codebases becomes critical. Large language models, while powerful, are susceptible to hallucination, generating plausible but incorrect outputs when source material is ambiguous or inconsistent. In the profession of engineering, these hallucinations could have disastrous results.

By creating engineering methods in transparent, peer-reviewed codebases, we provide a firm foundation for AI tools and agents to reason from. This grounding ensures that automated outputs reflect verified standards and logic, rather than inferred assumptions. For engineering, this means open-source notebooks and libraries can serve not only as tools for engineers but as training data and reference material for next-generation AI systems. In the long term, open engineering repositories will form the backbone of future engineering AI systems. These repositories should be developed and maintained by the engineering profession to ensure quality and robustness.

CONCLUSION

The engineering of non-structural elements (NSEs) is at a critical inflection point. While traditional codification has been slow to develop, the profession now has an opportunity to establish robust, shared practices through open-source collaboration. As this paper demonstrates, the use of computational notebooks and standardised libraries offers a transparent, reproducible, and extensible alternative to conventional methods.

The case study of plasterboard ceiling design highlights how structured Python notebooks, aligned to AS/NZS standards, can deliver reliable, auditable outputs while remaining adaptable to future code changes. When hosted on platforms like GitHub, these methods become living tools: open to critique, refinement, and reuse.

Importantly, this approach offers a pathway not just for NSEs, but for modernising structural engineering practice more broadly. By standardising routine logic and promoting shared tools, the profession can shift focus from calculation to insight, elevating the engineer's role as a contextual interpreter and strategic advisor. Open-source methods, while still emergent in the engineering profession, show promise as a model for broader reform across the discipline.

REFERENCES

Jan Stanway, Andrew Baird, Muhammad Rashid, Sara Hinz and Greg Preston 2024. ER95 Code of Practice for the Seismic Performance of Non-Structural Elements

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Canterbury Multi-use Arena: Structural Design Integration with Diverse Design Drivers

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ABSTRACT

This paper outlines the diverse and inter-connected functional requirements for the new Canterbury Stadium/Multi-Use Arena (CMUA), and their significant influence on the structural design. Aspects of this to be covered briefly in the paper (and presentation) include;

- The interaction of form requirements to meet turf-health, daylight, natural ventilation and thermal comfort, and complex internal and external acoustic requirements.
- Crowd dynamics and crowd sight-line performance.
- Structural + Geotechnical/Seismological interaction included a PSHA study.
- Design configuration of the 175m x 211m clear-spanning ~45m high steel roof structure that encapsulates the other 8 major structures of the rest of the facility.
- A shared foundation system for Roof and Bowl structures, whilst maintaining independent seismic systems (and clearances) above L00.
- Staged consenting and early main material procurement, and a fast-track programme.
- Careful load combination derivations.
- Resilience options for Primary and Secondary structure.
- Digital workflows to manage large amounts of steel connection design work, to local and international large-format-tube-connection requirements, and to process swift iterations of Non-linear Time History analysis.
- Early 2023 Main Works start on-site and April 2026 Completion.

The aim is to give a reader with an Engineering background an appreciation for how diverse and unique the design drivers were for this major ($683M NZD) project, and how the principal built-outcomes now visible on-site were shaped by that.

The Brief in a Nutshell

Essentially, the facility has to be optimised for the grass of the pitch to grow well year-round, and for a diverse range of performances/performers and the public to be attracted to the space and enjoy an 'atmosphere' while there: The Structure must enable these outcomes, not be the star of the show. Specific dot-points prescribed by the Christchurch City Council client were;

- A rectangular permanent grass Pitch
- A fully covered clear-span Roof, with partial transparency
- Raised and continuous Concourse (for ease of zone/stand transitioning)
- Various crowd capacities for major Events: 25K+5K sports, 35k full + 15k small concert
- Suitability for 'other smaller events' and for non-event day-use. IE: very multi-use

Turf-health, daylight, natural ventilation & thermal comfort, and complex internal and external acoustic requirements.

The ‘dance’ between these competing requirements/demands is the heart of the design process for CMUA. If the facility only needed to balance solar-gain and internal comfort, or solar-shading and noise breakout it would have been conventional and easy. But balancing solar (heat+light) gain, acoustic performance for noise-spill control outside and inside for 2-different-modes (Concert for sound quality, Sports for the roar), natural ventilation for airflow and thermal comfort, AND suitable grass turf health required something new. These had to be optimised to the ‘sweet spot’, with all other engineering outcomes secondary to that.

A shared workflow was established between the architectural, façade, structural and building sciences teams to coordinate models produced by each discipline, including the structure, pitch health, reverberation and noise spill, and airflow within the arena. This allowed for all critical elements to be considered simultaneously and compared through a multi criteria analysis in the early design. Initially, this didn’t influence the structural system optioneering greatly, but as the architecture+facade and engineering sciences teams converged on their shortlist of requirements for the optimisation of these outcomes, the interface of structural design on these (& cost & programme impacts) became a key part of the final selection process.

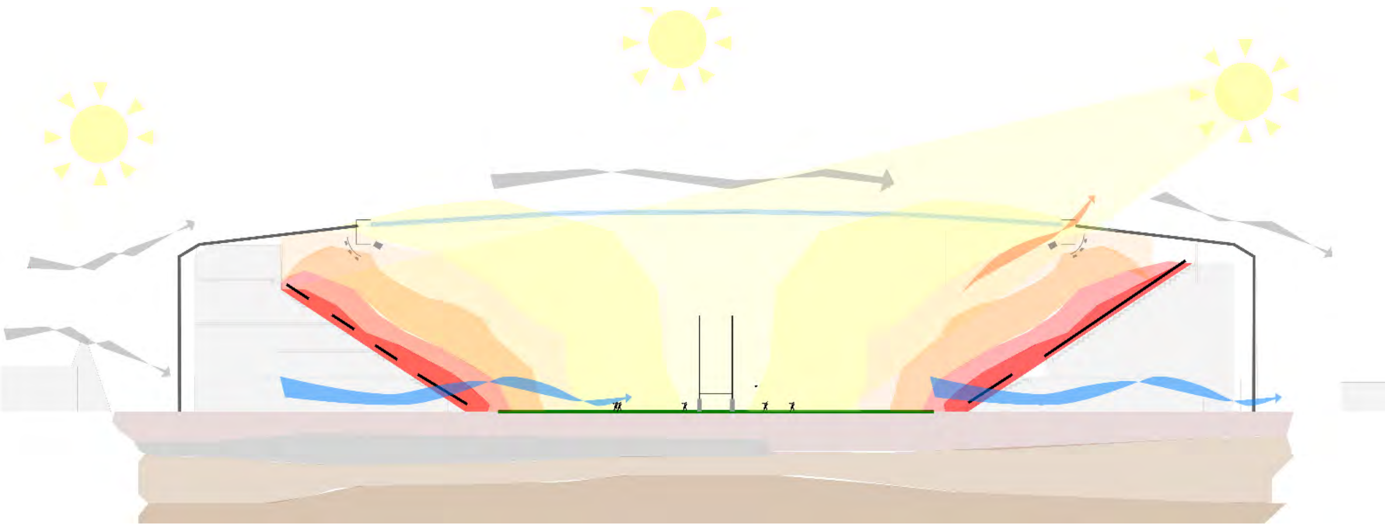

Figure 1: Principal E-W cross-section, representitive of the mixed solar/heat, air, and noise demands

Structural + Geotechnical/Seismological study interaction (+PSHA)

This aspect of the project is worthy of it’s own paper, and fortunately the Golders/WSP team have written one as was presented at the 2024 NZSEE Conference: ‘Challenges associated with the geotechnical design of Te Kaha – Canterbury’s new Multi-Use Arena’: (Murashev, Baker & Fearnley, 2024). This is recommended reading for more detail but in summary;

- The site was assessed as with some liquefaction potential, even though this was not observed at the surface in the February 2011 earthquake sequence.
- Presence of hard gravel layers within the geology posed a risk to deep piling.
- Rammed Aggregate Piers [RAPs] were selected for Ground Improvement across the Arena footprint + 5m beyond.
- A PSHA was undertaken early, highlighting a likely hazard level greater than the 1170.5 requirements at the time of design but reasonably well-aligned to the new draft NSM predictions. This was therefore adopted for the project, and underpinned the design of all of the structural (and façade) elements from Preliminary Design onwards.
- An iterative approach was adopted for soil structure interaction (SSI) analysis for the structural and geotechnical design of the foundations.

Staged consenting, early main material procurement, and fast-track programme.

Figure 2 below demonstrates how crucial a staggered, overlapping, fast programme was to achieving the targeted primary structure completion milestone and enabling an April 2026 finish.

For the structural team, this meant managing multiple large design package teams in-parallel and in co-ordination with each other. For example, the foundation design had to be confident of supporting an envelope of the likely final-design demand forces acting on it from structures above, as did the hold-down anchors for the Roof (installed nearly 1 year before Roof erection).

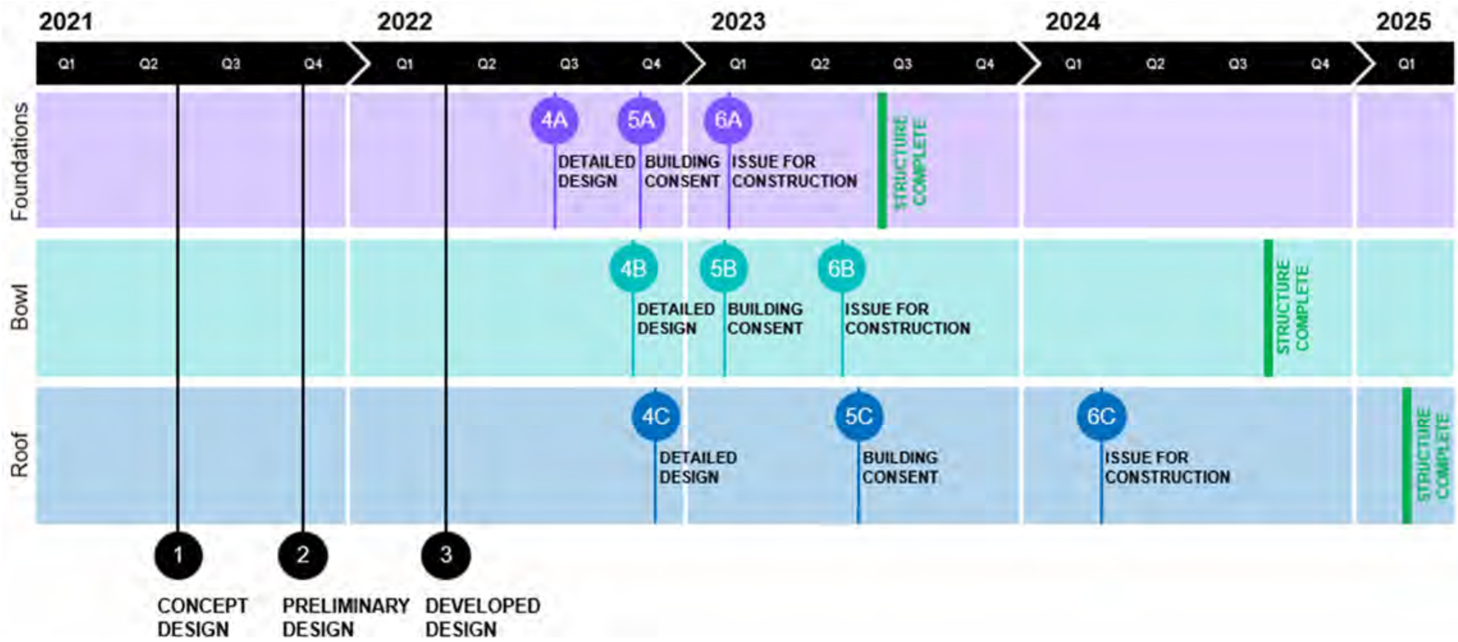

Figure 2: Primary Structure Design, Consenting, and Construction Timeline

The other crucial early-procurement decision made by the Main Contractor was to mandate concrete construction systems from Ground to L1, with Steel systems above. This enabled an

early-indent steel order when the design was further progressed and bought extra time for steel fabrication and delivery: the trade-off was a need for complex L1 structural node design.

Encapsulation of the entire facility with a Fixed Roof

This was one of the most important and influential structural decisions on the project, but it was initially approached from a perspective of how to optimise the programme (design, consenting, and construction) and the cost alongside how different options would support Turf Health, Ventilation and Internal Environment, and Acoustic outcomes. Once the studies had been completed and it was clear that a solution would align to all those drivers where the Roof structure <u>encapsulated</u> *and was separate from* the rest of the facility/structures, then;

- The Bowl could then be divided up into sensible separate structures of its own based on their differing performance requirements: Multi-function West Stand, seating focussed East + South Stand, NW Stand with Truck Dock, and simple Lower Tier areas.
- The Roof could be optimised for its key drivers and more intertwined nature with the Ventilation + Solar + Acoustic assessments by/with the Engineering Sciences team.
- The derivation of the necessary Roof-Bowl seismic gap values near the roof 'knee', and the interaction of net Foundation demands, then became the two remaining key interfaces between the Bowl Structural design team the Roof Structural design team: enabling them to run in-parallel and with the necessary design programme speed.

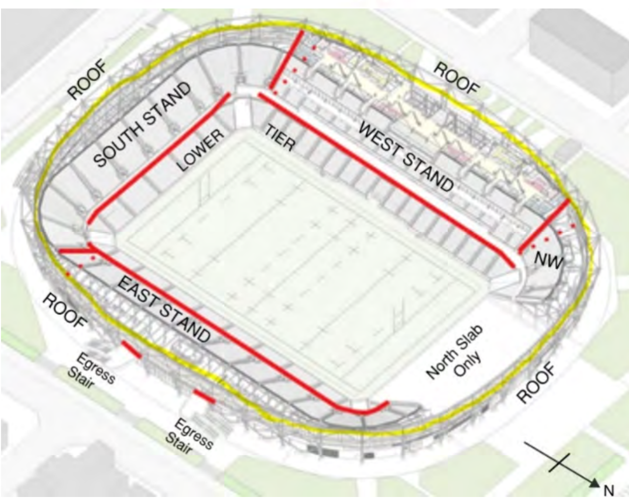

Figure 3: Typical E-W Isometric cross-section, + Seismic Joints between Bowl Structures & Roof

Support of the entire facility on a Shared Foundation

Once the above decision about Roof-Bowl (dis)connectivity was made, and the principal strategy for Ground Improvement (not conventional deep piles) was set, the specifics for the foundation design could be tackled:

- The 'Ring' of 40# Primary Roof columns clearly required a Ring-beam foundation below it, with Tension-capacity Rammed-Aggregate-Piers (T-RAPs) adopted at locations of net-tension at the ring beam base in the vicinity of the Roof brace touch-downs.
- The West Stand required a Raft-Slab of varying thickness: thicker in areas of higher overturning resistance from the braced bays above, especially the taller back-and-sides.
- The East & South Stands were able to be optimised away from a Raft and to a Ground-beam Grillage arrangement, helped by their very regular radial grid and the presence nearby of the tied-in ring-beam around the taller back of the Stands where the Bowl BRB loads were then resolved with no further net-tension elements required.
- Ultimate base-shear take-out was through simple friction of the base of the integral foundation 'donut' with the ground.
- Differential settlement was looked at carefully for each foundation type plus junctions.
- The Ground floor slab FFL in the West Stand was set above the expected 1:150 flood water level, and the GF Slab in the East & South Stands (between the Ground Beams) was designed as 'secondary structure' meaning it could be replaced or removed with no impact of the primary structure foundation ties or diaphragm (as functions change).
- The North Stand Slab is also designed without contribution to the support or take-out of load from any of the other structures as it is designed solely to withstand: loads from a temporary (1-2 times a year) 5,000 seat scaffold 'bump in' stand, and loads from Major Concert stage setup and truck & EWP access.

Crowd dynamics and crowd sight-line performance

Dynamic crowd/vertical excitation behaviour in large venues / Arenas is a specialist area and we were able to bring our global experts onboard to assist the Kotui team on this aspect. This informed additional areas of stiffness to the long-span East & South & West Stand raker-beams.

Crowd management design in the horizontal plane also influenced the placement of bracing lines and column grids, in particular shaping the structural configuration for the L1 & L2 West Stand lounge spaces (key to the operation of 'other small events' from the Brief), incorporation of corridor accessways directly between k-brace lines, and the bracing configuration to provide the necessary openness and access through the East & South Stand L1 Concourse plane.

Detailed patron sightline analysis (for every one of the 25,000 seats!) was undertaken by the Architects from early on and was key to setting heights, slopes and extents of the upper-tiers (including the 'wave' line to the back of the East & South Stands). From the start a key feature they also drove was the means of obtaining a good 'viewshaft' to the pitch from the L1 Concourse area: as the figure below indicates, this pushed the raker end up 2.5m in the air and set the need for a large brace element to transmit the above seismic loads across the gap.

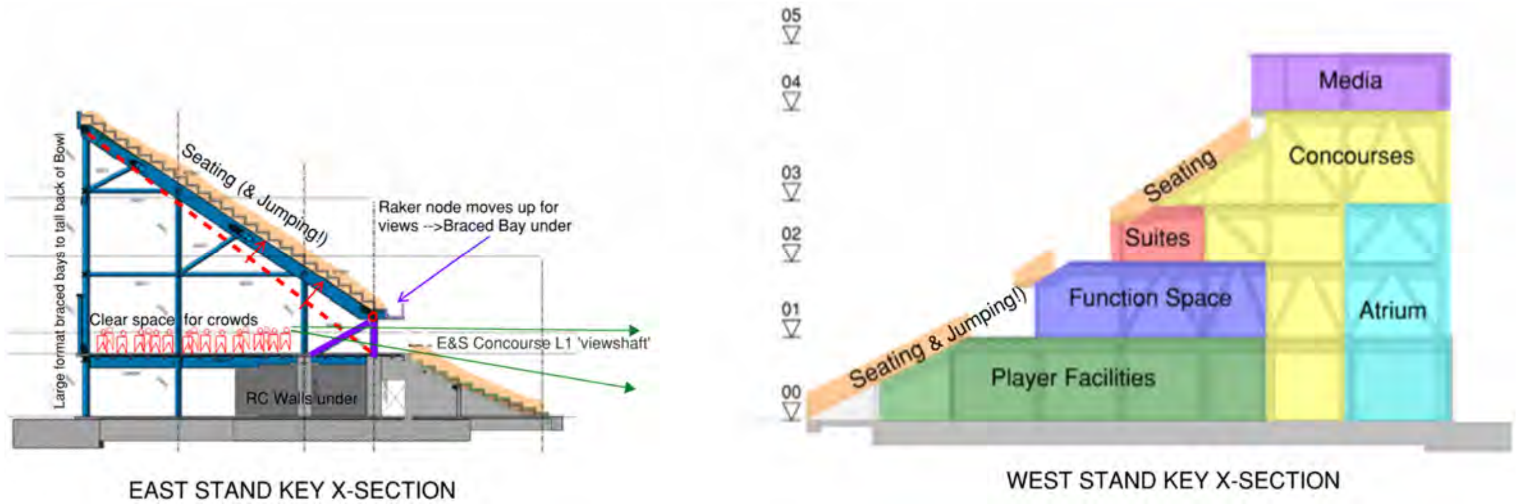

Figure 4: East and West Stand cross-sections showing key Crowd/People demands

Resilience options for Primary and Secondary structure.

Given that based on the above points the project was already on-track to deliver a design with;

- A Ground Improvement 'mattress' supporting a robust integrated foundation system, reducing the expected impact of liquefaction and differential settlement.
- IL3 ULS & SLS seismic design loads derived from a PSHA giving coefficients aligned to the new higher NSM predictions instead of the current 1170.5 values (a Cdt basic of 1.4(g) for the Bowl structures).
- The adoption of regular distributed bracing systems, with capacity design principles.

The detailed structural design of the CMUA buildings meet the client brief for seismic resilience.

Where the project was able to go further, but within the budget, included;

- Adopting repair-able / replace-able structural 'fuse' systems for the super-structures: Buckling Restrained Braces (BRBs) for the East & South Stands and the Roof, and bolted-link Eccentrically Braced Frames (EBFs) in the West Stand. Several of the required BRB units for the Roof were uncommonly long (~12m) and the project unlocked access to a local Christchurch testing facility for these *and* to local NZL fabrication for them, enabling the team to work in closely and quickly to agree the final detailed parameters and design for these with no net loss of programme.
- Detailing drift-capacity and/or suitable restraint into the Façade and Services designs.

In addition, the client is exploring the options for monitoring instrumentation (accelerometers, strain gauges, etc) of key structural elements to assist with swifter post-event operations.

Digital Workflows for Selected Structural Computation Tasks

On the Roof structure this was particularly important for the delivery of an optimised solution. The team used tools enabling an integrated computational workflow across architectural, structural, and specialist disciplines. This minimized individual discipline modelling and promoted integration (and speed!). Adopting these workflows allows for rapid multicriteria analysis, and then significant design optimization, leading to greater design and cost certainty and reduced risks and to achieving a sweet-spot between the multiple competing criteria.

Specific to structure, digital workflows were key to;

- Iterations of a long-list of load combinations (snow, rigging at 100ton allowance, erection, wind, thermal) with the large roof truss CHS member and node-connection design, and that influence on roof self-weight and *its* impact on the seismic re-assessment cycles,
- Processing roof shape iterations for Solar/Pitch Health, Natural Ventilation, & Acoustics,
- And to process swift iterations of the Non-linear Time History seismic analysis, including the impact of different BRB configurations.

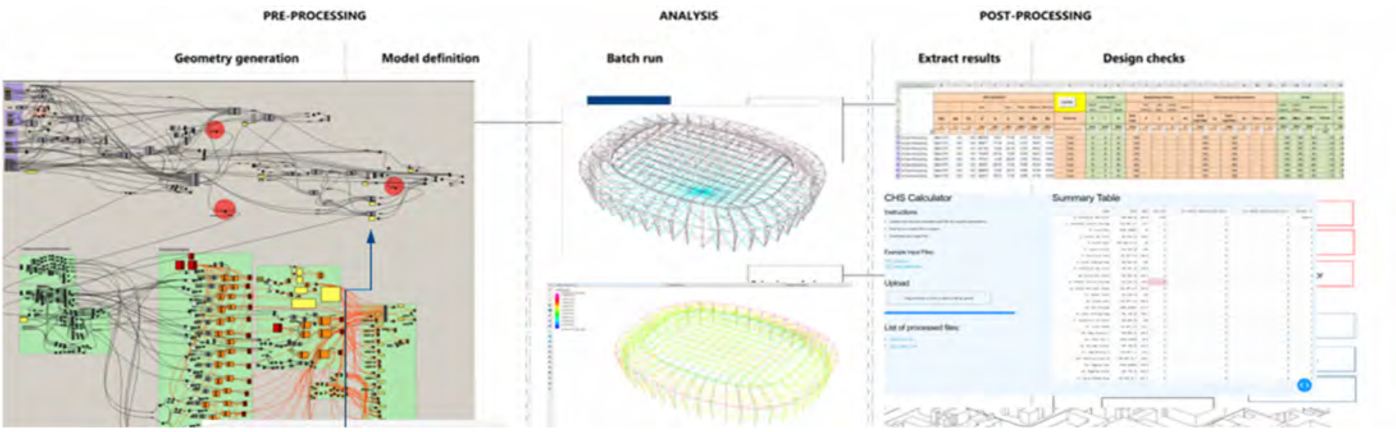

Figure 5: Sample roof digital workflow snippet

Extra for Experts: Construction Sequence & Temporary Works: From Ink to Insitu

The design of the temporary works and the detailed construction sequencing and stability, particularly for the 211m x 175m clear-spanning roof structure, was a massive multi-party undertaking in its own right. This could easily be the topic of another separate paper, however the below marked-up figure captures the basic approach. In particular note the 'soft-link' element integrated into the temporary prop/support from the back of the Bowl to the Roof columns during erection, with the ductility and capacity of that connection (under scaled PSHA loading) synced/capped to the elastic capacity of the in-construction Bowl framing. This enabled a self-stable Southern portion of the roof to go up swiftly, and the full-height temporary towers for the rest of the Roof to be positioned without clashing with the Bowl seating extent.

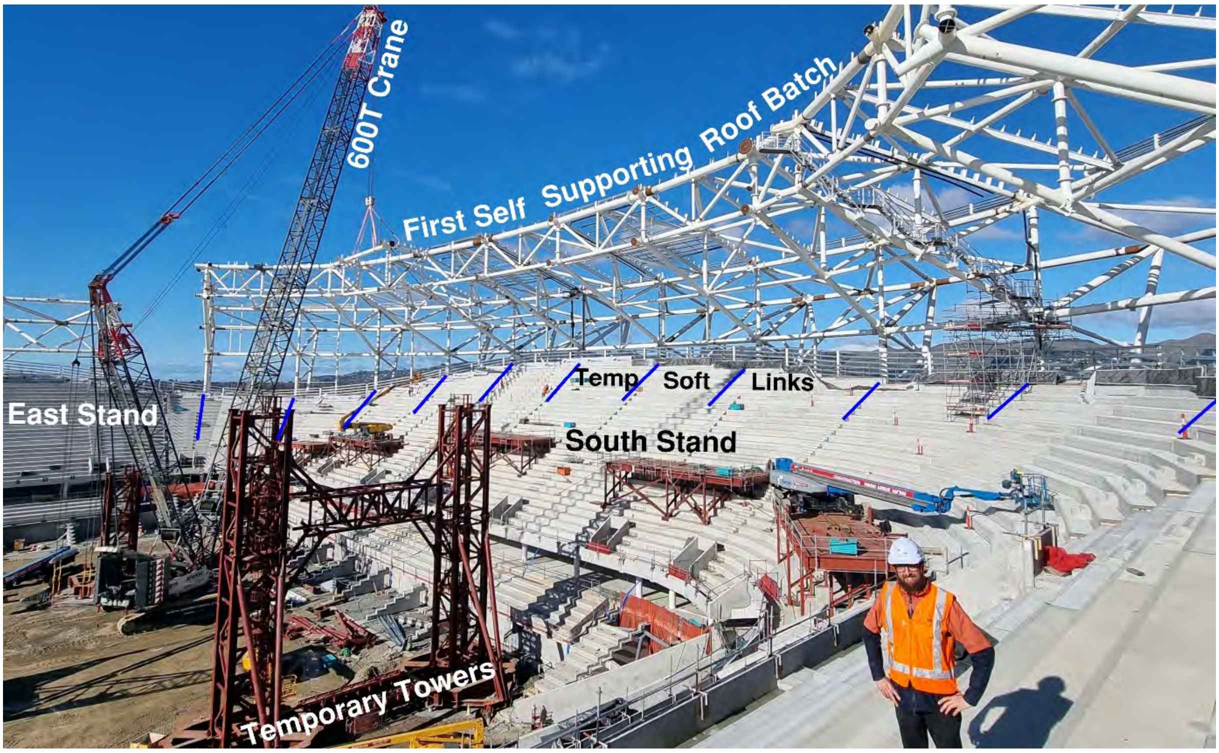

Figure 6: South end of CMUA under construction, including Roof erection elements

Conclusions

The 'dance' between the key desired outcomes and competing requirements for; Acoustics in and out, Turf Health, Solar Gain Control, Natural ventilation, Crowd Comfort & Dynamics, and attaining an attractive venue 'atmosphere', were all at the heart of the design process for CMUA.

Together with Cost and Programme drivers, and the elevated results of the PSHA study for the site, these led to a number of key design decisions for the project: Roof-Bowl dis-connectivity being the key decision that all other structural design decisions flowed from. Many of these are visible in the built-form on-site, but some are not: the aim of this paper has been to bring these 'into-the-light' for when the inquisitive engineers of New Zealand enjoy a visit to the Arena (under its new name of 'One New Zealand Stadium at Te Kaha').

<u>Acknowledgement:</u> The CMUA project is a true multi-discipline effort, with the following team;

Main Contractor & Kōtui Consortium Lead: *BESIX Watpac*, Contracting partner: *Southbase,* Architect: *Populous + Warren & Mahoney*, Structural engineer: *Mott MacDonald*, Structural peer review: *Lewis Bradford*, Façade peer reviewer, Stadium Sciences, & Wind engineering, Vertical transportation, ICT, AV, & security systems: *Mott MacDonald*, Mechanical, Hydraulic & Electrical (to DevD), & Acoustics & Civils: *Mott MacDonald + Powell Fenwick*, AI Consultant; *None,all human ingenuity*, Fire engineering + fire protection: *Holmes Fire*, Traffic engineer: *Abley*, Geotechnical engineer: *Golder/WSP*, Peer review of Geotechnical PSHA: *Bradley Seismic Limited*, Turf Consultant: *Sports Surface Design & Management*, Landscape Consultant: *LandLab*, Kitchen Consultant: *Wildfire*.

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Neglect Secondary Structures at your Building's Peril

A.G. Cattanach

Dunning Thornton, Wellington.

ABSTRACT

The assistance a building's Structural Engineer provides to a project outside the primary structure varies significantly across practice, region and project. Damage to non-structural elements in recent NZ quakes has also varied significantly.

This paper postulates an active approach intended to maximise the value from an Engineer's skills, without imposing significant additional burden's on their scope of work. An everyday approach to low-damage design, these techniques should not add significant project cost. Enabling the design team and the builder to use their skills to contribute is an important part.

A number of techniques will be discussed, and projects where they have been used provided to illustrate the technique and its effect on that project's design.

The Structural Engineer often has the best three-dimensional understanding of the structure and the spaces around it. By doing our part to masterplan how the rest of the building moves with the primary structure, we can achieve better buildings at a minimal price tag.

INTRODUCTION

This paper has been precipitated by recent interactions with contractors, project managers and quantity surveyors, all suggesting recent cost blowouts related to secondary structure in projects. This, unfortunately, is a little bit of a poor indictment on our profession: regardless of how efficient or well documented our primary structures may be, secondary structures have created time and cost over-runs (causing this paper's slightly provocative title).

Secondary structures can fall in the gap between architecture, building services restraint and passive fire, and cause significant delays, confusion, and of course, cost, increases during the construction period.

This paper identifies a number of techniques and approaches that the author has found useful.

It's not just a case of documenting more items. It's more a case of taking the time to organise an efficient secondary plan, sharing responsibilities with the team, and documenting this so who does what is well organised and complete. The costs of doing this are small in terms of consulting, but do require broad thinking and careful communication with other team members.

MASTER PLANNING THE CEILING CAVITY

The organisation of non-structural elements within a building is a shared Masterplan between the architect, services engineer, the fire engineer and the structural engineer, While certain aspects on the visible parts of the building are often driven by the architect, those behind the interior surfaces can sometimes fall in the gap between the needs of fire, services and seismic restraint of services to have the good organisation that makes secondary structure efficient.

This paper suggests that the structural engineer is one of the best people to carry this out. We think in three dimensions, we have a picture of what the underside of our structure looks like, and we understand the building's intended movements inherently from our design of the primary structure. Many building services engineers and architects I have dealt with have struggled to understand the master planning of the ceiling cavity because they're not taught this “non-visual” aspect. Communication is key, and later I will illustrate simple diagrams for how this can be done.

This master planning skill requires a high-level understanding of the fire engineering, the building services and the Architect’s intent, if a Structural Engineer is going to provide good integrated design. This integrated design is both primary structure in the right place to assist the organisation on non-structural elements, and the secondary structures that facilitate them.

Firstly in plan, one should not just consider the column grid, but also the rhythm of the beam grid versus floor span. If, for example, the building has fan coil units (especially if large), a greater floor span between the secondary beams may be appropriate.

Secondly, thinking in section, first traditional approach is to take the primary structure up as high as possible, allowing free movement of the services below. Whilst this may initially seem simple because it absolves the engineer being involved in master planning the services layout, it inherently pushes the services away from the structural soffit and therefore requires each brace to the building services and ceiling to be longer than it would if the cavity was shallower. A second strategy is to provide a deeper structure and to thread the services through it and reduce the cavity depth (and secondary bracing length). This requires a lot more communication between the services engineer and the structure engineer. Co-ordination is required for not only the initial layout, but the inevitable changes that will occur throughout the life of the building, and even sometimes changes of the fit out given during the construction period and design period.

Whilst these seem like simple moves from a primary structural point of view, the knock-on effects to the size, regularity and complexity of the secondary structure is significant. An example of this planning is the biochemistry Building at Victoria University of Wellington (Cattanach 2016) we arranged Vierendeel trusses at a 6.6 metre grid, matching the lab planning grid. The verticals in the trusses considered the longitudinal services, the bottom chords of the trusses the lateral distribution and restraint of the ceilings.. This is shown in the diagram below.

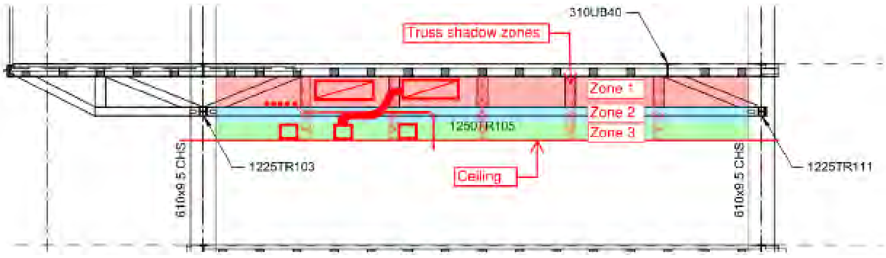

Figure 1: Section through TTR lab floors

Services that run close to the soffit are obviously cheaper, not requiring BSR. Unwittingly, this is a low damage design discussion: those attached in a rigid manner are shaken less hard than those on flexible mounts. This reduced acceleration for rigid attachment of parts is included in the draft DZTS 1170.5 issued last year. It follows that any rigidly attached services are going to be a lower risk of damage during seismic shaking, and so this simple move can significantly reduce the risk of services, leaks or breaks. The design team can explicitly balance the following pros and cons for the heights that ceilings and services are placed below the soffit.

Another tool to put on the table at the design meeting, is the lengths of hangers for services for when those services do not require bracing, proximity again saving secondary structures.

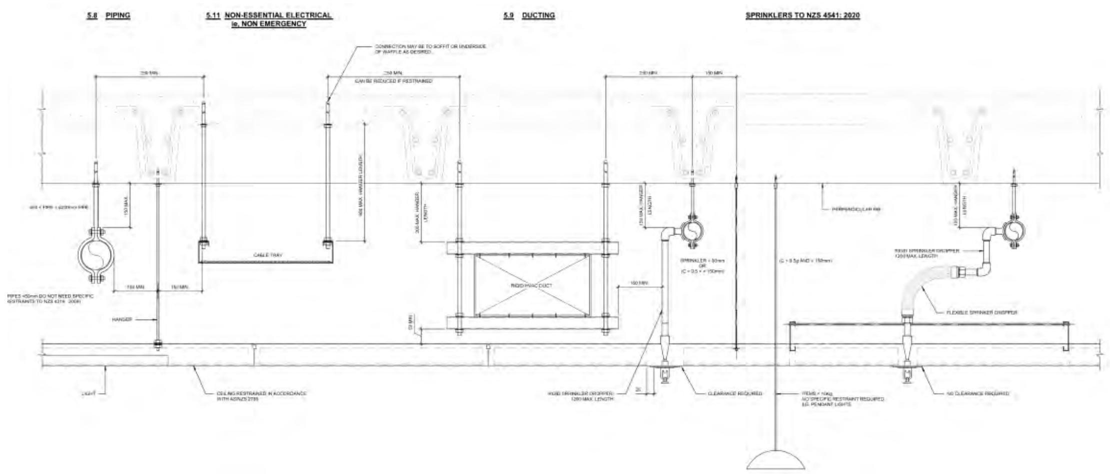

Figure 2: Hangar lengths that do not require bracing

CEILING AND WALL DIAGRAMS

It is very simple for the engineer to draw cross sectional diagrams through a floor in a cartoon manner to reflect how the building moves.

There are two basic strategies to communicate. Firstly, full height partitions move as the build with building drifts. Alternatively, partitions braced at ceiling level can keep the ceiling cavity rigid decreasing the risk of damage within the ceiling cavity, at the expense of concentrating all the partition movement between the floor and the ceiling. Bulkheads are also most often braced to ceiling above. Risers often have full height partitions, but in the detailing required around riser exits implies a bulkhead. This requires team buy in to one or other strategy, and involves considering the complexity of fire sealing, air sealing, access for maintenance, and of course, architectural integration.

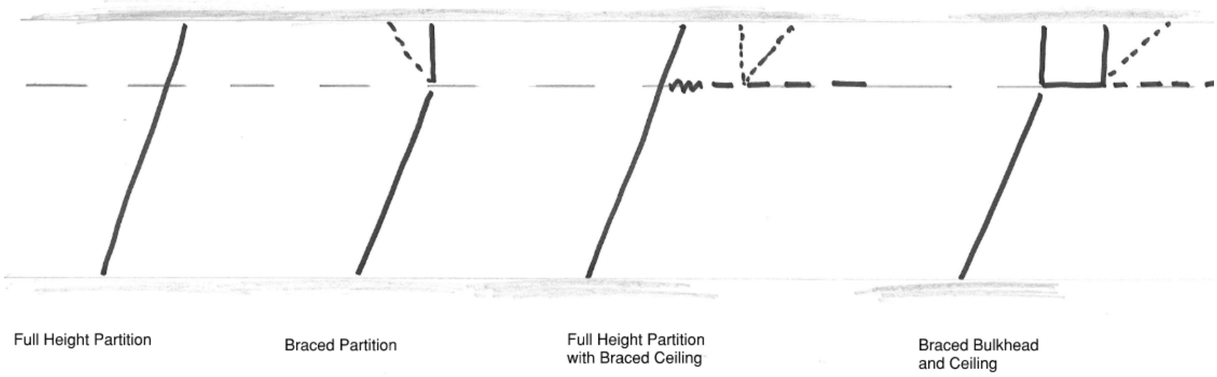

Figure 3: Wall and Ceiling Diagrams

These diagrams should be incorporated in the documentation passed to the contractor for those proprietary designers that come during the construction phase. This allows the contractor to assist communicating these strategies to the non-structural trades. For example, suspended ceiling designers will be very interested in the movement intent for different partition types, so they can decide where the ceiling can be restrained to partitions and where movement joints to walls are needed.

USEFUL RULES

Scale

Scale is important. An Architect does not have engineering training but can experience the strength and movement of a component of a certain size by pushing on it. When the scale is larger than what a person can physically experience, this is where the Structural Engineer can use maths and physics to extrapolate beyond this body experience.

To bring building scale to a human size is what secondary structures do. For example, a wall six metres high tends to be twice as thick and move twice as far as an Architect is used to experiencing. One can just to use bigger studs to support it, a stronger movement head, bigger connections: all of these typically require specific design. Also, the midspan movement of a tall wall even when normally stiff are not often easily coped with using standard detailing at a corner or end. Alternatively, a secondary frame can reduce effective partition heights to those more commonly experienced but needs to be balanced against the secondary frame's cost.

Arm span rule:

In the Fisher & Paykel project in Auckland, the "Shed" is the R&D lab and has significant servicing and requirements. A floor height of six meters was established early with the space above for servicing a broad variety of spaces and allowing for the many expected changes that will occur over the building's life. We created two concepts. Firstly, an open secondary ceiling structure between reticulation and the research spaces below. Second, organising the longitudinal services runs into an exposed "services highway" for the main feeds to the lab spaces.

All of these secondary structures have been invaluable in coordinating and restraining the very complex services within this multi-use lab building. The building services restraint KCL could then use normal scale restraints to attach services to a piece of the secondary In our office we used the "arm span" rule in our design. If you put one hand on something that needed bracing, you should be able to reach a piece of primary or secondary structure with your other hand if you were in the ceiling space. This was particularly useful in the upper floor where we had a lightweight roof.

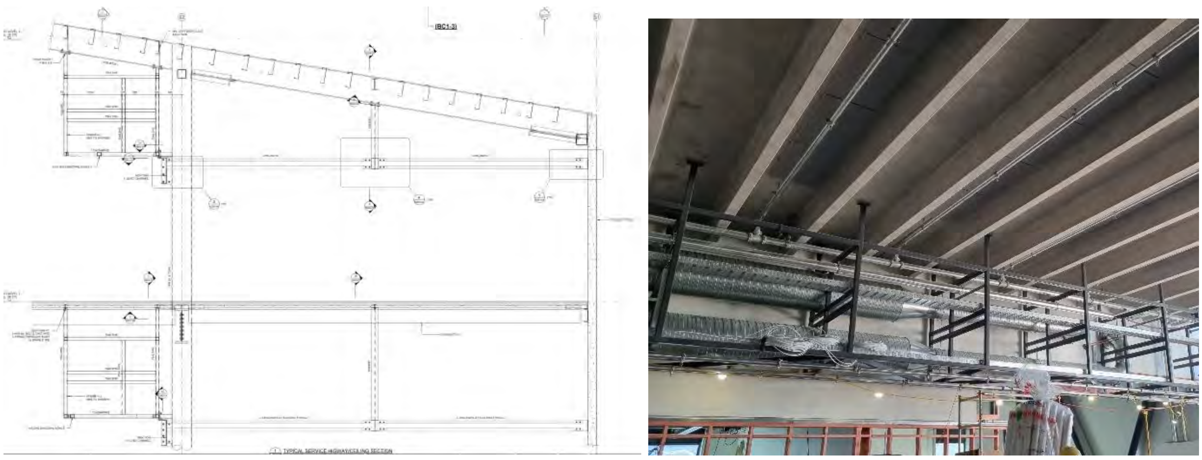

Figure 4: Section through F&P "Shed" and photgraph of the services highway

Lightweight Roofs Are Not a Slab

Most guidance for the bracing of services and ceilings is for a position below slabs. In New Zealand, with low-rise construction common, and with structural engineers trying to keep mass at height as small as possible for seismic reasons, we often have lightweight roofs to the top floors of our buildings. This needs a whole different approach for the bracing of services for two reasons.

One is slope: that standard details cannot repeat: at some point, the roof will get a sufficient distance above the ceiling that bracing becomes too flexible to be either efficient or effective.

Secondly, attachment for braces needs proper consideration: a purlin in bottom flange, whether timber or light gage steel, has very little capacity. If there is no plywood sarking, there is no diaphragm.

Instead consider a master planning exercise where services are concentrated in discrete strips, and support and bracing structure provided here. The Tākina project in Wellington is a good example of where this was done effectively, in a large span roof truss. A strip of ComFlor 210 running between bottom chords of secondary trusses provides an organising element for electrical, fire and data distribution, as well as restraining the bottom cord of the trusses.

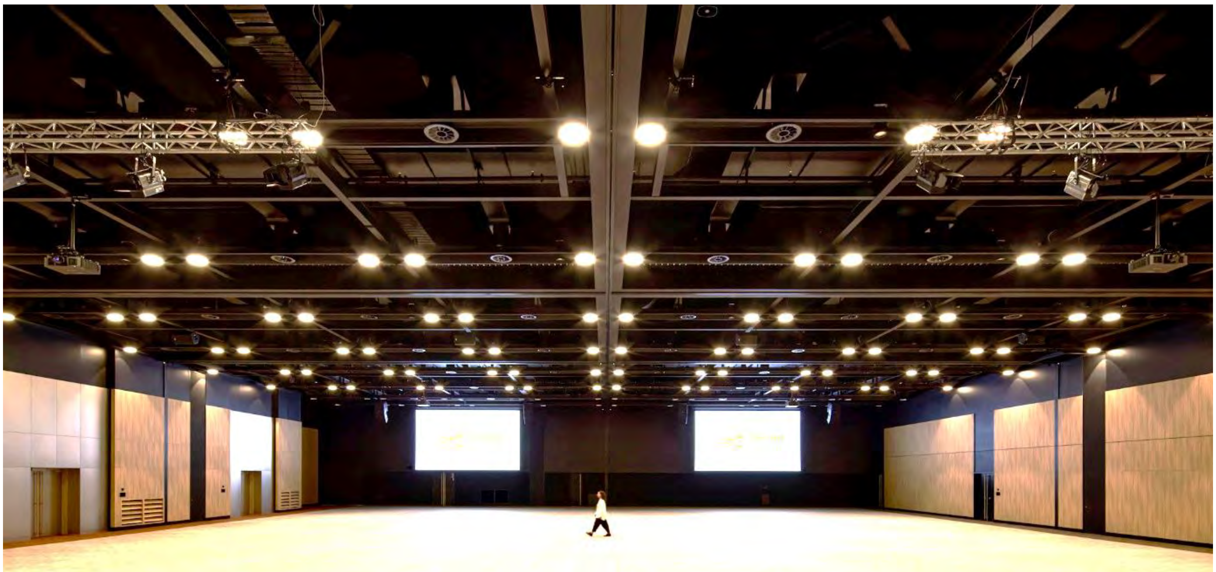

Figure 5: Tākina Servicing: the ComFlor runs left to right with lights, fire services and difuserers

By providing these secondary elements as structural they are often priced at a primary structure cost. Secondary structure is sometimes priced differently, especially if provided by the metalworker rather than the main structural steelworker

Light gauge steel purlins as cable trays

These are typically stronger than cable trays and can be used to mount services and piping as well as light objects like lights or small fans. Aan example where this was done against a complex roof is the Mount Pleasant Community Centre, where these purlin cable trays run down each side of the main hall, carrying both data and power and providing support to the main ducts for extract air.

Figure 6: Mt Pleasant Community Centre Services

Portal Lateral Restraints

The areas the structural engineer most often needs their lateral restraint around the knee and apex of a portal is often also the area where services run longitudinally to the building. If these bracing structures can be used for carrying services too, they're economic in providing two functions with one form.

The “Bounce Post”

This is a simple concept we derived from trying to control deflections in buildings where a cantilever happens on multiple floors. If these cantilevers are connected together with relatively small posts, the footfall on one floor is directly transmitted to multiple floors, hence stiffening the whole structure and reducing vibration. These posts can be small because only the differential load is transmitted by the post. These do not need to be limited to cantilevers.

A simple bounce post near a riser can fit within a normal core wall thickness, that can significantly reduce the requirements for movements at rise or exit. If this is done carefully, the need for movement heads in partitions can be eliminated, and the corresponding costs of flexible connections and ducts and the way that fire pacifier ceiling is done In this area, highly simplified.

CROSS-DISCIPLINE OWNERSHIP

A suggested approach is for the Structural Engineer to provide the master planning to the ceiling spaces to the preliminary design level. This can be done very easily on a plan at a highlighter level. Busy areas by the risers can involve integrated structural steel frames, which would typically cantilever down from the soffit where there is not sufficient space for diagonal placing. As we move out from these congested areas the services can move from integrated cantilever hangers to shared frames but with braces, then to individual hangars/braces for each trade. The intent diagram can remain in the Structural Engineer's Specification (refer below), and the detail around this intent be captured in the Architect and Services Engineer's documentation.

SAFETY IN DESIGN / MAINTENANCE

Maintenance can be addressed through this process. Deep ceiling cavities are awkward to access and can be organised by secondary structures. One strategy was demonstrated in the Alan MacDiarmid where we included a lightweight plywood purlin separation in the triangular roof cavity. Nicknamed "ghost floor" by the Contractor, the structure proved invaluable in separating the services that were running to the plant room from the standard services accessed from the ceiling below.

Figure 7: Fume extracts in Alan MacDiarmid running over the "Ghost Floor" (bottom of image) below lightweight roof

SPECIFICATION

The discussions had in the design team forum end up on the drawings as the “what”, but often we forget to communicate the “why” to the Contractor. We draw the finished building, but not the reasons these decisions were made. The diagrams made in the design process can be passed on. If the Contractors know what you intended, they can also help you check the job is being done by the subcontractors.

Our practice uses the Non-structural elements specification to identify requirements. The Seismic floor accelerations, building movements and loadings that are normally covered, but also guidance on how fixings can be done in different areas of the buildings to the various floor systems, beams (or not) and wall elements. The diagrams used in the design phase can be included to describe the secondary movement strategies from in the wall and ceiling diagrams and layout master planning.

MASS TIMBER

Mass timber buildings often require a slightly different strategy from conventional structures. There is more structure: successful services restraint can be through direct attachment with simple carpentry screw fixings, rather than the usual bracket and strut systems.

Mass timber plantrooms require a different approach. Mass timber floors rarely have the strength to cantilever heavy items of plant in the practical size of fixings that can be provided. We have overcome this in two ways before. A structural concrete slab above the mass timber slab can spread loads and assist with acoustic separation, and only have more discrete attachments to the structure below. A second approach is to provide a greater capacity in the roof of the plant room in terms of bracing and as a diaphragm, and to show the service restraint engineer that any tall elements need to have strongback elements that span from plant floor to plant roof to restrain tall plant.

CONCLUSION

By being involved in the master planning of how the non-structural elements run through the building, the Structural Engineer can provide significant value to the project by simplifying the masterplan of how services routes, ceilings and walls can work together. This approach to integrated design takes a little more thinking time but should not cause significant additional documentation. The cost is simply spending the time empowering the Architect, Services Engineer, the Fire Engineer, and importantly the Contractor in understanding the Masterplan: the big idea of how everything inside the building should move and be braced together. The more detailed tasks associated with restraint, framing movements and bracing can be done by others if you've communicated the high-level ideas effectively.

If this is done, well, we make our buildings more economic and reduce the risks of seismic damage.

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Sensitivity: General

SESOC CONFERENCE 2025

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Draft New Zealand Guidelines for the Assessment of Reinforced Concrete Masonry

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ABSTRACT

This paper describes the intent and form of the proposed guidelines for the assessment of Reinforced Concrete Masonry (RCM) structures, which is scheduled to be open for public submission around the time of the SESOC conference. This is a Joint Committee for the Seismic Assessment and Retrofit of existing buildings led initiative, supported by the Ministry of Business, Innovation and Employment. The initiative's purpose is to produce a new chapter for inclusion in The Seismic Assessment of Existing Buildings, the New Zealand guidelines used to assess buildings' *%NBS* ratings. At time of writing, the final form of the RCM guidelines has not been voted on, so some change could occur between the information presented in this paper and the draft guidelines that are issued for public comment.

INTRODUCTION

The Seismic Assessment of Existing Buildings (MBIE, 2017), commonly referred to as the Guidelines, has become a vital resource for New Zealand structural engineers. The Guidelines state that they "*provide engineers with the means to assess the seismic behaviour of existing buildings…*", and also provides the mechanism for the engineering assessment of Earthquake Prone Buildings. However, the Guidelines do not include specific guidance for the assessment of Reinforced Concrete Masonry (RCM) buildings or components. This has led to uncertainty in the industry and inconsistency in assessment of RCM elements.

The Joint Committee for the Seismic Assessment and Retrofit of existing buildings (JC-SAR), sought to address the gap by producing a new section for the Guidelines that addresses RCM. The Ministry of Business, Innovation and Employment (MBIE) has financially supported the initiative. The chapter is scheduled to be issued as a draft for public consultation around the time of the SESOC conference.

This paper addresses important aspects that have been considered during the development of the RCM section:

- Observation of RCM behaviour in earthquakes
- How to determine probable strength for RCM
- When the concrete chapter of the guidelines is appropriate for assessment of RCM
- Differences in RCM and reinforced concrete (RC) assessment of flexure and shear

Observed Damage to RCM in Earthquakes

Fortunately, the 6.2 M_W 2011 Christchurch Earthquake did not result in any fatalities in RCM buildings (Royal Commission 2012). Nevertheless, earthquakes can impose greater loading than RCM can elastically sustain. The new RCM section provides examples of common damage, noting the common spalling of concrete block face shells (Figure 1).

Figure 1: RCM damage in earthquakes. Note in the right hand image that many cells are unfilled

RCM Probable Strength

RCM should be inspected as part of any assessment. Unfortunately, the most common construction defect is not readily detectable from external visual inspection alone. Unintended grout voids are often reported when intrusive investigations are performed. This issue can have a drastic impact on the performance of an RCM element

Figure 2: Voids found via intrusive inspection in RCM wall. This wall was constructed in 2024 with construction monitoring undertaken by structural engineers. Note the right image shows Ground Penetrating Radar scans of areas of 'low density'. A skilled scanner is needed to interprete these results as not all highlighted areas are voids. Photo credit: Beca Ltd.

The draft guidelines provide two methods to deal with the possibility of unintended grout voids:

- Adopt a lower bound strength of 5.5MPa for the masonry compressive strength f'_m, or
- Undertake intrusive inspections on a representative sample of important RCM elements

The extent of effort for the intrusive inspections should be determined by the significance of the structural element, however at a minimum it involves all of the following:

- Ferro scanning to confirm presence of reinforcement
- Drilling with a ~5mm diameter masonry bit 50-80mm into the block to confirm whether grout is present.
- Using a hammer to knock on the masonry to determine areas of void and area of fill. The knocking method requires experience to interpret whether a knock sounds 'hollow' and may need calibration against known areas.

With the above investigation, the usage of fully filled or partially filled construction can be confirmed, and it further provides adequate confidence to allow higher probable compressive capacity (f'_m) via:

$$f'_{\mathrm{m}} = \left[0.6\alpha f_{\mathrm{cb}} + 0.9(1-\alpha) f_{\mathrm{g}} \gamma_{\mathrm{age}}\right] \gamma_{\mathrm{prob}}$$

where:

α	=	the ratio of the net concrete block area to the gross area of the concrete block. May be taken as 0.45
f_{cb}	=	the characteristic strength of the concrete block
f_{g}	=	the characteristic strength of the grout
γ_{age}	=	the strength gain due to aging beyond 28 days. May be taken as 1.2.
γ_{prob}	=	the ratio of probable strength to characteristic strength for reinforced masonry. May be taken as 1.2.

Historic characteristic strengths for grout and concrete block are provided within the RCM section. This is an adaptation of the strength calculation provided in NZS4230:2004, with modification to also allows benefit to be derived from strength gain with age.

Can't I use the Reinforced Concrete Section?

The Guidelines already include a comprehensive (and recently updated) Section C5 addressing reinforced concrete. There are obvious similarities between RCM and reinforced concrete, and the new RCM section frequently references Section C5 rather than repeat guidance twice.

However, RCM does present unique considerations that are not present in Section C5. Required lap lengths, construction defects and material considerations are detailed in the new RCM section. It is intended that these additions will make assessing RCM faster and provide more consistency across assessments.

RCM Shear and Flexural Probable Capacity

The shear and flexural strength of an RCM element remains similar to that of reinforced concrete. Out-of-plane assessment of partially filled RCM does not have a direct reinforced concrete analogy, so the new guidelines state that yield line assessment should be used in this situation. The assessment of RCM shear strength is based on the design code for RCM structures, NZS4230:2004, with changes to strength reduction factors and adopted masonry strength.

The ultimate compression strain of RCM is limited to 0.003, in recognition of the lower-level steel confinement in comparison to reinforced concrete.

CONCLUSIONS

The JC-SAR initiated draft guidelines for the assessment of RCM structures aim to address the current gap in the seismic assessment guidance for these buildings. This new section will provide specific guidance for RCM buildings, creating greater certainty and consistency in the industry.

The new RCM section will cover; common RCM construction practices, observed RCM behaviour in earthquakes, determining probable strength for RCM, and methods to assess RCM elements for flexure and shear. The guidelines will also provide methods to mitigate investigate or otherwise account for unintended grout voids.

By incorporating these new guidelines, the assessment of RCM buildings will become more consistent and reliable, ultimately improving the safety and resilience of these structures in seismic events.

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Highlighting the impact of a window opening on the seismic vulnerability of a light timber-framed wall

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ABSTRACT

New Zealand (NZ) has a long tradition of building standalone light timber-framed (LTF) wall buildings for housing, typically in accordance with NZS3604. Past earthquakes, locally and internationally, have demonstrated that one- and two-storey LTF houses provide good life-safety performance but suffer high financial losses. Recently there has been a surge in medium-density housing solutions of 3-storeys or more, constructed in a similar manner to one- and two-storey houses but outside the scope of NZS3604. There is concern that more guidance is required for the seismic design of such buildings as they are expected to behave differently to compliant NZS3604 buildings. Indeed, two three-storey LTF houses collapsed in the Canterbury earthquakes. LTF wall design in NZ typically relies on P21 testing, which assesses straight walls, without axial load, without openings and with a specific end restraint. However, walls in three-storey buildings are potentially subject to higher axial load, frequently include openings and return walls, potentially altering their seismic behavior. To address this, seven tests of LTF walls, all possessing perpendicular return walls, were completed at the University of Canterbury to examine the effect of axial load, openings and hold-downs. This paper reports on the quasi-static cyclic force-displacement behavior of two NZ LTF walls. Results suggest that the presence of a window opening halved the drift at which plasterboard would require replacement and reduced the strength and stiffness of the wall by almost 50%. In addition, NZS3604 bracing capacity ratings are 44% and 32% of the maximum peak force achieved in these tests, for the wall without and with the window opening respectively.

RESEARCH MOTIVATION

New Zealand (NZ) has a long tradition of building standalone light timber-framed (LTF) wall buildings for housing. LTF housing in NZ is typically constructed in accordance with *NZS3604: Timber-framed buildings* (Standards New Zealand 2011). NZS3604 specifies proprietary LTF walls, evaluated with the P21 test procedure (Shelton 2010a), which allows building practitioners to self-certify structural elements to resist vertical and lateral load without the need for specific engineering input, for one- and two-storey houses. Buchanan et al. (2011)

Classification: In-Confidence

investigated the performance of housing during the February 2011 Christchurch earthquake and found that single- and two-storey houses, constructed in accordance with NZS3604, performed well from a life-safety perspective. However, although characterised with good life-safety performance, the financial losses associated with residential construction in the 2010-2011 Canterbury earthquake sequence was high. It is estimated that of the total NZ$40 billion in losses, $16 billion was attributed to losses in residential construction, and within 90 days post the September 2010 Darfield earthquake approximately 130,000 residential insurance claims were made (Horspool et al. 2016; King et al. 2014; Wood et al. 2016).

Whilst the life-safety performance of 1- and 2-storey buildings was good, Buchanan et al. (2011) observed soft-storey collapse of two NZ three-storey LTF wall buildings, this is shown below in Figure 1. These buildings are considered outside the scope of NZS3604 and clearly did not perform well from a life-safety perspective. This poor performance is of concern, since there were not many 3-storey timber framed buildings in Christchurch at the time of the 2010-2011 earthquakes and because there has recently been a surge in medium-density housing solutions of 3-storeys or more, constructed in a similar manner to one- and two-storey houses but outside the scope of NZS3604. Indeed, MBIE (2023) report that the percentage of medium density houses in NZ's building stock has more than doubled since the 2010-2011 Canterbury earthquake sequence.

a) b) c) d)

Figure 1: a) Case study 1 before and b) after 22nd of Februrary 2011, c) Case study 2 immediately after the earthquake and d) soft storey collapse due to wind several days later. (Buchanan et al. 2011).

RESEARCH OBJECTIVE

Given that the design of LTF walls in three-storey housing typically relies on P21 testing, which assesses straight walls, without axial load, without openings and with a specific end restraint (Shelton 2010a). It would be desirable to conduct testing outside the scope of the P21 test procedure to quantify these effects. Past studies have shown the failure mechanisms and damage propagation can be different to those observed in past earthquakes (Buchanan et al. 2011; Liew et al. 2002; NAHB Research Center 1994). This study compares the likely seismic performance for two LTF walls, which are a subset of seven unidirectional quasi-static cyclic LTF wall tests recently conducted at the University of Canterbury. The impact of introducing a window opening in the middle of a LTF wall is highlighted. Information on the strength and stiffness is reported and relationships with observed damage and drift demand will be established.

EXPERIMENTAL TESTS OF THE LTF WALLS

General framing, sheathing and fastener details

Both test specimens include walls that 2.4 m tall, 2.4 m long and with 0.6 m long perpendicular return walls at each end. The walls are fixed to a reinforced concrete base slab and include a timber floor diaphragm at their top, as shown in Figure 2, which also shows the general test setup. Two different framing configurations are considered in this study: 1) typical framing without a window opening built in accordance with NZS3604, 2) typical framing with a 1 m wide window opening built in accordance with NZS3604. The framing plan and an image of the final test setup is all shown in Figure 2.

All wall framing was constructed with 140 × 45 mm SG8 timber studs spaced at 600 mm. The external sheathing was sheathed with 7 mm thick plywood, nailed to the framing at 150mm centres. The interior sheathing elements for both specimens consisted of standard plasterboard, 1200 mm wide and 10 mm thick, with screws used to fix the plasterboard to the timber wall elements. The the timber floor diaphragm at the top of the walls was connected to the walls using four nailon plates, in each corner of the roof diaphragm. M10 post-fixed anchors were evenly spaced at 750 mm centers and hold-downs are used at each end of the walls. Configuration 2 contained an additional two hold-downs beside the window opening, and the M10 anchors at 750 mm centers were removed. All surfaces were sanded, stopped and painted with a bottom coat of primer and two coats of white wall/ceiling paint, consistent with common NZ finishing practices. For further details of the framing, sheathing and fastener details, refer to Dawson et al. (2025).

Classification: In-Confidence

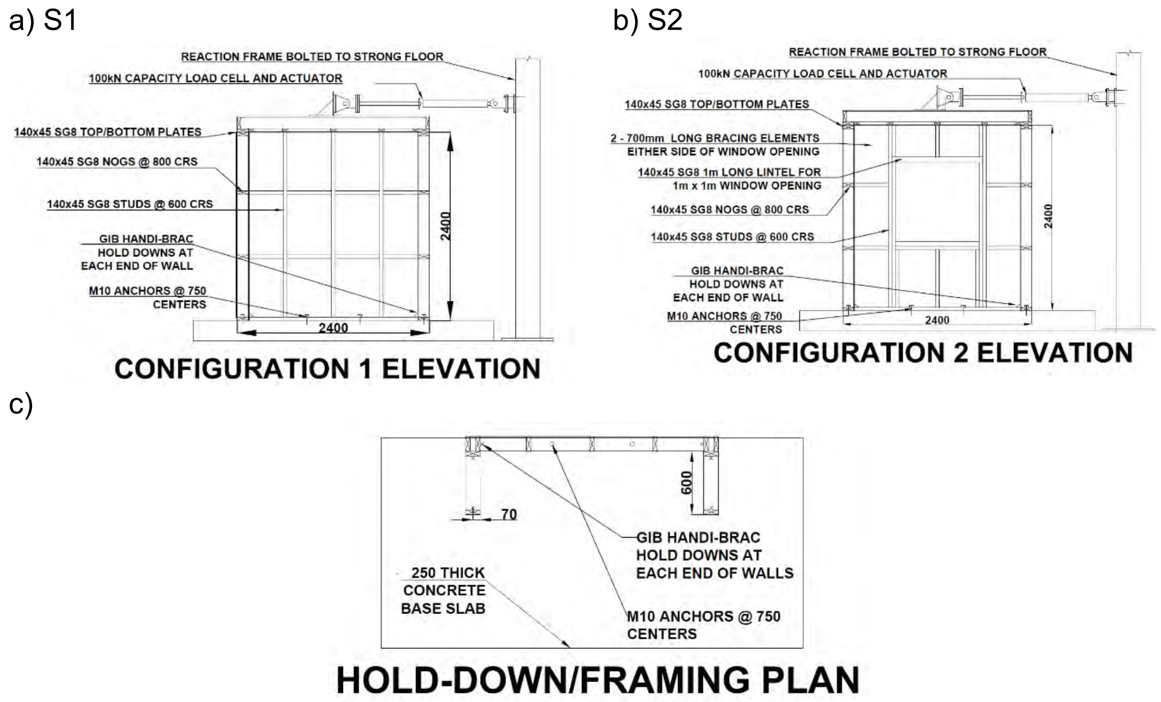


Figure 2: a) Framing configuration 1 elevation for Specimen 1 (S1), b) Framing configuration 2 elevation for S2, and c) Hold-down/framing plan view

Test setup, loading protocol and data acquisition

The test frame, shown in Figure 2, was used to support a 100 kN actuator with a ± 200 mm stroke. This was attached to the framing of the top floor/ceiling diaphragm using screwed connections. The bottom plate of the specimens was fixed to a 250 mm thick reinforced concrete slab. The load applied to the specimen was recorded using a 100 kN load cell. Axial load was applied to the specimen using steel plates which were placed on the plywood diaphragm above the wall. Axial load was calculated in accordance with the engineering basis of NZS3604 by assuming a light roof structure and assuming a 1.6 m tributary width (this was to also account for a soffit), this resulted in 0.8 kN being applied to the top of the walls (Shelton 2010b). In addition, 20 mm spring linear potentiometers were used to record specimen uplifts due to the applied lateral displacement.

The specimens were subjected to a displacement-controlled unidirectional quasi-static cyclic loading protocol, developed as a part of the CUREE-Caltech Woodframe project (Krawinkler et al. 2001), specifically for testing LTF structures.

The CUREE loading protocol for a cyclic test is defined by variations in deformation amplitudes, which uses the reference displacement as the absolute measure of deformation amplitude. It is recommended to conduct an initial monotonic test to determine the reference displacement, which can vary depending on the details of the LTF specimen. However, given the aim of this study is to test a range of different wall specimens, conducting a monotonic test prior to cyclic testing would be impractical. Therefore, a reference displacement of 24 mm (corresponding to 1% storey drift) was selected for all wall tests. Evidence from past experimental testing and numerical modelling of gypsum plasterboard walls suggests this value is reasonable (Francis et al. 2023; Lafontaine et al. 2017) and resulted in testing to peak drifts (for each of three cycles) of 0.05%, 0.075%, 0.10%, 0.2%, 0.3%, 0.4%, 0.7%, 1% with subsequent peaks increasing at 0.5% increments.

Damage State Definitions

Table 1 below shows damage state (DS) definitions of plasterboard partition walls reported by Taghavi and Miranda (2003), which has been used in more recent studies in NZ applications (Liu and Carradine 2023). It is viewed as a good tool as it establishes general relations between different levels of observed physical damage and the required repair actions. Minimal damage was observed in the plywood sheathing (some nails failed at high drift demands); hence it is not included in the table below.

Table 1: Damage state definitions of plasterboard walls (Taghavi and Miranda 2003).

Damage State (DS)	Damage Definition	Potential Repair Actions
0	Cosmetic wrinkling or hairline cracking of paint over tape joints	No repair needed
1	Cracking that can be repaired with tape, plaster and paint	Add more screws, if necessary, tape, plaster, finish and paint
2	Replacement of plasterboard sheathing but not timber framing	Replace damaged plasterboard with new sheets, attach new plasterboard to frames, tape, plaster, finish and paint
3	Total replacement	Demolish plasterboard walls and ceilings, build new walls and ceilings

EXPERIMENTAL RESULTS OF THE LTF WALLS

Figure 4 below shows the force-displacement response of both LTF walls considered in this study. Table 2 shows key numerical results, which may be of interest to structural engineering practitioners. To generate the backbone force-displacement approximations the least squares linear regression method was used between the data points of each primary cycle. The backbone approximation was generated by choosing an initial guess of the yield displacement (i.e. the intersection point between the elastic and post-yield region), which informed the guess of the yield force by using the slope of this line. Then this process was iterated to optimise until the largest R^2 value was found. Note that the average R^2 of both the elastic and post-yield region was considered in this process. If the specimen possessed trilinear backbone behaviour, the intersection point between the post-yield and softening region was taken as the peak force and displacement associated with that peak force. All R^2 values were greater than 0.9. This process was done for data in both the push (positive) and pull (negative) directions and results for both directions are shown in Table 2.

Table 2: Key numerical results from the experimental tests of the LTF walls.

Backbone Model Parameters	**S1**		**S2**	
	Push (+)	**Pull (-)**	**Push (+)**	**Pull (-)**
Yield Displacement (mm) - Δ_y	12.3	11.1	11.6	11.5
Yield Drift (%) - θ_y	0.51	0.46	0.48	0.48
Yield Force (kN) - F_y	27.1	21.5	15.1	16.1
Peak Force (kN) - F_{peak}	41.0	33.4	25.1	24.5
Ultimate/Yield Force Ratio - F_{peak}/F_y	1.51	1.55	1.66	1.52
Displacement at Peak Force - Δ_{peak}	33.8	24.0	72.0	72.0
Drift at Peak Force (%) - θ_{peak}	1.41	1.00	3.00	3.00
Initial Stiffness (kN/mm) - K_i	2.20	1.93	1.30	1.40
Post-Yield Stiffness Multiplier - r_1	0.64	0.92	0.17	0.14
Softening Region Multiplier - r_2	-0.66	-0.41	N/A	N/A

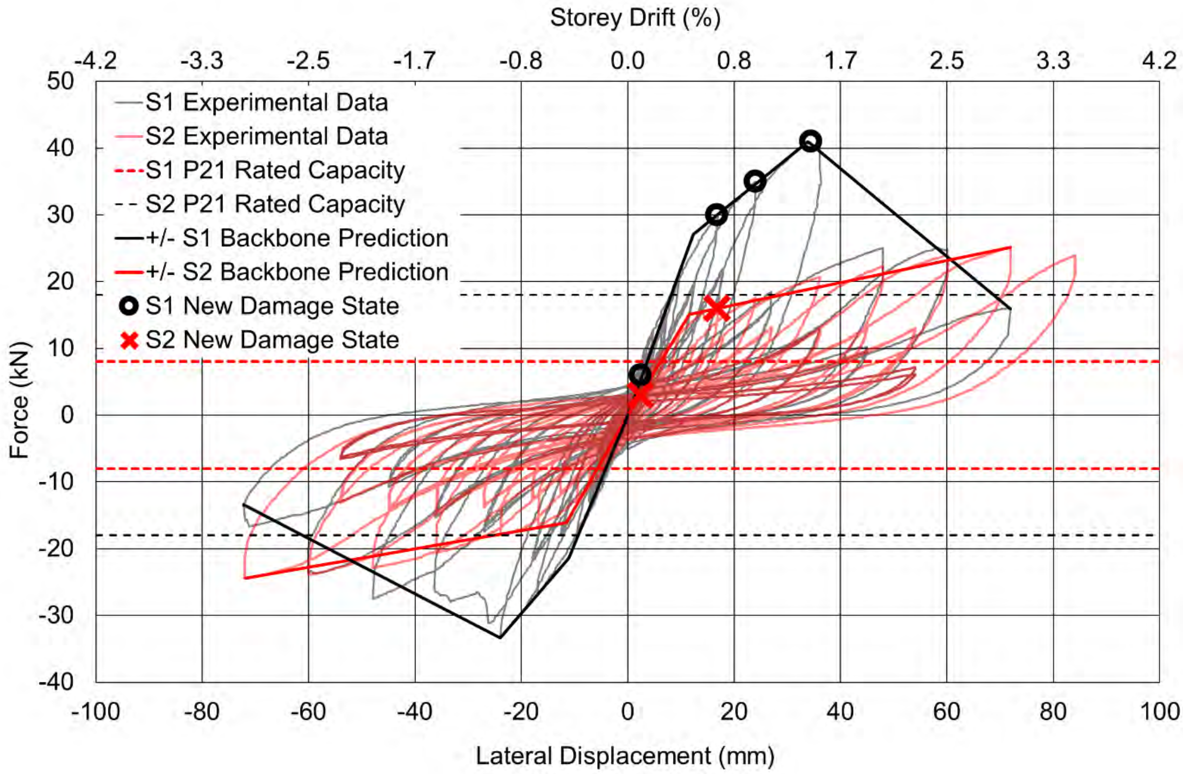


Figure 4: Experimental results for S1 and S2.

Damage Observations and Damage State Definitions

At each new level of drift demand in the CUREE loading protocol damage observations were made by taking photographs and visual inspections. This would be conducted at each new step in the loading protocol once the primary and subsequent trailing cycles were complete, and the specimen was back at its zero-displacement position.

LTF Wall Without a Window Opening (S1)

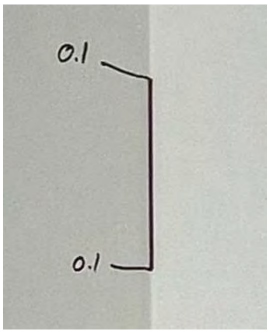
Cosmetic damage began to propagate in the specimen at 0.1% drift, in the form of vertical hairline cracks in the corners of the specimen as shown in Figure 6a. However, these were barely visible and would require careful inspection of walls to be noticed in practice. At 0.7% drift local cracking was observed near the plasterboard screw heads fixed into the bottom plate, indicating stress on the screws, shown in Figure 6b. It is noted that some of this damage may be hidden by skirting boards. At 1% drift significant local cracking/crushing is observed in the bottom corners of the plasterboard, shown in Figure 6c. At 1.5% drift the plasterboard significantly buckles out-of-plane and the load carrying capacity of the LTF wall drops, as

shown in Figure 6d. At 2% drift, minor nail slip is visible in the corners of the plywood panels, followed by some nails withdrawing from the timber studs as the plywood panel slightly buckled out-of-plane at 2.5% drift. At 3% drift some plywood nails fractured. However, such damage to the plywood would often be hidden by external cladding systems. There was no damage to the ceiling plasterboard, return walls or timber framing throughout the test. Figure 6 shows typical damage observed which triggers new damage states throughout the testing for S1. The drifts at particular damage states for S1 were similar to that reported by Liu and Carradine (2023).

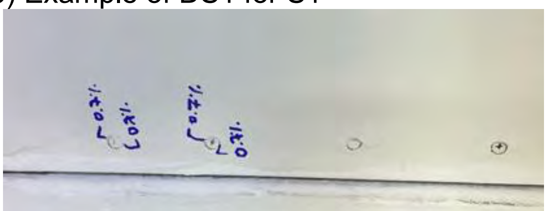
LTF Wall With a Window Opening (S2)

Like the first specimen, cosmetic damage began to propagate in the specimen at 0.1% drift, in the form of vertical hairline cracks in the corners of the specimen. Again, these were barely visible, as shown in Figure 6a. At 0.7% drift, significant damage was observed to the plasterboard, in the form of diagonal cracks occurring around the window opening, this is shown in Figure 6e. Nail slip was visible in the plywood at 1.5% drift, but nails did not withdraw from the plywood until 3% drift. At 3% drift the load carrying capacity appeared to still be increasing, but once reaching 3.5% drift, this appeared to slightly drop the load carrying capacity due to the window glazing fracturing. Again, like the test without the window opening, there was no damage to the ceiling plasterboard, return walls or timber framing. Figure 6 shows typical damage observed which would trigger a new damage state throughout the testing for S2.

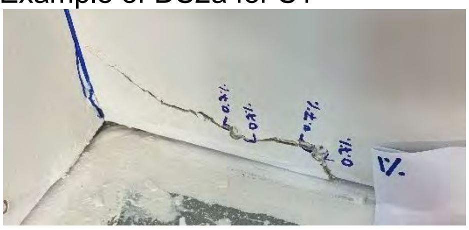
a) Example of DS0 hairline cracking in the corners of S1 and S2.



b) Example of DS1 for S1



c) Example of DS2a for S1



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d) Example of DS2b for S1 e) Example of DS2b for S2

Figure 6: Examples of observed damage for S1 and S2 at different DS'.

DISCUSSION OF RESULTS

Damage State Quantification of LTF Walls

Table 3 below shows the damage state definitions and the quantification of each damage state for both LTF walls considered in this study. Note that even at 3% drift or greater, there was no significant damage to the ceiling or framing and the test was subsequently stopped. To reach DS 3, drift demands well beyond the building code limit of 2.5%, specified in NZS1170.5 (Standards New Zealand 2004) would need to be imposed on the walls.

Table 3: Damage states, likely required repair actions and storey drifts of the LTF walls.

Damage State	**Damage Description**	**Potential Repair Actions**	**Storey Drift % (Top Plate Deflection (mm))**	
			S1	**S2**
0	Cosmetic wrinkling or hairline cracking of paint over tape joints	None	0.1% (2.4 mm)	0.1% (2.4 mm)
1	Cracking that can be repaired with tape, plaster and paint	Add more screws, if necessary, tape, plaster, finish and paint	0.7% (16.8 mm)	-

2a	Plasterboard damage - local crushing or cracking within sheets	Replace damaged areas of plasterboard, tape, plaster, finish and paint	1% (24 mm)	-
2b	Plasterboard detached from framing. Out-of-plane buckling of plasterboard. Load carrying capacity drops.	Replace damaged plasterboard with new sheets, attach new plasterboard to frames, tape, plaster, finish and paint	1.5% (36 mm)	-
2b (For Window Opening)	Large diagonal cracks around the corners of opening	Replace damaged plasterboard with new sheets, attach new plasterboard to frames, tape, plaster, finish and paint	-	0.7% (16.8 mm)
3	Total replacement	Demolish plasterboard walls and ceilings, build new walls and ceilings	> 3% (72 mm)	> 3% (72 mm)

The addition of a window opening has significant implications for the seismic vulnerability of a LTF wall. The drift at which the plasterboard required replacement was approximately halved, along with the strength and stiffness. In practice, when evaluating the strength of these walls the NZS3604 bracing rating would be used, a 2.4 m length would be considered for S1, and 1.4 m would be considered for S2, ignoring the contribution of coupling elements above and below the window. When comparing this capacity used in practise, relative to the maximum peak force observed in this testing, this shows the P21 test rating is 44% and 32% of the peak force for S1 and S2 respectively. Indicating strength can increase significantly when return walls are included, similar to other observations made in multiple studies (Liew et al. 2002; Morris et al. 2018).

Current NZ seismic design provisions are primarily developed to achieve life-safety. However, at a 1/25-year earthquake intensity level it is recommended, according to AS/NZS1170.0 (Standards New Zealand 2002), that the mid-height deflection of LTF plasterboard walls be limited to the wall height divided by 300, when subject to in-plane loading. This is specified with the intention of limiting lining damage and equates to approximately 8 mm lateral displacement (0.33% drift). Based on observations in this testing, there was no significant damage that would require repair action outside of routine maintenance (e.g. painting), in both specimens at this level of drift demand, indicating that it appears to be a reasonable limit. The testing observations also indicate that the current in-plane drift limit of 2.5% for ULS design intensity shaking, appears reasonable from a life-safety perspective but would imply significant repair costs.

CONCLUSIONS

This study compared the likely seismic performance for two LTF walls, one with and one without a window opening. The design of LTF walls in three-storey housing typically relies on P21 testing, which assesses straight walls, without axial load, without openings and with a specific end restraint (Shelton 2010a). It would be desirable to conduct testing outside the scope of the P21 test procedure to quantify these effects as past studies have shown the failure mechanisms can be different to those observed in past earthquakes. Several interesting observations were made between the two LTF wall tests and these are summarized below.

- The P21 rated capacities, as used in practise, were 44% and 32% relative to the peak force resistance observed in these tests for S1 and S2 respectively.
- Directly comparing S1 and S2 LTF wall tests suggests, introducing a window opening decreased the strength and stiffness by 43%.
- Without a window opening, plasterboard buckling-out-of-plane is likely at 1.5% drift. The introduction of a window opening, resulted in significant damage in the form of large diagonal cracks at 0.7% drift. Hence a window opening approximately halves the drift at which the plasterboard requires replacement.
- Based on damage state classifications developed in prior studies, relationships between storey drift and the damage states of two NZ LTF walls, typical of residential construction practise have been suggested.

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Design method for seismic strengthening of diaphragms using FRP ties

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ABSTRACT

Many existing buildings are seismically deficient due to outdated design standards, increased seismic demands, material degradation, or change in use. A recurring vulnerability is inadequate diaphragm action in concrete floors, often caused by poor detailing, corrosion, or insufficient reinforcement. In New Zealand, engineers frequently retrofit diaphragms using a strut-and-tie design approach, in which strips of fibre Reinforced Polymers (FRPs) are used to create the tension ties.. However, available design guidance is limited and design is typically based on provisions not specifically intended for this application. . This paper introduces a new design methodology for FRP tension ties in diaphragms, developed following an extensive research programme. The proposed method addresses the limitations of existing design approaches and aims to support more reliable and standardized seismic retrofitting practices.

BACKGROUND

Existing buildings are often deemed to be seismic deficient, either because they were not designed to modern standards, because seismic hazards are increasing in many jurisdictions, because of change of use, or degradation (e.g. corrosion), or a combination of these factors. A common issue is that concrete floors in buildings cannot fulfil their seismic roles as diaphragms, tying the building together and transferring the inertial forces to the lateral load resisting system. Common reasons are insufficient steel reinforcement, degradation, poor detailing, or a combination of all of them. Therefore, additional tension capacity is needed on the diaphragms, and its location is typically determined using strut and tie analysis or grillage models in the New Zealand environment.

Fibre Reinforced Polymers (FRPs) are a composite material made of a woven fabric of fibres (typically carbon) and a matrix (typically epoxy resin). In the civil engineering industry, the most commonly used application is called wet lay-up, where unidirectional fabrics are saturated onsite with the resin before being glued or bonded to the external surface of the structure (del Rey Castillo et al 2019a, Amran et al 2018). This method is called Externally Bonded FRP, or EBFRP, and it is prevalent in seismic environments and especially in New Zealand. The use of FRP to provide tension capacity to diaphragms is gaining traction among practicing

engineers because of its versatility, light weight, strength to thickness ratio, and ease of installation, which makes it a cost-effective solution. The main problem when designing FRP ties for seismic strengthening of diaphragms is that there is not an established design methodology, and the engineers rely on design provisions not originally intended for diaphragms and that were developed for much shorter and thinner FRP ties. This paper presents a proposed design method for FRP ties that the authors in this team have been developed following an extensive research programme.

TIE DESIGN

The most important variable to consider when designing FRP is the design strain. FRP is elastic until failure, which results in a brittle failure when this fibre rupture failure mode governs. Thus, the design strain should be significantly lower than the rupture strain to prevent this brittle failure mode. On the other hand, the load path from the FRP to the concrete relies on the bond of the epoxy resin to the concrete substrate, with concrete strength typically governing the bond strength.. The debonding of the FRP tie from the concrete substrate occurs at some threshold strain, which is a function of several factors and which linearly translates to a force depending on the cross-sectional area of the FRP and its modulus of elasticity, as long as a minimum development length, calculated per ACI 440.2-23 Eq. 11.4.1.2c and reproduced below in Equation 1, is provided. Bonded lengths above the minimum provided by Equation 1 do not result in an increase in force at debonding but may provide deformation capacity as the debond line progresses from the loaded end (the crack) to the anchored end. For further details, the reader is referred to Zhang et al (2024a).

Equation 1 $$L_e = \frac{23{,}300}{(Nt_f E_f)^{0.58}}$$ ACI 440.2R-23 (Eq 11.4.1.2c)

The total tension capacity of the tie, reduced by strength reduction factor ϕ, as required, must be equal or larger than the demand, as represented by Equation 2. A strength reduction factor of ϕ =1 is typically used when an overstrength demand has been calculated, otherwise a strength reduction factor of ϕ =0.75 should be used as per NZS 3101 (2006) 2.3.2.2h.

Equation 2 $$\phi N \geq N_u$$

The capacity is the sum of the capacity provided by the ductile reinforcing steel in the diaphragm and the capacity provided by the FRP, which multiplied by a reliability factor ψ_f.=0.85 (Equation 3).

Equation 3 $$\phi N = \phi(N_s + \psi_f N_f)$$

The contribution of the FRP is determined by the design strain (ε_f) multiplied by the modulus of elasticity in the FRP and the cross-sectional area of the FRP tie. As the FRP is an elastic material, the effective stress in the FRP is determined by the multiplication of the modulus of elasticity E and the design strain ε_f. The cross-sectional area of the FRP is given in Equation 4by the width w_f, multiplied by the number of layers N and thickness of one layer (t_f). The designer should ensure that the mechanical properties of laminate, i.e. resin saturated and cured, FRP are used.

Equation 4 $$N_f = E_f \varepsilon_{fe} w_f N t_f$$

The key parameter is the design strain (ε_d) as mentioned above. Published research has shown that the codified / standardised models (e.g. ACI 440.2 and fib Bulletin 90) to calculate the strain at which thick FRP ties (cured/laminate thickness >2 mm) debond are not suitable (del Rey Castillo et al. 2022, Zhang et al. 2024a). The design strain for the ties is based on the equations 5 and 6, which is based on an analysis of over 1800 datapoints, including thick and long ties, and is currently under consideration for inclusion in ACI440 as a replacement of the current provisions for pure axial strengthening in Section 12.4 (Zhang et al. 2024a). Equation 5 represents the the probable value of strain at debonding, while equation 6 results in a 95 percentile or design value. The equation is very similar to the debonding strain equation proposed by Chen and Teng (2001), widely accepted to reflect the mechanics of FRP debonding. The power relation between strain and tie stiffness ($NE_f t_f$) results in diminishing returns for increased tie thickness, to the point that perhaps it is not possible to design the FRP ties at the debonding strain in certain applications. This scenario should be avoided, but if unavoidable the designer should consider the deformation implications of using a strain where the FRP is likely to be debonded at the design level.

Equation 5 (predictive, probable value) $$\varepsilon_{fe} = 0.45\sqrt{\frac{\sqrt{f'_c}}{NE_f t_f}}$$ Zhang et al (2024a)

Equation 6 (design, 95 percentile equation) $$\varepsilon_{fe} = 0.34\sqrt{\frac{\sqrt{f'_c}}{NE_f t_f}} \leq 0.004 \leq 0.75\varepsilon_{fu}$$ Zhang et al (2024a)

The method detailed above allows the designer to calculate the force that the FRP tie can transfer at the onset of debonding, but not the deformation that the tie will exhibit or how this deformation influences the rest of the diaphragm and the building. FRP is a linear elastic material, so the deformation between two given points (anchorage points) is as simply a product of the strain and the distance between the two points. For example, a strain of 0.002 over a distance of 6 meters results in a deformation of 12 mm. Using the debonding strain to design the FRP permits the FRP ties to have some measure of adequate area, and the associated stiffness, to mitigate cracking in the diaphragm.

ANCHORAGE DESIGN

The cracking in the concrete may be minimal, even at the design demand, permitting the FRP tension ties to function without debonding. For any significant seismic hazard, the bond between the FRP tie and the concrete may be an unreliable design parameter for seismic design, not only due to the potential of the actual demand to exceed the estimated demand, but also due to unforeseen deformation demands on the FRP that would require very detailed non-linear modelling. Therefore, anchorage of the FRP is crucial to provide the added redundancy required in a seismic environment. The recommended demand to design the anchors (N_a) shall be at least 1.2 times the demand on the tie (N_f) as per equation 7 below. Due to the spread between design and probable debonding capacity (0.45 is ~32% higher than 0.34), the design capacity of the anchors must also be at least 1.2 times the probable debond capacity of the tie.

Equation 7

$$N_a \geq 1.2\phi(N - N_s)$$

$$N_a \geq 1.2\phi\psi_f N_f$$

FRP anchors have been heavily investigated in recent years to provide this load path. Various failure modes can affect FRP anchors and design equations have been developed to calculate the failure load for each failure mode (del Rey Castillo et al. 2019b; del Rey Castillo et al. 2019c; del Rey Castillo et al. 2019d). The equations have been summarised below for convenience, but reading the original sources is recommended for a fully comprehension of the details. The designer should note that the fibre rupture equations are based on the use of the probable properties of the FRP, rather than the design properties. These equations were then transformed into a prescriptive table and incorporated into ACI 440.2-23 for anchoring U-wraps for shear strengthening of beams and may be used for design of anchors. Project specific testing could also be performed to design the anchorage.

Equation 8 (Fibre rupture of straight anchors)

$$N_{fr}^{95\%} = 3.1E_a\varepsilon_f 10^{-3}A_{dowel}^{0.62}\left(\frac{90-\alpha}{90}\right)$$

del Rey Castillo et al (2019d)

Equation 9 (Fibre rupture of 90 Deg bent anchors)

$$N_{fr}^{95\%} = 2.2E_a\varepsilon_f 10^{-3}A_{dowel}^{0.62}\left(\frac{90-\alpha}{90}\right)$$

del Rey Castillo et al (2019d)

Equation 10 (concrete cone failure)

$$N_{cc}^{95\%} = 9.68h_{ef}^{1.5}\sqrt{f_c'}$$

Kim & Smith 2009

Equation 11 (combined concrete failure)	$N_{cb}^{95\%} = 4.62\pi d_0 h_{ef}\sqrt{f_c'},\ f_c' < 20MPa$ $N_{cb}^{95\%} = 9.07\pi d_0 h_{ef}\sqrt{f_c'},\ f_c' \geq 20MPa$	Kim & Smith 2009
Equation 12 (effect of insertion angle on concrete failure)	$\kappa_\beta = 2.34\left(\frac{\beta}{2\pi}\right) - 0.33$	Kim & Smith 2012
Equation 13 (fan-sheet debonding failure)	$N_{sd} = 5A_{fan}$	Kanitkar et al 2016

Arguably, proper detailing of the FRP anchors is even more important than the design capacities. Yet unpublished research has shown that anchors with an embedment depth of 100 mm or more typically result in fibre rupture failure mode up to 32 mm in diameter, but shallower anchors may have various failure modes depending on concrete properties and dowel dimensions (Zhang et al 2024b). If a lot of cracking and damage is expected in a particular location, deeper anchor embedment of 150mm to 200mm is recommended. Another observation is that are close together or too close to the edge of the concrete element may break the concrete, so these failures modes should be checked using well-established methods such as Chapter 17 of ACI 318. Recent research has also shed light onto when anchors placed in series (i.e. in a line parallel to the fibres of the fabric) share the load and thus act together as a group of anchors (Zhang et al 2023 a, b). Anchors placed up to 610 mm apart (2 feet) (orange and green curves in Figure 1) were observed to share the load and behave as a group of anchors, while anchors placed at 915 mm (3 feet) did not share the load and failed progressively in a zipper type manner (purple line in Figure 1). Proper installation is also crucial for anchors (del Rey Castillo et al 2021). Aspects that have not been investigated include cyclic loading of anchors, anchors installed in cracks and anchors subjected to bidirectional loading.

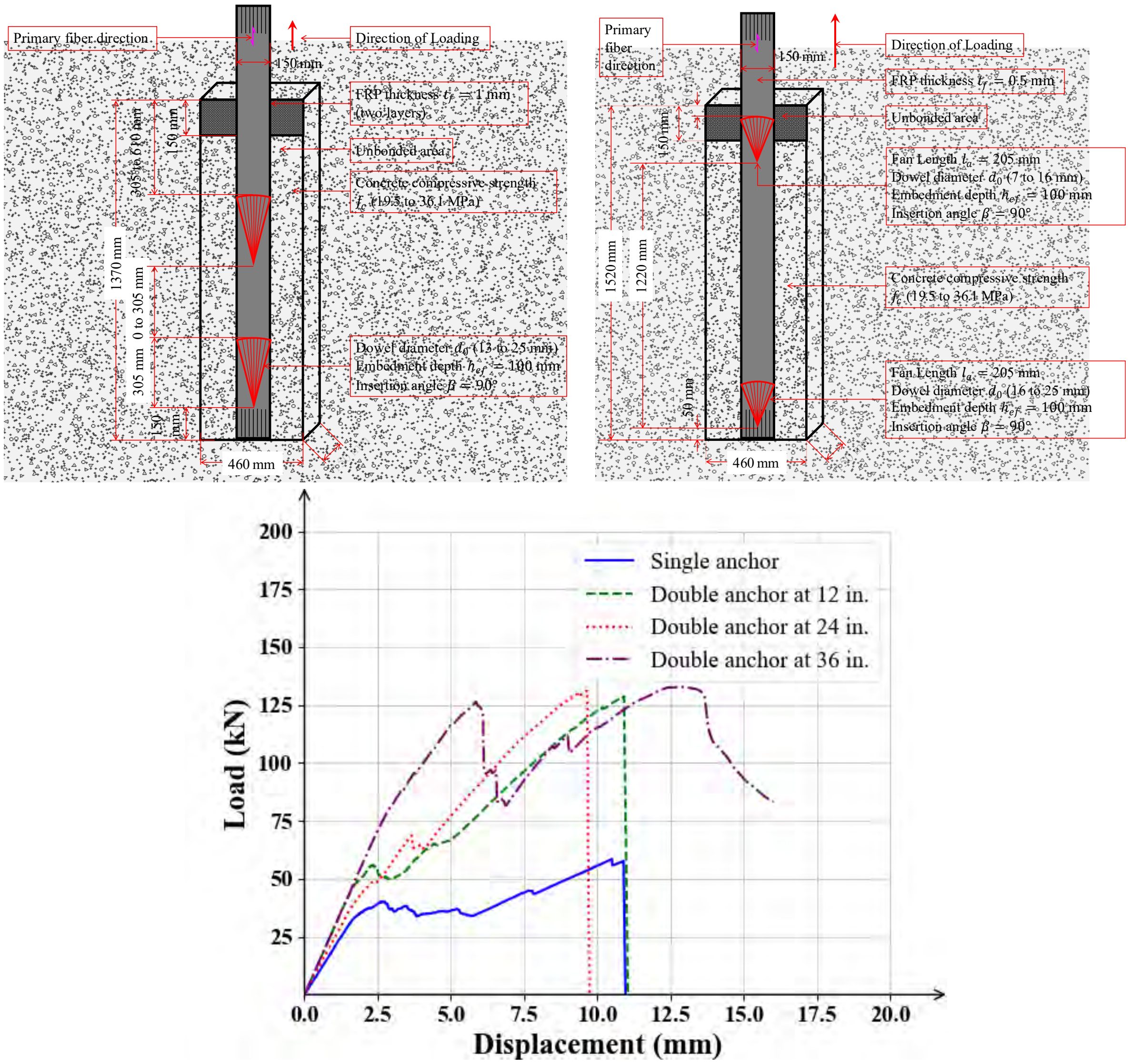

Figure 1 Comparison between single anchor and doble anchor at various distances (del Rey Castillo et al 2024)

ADDITIONAL CONSIDERATIONS

A critical element of a strut and tie model that receives comparatively little attention during design is the nodes. The nodes need to be sufficiently reinforced to transfer the forces from the strut to the tie and vice versa, i.e. need to be sturdy enough for the strut to push against and stiff enough for the tie to pull from. Extensive reinforcing is typically placed in these locations in reinforced concrete design, so the designer should consider these issues.

A tie is not just a force between two points, it must also be able to resist the deformations of the floor while maintaining stiffness. The FRP can sustain the rotational deformations between the floor and the beam, as demonstrated with ample experimental work that is starting to be published (Salimian et al 2024), but the designer should consider what other deformations the tie may be subjected to. Another key aspect is the lateral drift that the building is expected to sustain during the seismic event, and whether that lateral drift is compatible with the expected deformation from the tie design. Figure 2 shows the results from three tests where the response of the FRP tie is analogous to that of a single tie, as shown in Figure 1 (blue line).

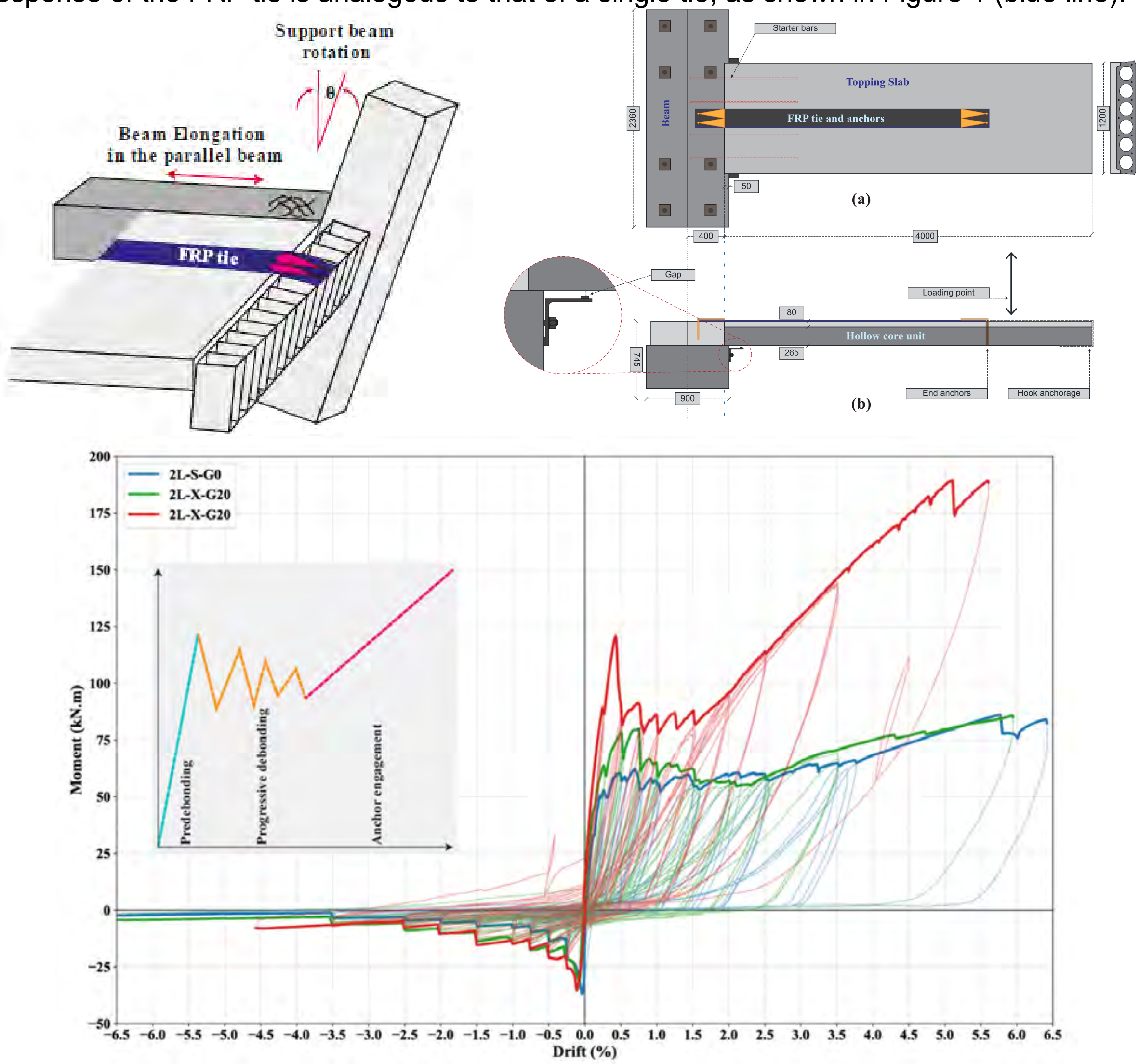

Figure 21 Plot of imposed moment and drift assciated with rotation at the support beam (Salimian et al 2024)

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Strengthening CPEng standards, trust and public safety

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ABSTRACT

The CPEng Rules have been updated and in the process of coming into force. We will outline these changes and give an overview of the implementation of classes.

Strengthening CPEng

The Registration Authority is committed to strengthening the chartered professional engineers (CPEng) system to ensure it remains fit-for-purpose and continues to uphold public safety and professional standards. Following consultation in late 2024, we have begun the process to implement new rules, building guidelines for how classes will work, improve the continued registration process, and strengthen the complaints and disciplinary system.

Strengthening CPEng using regulatory principles

Right-touch regulation is a regulatory approach that aims to use the minimum necessary level of intervention to achieve desired outcomes, while prioritizing public safety and minimizing costs. It's based on a thorough evaluation of risk and focuses on proportionality, consistency, and agility in regulatory decision-making.

We apply these principles to CPEng regulation. This will enable the Registration Authority to regulate efficiently and effectively.

New CPEng Rules

An overview of the rules changes are as follows;

- Annual declarations will be required from September 2025.
- Changes to the complaints process will be implemented immediately after the new Rules are published, in the second half of 2025.
- Updates to the continued registration assessment process will take effect in early 2026.

- Consultation on new classes will begin in 2026, with the first classes expected to be introduced from mid-2026.

Classes

The first classes to be introduced are unknown at this time. The CPEng Board will provide guidance on this in late 2025 and there will be consultation with relevant technical groups, registrants and other stakeholders (such as Councils). Based on consultation feedback, the priority areas for new classes include;

- fire,
- structural, and
- geotechnical engineering.

When implementing classes the Registration Authority will need to consult on the following criteria;

- describe the class and the practice area to which it relates; and
- specify the minimum standards (competence criteria) for registration in the class; and
- specify the minimum standards for continued registration in the class; and
- specify any conditions, and compliance responsibilities that apply to the class.

Other classes may be introduced where a clear public safety or regulatory need is identified. Some practice areas are currently regulated by legislation other than the CPEng Act due to safety requirements. These practice areas may also be considered for classes, such as;

- Design Verifier (Passenger Ropeways)
- Design Verifier (Cranes)
- Design Verifier (Pressure Equipment)
- Recognised Engineer (Potential Impact Classification)
- Recognised Engineer (Dam Safety Assurance Programme)

We will work with stakeholders when consulting on classes. This will include other regulators who rely on the CPEng mark of quality such as building consent authorities. We would like to deliver a regulatory system that provides all of the information regulators need to manage the risks they have in determining whether an engineer is competent. For example we would like to avoid the need for building consent authorities to have duplicate registers.

Reassessments

Previously, all CPEng registrants had to provide work samples and undergo an interactive reassessment every six years. Moving forward, requirements for work samples and interactives will be at the discretion of the Registration Authority. Continued registration assessments will be risk-based (probably using practice areas such as classes), meaning engineers in higher-risk areas may require more rigorous checks, while others will have a less burdensome process.

The reason for this change is that the reassessment process is not efficient in identifying risks and ensuring safe practice in its current form. An ineffective reassessment process in a professional regulatory system can lead to several negative consequences, including a decline in regulatory effectiveness, increased risks of harm to the public, and a lack of adaptability to changing circumstances. Poor reassessment can also erode public trust in the regulatory framework.

The Registration Authority will issue policy and guidance to support this process of transitioning to applying reassessments where they are required and where outcomes improve safety and competence.

Annual Declarations

Annual declarations will reinforce commitment to the Code of Ethical Conduct and ensure that all registrants remain fit and proper to practice, including confirming they have no health or legal issues that would impact their ability to practice safely. This aligns with best practices in other professional regulatory bodies.

Annual declarations are a new way of assessing and addressing risk. Annual declarations will be required for all CPEng. Currently risks are only addressed through complaints and reassessment, declarations will improve our ability to regulate using right touch regulation.

Complaints and Disciplinary process

Public confidence in the complaints process has been eroded. It is inefficient and has not delivered outcomes that provide confidence in the CPEng mark of quality. Key changes include:

- Allowing the Registration Authority to dismiss frivolous or vexatious complaints.
- Granting the investigating committee more powers, including requiring an engineer to undertake competence assessments or professional development.
- Allowing complaints to be referred to the Registrar so that the engineer’s competency can be reviewed.

- Improving efficiency by allowing complaints to proceed based on merit rather than requiring complainants to push forward cases.

A well-designed complaints process is crucial for professional regulation as it enhances public trust, facilitates learning and improvement, and ensures accountability. By providing a clear and accessible pathway for individuals to voice concerns, we can address issues promptly, protect the public, and maintain the integrity of the profession.

Conclusion

Good regulation is designed to achieve specific objectives effectively, consistently, and fairly, while minimizing negative impacts and promoting economic efficiency. It involves a proactive and collaborative approach, ensuring that regulatory systems are fit for purpose and adapt to changing circumstances.

The new CPEng rules are published on the Registration Authority website. We believe they are fit for purpose today, we will continue to review and adapt to changing circumstances and ensure the regulations are protecting the profession, maintaining minimum practice standards and protecting the public.

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Construction Facts of The Kingdom Tower: The World’s Tallest Building

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Previously: On-Site Senior Structural Engineer at Kingdom Tower, Jeddah (2014-2018)

Abstract

The Kingdom Tower, also known as Jeddah Tower, is poised to become the world's tallest building, surpassing the Burj Khalifa. This paper explores some of the structural facts and construction challenges of this monumental project.

In this project I acted as the on-site consultant from 2014-2018, and my role was to ensure construction quality, adherence to specifications, and compliance with project-specific requirements. During this period, I Acquired extensive expertise in the construction methodologies of super tall buildings.

The construction challenges included pumping concrete to extreme heights, climbing formwork systems, high tower cranes and hoists. Additionally, the project faces issues related to health monitoring, precise GNSS survey, casting mass concrete in very hot climate, accounting for concrete shortening/creep and managing labour flow and vertical transportation.

By addressing these obstacles with innovative solutions and meticulous planning, the Kingdom Tower sets a new benchmark in architectural and engineering achievements. This paper provides a brief overview of the project's insights into the complexities of building the world's tallest structure.

Structural Facts

The Kingdom Tower, is a supertall skyscraper designed to reach a height of at least 1,008 meters (3,307 feet), making it the tallest building in the world upon completion. Engineered by Thornton Tomasetti and designed by Adrian Smith, the tower features a distinctive Y-shaped footprint that enhances structural stability and maximizes views of the Red Sea. Its tapering form is specifically crafted to reduce wind loads, a crucial consideration for buildings of such extreme height. Constructed using a combination of reinforced concrete and steel, the tower is clad in an all-glass façade that provides a sleek, modern appearance while allowing abundant natural light. The foundation comprises a deep pile system topped with a thick raft to support the immense weight of the structure. To ensure efficient vertical transportation across its more than 160 floors, the tower will be equipped with 59 elevators, including advanced double-deck models.

Tower Structural System

Core and Wings: The structural system comprises a central triangular reinforced concrete core augmented by three wing assemblies. These wings extend approximately 36 meters from the core at the base of the tower.

Figure 1: Tower Structural System

Wind Resistance: The sloping ends and changing width of the wings prevent the organization of wind vortices along the height of the tower, reducing aerodynamic wind forces.

Concrete and Rebar Strength: The design concrete compressive strength for tower walls up to level 99 is 85 MPa, decreasing to 75 MPa from level 100 to 167, and 65 MPa up to level 240. Reinforcing bars are grade 75 steel with a yield strength of 520 MPa for walls and grade 60 with a yield strength of 420 MPa for slabs and beams.

Steel Pinnacle: The concrete walls extend up to 962 meters, with the pinnacle structure comprising steel framing extending beyond the concrete walls by 48 meters.

Steel Sky Terrace: Located at Level 157 (+630 meters), the sky terrace is a cantilever viewing platform with a maximum hanging distance of approximately 33 meters.

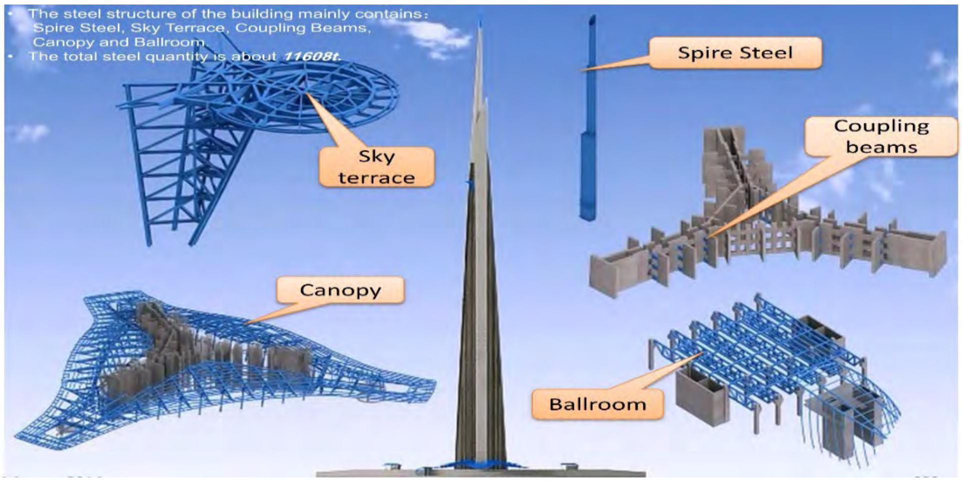

Figure 2: Steel Structures Used in the Tower

Tower Cranes

The construction of the Kingdom Tower requires the use of some of the world's largest and most advanced tower cranes. "Liebherr" and "WolffKran" have supplied custom-built 6 climbing cranes that can ascend with the building as it rises[1]. These cranes are designed to withstand harsh weather conditions and strong winds at high altitudes. Because of the sloping feature of the tower (reduced footprint over height) each tower crane will stop at certain level but the main one will continue up to level 245 (982 m). The same tower cranes used for the construction of the Kingdom Tower will also play a crucial role in their own dismantling. As the building reaches its final height, these cranes will be systematically disassembled in stages. The last standing crane will lift a smaller derrick crane to the sky terrace level. This derrick crane will then be used to carefully dismantle the remaining tower crane.

Structural scope was in designing the structural braces to the tower at higher levels that aren't covered by local standards, so we have relied on some outputs from the wind tunnel tests. Moreover, we have also designed the transfer system that the tower cranes utilized to ascend upward.

Same has been done for the Tower hoists that are used to lift workers and materials. 8 hoists are used in this tower (some of them are double deck) as per the vertical transportation study recommendations.

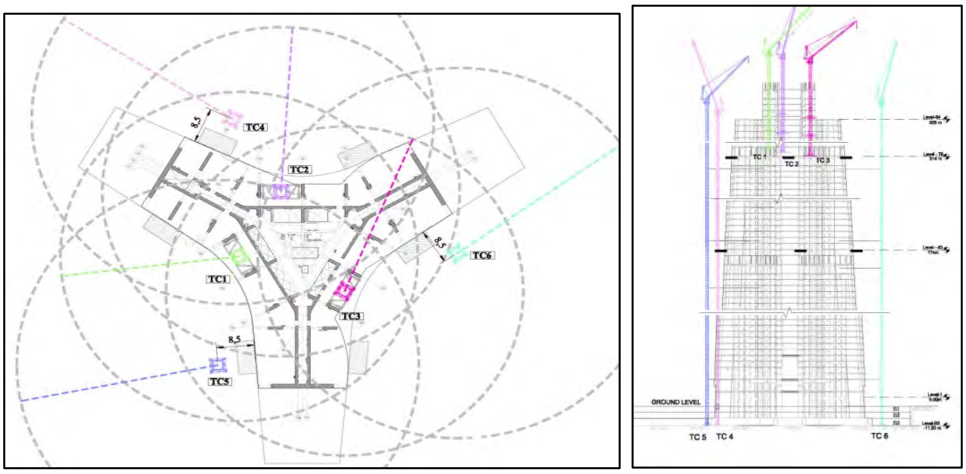

Figure 3: Tower Cranes Used

Pumping Concrete

Pumping concrete to the extreme heights required for the Kingdom Tower in Jeddah presents several structural challenges. One of the primary issues is maintaining the quality and consistency of the concrete mix as it is pumped to heights exceeding 500 meters. High-strength, self-consolidating concrete is used to ensure the structural integrity of the tower, but the pressure required to pump the concrete to such heights can cause segregation and loss of workability. This necessitates the use of advanced pumping equipment and techniques to maintain a consistent flow and prevent blockages. Additionally, the concrete must be designed to withstand the high pressures exerted during pumping, which can lead to micro-cracking and reduced durability if not properly managed specially hen casting in hot climate like Saudi Arabia. To mitigate all these risks about concrete mixes, thousands of mixes have been tested in the below shown pipeline which extends for 1000 m. Different mixes are pumped at one end, then tested for different properties when received at the other end.

For floors above 500m high we will use double pumping. First pump will push concrete to this level, then another pump located at this level will push the concrete for the remaining height as shown in the below picture.

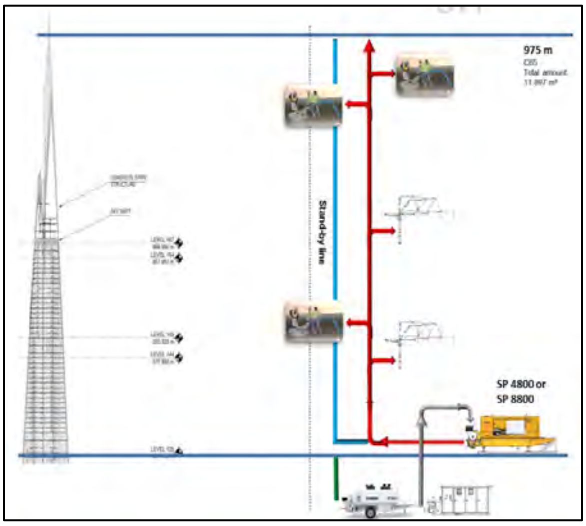

Figure 4: Pumping Concrete

Formwork

The use of Doka's SKE100 climbing formwork in the construction of the Kingdom Tower in Jeddah is essential for efficiently and safely erecting the super-tall structure. This advanced formwork system allows for the continuous and systematic construction of the tower's vertical walls, by climbing with the building as it rises. The SKE100 system is fully hydraulic and crane-independent, which is crucial for a project of this scale, as it minimizes reliance on cranes and enhances construction speed and safety. With this design variant, work can proceed simultaneously on several levels. Rising working platforms of this system allow the forming and rebar operations to be 'de-linked', so as to shorten the cycle times

One of the primary structural issues associated with using the SKE100 climbing formwork is ensuring the alignment and stability of the formwork system at extreme heights. Any misalignment can lead to deviations in the verticality of the walls, which can compromise the structural integrity of the tower. The SKE100 system addresses this challenge by providing a stable and adjustable platform that can be securely anchored to the building, ensuring consistent and accurate construction. Additionally, the formwork must withstand the high pressures exerted by the concrete during pouring, which requires robust design and precise engineering.

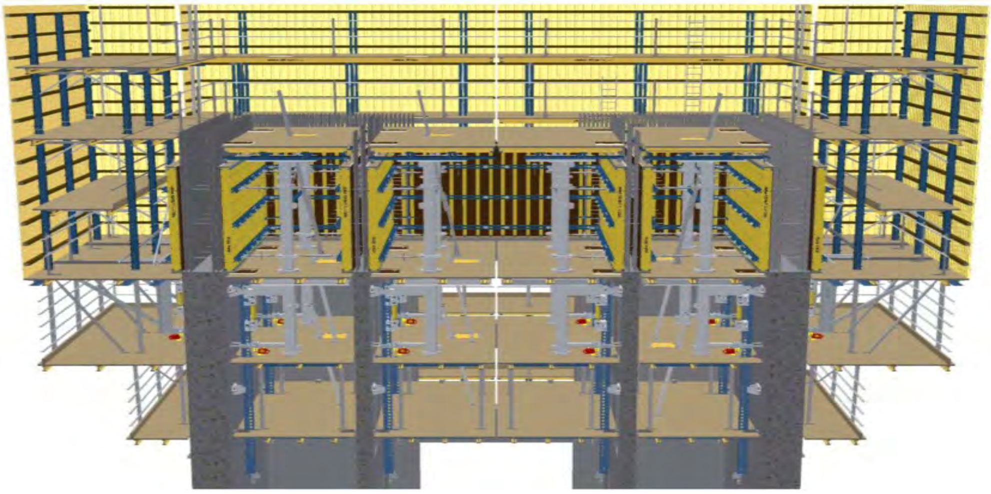

Figure 5: DOKA SKE100 Self-Climbing Formwork

Precise Survey

Kingdom Tower is designed to be the highest structure on earth ever built, with a height exceeding 1 kilometre. Survey works play a major role in achieving this goal, which has never been reached before in the history before, within acceptable height deviations. Horizontal and vertical deformations occur during construction due to different internal and external effects such as elastic, creep and shortening strains, wind load, sand dunes, crane loads and sun radiation effect.

The Core Wall Survey Control System was proposed using Leica Geosystems technology to control the construction of Kingdom Tower's core wall works.

The proposed Survey Control System consists of (but is not limited to) GNSS (Global Navigation Satellite System) reference stations. Leica Geosystems' Core Wall Control Survey System (CWCS) delivers precise and reliable coordinates on demand that are not influenced by building movements.

The information from the inclination sensors is logged at the survey office and the exact amount in Δx and Δy that the building is offset from its vertical position is applied as corrections to the coordinates of the Active Control Points.

The real advantage is that the surveyor can continue to set control – even when the building has moved "off centre" – confident that he will construct a straight concrete structure. The analysis isolates factors such as wind load, crane loads, and raft slab deformation and also relates movement to the construction sequence.

Figure 6: LEICA GNSS System for precise Survey

Casting Mass Concrete in Hot Climate

Pouring mass concrete for the Kingdom Tower in Jeddah involves several meticulous procedures to ensure structural integrity and durability, especially given the extreme height and climatic conditions. To mitigate the effects of high temperatures, concrete casting is often performed at night when temperatures are cooler. The mixing water is chilled, and ice is added to the mix to further reduce the temperature. During casting, a mist water network is used to maintain a cool environment and prevent rapid evaporation. The temperature of the concrete in arriving trucks is continuously monitored and kept below 24°C to ensure consistency. Additionally, thermosets are embedded within the mass concrete elements to monitor the internal temperature. This ensures that the temperature differential between the core and the surface does not exceed 20°C, preventing thermal cracking and ensuring the concrete cures properly. These measures are critical for maintaining the quality and performance of the concrete, which is essential for the stability and longevity of the Kingdom Tower.

Concrete Mix

The Kingdom Tower in Jeddah utilizes ultra-high-strength, self-consolidating concrete (SCC) with a compressive strength of 85 MPa for its shear walls. This concrete mix incorporates a variety of additives, including fly ash and silica fume, to enhance its performance. Admixtures such as superplasticizers and air retainers are also used to improve workability and durability. The mix design includes carefully selected strong, round aggregates and maintains a minimal water-to-cement ratio to achieve the desired strength. The SCC is designed to flow easily into the highly congested reinforcement without the need for mechanical vibration, ensuring complete filling of the formwork and eliminating voids. Additionally, the concrete must be pumpable to reach the extreme heights of the tower. To prevent thermal cracking, the mix is engineered to exert controlled heat during curing to avoid shrinkage cracks

Labour Flow and Vertical Transportation and Hoists

Although it is not structurally related, I found it worthy to mention how mobilization for labors and materials in super tall buildings are handled. Morning shifts had around 1500 labor and night shift around 1000 labor. Managing their flow and vertical transportation in a building of this height is a significant challenge. In addition to the hoists the contractor (at a certain stage of the project after reaching level 65) also used the Tower advanced double deck elevator systems (KONE) capable of traveling at speeds over 10 meters per second. Our structural input was designing hoists supports and protruding loading platforms for storing materials.

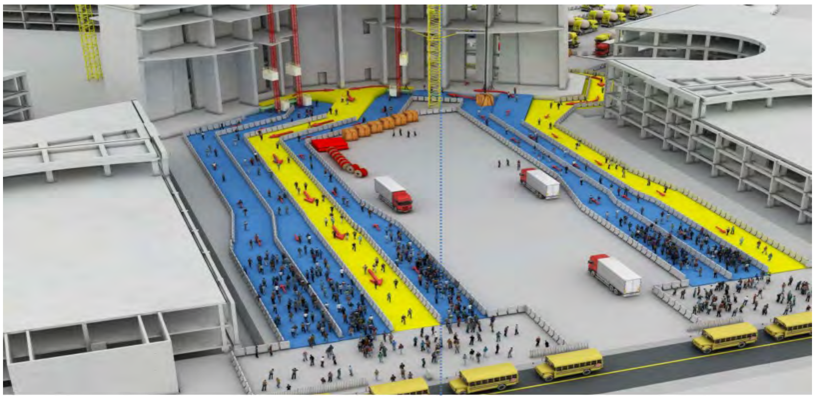

Figure 7: Vertical Transportation in the Tower

Conclusion

The construction of the Kingdom Tower in Jeddah is a monumental undertaking that has faced numerous challenges. Overcoming these obstacles requires innovative solutions, meticulous planning, and effective coordination. The successful completion of the tower will not only set a new benchmark in architectural and engineering achievements but also inspire future projects in the realm of super-tall buildings.

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Low carbon design: Is French flair at play and where are opportunities for New Zealand to up the game

A.R. Guennoc & J.A. Ortiz

ABSTRACT

As awareness on the climate change crisis rises internationally and as the reduction of carbon emissions is identified as a necessary path to a sustainable future, the construction sector, being a significant contributor to carbon emissions, is expected to play a pivotal role in leading change.

In January 2022, the Environmental Regulation RE2020 came into force in France, making it one of the first countries to adopt legislation with regulatory thresholds on whole-of-life embodied carbon for buildings. Following an overview of the French journey to RE2020 and an outline of the regulation's framework, thoughts are shared on opportunities for New Zealand structural engineers to accelerate change on low carbon design considering the country's cultural and organisational specificities.

INTRODUCTION

In response to the global climate crisis, many countries – including France and New Zealand – have committed to net-zero carbon emissions by 2050, with the construction industry playing a key role in that transition.

While climate change affects all nations, the shared goal of carbon reduction creates opportunities for global collaboration and mutual inspiration. However, effective low-carbon strategies must be tailored to local cultural and industry contexts.

This paper explores France's pioneering approach to low-carbon design - one of the first to legislate quantified carbon targets, though not widely publicised internationally - aiming to

identifying opportunities for New Zealand structural engineers to embrace and influence global low-carbon design practices.

RE2020: HOW FRANCE TRIGGERED CHANGE THROUGH EXTENSIVE DATA COLLECTION, CONSULTATION AND REGULATION PROGRESSIVITY

In January 2022 came into force in France the RE2020, a new energy and environmental regulation for new constructions. Its three main objectives are:

- Energy sobriety and decarbonization of energy
- Reduction of the carbon impact of buildings
- Comfort during heatwaves

In this paper, we focus on describing the challenges of its second objective on carbon emissions, and more specifically carbon emissions of structural components of buildings, keeping in mind these are part of broader regulatory objectives.

An adaptative process through a series of working groups to collect data and provide guidance

RE2020 was designed to be evolutionary, allowing time for stakeholders to adapt, provide feedback, and ensure alignment with France's National Low Carbon Strategy (SNBC).

It was preceded by the E+/C- experimentation (Positive Energy & Carbon Reduction experimentation), launched in 2016 and co-led by the French Government and the Higher Council of Construction and Energy Efficiency (CSCEE). This initiative aimed to upskill the sector on climate issues and collect large-scale performance data in preparation for future regulation.

The E+/C- experimentation enabled to establish the foundations of a building environmental performance evaluation method, however several technical subjects still needed further exploration. Thus, the following working groups were set up:

- Experts Groups (GE): Started in Autumn 2018, they identified several scenarios for the evolution of the calculation methods (from E+/C-) and highlighted the advantages and disadvantages of each scenario to provide guidance to future Consultation Groups on possible RE2020 calculation methods and data production.
- Modelisateur Working Groups (GTM1 & GTM2): involving design consultancies, quantity surveyors and research agencies, they carried out in-depth simulation work with the objectives to collect further data (technical and cost) to provide guidance to future Consultation Groups on possible RE2020 indicators and thresholds. The first phase GTM1 included residential, offices and schools and is now complete. GTM2 is for other tertiary buildings and is still in progress at first semester of 2025.

- Consultation Groups (GC): Between 2019 and 2020, stakeholders across the whole industry aimed to improve the calculation methodology and capture expectations on requirements and priorities of the regulation. Their final recommendations were submitted to CSCEE (Higher Council for Construction and Energy Efficiency) for preparation of the RE2020.

An adaptative process through progressive thresholds of regulation

The first challenge, and main objective of defining a first phase spanning from 2022 to 2024, was to enable the industry to familiarise with the life cycle analysis (LCA) methods. It improved the accuracy of LCA data, previously showing up to 30% error margins during E+/C-, and encouraged manufacturers to publish environmental data. This phase promoted low-carbon materials without mandating specific ones, with thresholds kept deliberately achievable.

The second challenge, and the objective of a second phase with gradually reviewed thresholds (in 2025, 2028, and 2031), was to drive adoption of underused low-carbon methods and innovative materials.

To ease implementation, RE2020 adopted a staggered rollout: starting with dwellings and collective housing in January 2022, followed by offices and schools in July 2022, and small buildings and extensions in January 2023. Tertiary and industrial buildings remain under consultation. Notably, RE2020 currently applies only to new buildings, with studies on existing buildings underway in 2025.

The RE2020 Observatory, managed by DGALN and operated with CSTB, monitors the implementation of RE2020 by tracking construction performance, informing future regulatory updates, and supporting industry upskilling. It gathers data from standardized energy and environmental summaries (RSEE files) submitted at project completion to provide transparent, sector-wide insights.

An overview of the RE2020 framework and its requirements

RE2020 encompasses numerous innovations, including the introduction of mandatory carbon accounting through Life Cycle Analysis (LCA), in accordance with the French SNBC and aligned on the European ISO 14040 and ISO 14044.

The regulation introduces the concept of dynamic LCA, as opposite to static LCA (as per E+/C- experimentation). It considers the temporality of emissions and therefore the effects of embedded carbon by assigning greater weight to today's emissions than to future ones. Thus, materials emitting little during manufacture and storing carbon, such as wood or bio-based materials, are favoured. Yet, the thresholds set in RE2020 maintain a focus on outcomes rather than means, giving designers and builders the freedom to choose the materials and techniques they wish to implement optimally.

Project owners of buildings subjected to RE2020 are required to sign a first declaration at building permit application as an engagement of compliance with RE2020 (which requires having conducted an LCA), and a second attestation upon completion of work that confirms set objectives were met. Failure to comply with set environmental objectives at the end of construction could mean the Local Authority could refuse to validate the Declaration Attesting to the Completion and Conformity of work) and request either the demolition or further work to achieve compliance.

The environmental performance evaluation process consists of providing as inputs both products and project data, then selecting an appropriate LCA software that will provide as an output a calculation of the environmental performance, as detailed in table below.

Table 1: Description of the process of evaluation of environmental performance.

	Description	**Provided by**	**Comment**
INPUT #1: **Project Data**	Quantity of **functional unit** of building products and equipment (e.g. volume or linear of structural elements, number of ventilation units, water consumption of site)	Project designer, project manager, environmental engineer or other project's stakeholder	An additional factor can be needed to adapt to Environmental Data, e.g. to adapt lifespan, quantity differing from functional unit, etc or to dynamic weighting
INPUT #2: **Product Environmental data**	Describe impact of a product on 36 environmental criteria, including "Impact on Climate" (Climate Change) Can be either Specific Environmental Data* or Default Environment Data*	Product's manufacturer (Specific Data) or Ministry of Construction (Default Data) Accessible on INIES database	RE 2020 doesn't regulate all 36 environmental criteria but an evaluation of the 36 criteria is carried out by LCA software
LCA software	Must perform environmental evaluation for each element of scope for each LCA phase, and create as output a Standardised Summary file for Energy and Environmental studies (RSEE)	Independent software editors. Approved software listed on RE-RT-Batiment website	Have limited time validity, and requires a first self-verification of results by editor, followed by a verification by Cerema prior to final ministerial approval
OUTPUT: Environmental performance	**$IC_{construction}$ and IC_{energy} , in $kgCO2eq/m^2$**		Many intermediate indicators related must be calculated according to RE2020 without being subjected to minimal requirements.

IC is the indicator introduced by RE2020 to measure the "impact on climate" (or "climate change"). Although Environmental Data must include 36 environmental criteria, RE2020 currently only regulates one of them, namely the impact on climate.

$IC_{construction}$ and IC_{energy} are amongst the key values measured and subjected to thresholds in RE2020 and are expressed and related as follows:

$IC_{building} = IC_{componants} + IC_{energy} + IC_{site} + IC_{water}$

$IC_{construction} = IC_{componants} + IC_{site}$

$IC_{building}$ and IC_{water} are required to be calculated as per RE2020 but are not submitted to regulatory thresholds.

An overview of the impacts of RE2020 on structural engineers' work

Structural elements significantly impact the calculation of $IC_{construction}$, typically contributing to 30-40% of its total value. Therefore, the implementation of RE2020 has a significant impact on structural engineers' scope of work.

The impactful decisions on low carbon are essentially made during the early stages of the design. However, detailed environmental impact assessments (LCAs) are usually done later in the design process, therefore it is critical that structural engineers learn how to perform LCA calculations to narrow down options for the structural components from the outset of a project.

Moreover, structural elements are central to other performances of buildings, such as acoustic, fire or thermal. If these performances are not considered early in the design process, additional materials may be needed later in the design, potentially compromising carbon reduction goals. Since structural engineers specify structural elements, they should expand their design criteria to include acoustics, fire safety, and other relevant factors from the start.

Additionally, as calculations of $IC_{construction}$ include the impact of the construction site, a good understanding of the site methodologies and procurement are required to make informed decisions at early stage of design.

THOUGHTS ON OPPORTUNITIES FOR NEW ZEALAND STRUCTURAL ENGINEERS TO ACCELERATE CHANGE ON LOW-CARBON DESIGN

New Zealand has a timely opportunity to lead in low-carbon structural design in the Asia-Pacific region by equipping engineers with practical tools and training. This includes familiarising professionals with LCA methodologies, endorsing trusted software, and creating a platform to share updates on low-carbon materials and best practices. Tools like BETie could help generate more accurate product data, enabling better-informed design choices and reducing reliance on generic, often penalising, assumptions.

To ensure lasting impact, future regulations must be ambitious, adaptable, and shaped by meaningful industry input. Learning from France's RE2020 experience, New Zealand can

structure engagement in phases – starting with expert groups and expanding to the broader sector – while avoiding over-consultation that may dilute intent. A shift from prescriptive compliance to performance-based outcomes will also foster innovation and flexibility as the climate and available solutions evolve.

Beyond regulation, consultancies should empower engineers to engage in low-carbon design through real projects and CPD initiatives, regardless of projects' targets. Anticipating potential conflicts between carbon goals and other design requirements, early discussion around professional responsibilities of engineers signing off the design will be essential.

Finally, engineers have a critical role in raising public awareness – helping communities understand the urgency of climate action and inspiring collective confidence in the transition ahead.

CONCLUSIONS

As the climate crisis deepens, the construction sector – particularly the structural design discipline – has a critical role to play in accelerating carbon reduction. France's experience with RE2020 demonstrates that a phased, adaptable regulatory framework, underpinned by robust life cycle assessment (LCA) practices, can provide a clear and practical path toward lower embodied carbon. By first focusing on upskilling and voluntary experimentation, then progressively tightening targets, France has enabled industry transformation without paralyzing innovation or delivery. The existence of supporting structures such as the RE2020 Observatory is also instrumental in providing transparency, data collection, and a feedback loop between policy and practice.

New Zealand, though at an earlier stage, has strong foundations to build upon – particularly through its professional bodies, tight-knit industry networks, and emerging government-led initiatives. What's needed now is the strategic implementation of ambitious yet flexible regulation that promotes outcomes over prescriptive methods, fosters early engagement with expert practitioners, and evolves with feedback and technological advancement. Structural engineers must be empowered with tools, data, and time to integrate carbon considerations into everyday design, and firms should actively support this cultural shift by encouraging exploration and learning within their teams.

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8. IFPEB - Le Hub des prescipteurs bas carbone: https://www.ifpeb.fr/travaux/le-hub-des-prescripteurs-bas-carbone/

9. IFPEB – Low Carbon Building HUB: https://www.ifpeb.fr/low-carbon-building-hub/

10. INIES - https://www.inies.fr/

11. Ministère de l'écologie – Declaration environnementale de certains produits de construction et équipements destinés au batiment - https://www.ecologie.gouv.fr/politiques-publiques/declaration-environnementale-certains-produits-construction-equipements

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Eastern Busway Alliance - Design and Construction of Rā Hihi

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Abstract

Rā Hihi, a major flyover, is part of the Eastern Busway project in east Auckland. When completed next year, it will provide a direct link between Pakūranga Road and Pakūranga Highway, running directly above Reeves Road, and provide grade separation across the busy Ti Rakau Drive to alleviate congestion around Pakūranga Town Centre.

This paper examines the development of Rā Hihi, focusing on the design and construction considerations that shaped its form. We explore structural type optioneering, structure lengthening to avoid costly ground improvement, use of large diameter monopiles, column plunging operations, three-dimensional modelling of the structure and reinforcement using Revit software, thermal modelling and maturity analysis for large element concrete pours, and construction challenges and sequencing in the built-up Pakūranga Town Centre.

Project Background

Eastern Busway Alliance is currently constructing the Pakūranga to Botany section of the busway, a significant project for East Auckland. Upon completion, the busway and Rā Hihi will enhance local connectivity and efficiency, offering improved sustainable travel options for all users. The project will establish a reliable public transport service between Botany and Waitematā Station (Britomart), forming a crucial part of the region's rapid transport network.

Figure 1: Eastern Busway Zone 2 overview including Rā Hihi

Rā Hihi will feature two general traffic lanes in each direction, connecting Pakūranga Highway to Pakūranga Road. This flyover will reduce traffic on Ti Rakau Drive by redirecting predominant flows between Pakūranga Highway and Pakūranga Road. The new configuration allows for the narrowing of Pakūranga Road, enhancing pedestrian and cyclist facilities. The space beneath Rā Hihi along Reeves Road will transform into a new public area, fostering connections between Pakūranga Town Centre and nearby community and recreational facilities.

Structure Development

Rā Hihi (formerly Reeves Road Flyover) has been the subject of several studies in the past with many commonly adopted bridge solutions explored to address the key site constraints, including spanning Ti Rakau Drive, fitting within the Reeves Road corridor, and creating an aesthetically pleasing structure with inviting space beneath. In 2019, AECOM completed the specimen design, proposing a 450m long, 45m span post-tensioned balanced cantilever concrete box girder supported on single flared columns.

The evolution of Rā Hihi's design focused on optimizing structural efficiency, cost-effectiveness, and construction feasibility while addressing site-specific challenges. During the early design phase, the project team considered four superstructure options:

1. Concrete box girder
2. Single steel box girder
3. Triple steel box girder
4. Super T beam and slab

Additionally, two pier foundation options were evaluated:

1. Single 3.0m diameter bored pile
2. Reinforced concrete (RC) pile cap with four 1.5m diameter bored piles per pier

The Alliance sought input from design and construction teams, external contractors, and industry experts. A Multi Criteria Analysis (MCA) determined the structural form for detailed design. The Super T superstructure emerged as the preferred option due to potential cost savings and familiarity with its construction methodology in New Zealand. The 3.0m monopile foundation option was selected for its potential programme and cost savings, reducing the number of piles and in-situ pours.

During preliminary design, the structure was lengthened by 145m, adding four additional piers and spans. This modification eliminated the need for costly continuous flight auger (CFA) pile ground improvements under the approach embankments and protection structures over existing services. Lengthening the structure resulted in approximately four months of programme savings.

Figure 2: Rā Hihi Concrete Box Superstructure Option

Structural Form

The final design of Rā Hihi is a 587m-long, 17-span flyover. The superstructure consists of 8 precast pretensioned Super T beams, each 1825mm deep with a 200mm thick cast in-situ reinforced concrete (RC) deck slab. The substructure features flared piers with a 2.6m diameter column core supported by 3.0m diameter bored monopiles. Monolithic connections are provided between the deck and piers at most internal support positions. End piers are articulated with two spherical bearings and shear key atop the pier flares, designed as corbel elements. The entire length of flyover is continuous between end abutments with no intermediate movement joints. Steel finger movement joints and spherical bearing under individual beam are located at abutments.

Rā Hihi is subject to extensive scrutiny at close quarters by the public due to its location in Pakūranga Town Centre. The design team has worked closely with the urban designer, bridge architect, mana whenua and an Iwi artist to address the aesthetic needs. The approach to the form of the bridge has been to provide a simple, aesthetically pleasing structure having consistent form and proportion. Architectural patterns and lighting are provided at abutments, flared piers, barriers, and extended barrier skirt. Deck diaphragms are flush to the soffit with no headstocks to reduce the bulk of the structure.

Figure 3: Rā Hihi Architectural Illustration

Design

The structures alignment consists of a 215 to 275m radius curve at the north and south approaches with a 250m long straight section above Reeves Road. The super T flanges are varied in width to accommodate the horizontal curve. The bridge has a varying vertical gradient and begins partway through a vertical curve at the north and south approaches. The in-situ RC deck thickness is varied to accommodate the vertical curve on the 'straight' Super T beams. The overall width of the bridge is limited by the adjacent properties along Reeves Road. The width between edge barriers was kept to 16.0m by reducing the lane shoulder widths.

Rā Hihi spans multiple obstacles, including the Ti Rakau drive over dimension route, Reeves Road, the intersections of Cortina Place and Aylesbury Street, and the entrances to neighboring properties. The bridge alignment maintains less than 2.0m clearance from deck level to the existing Transpower 110kV overhead lines. The towers on either side of the bridge were replaced to provide the necessary clearance to the lines and allow sufficient room for plant to construct the bridge.

The structure was designed to the seismic design requirements of the Bridge Manual and NZS1170.5 with a site subsoil Class C. A Site Specific Seismic Hazard Assessment (SSSHA) was undertaken to determine the structures site-specific spectra. Due to poor ground conditions the height of approach MSE walls were restricted without costly ground improvements. Pier and abutment piles are 22-30m in length, socketed into the underlying East Coast Bays Formation rock layer. Thermal integrity profiler (TIP) wires were used to evaluate pile integrity by measuring heat generation in the curing concrete. 3.0m diameter monopiles were utilized to enable the plunging of the central core pier column reinforcing cage into workable wet concrete. This construction sequence eliminated the need for an in-situ pile cap and allowed off-site prefabrication of the flared pier cage. The long, continuous structure

experiences significant temperature and shrinkage effects that are relieved by additional bearings at end piers.

Three-dimensional modelling of the structure was completed in Revit software and integrated into a project wide Building Information Modeling (BIM) model. Reinforcement within congested concrete sections including piles, piers, diaphragms, and abutments were modelled in Revit to refine detailing and resolve potential clashes on site.

Thermal analysis was undertaken for the large concrete elements including monopiles, piers, diaphragms, and abutments in accordance with CIRIA C766. Semi adiabatic hot-box tests as described in the Concrete Institute of Australia (CIA) Z7/07 were undertaken for the proposed concrete mixes. An assessment for heat of hydration and early thermal cracking was completed to inform maximum and differential temperature limits, formwork insulation and monitoring probe locations within the sections.

Construction

Rā Hihi demonstrates that a 10,600m^2 flyover through a built-up urban environment can be constructed in 15 months through detailed planning. Maximizing off-site fabrication was a key contributor to the efficiencies gained during construction. This minimised disruptions to neighbors and reduced crews working at height or over live traffic creating a safer work site.

In the construction of the substructure, 3.0m diameter monopiles were utilised, the largest rock socketed piles in New Zealand. The monopiles reduced the amount of reinforcement tying on site from a traditional pile cap and allowed the prefabrication of pier cages. A polymer plant was constructed on site to manage and recycle the fluid used to support the pile shaft during drilling.

The flared pier reinforcing cages were assembled off site in a purpose-built jig which ensured a more precise controlled process and minimised delays due to access and coordination. The quality of the cages could be monitored closely to reduce errors and potential clashes on site from the complex reinforcing tying procedure. Once the bored pile concrete pour reached top of pile level the prefabricated pier cages are delivered to site in their jig and lowered into an A-frame positioned over the bored pile. Once the pile concrete has been poured to cut off level, the pier cage is plunged into the workable wet pile concrete and locked off at the design height as shown in Figure 4. The A-frame allowed the pier position and height to be accurately set beforehand, reducing errors and time spent surveying while the concrete hardened.

Figure 4: Off-site fabrication of pier flare and plunging operation

Using purpose-built steel forms for the pier and diaphragm construction created a repetitive workflow which enabled the crews to gain speed. The flared pier form consisted of modular sections that enabled the 16 different pier heights and shapes, as seen in Figure 5, and weighed approximately 20 tonnes. A rubber form liner installed on the inside of the steel form created the fine architectural pattern on the pier. The pier concrete was bottom pumped through valves in the pier form using a high slump self-compacting concrete mix which minimised imperfections and remediation of the finished surface. Foam insulation installed on the steel form reduced differential temperature effects and a custom tarpaulin blanket allowed early formwork removal. The modular pier form required complex planning, handling, and assembly, however, over time the crew became adept at maneuvering the form and reduced the cycle time between pier pours from two weeks to four days.

Figure 5: Flared pier flare formwork and architectual pattern

The Super T beams were delivered with the sacrificial timber deck formwork in place, permanent drainage and lighting works installed and a foldable handrail and edge form system which enabled safe access to the deck quickly once the Super T's were placed, as seen in Figure 6. The hardwood packers, which determined the heights of the Super T placement were prepared in a workshop to ensure precision and allowed for them to be checked before installation.

Figure 6: Super T beam delivered to site with handrails, edge forms and drainage installed

The temporary works supporting the Super T beams was a 40 tonne modular, leapfrogging system designed to maximize the efficiency of the four sets of falsework. A heavy-duty system made up of steel frames on precast footings with granular hardfill build-up over live services supporting approximately 1220T. In comparison, a proprietary system would have reduced the space available on site due to the additional propping required. Similar to the steel pier forms, reusable diaphragm infill shutters allowed for diaphragm formwork to be installed within three days of landing adjacent Super T's seen in Figure 7, significantly reducing time spent compared to traditional timber forms.

Figure 7: Superstructure falsework and steel diaphragm formwork between Super T beams

A maturity curve was developed for the pier and diaphragm concrete mixes as defined in ASTM C1074. Through the use of temperature sensors and the concrete maturity curves, the strength development of the larger concrete elements could be determined with sufficient confidence to reduce formwork stripping times. This contributed to programme savings for Rā Hihi as in situ strengths could be monitored in real time, reducing the waiting times for cylinder results.

Upon removing the falsework, a crew moved in immediately to complete repairs or any snagging items identified which improved quality assurance close out times as common issues are remediated as soon as the physical work is complete.

Overall, the success of the construction of Rā Hihi can be attributed to driving and staying committed to the programme, fostering a collaborative relationship with subcontractors and suppliers. A creative and experienced engineering team paired with a proactive, efficient site team with a good eye for detail and safety culture has allowed innovation to be put in practice, creating a new standard for construction in New Zealand infrastructure. This has been acknowledged by Auckland Transport, the subcontractors and suppliers, and the public who have been astounded by the fast build.

Temporary Works: Lessons Learnt and Future Focus

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Chair: Temporary Works forum NZ.

ABSTRACT

Historically, “temporary works” have not been tightly regulated in New Zealand with some work being done well, and some done poorly. Attention has been given to the permanent structure whilst temporary works have received less consideration and generally left to the contractor. This has not always worked out well.

The collapse of the scaffolding on the DIC Building in Wellington in 1957 was essentially a temporary works failure where two members of the public died. This led to a Royal Commission of Enquiry and the passing of the Construction Act 1959 administered by the Department of Labour.

Ramp “A” collapsed in 1975 during the construction of Auckland’s central motorway junction. This was another temporary works failure.

As recently as 2017, scaffolding supported from the Panmure Bridge in Auckland collapsed and six workers fell into the Tamaki River.

This paper calls for structural engineers to support the self-regulating initiatives of the Temporary Works forum (an Engineering New Zealand Technical Group) and not wait for a serious incident to force our hand.

What are Temporary Works and who designs them?

Temporary works are the structures that are used to support and enable the construction of a permanent structure such as a bridge or building. In the past, their short, transient lifespans have kept them in the background of structural engineering. Little attention was given to temporary works by the permanent works designers leaving most of the responsibility to the contractor. In recent years there has been growing interest in temporary works by clients, consultants and contractors. Some consultants now specialise in temporary works and most larger contractors have in-house temporary works design teams.

Lessons from the Past

Historically, the risks accepted by the construction industry and the public were high unfortunately resulting in injuries and on occasion, death. This was the case in 1929, for example, when the Civic Theatre was built in Auckland. Productivity and lowest cost were valued above the welfare of workers.

Figure 1: A worker dangerously chocks the wheel of a brake-less truck as it is towed up a slope during the excavations for the construction of the Civic Theatre. (source: TVNZ)

There was a lack of engineering knowledge of the load paths and structures involved in temporary works resulting in failures and disproportionate collapse. Two of these cases are discussed below as they have a bearing on the evolution of temporary works in New Zealand.

The DIC Scaffold Collapse, 1957

On the 8th of May 1957, the scaffolding on the face of the DIC building in Wellington collapsed and two members of the public died.

Figure 2: Collapse of the scaffold – DIC Building, Wellington. (source: Alexander Turnbull Library)

A Royal Commission of Enquiry[1] was launched, and it was found that overloading had occurred whilst dismantling and stockpiling scaffold materials causing buckling of the standards and failure of the verandah structure on which the scaffold was built. Some back-propping of the verandah had been done, but there was no proper understanding of the verandah structure, or the loading imposed by the stockpiled materials.

This was a temporary works failure.

Following the commission of enquiry, the government of the day passed the Construction Act 1959 which was administered by the Department of Labour (DOL). Inspectors from the DOL were empowered to uphold the Act and Regulations, visit sites, stop the work if necessary and initiate legal proceedings against contractors who failed to comply.

The DOL and the subsequent government organizations (OSH, WorkSafe) have worked to improve methods and reduce harm in the construction industry, but it is regretful that the catalyst for their necessity was the fatal consequences of the DIC collapse.

Incidentally, the Health and Safety in Employment Act 1992 superseded the Construction Act, and it is this legislation and its successors that primarily govern temporary works today.

The Ramp A Collapse, 1975

On the 6th of May 1975, a 55m span of the post-tensioned concrete box girder for Auckland's central motorway junction bridge collapsed during construction. There were fortunately no injuries during this event as it occurred during the workers lunch break.

Figure 3: Collapse of Ramp A – Auckland. (source: WSP)

This occurred one day after a falsework collapse on the South Rangitikei rail bridge which again, fortuitously, did not result in injury. Both incidents were reported in NZ Engineering[2].

The Ramp A span required temporary support until such time as the final post-tensioned cables were installed and tensioned. This temporary support was provided by scaffolding.

It was found that the post-tensioning operation had caused arching of the span which in turn shed load to the scaffold standards at the ends and away from those in the middle. Furthermore, the ground at one end was partly fill material allowing uneven settlement of some of the standards and exacerbating the load on the others. A progressive collapse ensued.

This again, was a temporary works failure but with no injuries or deaths, there was no Royal Commission and no new Act of Parliament.

However, the Ministry of Works and Development (MWD) did respond to these two events by conducting an investigation and as a result, produced a Code of Practice (CoP)[3] for Falsework that was intended to be widely disseminated for discussion and gain general acceptance. This document was used by many engineers specialising in temporary works.

The MWD investigation found that amongst other things, that there were 'procedural' problems in the way temporary works were carried out in New Zealand. Although the CoP contained some very good guidance for procedural control, this was focused on falsework, not temporary works in general.

Ongoing Temporary Works Failures

On the 21st of February 2017, scaffolding suspended from the Panmure Bridge collapsed and six workers fell into the water below.

Figure 4: Scaffold collapse – Panmure Bridge – Auckland. (source: RNZ News)

A WorkSafe investigation[4] determined that the collapse was due to overloading. The scaffolding company was fined and ordered to pay reparations and WorkSafe accepted Enforceable Undertakings from both the main contractor and the Engineer to the Contract.

It was fortunate that the workers affected were not seriously injured but this incident, like the previous two were gravity load failures and were avoidable. Environmental loads such as wind and earthquake are variable and we have codes to determine the load intensity to be

resisted, but these loads could be exceeded. Not so with gravity – the load intensity is calculable and a reliable load path down to the ground should be established by the designer.

In all of these incidents, gravity loads were not thoroughly considered.

The UK Experience

In 1972, just 3 years before New Zealand's Ramp A collapse, there was a falsework collapse on the River Lodden in the UK. This was also a gravity load failure. There were 3 fatalities leading to an investigation by Her Majesty's Factory Inspectorate.

The final report, known as the Bragg Report, made a number of recommendations for changes to how falsework was designed, constructed and dismantled. This in turn led to the publication of BS5975 which eventually became a Code of Practice for Temporary Works, establishing procedures and roles including the role of a Temporary Works Coordinator.

The Temporary Works forum UK was incorporated in 2010 and exists as a not-for-profit company providing guidance and leadership to the industry in the field of Temporary Works.

The NZ Temporary Works forum

The Temporary Works forum NZ (TWfNZ) came about when a group of temporary works engineers decided in April 2018 that we should not wait for the next serious harm incident. We decided that best practice temporary works should be written down, published and promoted in New Zealand, not as a response to a serious harm or fatal incident, but rather, because we don't want there to be a serious harm or fatal incident.

The TWfNZ was established to encourage open discussion on any matter related to Temporary Works, for the benefit of the New Zealand Construction Industry. It was inspired by the TWfUK who were supportive towards the New Zealand initiative. Initially a special group of SESOC, the TWfNZ has now become a Technical Group of Engineering NZ. As is the case in other Technical Groups, time spent by committee members is volunteered by them and their home organisations. Over the past 7 years, the TWfNZ committee has met 63 times, run 17 Public Meetings, produced 16 publications and currently has 386 members.

The first aim of the TWfNZ was to produce a Good Practice Guide for Procedural Control[5]. This was produced by the initial committee made up of contractors and consultants and was published in 2019 after a period of public consultation. The first update for this document is currently underway and is expected to be published late 2025.

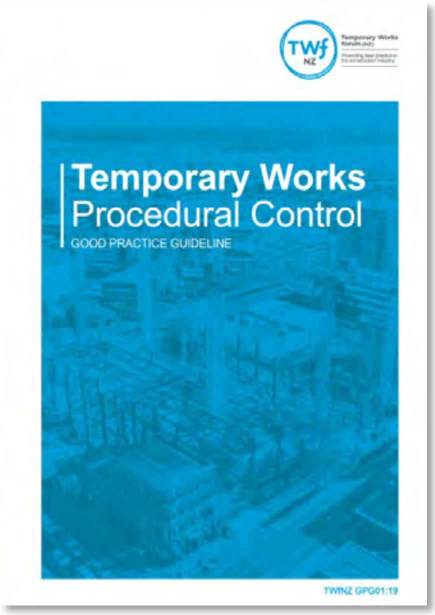

Figure 6: The Good Practice Guide for Temporary Works Procedural Control

The Good Practice Guide (GPG) has been widely accepted by many clients, contractors and consultants and has been endorsed by Construction Health and Safety NZ (CHASNZ). Some companies have reproduced the GPG as part of their Standard Procedures, or in the form of a project specific “Temporary Works Management Plan” with their own branding – something that is encouraged by the TWfNZ.

Focus for the Future

Our effectiveness as competent structural engineers who design and inspect temporary works depends on how committed we are to following good process. The TWfNZ has focussed much of their efforts on contractors, but parallel support from structural engineers in the consulting sector is crucial.

If there is a future serious harm incident involving temporary works, there may well be an enquiry and a response from central government in the form of new rules and regulations.

Right now, we have the opportunity to self-regulate and determine the future of temporary works through robust debate, documenting and promoting good practice as a united group of structural engineers. This can be done by joining the Temporary Works forum, contributing to the working groups who are producing Technical Guidance Notes and commenting on documents that are sent out for public comment.

Conclusions

Temporary Works failures from the past have resulted in injury and death and in most cases the lack of good process was a major contributor.

In 2018, the Temporary Works forum was formed by engineers who wanted to pre-empt a serious harm incident by producing a GPG for Procedural Control along with other Technical Guidance Notes. A commitment to self-governance can not only prevent a future failure and save lives, but also means that we, the practicing engineers, get to write our own rules.

Support for the Temporary Works forum from engineers who design temporary works can be offered by:

- joining the Temporary Works forum
- contributing to the working groups who are producing Technical Guidance Notes
- commenting on documents that are sent out for public comment.

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Wind Comfort Modelling for the Sustainable Built Environment: A Structural Engineering Perspective

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ABSTRACT

The integration of environmental investigation such as wind comfort analysis into the early stages of structural and architectural design has become increasingly critical. Computational Fluid Dynamics (CFD) modelling and wind tunnel modelling offer an efficient solution to assess pedestrian wind comfort and iterate to efficient massing. This allows for massing and structures to efficiently address the environmental issues and provide a sustainable response to the design brief. Our presentation will showcase the optimization of structural forms and layouts to mitigate adverse wind effects, potentially leading to a reduction in unnecessary material usage, enhanced occupant well-being, and improved urban microclimates. It will incorporate case studies of Holmes projects to demonstrate how this workflow has been incorporated.

Wind Comfort Modelling for the Sustainable Built Environment: A Structural Engineering Perspective

Introduction

As urban centers grow in density and the requirement for sustainable development intensifies, we see a demand for a more holistic and integrated design approach. Structural engineers have traditionally been focused on their own discipline leaving the arhtct to look at the overall aspects of the project. Whilst crucial to the structural integrity of the project, this perspective can leave the impact of the developments on its surroundings lacking.

A successful sustainable project will consider life cycle assessment and will also consider the impact of the design on its surroundings and how occupants interact with it. A successful wind microclimate experienced by occupants is a critical factor in successful project delivery.

The Importance of Wind Comfort in the Built Environment

Pedestrian Wind Comfort (PWC) study relates to how wind conditions affect occupants using outdoor and semi covered outdoor spaces. It is a correlation of wind speed and frequency of occurrence which are then compared to international criteria to allow us to quantify acceptable conditions for different activities (e.g., standing, sitting, walking).

While a minimum criteria is often dictated by local authorities at a resource consent stage, the importance of wind comfort extends far beyond code compliance and is intrinsically linked to the success, activation and sustainability of a development:

Usability and Amenity:

Successful developments are comfortable ones that are engaged with by occupants. In Auckland alone, there are many public spaces between developments that are known as uncomfortable wind-swept spaces. Persistent high winds will deter the use, negatively impacting the viability of retail / outdoor seating, the enjoyment of public spaces, and even entering or exiting a building. Comfortable spaces invite activity and contribute to urban vibrancy and a well designed space will contribute of the sustainability of the development. It is also possible to combine the effects of wind and thermal analysis to provide wholistic comfort assessment.

Safety:

Beyond discomfort, excessive wind speeds can pose genuine safety risks. This is particularly important for taller development which have the potential to make wind conditions worse in the surrounding streetscape.

An extreme example of a windy development causing harm is the Leeds (UK) Bridgewater development. The shape and form of the structure made wind conditions less safe and lead to a death and several injuries. The costly mitigations to solve this issue included large baffles and fins and extensive landscaping.

Figure 1: Bridgewater, Leeds, mitigations required to decrease wind related hazard around development.

Impact of structural engineers on wind comfort.

Whilst wind comfort is primarily driven by the architecture, massing and landscaping of a development, there is a flow on effect to structural engineers. who may have to change their designs if the wind comfort is not considered at the onset of the design.

The most effective measure to ensure a development is comfortable from a wind perspective is to assess the massing at the onset of the development and tweak it to align with the function and expectations of the space. This is most effective at concept stage before the constraints of a resource consent.

Mitigations implemented beyond the point where layouts are frozen will likely be less effective and more difficult to design. These are likely to involve the retrofit design of screens, canopies or landscaping to reduce wind speeds in activated spaces.

Retrofit mitigations will typically require a structural engineer to think outside the box and connect these mitigations to a structure that may mot be adequate for this purpose.

It is therefore more sensible to consider the impact of wind comfort early in a project.

Methodologies for Assessing Wind Comfort

Predicting and assessing wind comfort conditions requires tools that simulate the complex interaction between wind and the built form. The two primary methodologies are Computational Fluid Dynamics (CFD) and Wind Tunnel (WT) testing.

Historically wind tunnel testing has long been the preferred method for testing configurations and is still important for validation of models. It is an accurate and standardized process but it is also expensive with long turnaround times. CFD by comparison is relatively new has seen a growth in use since the late 2010s when improved turbulence models, mesh generation techniques, and validation studies made CFD increasingly accepted for regulatory purposes. In places like London, this type of assessment is commonly used to assess the impact of new developments.

CFD is affordable, fast and allows for efficient design optimization by testing more scenarios. It is an efficient way to showcase the mechanics of wind flows around buildings to the team. A CFD model typically requires a 3d model of the surrounding city and can incorporate a large extent of modelled detail such as canopies, atrium spaces, balconies which would be difficult to create in a scale model for wind tunnel testing.

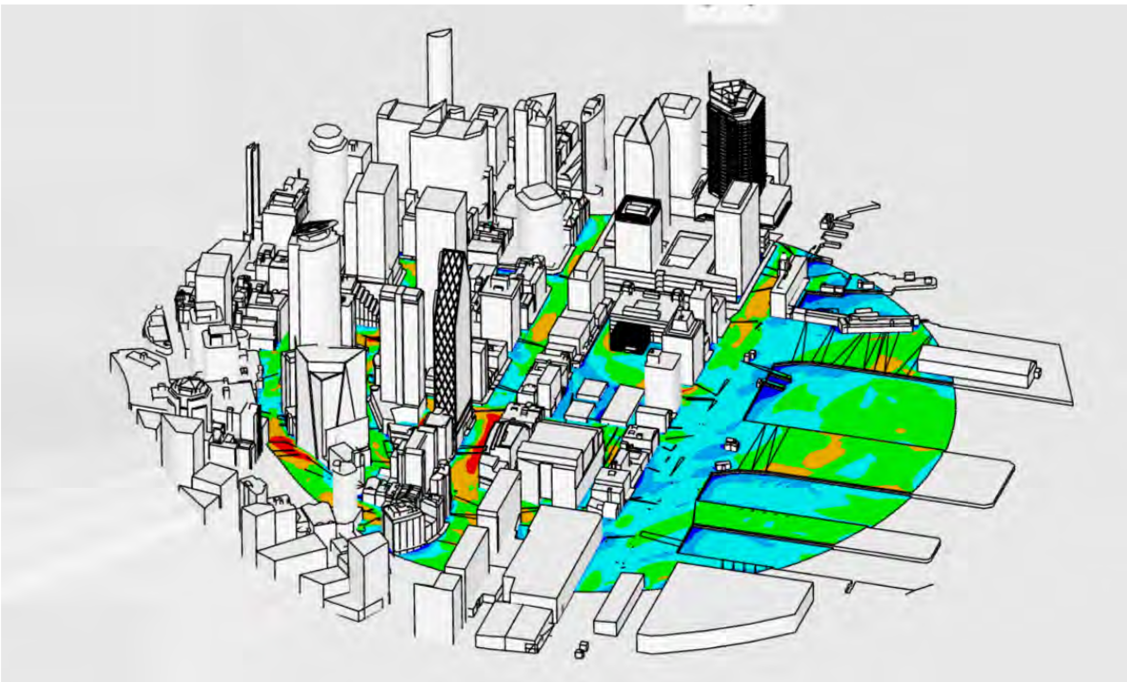

Figure 2: Example CFD model geometry and output of wind speeds Red colours indicate higher wind speeds

The model is calibrated to local wind parameters using data from local sources. The output of the modelling will be wind speeds (typically averaged) for each direction of wind analysis (typically 8 to 12 directions). This output can then be postprocessed to provide a statistical category of wind climate used to quantify the impact of development for resource consents.

Case study of CFD modelling

On a recent Holmes project the CFD modelling was undertaken in close collaboration with the structural engineer, this enabled the structure of the wind mitigations to be incorporated early in the design. The below figures show an extract from an assessment of the ground level wind speeds in an open atrium of a central Auckland project at the Preliminary Design Phase. A public passageway was planned though the middle of the development at ground level.

One of the Wind/Structural components to this assessment was the incorporation of large moveable wind screen to create a calmer wind microclimate inside the retail atrium of the space. The screens (circa 6m high by 5m wide) were proposed on a track system placed in the middle of a 12m span of the structure. It was therefore important to coordinate the placement and effectiveness to make sure these were required and design transfers to support these.

A number of different wind directions were assessed as well as a number of different configurations to arrive at the conclusion that the screens would make a significant difference in the use and ability to have a successful retail environment on the ground floor of the development.

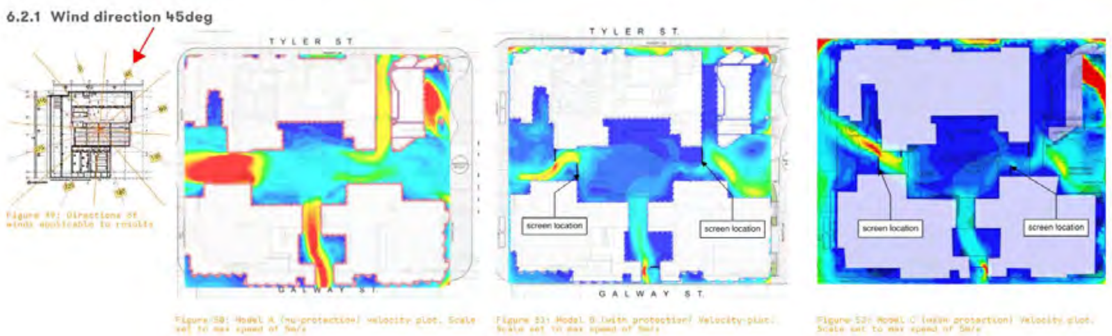

Figure 3: comparison of options for screen effectivess for winds in 45degree direction. Red colours indicate higher wind speeds

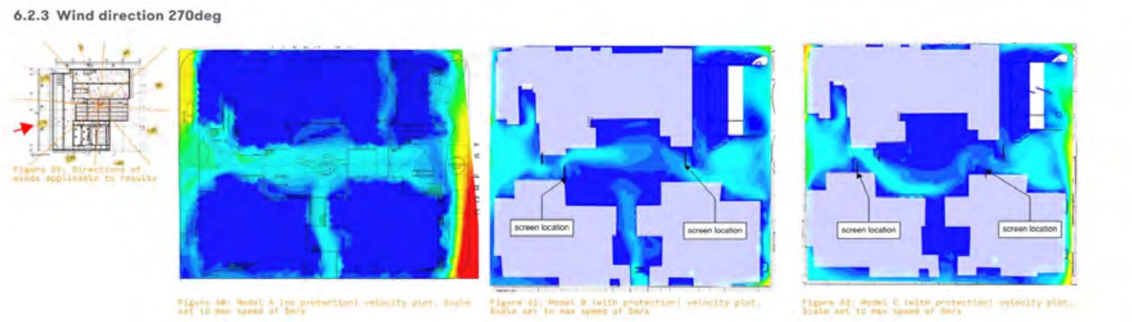

Figure 4: comparison of options for screen effectivess for winds in 270 degree direction. Red colours indicate higher wind speeds

The possible Future of CFD modelling for Structural Engineers

Whilst the main current applications of CFD are related to wind comfort, there are new developments which will be directly related to how Structural Engineers design tall buildings.

The current limitations of variability of results between CFD and wind tunnel testing is being eroded by developments of new modelling methodologies and computing ability. At present no codified pathway exists to assess the below but all methodologies

Evaluation of structural performance and aerodymanics

Traditionally wind tunnel is used to undertake the assessment of wind related overturning demands or pressure mapping of a tall structure due to aerodynamic and site specific conditions. The assessment of the wind dynamics looks at the impact of building forms, pressure distributions and vortex shedding patterns to provide a detailed understanding of the lateral stability system requirements.

CFD enables a cost effective iterative approach to this analysis that can be undertaken to compare the benefits of shape and form on baseshear reduction. Whilst similar to the wind tunnel test simulations, the real advantage is one of speed and efficiency reacting and driving the process rather than reacting to the imposed architecture.

The current limitations of CFD currently require a baseline comparison rather than providing definitive baseshears. A recent example of a CFD study to assess the impact of the building shape on wind loads was undertaken by Adrian Smith + Gordon Gill. It modelled different shapes and aerodynamic profiles which enabled its concept to reduce overturning demands by more than 1/3. Such studies done early have the potential to make meaningful impacts on the extent of structure required for a building development. This leads to a reduction in cost but also a better more sustainable design.

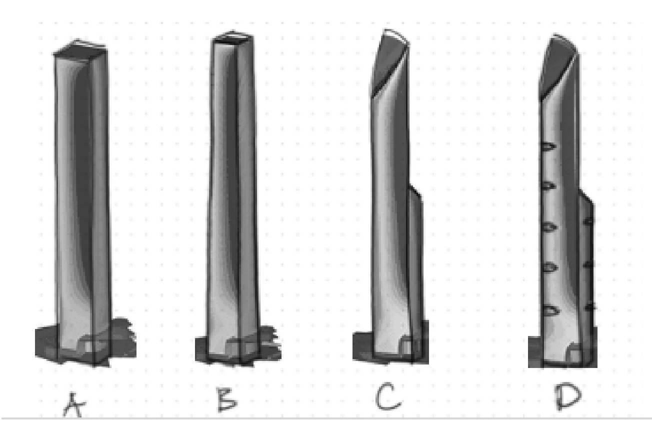

Figure 5: Example study of Baseshear reduction in CFD testing. The shape of the strucutre in comparison to a baseline square shape (A) reduces with model B by 4.5%, with model C by 25% and with model D by 35%

Conclusion

The integration of wind comfort analysis into structural and architectural design represents a significant step toward creating more sustainable built environments. Case studies show that early-stage CFD modelling provides the design team, with potential impacts of the massing on wind comfort.

By addressing wind comfort at the conceptual stage, engineers can help design teams optimize building massing, orientation, and structural elements to create comfortable, usable spaces while potentially reducing material requirements. This proactive approach not only enhances the occupant experience but also contributes to the overall sustainability goals of the project by ensuring spaces are fully activated and fit for purpose.

The evolution of CFD technology continues to expand the role structural engineers can play in environmental design. As computational capabilities advance, the gap between CFD and wind tunnel testing will narrow and make the technology more accessible to designers.

For structural engineers, embracing these tools and methodologies means moving beyond a reactive role to become active participants in shaping the environmental performance of our built environment. By considering wind comfort early and collaborating closely with architects and other disciplines, we can deliver structures that are not only safe and efficient but also contribute positively to urban microclimates and the broader sustainability agenda.

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New Zealand Residential Construction – Engineering realities – Foundations

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ABSTRACT

This paper is part of a series looking at residential construction in New Zealand, where there is some engineer involvement. This paper discusses geotechnical and foundation issues. The paper first outlines the intended pathway for investigation, design and compliance for foundation systems. We then draw upon the experience of members of the EngNZ Engineering General Practitioners group to highlight and address common issues and engineering discrepancies which appear to exist.

We review geotechnical investigation requirements for residential construction, and discuss the simple applicability of 'Good Ground' definition under increasing awareness of natural hazards, and the potential increased risks given climate change. We discuss proprietary systems and highlight some challenges Engineers face in specifying such systems when undertaking specific design. We discuss the actual use of the B1/WM4 methods, and their interaction with both NZS 3604:2011, and Building Code Clause H1, particularly when looking to balance cost, life safety and energy efficiency.

We recognise the time lag in the production and publication of standards, which can lead to competing and conflicting information, but look to propose a route for navigating change with a focus on the future for residential construction in New Zealand.

Introduction

This paper discussed issues which have been raised in the Engineering General Practitioners Special Interest Group Slack channel (egp-sig-nz.slack.com). The channel has been running for around four years and has around 350 members. There is regular and animated discussions relating to engineering matters, and the channel serves as an open forum for discussion of engineering issues facing General Practitioner Engineers. The majority of members and discussions have a structural engineering focus, but there is significant discussion on other technical aspects such as geotechnical and fire engineering, and around compliance pathways, legal and insurance issues.

This paper focusses on issues raised relating to foundations and structural engineering, and is a snapshot of the 240 threads to date relating to foundations.

The EGP group focuses typically on residential and light commercial type construction, often working with NZS3604:2011, and undertaking Specific Engineering Design (SED) for elements of a building generally designed to NZS3604. The interplay of SED and design to NZS3604 is a skill which requires a good understanding of the behaviour of proprietary products, their testing and published technical information, and keeping abreast of the wide range of BRANZ and other specialist bodies information. When combined with the complexities of New Zealand soils, this makes the design of residential foundations a job with many potential pitfalls which this paper seeks to elucidate. It is hoped that the majority of the aspects discussed here are well known to New Zealand Engineers, but the Slack forum has proved useful in identifying and challenging the thoughts of even very experienced engineers.

The paper first discusses geotechnical investigations, then issues with concrete slab foundations, and later timber pile foundations, before drawing some general conclusions.

Geotechnical investigation and reporting

The definition of 'Good Ground' as outlined in NZS3604 is well suited to simple lightweight residential construction. Most testing is undertaken with a 'Scala Penetrometer' or a shear vane. It is testament to its utility that a test developed for road and runway design (Scala, 1956) has become the mainstay of New Zealand residential construction. However with easily developable land becoming increasingly scarce, the number of 'Good Ground' sites is declining, and more complex methods may soon be required for investigation. There are many situations where subdivision level earthworks have been undertaken, and it may be that no investigation is required, and other situations where a scala penetrometer or a shear vane is not suitable for identifying wider natural hazards. This is usually picked up at Resource Consent stage, however, increased use of slightly more complex investigation sampling techniques (such as window sampling) may prove beneficial to reduce risk across New Zealand residential construction.

Concrete slab foundations

Typical residential construction in New Zealand usually features concrete slabs, generally with a concrete or block perimeter, internal thickenings and solid insulation below.

Frost Heave

While Frost Heave is considered a significant issue across Europe and much of North America , there appears to not be significant guidance in New Zealand. Some geotechnical firms add a comment to their report and some local councils may require a specific foundation depth. It is expected that the geographical areas affected would be similar to where SED snow loading

design is required - such as areas above 400m. A typical depth to prevent frost susceptibility would be around 400mm below cleared ground level.

Expansive Soils

The Building Code has recently been updated to offer guidance on Expansive Soils (amendment 19 and 20 (2019 and 2021)). This went some way to addressing the ‘regulatory void’ previously identified (Cook, 2019). However, there does still not appear to be a consistent industry approach to the design of foundation solutions on Expansive Soils. Some engineers favour deepened perimeter footings which serve to shield the subgrade from moisture content changes, while others design internal footings to the depth of the perimeter footings. Such variations in methodology could be incurring additional expense or additional risk, but it is difficult to currently determine which is the correct approach or if multiple approaches have similar merit.

Brace wall hold downs

Issues and queries around residential hold downs have been discussed at length over the past few years ((ENZ 2022) and there has been some industry response (GIB, 2022)). This paper does not seek to reproduce those arguements. Nevertheless, it would seem that the queries raised in 2022 have not yet been adequately addressed, either through additional testing, or through redesign of elements.

The increasing tendency to place external slab insulation, as recommended following recent H1 updates as lead to screw anchor edge distance issues, which are noted by manufacturers, but not necessarily taken up by all engineers. The typical requirement for bracing panel hold downs is usually given as 15kN for concrete anchors. Pryda explicitly states that only 14kN could be expected from a 90mm timber framed wall (Pryda 2024).

The adoption of waffle slabs with a 100mm or thinner concrete means that the required drill depth of 96mm can only be achieved with really quite precise works on site. Therefore brace elements would tend to have a thickening placed beneath them. While most engineers would tend to specify a thickening, it is certainly possible for builders to argue that one is not necessary.

The edge distance requirements for the screw anchor cannot be met where the recommended H1 insulation is adopted and a typical 90mm framed wall is overhung (GIB, 2024), and therefore a 140mm framed wall is generally required to provide any manufacturer compliant bracing solution.

While these challenges are acknowledged by the industry, with advice to ‘carefully consider the placement of bracing elements’, the development and testing of a system with a lesser embedment or reduced edge distance would seem prudent, allowed reduced slab thickenings,

and increased perimeter insulation thickness. We note that further testing was planned, but the results of this do not appear to be publicly available.

Adoption of H1

The increasing provision of EPS (Expanded PolyStyrene) and XPS (eXtruded PolyStyrene) insulation below perimeter footings is welcomed as providing additional insulation below the slab. However the long term performance of EPS under load is not well understood, and therefore its use for the design life of the building is uncertain. A typical detail recommends insulation below the slab, which is then wrapped underneath the perimeter foundation and up the face. The majority of the vertical load that a house perimeter foundation will be seen during the construction, and therefore any compression of the insulation will occur at the beginning of the building's life. However there may be certain locations such as below large lintels, or at post locations, where it may be necessary to remove the insulation and bear directly to ground. This is currently a source of engineering judgement and advice on the structural behaviour of insulation would be greatly appreciated.

Timber pile foundations

Lateral load resistance of piles

Anchor piles are often specified for bracing, with piles taken deeper than 'Ordinary' Piles. Anchor piles are usually 125mm diameter square, and taken from the whole log, and suffer less strength reduction as a result of cutting across the grain of the timber. There piles are stamped with an 'A' and have a nominal bending strength of 22.1MPa based on BRANZ SR046 (Thurston, 1993), which was downgraded to 17.5MPa by NZS 3605:2001.

These square 'Senton' piles are used for retaining, and is it important to note that only A stamped piles should be used for the design and a bending strength of 17.5MPa adopted for a nominal 125mm square pile.

Good Ground at depth

There are some situations where lower strength or stiffness soils exist above what would otherwise be termed 'Good Ground'. There is usually a good reason for lower ground strength, either through uncontrolled fill material, topsoils, or expansive soils leading to lack of support at the soil column. This depth is often quoted in geotechnical reports, with a requirement to increase the depth of timber piles to reach the 'Good Ground'. However the consequence of increasing the pile length beyond NZS3604:2011 requirements would increase the bending moment on the pole, and the expected displacements at the head under a nominal 12kN bracing load (Thurston 1993, Shelton 2007). While most anchor piles are encased in concrete, which is thought to give additional bending capacity for the section, the analysis methods for

this (for example Wood, 2021) are reasonably complex and not necessarily suited to the NZS3604 approaches. Some practitioners recommend driving nails; screws or reinforcing bars into the pile to aid shear transfer, but the magnitude of this is difficult to quantify.

The lateral resistance of soils where 'Good Ground' is not present at the surface should also be questioned. While some lateral resistance can be assumed, the strength and stiffness characteristics of the soil cannot be determined with simple methods.

For situations where Good Ground is at depth, it may not be suitable to simply lengthen the anchor pile and adapt the Senton piles otherwise destined for retaining wall use. However options to increase hole diameter, or even to provide nominal lateral resistance values for a range of pile depths to NZS3604 may benefit to industry through reduced material use.

When Specific Engineering Design is undertaken, the usual approach is to adopt the methods of Broms' as outlined in B1/VM4 (MBIE, 2023). While it is not common international practice for detailed formula and textbook type approaches to be present directly in Building Codes, the detailed methods given in B1 are somewhat useful. However, as recognised by Broms, and outlined in Pender, 2000 the original approach may be somewhat conservative. Indeed, the SESOC Soils programme adopts the Pender approach. When this is combined with the large possible variance in both the demand and capacity multipliers outlined in B1; Shelton (2007); NZS1170 series; NZS3603:1993 and NZS AS 1720:2022, there is a large potential variance in the pile lengths and expected resistance. While some of this can be attributed to 'Engineering judgement', there is certainly room for further guidance on the specific design of piles.

Brace Wall hold-downs

Hold down bolts for brace walls in timber floors exists in with timber floors, where a M10 x 140mm long bolt or similar is fixed into a 45mm thickness of timber and is rated as having 12kN uplift capacity, and 'timber floors constructed in accordance with NZS3604 do not require additional SED to achieve 120BU/m' (GIB, 2022). While manufacturers explicitly state this, and it is based on decades of NZ experience, changing timber grades and improved screw anchor manufacturing techniques would warrant additional testing to confirm the continued use of such systems. Such fixing would not typically be recommended by engineers, indeed the Pryda documentation makes specific reference to 10.7kN tension capacity only being available if the bolt has 25mm edge distance (Pryda 2024). Therefore there appears to be a mismatch between the Engineering requirements and the standard practice of 45mm wide timber joists and bearers common across New Zealand construction.

Conclusions

There are two main issues which this paper has raised: 1. The increasing movement away from redundancy, and the gradual intrusion of SED buildings into NZS3604:2011 space; and 2. The myriad of overlapping, guidance, standards and approaches, which require significant engineering judgment and can lead to varying solutions to the same problems. These can both be addressed through the provision of more information, and through the free exchange of information between engineers.

The intent of NZS3604 has always been ‘a rational engineering approach..(with) advantage,, taken of the redundancies, additional strength, and other favourable factors known to be present in light timber frame buildings’ (NZS3604:1978). NZS3604 type foundation solutions have developed over the years in response to New Zealand specific conditions and products. Over time these solutions may drift away from their original design intent, tested parameters, indeed the original test results on which engineers are required to rely is often unavailable or buried within old reports and lost to newer engineers.

It is expected that the forthcoming updates to NZS3604 address some of the issues raised in this paper, and that manufacturers work with the New Zealand industry to provide fit for purpose products with a focus for the future.

The excellent Engineering Basis of 3604 (Shelton, 2013) should be updated, and perhaps extended. While knowledge of the basis of the code is good, its integration into Specific Engineering Design, analysis methods and use with specific products would also be of particular help to Engineers.

Initiatives such as the Engineering General Practitioners Slack channel facilitate robust and interesting discussions and could be taken up across the wider engineering community.

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Producer Statements and their Changing Role in Litigation

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ABSTRACT

Producer Statements have been enigmatic documents from a legal perspective over the years. Originally defined and included in the now-repealed Building Act 1991, they were later excluded from the Building Act 2004. Despite this, Producer Statements remain a pivotal part of New Zealand's building regulatory environment.

Recent legal attention and judgements mean that the way Producer Statements are viewed and the significance that is placed on them is changing. Notably, issues such as fraudulently issued Producer Statements and a recent Court of Appeal decision confirming that Producer Statements can be considered building work – making engineers exposed to criminal prosecution under the Building Act – have brought Producer Statements under the microscope.

Producer Statements are frequently relied upon by Building Consent Authorities (BCAs) to establish reasonable grounds to grant Building Consents or Code Compliance Certificates, but the extent to which they can be relied on, the liability they attract, and other aspects of these crucial documents are often points of argument in building defect litigation.

This paper will provide some key learnings and observations from the perspectives of both a structural engineer – who authors Producer Statements (PS1, PS2 and PS4), carries out regulatory reviews for a BCA (which influences the acceptance or rejection of these statements) and regularly participates in legal proceedings as an expert witness where Producer Statements play a crucial role in the evidence and arguments presented by all parties involved – and a litigator with significant experience and interest in the role that Producer Statements play.

INTRODUCTION

As defined in the Building Act 1991, a producer statement was:

> *any statement supplied by or on behalf of an applicant for a building consent or by or on behalf of a person who has been granted a building consent that certain work will be or has been carried out in accordance with certain technical specifications.*

Under the Building Act 1991, a territorial authority could, at its discretion, accept a producer statement as establishing compliance with any (or all) provisions of the building code. Such references were removed from the Building Act 2004, and yet structural engineers in particular have continued to issue producer statements, and rightly or wrongly, territorial authorities have continued to rely on them. A producer statement is now generally recognized as:

> *a professional opinion based on sound judgement and specialist expertise*

The Engineering New Zealand website (https://www.engineeringnz.org/engineer-tools/engineering-documents/producer-statements/) states that:

> *Producer statements give authorities confidence that building work will be or has been constructed to meet the Building Code and approved consent requirements.*

As such, producer statements have played an important role in the evidence and arguments presented in many disputes relating to defective design and building work. More often than not, these disputes are settled in alternative dispute resolution processes and do not proceed to a trial, which may establish legal precedent. As a result, many aspects of the reliance on producer statements have not been clarified in law, however the Court of Appeal has recently confirmed that issuing a producer statement confirming that building work complies with the building consent can give rise to criminal liability under section 40 of the Building Act 2004 when that work is found not to be compliant with the consent.

TERRITORIAL AUTHORITY RELIANCE

Producer statements were originally introduced for the explicit purpose of allowing a territorial authority to establish compliance with the building code, typically and of relevance in this forum, for clause B1 – structure. With the removal of any reference to producer statements from the Building Act 2004, that purpose was made somewhat less clear.

The current situation sees different territorial authorities throughout the country treating producer statements quite differently.

Auckland Council require engineers to apply to become a producer statement author, including submission of documents such as references from peers, in some respects duplicating the work that is required to either gain or retain Chartered Professional Engineer (CPEng) registration. Auckland Council will only accept producer statements from approved authors who are on their register. At the other extreme, we know that many smaller territorial authorities, who don't have the resources of the larger organisations, simply accept producer statements from a CPEng at face value. Other territorial authorities will accept producer statements but undertake their own reviews, with different levels of detail and robustness.

At the very least Authors of producer statements should expect territorial authorities to apply the standards set out by the Courts, to ensure independence from the project on the part of the Engineer, and that the Engineer is practicing within their expertise *(Body Corporate 326421* v *Auckland Council,* at [115]). However, in order to prevent fraud, we are seeing some Council's take extra steps to verify authorship by contacting the CPEng signatory.

Territorial authorities should be able to accept and rely on a producer statement from a CPEng, but we know from experience, both in reviewing applications for building consent on behalf of a territorial authority, and from providing litigation support in defective building claims, that all too often producer statements are simply not worth the paper they're written on.

The question is, should that be a territorial authority's problem? The answer, unfortunately for ratepayers, is that 9 times out of 10 it becomes the territorial authority's problem, and worse, it becomes the territorial authority's burden to make it the engineer's problem.

COMMON MISTAKES AND ISSUES

From experience in litigating defective building claims and reviewing applications for building consent there are several issues that arise from producer statements, aside from the design or construction work that they support being flawed, that come up regularly and should be avoided.

Know what you're signing

A fundamental requirement of signing a producer statement is that the CPEng who is signing the producer statement must be familiar with the project and the work that has been done. The signatory must be confident that the work has been carried out competently and that there have been appropriate checks and balances to ensure that design or construction work that they are signing off as compliant with the building code or building consent is actually compliant.

Whether it be a producer statement for design, design review, or construction review, you must have reasonable grounds for signing a statement that the work either will comply with the building code or does comply with the building consent.

But it wasn't my scope…

In one case, a design for an industrial building was produced by Company A. Company A did not employ a CPEng and so engaged Company B to provide the producer statement for design (PS1) sign-off by a CPEng. Company C was engaged to peer review the design and provide a producer statement design review (PS2).

During the construction stage, concerns were raised by the steel fabricator, and a further, independent structural engineer. Company D was engaged by the owner to provide yet another review of the design. Company D identified a number of significant design issues, and through a collaborative peer review process with Company A, identified remedial and strengthening works that would be required to ensure the building would comply with clause B1 of the building code.

Proceedings were filed by the owner against Company A, Company B and Company C.

The Director of Company B had signed the Producer Statement PS1 – Design for the building but had not reviewed the calculations or drawings produced by Company A, because he understood that the design would be peer reviewed and a Producer Statement PS2 – Design Review issued. That happened (at least, a PS2 was signed), however, as noted above, there were still significant issues with the design. Company C maintained that its scope was only to provide the producer statement for design.

The Director of Company B should not have signed a producer statement without carrying out the necessary due diligence to ensure that he had reasonable grounds to do so.

This is a case in which a territorial authority found itself faced with two producer statements (design and design review) for the same design that were both plainly wrong. The building would not have complied with the building code had it been built in accordance with the design, yet two separate CPEngs had signed statements indicating that it would.

While this is an extraordinary situation with a building that had both a PS1 and a PS2 being flawed, the key lesson for producer statement authors is to make sure that you have reasonable grounds before signing.

Don't take my word for it…

In another case, a residential dwelling had suffered from some significant issues in the early part of its construction, all due to poor workmanship on the part of the builder. One such issue was that the foundation slab had been poured approximately 600mm too low.

A remedial approach was devised and details provided by both the architect and the engineer. From an engineering perspective, the solution included drilling and epoxying starter bars into

the already poured slab to accept new concrete masonry foundation walls to raise the slab level.

The engineer did not monitor the installation of the new post-installed starter bars but was asked to sign a producer statement for the construction review of that aspect of the work. Rather than declining or requiring the builder to carry out some destructive work to prove that the bars had been properly installed, the engineer elected to sign a PS4 on the basis of a brief statement from the builder (who had already established a track record of poor work on the project) and a photograph of a cartridge of epoxy.

This building also became the subject of legal proceedings. It was later proven that most or all the bars were imbedded less than 50% of the depth specified by the engineer and the holes contained insufficient epoxy to be effective, in some cases there was no epoxy in the holes.

This again highlights the need for producer statement authors to ensure that there are reasonable grounds before signing.

A useful litmus test may be to ask oneself, how would I feel if I had to give my reason for signing this producer statement in court, in defense of a claim against me?

Be accurate

It is important to be accurate in the preparation of a producer statement, and if the signatory has not prepared it themselves, they should review it for accuracy before signing.

For example, if a design is carried out to a standard that is not referenced in B1/VM1 – and a number of structural systems are not, then VM1 should not be listed as the (sole) means of compliance. If a design is, either wholly or partly, an alternative solution, then that should be noted.

It is therefore incumbent on engineers signing producer statements to know what the means of compliance is for what they are signing off, and to know if all the standards used are referenced by the means of compliance listed. Territorial authorities should also be aware of standards that are referenced in VM1 and should be pushing back on engineers if unreferenced standards are listed on producer statements, or if structural systems not covered are included.

It is also important to be accurate in defining what you are taking responsibility for. Be clear in noting what is included in the sign off and what is not, if that is appropriate. For example, in signing off the design or manufacture of precast concrete building components, a producer statement author may choose to clarify that they bear no responsibility for the structure surrounding and encompassing the precast units, such as a topping slab/diaphragm over prestressed concrete flooring units.

When signing a Producer Statement PS4 – Construction Review, ensure that the selected Construction Monitoring (CM) level is aligned with the construction monitoring that has been undertaken. This should be supported by attaching the relevant site records, but in any case, selecting CM4 on the PS4 and signing it on that basis when CM2 (or any other CM level that is not CM4) inspections may have been undertaken is not advisable. We are aware of a situation where carelessness in selecting the CM level on a PS4, combined with site reports that left room for interpretation in relation to the scope, may have resulted in an engineers inclusion in proceedings that may otherwise have been avoided.

Further, when signing a PS4 where another engineer has carried out the construction monitoring (including a junior engineer from your own firm) you should turn your mind to whether the reports provide you with the confidence to conclude that that the building work complies with the building code. We suggest that it is not sufficient to simply rely on the fact that another engineer has carried out construction monitoring.

LIMITATION OF LIABILITY

When signing a producer statement, an engineer would generally assume that they are covered by the limitation of liability contained in the fine print at the end of the producer statement, typically $200,000. The reasoning is that this limit of liability would be in addition to any liability to the engineer's client, and that by accepting the producer statement, the territorial authority is therefore agreeing to that limit of liability.

Limitation of liability, both contractually and within the fine print of a producer statement has been the subject of proceedings filed in the High Court. The proceedings arise in relation to the Harrington Street Transport Hub in Tauranga (*Tauranga City Council* v *Harrison Grierson Holdings Limited*).

The High Court's decision in this matter has been appealed to the Court of Appeal. The hearing in the Court of Appeal is set down for early next year. While some comment from a senior court on the issue will be well received, it is unlikely to resolve the matter outright.

Given Tauranga City Council's involvement in the proceedings is as the asset owner, and not the building consent authority, there is a risk that any judgment from the Court of Appeal does not deal with the limitation issues where there are contribution arguments in play in typical defective building litigation.

The reasoning to counter the applicability of the limitation of liability on a producer statement considers a theoretical case where a territorial authority is sued in relation to a defective building – that may have been defectively designed, defectively constructed, or some combination thereof – where the original owner (who engaged the engineer) is no longer in existence or able to be sued. If the limit of liability were to hold in that case, then the territorial authority (and by extension, rate payers) may be left bearing a substantial portion of the cost

of remediating the defective building, including costs which reasonably should have been borne by the engineer (or their insurer) if they had in fact been negligent in carrying out their work.

More generally, concerning liability we note that the Court of Appeal has recently concluded that issuing a PS4 is building work (*Solicitor-General's Reference (No 1 of 2022))* for the purposes of the Building Act 2004. What this means practically is that the issuer of a PS4 could be liable criminally under section 40 of the Building Act. Section 40 creates an offence where building work is carried out otherwise than in accordance with a building consent. When making an assessment whether a PS4 was issued not in accordance with a building consent, the key question seems to be whether the signatory carried out construction monitoring to the required standard and has reasonable grounds to support the statements made in the PS4.

Lastly, litigation is expensive for everyone involved. Taking the time to clearly establish the parameters within which work is to be completed can save money and time. As an example, we are aware of proceedings where a civil CPEng completed work which would typically be associated with the geotechnical discipline. While the general consensus was that the civil engineer was able to complete the design work, as they were working within their competence in accordance with the CPEng Rules , we consider if the engineer had included the reasons for which they thought they could carry out this work as supporting information in their design package, that may have saved this particular engineer a lot of time and money in responding to the dispute post construction.

CONCLUSION

Despite not being recognised in law, producer statements and their widespread use are likely to continue, and indeed they play a valuable role in the building industry. They are therefore also likely to continue to play an important role in the evidence and arguments presented in building disputes.

Preventing disputes will always be the cheaper and less time-consuming option. In order to do this effectively an engineer needs to carefully understand the scope of their role and the value that will be placed on the producer statement from project to project.

Spending time ensuring accuracy and understanding the audience of the producer statement (and accompanying package) will also save time, not just in terms of disputes after the fact but by reducing the need for Requests for Information from consenting authorities. Not approaching producer statements with the necessary care and attention to detail has the potential to land an engineer in a dispute, even if they have carried out the bulk of their work diligently and accurately.

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Seismic Performance of Steel Frames with Buckling Restrained Braces, Friction Braces, and Braces with Resilient Slip Friction Joint dampers

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ABSTRACT

Recent earthquakes have underscored the necessity for resilient structural systems that minimize permanent damage. This shift in approach has led to the exploration of innovative Lateral Load Resisting Systems (LLRS) that enhance energy dissipation and ductility, ensuring the protection of both structural and non-structural components. This study presents a comparative analysis of three advanced bracing systems: Buckling Restrained Braces (BRBs), pure Friction Braces (FB), and Resilient Slip Friction Joints (RSFJ), implemented in 4-story and 8-story steel braced frames designed in accordance with NZS1170.5. Numerical models of these structures were developed in ETABS, and their seismic performance was evaluated using Nonlinear Static Pushover (NSP) and verified through Nonlinear Time History Analysis (NLTHA) under Design Level Earthquake. The numerical results indicate that FB and RSFJ systems can achieve a ductility factor of 3.0, comparable to BRBs while exhibiting reduced inter-story drifts. Furthermore, FB and RSFJ systems demonstrated potential for higher ductility factors ranging from 3.0 to 3.2 with code-compliant drifts under all design-level seismic demands. The RSFJ system, in particular, displayed inherent self-centering capability, ensuring reduced residual drifts. The findings highlight that while all three systems provide robust seismic protection, their selection should be based on specific project requirements, including post-event functionality, architectural constraints, and economic considerations.

INTRODUCTION

Recent major earthquakes have highlighted the urgent need for structural systems that not only ensure life safety but also minimize damage, reducing economic losses and post-earthquake downtime. Conventional seismic design approaches often allow for significant inelastic deformations in structural components, resulting in costly repairs and extended recovery periods. This has driven the development of advanced Lateral Load Resisting Systems (LLRS) that enhance energy dissipation and structural ductility, improving both immediate safety and long-term resilience. Among these, innovative bracing systems have gained prominence, offering improved seismic performance by efficiently controlling structural response and dissipating earthquake-induced energy.

Seismic dampers, functioning similarly to shock absorbers in vehicles, are crucial in protecting structures by dissipating energy during intense ground motions. The fundamental principle of damping involves reducing oscillatory motion by converting kinetic energy into heat, thereby mitigating excessive structural displacements (Crandall, 1970). During seismic events, these dampers undergo controlled deformation or frictional sliding to limit the force transmitted to the primary structure. The behavior of such dampers is typically represented by hysteretic load-deformation relationships (refer to Figure 1), where inelastic deformations beyond predefined thresholds contribute to energy dissipation. Unlike traditional seismic design, which relies on plastic hinging in structural elements, modern approaches increasingly incorporate damping devices to concentrate energy dissipation in replaceable or repairable components. Experimental studies indicate that these devices can dissipate over 90% of the total seismic energy imparted to a structure (Soong & Dargush, 1999).

INNOVATIVE LLRS FOR STEEL FRAME STRUCTURES

Buckling Restrained Braces (BRBs)

Buckling-Restrained Braces (BRBs) have emerged as a highly effective seismic-resistant bracing system, providing significant improvements in energy dissipation and stability. Unlike conventional bracing, BRBs are designed to resist both tensile and compressive forces without experiencing local or global buckling, ensuring a symmetric and predictable hysteretic response (Fahnestock et al., 2007). This behavior enhances structural resilience by maintaining energy dissipation capabilities throughout multiple loading cycles, reducing the likelihood of strength and stiffness degradation. Over the years, BRB technology has evolved, with designs ranging from concrete-filled steel tubes to fully steel-based configurations, further optimizing their performance (Hoveidae & Radpour, 2021). Research has demonstrated that BRBs can dissipate more than 75% of seismic input energy while maintaining inter-story drifts within acceptable limits (Tan et al., 2021). However, a primary limitation of BRBs is the potential for significant residual deformations following severe earthquakes. These large permanent displacements may complicate post-earthquake repair and structural rehabilitation, posing challenges for long-term functionality (Chen et al., 2019).

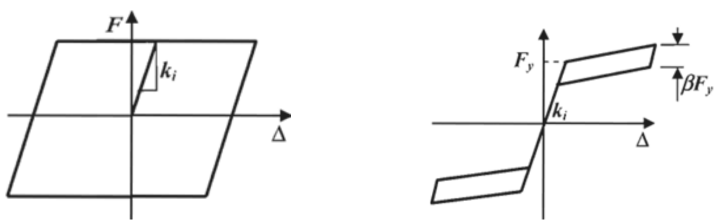
Resilient Slip Friction Joint (RSFJs)

Resilient Slip Friction Joints (RSFJs) are an advanced type of passive damper that combines effective energy dissipation with self-centering capability, addressing one of the main drawbacks of traditional BRBs residual drift. These joints rely on frictional sliding between plates coated with a proprietary material, with ridged sliding surfaces and disc springs facilitating self-centering behavior. When the applied force exceeds the frictional resistance, the plates slide, dissipating energy, while the disc springs restore the system to its original position. A key advantage of RSFJs is their ability to accommodate up to 1.75–2 times their displacement capacity due to a built-in secondary fuse mechanism, ensuring structural integrity even under extreme seismic demands. Additionally, they exhibit a reserve force capacity of approximately 1.35 beyond the design displacement, attributed to the strain hardening of bolts after yielding (Hashemi et al., 2018; Hashemi, Zarnani, et al., 2020). By offering stable energy dissipation and reliable re-centering capability, RSFJs mitigate permanent structural deformations, enhancing both safety and post-event functionality.

Friction Dampers (FD)

Friction dampers (FDs) represent an efficient passive seismic energy dissipation mechanism that utilizes the principle of solid friction to control structural response. These devices dissipate energy through relative sliding between metallic surfaces under normal pressure, converting kinetic energy into heat. Typically installed at brace-to-frame connections or within bracing members, FDs activate upon lateral deformation of the structure, thereby reducing peak forces transmitted to primary load-bearing elements. Unlike conventional braces, which dissipate energy through plastic deformation, friction dampers provide a stable elasto-plastic (EPP) hysteretic response without causing permanent damage to structural members (Calvi et al., 2007). This characteristic allows for easier post-earthquake inspection and maintenance, as friction dampers can be retightened or replaced without requiring extensive structural repairs. Their predictable and repeatable energy dissipation behavior makes them a highly attractive solution for seismic applications, particularly in structures requiring enhanced resilience and rapid post-earthquake recovery.

This study examines the seismic performance of 4-story and 8-story steel-framed structures by incorporating three different types of lateral load-resisting systems (LLRS) in the transverse direction: Buckling Restrained Braces (BRBs), Concentrically Braced Frames (CBFs) with Resilient Slip Friction Joints (RSFJs), and Concentrically Braced Frames (CBFs) with Friction Dampers (FD). The load-deformation curves for BRBs and CBFs with Friction Dampers (FD) are illustrated in Figure 1 (a), while Figure 1 (b) shows the flag-shaped load-deformation response of the CBFs with RSFJs. In the longitudinal direction, the LLRS consists of Steel Moment Resisting Frames (SMRF). The seismic performance is evaluated in accordance with the provisions outlined in New Zealand's Earthquake Design code (NZS1170.5:, 2004) under the Design Level Earthquake (DLE).



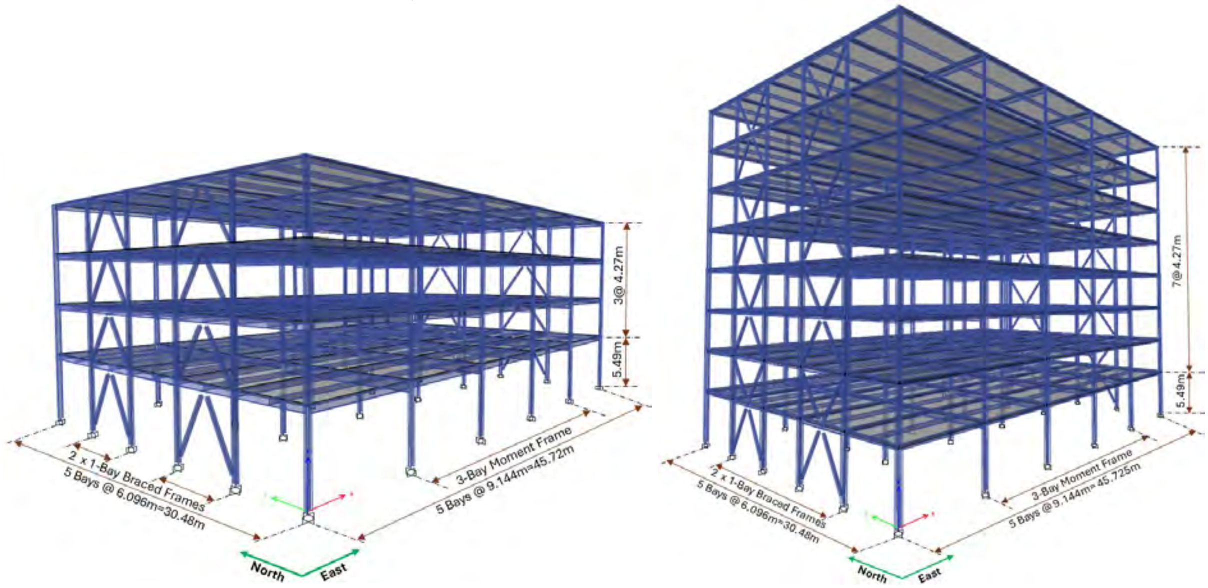
(a) (b)

Figure 1. Load-deformation (Hysteresis) curve: (a) Elasto-plastic (EPP), and (b) Flag-shape (FS) (Calvi et al., 2007).

METHODOLOGY

Overview of 4-storey and 8-storey Steel Framed Building

This study investigates the seismic performance of two steel-framed office buildings, one with four (4) stories and the other with eight (8) stories (refer to Figure 2), based on archetypes adopted from NIST TN 1863-4 (Speicher & Harris, 2019). Advanced Lateral Load Resisting Systems (LLRS), including Buckling Restrained Braces (BRBs), CBFs with Resilient Slip Friction Joints (RSFJs), and Friction Dampers, are applied in the transverse (N-S) direction to assess seismic performance in terms of base shear, drift, and acceleration. LLRS are symmetrically placed along the building perimeter, while Special Moment Resisting Frames (SMRFs) provide longitudinal (E-W) stability. The seismic performance evaluation of SMRFs is outside the scope of this study.



(a) (b)

Figure 2. Case study archetypes: (a) 4-story structure and (b) 8- story structure.

Both buildings have rectangular floor plans measuring 45.72 m (E-W) by 30.48 m (N-S), with five 9.144 m bays and five 6.096 m bays in each direction. The first story is 5.49 m high, and the upper stories are 4.27 m tall. The 4-storey structure uses chevron bracing, while the 8-storey uses X-bracing (see Figure 2). Table 1 presents the roof and floor loads. Total seismic weights are 22,864 kN for the 4-storey and 46,862 kN for the 8-storey building. For modeling simplicity, both buildings exclude elevators and stairwell diaphragm openings.

Table 1. Gravity loads on respective levels of the buildings

Load	Type	Magnitude (kPa)
Dead, D	Dead	2.20
Floor Superimposed Dead, SD	Dead	0.72
Roof Superimposed Dead, SD	Dead	0.48
Façade Dead (Curtain Wall), SD	Dead	3.65
Unreduced Design Floor Live, Lo	Floor Live	2.40
Unreduced Design Roof Live, Lo	Roof Live	1.44

Seismic Analysis

Numerical models for the 4-storey and 8-storey steel-framed buildings were developed in ETABS (Computers and Structures Inc, 2021) to evaluate the seismic performance of various bracing systems. Both Nonlinear Static Pushover (NSP) analysis and the Capacity Spectrum Method (CSM) were performed, with additional verification through Nonlinear Time History Analysis (NLTHA) to capture response variability. The hysteretic behavior of Friction Dampers and Resilient Slip Friction Joints (RSFJs) was simulated using "Plastic Wen" and "Damper - Friction Spring" link elements in ETABS. The modeling of RSFJs as friction springs has been validated by several studies (Assadi et al., 2023; Hashemi, Bagheri, et al., 2020; Hashemi, Zarnani, et al., 2020). The buildings are assumed to be located on Soil Class D in Christchurch and classified as Importance Level 2, with an annual ULS exceedance probability of 1/500. A near-fault factor of 1.0 was applied. Drift limits were set at 1.5% for ULS and 0.33% for SLS (annual probability of 1/25). A structural performance factor (Sp) of 0.85 was conservatively used, consistent with the typical range of 0.7 and 1.0. Table 2 summarizes the key characteristics of the modeled buildings.

Table 2. Overview of the case study buildings

No.	Case Study	LLRS Configuration	No. of connections	Direction
1		BRB	32	N-S
2	4 story-steel Frame Structure	RSFJ	32	N-S
3		FD	32	N-S
4		BRB	64	N-S
5	8 story-steel Frame Structure	RSFJ	64	N-S
6		FD	64	N-S

Selection of ground motion records and scaling

To evaluate the seismic performance of the case study building archetypes, nine ground motion records (comprising 18 horizontal components) were selected to represent Zone South C conditions specific to the South Island of New Zealand (Burlotos et al., 2022). These records were scaled according to New Zealands seismic design guide (NZS1170.5:, 2004) to ensure

alignment with local seismic design standards. Nonlinear Time History Analysis (Direct Integration) was performed on the six case study structures to assess the performance of various Lateral Load Resisting Systems (LLRS). The comparative evaluation was based on key response parameters, including base shear, inter-story drift, residual drift, and floor acceleration. Details of the selected ground motions are presented in Table 3.

Table 3. Selected ground motion records for NLTHA

Event	Country	Year	M	Fault Mechanism
ChiChi-TCU051	Taiwan	1999	7.6	Reverse Oblique
Christchurch-CBGS	New Zealand	2011	6.2	Reverse oblique
Christchurch-RicartonHschool	New Zealand	2011	6.2	Reverse oblique
ImperialValley-02	USA	1940	6.9	Strike-slip
ImperialValley-06	USA	1979	6.5	Strike-slip
ImperialValley-ElCentroArray#9	USA	1979	6.5	Strike-slip
Kobe-PortIsland	Japan	1995	6.9	Strike-slip
Valparaiso-Llolleo	Chile	1985	7.8	Reverse/Thrust
Victoria-Chihuahua	Mexico	1980	6.3	Strike-slip

Results and Discussion

Capacity Spectrum Method (CSM)

The Coefficient Method (CSM) was initially standardized in FEMA 356 and ATC 40 and later refined in FEMA 440. In New Zealand, a simplified approach to seismic assessment is outlined in C2: Assessment Procedures and Analysis Techniques by the Ministry of Business, Innovation, and Employment (2018). CSM involves plotting the backbone curve from a nonlinear pushover analysis (NPA) against the modified Acceleration-Displacement Response Spectrum (ADRS), adjusted for 5% damping for both Serviceability Limit State (SLS) and Ultimate Limit State (ULS). An optimal system should position its minimum yield point above the SLS ADRS curve and intersect the ULS ADRS curve at force and displacement levels beyond the design values. According to FEMA 440 and C2, higher mode effects are negligible if the square root of the sum of the squares (SRSS) of shear in any storey, considering modes contributing at least 90% of the mass, does not exceed 130% of the first mode-only storey shear (Assadi et al., 2023).

Figures 3 and 4 present the Acceleration Displacement Response Spectrum (ADRS) curves showing performance points for three Lateral Load Resisting Systems (LLRS); Buckling Restrained Braces (BRBs), Resilient Slip Friction Joints (RSFJs), and Friction Dampers (FDs) in a 4-storey and 8-storey steel-framed structures along the N-S direction. Analyzed at 1.5% drift, the performance points, determined via the Capacity Spectrum Method (CSM), represent the intersection of the pushover curve and damped spectra. For the 4-storey structure, base shears were 4235.6 kN, 4535 kN, and 3087 kN for BRB, RSFJ, and FD systems, respectively, with inter-storey drifts of 0.9%, 1.1%, and 1.25%. The 8-storey structure exhibited base shears

of 4900 kN, 5564 kN, and 3800 kN, with inter-storey drifts ranging from of 0.9%, 1.0%, and 0.95% for BRBs, RSFJs and FD, respectively.

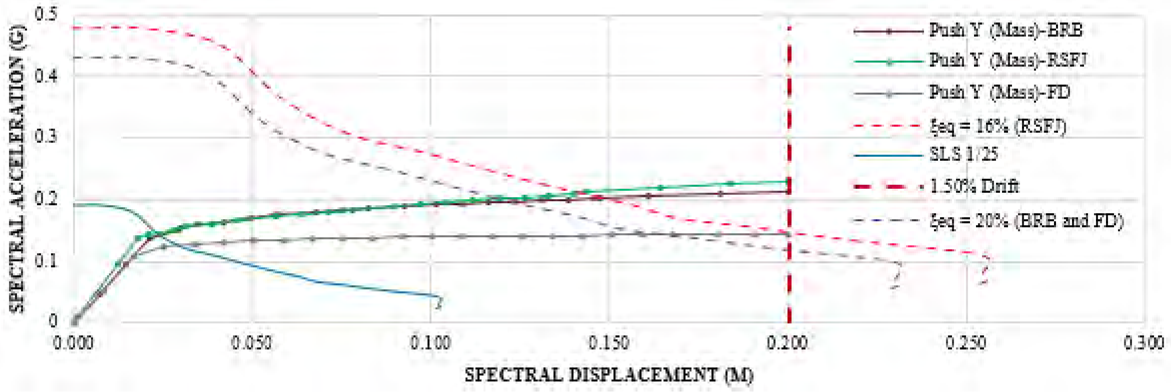


Figure 3. ADRS Performance point for 4-storey LLRS configuration with BRB, RSFJ, and FD _ULS

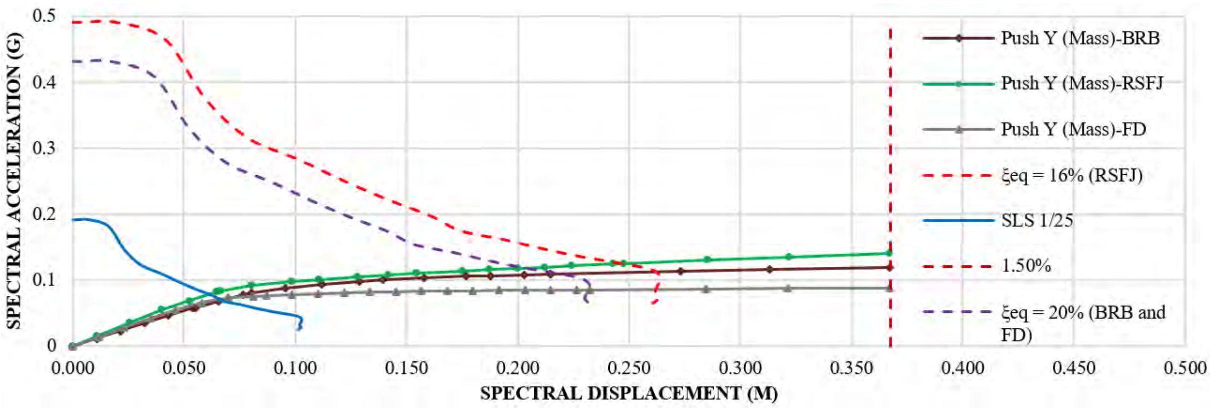
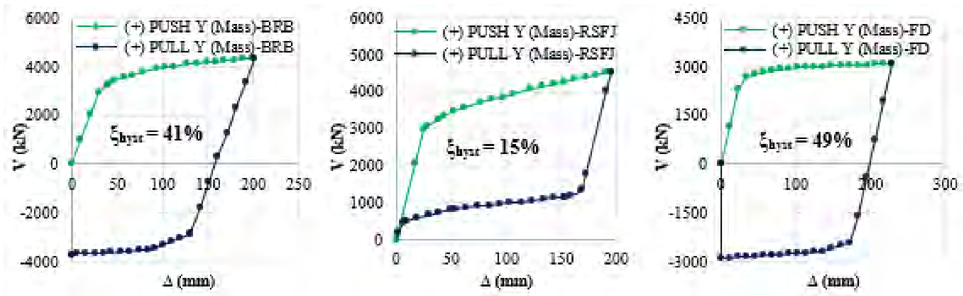


Figure 4. ADRS Performance point for 8-storey LLRS configuration with BRB, RSFJ, and FD _ULS

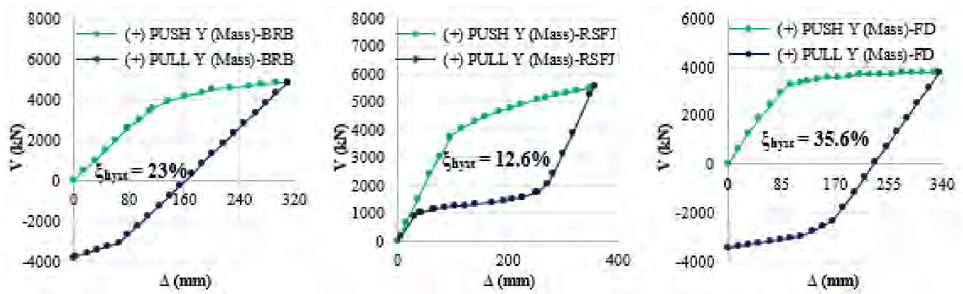
Nonlinear Static Pushover (NSP) analysis

Nonlinear Static Pushover (NSP) curves are plotted at drift levels obtained from ADRS, with area-based damping calculated using Jacobsen's method (Chan et al., 2021). NSP identifies potential hinge formations and defines the backbone curve, representing the lateral force-displacement relationship under cyclic seismic loading. The initial elastic response transitions to inelastic behavior at the yield point, indicating plastic deformation. Figures 5 and 6 show half-cycle NSP curves for BRBs, RSFJs, and FDs in the N–S direction of 4-storey and 8-storey steel-framed structures. The FD system exhibits the highest energy dissipation, with hysteresis damping ($\xi_{hys}t$) of 49% and 35.6% for the four storey and the 8-storey archetypes, respectively. BRBs demonstrate moderate performance, with damping ($\xi_{hys}t$) decreasing from 41% to 23% as building height increases. RSFJs show the lowest hysteresis damping (15% and 12.6%) but maintain full self-centering capability and post-earthquake functionality. All systems exhibit reduced energy dissipation in the 8-storey structure, highlighting the impact of building height on seismic performance. This suggests that additional design strategies may be required for taller buildings to ensure equivalent energy dissipation and seismic resilience (see Figures 5a–c and Figures 6a–c).



(a) (b) (c)

Figure 5. NSP cyclic curves for 4-storey LLRS : (a) BRB, (b) RSFJ, and (c) FD_ULS



(a) (b) (c)

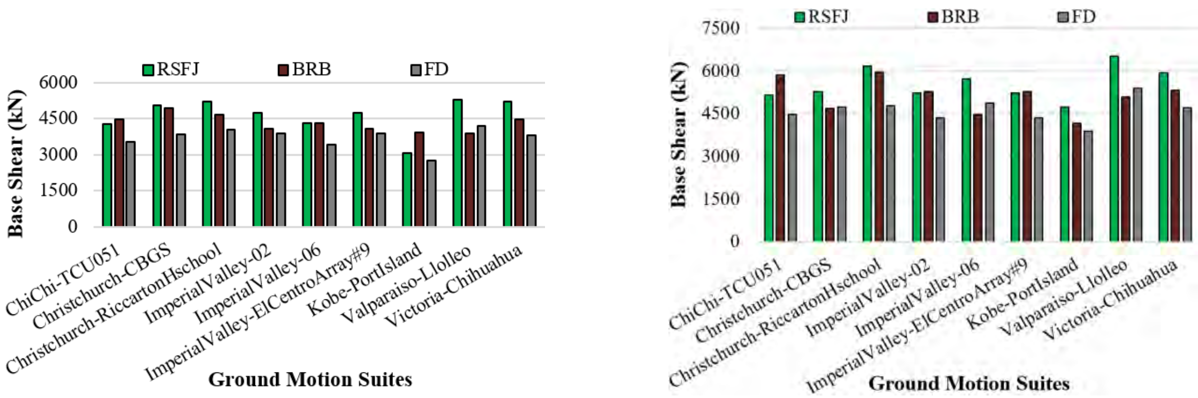
Figure 6. NSP cyclic curves for 8-storey LLRS : (a) BRB, (b) RSFJ, and (c) FD_ULS

Nonlinear Time History Analysis (NLTHA)

Non-linear Time History Analysis (NLTHA) was performed to assess the seismic performance of the two buildings with different lateral load-resisting systems (LLRS) subjected to ground motions scaled to the Design Level Earthquake (DLE). A Rayleigh damping model with 2% inherent damping was applied to both the first mode and the mode, contributing 90% mass participation. Key seismic response parameters such as base shear, drift, and floor acceleration were evaluated using the mean of nine ground motion records, as recommended in response history analysis guidelines (Bradley, 2014). While the New Zealand seismic design code (NZS1170.5:, 2004) permits analysis with as few as three records, this study uses nine to capture the variability in structural response better, enhancing the robustness of the analytical results within the study's scope.

Base Shear

The Non-Linear Time History Analysis (NLTHA) results for the 4-storey and 8-storey steel-framed structures reveal significant variations in base shear demands among lateral load-resisting systems (RSFJ, BRB, and FD). For the 4-storey structure, RSFJ exhibited the highest base shear demand at 4655 kN, 7.3% higher than BRB with 4316 kN and 20.3% higher than FD, which yielded 3708 kN (see Figure 7a). A similar trend was observed in the 8-storey structure, where RSFJ had the highest average base shear of 5554 kN, exceeding BRB by 7.8%, which yielded 5123 kN, and FD by 16.9%, which exhibited 4616 kN (see Figure 7b). FD consistently showed the lowest base shear demands, with reductions up to 20.3% and 16.9% in the 4-storey and 8-storey structures, respectively, compared to RSFJ. FD demonstrated superior force reduction, making it a more efficient option for minimizing seismic forces.

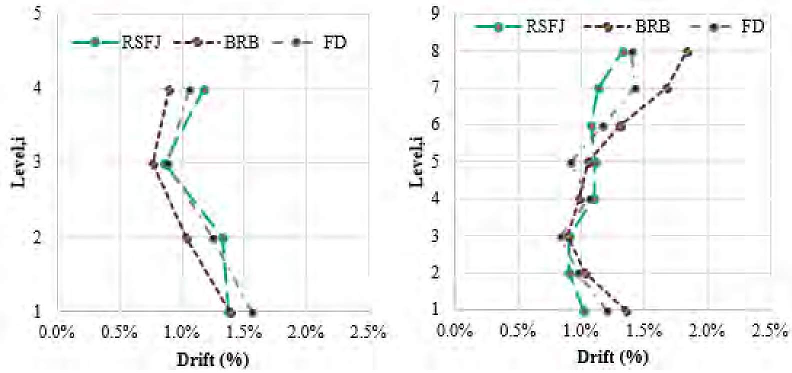


(a) (b)

Figure 7. Comparison of ULS Base Shear in N-S direction: 4-Storey and 8 -Storey archetype.

Inter-story drift

Drift distribution analysis of the 4-story and 8-story steel-framed structures shows distinct behavior across the lateral load-resisting systems. In the 4-storey structure, FD exhibits the highest drift at Level 1 of 1.6%, while RSFJ and BRB show similar drifts at 1.4% . At upper levels, drift converges to 0.8–0.9%. The average drift is similar for RSFJ and FD, exhibiting 1.2%, while BRB has a slightly lower drift of 1.0% (see Figure 8a). For the 8-storey structure, BRB experiences the highest drift at upper levels of 1.8%, while RSFJ and FD maintain lower drifts. BRB exhibited the highest average drift of 1.3%, and FD showed the most consistent performance with maximum average drifts capped at 1.1% similar ot RSFJs(see Figure 8b).

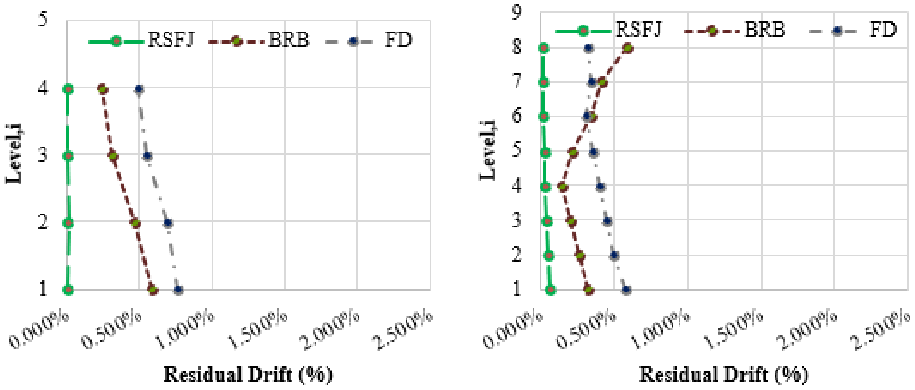


(a) (b)

Figure 8. Comparison of Inter-storey drift in N-S direction: (a) 4-Storey archetype, and (b) 8-Storey archetype_ULS

Residual drift

The comparative analysis of three structural systems in 4-story and 8-story buildings shows consistent performance trends. The RSFJ system demonstrates superior performance with near-zero residual drifts (approximately 0.000%) at all floor levels in both structures (see Figures 9a and 9b). The BRB system shows moderate performance, with residual drifts ranging from 0.250-0.500% in the 4-story building and increasing to 0.600-0.700% in the 8-story structure. The FD system consistently exhibits the highest residual drift, reaching 0.750% at Level 1 in the 4-story building and 0.500-0.750% in the 8-story structure Figures 9a and 9b). While all systems control residual drifts, the RSFJ system provides the best self-centering capability, reducing post-earthquake repair costs, minimizing downtime, and enhancing resilience by enabling immediate occupancy after major seismic events.

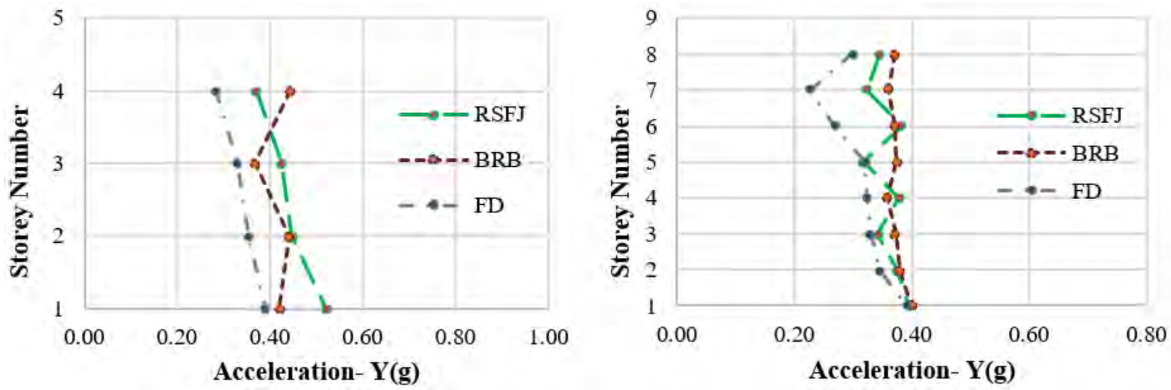


(a) (b)

Figure 9. Comparison of residual drift in N-S direction: (a) 4-Storey archetype, and (b) 8-Storey archetype_ULS

Floor Acceleration

The primary concern for any structure is ensuring human safety, as high-floor accelerations can lead to discomfort, injuries, or fatalities. Keeping acceleration low during seismic events reduces the risk to occupants. In the 4-storey structure, RSFJ recorded the highest acceleration at Level 1 (0.52g), followed by BRB (0.42g), while FD exhibited the lowest (0.39g) at level 1, reducing acceleration by 25% compared to RSFJ. This trend continued across the structure, with RSFJ consistently showing higher accelerations Figures 10a and 10b). For the 8-storey structure, acceleration distribution was more uniform at upper levels, with FD maintaining the lowest accelerations in lower and middle stories. These results suggest that FD provides the best acceleration reduction, particularly in lower stories, benefiting sensitive non-structural components and equipment (see Figures 10a and 10b).



(a) (b)

Figure 10. Comparison of floor acceleration in N-S direction: (a) 4-Storey archetype, and (b) 8-Storey archetype-ULS

Conclusion

This study demonstrates the effectiveness of implementing three advanced Lateral Load Resisting Systems (LLRS); Buckling Restrained Braces (BRBs), Resilient Slip Friction Joints (RSFJs), and Friction Dampers (FDs) in 4-storey and 8-storey steel braced frame structures. These systems significantly reduce force demands while maintaining structural integrity and minimizing damage. RSFJs and FDs achieved equivalent ductility factors of 3.1 and 3.7 in the 4-storey archetype and 2.8 and 3.8 in the 8-storey archetype, respectively, outperforming BRBs, which yielded 2.7 and 2.6. This highlights the robustness of FDs in energy dissipation and RSFJs in controlled, resilient response across varying building heights. As building height increases, a general reduction in equivalent ductility is observed, but the inherent calibrating properties of RSFJs and FDs offer flexibility for optimizing designs to meet project-specific

requirements. The results also demonstrated that all LLRS configurations achieved comparable inter-story drifts ranging from 1.0% to 1.3%, effectively minimizing damage to non-structural components such as façades and mechanical systems. Floor accelerations were similar across all configurations, underscoring their ability to control seismic response.

The Capacity Spectrum Method (CSM) was validated as an efficient seismic evaluation tool, with results aligning closely with Nonlinear Time History Analysis (NLTHA). While BRBs and FDs comply with New Zealand seismic codes and demonstrate favorable energy dissipation, their low post-yield stiffness can lead to residual drifts, especially in the 4-storey archetype. These residual displacements increased in the 8-storey structure, indicating the need for additional Moment Resisting Frames (MRFs) to restore post-earthquake functionality. In contrast, RSFJs exhibited superior self-centering performance with negligible residual drifts, thus requiring minimal maintenance post-event and supporting rapid re-occupancy. In conclusion, while all LLRS systems studied improve seismic performance, reduce force demands, and enhance ductility, the optimal selection must consider specific project needs such as residual drift tolerance, self-centering behavior, cost, and maintainability. Strategically integrating the most suitable system can maximize resilience and safety, contributing to the development of high-performing, repairable, and earthquake-resilient steel structures.

Acknowledgement

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Non-Structural Seismic Restraint Strategies and Procurement Options

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ABSTRACT

The design of Non-Structural Element (NSE) Seismic Restraints has gained significant attention from the New Zealand industry in recent years due to several local seismic events. Both councils and clients are demanding a higher level of design of NSE restraint at earlier stages in the project lifecycle**.** The contractor procurement process and contractual models can greatly influence what the seismic restraint engineer can deliver and at what stage in the project. These changes are in focus and have a close link to construction present challenges and questions for the structural engineer, such as the extent of changes expected when the contractor completes the services design, and the level of development required for the project model, and the complexity to achieve Code Compliance at Practical Completion.

This paper explores strategies for delivering Code Compliant NSE restraint designs in the context of different project structures and contractual arrangements. Case studies have been presented, to examine different restraint strategies and delivery methods, and compare their respective advantages and disadvantages.

Introduction

Non-Structural Elements (NSE) have been in the spotlight in recent times based on their often-poor performance in recent seismic events. However, it is well recognized by industry that there is some distance to go before the holistic building design that incorporates NSE design and documentation is delivered in harmony with how the contractor implements the NSE seismic design on-site. Industry guidance documents have recently been released to support the seismic design of NSEs; notably The Code of Practice for the Seismic Performance of Non-Structural Elements (NSE CoP) in 2024 (BRANZ / Building Innovation Partnership, 2024). As the industry begins to digest and learn from these studies and previous experiences the challenge remains to navigate through delivering compliant, coordinated NSE seismic designs while providing documentation that the contractor can accurately price, and minimizes the risk for change during construction.

Historical methodology for NSE seismic design and installation

Historically, NSE seismic design has been completed as a contractor design element. At the completion of Detailed Design, the consultant team would issue their tender drawings, following this the contractor would price the work to both complete the building services design, and design of NSE restraints. Often there would be little consideration of the NSE seismic loads to be carried by the primary structure, and little holistic design and coordination including seismic design of NSEs.

This process led to the contractor pricing the risk associated with coordinating seismic restraints during the construction phase as well as needing to assume the structure has sufficient capacity to resist the gravity and seismic NSE loads. In this scenario, there is an attempt to pass the design and coordination risk to the Contractor, but typically Contractor's limit their scope of work and the risk ends up sitting with the client. Often the outcome is unexpected variations, costs and programme delays. Additionally, this approach has often led to undesirable outcomes where compromises to compliance have to be made to resolve issues. Figure 1 shows an example of a new build structure (~2016) with non-existent NSE restraint.

Figure 1 - Example of a non-compliant NSE in a new build structure ~2016.

Recent changes to methodology for NSE seismic design and installation

As a result of recent industry guidance, along with Building Consent Authority and Client pressures, NSE seismic design is being undertaken earlier in the design process with designs being developed by the consultant team and being provided to the contractor to price and implement. This new approach aims to ensure compliant NSE seismic designs are achieved and provides the contractor with a design to price and implement. However, based on recent experience there are still a number of challenges, including:

1. Alignment of design deliverables. If the building services and architectural consultants document components by performance specification, then how can a compliant and coordinated NSE seismic design be delivered at tender?
2. Managing the expectation of change. To what level of completeness should the NSE seismic design be completed to prior to hand over to the contractor? Who bears the cost of changes during construction and how are changes undertaken in construction coordinated with the main design to ensure the design intent is not compromised and product substitution(s), where on a product c.f. product basis indicates potential cost saving, does not result in significant additional costs when considered in the wider context of the holistic building design?
3. Contractor designed elements. When the NSE seismic design is handed over to the contractor to complete, is what is being asked of them within their capability (and scope) to resolve?

This paper will investigate how different NSE restraint strategies can be implemented in a range of different project delivery models and how these impact the challenges raised above.

Restraint Strategies

Traditional restraint strategies have focused on individual bracing of components via adapting gravity supports for NSEs. While this is a sound methodology for more routine low-rise commercial style buildings with little change risk during construction, for more complex projects this can lead to unresolvable clashes. Two other methods are presented herein which can streamline NSE restraint and reduce issues onsite.

Shared Service Runs

A more holistic approach to seismic restraint is being recommended in the latest industry guidance (BRANZ / Building Innovation Partnership, 2024), which considers a Seismic Restraint Strategy combining considerations drift and acceleration sensitive components. Unlike the traditional approach the combined support model looks to support multiple services on combined support hangers. These restraint frames can support services only or incorporate ceilings and partition support also. These methods have the added benefit of being able to be constructed using offsite prefabrication and modularization for efficiencies in construction. Figure 2 illustrates an example of combined corridor support frames.

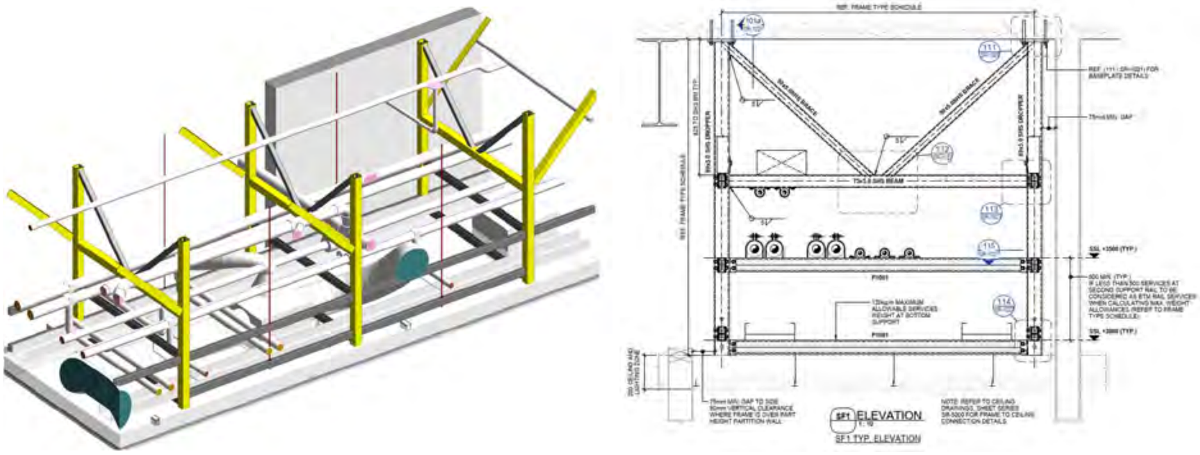

Figure 2 - Example of combined corridor restraint frame incorporating support for linear services, ceilings and partitions.

<u>Common Seismic Restraint Grid (secondary structure)</u>

As an extension to the shared service run approach another method which has been implemented is a common seismic grid. With this approach a secondary structure grid is added at a common datum above the ceiling level. This can then be used to direct-fix ceilings, services and partitions and greatly reduce the quantum of additional seismic braces. Figure 3 show an example of a common seismic grid for a commercial project.

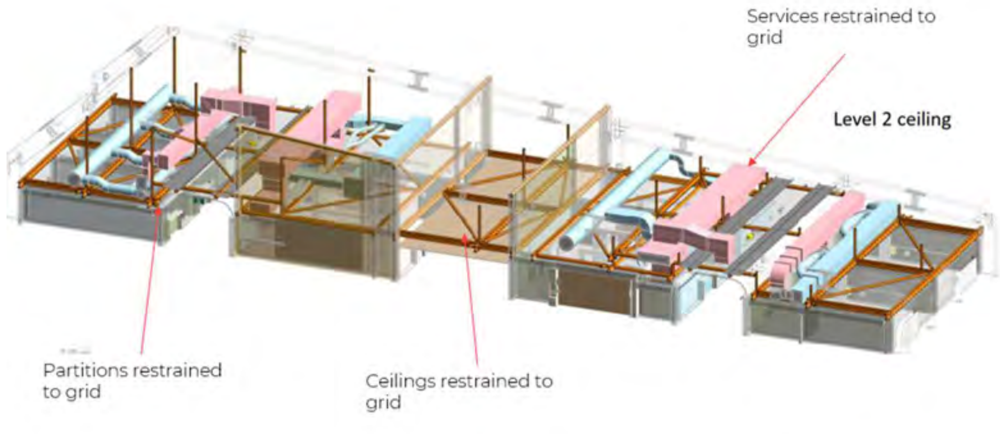

Figure 3 - Example of a common seismic grid for combined restraint of ceilings, services and partitions

Key advantages and considerations of shared support runs include:

- Common datums for fixing of services leading to a consistent seismic restraint and resulting response

- Reduced overall quantity of bracing, and single point of responsibility for seismic restraint installation
- Better able to quantify the expected movement of the services for detailing of passive fire requirements
- Requires a high level of coordination during the design phase
- Often higher material cost, which needs to be justified earlier in the project

Defining NSE Seismic Restraint Scope & Contractual Models

The success of any chosen NSE seismic strategy and delivery approach is heavily dependent on the adopted contractual model and levels of engagement with the supply chain and contractors during design. Fully coordinated designs at tender will not be possible without input from key subcontractors, and equipment selection being finalized. This means that definition of the NSE restraint scope and expectation of the completeness of deliverables is important.

In discussions with contractors in the industry, where subcontractors are able to be bought onboard early (through a D&C type contract) to provide inputs into the seismic design for NSE it has proven highly beneficial. They have been able to contribute to developing solutions which have reduced cost and simplified design to increase efficiency onsite. This flows through into reduction of clashes or construction issues on site with higher levels of coordination.

Similar benefits can be achieved through ECI type arrangements but does require the ability to engage subcontractors early. This can often be perceived as a reduction to commercial competitiveness. However, on the contrary, this can lead to increased cost certainty early, reduce overall cost and reduce the risk of constructability issues and change, which can lead to claims during the build – these benefits are worth considering as it can offset the loss of competitiveness in tendering.

The client needs to understand procurement models and construction risk prior to confirming the scope of work. Should a traditional design then construct procurement model be chosen, the extent of coordination of the NSE seismic design with the main building design must be confirmed at the start of the project. As previously described, if some elements of the architectural or services design are performance specified then it is important to recognize that there will be a requirement for updates to the design during the construction phase. An understanding of how this will be addressed from a cost and program perspective is important. Recent contractor experience highlights that having a consultant design pre-tender is effective at reducing potential issues and risks of clashes developing later in the project but does lead to varying degrees of design changes to deal with clashes or changes in products used.

The Project Structural Engineers Role

Irrespective of the procurement model chosen for each project, to successfully deliver NSE restraints the project structural engineer needs to ensure they design the structure for all of the loads the building may experience during its design life. The seismic loads resulting from NSEs needs to be included in the design of the structure. In our experience when lean structural designs are provided, particularly at the roof level, seismic loads from NSEs are not adequately considered and significant challenges arise when the structure has insufficient capacity to resolve NSE restraint reactions. This challenge is often at large cost to the client, where additional secondary steel or more onerous NSE restraints are required. These costs are not insignificant and can add high cost and programme delays to the project.

The project structural engineer needs to understand that they are responsible for ensuring their structural design can resolve all loads the building may experience.

Economics of NSE

The MBIE study into the Economics of NSE in 2017 (WSP NZ, 2017), looked into how contractors were pricing NSE restraint within their projects. These elemental cost breakdowns have been compared with reference present day projects below:

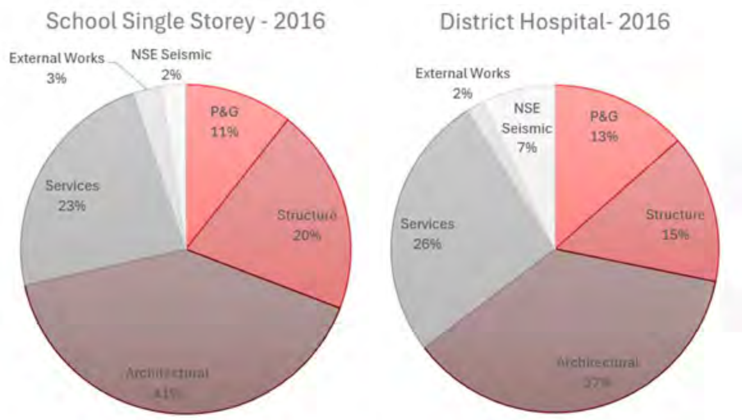

Figure 4 – Elemental cost breakdowns from 2016 (WSP NZ, 2017).

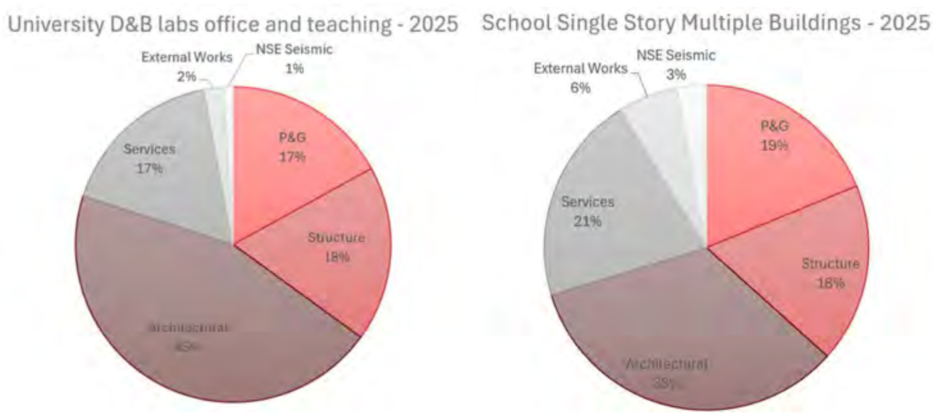

Figure 5 – Cost breakdown of building elements in 2025 for Left D&B Teaching and Lab Space, and Right Traditional School Project.

This updated snapshot of the industry data provides some interesting insights into how the delivery of NSE restraint has been maturing over the past 8 years. Whilst the NSE seismic restraint for a simple building using traditional design then construct procurement is similar, it shows that the contractual procurement model can have a significant effect on the cost to deliver NSE seismic restraints. The D&B delivery model has noticeably lower costs to deliver NSE seismic restraints.

Conclusion

The successful implementation of Non-Structural Element (NSE) seismic restraints firstly requires early coordination of design disciplines. Any failure to coordinate NSE seismic restraint in the design, inevitably results in variations and inefficient solutions during construction. With a traditional design, procure and construct approach, even a well-coordinated design to LOD300 will require some adjustment of NSE restraint details in the construction phase due to either development of shop drawings, and/or redesign to accommodate contractor product selections. This needs to be understood and communicated early in design. From recent project experience and as supported by contractor review, engaging subcontractors during design, or taking a Design and Build approach enables NSE restraint design to be completed efficiently, and significantly reduces the risk of variations related to fitout during construction. An ECI approach can be taken to provide input to NSE restraint design, but to be truly effective needs to include key subcontractors, which can be perceived as reducing commercial competitiveness.

To facilitate coordination of NSE designs in highly serviced buildings, use of standardised modules or components or a common restraint systems are recommended to deliver a well-considered cost-effective solution.

The role of the project structural engineer for the overall project outcome is critical. The structural engineer muse ensure that the structural design can accommodate all loads, including gravity and seismic NSE loads throughout the building's lifecycle as well as consideration of appropriate building drift and the implications of drift on the seismic performance of NSEs. Lean structural designs that do not adequately consider the seismic design of NSEs, particularly at the roof level, can pose significant challenges and lead to costly modifications if not adequately addressed.

Overall, early NSE design coordination not only reduces project cost risk but also provides valuable options for contractors, ultimately contributing to the project's success.

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Innovation Through Constraint: The Chatham Island Renewable Energy Project

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ABSTRACT

Chatham Island, approximately 800 km from mainland New Zealand, relies on ship-supplied diesel generators for electricity. This results in the highest electricity costs in the country, and blackouts can occur due to shipment delays. The Chatham Island Renewable Energy Project (CIREP) was commissioned to address this predicament– a small wind farm intended to strengthen the island community's future energy resilience and sustainability.

The project included multiple unique constraints that affected foundation design. The absence of a concrete batching plant on the island made a traditional mass in-situ concrete solution unfeasible without the costly transport of a certified mixing facility and raw materials. Furthermore, shipping logistic constraints required any precast solution to take a modular design approach, with a limit on individual component weight.

This paper explores how the unique challenges and constraints shaped a design solution that met the project constraints – from utilising modern FEA with complex geometry, to detailing a modular precast, post-tensioned solution that optimises material usage and reduces on-site construction time. The delivered solution sets a sustainable and replicable precedent for wind farm foundations across the country while demonstrating the role of constraints in driving innovation.

INTRODUCTION

Project Background

Chatham Island is the largest island in the Chatham Islands archipelago, located approximately 800 kilometres east of Christchurch in the South Pacific Ocean (Fig. 1). The island's remote location imposes significant logistical and economic challenges on its circa 750 residents. All goods must be shipped to the island, resulting in elevated costs for many essential supplies. The cost of electricity is one of the most substantial burdens, with unit prices reaching $1.37 per kilowatt-hour (Chatham Islands Council, 2023) – 400% higher than mainland New Zealand.

Chatham Island relies on diesel generators for electricity generation. Diesel is shipped to the island and trucked to the power station, leaving the island's energy supply vulnerable to periodic power outages from delayed shipments. Chatham Island Wind, a private company, attempted to diversify the energy supply by installing two 225 kW wind turbines in 2010. However, the wind farm ceased operations in 2017 due to the liquidation of its parent company.

Figure 1: Location of Chatham Islands and Point Durham site (Toitū Te Whenua Land Information New Zealand, 2025).

On 18 May 2023, the New Zealand Government announced it would be allocating $10.7 million to install a renewable electricity system on Chatham Island (Government of New Zealand, 2023). Chatham Island Enterprise Trust commissioned the project with Pachlan Limited chosen as the principal project manager. WSP were selected to complete structural and geotechnical engineering of three turbine foundations in May of 2024.

CONSTRAINTS

Site Conditions

The chosen site for the wind farm at Point Durham (Fig. 1) is approximately five hundred meters from the shoreline. The corrosive marine environment meant durability was a factor in the selection of the preferred structural form. BRANZ had previously completed a corrosivity study at multiple locations across the island which allowed for quantification of the site corrosivity. Reinforced concrete outperforms steel in highly corrosive environments, making it the preferred choice for both environmental and maintenance considerations.

Geotechnical

Preliminary site investigations indicated the presence of a strong basalt layer below approximately 2-3 meters of weak clay soils. The basalt was assessed to provide an appropriate founding layer, with the unsuitable soil above requiring excavation. WSP received two designs as part of the bid documents: a reference design from the turbine supplier, Vestas, and an early concept design developed by another consultancy. Both designs were similar, utilising a piled foundation tied together by a shallow in-situ concrete slab. With no piling rig on the island and an accessible competent founding layer, a shallow foundation would be cheaper and easier to construct on-site.

Remote Construction Limitations

In-situ mass concrete pads are the industry standard for wind farm foundations. This approach creates multiple obstacles for this project, primarily that Chatham Island does not have a

concrete batching plant. To facilitate mass in-situ concrete production, a batching plant would have to be shipped out to site along with the raw materials. The plant would require certification and significant onsite quality assurance (QA) processes; cylinder test samples would be required more frequently and would require shipping back to mainland New Zealand for analysis. Together these factors would significantly increase project risk, onsite labour, and shipping costs.

A prefabricated system would help to minimise these issues. Fabrication on mainland NZ would be cheaper and allow for more thorough and accurate QA. The fabricated system could then be shipped piece-by-piece and assembled on site, reducing the higher-cost on-site labour.

Shipping and Logistics

There were multiple transport stages to consider for the project: shipping to the island, truck transport between Waitangi Port and the Point Durham site, and on-site craneage. Chatham Islands Shipping Ltd provided shipping limits that were governed by the 20-tonne single-lift capacity of the Waitangi Port cranes. On-site craneage and truck net weight capacity were both assessed but neither were critical over the port limit. Reducing the number of ship berths would allow for streamlining of the construction timeline and minimise shipping costs. Items under 10 tonnes could be stowed in the ship's hold which would allow segments to be shipped both above and below deck. Any option that utilised on-island materials such as site-won soil or quarried aggregate would also decrease shipping requirements.

OPTIONS ANALYSIS

Global Precedents

Research was conducted into international precedents for precast modular wind turbine foundations (Jiménez Toña et al., 2024), (Li et al., 2024). Two systems were highlighted as inspiration: the ANKER foundation by ANKER Foundations GmbH, and the BXG foundation by RUTE. The ANKER system features many small, inverted tee-section legs, which are easy to manufacture and provide an efficient balance of bearing area and bending strength. The BXG system utilizes larger, radially post-tensioned box-girder style components, simplifying manufacturing and installation.

Multi-Criteria Analysis

A multi-criteria analysis (MCA) was used as a decision-making tool to evaluate the proposed options against key criteria based on the design, logistics, constructability and location-based constraints. MCAs are a flexible tool that can utilise as many criteria as required which are weighted by relative importance. Options are then scored against each criterion to determine the preferred option. The MCA was used to illustrate how the different criteria factored into each option. Weighting allows criteria with higher importance to exert more influence on the outcome. Working through the MCA with the client gave them a clear, rational basis that they could support for endorsing the preferred option.

Options Proposed

The first two options developed were variations of precast, modular, shallow, concrete solutions. Illustrated as Options 1 and 2 in Figure 2; the first drew inspiration from the international precedents and the second from traditional in-situ concrete pads.

The first concept utilised eight precast ‘legs’ arranged radially around a central cylindrical ‘hub’ component. Inverted tee-section legs provide an efficient strength-to-volume ratio while maintaining a large bearing surface. Post-tensioned bars would be utilised to connect the components while also reducing fatigue stress. This concept was highlighted as a complex design requiring careful detailing of the hub section; however, it did show potential for the best payoff in material volume/weight, repeatability, and constructability.

The second concept was a large square precast pad, split into components that would be joined on site. Post-tensioning in two orthogonal directions was expected to pose structural concerns and constructability challenges and so a one-way post-tensioned solution with insitu joints was proposed.

Three other concepts that did not fulfil all preferred characteristics were also considered for a thorough comparison. First was the steel spider foundation designed by WSP for use on the Scott Base Wind Farm project; second was the traditional concrete in-situ foundation; third was the piled slab foundation from the aforementioned reference design.

Further Options Development

After discussion with the wider design team, the two highest scoring options from the MCA – the two shallow precast concrete systems – were developed further before final selection. These were evaluated in depth against the most important constraints: complexity, constructability, and shipping requirements. This review included further assessment of key aspects including:

- Initial overturning calculations enabled estimates of overall geometry and component mass to confirm the individual components would meet the shipping crane and deck weight limits.
- A construction sequencing methodology was drafted to demonstrate how on-site assembly of the systems would be achieved.
- Onsite crane handling was assessed to verify the components could be safely handled on site. The number and weight of components and the amount of island-based fill material were estimated to compare total shipping demand.

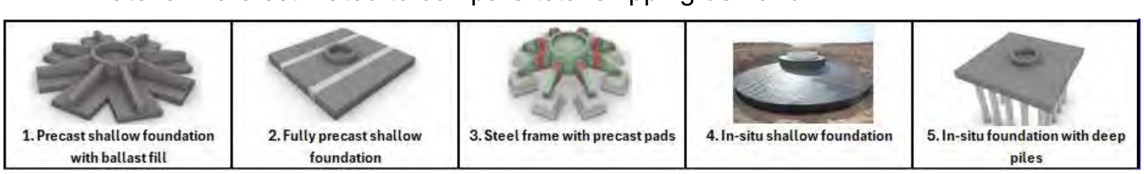

Figure 2: Concepts evaluated by the Multi-criteria analysis.

The foundation system comprising radial hub-and-legs was selected as the preferred design following this more detailed review and discussion with the client and wider design team. The full two-stage options analysis and concept refinement was extensive, but it gave the design team confidence both in the selected system and that the design approach and buildability requirements were understood and well-considered early. This was a conscious risk management approach that reduced the uncertainty associated with implementing a bespoke solution.

DESIGN HIGHLIGHTS

Ensuring both precise and accurate design of a bespoke solution with complex geometry presented a challenge. Two approaches were used to ensure an accurate and efficient solution: parametric modelling and Finite Element Analysis (FEA) allowed for precise modelling of the load distribution throughout the structure, while simplified hand calculations were completed to verify their accuracy. The solution selected also used approximately 20% less embodied carbon than the Vestas reference piled solution.

Parametric Modelling

Rhino is a CAD software that has been rapidly gaining popularity in structural engineering for its powerful parametric design tools which were utilised to generate the foundation geometry, allowing for fast design iteration and optimisation. The parametric geometry – shown in Figure 3 – was exported to SAP2000, finite element analysis (FEA) software used for structural analysis. The parametric workflow facilitated rapid iterations to optimize both post-tensioning design and key performance metrics, including global deflection, overturning stability, and rotational stiffness. Decreasing iteration time resulted in quickly finalising the geometry of a design that efficiently met all project constraints and design requirements.

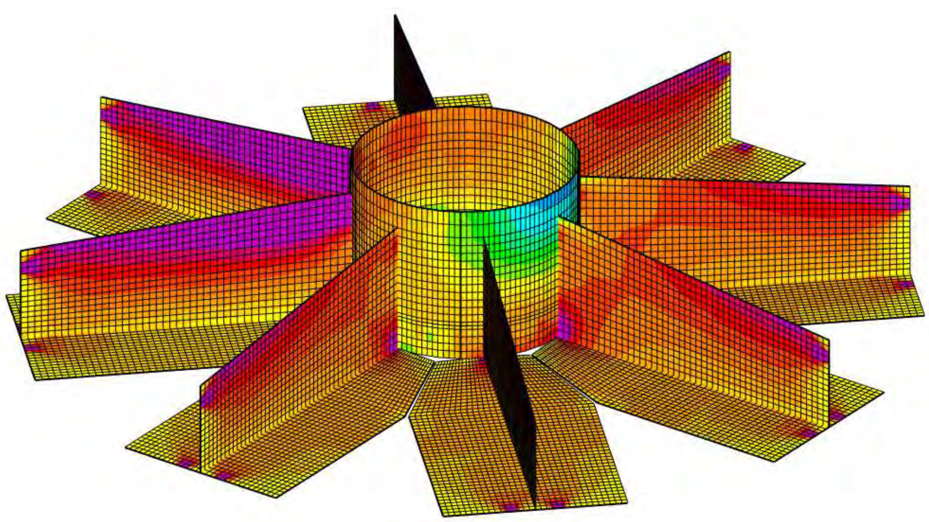

Figure 3: Geometric model used for FEA.

Serviceability Design

Following international best practice, this project incorporated the guidance outlined in the International Electrotechnical Commission (IEC) standards IEC 61400-1 and IEC 61400-6. These standards provide specific recommendations for the structural design of wind turbines and their foundations. A key consideration was the directive to prevent uplift at the foundation base under S1 loading conditions—a load level greater than the New Zealand serviceability limit state. This aimed to mitigate the risk of soil degradation beneath the foundation due to cyclic compaction. The no-gapping criterion governed the overall footprint of the structure by limiting the minimum effective radius of the foundation plan.

Fatigue Design

Fatigue design can be critical for wind turbine foundations due to the cyclic nature of turbine loading. New Zealand's concrete design standard NZS3101 does not provide explicit fatigue guidance for reinforcing steel. The FIB Model Code – referenced in the NZS3101 commentary document – was adopted to align with global best practice. The FIB approach, like the New Zealand steel design code NZS3404, employs a Wöhler S-N curve to quantify fatigue damage. Site-verified fatigue loading data provided by Vestas was used to verify fatigue performance of the structure. Design for the Fatigue Limit State (FLS) governed the reinforcing steel layout within the hub component due to the large reversal in stresses under cyclic loading.

CONSTRUCTION HIGHLIGHTS

Construction was completed in two distinct phases. From November 2024 to January 2025 the hubs, legs and aprons were fabricated in Christchurch. This allowed for regular pre-pour inspections by the design team and concrete production by an established and certified plant, ensuring high quality of the finished components. During design, the WSP design team modelled the hub reinforcement and additional penetrations required in the steel tower can. Modelling the complex hub reinforcement layout reduced fabrication clashes and complications, resulting in few issues being raised during fabrication.

The precast components were then shipped to Waitangi Port, unloaded, and trucked to site at Point Durham where site preparation had been started in December 2024. The initial investigation into shipping allowed for the PC legs to be stored in the lower ship hold and larger hubs on the deck. This reduced the total number of ship berths required for delivery to two. The prefabricated approach resulted in completion of on-site precast assembly and post-tensioning of the three foundations within four weeks with only a small labour crew required. Photos of the completed foundation assembly are shown in Figure 4. The early investigation into shipping and trucking requirements ensured no logistical issues arose.

Figure 4: On-site assembly photos of the foundation structure.

CONCLUSION

The remote location of the Chatham Islands Renewable Energy Project presented unique challenges for designing wind turbine foundations that made the traditional mass concrete approach unfeasible. In response, a tailored solution was developed to address all project constraints and deliver a high-quality solution to the client. Through a detailed options analysis and open communication with stakeholders, the ideal structural system was identified early in the design process.

By carefully considering all constraints and utilising a two-step MCA evaluation process, the team developed an innovative off-site prefabricated solution for this remote application that was more sustainable and required less on-site labour than the industry standard approach. Early investment in a thorough concept design phase enabled potential challenges to be resolved upfront, avoiding costly redesigns later on. This forward-thinking approach contributed to a smooth and efficient construction process both on the mainland and on-site. The use of parametric and FEA design tools and alignment with international best practice further helped streamline the process and ensure a high-quality result that will serve the Chatham community, delivering sustainable, and reliable energy into the future.

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Staged Analysis and Construction Sequencing of Innovative Pool Roof Box Girders for Hiwa, Recreation Centre

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Abstract

Hiwa, Recreation Centre at the University of Auckland is a world-class sports facility and is one of New Zealand's largest and most complex education construction projects.

Hiwa is an 8-storey structure inclusive of two basement levels. Above ground, the lateral system consists of perimeter steel diagrid mega-braces and concentrically braced frames. At the basement level, the perimeter concrete ground retention walls form the lateral resisting system.

The pool hall and plaza structure extend to the west of the main super-structure. Concrete shear walls form the perimeter of the pool hall, with the pool roof supporting the external plaza space above. The pool roof profile incorporates 11 different types of steel box girders, each spanning 28 metres and subject to varying degrees of cranked geometry. This creates a unique roof profile that facilitates the head height requirements for the diving platform and creates the plaza mound above. The plaza was used as a laydown space during construction and is subject to vehicle loading.

This paper focuses on the staged analysis performed in the design of the pool roof box girders. It considers the different loading conditions and construction sequencing requirements necessary to achieve the assumed design boundary conditions.

Introduction

Hiwa – The University of Auckland Recreation Centre finished construction in late 2024. The name Hiwa symbolises aspiration, growth and ambition, all of which is encompassed by this new world-class sports facility. Hiwa facilities include two sports halls, a rooftop turf, outdoor running track, squash courts, swimming pool, cardio and weights rooms, and group fitness studios.

Design started in 2018 with construction beginning in 2021. Early construction works included the demolition of the existing recreation centre and enabling works to facilitate Hiwa and the larger site footprint.

This paper provides a focused description of the design of the pool roof steel box girders which span above the swimming pool and form the profiled roof (Fig. 1).

There are 11 different types of box girders ranging from a simple box girder with no variation in geometry, to box girders that have varying degrees of cranked geometry. The arrangement of these box girders allows for the unique roof profile which forms the plaza mound above.

The complex boundary conditions affecting these box girders, such as the pool roof serving as the laydown zone during construction and the varying loading conditions, necessitated a staged analysis for their design. This approach ultimately determined the construction sequencing methodology and installation requirements for the box girders.

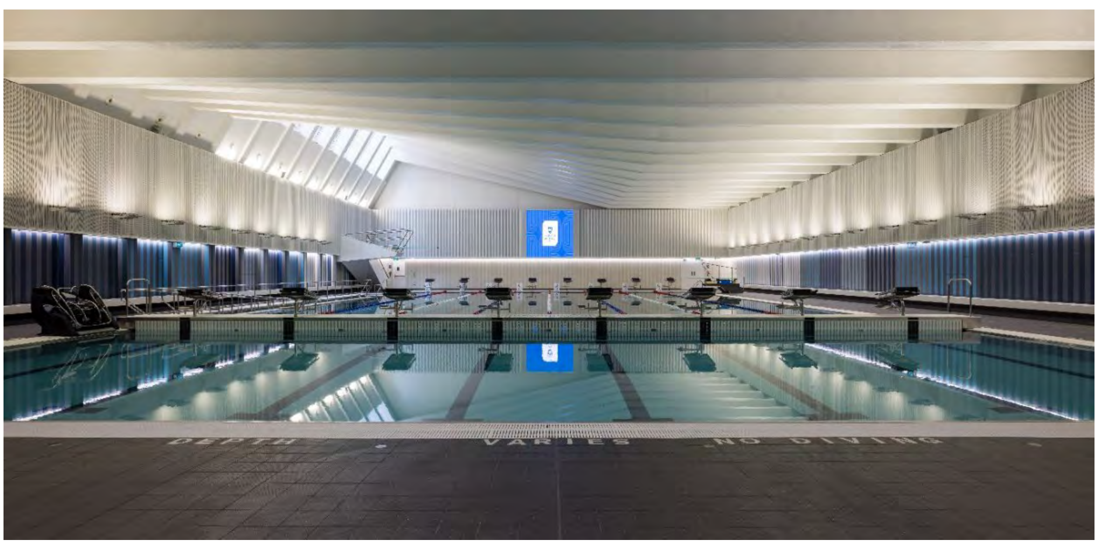

Figure 1 - Pool roof box girders forming the profiled pool roof (Scowen, 2024)

Building Overview

Hiwa is an 8-storey structure inclusive of two basement levels. Permanent ground retention has been provided around the perimeter of the basement. This retention system consists of concrete soldier piles to East Coast Bays Formation (ECBF) rock, a mass concrete infill between the piles,

and a concrete facing wall. The building's foundations consist of piles down to ECBF rock, while the swimming pool bears directly on rock.

The gravity system uses a composite metal deck flooring system supported by composite secondary steel beams, primary beams and columns. The large spans above the sports halls are achieved with deep steel trusses.

The seismic design philosophy adopted is a primarily elastic structure, with capacity designed connections. The lateral load-resisting system consists of perimeter steel diagrid mega-braces and concentrically braced frames (CBFs). Perimeter concrete ground retention walls transfer seismic demands from the diagrid to the ground, with piles resisting the seismic design actions from the CBFs.

The pool hall and plaza structure extend to the west of the main super-structure (Fig. 2). Concrete shear walls form the perimeter of the pool hall, with the pool roof forming the external plaza space above. The pool roof box girders span 28m between the shear walls and bear directly onto them. A flat soffit metal-deck slab spans between the box girders onto the top flange. This 180mm thick slab served as the laydown zone during construction. In the final state, a 100mm wear slab was added above to support vehicle movements and form the plaza mound build-up (Fig. 3).

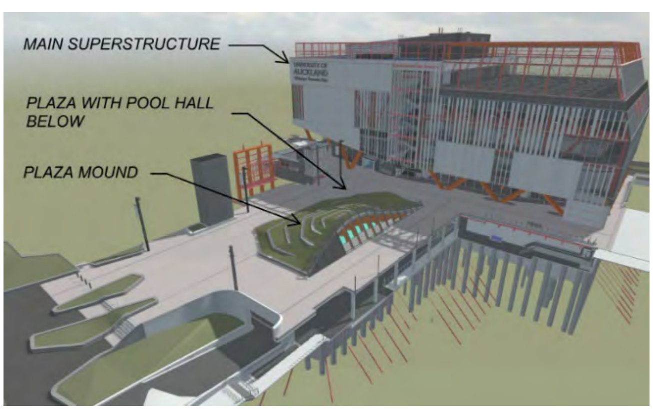

Figure 2 - Overall layout.

Figure 3 - Plaza mound directly above the pool hall, formed by cranking of the box girders.

Box Girder Geometry

For design purposes the steel box girders were categorised into two types: Type 1, which features no variation in geometry along the girder's length, and Type 2, which includes variation in geometry along the girder's length. The Type 2 box girders form the plaza mound.

The Type 1 box girder is 1200mm deep, 400mm wide. The box girder was pre-cambered to minimise any perceived gravity deflections which could be apparent from the high level viewing gallery in the final state.

There are 10 variations of the Type 2 box girder. The geometry of the box girder can be broken into three segments. The first two "beam" segments are 1200mm deep, whilst the third "post" segment is 600mm deep. To accommodate the gravity deflections in the final state the box girder is preset upwards at the transition between the straight segment and inclined segment.

A typical cross section through the 1200mm deep girder is shown in Figure 4. The difference between a Type 1 and Type 2 box girder is shown in Figure 5.

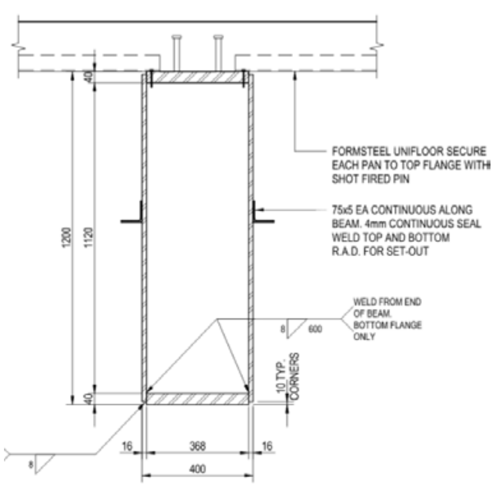

Figure 4 - Typical cross section for the 1200 deep box girder.

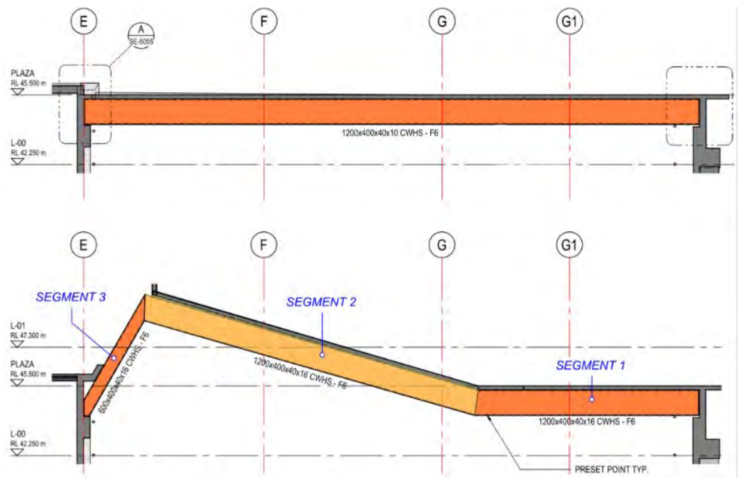

Figure 5 - Type 1 box girder (top) and a Type 2 box girder (bottom).

Design Approach

Design Loads

The self-weight allowance included the box-girder weight and 180mm metal-deck slab. A significant superimposed dead load allowance was provided for the 100mm wear slab, insulation, skylights, services and plaza landscaping.

The plaza is subject to vehicle loading, whilst the plaza mound is inaccessible by a vehicle. Three live load cases were considered to cover both construction loading and end state plaza use:

- Case 1: a 10kPa area load
- Case 2: a 60kN point load
- Case 3: 0.85HN Bridge Manual Loading

The following ULS load combinations were considered:

$$1.35G$$

$$1.2G + 1.5Q$$

The critical combination is using the Case 3 live load.

Additional design cases considered included the thermal effects of expansion/contraction of the steel box girders, and the natural frequency of the box girders.

Boundary Conditions

The following boundary conditions were assumed in the design of the box girders:

Type 1 Box Girders:

- Girders are simply supported and considered to have pin-end restraints.
- Slotted holes have been provided at both ends for construction tolerance and thermal effects.
- No composite action relied on for strength, partial composite action relied on for deflections.

Type 2 Box Girders:

- Girders have a pin-end restraint and a roller-end restraint during construction stage.
- 'Arching' action will occur if the ends are not free to spread laterally.
- Slotted holes are only provided at the roller-end (segment 1) during construction.
- The roller allows for the release of the horizontal action as the beam spreads.
- In the final stage the roller-end becomes a pin-end, and the horizontal slab diaphragm resist the thrust from the beam arching.
- No composite action relied on for strength, partial composite action relied on for deflections.

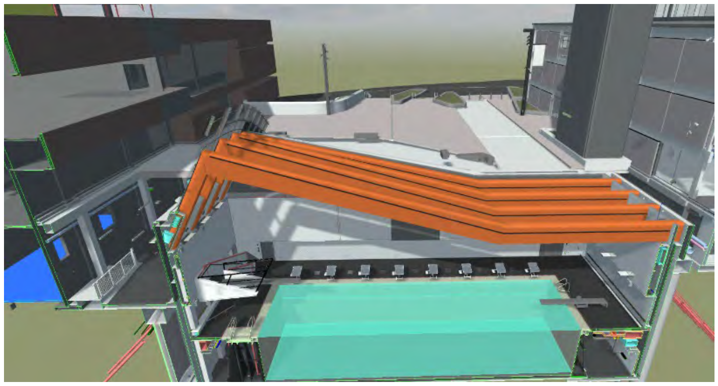

Figure 6 - Construction: box girders spanning between shear walls.

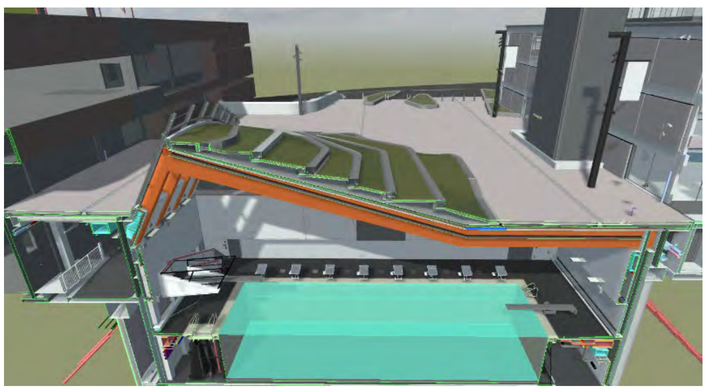

Figure 7 - End state: plaza slabs on both sides can resist the thrust.

Staged Design Analysis

For the Type 2 box girders, a staged design analysis was performed to determine the amount of axial load that could be effectively released through the lateral spread of the box-girder.

In construction, the absence of the plaza slabs on both sides of the shear wall, meant that the shear walls could be over stressed in out-of-plane bending if they were required to resist the large thrust actions. To address this, the box girders were allowed to spread horizontally by incorporating 80mm long horizontally slotted holes at the segment 1 end. Figure 6 illustrates this initial setup of the box girders between the shear walls.

In the end-state, the horizontal diaphragm can resist the thrust created by the beam arching. Figure 7 shows this final stage with the surrounding plaza slab.

The following methodology outlines the staged design analysis:

Stage 1: Determine the maximum axial demand by considering both end restraints to be pins. (Fig. 8)

Stage 2: Determine the maximum horizontal spread by considering a pin-end restraint and a roller-end restraint. (Fig. 9)

Stage 3: Assume the 180mm slab has been poured. Allow the box girder self-weight and the slab self-weight to be released by slotted holes. Consider a pin end restraint and a roller end restraint. (Fig. 10)

Stage 4: Remove the stage 3 axial demand from the Stage 1 axial demand to determine the reduced axial demand. (Fig. 11)

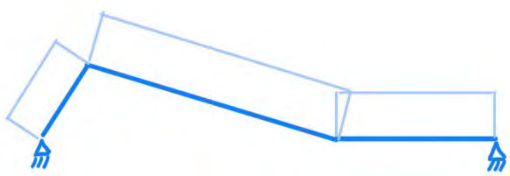

Figure 8 - Maximum Axial Force Diagram.

Figure 9 - Introduction of a roller-end restraint.

Figure 10 - Determining the horizontal spread.

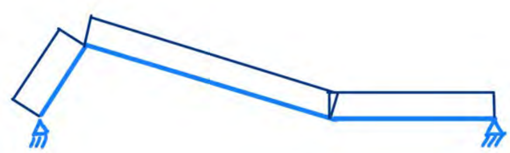

Figure 11 - Reduced Axial Force Diagram.

Construction Sequencing

The staging of the design analysis required a clear construction methodology to effectively communicate the boundary conditions assumed, and the sequencing that was required for the installation of these Type 2 box girders.

Step 1: Shear walls to be in their final state to allow for installation of box girders.

Step 2: Install box girders. Each 28m box girder was transported to site as one complete beam. (Fig. 12)

Step 3: Snug-tighten the hold-down bolts and dry-pack the baseplate on the segment 3 side.

Step 4: Position the baseplate on a stainless-steel shim with hold-down bolts hand tightened only on the segment 1 side.

Step 5: Pour the 180mm thick metal-deck slab. Allow the beams to spread at the segment 1 support point and record this movement. Submit to the engineer to review these deflections. (Fig. 13)

Step 6: Remove the stainless-steel shim, snug-tighten the hold-down bolts, and dry-pack the baseplate on the segment 1 side.

Step 7: Grout at both ends of each box girder between the girder end and shear wall to lock them in place.

Step 8: Utilise the slab as a laydown zone during construction.

Step 9: Pour the 100mm wear slab, record vertical deflections of the box girders, and submit these to the engineer for review.

Figure 12 - Box girders installed.

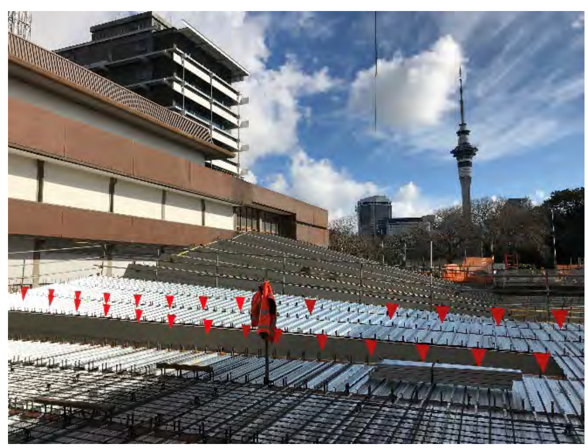

Figure 13 - Metal-deck placed for 180mm slab.

Conclusion

Hiwa showcases innovative engineering in one of New Zealand's largest and most complex education construction projects. This paper has outlined the design and staged construction approach for the pool roof box girders, which feature varied geometry, cranking profiles, and complex boundary conditions. These innovative girders not only support the functional requirements of the pool hall but also create the unique architectural profile of the plaza mound above.

The staged design analysis was key to reducing axial loads and horizontal reactions on shear walls by releasing locked-in forces during construction. This approach optimised structural performance while minimising material usage, demonstrating how thoughtful sequencing can contribute to sustainable design solutions and future focus.

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Practical Considerations for Seismic Design of Temporary Works in the Strengthening of Existing and Construction of New Buildings

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ABSTRACT

The seismic design of temporary works is a critical yet often underappreciated aspect of construction. Temporary works must comply with both the Building Act and the Health and Safety at Work Act (HSWA), necessitating robust design practices to mitigate risks during construction. Failures in temporary works during seismic events, both in New Zealand and globally, have resulted in catastrophic consequences, including structural collapses, loss of life, and significant damage. The construction of new buildings and the modification of existing ones present opportunities to reduce overall risks in New Zealand's built environment. Encouraging greater collaboration between designers and constructors, including the development of safe construction or strengthening sequences, is vital to fulfilling Safety in Design requirements.
This paper serves as a companion to the forthcoming New Zealand Temporary Works forum (TWf NZ) technical guidance note, *Seismic Design for Temporary Works*, currently out for public consultation. It summarizes some of the same information but also discusses underlying justifications and background not contained in the guidance itself.

Risk and Complexity Evaluation of Temporary Works

A key aspect of procedural control and management of temporary works is risk and complexity evaluation. This is an aspect of temporary works that is important for all stakeholders to understand and may have practical applications that can be possibly applied to other aspects of design including permanent works. This is covered in detail in TWf NZ Technical Guidance Note TGN 05.23: Temporary Works Risk Assessment and Categorisation.

The consequence of failure is an important thing to consider, and it can be correlated directly to importance levels as defined in AS/NZ1170.0.

It should be noted there is a certain level of subjectiveness to this assessment, and other controls can be used in conjunction with engineered controls to affect the consequences and likelihood of failure (also the consequences may vary depending on load cases, refer to the section on Temporary Works Return Periods and Seismic Design).
Refer to table 1 for Consequences of Failure Risk and Corresponding Importance Levels.

Table 1: Consequences of Failure Risk and Corresponding Importance Levels

Minor (Importance Level 1)	Low impact and entirely within site; inconvenient but personal injury unlikely.
Significant (Importance Level 2)	Significant impact and entirely within site; potential for personal injury but fatality unlikely.
Major (Importance Level 3)	Potentially major effect, but failure, while potentially of major impact (for instance involving fatalities and injuries) would not initiate any secondary or chain reaction of major incidents.
Catastrophic (Importance Level 4)	Failure, should it occur would be catastrophic or, even if minor, might initiate a secondary or chain reaction of major or catastrophic incidents.

Table 2, Design Complexity shows various levels of design complexity and description of each category.

Table 2: Design Complexity

Basic	Built to a simple specification or to industry guidance (NZS3604 or the Scaffolding in New Zealand), engineering input not required. (Seismic design implicitly covered in most cases)
Simple	Simple design that can be completed by suitably experienced technicians using software or simple calculations.
Involved	Understanding of structural engineering principles, specific temporary works design and specific knowledge of relevant standards required.
Complex	Highly complicated with a requirement for subject specific knowledge and experience.

Check Category		Consequences of Failure Risk			
		Minor	Significant	Major	Catastrophic
Design Complexity Risk	Basic	Cat 0	Cat 0	Cat 1	Cat 2
	Simple	Cat 1	Cat 1	Cat 2	Cat 2
	Involved	Cat 2	Cat 2	Cat 2	Cat 3
	Complex or innovative	Cat 2	Cat 3	Cat 3	Cat 3

Check Category	Minimum Competency	Independence of Checker
0	The Designer and Checker should be suitably competent and experienced.	The check may be carried out by another member of the site or design team.
1	The Designer and Checker should be suitably competent and experienced.	The check may be carried out by another member of the design team.
2	The Designer and Checker should be suitably competent and experienced. The checker should be a CPEng engineer.	The check should be carried out by an individual not involved in the design and not consulted by the designer.
3	The Designer and Checker should be suitably competent and experienced. The designer and checker should be CPEng engineers.*	The check should be carried out by another organisation.

Figure 1: Temporary Works Risk Assessment Matrix and corresponding requirement for peer review

Current Compliance Pathways

Temporary works are subject to additional structural requirements compared to permanent works. Temporary works designers use the same environmental loadings and materials codes as permanent works designers but must also be aware of WorkSafe NZ documentation and the wide-ranging powers of the HSWA. While permanent works typically fall under the definition of a "Building," and all building work must comply with the Building Code, Temporary works, also referred to as "Siteworks" are also covered by section B1 of the Building Code, while also needing to adhere to additional specific guidelines under the HSWA.

Verification Method B1/VM1 deems compliance with the Building Code if NZ loadings and materials standards (AS/NZS 1170, NZS 3101, NZS 3404, etc.) have been followed. In contrast, the HSWA does not have a prescribed set of Verification Methods and Acceptable Solutions that deem compliance with the Act. However, the Act (s.20) allows the Minister, through the Ministry of Building Innovation and Employment (MBIE) and its predecessors, to produce Approved Codes of Practice (ACOP) for various aspects of temporary works. ACOPs have also been issued through WorkSafe NZ, and while their wording is cautious, the message is that undertaking work in compliance with these documents will meet the requirements of the HSWA (e.g., "Prevention of Falls," April 2000).

Since many temporary works items generally support a building during construction, most propping is bound by both acts, and the most onerous requirements should be complied with. The relevant codes (AS/NZS 1170, NZS 3101, NZS 3404, NZTA Bridge Manual etc.) should therefore be referenced. Designers should adhere to the most stringent requirements and reference relevant codes (e.g., AS/NZS 1170, NZS 3101, NZS 3404).
Additionally, there is an implicit requirement in New Zealand guidance that a Chartered Professional Engineer must verify most temporary works items beyond basic temporary works. There are few ways for an Engineer to conduct these calculations without using codes intended for building design, such as AS/NZS 1170, NZS 3101, and NZS 3404.
In conclusion, temporary works in all regions of New Zealand require consideration of seismic loads in their design. However, depending on the risk and complexity of the project, explicit seismic design may not always be necessary, and other controls or mitigation measures could be employed. When explicit calculations are required, these will typically rely on established building design standards, such as those outlined in AS/NZS 1170, NZS 3101, NZS 3404 and the NZTA Bridge Manual.

Temporary Works & Existing Structures

When designing temporary works for projects relating to existing structures, the permanent structures may not meet the temporary works seismic loading requirements of NZS1170.5 or the NZTA Bridge Manual (e.g. earthquake prone buildings) and/or may need to be further

weakened before being strengthened. When this is the case, it may be considered unreasonable to design temporary works to the full loading requirements of NZS1170.5 or the Bridge Manual. In these situations, discussions should be had between contractors, engineers, clients (and in some cases insurers) to determine what is considered reasonable for the specific project, considering the durations and failure consequences of the work.
While there is no current legal definition of what is reasonable in this case, context can be provided by referencing the following legislation:

- Earthquake Prone Building legislation – this sets expectations of accepted timeframes to strengthen buildings that are earthquake prone, whilst still being occupied as well as minimum strength to achieve where no occupancy restrictions are in place.
- Building Act Section 112 – outlines the Building Act requirements when carrying out alterations to existing buildings.

Temporary works design approaches that could be adopted could therefore include:

- Design temporary works to match current strength of the building or building element (i.e. make no worse than existing).
- Design temporary works to make the structure/element no longer earthquake-prone (>33%NBS).
- Accept partial weakening of elements or lateral lines of restraint for short periods of time
- Design localised temporary works elements to full temporary works loads, while accepting reductions to global strength during the works.

With any of these approaches, robust principles must still be applied, collapse, exposure to critical structural weaknesses and brittle failure mechanisms should be avoided and clear load paths must be provided.

Discussion on Temporary Works Return Periods and Seismic Design

In the context of seismic risk and temporary works, the minimum return period typically designed for is a 1/100-year event. At first glance, this may seem overly conservative for a structure that could be in service for significantly less than a few months. However, it's important to recognize that temporary structures, especially buildings under construction and scaffolds in urban settings or alongside critical infrastructure networks operate in environments where safety is a primary concern, particularly when they are in public spaces or areas where workers are at risk. Historically, some guidance (such as the Ministry of Works' standards) allowed for reductions in seismic loads for short-duration exposures. However, current standards do not provide a similar reduction for temporary structures, reflecting a more conservative approach. The recommended design life for temporary works is six months, which corresponds to a 1:100-year return period for Importance Level L2 structures.

In some cases, particularly for structures categorized under Importance Level 1, where a low degree of hazard is presented, the guidance strongly advocates for prioritizing safety in all instances.
The argument for reducing return periods based on exposure duration does not apply to seismic design in the same way it does for wind or snow loads. Earthquakes, unlike wind, rain or snow, cannot be predicted or mitigated with operational measures in most cases. This distinction justifies the more conservative approach to seismic design for temporary works. A designer must decide whether they can accept the risk of larger seismic events or whether they need to design for these larger loads to ensure safety.
Scaffolds are typically designed using conventional ties, which are generally sufficient for standard conditions, such as low to moderate seismic zones. For example, in locations outside high seismic regions like Wellington, traditional scaffold designs with conventional ties may adequately resist seismic forces. However, for more complex scaffolds—such as engineered structures without wraps and with considerable suspended weight—the seismic load becomes more significant. In these cases, nominal horizontal forces could pose a real risk, requiring more robust design strategies, or at least careful consideration.
This principle applies not only to scaffolds but to any temporary structure exposed to seismic events that could endanger workers or the public. The seismic design of these structures must consider the potential consequences of failure, not just the likelihood of seismic events during their design life.
Considerations should be grounded in public safety, practicality, and affordability:

- **Public Expectations**: Structural failures, even temporary ones, carry reputational, legal, and safety consequences. For instance, a scaffold collapse in a busy urban area could cause significant harm to the public, regardless of how long the scaffold has been in place. Reducing seismic loads to minimize costs may therefore be unacceptable, given the potential consequences.
- **Practicality**: Seismic risk is more challenging to mitigate than forces like wind or snow. While we can prepare for wind and snow through operational controls or design flexibility, earthquakes cannot be predicted or prevented in the same way. Therefore, the risk posed by seismic forces cannot be mitigated as easily.
- **Affordability**: While cost is always a consideration, it must be balanced against the potential consequences of failure. The cost of designing for larger seismic events may be high, but the consequences of failure far outweigh any potential savings from cutting corners. Designing for uncertainty ensures that temporary works remain resilient, and safety is prioritized over cost savings.

CONCLUSIONS

To conclude, as seismic risk is a constant consideration in New Zealand, it is important that stakeholders understand the risks during construction. Given that contractors have knowledge of construction and designers understand their designs, greater collaboration between designers and constructors is essential. This includes developing safe construction or strengthening sequences that fulfil Safety in Design requirements, resulting in good outcomes for both clients and the public.

This paper serves as a companion to the forthcoming New Zealand Temporary Works forum (TWf NZ) technical guidance note, *Seismic Design for Temporary Works*, which is currently out for public consultation. It includes key considerations and basic knowledge to assist. Below is a summary of the key points:

- Risk and complexity evaluation is crucial in managing temporary works, ensuring all stakeholders understand the potential consequences of failure. This process is linked to importance levels as defined in AS/NZ1170.0, with subjective judgment required in certain cases. Other controls can be used alongside engineered solutions to mitigate failure risks, which can vary depending on load cases. The TWf NZ Technical Guidance Note TGN 05.23 provides detailed guidance on assessing and categorizing these risks.
- Temporary works have additional structural requirements compared to permanent works, as they must adhere to both the Building Code and the Health and Safety at Work Act (HSWA). Designers must follow the same environmental loadings and materials codes as permanent works but also consider WorkSafe NZ documentation and specific guidelines under the HSWA. Verification Method B1/VM1 deems compliance with the Building Code, but the HSWA does not have predefined methods, relying on Approved Codes of Practice (ACOPs) from WorkSafe NZ. While temporary works may not always require explicit seismic design, when necessary, they should follow established building design standards like AS/NZS 1170, NZS 3101, and NZS 3404.
- When designing temporary works for projects involving existing structures, the permanent structure may not meet seismic loading requirements, such as those in NZS1170.5 or the NZTA Bridge Manual, especially in earthquake-prone buildings. In these cases, it may be unreasonable to fully design temporary works to meet these standards. Discussions between contractors, engineers, and clients should determine what is reasonable based on the project's duration and failure consequences. Potential design approaches include matching the current strength of the structure, improving earthquake resistance, accepting temporary weaknesses, or designing localized elements to full load requirements. Regardless of the approach, safety principles must be followed, and critical weaknesses must be avoided.
- In seismic design for temporary works, the standard return period is typically a 1/100-year event, reflecting a conservative approach. Although this may seem excessive for short-term structures, it ensures safety, particularly in public spaces or construction

areas where workers are at risk. Unlike wind or snow, seismic events cannot be predicted or mitigated through operational measures, justifying a more cautious approach. While scaffolds in low to moderate seismic zones may suffice with conventional designs, more complex structures may require enhanced design to address the higher seismic loads. The seismic design must consider the potential consequences of failure, prioritizing public safety, practicality, and affordability.

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SESOC CONFERENCE 2025

Navigating Change

With Integrity and Future Focus

25-27 June 2025 | Tākina, Wellington

MBIE/NHC/SESOC Low Damage Sesimic Design Technical Guidance

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ABSTRACT

Low Damage Seismic Design (LDSD) is a building design philosophy that achieves better than New Zealand Building Code minimum requirements. A key goal of LDSD is to deliver buildings that are less likely to be damaged and thereby limit disruption and losses in future earthquakes. SESOC, in partnership with MBIE and NHC, are developing technical design guidance which can be used by project managers, design consultants, contractors and facilities management to design, construct and maintain seismically resilient buildings. This paper will summarise some of the key technical design recommendations proposed for the document.

INTRODUCTION

The 2010-2011 Canterbury earthquake sequence caused significant damage to many modern buildings in Christchurch (Kam, Pampanin, & Elwood, 2011). More recently, the 2016 M_W 7.8 Kaikōura earthquake caused significant damage to numerous buildings in Wellington, despite the epicentre being some 200 km away to the southwest (Cubrinovski, et al., 2020). In the immediate aftermath of the 2016 Kaikōura earthquake about 11% (180,000 m^2) of the city's office space was closed for assessment and many office buildings had to be demolished (Elwood, Filippova, Noy, & Paz, 2020).

In light of such events, industry has recognised that a life safety focus is insufficient to meet community expectations and that communication about the likely performance of buildings in earthquakes should be improved (Kam et al., 2011 and NZSEE, 2024). Recognising this, Engineering NZ and Structural Engineering Society New Zealand (SESOC), in partnership with the Ministry of Business, Innovation and Employment (MBIE), established a LDSD project in 2019. This project built on ideas and concepts for LDSD that were circulating in New Zealand at the time (Hare, Oliver, & Galloway, 2012).

A three-volume LDSD Guidance Series, developed in partnership with MBIE, SESOC and the National Hazards Commission (NHC), represents the culmination of the original 2019 LDSD project.

LDSD GUIDANCE SERIES OVERVIEW

A three-volume LDSD Guidance Series is being developed and consists of the following documents:

- Low Damage Seismic Design Volume 1: Benefits, Options, and Getting Started.
- Low Damage Seismic Design Volume 2: Performance Framework.
- Low Damage Seismic Design Volume 3: Technical Guidance.

LDSD Volume 1: Benefits, Options, and Getting Started (MBIE, 2024) is intended to help building owners and tenants decide if LDSD is right for their project.

LDSD Volume 2: Performance Framework is being written for building owners, developers, tenants, project managers and design consultants. The document will define the recommended Performance Framework for LDSD projects and how LDSD projects fit within the New Zealand building regulatory system (Oliver, Brunsdon, & Sullivan, 2025).

LDSD Volume 3: Technical Guidance is being written for project managers, design consultants, contractors, and facilities management. It will provide designers with a methodology to achieve the LDSD outcome objectives and post-event performance goals.

WHAT IS LOW DAMAGE SEISMIC DESIGN?

LDSD is primarily good design practice by a well-coordinated design team. A key goal of LDSD is damage control whereby designers can deliver buildings that are seismically resilient and less likely to be damaged and thereby limit disruption and losses in future earthquakes. Good design practice means conscious, thoughtful, decisions are made about selection of building sites, typologies and components.

Observations of building performance in past earthquakes (Shiga, 1968), (Baird & Ferner, 2017), (Pettinga, Sarrafzadeh, & Elwood, 2019), (Lagos, et al., 2020) and (NHC, 2024) have consistently demonstrated stiff, regular buildings on good ground perform well in earthquakes provided secondary and non-structural elements are adequately restrained. Limiting building displacements is an effective means of mitigating damage to the primary structure and deformation sensitive secondary and non-structural elements (NSEs).

LDSD Outcome Objectives

Outcome objectives for LDSD buildings as detailed in LDSD Volume 2 are:

- Life Safety, a low probability of loss of life or significant injuries.
- Damage Control, a low probability of damage leading to significant economic loss.
- Improved Functionality, by reducing the probability of damage, the time to return to functionality is likely to be significantly lower than a minimum code compliant building.

A summary of the LDSD outcome objectives are presented below. For a more detailed explanation of the LDSD outcome objectives and related benchmarking studies refer to Oliver et al. (2025).

Table 1: LDSD Damage Control perfomance goals

LDSD Category	Expected Annual Loss (EAL)	Approximate probability of exceeding 5% replacement cost in an earthquake in a 10-year period
Level 1	N/A[1]	N/A[1]
Level 2	0.15%	5%
Level 3	0.05%	2%

Notes: 1. There is no explicit damage control performance goals requirements for LDSD Category Level 1 buildings.

The Building Code addresses the life safety outcome objectives by means of an Ultimate Limit State (ULS). The damage control outcome objective is the primary focus of LDSD and is addressed through the introduction of a Damage Control Limit State (DCLS) and the Damage Control performance goals detailed in Table 1. The expectation is that for most building designs there will be no need for design teams to explicitly demonstrate compliance with the limiting EALs and probabilities of exceedance detailed in Table 1. The design procedures detailed in LDSD Volume 3 are calibrated so the Damage Control performance goals will generally be met for most buildings.

Table 2: Annual Probabilities of Exceedance (APoE) for different design limit states

Limit State	Annual Probability of Exceedance		
	LDSD Level 1	LDSD Level 2	LDSD Level 3
SLS1	1/25	1/50	1/50
DCLS	N/A[1]	1/250	1/500
ULS[2]	1/500	1/500	1/500

Notes:
1. There is no specific requirement for LDSD Category Level 1 buildings to consider DCLS.
2. ULS APoEs detailed in Table 2 are for conventional Importance Level 2 buildings. Importance Level 3 and 4 buildings would be designed using ULS APoE’s of 1/1000 and 1/2500 respectively.

Earthquake design actions are to be determined using TS 1170.5 (SNZ, 2025). Table 2 details the APoEs to be used for determining earthquake design actions for LDSD buildings. As detailed in Table 2, the APoE for SLS1 will be reduced from 1/25 to 1/50 for LDSD Category Level 2 and 3 buildings. Reducing the APoE for SLS1 delays the onset of damage in smaller events and increases the probability a building will be in a repairable state following a damaging event beyond SLS1 (Pettinga, Sarrafzadeh, & Elwood, 2019). The reduced APoE is consistent with that used for new building design in Japan and performance-based design in the United States (PEER, 2017).

LDSD Category Level 2 and 3 buildings are expected to achieve Operational State Category OS II - Partial Functionality (refer Table 3) following a DCLS intensity earthquake provided

wider impacts beyond the building site, outside the control of the design team, do not occur. Wider impacts that could impact building functionality include public utility outages (i.e. power, water, wastewater and data), post-earthquake cordons established by emergency services limiting building access, and labour or material shortages.

Table 3: Indicative post-earthqauke building operational states.

Operational State Category	Operational State	Description
N/A	Safe egress only	A building which only meets life safety objectives without any expectation of post-earthquake occupiability or functionality.
OS-I	Shelter in place	A building for which post-earthquake functionality has been significantly reduced. Building services such as HVAC, water supply and electrical systems may be damaged and unavailable until necessary repairs are completed[1] The deformation capacity and strength of the building has not been significantly reduced due to the prior earthquake, and the building is able to withstand another major earthquake.
OS-II	Partial functionality	A building for which post-earthquake structural and non-structural damage is limited to the extent that the basic intended functions of the building's pre-earthquake use are maintained or can be restored within an acceptable time (usually measured in days rather than weeks)[2].
OS-III	Full functionality	A building for which post-earthquake structural and non-structural damage is limited to the extent that the intended functions of the building's pre-earthquake use are maintained or can be restored within an acceptable time (usually measured in minutes to hours rather than days).

Notes:
1. Residents who are sheltering in place will need to be within walking distance of a neighbourhood centre that can help meet basic needs.
2. Other repairs need to return the building to a full functionality state can be completed over a longer timeframe provided these can be undertaken outside normal working hours or as part of normal annual maintenance.

Critical non-structural components and systems necessary for OS II are to be identified by the project team. A strategy is to be established for each critical non-structural component and system to enable OS II to be returned within the acceptable timeframe.

DESIGN PROCESS OVERVIEW

The recommended LDSD design process is summarised in Figure 1. Some key recommendations from LDSD Volume 3 Technical Guidance are summarised below.

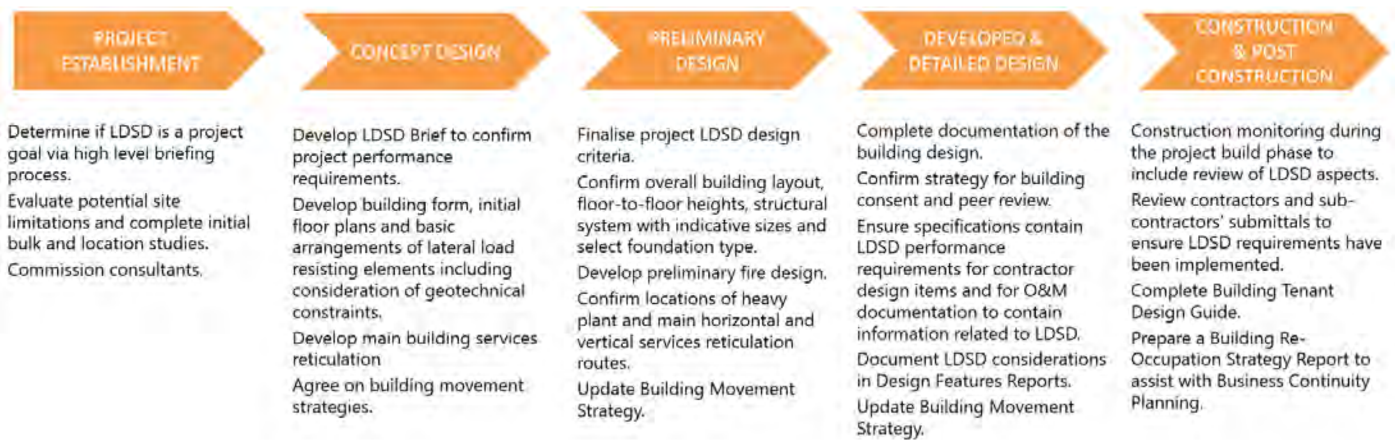

Figure 1: Recommended LDSD design process

Project Establishment

It is recommended project teams identify if LDSD is a project goal during the project establishment phase by means of a high-level briefing process. This is because LDSD can have a significant impact on site selection, building form and the design of structural and non-structural elements. Delaying the decision to adopt LDSD on a project can have significant cost and program implications. Good site selection is identified as being fundamental for achieving LDSD because for some poor sites it may not be practicable to achieve LDSD because of high costs associated with mitigating seismic hazards that affect the site. It is therefore important that geotechnical engineers are part of the core design team at an early stage to ensure site specific hazards are understood and appropriately considered.

Bulk and location studies are commonly undertaken during the feasibility phase of a project. The overall form of the building is fundamental to LDSD. While the basic building form is typically designed by the architect, it is recommended advice be sought from a structural engineer as a design is developed. This is because regular form can make a big difference to the seismic performance of a building, and it is important the client and architect consider this as part of their early decision making.

Figure 2: Recommended LDSD briefing process.

Concept Design

During the concept design phase, a key task is to initiate the process to translate the client's overall objectives for the project into technical design criteria which can be used by the design consultants for the project. Figure 2 illustrates the recommended briefing process. For non-specialist buildings, such as commercial offices or apartments, it is recommended one of the standard LDSD category levels be adopted i.e. Level 1, 2 or 3. However it is acknowledged that for some projects, specific bespoke LDSD requirements might be required.

Table 4 below is an example of client briefing resources included in LDSD Volume 3 which can be used by the design team and the client to establish the LDSD performance goals and inform the corresponding technical design criteria.

Table 4: Example discussion points included in LDSD Volume 3 to assist establishing the LDSD Brief.

Possible Discussion points/questions		**Commentary**
Primary question	**Secondary questions**	
What is the building to be used for?	What is its primary use? What is its secondary use?	Impacts on Building Importance Levels. Impacts LDSD criteria.
How soon do you want to be able to occupy your building after a large earthquake?	Minutes, hours, days, months? Or is this not a significant issue for the client?	Determines if recommended LDSD Category Level can be used. Informs bespoke criteria.
Any specific building functions or features that may warrant specific consideration of LDSD criteria?	Partial functionality	Examples include valuable contents in museums, laboratories, etc
Is some level of repairable damage tolerable in reasonably foreseeable events?	What potential repairs might be considered acceptable? By extent By cost By time to complete.	Can be used to determine a specific response if required.
How reliable is the infrastructure to the site?	Electrical Comms/Data Three Waters	Information on the seismic resilience of public infrastructure is not typically available, and hence any assessment is likely to be indicative only. Consider need for on-site water storage and power generation as below.

LDSD Volume 3 highlights the importance of ensuring design teams jointly discuss and agree on important design considerations such as overall building form, floor-to-floor heights, structural strategy, foundation strategy, building stiffness, spatial requirements for plant items and in-ceiling services etc. during the concept design phase. This includes the requirement for the design team to develop a Building Movement Strategy Report, as defined in the Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025).

As noted above, a structure that is stiffer and therefore deflects less during an earthquake, means that deformation sensitive building components, such as facades, ceilings, partitions, pipes, ducts, cable supports etc., do not have to be detailed to accommodate such large movements at interfaces. Correspondingly, flexible structures require these building components be detailed to accommodate with much higher movements. In complex building types, such as apartment buildings, this can have a major impact on the successful design and execution of deformation sensitive NSEs such as tiled membraned showers and inter-tenancy walls that have to achieve, and maintain, acoustic and fire performance requirements.

The Building Movement Strategy Report is a way for the project team to collaboratively identify at a more granular level some of the conflicting requirements for acoustic/fire /vibration/thermal /passive fire etc. vs seismic movements. Compromises maybe required in some locations where not all of the requirements can be met. In these locations the design team should discuss and agree what aspects are to take precedence and which can be compromised, and why the decision was made. This process should be documented in the Building Movement Strategy Report.

Preliminary Design

The preliminary design phase is critical for achieving LDSD because it is the stage when key, often irreversible design decisions are finalised. These include the following:

- Foundation selection including indicative solutions for basements and retaining walls.
- Overall building layout, grid layout, and cross section including floor-to-floor heights.
- Core layout with vertical circulation and egress paths.
- Structural system with indicative member sizes.
- Location of heavy plant and main vertical and horizontal services distribution routes.
- Confirmation of facade treatment and secondary elements like canopies etc.

In order to achieve a coordinated response that meets the LDSD brief, it is important to have a number of consultant workshops to resolve any conflicting requirements. The compatibility between the foundation and primary structure options should be tested and refined at this stage and preliminary sizing undertaken. Design of NSEs and their co-ordination and interfacing with structural elements should be considered. Any issues arising from the Building Movement Strategy Report should be managed at this design stage.

During the preliminary design phase, the design team should also capture critical LDSD design criteria and how these are addressed, and documented these in their respective Design Features Reports (DFRs). The LDSD design criteria forms the link between the client's aspirations, as captured through the briefing process, and the detailed building design.

It is expected design teams will use the Building Components section of LDSD Volume 3 to determine the technical design criteria for their project. The design criteria adopted will depend on building form and use. For example, a project team might adopt a low drift structure (perhaps via base isolation), allowing for conventional partition and façade detailing.

Alternatively, a project might adopt special partition and façade detailing to accommodate higher drifts of a more conventional structure. These decisions will need to be made in a collaborative way by the whole design team and recorded in the Building Movement Strategy Report.

Developed and Detailed Design

Developed Design is an important stage to resolve any outstanding inter-disciplinary design issues, although inevitably design coordination issues continue to be worked on during the subsequent Detailed Design stage. In the past it has often been left until the construction stage for the seismic design of NSEs. However, this approach can often lead to poor or inefficient project outcomes and should be avoided.

The Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) recommends there is a lag between the completion of the building design and the delivery of the NSE documentation. This is good practice but does not mean that the NSE Designer is not an active participant during these stages. There should also be the opportunity for other consultants to update their documents after the NSE documentation and before final issue.

The NZCIC Guidelines (NZCIC, 2023) provides a detailed list of the requirements for consultants' design and documentation for conventional projects. Additional requirements for LDSD projects at these design stages include:

- Ensure ongoing and well thought out coordination.
- Workshop any situations where there are incompatible requirements.
- Consider access for post-event inspections and repairs.
- Update DFRs with LDSD content.
- Update the Building Movement Strategy Report, in particular to record decisions on conflicting requirements and how these have been resolved.
- Ensure drawings and specifications align with the content of reports related to LDSD.
- Ensure specifications contain detailed requirements for Operating and Maintenance (O&M) Manuals to contain relevant LDSD information.

DESIGN METHODOLOGY

LDSD Volume 3 contains technical design criteria for building components which can be used by design teams to demonstrate a building design meets the requirements for LDSD. Technical design criteria are provided for the following building components:

- Foundations.
- Building Structure.
- Building Envelope.
- Building Services.
- Stairs and Ramps.
- Lifts.
- Ceilings.
- Partition Walls.
- Passive Fire.
- Building Contents.
- Landscaping and External Services

Technical design criteria provided not intended to provide detailed guidance for the general design of buildings, or to replace existing building standards and guidelines. Rather the LDSD design criteria are intended to act as an overlay and provide designers with a methodology to achieve the LDSD outcome objectives and performance goals as detailed in Volume 2.

LDSD Verification Pathways

Three verification pathways are available to demonstrate a building meets LDSD outcome objectives and post event performance goals:

1. Prescriptive Design Method;
2. Direct Design Method; and
3. Direct Assessment Method.

The Prescriptive Design Method is limited to buildings which meet certain size and regularity restrictions, or building components, elements or systems where expected performance is reasonably reliable, and the design method is covered by the Building Code generally through an Acceptable Solution or Verification Method.

It is envisaged the Prescriptive Design Method will be applicable to most modest-sized buildings. For buildings using this pathway, the LDSD design process is similar to the current Building Code design process, albeit there is an increased requirement for design collaboration and reporting. Provided the prescribed design parameters and/or methods are used, and the type of system or component fits within the scope, then the Prescriptive Design Method is deemed to satisfy the LDSD provisions.

The Direct Design Method pathway is used for building systems or components that don't meet the requirements of Prescriptive Design Method pathway. It is for those elements, systems or components that are considered innovative, new or alternative solutions via the Building Code. Demonstrating conformity using this method will require the design team to follow recognised analytical methods, or experimental testing in accordance with recognised standard testing procedures, to validate the performance of potentially affected building elements, systems or components will not compromise the LDSD performance goals detailed in Volume 2.

An example situation when the Direct Design Method pathway might be adopted by a design team would be for the case when a DCLS inter-storey drift limit of 0.75% is adopted. This exceeds the 0.5% inter-storey drift limit prescribed in the Prescriptive Design Method pathway. The Direct Design Method pathway could be used to validate adequate performance of drift sensitive components such as partition walls, façade systems, passive fire systems etc. This could include experimental testing of proposed 'improved' partition wall detailing to demonstrate that damage sustained by partition walls at 0.75% inter-storey drift met the Physical States damage criteria detailed in Volume 2. In a similar way, all other building elements and components would be required to sustain the 0.75% inter-storey drift while meeting the Physical States damage criteria. However, other unaffected building components (such as those that are sensitive to floor acceleration and not storey drift demands, or structural

elements in which damage is limited through ductility checks rather than storey drift) could still be design using the Prescriptive Design Method pathway.

The Direct Assessment Method pathway is an explicit assessment method that enables design teams to demonstrate compliance with the LDSD performance goals detailed in Volume 2. Design teams should use recognised performance assessment procedures such as the FEMA P58 (FEMA, 2018) approach. Fragility functions required for the Direct Assessment Method pathway should be adopted considering New Zealand construction practice.

Building Structure Design Criteria

A summary of some of the key proposed LDSD technical criteria in the Prescriptive Design Method pathway for the design of LDSD Level 2 and 3 structural systems is included below.

Proposed Prescriptive Design Criteria

In addition to the requirements of the NZBC, the primary structure is to be designed in accordance with the material standards cited in NZBC B1/VM1 with the additional design criteria given below.

The structural ductility factor, μ, used for the design of the primary structure should not exceed the maximum values detailed in Table 5 (further limitations for specific structural systems will be defined separately). The structural performance factor, S_p, for the DCLS should be determined in accordance with the methodology used in TS 1170.5 (SNZ, 2025) for the ULS.

Table 5: Maximum structural ductility factor

Building Limit State	Maximum structural ductility factor[1]
SLS1	1.0
DCLS	2.0
ULS	3.0

Notes: 1. As defined in TS 1170.5 Section 4.3 for use with equivalent static and modal response analysis method.

Member design capacities for DCLS design procedures should be calculated using strength reduction factors and nominal material strengths as detailed in AS/NZS 1170.0 (SANZ, 2011) for the ULS.

Peak floor accelerations (PFAs) for the DCLS should generally be less than 1.25g and should not exceed 1.6g unless the seismic performance of acceleration sensitive components have been validated for higher shaking intensities. PFAs should be calculated using TS 1170.5 Eqn 8.1 with the part spectral-shape coefficient, $C_i(T_p)$, and the part horizontal response factor, C_{ph}, taken equal to 1.0.

Except for single storey buildings, or the uppermost storey of multi-storey buildings, capacity design should be used to ensure that a suitable sway mechanism develops under very rare shaking levels. Potential plastic hinge regions should be detailed in accordance with the relevant material standard, and the additional LDSD requirements as defined in Volume 3, to ensure they have adequate deformation capacity to reliably resist anticipated deformation demands.

Structures should not have vertical or plan irregularities as defined in Section 4.5 of TS1170.5, except that a vertical stiffness irregularity is permitted for structures when capacity design has been used to provide a level of protection against the formation of a column sway mechanism.

No single line of lateral force resistance to have a single element or assembly resisting more than 60% of the total lateral demand in the building in that direction. When assessing if an element or assembly is resisting more than 60% of the total lateral demand in a building direction accidental eccentricity need not be considered.

Peak and residual inter-storey deflections for DCLS load combinations, including consideration of foundation flexibility, should not exceed the limits detailed in Table 6. Peak inter-storey deflections should be calculated in accordance with Section 7.3.1 of TS 1170.5.

Table 6: DCLS inter-storey deflection limits

Building Inter-storey Deflection	Deflection Limit[1]
Peak Deflection[2]	0.5%
Residual Deflection	0.5%

Notes: 1. Inter-storey deflection limit is expressed in terms of the percentage of the corresponding storey height.
2. Peak inter-storey deflection limit can be increased by means of the Direct Assessment Method.

Allowance shall be made for the deformations arising from member elongation for DCLS load combinations.

Upper limit shear design actions on reinforced concrete and masonry walls should be determined as 1.5 times the ULS seismic design action with μ = 1.25 and S_p = 0.9.

All elements that may require inspection following a DCLS event must be accessible.

Foundation flexibility, including piles and the supporting soils with which they interact, should be included in the analysis when they significantly affect the dynamic properties of the building.

Commentary:

The proposed LDSD design criteria align with recommended industry practice (NZSEE, 2022) whereby designers should deliberately proportion structures with redundancy and enough regularity so that it is possible to identify a clear plastic mechanism. The latter will enable successful implementation of capacity design which will deliver robust structures that can be expected to perform well even when subjected to stronger than expected ground shaking.

Loss modelling studies undertaken to validate the proposed design criteria confirmed the recommended approach of controlling building damage by limiting building deformations and

bracing non-structural components is effective for low to significant levels of ground shaking. However, under higher levels of ground shaking, acceleration sensitive components can be susceptible to shaking damage and this could result in building damage which exceeds that specified in Volume 2.

When PFAs exceed 1.25g design teams should consider alternative lateral force resisting systems which are characterised by lower floor accelerations or consider limiting the specification of acceleration sensitive components to components that have been validated for the levels of shaking expected. The 1.6g limit is a ‘hard’ limit which is not to be exceeded for LDSD Category Level 2 and 3 buildings unless the performance of acceleration sensitive building components is validated i.e. through seismic verification or similar.

The requirement for capacity design and limiting the structural ductility factor used for the design of the primary structure to be not greater than 2 at DCLS load levels, is intended to provide a high level of confidence that (1) any damage sustained to the primary structure is economically repairable and (2) the building will not sustain safety-critical structural damage that would require repair before the building could be reoccupied. Additional system specific local deformation checks will also be included in Volume 3 to mitigate the localisation of inelastic deformation.

For moment resisting frames a beam sway mechanism may be assumed when the following equation is satisfied:

$$\sum M_{n,col} \frac{L_{col}}{L'_{col}} \geq 1.15 \sum M_{n,beam} \frac{L_{beam}}{L'_{beam}} \qquad \text{Eqn 1}$$

where:

$\sum M_{n,beam}$ = sums of the nominal flexural strength of the beams at the faces of the beam column joint zones in the level being considered

$\sum M_{n,col}$ = sums of the nominal flexural strength of the columns at the faces of the beam column joint zones in the level being considered

L_{beam} = centre to centre spans of the beams

L'_{beam} = clear spans of the beams

L_{col} = centre to centre spans of the columns

L'_{col} = clear height of the columns

The sway index concept was originally developed by Priestley (1996) for reinforced concrete moment resisting frames. Consideration should be given to biaxial bending when calculating the nominal flexural strength of columns, $M_{n,col}$, that act as part of two-way frames. When calculating the nominal flexural strength of beams, $M_{n,beam}$, the contribution of slab reinforcement when present should be included in accordance with the relevant material design standard.

Where buildings do not meet regularity provisions, experience has shown these buildings typically perform poorly in earthquakes. For this reason, those buildings that do not meet the regularity limitations in TS 1170.5 have been excluded from the Prescriptive Design Method pathway.

Proposed drift limits specified in Table 6 are not intended to mitigate damage to deflection sensitive elements such as conventional partition walls, however, overall building damage is expected to be controlled such that the building damage performance goals detailed in LDSD Volume 2 are met.

Residual drifts are limited to 0.5% of the corresponding storey height to ensure building repair is possible (FEMA, 2018). This limit is consistent with work completed by McCormick et al. (2008). The limit has been derived with consideration given to both human comfort, building functionality and building safety. Buildings with residual drifts exceeding this limit may not be acceptable to occupy following an earthquake.

Residual inter-storey deflections can be calculated in accordance with FEMA P-58-1 Section 5.4 (FEMA, 2018). However, note that if a maximum drift of 0.5% is maintained then the residual drift limit can be assumed met and will not be critical.

CONCLUSION

This paper provides an overview of a new three-volume LDSD Guidance Series currently being developed MBIE, NHC and SESOC. Some of the technical design recommendations proposed for Volume 3 have been introduced, including a recommended design process for new LDSD buildings and proposed criteria for the design of LDSD Level 2 and 3 structural systems. Using this new guidance series design teams will be able to deliver buildings that are seismically resilient and less likely to be damaged and thereby limit disruption and losses in future earthquakes.

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Integrating Carbon Thinking in Structural Design to Enhance Building Sustainability Outcomes

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ABSTRACT

40% of global energy-related CO_2 emissions come from buildings, and 70% of a building's carbon footprint is determined during design. This paper showcases the impact of integrating carbon thinking into the structural design, using three real design examples, and how the structure can affect the carbon impact outcome.

INTRODUCTION

Balancing embodied carbon and operational energy is essential for reducing a buildings' overall carbon footprint. While efforts have traditionally focused on lowering operational energy, embodied carbon is becoming increasingly significant as buildings grow more efficient. Structural engineers play a key role in shaping sustainability outcomes, as early design decisions heavily influence a building's embodied carbon and limit future design flexibility.

CASE STUDIES

Case Study 1: Bader Ventura. The balance between Embodied and Operational Carbon.

Completed in June 2023, the Bader Ventura development in Māngere was the first Australasian public/ social housing project funded by central government and built to Passive House standards.

The life cycle assessment (LCA) focussed on building B3.1, for a lifecycle of 50 years, using LCAQuick3.6. The materials inventory was obtained by a full schedule of quantities performed by a quantity surveyor.

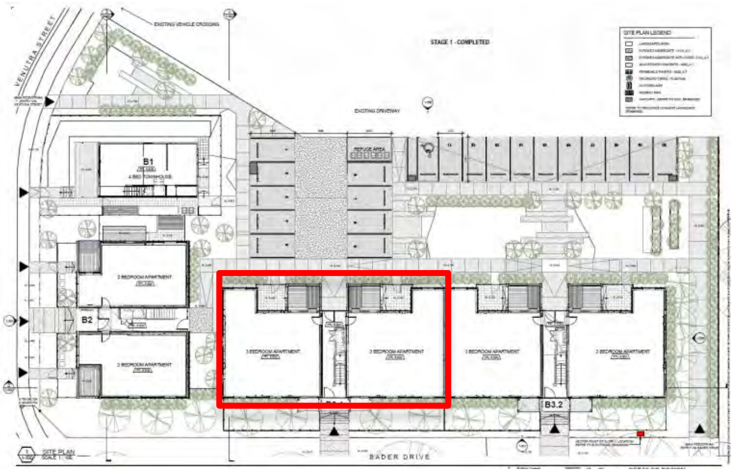

Figure 1: Bader Ventura Development. The study focused on building B3.1

The analysis of the whole-of-life LCA stages revealed that once energy demand was reduced through the passive house design philosophy, the largest carbon impact stemmed from the embodied carbon, particularly during the construction stage or upfront embodied carbon (A1-A5 stages). The remaining operational carbon was due to the hot water heating demand of the building.

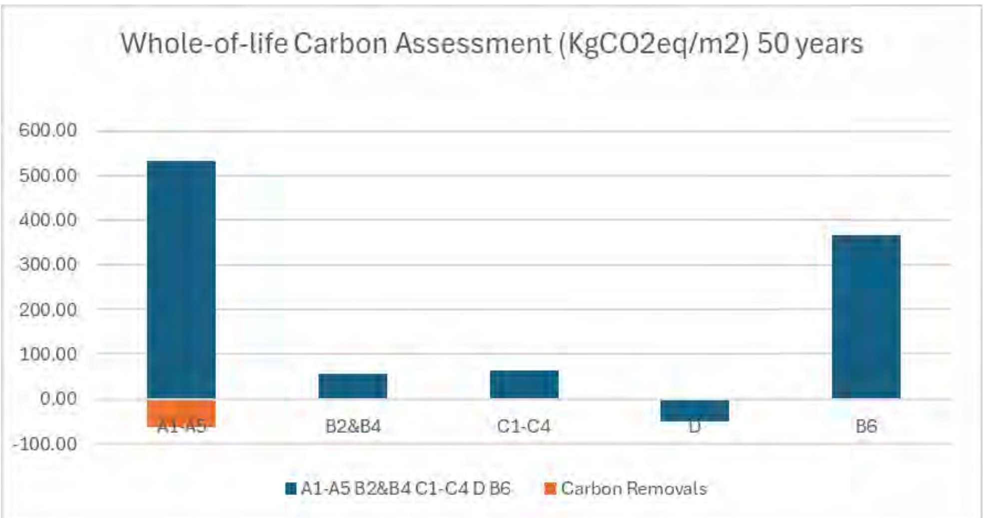

Figure 2: Whole-of-life Carbon Assessment

The analysis of the materials' contribution to embodied carbon highlights the significant impact of concrete and reinforcement.

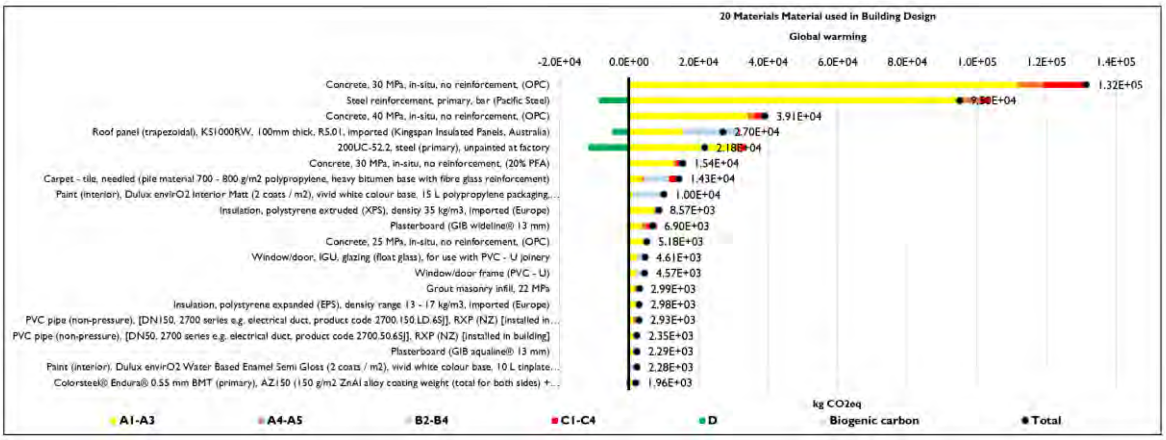

Figure 3: Materials carbon impact (KgCO_2eq)

A more detailed analysis showed a notable contribution of the external walls (excluding lining and paint). The external walls were constructed as a concrete sandwich, comprising a 150mm thick reinforced (rebar) concrete inner layer, a 60mm thick layer of polystyrene, and a 60mm thick reinforced (mesh) concrete outer layer. The steel reinforcement quantity in the inner layer (load bearing) was higher than the average amount, which was confirmed by the structural engineer. This additional reinforcement was necessary to support the external concrete layer. This approach was chosen for its durability, despite contributing to higher upfront carbon emissions, as it reduces the need for future replacements and maintenance.

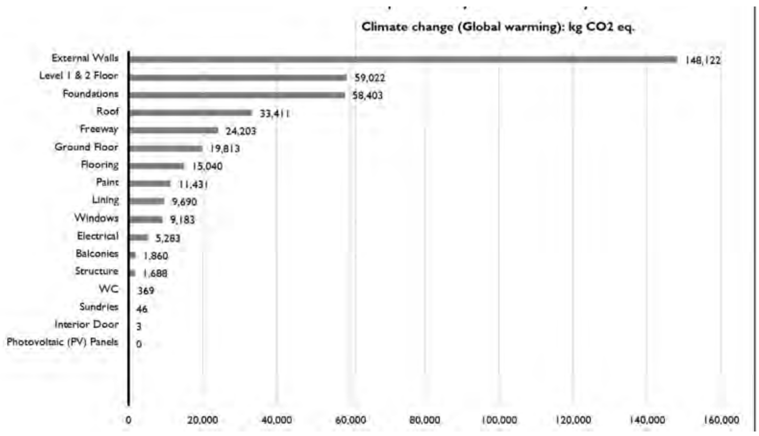

Figure 4: Building construction elements' contribution to the embodied carbon

The Bader Ventura design primarily focused on achieving high energy performance through the passive house standard, with no significant focus on embodied carbon reductions. The only action implemented to reduce embodied carbon was the use of 10-20% carbon-reduced concrete (a combination of fly ash and Neocrete) in the façade layer due to structural constraints.

Potential strategies for further carbon reduction could include alternatives to the outer reinforced concrete cladding, a different structural system with less weight, and the broader use of reduced-carbon concrete. These options could be explored through a desktop study on carbon reduction alternatives.

Case Study 2: Nga Kāinga Anamata: comparative analysis of different structural systems

The Ngā Kāinga Anamata project was designed to deliver 30 new homes within five three-level apartment buildings in Auckland's Glendowie. Each near-identical building used a different construction technology, in particular on the structural system. Unfortunately, the project was closed as the investment was no longer commercially viable, but it reached detailed design stage, enabling sustainability insights to be gathered on a range of building materials and systems.

Figure 5: External Wall materials carbon impact

The LCA assessment focused on three of the typologies (light timber frame, precast concrete, and cross-laminated timber) for a life span of 50 years, using LCAQuick 3.6. The materials inventory was obtained by a full schedule of quantities performed by a quantity surveyor. The results were shown in absolute and per m^2 GFA. The designs also focused on achieving passive house standards, reducing water heating energy demand, and offsetting the remaining energy with the use of solar panels.

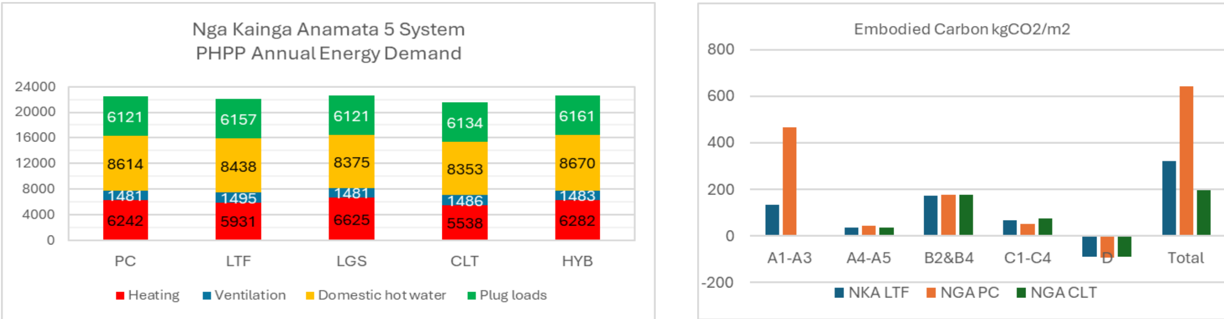

Figure 6: Passive House Energy Results per topology and Whole-of-life Carbon Impact of LTF, PC and CLT buildings

Table 1: Embodied carbon impact comparison between LTF, PC and CLT buildings.

Structural System	Absolute Biogenic Excluded ($kgCO_2eq/m^2$)	Biogenic Carbon ($kgCO_2eq/m^2$)	Absolute Biogenic Included ($kgCO_2eq/m^2$)
Light Timber Frame	549.8	228.8	321
Precast Concrete Panels	723.9	79.9	644
CLT Panels	640.9	445.5	195.3

While it is clear that concrete panel buildings have higher embodied carbon, CLT (cross-laminated timber) buildings can outperform light timber frame (LTF) systems when biogenic carbon is considered. On the other hand, LTF uses less wood, reducing the demand for natural resources (mass timber), lowering costs, and making the structure lighter. In contrast, CLT panels are multifunctional—they can reduce the need for additional materials, allow for more efficient construction with fewer connections, generate less waste, and require minimal temporary works. Another important factor is the cost associated with different construction methods. All of this highlights the importance of treating carbon as one of many factors in the design process – balancing it alongside other priorities to achieve more holistic outcomes and avoid what's often called 'carbon tunnel vision'.

Case Study 3: Alexandra Park Development. Absolute carbon vs Carbon intensity

The original assessment analysed the whole-of-life embodied carbon impact of the Alexandra Park (B1, B2 and Parking buildings) design. The original LCA analysis (Revision 1.0) considered the buildings as commercial (60-year life cycle). A second iteration considered buildings B1 and B2 as multi-rise apartments over a 90-year life cycle, using LCAQuick 3.5.

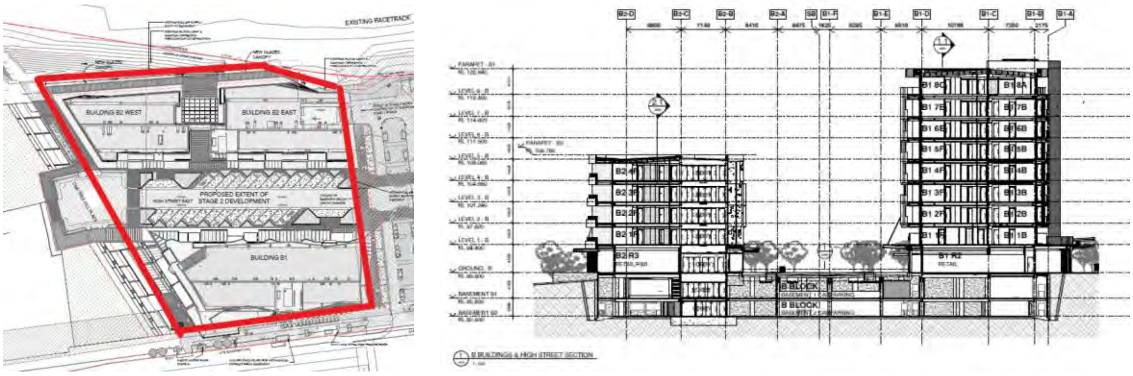

Figure 7: Buildings B1 and B2 footprint and elevation

The total global warming potential (GWP) impact of Alexandra Park complete building site was calculated to be 16,544,782.00 kg CO2 eq. The main materials contributing to the GWP of the complete building site are the steel framing (47%), in-situ concrete (22.6%), and reinforced concrete (14.3%).

A reference building was defined in LCAQuick3.5 with a value for the GWP of 702 kg CO_2eq/m². Building B1 is 0.91 times the Reference building, Building B2 is 1.45 times.

Table 2: Embodied carbon impact comparison between Buildings B1 and B2.

Building	Area (m^2)	Absolute ($kgCO_2eq$)	Intensity ($kgCO_2eq/m^2$)
B1	9533	6,139,127	644
B2	6197	6,315,533	1019

B1 and B2 have a similar absolute GWP; however, when both buildings are compared based on their relative GWP ($kgCO_2eq/m^2$ GFA), the value of B2 is noticeably higher. A further analysis showed that the amount of structural steel used in B2 per m^2 GFA was higher. We can state that the main reason for the difference in carbon intensity is the difference in layout between both building. B1 is more regular in plan and therefore, has a more efficient load path compared to building B2, where the offset in some of the loadbearing members added complexity and needed to be addressed with bigger structural members.

Figure 8: Buildings B1 and B2 Material Carbon Impact

CONCLUSION AND RECOMMENDATIONS

A summary of key findings and their implications for structural engineers is presented below:

- Decisions made at the earliest stages of design – particularly those related to the building's structure—lock in the majority of the project's embodied carbon. Therefore, a structural engineer's understanding of embodied carbon is critical to achieving low-carbon outcomes.
- Energy-efficient buildings with low operational carbon, such as Passive House designs, do not necessarily result in low embodied carbon structures. Reducing embodied carbon delivers immediate carbon savings, whereas operational carbon reductions benefit future emissions. Both aspects must be addressed in parallel.
- While absolute carbon values are important for reporting, presenting results relative to a functional unit enables more meaningful comparisons. Gross floor area (GFA) is commonly used, but alternative functional units—such as number of occupants—may offer better insights, especially for residential buildings.

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The use and misuse of soil-structure-interaction in NZ practice for deep foundations

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ABSTRACT

Soil-Structure Interaction (SSI) has a significant influence on the response of a structure when subjected to seismic ground motion. Nearly all structures are founded on a soil medium and during a seismic event, the deformation of the soil affects the response of the structure (kinematic interaction) and the response of the structure in turn affects the deformation of the soil (inertial interaction). A realistic simulation of a structural system should incorporate the effects of SSI especially when it is modelled for extreme seismic events. Rigorous modelling of SSI is complex and demands the use of advanced numerical models. The modelling process is further complicated by the uncertainties associated with the parametrization of the models used. This paper presents a critical evaluation of the state of NZ practice in SSI modelling especially as applied in the commercial industry by structural engineers for deep foundations while using Nonlinear Response History Analysis. This paper also presents an overview of the key considerations required when incorporating SSI within a structural analysis to arrive at physically consistent conclusions. The authors have come across many examples in which SSI has been “misused” and where unjustifiable conclusions were derived. The paper will consolidate some of these observations and discuss why such simulations result in unrealistic conclusions.

INTRODUCTION

With the increase in seismic hazard levels exhibited within the recently published National Seismic Hazard Model, there has been a noticeable shift within the industry from traditional methods of seismic analysis and design toward more advanced techniques. Among these, Nonlinear Response History Analysis (NLRHA) has emerged as a preferred method for compliance due to its ability to provide a more realistic representation of structural behaviour under dynamic loads. Unlike traditional pseudo-modal methods, NLRHA captures the effects

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of the inertia forces with greater reliability, offering insights that are critical for designing structures in high seismic regions.

While this shift toward NLRHA represents significant progress in seismic design methodology, some practitioners have taken this advancement further by incorporating Soil-Structure Interaction (SSI) effects of deep foundations into their processes. This integration acknowledges the reality that structures do not exist in isolation but interact dynamically with the soils they rest upon. By including SSI effects of deep foundations, engineers can achieve an even more realistic simulation domain, accounting for how soil properties influence the response of structures during seismic events.

However, this approach raises an important and pressing question: *Is structural engineering as a discipline sufficiently mature to fully incorporate SSI effects of deep foundations into its current practices?* While integrating SSI represents a step forward in terms of realism and complexity, it also introduces significant challenges and uncertainties. With correct usage, SSI can provide invaluable insights into the structural response and the effects of the supporting soils which will help improve the overall design. The authors argue that the field of structural dynamics, as currently applied in structural engineering, has not yet reached a level of maturity necessary to handle SSI effectively and consistently especially for structures with deep foundations. Deliberately we are avoiding discussing the NIST GCR documentation, as it largely focuses on shallow foundations and identifies that the determination of suitable nonlinear properties for use when modelling piles requires further research.

This paper provides evidence to support this argument by examining key gaps in knowledge and practice within structural dynamics. It highlights areas where existing methodologies fall short when attempting to account for SSI effects especially for deep foundations and discusses the potential risks associated with premature or poorly understood applications of these advanced techniques. By presenting these findings, the authors aim to raise some pressing issues to provoke critical reflections within the engineering community about how best to balance advance computational applications with robustness in seismic design practices.

DYNAMIC SOIL STRUCTURE INTERACTION (SSI): AN OVERVIEW OF THE PHENOMENA

Dynamic soil structure interactions evaluate the response of the combined superstructure, soil and foundation system. This is primarily a function of the stiffness and mass properties of the structure, stiffness properties of the soil and the damping characteristics of both soil and structure. SSI consists of two interactions: kinematic interaction and inertial interaction.

Kinematic interaction

Kinematic interaction is the result of the stiffness of the structure. Figure 1.0 shows a massless shallow foundation restraining the vertical movements of the ground which results in motions that differ from the free field motion attributed to the foundation. This restriction of the vertical motion is induced by the flexural stiffness of the foundation. Pile foundations will have an even more complicated interaction with the soil. This type of interaction of the foundation with the ground which is solely a result of the stiffness of the foundation is called the kinematic interaction. For more details on this, refer Kramer 1996.

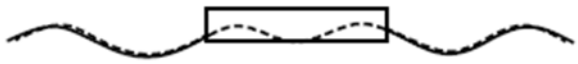

Figure 1.0 Illustration of foundation altering the vertical motion of the ground.

Inertial Interaction

Inertial interaction is a function of the inertia forces generated in the structure due to the displacements of the masses within the structure. These inertial forces transmit dynamic forces to the foundation. The compliance inherent in the supporting soils will result in foundations undergoing dynamic displacement. Figure 2.0 illustrates the inertial interaction phenomenon.

The dynamic displacement at the foundation–soil interface is the sum total of the free field ground motion, the displacement produced due to kinematic interaction, and the displacement produced due to inertial interaction. The portion of the soil adjacent to the foundation undergoes vibration produced by the inertial effects. This results in some energy being lost as it is expended to move the soil mass adjacent to the foundation. This energy loss is popularly known as radiation damping.

The combined effects of kinematic and inertial interactions result in the phenomenon of soil structure interaction.

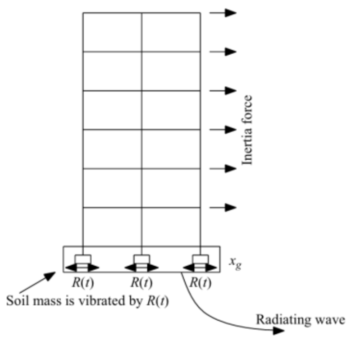

Figure 2.0 inertial interaction effects

In NLRHA models which include the whole foundation the ground motion is input at the base of the foundation model, either at bedrock or at some point below the surface of the ground at the structure's site. The soils themselves alter the ground motion as they act as a filter as the ground motion propagates vertically from the bedrock to the surface. This filtering is complex, as the soil properties, at least for strong shaking, are markedly non-linear. Furthermore, most recorded ground motions are recorded near the ground surface and not where the ground motions are needed to be supplied for the numerical models. Trying to determine a below ground motion from a surface motion is a perilous numerical process. If the foundation model is only representing the foundation compliance effects, this is not generally a problem. If one is only modelling a foundation compliance model, then the ground motion used is usually that at the ground surface.

SSI: AN OVERVIEW OF THE COMMON MODELLING ADOPTED BY STRUCTURAL ENGINEERS

In this section we describe an overview of the problem of SSI in a structural engineering modelling framework. There are broadly two modelling approaches as follows (Refer Kramer 1996 for more details):

Finite Element Modelling Approach

This is the most generic approach to incorporate SSI. Both the soil and the structure are both discretized using numerical models such as finite element models. Only a brief description of the approaches is given here and for detail modelling, readers should refer to Kramer, 1996.

Broadly for FEM based modelling there are two approaches:

- *Bounded Problem Approach*

Figure 3.0 shows a bounded problem schematically. In this example the soil layer is incorporated as 2D plane strain elements and the structure is modelled using line elements. In real analyses the finite element model of the soil is in 3D and similarly the model of the structure is also 3D. In these cases, the magnitude of the soil model will be much larger than that for the structure. If the model has well defined boundary conditions, then the Equation of Motion (EOM) maybe given as,

$$\mathbf{M}\ddot{\mathbf{v}}(t)+\mathbf{C}\dot{\mathbf{v}}(t)+\mathbf{K}\mathbf{v}=-\mathbf{M}\mathbf{I}\ddot{u}_g(t)$$

where,

M,C and **K** are the mass, stiffness and damping matrices for soil-structure combined system; $\ddot{\mathbf{v}}(t)$, $\dot{\mathbf{v}}(t)$ & $\mathbf{v}(t)$ are the relative acceleration, relative velocity and relative displacement with respect to the rigid base as shown in figure 3.0. **I** is the excitation direction matrix. The input motion required is at the bottom of the soil model.

In real structures, the lateral boundaries of the soil are not free boundaries which means that some means of accounting for the radiation of energy though those boundaries must be provided. If not, the energy is trapped in the soil model and is reflected by the lateral boundaries.

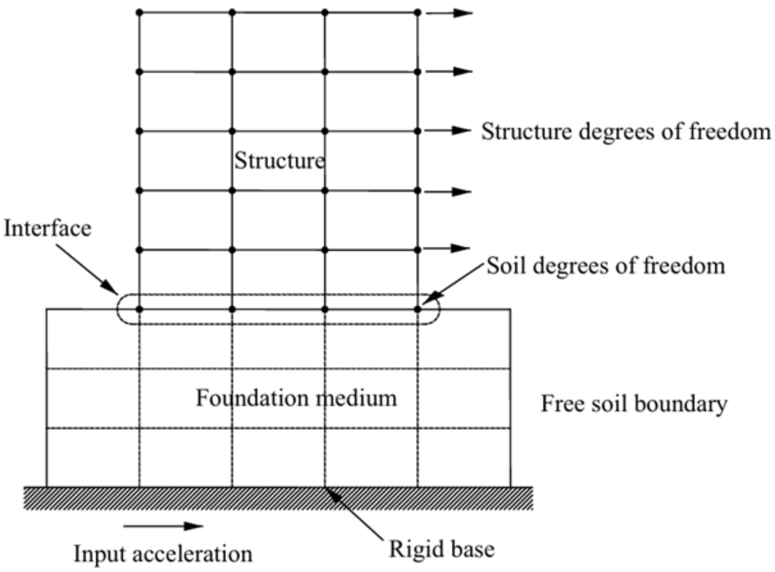

Figure 3.0 FEM approach to SSI

The bounded problem approach can also be done as a multi-step approach as shown in Figure 4. In the multi-step method, the problem is solved in 2 stages:

1. Kinematic stage

Figure 4a shows the kinematic stage where the mass of the superstructure is neglected. In that case, EOM gets modified as,

$$\mathbf{M}\ddot{\mathbf{v}}_i(t)+\mathbf{C}\dot{\mathbf{v}}_i(t)+\mathbf{K}\mathbf{v}_i=-\mathbf{M}\mathbf{I}\ddot{u}_g(t)$$

where,

M is the mass matrix where only the soil mass is present and $\mathbf{v}_i$ are the relative displacements produced at the degrees of freedom.

2. Inertial stage

In this second stage, the entire mass matrix (soil mass and structure mass) is accounted on the left side of the EOM, but the loading mass matrix only has structure mass in the matrix with soil mass initialised as zero.

The EOM gets modified for this case as,

$$\mathbf{M}\ddot{\mathbf{v}}_{ii}(t)+\mathbf{C}\dot{\mathbf{v}}_{ii}(t)+\mathbf{K}\mathbf{v}_{ii}=-\mathbf{M}_{structure}\left\{\ddot{\mathbf{v}}_i(t)+\mathbf{I}\ddot{u}_g(t)\right\}$$

The final response of the combined system is the sum of $\mathbf{v}_{ii}$ and $\mathbf{v}_i$.

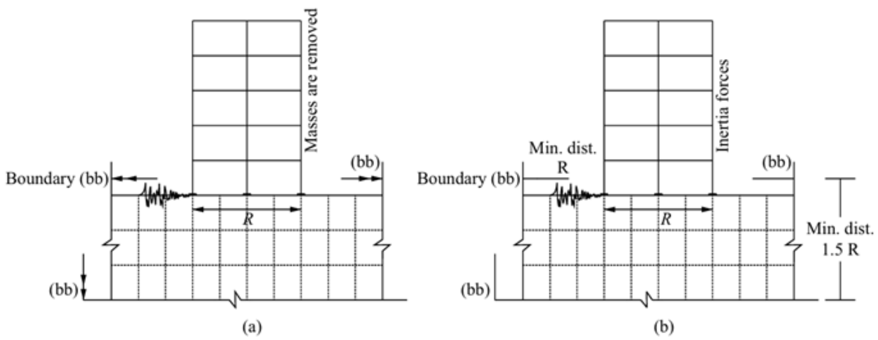

Figure 4.0 multi-step approach

For a large proportion of cases, the kinematic interaction is small and can be neglected. The foundation model provides for a compliance model structural behaviour. In that case, the EOM gets modified as,

$$\mathbf{M}\ddot{\mathbf{u}}(t)+\mathbf{C}\dot{\mathbf{u}}(t)+\mathbf{K}\mathbf{u}(t)=-\mathbf{M}_{structure}\mathbf{I}\ddot{u}_g(t)$$

In this case, as shown in figure 4.0, the ground motion is applied at the free field surface.

Discrete Modelling Approach

Though Finite Element modelling provides more realistic modelling approaches, it is computationally expensive to perform for a 3D structural system. Also, parametrisation of the

elements is extremely complex, and uncertainties associated with the process is also very high. The modelling also requires modelling of the domain boundaries to account for the propagation of wave motion through the boundaries. The other main issue is that the Finite Element approach to SSI demands a non-classical elemental damping matrix which is not yet commonly available in commercial platforms. A more pragmatic approach is to adopt discrete dynamic springs. The general approach is as shown in figure 5.0. This approach is also called a substructure approach.

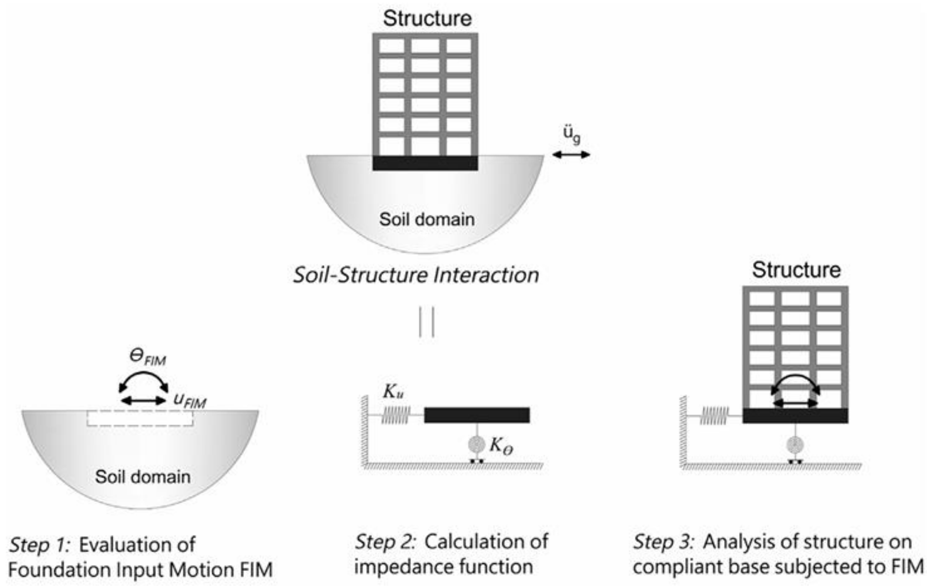

Figure 5.0 Substructure / Discrete-spring approach

Here a de-coupled approach is adopted between the soil and the structure, and an equivalent dynamic spring is determined as shown in the figure 5.0. Based on the type of foundations, broadly the approach can be classified as either shallow or deep as discussed below.

Shallow foundations

As shown in figure 5.0, the substructure approach assumes an elastic half space for which the (frequency dependent) impedance function is known. As the properties of the soils are frequency dependent then usually one is given either stiffness and damping properties or stiffness and added mass properties. There are also frequency independent equivalent spring and dashpot coefficients. As the sole purpose of the paper is not to discuss more on the shallow foundations, the interested readers are encouraged to refer the NIST SSI report.

Deep foundations

Deep foundations using the discrete spring modelling approach are mainly modelled using the p-y curves. Figure 6.0 represents the generation of p-y curve.

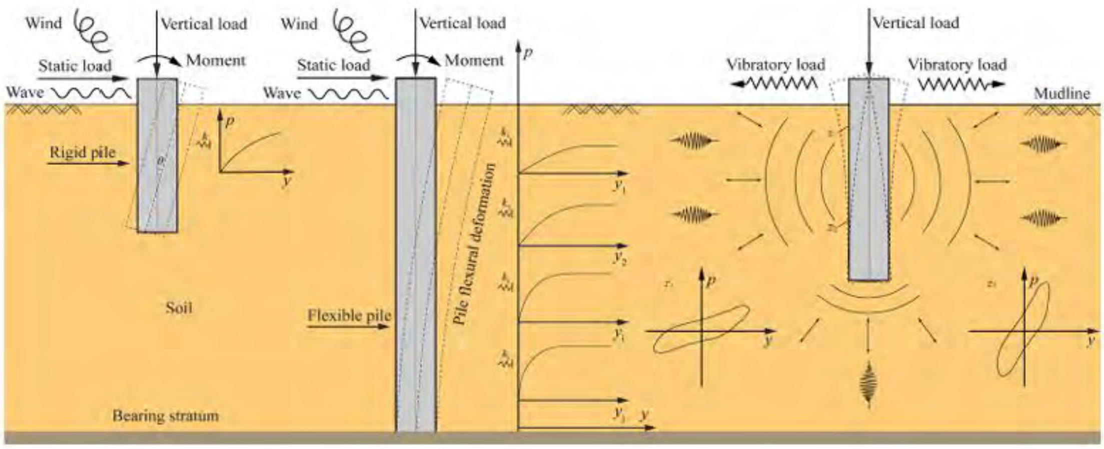

Figure 6.0 schematic generation of p-y curve (Ref Wu et al. 2024)

There are predominantly two types of p-y curves. One is the static p-y curve and the other one is the cyclic p-y curve. As the names suggest, static p-y curves are derived for static loading and cyclic p-y curves are derived for cyclic loading where the cyclic shear action between the pile and the soil is captured more realistically. For more details on this refer to Wu et al. (2024) which presents a state-of-the-art literature study.

Both of these approaches suffer from the fact the soil data available is frequency dependant whereas NLTHA requires time-dependant or displacement-dependant data which, generally, seems to be unavailable.

A TYPICAL STRUCTURAL PROGRAM FRAMEWORK

To put the issues that are going to be discussed in later section into context, a brief overview of the mechanics of time history analysis framework that is normally implemented in a structural program is presented.

Most computer programs for seismic engineering employ implicit time integration schemes to solve the dynamic equilibrium equation for the system.

Though there are a series of methods, most computer programs implement a version of the popular classical Newmark constant average acceleration method. In this section we present a brief overview of the method to highlight how the structural program considers springs.

The fundamental assumption in the classical Newmark formulation is that the acceleration is assumed to be constant during the time step with a value equal to the average of the accelerations at the beginning and end of the time step. The classical Newmark method starts with the difference in the response between two successive time steps ΔT apart. So the equation of motion at time t with $\{u(t)\}$, $\{\dot{u}(t)\}$ and $\{\ddot{u}(t)\}$ representing, displacement, velocity and acceleration becomes,

$$[M]\{\ddot{u}(t)\} + [C]\{\dot{u}(t)\} + [K]\{u(t)\} = \{P(t)\} \qquad 1$$

At time $(t + \Delta T)$,

$$[M]\{\ddot{u}(t+\Delta T)\} + [C]\{\dot{u}(t+\Delta T)\} + [K]\{u(t+\Delta T)\} = \{P(t+\Delta T)\} \qquad 2$$

Taking the difference between these two equations would result in,

$$[M_t]\{\Delta\ddot{u}\} + [C_t]\{\Delta\dot{u}\} + [K_t]\{\Delta u\} = \{\Delta P\} \qquad 3$$

In the above equation, M_t , C_t and K_t respectively are the tangent mass, tangent damping and the tangent stiffness matrices. Eq. 3 is in incremental equilibrium with $\{\Delta\ddot{u}\}$,$\{\Delta\dot{u}\}$ and $\{\Delta u\}$ being the corresponding acceleration, velocity and displacement increments. Eq. 3 has *n* equilibrium equations and *3n* unknowns including *n* displacements, *n* velocities and *n* accelerations. So, this incremental form of the equation can only be solved by expressing a relationship between the displacement, velocity and acceleration.

After mathematical manipulations, we get,

$$[\hat{K}_t]\{\Delta u\} = \{P(t+\Delta T)\} + [M]\left\{\{\ddot{u}(t)\} + \frac{4}{\Delta T}\{\dot{u}(t)\}\right\} + 2[C_t]\{\dot{u}(t)\} - [C]\{\dot{u}(t)\} - [K]\{u(t)\} \qquad 4$$

where dynamic or augmented stiffness $[\hat{K}_t]$ is given as,

$$\left[\hat{K}_t\right] = \frac{4}{\Delta T^2}\left[M_t\right] + \frac{2}{\Delta T}\left[C_t\right] + \left[K_t\right] \qquad 5$$

In all structural analysis software, eq. 5 is implemented and iteratively solved at every time-step during the analysis.

MISCONCEPTIONS WHILE USING DISCRETE SPRINGS WHEN MODELLING DEEP FOUNDATIONS

As described in previous section, cyclic *p-y curves* are commonly generated by geotechnical engineers when asked for soil springs by structural engineers to do NLTHA. Now let's look at the cyclic *p-y curves* more closely in conjunction with eq. 5.

Figure 8.0 shows a typical set of cyclic *p-y curves.*

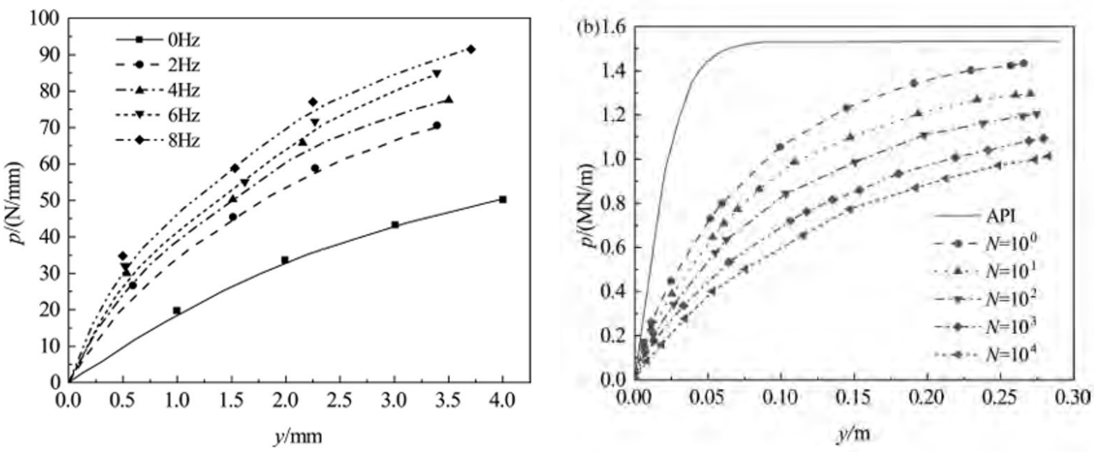

Figure 8.0 a typical set cyclic *p-y curve* with differing loading frequencies and cycle numbers (Ref Wu et al. 2024)

As shown in Figure 8.0, the *p-y curve* generated depends on the number of cyclic loads and loading frequencies. It also depends on load amplitude, loading type, long-term load action, pile type, pile diameter, and embedment depth. As shown in Figure 8.0, P-y curves, essential for modelling soil-structure interaction, are inherently complex as they are influenced by a lot of factors including frequency dependence and the number of loading cycles. When considering a suite of earthquakes, each event presents a unique spread of frequencies and varying predominant cycles. So, for each earthquake the net P-y curve applied should ideally reflect all these above factors including the frequencies and the cumulative number of cycles experienced. This means that every earthquake in the suite will/should have a unique definition of the curve in such a manner that it reflects all aspects including the frequency and cycle dependance.

The pressing question is *how can a unique spring definition be achieved by incorporating all these dependencies (frequency, number of cycles, loading amplitudes etc.) and structural engineers implement the same in a structural software platform?* To answer this question, let's first review what happens when structural engineer applies the spring given by the Geotechnical engineer. When structural engineers apply the springs given by a Geotechnical Engineer in a structural software platform, what happens is that the spring gets assembled in the $\left[K_t\right]$ matrix given in eq. (5). This assemblage means that the entire frequency, cycle count dependance, Load amplitude dependence etc. of the *p-y curve* is completely ignored. As any given ground motion will have a spread of frequencies, differing cycle numbers, load amplitudes etc., this will generate a high order of uncertainty in the observed responses. The degree of uncertainty generated by such an application is an order of magnitude more than what is normally acceptable in seismic designs in general. This means that to have a realistic SSI model the structural analysis should be able to switch back between these curves or the program should somehow have a capability to account for all these dependencies so that at any given instant the program can compute or uniquely formulate a P-y function. This is almost an impossible task as per the current state of the art of structural programs.

Studies like those from Rahmani et al. (2018) indicate that, regardless of whether static or more advanced dynamic p-y curves are used, the spring-based method fails to capture the true dynamic soil-pile interaction under seismic loading. The main reason for this is the above shortcomings of the structural programs.

So, considering all these aspects, the pressing question is, *for Performance Based Seismic Design of structures with deep foundations, is it appropriate to use dynamic p-y curves?* The authors believe that more careful consideration need to be made by structural engineers when making judgements and they need to be more aware of how the computation is carried out. From a mathematical point of view, the incorporation of these soil springs into the structural software has a clear disconnect due to these dependancies of their properties. There needs to be a holistic research approach where the gaps in the knowledge with respect to the implementation of the soil springs into the structural software platform can be alleviated.

PRACTICAL ISSUES WITH THE USE OF FINITE ELEMENT MODEL FOR SSI

As mentioned earlier, SSI modelling using FEM is more realistic than using discrete spring modelling approaches, mainly because the FEM provides a better description of the strength, stiffness and participating soil mass along with structural induced dynamics. These analyses seem more realistic, but they have a greater number of complexities which may have great implications for the results of the studies. To demonstrate this, Figure 9.0 shows a typical set of structures. Suppose we want to incorporate SSI using FEM for one of the long structures, the standard approach is shown in Figure 10.0 where the soil and structure is modeled using Finite Elements. Though this looks realistic and sophisticated, *the primary problem is that the model only models the structure as a stand-alone structure with no interferences from the neighboring structure*. This is far from reality and the conclusions drawn based on such modelling have no realism. To obtain realistic results from FEM based SSI modelling, a decision needs to be made as to how far the zone of influence is which may necessitate the entire set of neighboring structures also being included in the analysis model. See Rahman, Carr & Moss.

The other major difficulty is in the representation of the boundaries of the finite element model of the soil system. The foundation is part of what is an effectively infinite domain, and unless some form of transmitting, or silent, boundary can be deployed energy will be trapped within the finite element domain. Unfortunately, most silent boundary models are based on frequency domain solutions rather than the time-domain models required by the NLTHA programs.

Figure 9.0 a google image of a typical building group

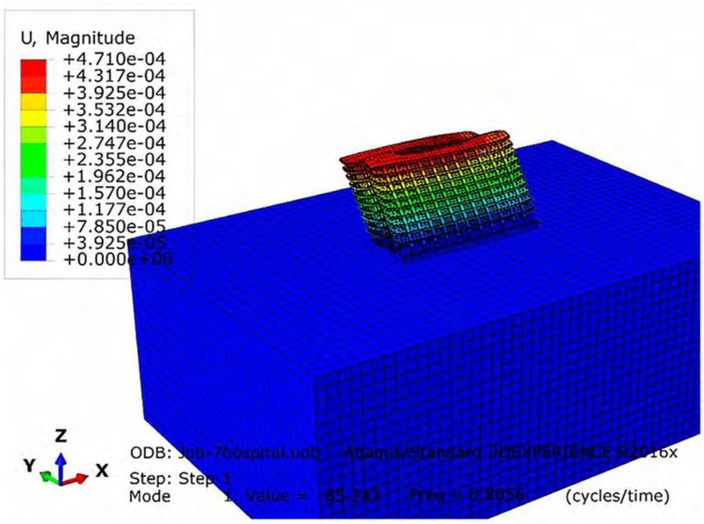

Figure 10.0 SSI incorporated FEM model of the long structure in Figure 9.0

Due to the availability of sophisticated FEM software, there is a growing trend in the industry to directly consider SSI within NLTHA as a means of assessing existing structures and arriving at what is deemed to be a “more realistic rating”. The use of “more realistic rating” is in many instances used as a justification for not doing work in the foundations or additional foundations etc., which would otherwise have been needed when considering a fixed based model. Also, most of the models look similar to what is depicted in Figure 10.0, with no acknowledgement of the effects of the surrounding structures. So, this so-called realism achieved by complex SSI incorporated FEM modelling becomes an illusion. The authors strongly believe that if FEM based SSI is used for structures in cityscape, then the effects of the neighboring structures should be included in the model along with uncertainty quantification of the parameters or characterization of the elements that have been included in the model.

There are instances where even simple SSI models provide a very powerful insight into the structural response, see Moss & Carr.

The conclusions drawn from the SSI analyses should take into account the uncertainties in the modelling on the finite nature of the boundaries of the finite element domain, the effects of any other structure that may have a bearing on the response of the subject structure as well as the uncertainties inherent in the material properties of the soil elements that have been included in the model. As most of the three-dimensional finite elements in use have relatively limited deformation capabilities compared with those available in most current structural analysis software for modelling superstructures, the effects of what is usually a relatively coarse mesh

for the soil foundation compared to that used in the modelling of the superstructure also needs to be kept in mind.

Whilst there are serious reservations in the use of these very elaborate foundation models, provided the objective of the NLTHA studies are well defined, and the limitations of the available soil data and models together with the limitations of the finite dimension models in what is intrinsically an infinite domain is acknowledged, the results of these analyses can provide a very valuable insight into the effects of the supporting foundation on the response of the structure.

As commented earlier, even quite simple SSI models can be used to provide a very powerful insight into the structural response.

CONCLUSION

The challenges associated with incorporating explicit Soil-Structure Interaction (SSI) into a structural engineering compliance pathway have been critically discussed. Through a theoretical review of the mechanics underlying structural dynamics software frameworks, it has been demonstrated that the use of *p-y curves* to account for Pile-Soil-Structure Interaction effects requires further research and validation. As such, relying on *p-y curves* as a method for modelling SSI in deep foundations may result in a degree of uncertainty which is an order of magnitude more than acceptable in general seismic design. Hence this approach is premature and should be approached with extreme caution.

If the Finite Element Method (FEM)-based SSI analysis is adopted for structures in a built-up area, it could be important to incorporate, or at least consider, the inertial interaction effects of neighboring structures in the model to achieve more realistic and accurate results. The effects of the finite domain of the computer model compared to the effectively infinite domain of the real foundation need to be assessed. Failing to account for these interactions can lead to outcomes that do not reflect real-world behaviour, potentially resulting in unconservative conclusions—such as decisions to avoid foundation enhancement—that could compromise structural safety and performance. Such conclusions should therefore be drawn only with extreme care and consideration of the limitations inherent in the current methodologies. Regardless of whether SSI is included or not, for uncertainty quantification purposes, responses based on fixed base models had to be treated as a primary median model for the design. Also, in all cases involving SSI, extensive uncertainty quantification and de-sensitization (minimizing parametric sensitivity) of the design need to be performed.

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QIC Motorsports and Speed Park: What I learned designing the world's next Formula One racetrack

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ABSTRACT

The Government of Saudi Arabia is undertaking the ambitious development of Qiddiya, a groundbreaking entertainment city set in the heart of the Arabian desert. Designed to become a global hub for entertainment, sports, and culture, Qiddiya's key attraction is the Speed Park and Motorsports precinct, which will contain a brand new Formula One racetrack. This world-class track weaves through surrounding theme parks, promenades, and hospitality spaces to create an immersive experience for both spectators and drivers. The track also features a world-first cantilevered track section, elevated 70 meters above the ground, known as the Blade.

WSP Middle East has led the structural design of twelve buildings across the Speed Park and Motorsports precinct. WSP New Zealand has provided support with the design of embedded connections for four of these buildings. These structures, composed of hybrid steel-concrete systems, required the design of over thirteen hundred steel-to-concrete connections, each presenting unique engineering challenges.

This paper explores the complexities of designing these critical connections, emphasising the role of design automation in streamlining the process through customised computational tools. It also examines the challenges of adapting to international design codes as an emerging professional and the importance of strong cross-border collaboration. Additionally, this paper will discuss the close coordination between engineers and drafters required to ensure efficiency and precision in a project of this scale.

INTRODUCTION

WSP New Zealand were engaged by WSP Middle East to provide design assistance on the QIC Motorsports and Speed Park project; a brand-new Formula One track being built in the Saudi Arabian city of Qiddiya. Developed by the Qiddiya Investment Company (QIC), the new purpose-built city of Qiddiya is set to host many notable sporting events including the 2034

FIFA World Cup, and the Saudi Arabian Grand Prix. Working on a project of a scale rarely seen in New Zealand provided a unique opportunity for 'on the job learning' - a key aspect of the development of junior engineers early on in their careers. This paper explores the challenges encountered in the delivery of the project from the perspective of an emerging professional. This included:

- Embedded connection design
- Design management and collaboration, both between engineers and with technicians
- Design automation
- Learning American and Saudi Arabian design standards

PROJECT BACKGROUND

The city of Qiddiya, developed by Qiddiya Investment Company (QIC), is an entertainment destination 45 kilometres from the Saudi Arabian capital, Riyadh. Designed as a global hub for entertainment, sports, and culture, Qiddiya is a cornerstone project of Saudi Arabia’s Vision 2030 initiative, aiming to diversify the country’s economy and create new opportunities for tourism and recreation.

One of Qiddiya’s premier attractions is the Motorsport and Speed Park precinct, which will feature a brand-new Formula One racetrack. Starting in 2027, this track is set to replace the current circuit in Jeddah as the host of the Saudi Arabian Grand Prix.

The Motorsports and Speed Park Precinct contains a total of twelve buildings as shown in Figure 1 and listed below.

- Primary Pit (J1)
- Secondary Pit (K1)
- Motorsports Experience Centre (N1)
- Clubhouse (G1)
- Service Undercroft (U1)
- Arena Town Square (D1)
- World of Bikes (F1)
- Museum (E1)
- Subzero to Sixty (C1)
- Gravity Karting (T1)
- Medical Centre (V1)
- Operational Compound (W1)

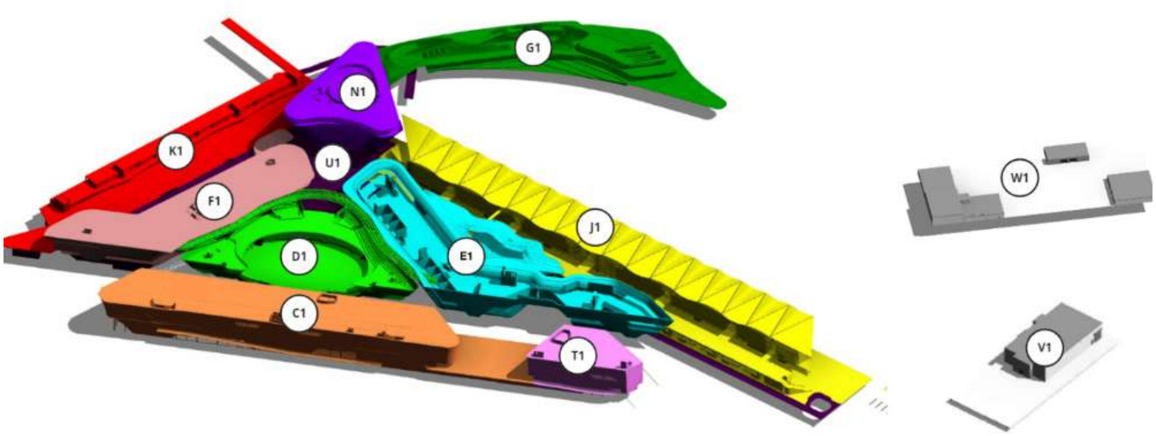

Figure 1: Map of all buildings in QIC Motorsports and Speed Park Precinct.

All buildings within the Motorsport and Speed Park precinct comprise hybrid steel and concrete systems. Flooring systems are typically in-situ concrete slabs spanning to structural steel beams and/or long-span trusses. Beams and trusses are then supported by vertical elements (columns and access/shear cores) comprising reinforced concrete, a decision made in order to keep steel tonnage, and therefore building costs, down. Foundations are a mix of rafts, strip footings and piles, depending on the geotechnical environment for each building. WSP New Zealand was allocated the design of the embedded connections between the structural steel beams/truss and the reinforced concrete vertical structural elements for the Primary Pit (J1), Gravity Karting (T1), Subzero to Sixty (C1), and the lower levels of the Motorsports Experience Centre (N1). This resulted in over 1,300 embedded steel connections of varying complexity to be designed and documented.

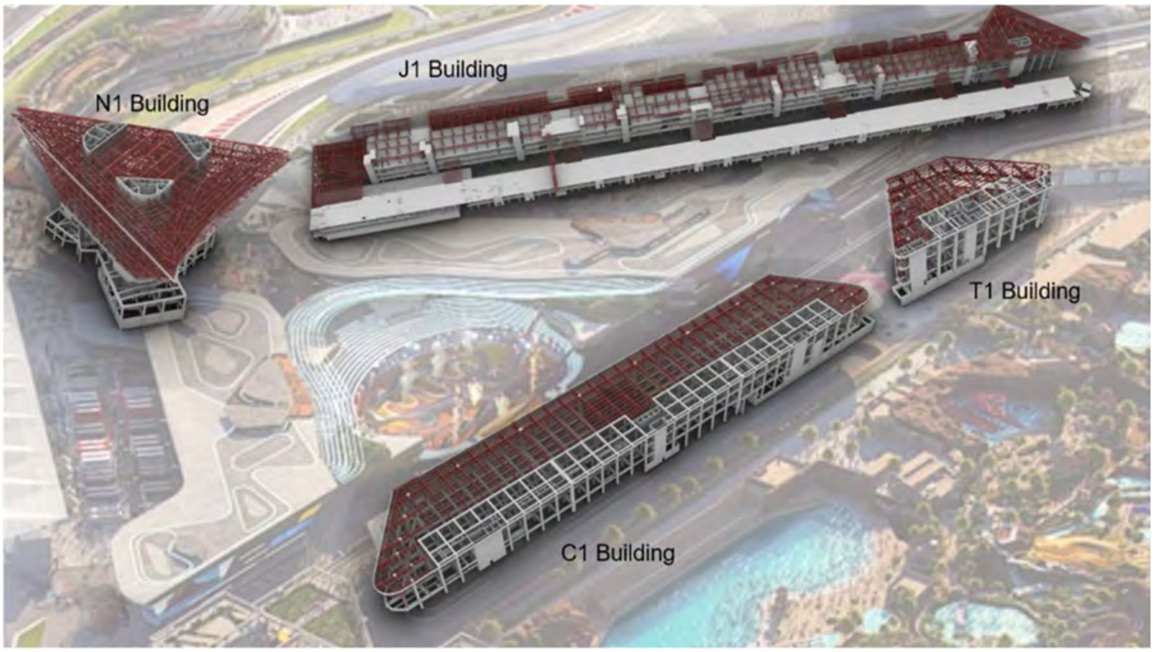

Figure 2: Overview of buildings WSP NZ worked on

Engineering Challenges & Design Solutions

To effectively manage the design of 1,300 steel-to-concrete connections across four buildings, the connections were categorised into categories with similar design processes. First, they were split into two broad categories, ‘simple’ and complex’.

Simple connections were those that could generally be designed with hand calculations only and included shear-only connections into walls or beams, moment connections into beam-column joints or walls, baseplates, and lift beam connections. The solution implemented for the simple connections typically included cast-in plates with a combination of shear studs and welded reinforcement. In cases with higher demands, backing plates were used to help carry tension loads back into the concrete. These connections had high repeatability and standardisation across all buildings.

Complex connections were connection types with complicated geometric constraints which generally required some form of finite element analysis. Examples of complex connection scenarios include where beam/truss members node into reinforced concrete columns or beams (often on multiple axes), four-way steel beams nodding into a single reinforced concrete column. Sliding link bridge steel connections into reinforced concrete corbels, and pinned bridge connections into shallow reinforced concrete beams. In more complex cases, a number of embedded steel sections were used to help carry high tension loading into the existing structure.

The nature of the contractual arrangement on the project meant that WSP was responsible for the design of all structural elements up the face of the columns/walls, while the final design of the incoming beams/trusses was the responsibility of the steel fabricator. This meant WSP had to work with incoming structural member sizes that were 'locked in' and not subject to change. A summary of the different connection types is shown in Figure 3.

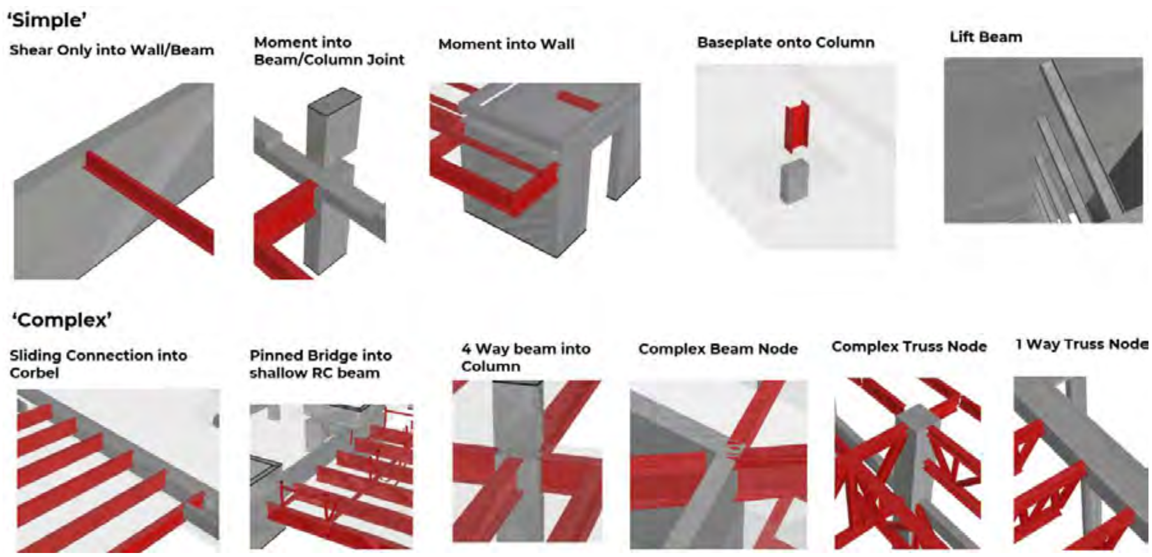

Figure 3: Summary of different 'simple' and 'complex' connections

Design Automation

As a junior engineer on the project, I was responsible for the simple connection packages, which included the design of over 500 shear-only connections. To handle such a large number of designs, and cater for varying input parameters for each connection, an automated procedure for cast-in plate design was developed in Excel.

First, a spreadsheet for analysing and designing cast-in plate to the Saudi concrete code was developed. Then, a batch processing system, allowing many design instances to be processed simultaneously, was implemented. The process generated individual calculation reports (i.e. spreadsheets) for each individual connection.

The design spreadsheet required several key inputs, including:

- Loading demands from ETABS for 454 individual load cases
- Steel beam size
- Geometry of the support concrete element, including edge distances
- Cast-in plate size
- Existing reinforcement within the concrete member

Revit Tag	Worksheet Name	Beam Depth	Minimum depth of fin plate	Cast-in Plate Type	CIP Depth	CIP Set-out	Axial load from the beam to the fin plate (worst case combo)	Shear load from the beam to the fin plate (worst case combo)	Edge distance 'west'	Edge distance 'east'	Edge distance 'north'	Edge distance 'south'	Concrete thickness	Concrete strength	Concrete cracked a services i
			$d_{min,fin}$				N_{beam}	V_{beam}	c_{west}	c_{east}	c_{north}	c_{south}	h_c	f'_c	
J1-L03-BM-001	001	610	397	B06-04	480	Centred	-17.7	144.1	-100	1900	305	305	400	40	
J1-L03-BM-003	003	457	297	A04-00	280	Top Flush	-6.6	141.3	1900	1900	240	417	400	40	
J1-L03-BM-004	004	457	297	A04-00	280	Top Flush	-5.3	141.3	1900	1900	240	417	400	40	
J1-L03-BM-005	005	610	397	C06-03	560	Top Flush	0.7	288.3	1925	1925	305	355	750	45	
J1-L03-BM-009	009	610	397	C06-03	560	Top Flush	-115.6	478.1	125	125	2105	2155	800	45	
J1-L03-BM-012	012	610	397	C06-03	560	Top Flush	-120.0	481.7	125	125	2105	2155	800	45	
J1-L03-BM-015	015	610	397	C06-03	560	Top Flush	124.3	346.1	125	125	2105	2155	800	45	
J1-L03-BM-018	018	610	397	C06-03	560	Top Flush	119.9	335.0	125	125	2105	2155	800	45	
J1-L03-BM-019	019	406	264	A04-00	280	Top Flush	20.2	113.6	1900	1900	240	366	400	40	
J1-L03-BM-020	020	406	264	A04-00	280	Top Flush	18.8	142.2	1900	1900	240	366	400	40	
J1-L03-BM-021	021	406	264	A04-00	280	Top Flush	17.9	105.0	1900	1900	240	366	400	40	
J1-L03-BM-023	023	610	397	C06-05	560	Top Flush	35.4	172.1	10	1900	280	330	400	40	
J1-L03-BM-025	025	457	297	C06-03	560	Top Flush	41.6	132.4	125	125	2105	2002	800	45	
J1-L03-BM-026	026	457	297	C06-03	560	Top Flush	42.2	136.4	125	125	2105	2002	800	45	
J1-L03-BM-027	027	457	297	A06-00	480	Top Flush	19.5	135.4	1900	1900	240	217	400	40	
J1-L03-BM-028	028	914	594	C08-00	780	Top Flush	107.1	459.8	1900	1900	280	434	400	40	
J1-L03-BM-029	029	1000	650	C08-00	780	Top Flush	-52.9	730.5	1900	1900	280	520	400	40	
J1-L03-BM-030	030	457	297	A06-00	480	Top Flush	16.3	109.8	20	1900	240	217	400	40	
J1-L03-BM-031	031	610	397	C08-03	760	Top Flush	122.7	220.0	10	1900	280	130	400	40	
J1-L03-BM-032	032	610	397	C08-00	760	Top Flush	-59.6	323.2	1900	1900	280	130	400	40	
J1-L03-BM-033	033	610	397	C06-03	560	Top Flush	120.5	540.4	125	125	2105	2155	800	45	
J1-L03-BM-034	034	700	455	C06-03	560	Top Flush	319.4	194.4	125	125	2105	2245	800	45	
J1-L03-BM-035	035	610	397	C06-03	560	Top Flush	-108.4	437.1	125	125	2105	2155	800	45	
J1-L03-BM-036	036	610	397	C06-03	560	Top Flush	94.3	432.8	125	125	2105	2155	800	45	
J1-L03-BM-037	037	457	297	A06-00	480	Top Flush	-3.4	135.4	1900	1900	240	217	400	40	
J1-L03-BM-038	038	457	297	A06-00	480	Top Flush	16.3	127.3	1900	1900	240	217	400	40	
J1-L03-BM-039	039	457	297	A06-00	480	Top Flush	14.0	164.0	1900	1900	240	217	400	40	
J1-L03-BM-040	040	900	585	C08-00	780	Top Flush	-136.0	567.0	1900	1900	280	420	400	40	
J1-L03-BM-041	041	700	455	C06-04	560	Top Flush	93.8	433.7	1900	1900	280	420	400	40	
J1-L03-BM-042	042	700	455	C06-04	560	Top Flush	70.3	313.5	10	1900	280	420	400	40	
J1-L03-BM-044	044	533	346	C06-03	560	Top Flush	2.9	313.7	125	125	2105	2078	800	45	
J1-L03-BM-045	045	533	346	C06-03	560	Top Flush	0.4	314.0	125	125	2105	2078	800	45	
J1-L03-BM-046	046	914	594	C08-01	780	Top Flush	40.7	213.2	125	125	2105	2259	800	45	

CONNECTIONS | Anchor Data | CAST-IN-PLATES | DEMANDS | EDGE DISTANCES | Calculation Master | 001 | 003 | 004 | 005 | 009 | 012 | 015 | 018 | 019

Figure 4: Screenshot of cast-in plate spreadsheet design tab showing key inputs.

The master spreadsheet then produced individual calculation sheets for each unique connection and consolidated the results in the main tab. This provided a summary of which connections were passing or failing and their subsequent utilisation ratios.

Revit Tag	Cast-in Plate Type	Tension	Shear	Combined	Robustness	Pryout Included in design checks	Plate Bending Failure Mode	Tension	Shear	Combined	Robustness	Pryout Included in design checks
		$N_{ua,top}/\phi N_{n,top}$	$V_{ua,top}/\phi V_{n,top}$	$N_{ua,top}/\phi N_{n,top}+V_{ua,top}/\phi V_{n,top}$	$(N_{robust}/n_{rows})/\phi N_{n,top}$			$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	$N_{ua}/\phi N_n+V_{ua}/\phi V_n$	$N_{robust}/\phi N_n$	
J1-L03-BM-001	B06-04	-0.11	0.56	0.45	-0.21	Yes	Mode 3	0.09	0.26	0.35	-0.48	No
J1-L03-BM-003	A04-00	0.19	0.51	0.70	0.30	Yes	Mode 3	-0.02	0.73	0.71	0.30	Yes
J1-L03-BM-004	A04-00	0.18	0.51	0.69	0.30	Yes	Mode 3	-0.02	0.59	0.57	0.30	Yes
J1-L03-BM-005	C06-03	0.13	0.30	0.43	0.17	Yes	Mode 3	0.00	0.65	0.65	0.54	Yes
J1-L03-BM-009	C06-03	0.10	0.49	0.59	0.31	No	Mode 3	-0.33	0.60	0.27	0.92	No
J1-L03-BM-012	C06-03	0.10	0.73	0.83	0.31	Yes	Mode 3	-0.35	0.60	0.26	0.93	No
J1-L03-BM-015	C06-03	0.31	0.52	0.83	0.22	Yes	Mode 3	0.36	0.43	0.79	0.67	No
J1-L03-BM-018	C06-03	0.30	0.51	0.80	0.21	Yes	Mode 3	0.35	0.42	0.77	0.64	No
J1-L03-BM-019	A04-00	0.24	0.41	0.65	0.25	Yes	Mode 3	0.13	0.53	0.66	0.51	Yes
J1-L03-BM-020	A04-00	0.28	0.51	0.80	0.32	Yes	Mode 3	0.13	0.66	0.79	0.63	Yes
J1-L03-BM-021	A04-00	0.22	0.38	0.60	0.23	Yes	Mode 3	0.12	0.49	0.61	0.47	Yes
J1-L03-BM-023	C06-05	0.44	0.39	0.83	0.42	Yes	Mode 3	0.24	0.72	0.96	0.77	Yes
J1-L03-BM-025	C06-03	0.17	0.20	0.37	0.08	Yes	Mode 3	0.12	0.54	0.66	0.25	Yes
J1-L03-BM-026	C06-03	0.17	0.21	0.38	0.09	Yes	Mode 3	0.12	0.56	0.68	0.26	Yes
J1-L03-BM-027	A06-00	0.25	0.33	0.57	0.19	Yes	Mode 3	0.08	0.44	0.52	0.36	Yes
J1-L03-BM-028	C08-00	0.27	0.34	0.60	0.31	Yes	Mode 3	0.22	0.75	0.96	0.62	Yes
J1-L03-BM-029	C08-00	0.28	0.53	0.82	0.49	Yes	Mode 3	-0.11	0.56	0.46	0.98	No
J1-L03-BM-030	A06-00	0.58	0.41	0.99	0.43	Yes	Mode 3	0.12	0.69	0.82	0.49	Yes
J1-L03-BM-031	C08-03	0.75	0.30	1.05	0.28	Yes	Mode 3	0.23	0.39	0.62	0.28	No
J1-L03-BM-032	C08-00	0.04	0.24	0.27	0.08	Yes	Mode 3	-0.07	0.83	0.76	0.25	Yes
J1-L03-BM-033	C06-03	0.35	0.82	1.16	0.30	Yes	Mode 3	0.30	0.68	0.98	0.90	No
J1-L03-BM-034	C06-03	0.29	0.29	0.58	0.11	Yes	Mode 3	0.80	0.24	1.05	0.33	No
J1-L03-BM-035	C06-03	0.09	0.66	0.75	0.28	Yes	Mode 3	-0.31	0.55	0.24	0.84	No
J1-L03-BM-036	C06-03	0.32	0.65	0.97	0.28	Yes	Mode 3	0.27	0.54	0.82	0.83	No
J1-L03-BM-037	A06-00	0.19	0.33	0.52	0.20	Yes	Mode 3	-0.02	0.45	0.43	0.55	Yes
J1-L03-BM-038	A06-00	0.24	0.31	0.55	0.19	Yes	Mode 3	0.10	0.43	0.53	0.52	Yes
J1-L03-BM-039	A06-00	0.29	0.40	0.68	0.24	Yes	Mode 3	0.09	0.55	0.63	0.67	Yes
J1-L03-BM-040	C08-00	0.12	0.41	0.54	0.27	Yes	Mode 3	-0.19	0.92	0.72	0.53	Yes
J1-L03-BM-041	C06-04	0.23	0.42	0.66	0.27	Yes	Mode 3	0.26	0.83	1.09	0.81	Yes

Figure 5: Utilisation Ratios for Connections shown in spreadsheet.

Cast-in plate types were designed based on the depth of the steel beam member and the magnitude of the loading. They were classified into three types depending on the anchor configuration:

- Shear studs only (A)
- Shear studs with a top row of headed bolts (B)
- Headed bolts only (C)

Variations within these types depended on the number of anchor rows and the length of the headed bolts. Generally, type A cast-in plates were for those with low shear demands, type B were for these with higher shear demands (that generated significant tension in the top row of anchors due to the eccentricity of the applied shear force), and type C were for those with high axial demands.

Through the implementation of design automation, the WSP New Zealand team efficiently designed a large volume of connections while improving accuracy and reducing manual input. One of the key advantages was the ability to conduct independent checks on all 454 load cases from ETABS, rather than relying on worst-case envelope loading for shear and tension. This allowed for a more precise representation of worst-case combined actions, preventing overdesign and optimising the design of each connection. Another major benefit of automation was significant time savings, particularly given the iterative nature of design and the sheer volume of connections. Quick turnaround times were essential when design changes occurred, especially when coordinating with the WSP Middle East team. One such challenge arose when the ETABS models, developed by the WSP Middle East team and handed over to the WSP New Zealand team, required updates after design work was already underway. Since these changes affected loading demands for all connections, the automation process made it significantly easier to update the spreadsheet as manual adjustments were limited to modifying the demands tab, rather than re-entering data for each connection individually. As the project progressed, the cast-in plate spreadsheet was further refined to enhance efficiency in other connection scenarios. An example of this was the moment-modified cast-in plate spreadsheet, which allowed moment connections with lower demands to be designed using the same batch process, significantly reducing design time and improving efficiency.

During the QIC project, several challenges arose in implementing design automation. One of the primary difficulties was managing the large volume of data. In one instance the limit of Excel rows within a single worksheet was exceeded by the spreadsheet requiring the WSP New Zealand team to manually modify the data once it went beyond 1,048,576 rows. While automation significantly reduced design time, manual input was still required for edge distances, beam sizes, and existing reinforcement. Processing ETABS loading data was also complex due to the high number of load cases and the need to correctly identify member ends before extracting the demands. Additionally, setting up the batch processing procedure required substantial technical expertise in Microsoft Excel and Visual Basic coding. However, despite these initial challenges, the investment in automation ultimately provided significant time savings and demonstrated the value of implementing automation tools early in a project to improve efficiency in later stages.

Adapting to International Design Codes

Adapting to the Saudi Building Code was one of the key challenges faced on the project, as it was the first time in my career working with international design standards. The Saudi Concrete Code (SBC 304) and the New Zealand Concrete Code (NZS 3101) in relation to cast-in embedded anchors and concrete bearing are both based on the American Concrete Institute (ACI 318). One key difference between NZS 3101 and SBC 304 is the stringent seismic design considerations in NZS 3101. SBC 304 does include seismic design provisions however it differs in classification and detailing requirements. For the case of the QIC project the location was in a low seismic zone and hence seismic considerations were not necessary. For embedded anchor design, both codes are based on the ACI 318 equations, with the primary difference being the strength reduction factors as shown in Table 1. SBC 304 applies less onerous strength reduction factors for most shear and tension failure modes.

Table 1: Strength reduction factors in NZS 3101 and SBC 304

Failure Mode	NZS 3101 φ Factors	SBC 304 φ Factors
Concrete Breakout in Tension	0.70	0.75
Concrete Pullout	0.70	0.75
Concrete Side-Face Blowout	0.70	0.70
Concrete Breakout in Shear	0.70	0.75
Shear Pryout	0.70	0.75
Bearing Strength	0.65	0.70

Working with SBC 304 in comparison to NZS 3101 provided valuable lessons as an emerging professional, particularly in understanding the fundamental theory behind building code equations. Fundamentally, the embedded connection design provisions both SBC 304 and NZS 3101 are based on the research used to develop ACI 318 and, hence, share the same core design principles. While their overall methodologies remain consistent, detailed understanding was still essential as small variations in factors, equations, and material assumptions must be understood to ensure compliance with local requirements. This experience reinforced the importance of attention to detail in design work, while also demonstrating that a solid understanding of design fundamentals enables the ability to work across geographical boundaries.

Cross-Border Collaboration & Coordination

Effective communication and coordination were essential to the success of the QIC project. One key example was our cross-border coordination between the WSP Middle East team, whom led the structural design of the QIC Project, and the WSP New Zealand team. Regular design meetings were scheduled with the WSP Middle East team, while informal communication via Microsoft Teams typically took place outside of New Zealand's business hours, either early in the morning or later in the evening.

Collaboration became essential when design challenges arose, specifically in the case of updating ETABS models. On occasion, the New Zealand team identified inconsistencies between the drawings and ETABS models, requiring clarification from the Middle East team before work could proceed. Additionally, clear communication to understand modelling assumptions (particularly those around construction staging impacts and ‘locked in’ demands) was critical to understanding when stress concentrations in the model were ‘real’ or just an artifact of the finite element meshing. Once design packages were issued, the WSP Middle East team handled Requests for Information (RFIs) from the contractor on behalf of WSP New Zealand, with the New Zealand team being contacted when additional clarification was required for the connection design.

Another critical area where collaboration was essential was between engineers and technicians within the WSP New Zealand team. As a junior engineer, the QIC project marked my first significant experience working closely with the drafting team to ensure connections were accurately modelled and detailed. Given the scale and complexity of the project, geometric constraints often dictated design solutions. After developing an initial series of connections for modelling, issues were sometimes identified due to limitations such as insufficient edge distances from steel plate to edge of concrete, clashes with existing reinforcement, or geometric constraints of concrete members. Such connections often required redesign and iteration with the technicians, with adjustments to eccentricity to ensure proper fit. Managing this iterative process on a project of this scale was challenging for both engineers and technicians, making clear communication and collaboration essential for efficiency and accuracy. One solution was to leverage the experience of the technician to flag potential geometric conflicts as a first step in the design process therefore minimising back-and-forth revisions later. Finally, due to the large number of connections requiring documentation, a standardised connection schedule was implemented whereby standard details were created for connections and specific parameters were referenced from a table.

Conclusion

Across four buildings of the groundbreaking QIC Motorsports and Speed Park Precinct, the WSP New Zealand team designed over 1300 steel-to-concrete embedded connections over eight months. The complexity of the project required unique engineering solutions where geometries and loading conditions were highly challenging. Design automation was key to ensuring efficient and accurate designs could be achieved within the required timelines.

As an emerging professional, this paper also reflects on the experience of working on an international project, particularly the challenges of adapting to a new building code. It highlights how collaboration between the WSP Middle East and WSP New Zealand teams, and between engineers and technicians, played an essential role in ensuring accurate and efficient delivery. Working on the QIC project has provided invaluable insights into international project workflows, design office coordination, and the expanding role of design automation in the engineering industry.

SEISMIC NSE HICCUPS & SOLUTIONS FROM BUILDING SERVICES PERSPECTIVE

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ABSTRACT

The structure of any building in New Zealand is governed by building Act and NZ Building Code B1 – Structure. We have our own *NZ Standard 1170 – Structural Design Actions* particularly part 5: 2004 which outlines the *Earthquake Actions – New Zealand.* The main reason to refer to these standards are to determine the *Ultimate limit State* (*Eu*) and Serviceability *Limit State* (*Es*).

When touching the non-seismic elements (NSE) of a structure system, it falls under *NZS 4219:2009 – Seismic Performance of Engineering Systems in Buildings*. This standard is for specific design of the NSE of the structure elements. However, it doesn't cover all types of Fire and Mechanical services. Such as but not limited to,

1. Fire Sprinkler System
2. Suspended Ceilings

These are just some of the examples and they have their own standards to comply with the NZBC B1 and of course the NSE of any structure in New Zealand.

As per NZS 4219:2009, there are 2 x pathways to achieve NSE for any type of building,

1. Specific Design Method

- Such designs are drafted by experience structural engineers such as CPEng as most of them falls under either AS or VM of the NZBC B1. However, it may touch to other Building codes such as *NZBC G10- Piped Services* and/or *NZBC G12 – Water Supplies*. These types of design could have generic approach on the NSE components from experience of the registered engineer.
-

2. Non-Specific Design Method

- Such designs could be done by NSE specialists as well as by the building services engineers such as Mechanical Engineer or Fite Engineer. Most of the NSE designs out there are done by this way determining the different parameters of the building and finally calculating the required restraint forces for all of the NSE components in question.

Guidance has been provided by BRANZ on the responsibilities associated with the producer statements, as below,

#	TYPE	BY	COMMENT
PS1	Design	Seismic Specialist or Building Services Contractor	Certifies that the proposed works, if constructed as designed, will comply with the requirements of the NZBC.
PS2	Design review	Seismic Specialist	Where a PSI is provided by the contractor, the seismic specialist should provide a corresponding PS2 to certify that the proposed works, if correctly constructed, will comply with the requirements of the NZBC
PS3	Construction	Building Services Contractor	Certifies that the works have been completed in accordance with the seismic specialist's design documentation.
PS4	Construction review	Seismic Specialist	Certifies that construction monitoring and information provided by the contractor indicates that works have been completed in accordance with the seismic specialist's design documentation.

As a building services engineer, I have been engaged over 10 x projects to acquire/supply of the NSE design for these projects specifically for the mechanical services and fire sprinkler systems. Below are some of the issues that I have faced during those projects as a building service engineer. I have outlined how to rectify them as well in the context.

PROPRIETARY DEISGN

Issue:

Each Structural / Seismic NSE engineer has their own specific design and recommendation for the product to be used. Due to this approach, any partial design by another NSE engineer or a building services engineer is not considered and only proposed to have a clean fresh design. There have been instances where NSE engineers have refused to even have a look at the design made by the building services engineer.

Solution:

As per the design and producer statement guidelines, a building services engineer can design the NSE system, and a seismic specialist can review and supply a PS2 for the design. In the process, the seismic engineer may ask for further evidence, calculations and can supply advises and recommendations to the designer before satisfying with the design and issuing a PS2. At least have a look on the proposed design by the building services engineers and communicate with them before failing or rejecting their design.

TIME FRAME

Issue:

The standard terms are to supply the PS4 within 10 days of supply of all the evidences and NSE engineer’s site visits. The building services engineers / contractors have to beg for the PS4 and most of the time it comes at the last day.

Solution:

We get that companies and individual engineers’ are busy. However, 10 days, is not a reasonable time frame considering this being one of the last cert for the completion of the project. Engineers for other services such as mechanical or Fire supplies the PS4 within 2 working days of the visit along with the final report if the visit was successful without any defect(s). If the companies or engineers’ are flooded with the work; they shouldn’t take on more work, simple is that.

CONSTRUCTION REVIEW VISITS

Issue:

For small to some medium new builds, the NSE engineer only visit the site once at the end of the project and that could be too late for the contractor to change/alter the seismic bracings/parts if failed by the NSE engineer. Needless to say that by the time a failed report comes through by the NSE engineer, the project would be at its final days of completion and ceilings are already in place.

Solution:

At least 2 x site visits must be proposed at the beginning or mid of the project and of course at the end so that the contractors will have enough time to rectify the defects.

DESIGN OF OTHER SERVICES FOR THE SAME PROJECT

Issue:

While sending out the short form agreement, we never been asked by the NSE engineer that who are doing the seismic design for other services for the same project. It is always better practically and of course financially for all services of a specific project to be designed by the same seismic engineer.

Solution:

A simple email to the people engaging the Seismic engineer asking about other services would be suffice to achieve this. If the building service that NSE engineers are engaged with doesn't have this answer, just ask for the name/contact of the main contractor or main builder for the project and then ask them the same question. At your surprise, its highly likely that you will get design work from other services if not all for that same project. And if that is the case; the co-ordination of NSE services would be much efficient and beneficial.

CONCLUSION AND SUMMARY

Non-Structure Elements have a vital role in the development of a structure and so does the NSE engineers. Should the proposed solutions be employed in day-to-day practices as outlined in above points and scenarios; a much better co-ordination and seamless sign-offs will take place which will be beneficial for the NSE engineer, client and the project of any scope.

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10 x projects requiring NSE design

The development of CarbonFlow: a Revit based LCA tool to track embodied carbon goals

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ABSTRACT

Beca has developed a Revit plug-in designed to expedite embodied carbon calculations to gain insights into carbon reduction strategies and track company progress on a 1.5-degree decarbonization pathway. By integrating upfront carbon results with a PowerBI dashboard, it enables engineers to review similar projects and understand areas of carbon intensity and enable informed design decisions.

During the development phase, a goal was established to find a consistent and efficient method to measure the upfront carbon footprint of designed projects. Recognizing the need for a tool tailored specifically to our structural modelling and metadata management processes in Revit, no other available products were deemed fit for specific structural purposes.

CarbonFlow was developed using existing projects with structural Revit models, therefore the tool relies heavily on the accuracy of the model. Through the development process, various challenges in establishing consistency, streamlining emission factors and accounting for model irregularity had to be addressed. By addressing these challenges, a process was created that is simple enough for engineers without extensive LCA training and accurate enough to give meaningful upfront carbon results.

The CarbonFlow tool is being continually refined in addition to adding upfront carbon data to the structural carbon insights dashboard with the goal to continually produce designs that are less carbon intensive using real portfolio data.

INTRODUCTION

CarbonFlow is a program developed by Beca to streamline the process of conducting upfront embodied carbon Life Cycle Assessments (LCA) and establishing carbon benchmarks for design projects. The tool tracks company progress on a 1.5-degree decarbonization pathway by integrating the upfront carbon data with a Power BI visual platform, referred to as the "Structural Insights Dashboard." This enables engineers to make data-driven design decisions

in the early stages of a project to optimize structural performance and carbon intensity. Beyond this, CarbonFlow cultivates “carbon-intuition” by bridging a visual tool like Revit with carbon data, encouraging engineers to instinctively consider the carbon impact of their design choices.

Decarbonization Pathway

In 2018 the Intergovernmental Panel on Climate Change (IPCC) issued their special report on the impacts of global warming of 1.5˚C above pre-industrial levels due to human-induced greenhouse gas emissions. The report warns that exceeding 1.5˚C will result in unavoidable increase in climate hazards and resulting risk to humans and ecosystems alike.

The United Nations Environment Programme (UNEP) 2019 Emissions Gap Report stated that to achieve a two-thirds chance of limiting climate change to 1.5˚C, emissions must drop 7.6 percent year on year from 2020 to 2030. To better understand the company’s carbon handprint[1], Beca has adopted an approach called the ‘Paris Aligned Decarbonization Initiative’ (PADI), which assesses project performance against the trajectory of the 1.5˚C decarbonization goal set by UNEP, as shown in Figure 1. At the time of development, there were no publicly available methods to apply a science-based emissions reduction approach at a project or portfolio level as opposed to an organizational level.

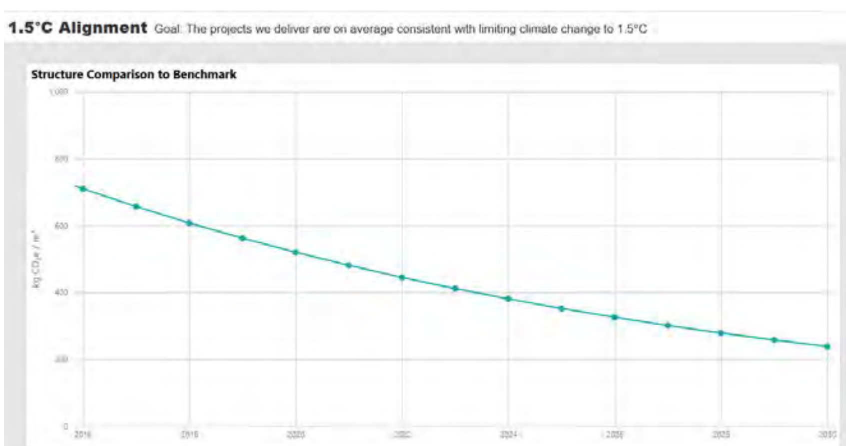

Figure 1: 1.5 C alignment curve projected out to 2030 as shown in Beca’s Structural Insights Dashboard

[1] Beca’s carbon “Handprint” refers to the direct actions being taken by Beca to reduce global warming and limit climate change.

Carbon Benchmarking

There are several sources for existing carbon benchmarks[2], however the most widely referenced and credible benchmarks are not New Zealand based and can be limited in their scope and versatility. While these benchmarks provide robust frameworks, they often lack relevance to New Zealand's construction practices and material supply chains. Since the development of CarbonFlow, New Zealand-specific carbon benchmarks are available through the Building Performance Government Website; however, these benchmarks do not fully reflect the diverse range of project types undertaken by many consultants.

Establishing specific carbon benchmarks for Beca designed projects is the first critical step of achieving our decarbonization goal. The ongoing follow up action is monitoring how carbon data changes over time. Doing so will help identify what carbon reduction strategies work for different project typologies, locations, and seismic hazards so that they can be used to help clients with their decision making in the New Zealand market.

TOOL DEVELOPMENT

Overview

CarbonFlow has been developed with the intention to aid structural engineers in their design decision making, therefore, only considering A1-A3 upfront emissions focuses the results on structure specific contributions. For further context on the rationale behind prioritizing upfront emissions versus whole-life carbon assessments, refer to the article “Upfront & Whole-of-life Embodied Carbon: What Structural Engineers Need to Know” (Symons 2024).

CarbonFlow has been designed as a Revit plug-in. It uses the materials as defined in the Revit model to streamline the quantity take-off process necessary for an LCA. The tool calculates material volume totals by grouping like-elements based on their Group and LCA category (Figure 2). Using existing New Zealand specific lifecycle inventory data[3], CarbonFlow multiplies the material specific carbon intensities by the calculated volumes.

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[2] London Energy Transformation Initiative (LETI), Carbon Leadership Forum (CLF), Royal Institute of British Architects (RIBA)

[3] BRANZ CO$_2$NSTRUCT V2.0

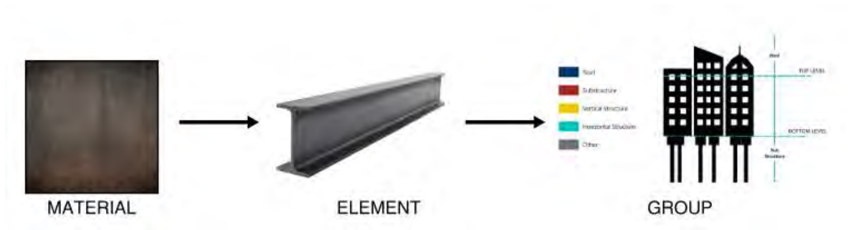

Figure 2: CarbonFlow categorisation levels

Process

The CarbonFlow process is broken down into the following general steps: Setup Groups, Material Configuration, Map EPD Data, Post Processing, and Verification. Figure 3 shows the CarbonFlow main page with all necessary steps outlined.

Figure 3: CarbonFlow mainpage

The purpose of these initial steps is to organize the data to be exported to PowerBI and to align with how LCA results are typically presented. There are five categories that the elements get assigned to: Roof, Substructure, Vertical Structure, Horizontal Structure, and Other.

The Map EPD Data step in the CarbonFlow process is where users review default emission factors. The tool automatically assigns materials and emission factors (A1-A3 GWP values using NZ product EPDs) based on the elements' name and family parameters in the Revit model. This is a crucial part of the CarbonFlow process as assigning the incorrect emissions

factor can significantly affect the results. This step also provides users with the opportunity to review and confirm the assigned “Group,” material type, and reinforcement type.

Manual review and calculation, or post-processing, is required to bridge the gap between the model and reality when the model contains simplifications, variable levels of detail, or unique composite elements (e.g. connection allowance). This is done by either using verified allowances or manual quantity take-off, often using the structural drawings to perform estimations.

Challenges

Project Setup Challenges

When the project is first uploaded to CarbonFlow, some initial effort is required to familiarize with the specific project, adjust the settings for the assessment to include the desired Revit work sets and phases. When developing the tool, the decision was made to exclude non-structural items, civil works (anything outside of the building dripline), connections and existing structures. This created more consistency in establishing carbon intensities on a per Gross Floor Area (GFA) basis. Unlike other excluded elements, connections get added back into the calculation, but as a general allowance due to the different model LOD requirements on projects.

Varying Reinforcement Rates

The material configuration stage involves designating baseline concrete strengths and reinforcement rates for concrete elements. In practice, concrete reinforcement rates vary drastically. Reinforcement rates were simplified into standardized categories: "light", "medium", and "heavy". An assessment was undertaken during development to determine what “light”, “medium”, and “heavy” reinforcement looks like in different elements for various projects. This involved manual calculation of reinforcement rates using existing design examples for piles, pile caps, raft foundations, pad footings, ground beams/strip footings, ground beams, slabs on grade, insitu slabs, beams, columns, walls, and composite flooring systems. The medium reinforcement rates are applied as the default, however project-specific designs calling for light or heavy reinforcement can be implemented if necessary.

Post Processing Challenges

The inherent model validation that occurs during the steps of CarbonFlow aid the user in identifying any inconsistencies between the Revit model and the structural drawings. Common elements that may differ from the drawings include façade secondary steel, purlins, elements designed by others such as screw piles, and composite elements. Typically, these have been excluded from the model due to the LOD the structural designers are working towards. Including these elements is an important aspect of the carbon accounting, however, it is more time efficient to add them manually as opposed to upgrading the detail of the model.

Through the process of developing CarbonFlow, it was observed that connections are often modelled to variable levels of development. As a result, the decision was made to exclude modelled connections from the quantity take off and account for them in the post processing stage. There are several standard allowances that get added into the carbon total including steel connections, timber connections, and either all roof purlins or purlin connections and laps only. These allowances are shown in Table 1 below.

Table 1: Summary of standard post processing allowances.

Item	Extra allowance (%)
Steel Connections	15
Timber Connections	25
Purlin Connections/Laps	15

To establish these connection allowances, the development team took observed ratios from existing carbon assessments and quantity surveying documentation. This method ensured that the allowances were realistic and reflects the majority of structural building types.

It was necessary to establish a process that enhanced accuracy and consistency amongst all projects and was straightforward enough for engineers without extensive LCA training. During the first CarbonFlow assessments, the potential post processing items were unknown. It was only by completing several projects that common differences between the model and actual materials were identified. These commonly identified elements and their significance to the data are included in Table 2.

Table 2: Common post-processing observations

Item	Observations	Significance
Composite members	Materials not modelled separately, including: • concrete filled CHS columns • concrete encased steel members	Omitting either inner or outer element can significantly underestimate carbon
Stairs	Stairs are often modelled as generic volumes and may have different materials for the treads or stringers	Generic volumes often get assumed to be concrete by CarbonFlow, leading to overestimations in carbon
Timber braced walls	Timber braced walls are commonly modelled as volumes without voids	Actual carbon is less, significance depends on the number of walls and spacing of studs
Piles	Pile depth often modelled at same depth across site	Actual pile depth varies over the floor plan depending on soil conditions can significantly impact carbon

It is expected that with more projects going through the process different challenges will be encountered. These observations highlight the importance of structural familiarity and care when using Revit-based LCA software.

RESULTS

Currently, the sample size of projects having gone through CarbonFlow is not large enough to draw significant conclusions on company progress towards the 1.5°C decarbonization pathway. The initial observations from the results align with previous carbon assessments conducted using other upfront embodied carbon LCA methodologies[4]. Some of these observations include:

- Foundation structure contributes a higher proportion of the total carbon for single to low rise buildings.
- Reinforced concrete was observed to be the highest contributing material for low to mid rise buildings.
- Specifying low carbon materials can significantly reduce structural embodied carbon.[5]
- Steel was observed to be the highest contributing material for mid to high rise buildings.

[4] Other commonly used LCA methodologies including the OneClick LCA software and QuickLCA (BRANZ)

[5] Observed by comparing CarbonFlow results with LCA results incorporating emission factors for EAF steel or low-carbon concrete

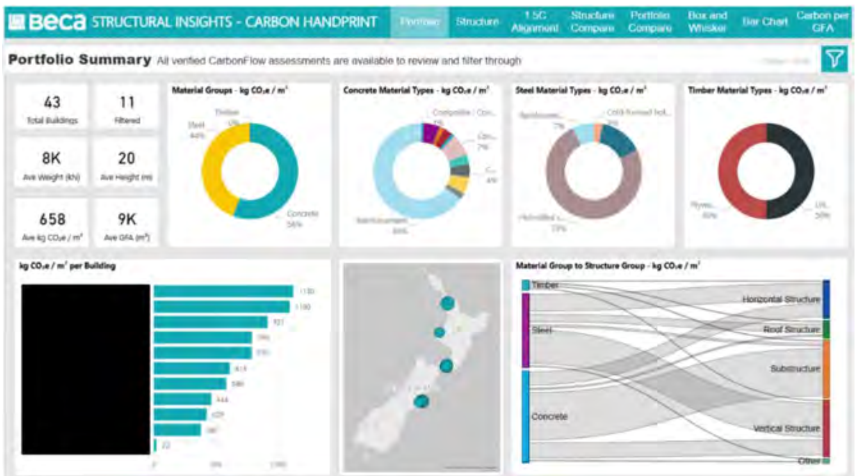

Figure 4: Overall CarbonFlow results summarised in the Structural Insights Dashboard (note project names have been redacted for anonymity)

Simplifications

The current version of CarbonFlow is limited to evaluating emissions from the A1–A3 lifecycle stages. End of life considerations, including reuse, recycling, or disposal, of materials, have not been considered. At this stage, CarbonFlow does not account for potential impacts from carbon sequestration or biogenic carbon. While it doesn't encompass the entire "cradle-to-grave" carbon footprint, CarbonFlow focuses on upfront emissions that are critical for guiding structural design decisions. Additionally, CarbonFlow uses a limited set of emissions factors which exclude low-carbon material alternatives. By reducing the scope of emissions factors, the results are centered on improving design strategies rather than focusing on material substitutions. The limited inclusion of emissions factors simplifies the 'Map EPD Data' process.

CarbonFlow has been designed for use with IFC Revit models with a LOD300 baseline. This minimizes discrepancies in the analysis from the as-built condition and provides a reasonable level of detail for counting elements significant to the total building upfront carbon. It does mean, however, that details are excluded and need to be added post-process.

The tool currently excludes strengthening and retrofit projects. This decision helps establish comparable data, as strengthening and retrofit projects typically have significantly reduced upfront carbon intensities when compared to similar new build projects.

CarbonFlow is not intended to report under green building certifications like Green Star as it is purpose built for improving designs and assisting with client decision making.

Future development of the tool

Efforts in developing and refining the tool further will continue as Beca works toward making CarbonFlow more widely adopted across projects, enabling its integration into everyday design workflows.

CarbonFlow is currently only developed for and being used to assess structures. Future efforts will focus on expanding the tool's capabilities to incorporate broader practices and disciplines such as building services and architectural elements. The future focus for CarbonFlow is to leverage the learnings from its current applications and solidify carbon conscious practices across the business. Learnings from the application will be an important driver to delivering improved outcomes for clients and the environment.

CONCLUSION

Integrating CarbonFlow into a 'business-as-usual' approach offers a further opportunity for embedding sustainability decisions in all future work[6]. It not only reflects the commitment to adopting data-driven strategy that supports Beca's handprint goals, but also provides engineers with a framework to develop 'carbon intuition', helping them make carbon-conscious design choices.

Many people at Beca have been involved in the development of the tool, all bringing unique insights in how to improve the tool and extend its use beyond measuring upfront carbon. CarbonFlow has been used to validate LCAs performed through other methodologies, provide material quantity data, and set a drafting standard for all future project models.

CarbonFlow is still in its early implementation stages, but the potential future applications of the tool are far reaching. Beca is excited to use CarbonFlow to drive company progress towards the 1.5˚C decarbonization goal and make impactful change in the New Zealand structural engineering industry.

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[6] Developing CarbonFlow would not have been possible without the hard work and technical knowledge of Eldho Raju, George Fitzpatrick, Ben Westeneng, Phoebe Moses, Paul Denmead, Zoya Mehrfard, and Manuelito Julaton.

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Hybrid Post-Tensioned Concrete and Steel BRBF Parking Garage – A Fresh Approach

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ABSTRACT

Conventional parking garages are comprised almost entirely of reinforced concrete. They require staging of various construction operations which are taxing to a construction schedule. Reinforced concrete vertical elements (walls, columns) are often obtrusive, occupying valuable footprint space while also creating security and visibility concerns.

The South Terrace Parking Structure at Utah State University in Logan, Utah overcame many of these challenges with a hybrid structure comprised of post-tensioned concrete decks while the vertical elements are structural steel with buckling restrained braced frames (BRBF). The use of a BRBF lateral system afforded a reduction of design forces of nearly 70% owing to improved system ductility coupled with a lengthened fundamental period and an overall lower seismic mass due to the removal of concrete walls and columns. These factors resulted in improved resiliency while reducing the total volume of foundation concrete by more than 50%, reducing the foundation costs and complexity. The innovative fabrication and erection sequences resulted in improved construction efficiency and reduced lag-time between each construction level, resulting in structural completion 8 weeks ahead of schedule. Elimination of formwork materials required for columns and walls eased storage and sequencing needs, benefiting the site's tight footprint. Transparency of BRBF frames enabled their strategic placement at motorist turning corners without compromising visibility. Typical columns measure only 250mmx250mm and with bucking restrained braces, the framework results in superior overall visibility and sightlines. This paper will detail the design and construction of this innovative system and the resulting benefits to project cost and schedule.

INTRODUCTION

Parking Garage Challenges

Parking garages are typically comprised of reinforced concrete. As a cast-in-place or precast solution, reinforced concrete typically provides the strength, stiffness, and durability to meet the demands of a parking environment. Drawbacks of concrete include various stagings of constructed systems which must alternate from level to level, often employing different crews and trades, ultimately impacting the construction schedule. Concrete elements are also

obtrusive, often requiring space which compromises visibility. Concrete shear walls can create 'blind corners' for motorists if not strategically placed and concrete frames in general (beams and columns) require large footprints and even overhead space that drives height requirements between floors. In cold environments, concrete requires costly heating during construction and/or insulation to ensure a sound finished product. Concrete lateral force resisting systems express significant damage as a result of design-level lateral forces and can be very costly to repair.

Hybrid Approach

Designers of the South Terrace Parking Structure at Utah State University in Logan, Utah, USA developed a hybrid concrete and steel approach which overcame many of the challenges often associated with parking garage construction. This project was driven heavily by schedule. The hybrid approach includes standard post-tensioned, two-way flat plate decks with vertical and lateral systems being comprised entirely of structural steel, with buckling restrained braced frames (BRBF) as illustrated in Figure 1. The hybrid approach summarized in this paper enabled the following:

- Minimized mobilization of crews for vertical construction since vertical construction accommodated multiple stories for each tier of steel.
- Facilitated operation of deck forming and casting crews unimpeded by the necessities of casting columns and walls between each level of deck
- Reduced need for surveying and layout crews between each constructed level
- Greater latitude in layout of the lateral force resisting system since braced frames are mostly transparent, which also improves security in general
- Significant reductions in design forces for the lateral force resisting system owing to a lengthened fundamental period and higher ductility afforded by the steel framed buckling restrained braced frame approach
- Reduced wintertime construction demands for insulating walls & columns
- Reduced seismic mass with the steel framework weighing about 25% of the equivalent concrete framework
- Reduction of restraint during post-tensioned concrete deck stressing operations which reduced in-plane restraint stress of decks
- Simpler development and detailing of vehicle restraint systems
- Improvement in seismic resiliency with buckling restrained braced frames in comparison to concrete lateral force resisting systems

With the benefits afforded by the hybrid steel and concrete system approach, several challenges became apparent for which creative solutions became necessary. These included:

- Accommodation of slab shrinkage and its influence over each tier of steel columns (1st Tier being 2 stories, 2nd Tier being 3 stories).
- Corrosion performance of exposed structural steel

- Development of integrity reinforcing
- Lack of familiarity with hybrid system

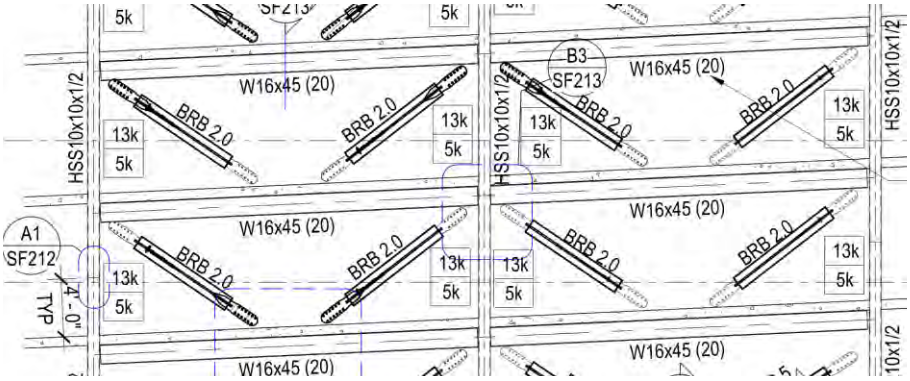

Figure 1: BRBF With Two-Way PT Flat Plate (Elevated View)

SYSTEM DESCRIPTION

Concrete Decks

The hybrid system makes use of 200mm thick post-tensioned, two-way, flat plate concrete decks. Such systems have proven highly reliable in the challenging environments of the Intermountain West which include major temperature swings, and prolific use of de-icing salts during winter. The post-tensioned concrete, which combined with other improvements such as direct corrosion inhibiting admixtures, enabled parking decks with a targeted design life in excess of 70 years. In addition to this, the two-way, flat-plate approach enables minimized story heights since the total structural system depth is confined to 200mm except at the beams of the steel braced frames.

Vertical Support

All columns in the parking garage are square structural steel members (SHS) measuring 250mm x 250mm, including the columns for the braced frames. Hollow structural shapes were chosen for both efficiency of design and for the relatively flat, relief-free surfaces which are expected to weather favourably. Likewise, the interface of the square columns with the post-tensioned decks provided for a clean, 4-sided rectilinear geometry for the concrete deck to column interface, a condition which would have been more difficult with traditional W-shapes. Column capitals are comprised of welded angles and plates extending 150mm beyond the faces of the column to provide a relatively large bearing area for the concrete decks to be supported by the columns. The capitals incorporated an arrangement of headed stud anchors

vertically from the capitol bearing surface and horizontally from the column (both within the slab thickness) through which tendons could pass to address concerns regarding integrity reinforcing (Figure 2). Column splices utilized bolted connections with hidden nuts and splice plates oriented vertically within the cavity of HSS columns.

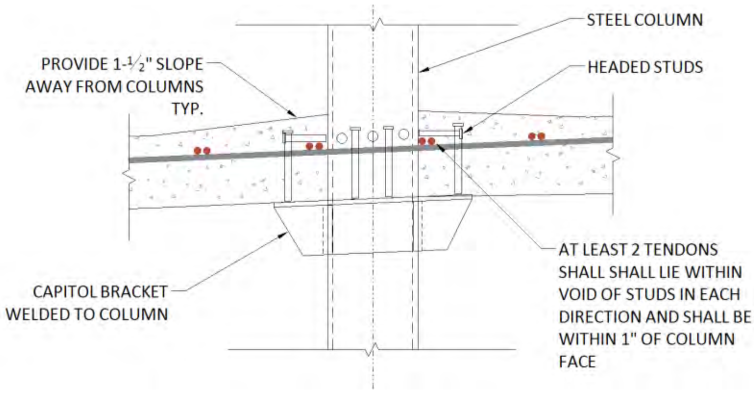

Figure 2: BRBF With Two-Way PT Flat Plate

Lateral Force Resisting System

The lateral force resisting system is comprised of the HSS columns, wide flange beams, and diagonal buckling restrained braced frames. The 'open' nature of the BRBF system mitigated concerns that often accompany a concrete counterpart as the frames offer nearly full visibility across the structural bays they occupy. As a result, us of BRBF frames at the interior of the garage resulted in no 'blind corners' for motorists (Figure 3). Likewise, security concerns were reduced with the BRBF system providing maximum visibility across the breadth of the structure.

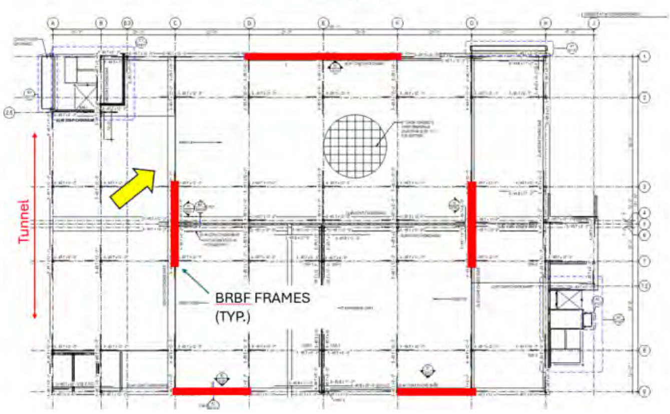

Figure 3: Typical Floor Plan View

SCHEDULE IMPACT

Steel Erection with Concrete Construction

The benefits of the BRBF lateral system included significant reduction in the volume of foundation concrete and therefore a major reduction in the cost and time of foundation construction. The hybrid approach allowed for the detailing and fabrication of structural steel concurrent to foundation construction efforts. Erection of primary structural steel occurred in two mobilizations. Following the first mobilization for structural steel erection, deck forming and casting crews began work and proceeded from level to level, unencumbered by the need to await construction of walls and columns between levels. This proceeded to the mid-height of the 5-story structure, upon which time the second mobilization for structural steel erection proceeded. After this, deck forming crews proceeded, continuously forming and casting decks all the way through the roof without awaiting layout or casting of columns or walls.

The overall result of the hybrid approach enabled a reduction in schedule of over 8 weeks (of a 12-month schedule overall), thereby assuring project completion well ahead of the owner's critical deadline.

SEISMIC DESIGN

Lateral Forces

Use of a buckling restrained braced frame enables a significant lengthening of fundamental period for the structure. The early design phase included consideration of a special reinforced concrete shear wall (SRCSW) lateral force resisting system, for comparative purposes. This design had two primary shear walls in each principal direction, each centrally located on each of the 4 sides of the structure. This approach yielded a fundamental period of approximately 0.4 seconds, coinciding with a spectral acceleration response at the peak of the prescribed design spectrum for the site. The BRBF lateral system enabled the introduction of tuned flexibility, resulting in lengthening of the fundamental period to greater than 0.8 seconds (Figure 4). The net result is a reduction spectral acceleration and therefore seismic design forces of nearly 50 percent.

Enhanced Ductility

The concrete shear wall comparative design required that the shear walls bear a significant amount of gravity load. Requirements of relevant codes included a response modification factor (R-factor) of 5 for the derivation of seismic design forces for the shear wall option. For the BRBF system, the response modification factor is 8 (a reflection of exceptional ductility), which, in comparison to SRCSW, affords a further reduction in seismic design forces by approximately 38 percent. This is illustrated in Figure 4 with the reduced spectrum of the BRBF design, developed as improved damping from higher overall energy dissipation.

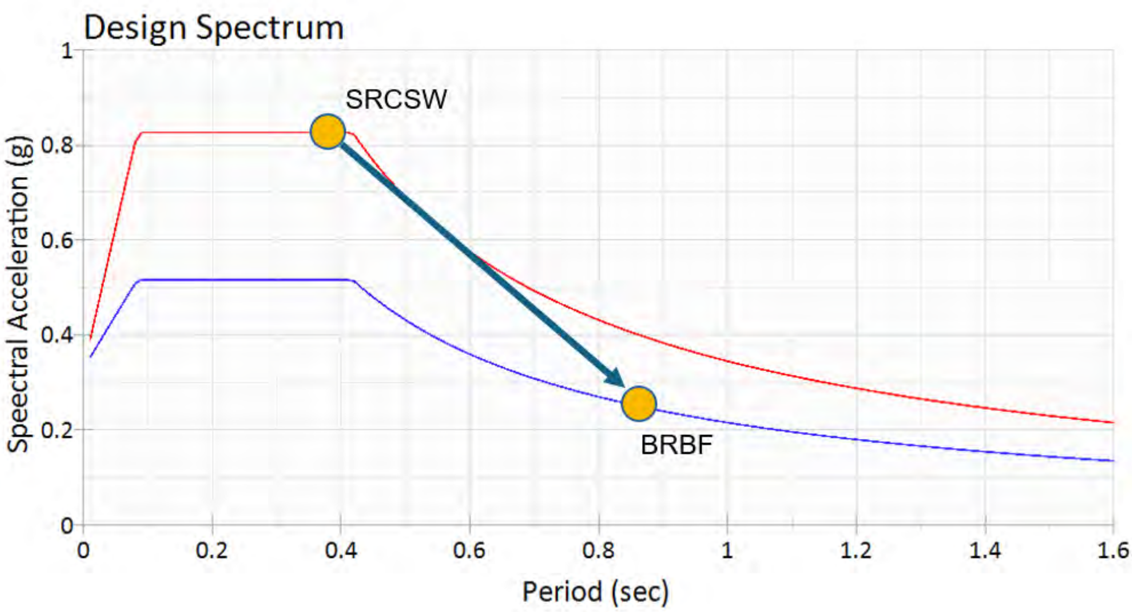

Figure 4: Design Response Spectrum

Taken jointly, the magnitude of total base shear required for the design of the hybrid system was reduced by approximately 70% when compared to the requirements of the parallel shear wall system. This reduction was driven principally by the lengthening of the fundamental period while the response modification factor also contributed significantly to the design force reduction.

Resiliency

For the hybrid system, steel the ductile frames are expected to exhibit exceptional resiliency. Should damage to the BRBF system be extreme, the need for replacement of braces, though deemed unlikely, would be far simpler and less costly than repair of a concrete system. Comparative evaluations using the SP3 Risk Model by Haselton Baker predicts structural repair costs for the hybrid garage being about 35% of those for a comparable concrete shear wall garage (SP3, 2025).

DETAILING & LAYOUT

Corrosion Performance

The location of the project is a region of extreme seasonal temperature differences plus it is a region where corrosive de-icing salts are used in abundance. Experience in the region has shown that post-tensioned concrete decks have superior performance expectations in comparison to other systems, especially with enhancements such as direct corrosion inhibitors for the concrete mixes. Using such approaches enabled a projected 75-year serviceable design life for the concrete decks, at which time routine and regular maintenance will likely be needed to keep the structure in good condition. Other strategies for improving corrosion performance became readily adaptable to the hybrid scenario. All structural steel was galvanized and casting of the concrete deck against and around the columns was sloped to shed water away from the column, a provision more easily accommodated with the hybrid approach because the steel column is present during the casting operation (Figure 2). This reduces the potential for water settling at potential voids of the steel/concrete interface. Likewise, a caulked joint at the anulus interface of the concrete deck and steel column further discourages the intrusion of water. These detailing approaches, yield favourable expectations in terms of overall corrosion performance.

Vehicle Cable Barriers

The hybrid approach required accommodation of vehicle cable rails at the perimeter of the structure and at the diverging interior ramps. While normally accommodated with column penetrations and anchorages to walls for a concrete garage, such accommodations become simpler in the hybrid approach with a fastening approach with welded angles exterior to the column footprint (Figure 5). This enabled the accommodation of cable anchorage sockets and connections of the cable rail to the structure to be handled through the steel framing package rather than the efforts typically part of the on-site concrete construction. Likewise, the approach enabled the cable rail to be outboard of the column footprint toward the outer side of the structure, thereby enabling more occupiable space at the interior.

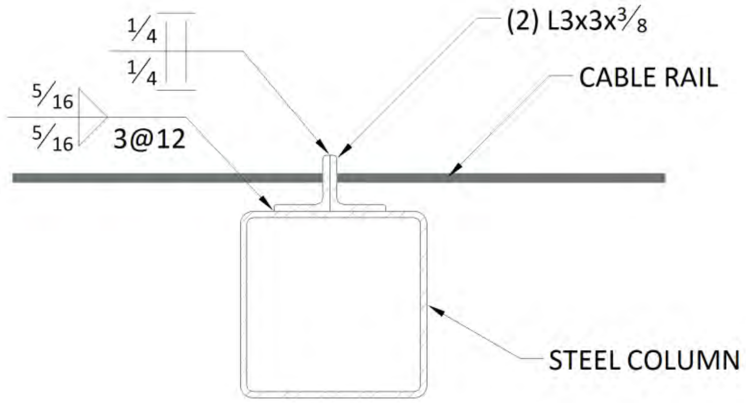

Figure 5: Cable Rail Attachment (Plan View)

Slab Shortening & Shrinkage

Shrinkage and shortening of post-tensioned slabs due to stressing, curing, and creep are often addressed by anticipating the magnitudes of these phenomena and deliberately constructing the vertical elements of the structural system (columns and walls) in and outwardly canted manner. The magnitude of the canting is typically benchmarked against the location of the column or wall with respect to its position and distance from the centre of the slab. As such, columns and walls furthest from the centre are constructed with the greatest magnitude of outward canting. Upon shortening, the vertical elements are pulled into a plumb condition.

For the hybrid approach, a principal benefit is erecting multiple stories of steel in one tier. The consequence of this is that the shortening effect as it manifests in a single continuous column becomes amplified. Each shortened slab pulls columns closer to the centre of the slab and this becomes amplified at the top of the column by the ratio of slab elevation to total column height. Multiple slabs pulling columns toward the centre contribute to this effect even more.

Addressing the effects of shortening was handled in the steel columns of the hybrid solution in a manner similar to a conventional concrete solution by canting columns outward to counteract the anticipated total effect of shortening. Figure 6a illustrates a typical condition of anticipated shortening of slabs and its effect on a column. Figure 6b illustrates the result of canting the column outward to overcome the effects of shortening. Note that the outward canting is developed as a linear manner shown in Figure 6b. However, the effects of shortening are nonlinear, occurring at each level at a different time and aggregating level by level. The final result is a slight curvature to the column as shown in Figure 6c. The curvature induces bending stresses in the column which increase utilization ratios by a maximum of about 7 percent. Final canting values were developed in a full 4-dimensional analysis utilizing the 3-d design model with staged construction of elements and slab shortening captured with temperature loading to replicate its effects (Computers and Structures Inc., 2024).

The consequences of shortening were addressed with the BRBF hybrid frame in ways that were not immediately apparent. First, connections for BRBF frames typically utilize oversized holes. Leaving bolted BRBF frame connections ‘loose’ until completing the stressing operation of the slab above afforded an opportunity to mitigate the effects of at least some restraint, an option not possible with a concrete system. Likewise, column splices were maintained loose and nuts of anchor rods at column baseplates were held off of the top of the baseplate. Column baseplate grouting operations were delayed until slabs immediately above were stressed and allowed to shorten, thereby reducing restraint and reducing column and slab stresses as a result of shortening. All bolts were tightened and grouting operations proceeded following the effects of slab stressing and slab shortening.

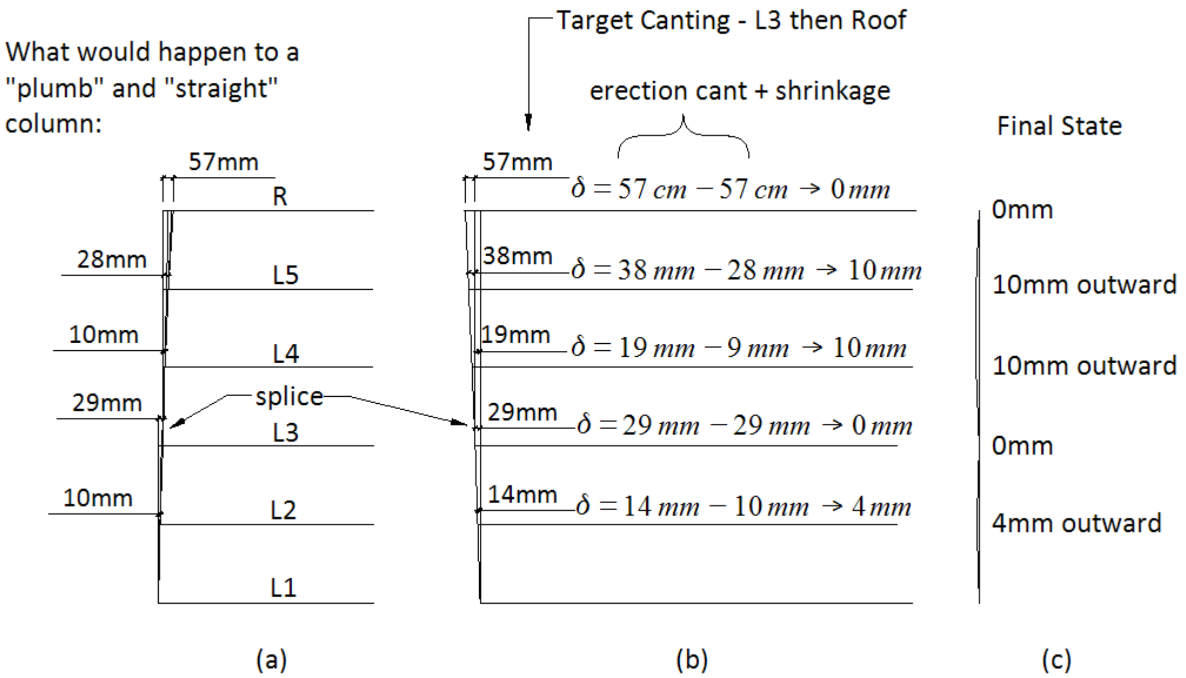

Figure 6: Column Canting Concept

Conclusions

The hybrid approach for parking garage construction represents a strong alternative that supports an accelerated construction schedule while also introducing significant benefits to the overall design. Chief among these is enhanced ductility coupled with softened seismic response. Other improvements include greater visibility, security, and overall resiliency.

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How contractors can influence sustainability outcomes whilst lowering costs

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Abstract

It is no secret that there is a certain stigma when it comes to driving for more sustainable outcomes on a project. The perception is that it comes with a cost premium, and therefore, is typically the first thing that is value engineered out of the project.

However, if you strip away certification costs, the principles of sustainable design run parallel with the principles of cost efficiency in construction. If you reduce materials on a project, you will reduce costs.

In today's market where cost blow-outs are prevalent and projects appear to be going towards more design and build, public-private partnership (PPP) model contractors have a key role to play in ensuring sustainable outcomes can be met.

This paper explores how a contractor can reduce costs whilst achieving sustainability goals through the principles of:

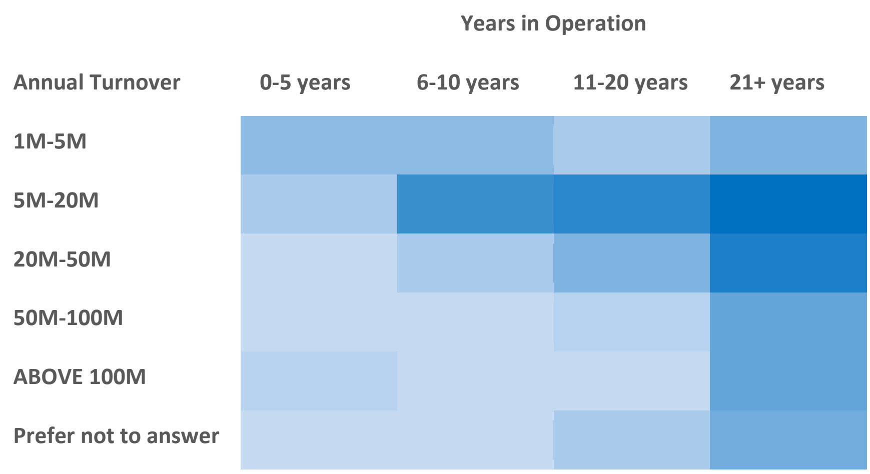
- Design efficiency
    - Pursuing alternative design methods
- Design out waste through design for manufacture and assembly
    - Choosing lengths and sizes that are common
    - Reduce temporary works, wastage and rework
- Procurement
    - Procuring cost-effective (or cost neutral) low carbon materials

Introduction

In December 2024 Preformance launched and industry wide survey to assess sustainability within the sub-contractor market in New Zealand. This paper summarises the findings of this research based on the feedback received from subcontractors in parallel with initiatives adopted by Preformance during the design process.

Over 100 subcontractors responded across various trades, sizes and years in operation. For this paper we will focus on the following key trades, steel, reinforcing steel, concrete, timber and modular. As the survey results are yet to be released feedback from subcontractors has been anonymised in this paper.

Table 1: Overview of respondents to survey



Akin to the success of a project the more time you spent adequately planning and designing efficiently upfront will lead to greater sustainability outcomes. In regard to planning, this is ensuring that the limitations of the supply chain are well understood, and procurement and design aligns with these limitations.

Procurement pathways for a contractor

To understand the impact subcontractors can have on the design of a project first we need to understand the different procurement routes.

The Principal (the client) will engage a Main Contractor (the builder) to construct their project. The main contractor manages the construction of a project and engages subcontractors to construct the build. There are two main procurement routes for the main contractor:

Build Only –The design is completed by the client and then tendered out to procure the builder. Typically, this procurement is undertaken in a relatively short period compared to the period of design. The client owns the design risk. Based on this procurement pathway the subcontractor typically doesn't have much influence over the design.

Design and Build – At a certain stage during the design process the client will tender out the project and seek to hand over all design responsibility to the builder. The design requirements for the project are outlined in a robust set of documents, known as the 'Principal's Requirements'. The builder procures the design team and owns the design risk. The builder may choose to engage with subcontractors early to either lead design and / or secure material and cost.

In both scenarios above the builder commits to a programme and cost for the project at time of tender.

In more recent years **Early Contractor Involvement** (ECI) has become more prevalent where during a Build Only project the builder will be procured as a member of the design team. In this scenario the builder will provide 'buildability advice' to steer the design to be more efficient for construction. However, the design team is still engaged by the client and in most cases the procurement pathway is set, thus engaging subcontractors early for design input and procurement may be limited.

The design process and contractor's influence on sustainability

There is a stigma that a more sustainable design usually leads to higher costs. For the purpose of this paper the focus on sustainability will be on upfront embodied carbon of buildings, however, the same principles can be applied to horizontal infrastructure. With a more sustainable design meaning a reduction in embodied carbon. The structure of a building typically makes up most of the embodied carbon for a project, hence the focus on this opportunity of sustainability.

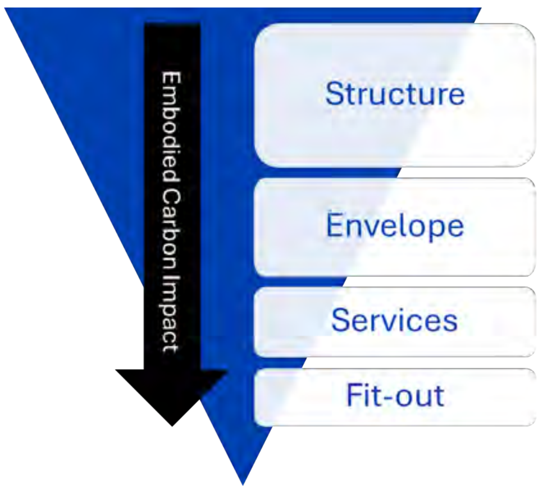


Figure 1 : Embodied carbon impact for a building project

General feedback from subcontractors (who price projects) say higher costs are primarily due to tight tender programmes and the siloed approaches of design vs construction, that doesn't allow for innovation.

As a project progresses the ability to influence sustainability opportunities diminish as more of the design gets locked whilst the contractor and client try to keep the project within budget and programme.

The following sections explore different examples how a main contractor can drive for greater sustainability outcomes through early engagement of subcontractors, all whilst not having a negative impact on cost.

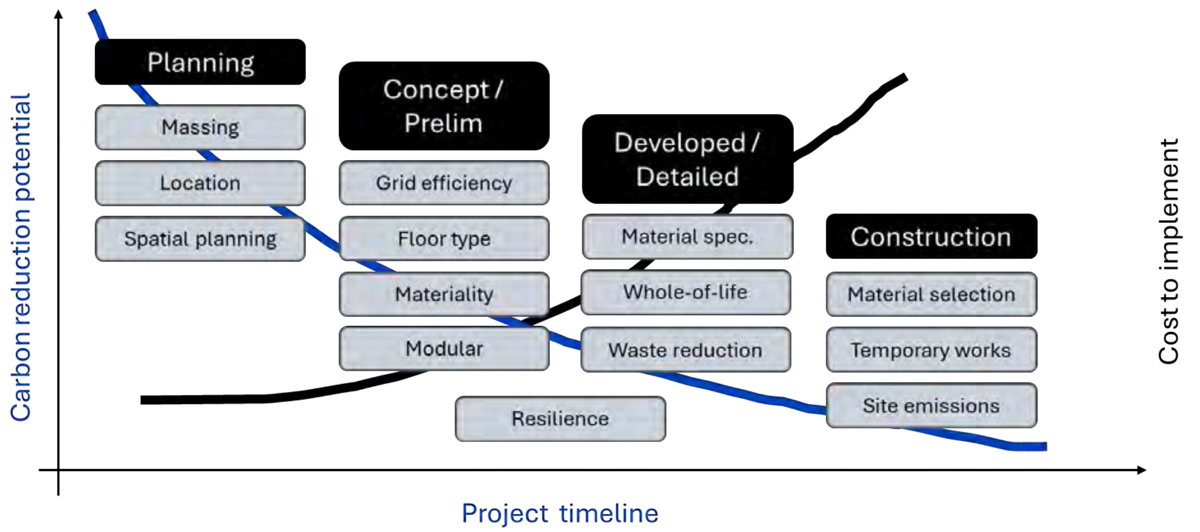


Figure 2: Carbon reduction potenial vs time vs cost

Design efficiency

Suspended flooring

A common design approach for suspended flooring is to use steel primary and secondary beams to support flat soffit metal decking. As steel has a higher embodied carbon, per kg, than concrete this can have a significant impact on the embodied carbon of a project. Design opportunities such as the use of longer span precast elements to mitigate the need for secondary beams can have both cost and carbon benefits.

However, the use of post-tensioned (PT) slabs can also have further significant impacts. A subcontractor for PT noted that it is the most effective product for repeatable and high floor plates. Even though it is more expensive than traditional steel and concrete most quantity surveyors don't consider the reduction in materials as a result of adopting a PT design. With the flooring alone compared to a traditional project having a reduction of up to 30% concrete and steel. This doesn't consider the savings made in beams as PT slabs can span further as well the reduction in services coordination and rework with having a flat-soffit slab. This same subcontractor noted that typically by the time they are engaged to steer the design towards PT they are often thwarted due to the re-design cost and program implications.

Seismic restraint

Traditionally seismic restraint will be fully designed by the seismic engineer and follows a standard set of details that utilises a different array of products e.g. Rondo, Sikla, Unistrut and Hilti. The gravity support is to be designed by the subcontractor. When the seismic restrained package is tendered out each individual trade (services and architectural subcontractors) will construct the seismic restraint associated with their package. Efficiencies are typically not pursued as there is not sufficient time to change the design and get it reviewed by the structural engineer.

Homing in on the services seismic restraint package an alternative approach would be to engage with a seismic restraint supplier early and nominate them as the preferred product. This supplier can be leveraged to provide design services to assist the design team. Where the services design is design and build the seismic restraint design can be optimised to conform with shop drawings.

One subcontractor noted that on recent data centre projects they were able to save ~30% of steel tonnage by:

- Modularising the design
- Combining both seismic restraint and gravity supports
- Combining different trades into a combined service support

Even though their product was more expensive than an alternative, through the saving of steel tonnage and the detailed modelling early on the project they had an overall cost saving through reduction in materials and rework onsite. This reduction in material resulted in an overall reduction in carbon.

Design out waste through design for manufacture and assembly

Timber

Timber framing and timber sheets come in standard sizes. By simply adopting standardised framing heights and grid spacing the quantum of timber offcuts and nogs required for sheets can be drastically reduced. A modular subcontractor of cassette framing (2-D panelized timber) noted that they typically they try to redesign framing for load-bearing or cladding walls to optimise the manufacturing process, this can see wastage rates reduced to ~3-5% when compared to ~13% - 20% if built onsite (BRANZ 2025). However, they noted they were only able to do this on a design-build project where they were engaged early enough in the design process so as to not affect the construction programme. They worked collaboratively with the structural engineer to reduce material (and in turn wastage) during the detailed design of the project. Effectively shop drawing the structure during detailed design.

Reduction in rework and temporary works

Both rework and temporary works pose health and safety risks, with ~70% of health and safety incidents occurring whilst undertaking rework on site (Love, Smith, Ackermann & Irani 2019). Not only that they also contribute negatively to sustainability with the requirement for additional materials. Feedback from a modular subcontractor was that on a recent project they modularized timber façade panels for a project. This led to the building not having to be scaffolded and temporary edge protection not having to be built onsite. Furthermore, with the timing of the fabrication of the panels to arriving onsite for erection there was no time for rework, so they ran virtual scenarios on test fitting the structure prior to fabrication to ensure that any issues could be resolved. This led to the panels arriving onsite and craned into place without any rework or temporary works requirements, thus reducing wastage of materials onsite.

Procurement

Low carbon steel

Steel is typically indented from overseas if the quantity is >50 tonnes. This involves fabricators going through third party merchants to secure product from a variety of mills around the world. From order to material arriving usually takes 4 months (SCNZ 2025). If the quantity is less than this than the material is procured from local stock held by merchants.

It is more challenging to get low carbon steel locally as the material is already procured hence the ability to influence the source of material is limited. Whereas the general feedback from steel fabricators is indenting low carbon steel can come at no cost premium if the indent order early enough (an additional month or two) that they can steer their merchant to procuring from more sustainable mills i.e. electric arc furnace (EAF) mills rather than blast furnace mills (BF). In practice this would typically involve procuring a steel fabricator off the completion of developed design with the structural engineer setting an average GWP value for the supply of steel. Note, EAF produced steel can be up to 70% less embodied carbon than BF steel.

Low carbon steel reinforcing

Depending on the design, steel reinforcing can have a significant impact on the embodied carbon of concrete. Thus, securing low carbon steel reinforcing, from an EAF mill, can potentially save you up to 50% embodied carbon in your concrete trade.

One NZ supplier noted that they procure low carbon reinforcement from overseas in coils that they be bent and cut locally. For the cost to be neutral this order had to be put in at least 3-4 months before the steel was to be onsite. Foundations are the first thing to be built onsite thus this lead time proves to be difficult on traditional build-only projects where they are only procured after the design is fully complete. Whereas the supplier noted on a design-build contract the main contractor committed to an order at developed design for majority of steel

reinforcement. As the design progressed any overs were procured locally and unders were a credit back to the main contractor.

An alternative supplier noted that all their steel reinforcement comes from an EAF mill thus their base product is low carbon. As far as we know this is the only concrete subcontractor in NZ that does so. Thus, to have some competitive tension in procuring low carbon steel reinforcing it would be beneficial to do so at the conclusion of developed design.

Conclusions

As described in the limited examples above contractors can have great influence over the sustainability outcomes for a project. However, to do so they need to be engaged early enough to provide feedback on the design and the secure long lead items. This will allow for any innovations proposed to not affect the programme and to be captured in the design as well ensuring procurement of low carbon materials are kept cost neutral. As the opportunity to influence the sustainability (and cost) outcome for a project greatly diminishes once the design is complete on traditional build-only projects this may be why clients are seeing a cost premium applied to sustainability, as the market does not have sufficient time to respond to requirements during a short tender period.

By working more collaboratively, between designers and contractors we can ensure that sustainability outcomes for projects are met, whilst maintaining budget and programme.

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Out-of-Plane Seismic Force for RC Walls of Industrial Buildings

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ABSTRACT

Industrial buildings are commonly constructed using steel portal frames and concrete wall panels (precast or tilt-up) as their primary structural systems, serving the dual purposes of load resistance and cladding. Although this structural system is simple and efficient, New Zealand's seismic design standard, NZS 1170.5, does not provide sufficient guidance on estimating out-of-plane seismic forces of these RC wall panels. This study evaluates the provisions of NZS 1170.5, BRANZ guidelines, and international standards, such as ASCE 7-22 and ASCE 41-23, to identify and address inconsistencies. Findings reveal that while BRANZ acknowledges the amplification effects of diaphragm flexibility, it does not explicitly address them. Its recommendations to apply Section 8 of NZS 1170.5 lack robust justification and result in excessive conservatism, which not only increases the costs of new designs but also reduces New Building Standard (NBS%) ratings during seismic assessments, potentially misrepresenting the actual performance of these walls and the overall building. Conversely, NZS 1170.5 states that ground-supported RC walls should be designed as independent structures under Section 5, without dynamic amplification. However, ambiguities in the standard's wording have led to misinterpretations in practice. International standards on the other hand, provide clearer distinctions between structural and non-structural components, offering rational methodologies for seismic load calculations, including diaphragm flexibility effects.

This study presents a case study analysis and concludes that applying Section 5 of NZS 1170.5 is consistent with international best practices. It achieves the intended life safety performance without undue conservatism. However, on the other hand, using Section 8 of NZS 1170.5 appears to be inappropriate and overly conservative.

Keywords: Out-of-plane seismic force, RC structural walls, Industrial buildings, Seismic performance, Part and component.

1 Introduction

Industrial buildings are typically constructed using steel portal frames paired with concrete wall panels, such as precast or tilt-up walls. These components serve a dual purpose: they provide primary gravity and lateral load-resisting systems while also functioning as cladding for the structure. Due to their efficiency and versatility, such buildings are a common sight in industrial zones, often forming the backbone of these areas. Consequently, the structural design of these

buildings should follow well-established, standardised practices to ensure both reliability and efficiency.

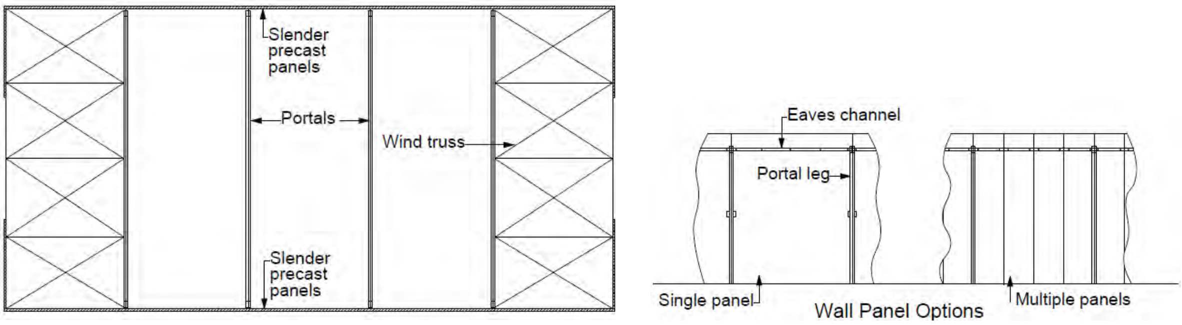

Figure 1: A typical plan and common wall options of industrial building (BRANZ)

In New Zealand, NZS 1170.5 outlines some requirements that are ambiguously worded, leading to varied interpretations among engineers and discrepancies in practical applications. Documents such as the BRANZ publication and Determination 2013/057 have promoted designing reinforced concrete (RC) walls as "Part" under Section 8 of NZS 1170.5. However, these references do not provide robust evidence or authoritative support for this interpretation. This study aims to provide clarity on conflicting recommendations and their implications for RC walls of industrial buildings through an evaluation and comparison of different methodologies.

2 BRANZ recommended approach

BRANZ, in Clause 10.1, emphasizes the critical importance of the connection between panels and the eaves beam in resisting earthquake forces. The report highlights that during an earthquake, the robustness of this connection plays a pivotal role in ensuring the structural system's seismic performance. The clause further states that dynamic computer analyses indicate that connection forces can be amplified by up to 1.25 times as a result of the flexibility of the roof structure. As a result, it is imperative to design these connections to withstand the amplified forces effectively. While BRANZ discusses the amplification effects arising from diaphragm flexibility, it provides an example in Appendix C for calculating earthquake forces using Section 8 ("Part") of NZS 1170.5. However, the rationale for referencing Section 8 in this context is not explicitly clarified. It seems likely that the decision was influenced by the identified amplification due to diaphragm flexibility. Nevertheless, Section 8 of NZS 1170.5 primarily addresses other types of dynamic amplifications specific to structural parts and does not directly account for diaphragm flexibility effects. This creates a mismatch between the amplification mechanism discussed and the provisions of the standard used by BRANZ, potentially leading to ambiguity in its practical application.

BRANZ further notes that Section 8 (Part) of NZS1170.5 is focused primarily on multi-level buildings and its application to single-storey slender panel structures, such as what is used in industrial buildings, "*requires a **critical assumption** to be made that the driving force*" is taken

as half the sum of the calculated forces at the base and top of the panel. In other words, the report effectively recommends designing the panel using the average of the forces calculated from Section 5 of NZS 1170.5 for the base of the panel and Section 8 for the top of the panel. While this approach seems to aim at reducing the overly conservative results from Section 8, the report does not provide a clear explanation or justification for combining these distinct sections of the standard. This ambiguity is compounded by the recommendation in the report appendix to design the panel's top connection using the force calculated from Section 8, without applying the same relaxation used for the panel itself. This approach seems to indirectly account for the diaphragm flexibility effects at the top of the panel, but it lacks clarity and consistency. These inconsistencies highlight the need for more explicit guidance on incorporating amplification effects, such as those caused by diaphragm flexibility, into the design process.

To address these gaps, this study explores the provisions of NZS 1170.5 and other international standards to identify appropriate out-of-plane seismic loads for wall panels and their connections. The goal is to eliminate unnecessary, overly conservative assumptions and provide a more rational basis for design.

3 NZS1170.5 prescribed approach

The classification of single-story reinforced panels, extending from the base to the eave, as "Part" under NZS 1170.5 has been a point of debate among engineers, particularly in the assessment of existing buildings. The use of conservative assumptions suggested by BRANZ often leads to costly strengthening work, which may not even be feasible in some cases. To address this issue, it is crucial to review the scope and intent of Section 8 of NZS 1170.5, which outlines the calculation of lateral forces acting on parts and components. Clause C8.1.1 of NZS 1170.5 offers guidance, stating: "*An element of the primary seismic structural system **may** also be considered as a part when it is loaded by earthquake actions in a direction not considered in the design of the primary seismic load-resisting system*." The use of the term "***may***" introduce ambiguity, enabling varied interpretations. Anecdotally, this has led to interpretations, such as those in Determination 2013/057, which classify single-story reinforced concrete panels as "*Part*" when subjected to out-of-plane forces. This interpretation, however, is overly conservative and appears inconsistent with the intent of the standard. This misclassification can be refuted on two grounds: The standard itself provides specific examples to clarify what qualifies as a "part". Continuing the first explanation, clause C8.1.1 elaborates: "*Such situations would include face loading on an unreinforced masonry shear wall or vertical seismic loading on a cantilever*." This explanation confines the classification of "part" for out-of-plane face loading specifically to unreinforced masonry walls. Notably, it does not adopt the broader phrasing, "such situations would include but are not limited to...," which is used elsewhere in the clause to encompass additional scenarios. Consequently, reinforced concrete (RC) walls of industrial buildings do not fall under this definition.

Additionally, Clause 8.2 of NZS 1170.5 explicitly states that when a structural component or part (assuming the wall is classified as a part) is directly supported on the ground, it must be

designed as an independent structure. In such instances, design actions are calculated using the provisions of Section 5, based on the structural properties defined in Section 4. The standard commentary on Clause 8.2 further clarifies that components or parts directly supported by the ground are ***not subject to any dynamic amplification*** from the building's seismic response. Therefore, their design actions are determined solely under Section 5, without adjustments for interactions with the building. The following sections examine whether applying Section 5 of NZS 1170.5 is sufficient to ensure the required performance of reinforced concrete (RC) walls in the out-of-plane direction.

4 ASCE7-22 and ASCE41-23 prescribed approach

Insights from American standards, such as ASCE 7-22 and ASCE 41-23, further support the interpretation using section 5 of the NZS1170.5 approach. ASCE 7-22 clearly distinguishes non-structural components, discussed in Section 13 (Seismic Design Requirements for Non-structural Components), from structural walls and their anchorage in the out-of-plane direction, addressed in Section 12.11. Similarly, ASCE 41-23 provides explicit distinctions between structural and non-structural components, ensuring reinforced structural walls are not misclassified as non-structural components or parts in the out-of-plane direction. Based on the clear distinctions outlined in the American standards, single-story reinforced concrete panels spanning from the base to the eave should not be classified and designed as "Part" when subjected to out-of-plane forces. However, both standards explicitly apply dynamic amplification for the diaphragm flexibility effects in the design of the panels' connections.

4.1 Structural walls and their anchorage out of plane design force: ASCE 7-22

ASCE 7-22, Section 12.11, provides clear and detailed requirements for designing structural walls and their anchorage. These requirements aim to ensure that structural walls can resist out-of-plane forces effectively and that their connections to diaphragms or other supporting elements are robust.

- **Design Force for Structural Walls:**

According to ASCE 7-22, structural walls must be designed to resist a force acting normal to their surface. This design force is calculated as:

$$F_p = 0.4 S_{DS} I_e W \geq 0.1W \qquad (1)$$

Where:
F_p: Design force acting on the structural wall (perpendicular to the surface).
S_{DS}: Design spectral response acceleration parameter at short periods (=C_h (0). Z for short periods in NZS 1170.5 Eq.3.1(1)).
I_e: Importance Factor, which reflects the building's seismic importance category (=R_u in NZS 1170.5 Table 3.5)
W: Weight of the structural wall.

- **Anchorage Requirements for Structural Walls**

Structural walls must also be anchored securely to diaphragms or other supporting elements. The anchorage must provide a direct connection capable of resisting a force calculated using the following formula:

$$F_p = 0.4 S_{DS} I_e k_a W_p \tag{2}$$

Where:
F_p: Design force in the individual anchors.
K_a: Amplification factor for diaphragm flexibility.
W_p: Weight of the wall tributary to the anchor.
The amplification factor, k_a, accounts for **the flexibility of the diaphragm** supporting the wall and is calculated as:

$$k_a = 1.0 + (0.0032\, L_f)/100 \leq 2.0 \tag{3}$$

Where:
L_f: Span (in mm) of the flexible diaphragm providing lateral support for the wall, measured between vertical elements supporting the diaphragm. For rigid diaphragms (L_f=0), ka is 1.0. The design force (F_p) for individual anchors must not be less than the larger of 0.2 Wp and 0.24 kN/m2 times the area of the wall tributary to the anchor.

The first term in Equations 1 and 2, "$0.4S_{DS}\ I_e$," aligns with the PGA definition in NZS 1170.5 Supp1 Appendix A, which is recommended for inertia loads when determining the design forces for lower-floor diaphragms at the building's overstrength level.

Adjustments for Non-Roof Anchorage and Rigid Diaphragms: For anchors not located at the roof level when diaphragms are not flexible, the force F_p can be reduced by applying a height-dependent factor:

$$\text{Reduction Factor} = (1+2\, z/h) / 3 \tag{4}$$

Where:
z: Height of the anchor above the structure's base.
h: Total height of the structure to the roof.

Commentary of ASCE 7-22 Cl.12.11 has explained that some force reductions have been allowed in equation 1 considering the combined inelastic behaviour of the main frame, wall (out of plane), and its anchors. However, it is hard to achieve the high level of reduction required for the seismic category of D and higher (i.e., $C_h(T)$. Z >= 0.5 for importance levels of 1 to 3) through the anchors alone. Therefore, the reduction must come from other parts of the structure yielding, such as the vertical seismic-resisting elements, diaphragms, or walls acting out of plane. The commentary has recommended the minimum required R-values for structures' lateral resisting systems in different seismic design categories:

- Seismic Category D (high seismic risk): Minimum R = 3.25, except for certain systems.
- Seismic Category C (moderate seismic risk): Minimum R = 2.0.
- Seismic Category B (low seismic risk): Minimum R = 1.5.

Establishing a clear correspondence between the ductility factor (μ) in NZS 1170.5 and the response modification coefficient (R) in ASCE 7 is challenging because the two standards approach seismic design from different perspectives. NZS 1170.5 incorporates ductility as a measure of the structural system's ability to deform plastically and dissipate energy, whereas ASCE 7 uses prescribed R values to represent the reduction in seismic forces achieved through inelastic behaviour, overstrength, and redundancy. However, review of Table 12.2-1 in ASCE 7-22, which specifies allowable R-values for various structural systems, reveals some useful insights. For instance, ordinary RC shear walls and ordinary steel moment-resisting frames achieve R-values of 4 and 3.5, respectively. Those system corresponds to nominally ductile systems as defined in New Zealand standards. This comparison suggests that systems designed for the minimum of nominal ductility under NZS 1170.5 could achieve similar performance levels as systems assigned R-values in ASCE 7-22 clause 12.11. To minimize confusion when drawing parallels between μ and R, it is useful to examine the framework provided in ASCE 41-23. Unlike ASCE 7-22, which simplifies seismic design for prescriptive compliance, ASCE 41-23 focuses on building performance, providing a more nuanced understanding of structural behaviour during seismic events. By using ASCE 41-23 as a supplementary reference, engineers can bridge the conceptual gap between NZS 1170.5 and ASCE 7-22. This approach offers greater clarity in aligning μ with R by focusing on equivalent performance outcomes rather than simplified force-reduction factors.

4.2 Structural walls and their anchorage out of plane seismic force: ASCE 41-23

Clause 7.2.13 of ASCE 41-23 addresses the evaluation and retrofitting requirements for structural walls and their anchorage. It distinguishes between structural walls and nonstructural walls, emphasizing that their design and evaluation must follow different provisions. Structural walls and their anchorage must be assessed for out-of-plane inertia forces per the requirements of Clause 7.2.13, while non-structural walls are evaluated using the provisions of Chapter 13, which deals with parts and components. Additionally, it has been highlighted that the actions resulting from the application of forces specified in Clause 7.2.13 are classified as **force-controlled**, meaning not allowed to exceed the nominal strength of the element being evaluated (i.e., no allowance for inelastic deformation). This requirement ensures that the wall and its anchorage design account for the full seismic demand without reliance on inelastic behaviour.

- **Out-of-Plane Seismic Force for Structural Walls**

According to ASCE 41-23, structural walls must have sufficient strength to span between locations of out-of-plane support when subjected to out-of-plane forces calculated using Equation 5:

$$F_p = 0.4 S_{XS} \times W \geq 0.1W \qquad (5)$$

Where:
S_{XS}: Spectral response acceleration parameter at short periods, which is equal to 90% of S_a.

S_a=2/3 S_{aM} where S_{aM} is the MCE$_R$ spectral response acceleration. Approximately, S_{XS} can be considered equivalent to 0.9C$_h$(T). Z for short periods in NZS 1170.5 Eq.3.1(1).
W: Weight of the structural wall.
X: Performance Factor from Table 1.

Table 1: Factor X for Calculation of Out-of-Plane Wall Force

Structural Performance Level	X
Collapse Prevention	0.8
Life Safety	1.1
Immediate Occupancy	1.7

- **Seismic Force for Out-of-Plane Walls Anchorage to Diaphragm**

Structural walls must also be anchored adequately to diaphragms or other supporting elements. The anchorage must establish a direct connection capable of resisting a force calculated using Equation 6.

$$F_p = 0.4S_{XS}.\, k_a\, .k_h\, .X\, .W_p \qquad (6)$$

$$F_{p,\,min} = 0.2\, k_a.X.\, W_p \qquad (7)$$

Where:
K_a: Amplification factor for diaphragm flexibility as per Eq. 3, equal to 1.0 for rigid diaphragms and need not exceed 2.0 for flexible diaphragms
K_h: Reduction factor as per Eq. 4 to account for variation in force over the height of the building when all diaphragms are rigid; for flexible diaphragms, use 1.0.
W_p: Weight of the wall tributary to the wall anchor
X: Performance Factor from Table 2.

Table 2. Factor X for Calculation of Out-of-Plane Wall Anchorage Force

Structural Performance Level	X
Collapse Prevention	0.9
Life Safety	1.3
Immediate Occupancy	2.0

A comparison between ASCE 7 and ASCE 41 reveals that the face load in the Life Safety performance level is the same for the panel (i.e., 0.9×1.1) and increased by a factor of 1.17 (i.e., 0.9×1.3) for its connection in ASCE 41. However, ASCE41, which is a standard specifically tailored for the seismic assessment of existing buildings, permits the use of the expected capacity of materials rather than the nominal capacity mandated by ASCE 7 and New Zealand's design standards (e.g., NZS 3101 or NZS 3404). Additionally, ASCE41 eliminates the requirement to apply strength reduction factors commonly used in new design standards. The permission for using expected capacity and the absence of strength reduction factors effectively offset the impact of the increased load factors in ASCE 41 for the connections. As a result, the overall structural performance requirements align closely with those in ASCE 7,

ensuring consistency in achieving reliable and safe seismic performance across both standards.

5 Case study and comparison between different approaches

This section evaluates the seismic face load on panels and their connections described in BRANZ Appendix C. The results are compared against different methodologies outlined in NZS 1170.5, ASCE 7, and ASCE 41 to assess their relative implications and accuracy. By comparing the seismic demands derived from these references, the study aims to highlight discrepancies, assess the conservatism of each approach, and identify any adjustments to achieve a balance between safety and cost-effectiveness in the design of RC panels and their connections.

BUILDING DESCRIPTION: Consider a warehouse structure with plan dimensions of 48 m by 30 m, located in Auckland on Subsoil Class C. The building features precast concrete panels along its sides, measuring 8.0 m high and 125 mm thick. These panels serve as both weather protection and structural bracing. The roof is supported by steel portal frames spaced at 8.0 m intervals, spanning the 30 m direction. The panels are supported at the eave level, 8.0 m above the ground. The concrete used in the panels has a compressive strength of 40 MPa. This configuration represents a typical industrial building and serves as a basis for evaluating seismic face load demands and design methodologies for the panels and their connections.

- **BRANZ Approach**

As detailed in Section 2, the BRANZ approach calculates the seismic face load on the panel by assuming an average of the forces at the panel's top and bottom, derived from different sections 8 and 5 of NZS1170.5, respectively.

Face Load for Panel Design: Force at the top of the panel (Part – NZS 1170.5 Section 8):

$f= 1.875^2\ (EI/m)^{0.5}\ /(2\pi H^2)= 0.34\ \text{Hz}\ (T_p=1.05s)$
$C_i\ (T_p)=2\times(1.75-T_p)=1.4$
$C\ (0)=C_h(0)\times Z\times R\times N(T,D)=1.33\times0.13\times1.0\times1.0=0.17$
$C_{hi}=1+h_i/6=1+8/6=2.3$
$C_p\ (T_p)=C\ (0)\times C_{hi}\times C_i\ (T_p)=0.17\times2.3\times1.4=0.55$
$C_{ph}(\mu=1.25)=0.85$
$F_{ph}=C_p\ (T_p)\times C_{ph}\times R_p\times W_p=0.55\times0.85\times1\times W_p=0.47W_p$

Force at the bottom of the panel (Independent – NZS 1170.5 Section 5):

$C=C_h(T)\times Z\times R\times N(T, D)=1.19\times0.13\times1\times1=0.15$
$F_{ph}=C\times W_p=0.15W_p$

Average force:

Mean= (0.47+0.15)/2×Wp=0.31Wp

Face Load for Panel Top Connection Design: Force derived using NZS 1170.5 Section 8 (assuming elastic connection).

$C_{ph}(\mu=1.0) = 1.0$
$F_{ph}=C_p\,(T_p)\times C_{ph}\times R_p\times W_p=0.55\times 1.0\times 1.0\times W_p=0.55W_p$

- **NZS1170.5 Approach**

The NZS 1170.5 methodology, as described in Section 3, calculates the face load on the panel and its connections based on Section 5 without considering the "Part" provisions in Section 8.

Face Load for Panel Design (assuming nominally ductile RC panel):

$T_p=1.05s$
$C(T1) = C_h(T)\times Z\times R\times N(T, D)=1.19\times 0.13\times 1\times 1=0.15$
$F_{ph}=C(T1)\times S_p\times W_p/\mu=0.15\times 0.9\times W_p/1.25=0.11W_p$

Face Load for Panel Connection Design (assuming elastic connection):

$F_{ph}=C(T1)\times S_p\times W_p/\mu=0.15\times 1.0\times W_p/1.0=0.15W_p$

- **ASCE7-22 Approach**

The ASCE 7-22 approach calculates seismic face loads using PGA-based inertial forces.

Face Load for Panel Design:

$F_{ph}=0.4S_{DS}\times I_e\times W_p=0.4\,(2.36\times 0.13)\times W_p=0.12W_p$

Face Load for Panel Top Connection Design:

$k_a=1.0+0.0032\times L_f/100=1.25$
$F_{ph}=0.4S_{DS}\times I_e\times k_a\times W_p=0.15W_p$

- **ASCE41-23 Approach**

The ASCE 41-23 methodology incorporates performance factors for seismic evaluation.

Face Load for Panel Assessment:

$F_{ph}=0.4S_X\times X\times W_p=0.12W_p$

Face Load for Panel Top Connection Assessment:

$Fph=0.4S_X\times k_a\times k_h\times X\times W_p=0.17W_p$

In summary, the results indicate that the NZS 1170.5 Section 5 approach aligns well with ASCE7 and ASCE41 in achieving life safety performance. However, the BRANZ methodology appears overly conservative (i.e., more than 3 times) and inappropriate to use in the design and assessment of industrial buildings' structural walls.

Table 3. Face Load Comparison

Approach	For Panel	For Panel top connection
BRANZ	0.31 W_p	0.55 W_p
NZS1170.5 (Section 5)	0.11 W_p	0.15 W_p
ASCE7-22	0.12 W_p	0.15 W_p
ASCE41-23	0.12 W_p	0.17 W_p

In general, a comparison between the NZS 1170.5 and ASCE 7-22 approaches for calculating panel face loading reveals that while the Section 5 NZS1170.5 approach does not explicitly allow for the diaphragm flexibility amplification, it is more conservative than the ASCE 7 approach when the panel is stiff enough. In other words, for the common spans between the portal frames (Lf<10m), when nominal ductile panel's period is less than 0.9 seconds for subsoil classes A, B, and C, and less than 1.3 seconds for subsoil class D (Figure 2). Conversely, for more flexible panels with longer periods exceeding these thresholds, the ASCE 7 approach becomes more conservative. A similar trend can be observed for the panel's elastic top connection (i.e, S_p=μ=1.0).

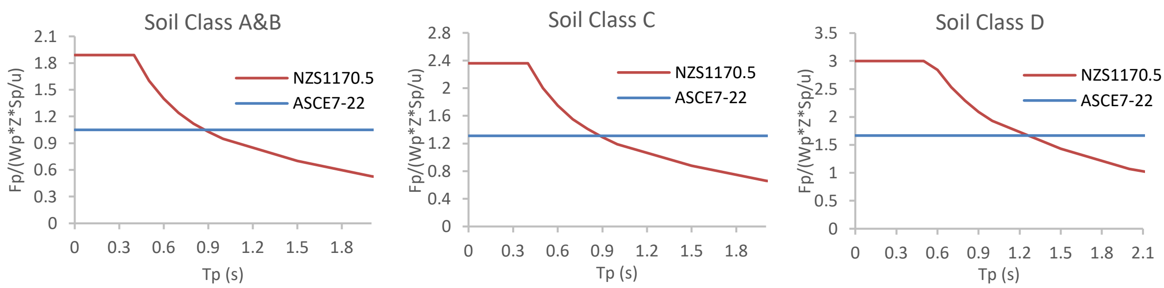

Figure 2. Comparison between panel face load in different approaches

6 Conclusion

Steel portal frames with concrete wall panels (precast or tilt-up) are prevalent in industrial buildings, serving as primary gravity and lateral load-resisting systems as well as cladding. However, current practices in New Zealand face challenges stemming from ambiguous provisions and guidance that lead to inconsistent interpretations and applications. In particular, the classification of RC walls as "Part" under Section 8 is contentious and may result in overly conservative designs. This study evaluated and compared different approaches, and the key findings are summarized below:

- While amplification due to diaphragm flexibility is recognized by BRANZ, its application remains insufficiently justified.
- BRANZ recommends applying Section 8 of NZS 1170.5 for out-of-plane seismic loads, which can result in excessive reinforcement and overly stringent connection requirements (approximately more than 3 times).
- According to NZS 1170.5, RC walls directly supported by the ground should be designed as independent structures under Section 5 without applying dynamic amplification. However, ambiguities in the standard's wording often lead to their misclassification as "Part".
- American standards, such as ASCE 7-22 and ASCE 41-23, clearly distinguish between structural and nonstructural components. These standards ensure that RC walls are designed under structural provisions, not as "Part," and provide explicit formulas for design forces and anchorage requirements, incorporating diaphragm flexibility effects.
- The findings suggest that the Section 5 approach in NZS 1170.5 aligns well with the methodologies outlined in ASCE 7 and ASCE 41, achieving life safety performance effectively. As a result, it is recommended to avoid misclassifying ground-supported RC walls as "Part" and instead use Section 5 for a more rational and efficient design.
- The research revealed while the Section 5 NZS1170.5 approach does not explicitly allow for the diaphragm flexibility amplification, its method is more conservative than the ASCE 7 approach once the nominally ductile panel is stiff enough with period of less than 0.9 seconds for subsoil classes A, B, and C, and less than 1.3 seconds for subsoil class D. A similar trend can be observed for the panel's elastic top connection (i.e, Sp=μ=1.0).

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Seismic Force for Diaphragm Design of Tall Buildings

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ABSTRACT

The increasing demand for high-rise buildings in New Zealand has highlighted significant gaps in current seismic design standards, particularly regarding floor diaphragm forces. While NZS 1170.5 and its commentary provide guidelines for diaphragm design in buildings up to nine stories, their applicability to taller structures remains uncertain. This study evaluates the limitations of the pseudo-Equivalent Static Analysis (pESA) method prescribed in NZS 1170.5 and its potential "misapplication" in mid- and high-rise buildings design. Additionally, the study examines the findings of the UOC-6378 report on diaphragm seismic performance, which recommends extending the pESA method to buildings beyond nine stories under specific conditions.

A series of Response Spectrum Analyses (RSAs) were conducted on five buildings of varying heights (5 to 40 stories) and plan irregularities, considering different soil conditions in Auckland and Wellington. The results indicate that while the pESA method is adequate for low to mid-rise geometrically regular buildings, it significantly underestimates diaphragm forces in taller and irregular structures. This study underscores the need for revising NZS 1170.5 to incorporate more accurate force estimations for high-rise buildings. It advocates for a more refined methodology that aligns with international best practices, ensuring seismic resilience and structural safety in the design of high-rise buildings in New Zealand.

Keywords: Diaphragm acceleration, tall buildings, irregular buildings, soft soil condition, seismic performance

1 Introduction

While New Zealand standards, such as NZS 1170.5 and its commentary, provide comprehensive guidance and well-established requirements for designing structural diaphragms and their components in low- to medium-rise buildings up to nine stories, they fall short in addressing the seismic design forces required for taller buildings. This gap in the standards has led to notable challenges in designing diaphragms for high-rise buildings, primarily due to ambiguous interpretations of appropriate seismic design forces. These uncertainties have introduced inconsistencies in design practices, potentially compromising the safety, efficiency, and performance of tall structures under seismic conditions in the New Zealand environment.

This study evaluates the provisions of NZS 1170.5 and the UOC-6378 report "Seismic Performance of the Diaphragms of Buildings" and also examines the effects of height, irregularity, and soil condition on diaphragm design force. The primary objective is to develop clear and robust guidelines that align with the unique demands of New Zealand's high-rise structures. Furthermore, the study advocates revisions to address current ambiguities and bridge the gaps in existing design requirements for tall buildings

2 NZS1170.5 and NZS1170.5 Supp 1 prescribed approach

According to NZS1170.5 clause 5.7, diaphragms shall be considered integral to the primary structure, with clearly defined internal load paths to transfer forces between the diaphragms, their connected elements, and the lateral force-resisting system. All diaphragm components and their connections to the lateral force-resisting system must be capable of accommodating both the imposed displacement and force demands. Additionally, it has prescribed that diaphragm design actions shall account for all the following:
(a) Permanent (dead) and imposed (live) loads acting on the floor.
(b) Floor design accelerations.
(c) Force transfer between interconnected lateral force-resisting elements.
(d) Interaction with vertical elements supporting the diaphragm.
(e) Deformations between diaphragm components.
It also emphasized that the diaphragm design shall also consider higher mode effects and overstrength effects from the overall structure, as specified in Clause 5.6.

The broadly defined requirements of Clause 5.7 allow for varied interpretations and applications, potentially leading to inconsistency in structural design approaches. However, NZS 1170.5 Supp 1 Appendix A to C5.7 provides more explicit guidance on determining diaphragm design loads using the pseudo-equivalent static analysis (pESA) method. This method, widely recognized and generally applied to buildings up to nine stories, is derived from the Equivalent Static Analysis (ESA).

While ESA is permitted only for buildings less than 10 m high (approximately 3 storey) as per NZS1170.5, the pseudo-ESA (pESA) method extends the height limitation from 3 to 9 storey. NZS 1170.5 Clause 6.2.1.3 and its commentary apply a simplified higher-mode correction by introducing a lateral force $F_t = 0.08V$ at the top level only, assuming that higher-mode effects primarily influence the uppermost storeys. However, this simplified higher mode correction is not applied at mid to low levels, making both ESA and pESA methods inadequate and potentially misleading for medium and high-rise buildings.

Additionally, irregular buildings experience complex dynamic behavior that the ESA method does not adequately capture, further limiting its applicability. Consequently, since the pESA method is built upon ESA principles, it inherits the same fundamental limitations.

The following sections further explore and critically analyse those issues, assessing the suitability of the pESA method for such scenarios.

3 The UOC-6378 report [3] recommendation

In 2018, Holmes Consulting was engaged by the University of Canterbury and the Ministry of Business, Innovation and Employment (MBIE) to research the seismic performance of concrete diaphragms (floors) in reinforced concrete buildings. The study aimed to evaluate various methods used both in New Zealand and internationally for determining the lateral force distribution to floor diaphragms during earthquakes, ensuring a reasonable estimation of the forces generated.

A key focus of the study was comparing these methods to the "pseudo-Equivalent Static Analysis" (pESA) approach outlined in NZS 1170.5 commentary. Additionally, the report incorporated findings from the PTL research, which compared peak floor accelerations across seven different building types with lateral force distributions reported from various references and nonlinear time history analysis (NLTHA) results. PTL's analysis of the diaphragm acceleration time histories revealed that acceleration peaks were generally of short duration, suggesting that maximum accelerations were influenced by high-frequency energy content. PTL concluded that while these peaks are present, their potential to cause significant structural damage is debatable, as structural damage is more closely associated with displacement rather than force. To filter out high-frequency energy content from the acceleration response, PTL proposed a low-pass filter to be applied to the numerical results. This filtering process removed energy from frequencies above a designated cutoff value, which was determined for each building to ensure that at least 90% of the total modal participating mass was included. This approach aligns with the response spectrum analysis principles outlined in NZS 1170.5 Cl.6.3.3.
Holmes' report attempts to answer a critical question of "*Can the pseudo-Equivalent Static Analysis method (pESA) be applied to buildings taller than nine stories, despite the limitation suggested in NZS 1170.5:2004?*" The report ultimately concluded that "*the PTL study indicated that for one building in the analysis, the pESA method could potentially be extended to 12 stories. More broadly, it may be possible to apply the pESA method regardless of height, provided that the dominant displaced shape of the building follows the 'first translational mode'.*" In our opinion, Holmes' report conclusion is not totally definitive since the study has examined a limited sample of buildings, almost all of which were under nine stories. The only exception, Building D, a 12-story building with soil class C, showed that the pESA method underestimated diaphragm forces by 30% to 100% when compared to nonlinear time history analysis (NLTHA), as shown in Figure 1.

Additionally, Holmes' report has not considered the effects of irregularity and changes in the soil subclass on the floor diaphragms' acceleration. In the next section of this paper, these issues are investigated further by performing response spectrum analyses for a series of buildings with different heights, irregularity, and soil subclasses to assess the validity of the pESA method.

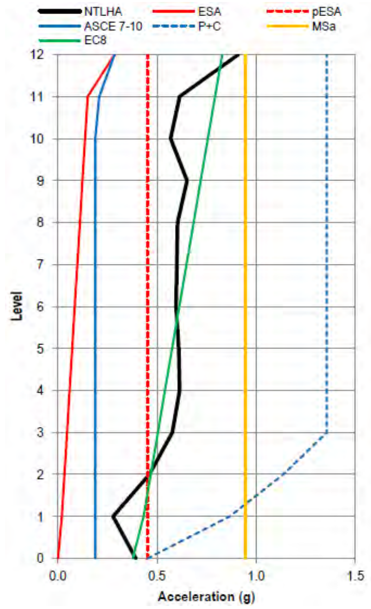

Figure 1: PTL's Building D. Diaphragm acceleration low pass filtered envelopes.

4 Building Descriptions

This paper investigates five office buildings that have horizontal irregularity in one direction (Y-dir.), as shown in Figure 2. The buildings have a similar geometry of rectangular shape, including 7.0m spans in each direction and different heights of 5, 7, 9, 25, and 40 stories. The buildings' lateral resisting system consists of RC shear walls (Core walls). The floors are 200 mm thick suspended slabs that are supported by 20 gravity columns positioned every 7 m span. The buildings are investigated using Response Spectrum Analysis (RSA) for the scenarios of Soil Classes C and D and buildings located in Auckland and Wellington. CSI ETABS Ultimate version 22.1.0 was used to model and analyse the buildings.

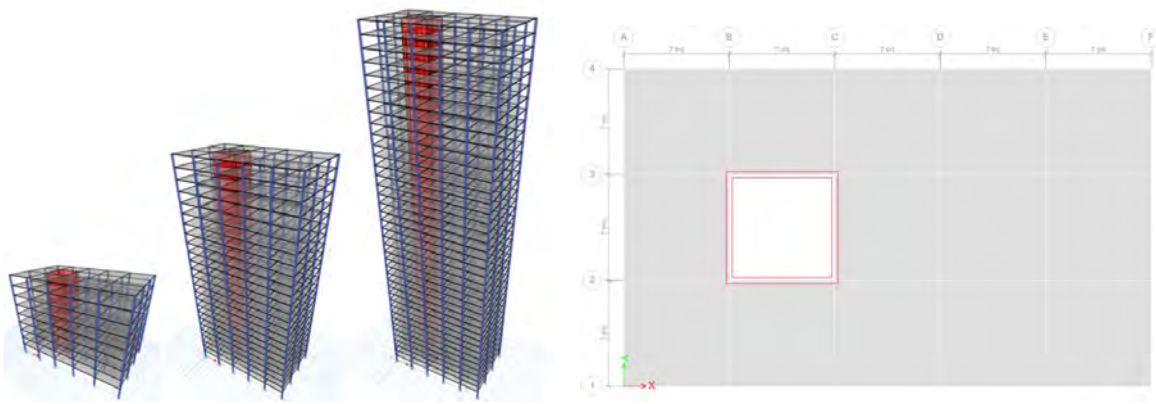

Figure 2: Nine, twenty-five and forty storey buildings' 3D and plan views

The structural walls have typical dimensions of 7 m in length and 3 m in inter-storey height at all levels. The walls' thickness varies in height and consists of 400, 300, and 200 mm. It is assumed that the walls have been designed and detailed following the provisions outlined in NZS1170.5 and the nominal ductile detailing requirements of NZS3101.

5 Results and Discussion

In this study, a series of Response Spectrum Analyses (RSAs) have been conducted for all the buildings described in Section 4 of this paper. The analysis includes a sufficient number of modes to ensure that at least 90% of the total mass of the structure participates in the direction under consideration, providing a comprehensive representation of dynamic behaviour. To mitigate the influence of high-frequency modes, which primarily contribute to localized vibrations rather than significant structural damage, the number of included modes has been carefully controlled. Only the necessary modes have been considered to achieve the 90% cumulative mass participation threshold, ensuring that the analysis remains both computationally efficient and structurally relevant.

For consistency in comparison, an overall building overstrength factor of 2.08 ($Ø_{ob} \approx 1.5\mu/S_p = 1.5 \times 1.25 / 0.9$) has been applied to both the RSA accelerations (Ry and Rx) and the ESA accelerations. While this factor may be higher for Wellington, it does not impact the comparative analysis since it has been uniformly applied across all cases. Additionally, to account for the strength reduction factor of 0.75, as specified in NZS 1170.5 Supp 1, the Peak Ground Acceleration (PGA) has been amplified by a factor of 1.33, as illustrated in Table 1. This adjustment ensures that the analysis appropriately reflects the intended seismic demand while maintaining consistency in methodology. As presented in Table 1, the PGA for soil subclass C is more than Class D. While softer soils (Class D and E) amplify motion, they tend to dampen the acceleration due to increased energy dissipation (e.g., through plastic deformation). In contrast, Class C soils retain more of the seismic energy, leading to a higher observed PGA.

Table 1: PGA used for pESA acceleration estimation

	Soil Subclass	PGA =C (0)	1.33 PGA
Wellington	D	0.45	0.60
	C	0.53	0.71
Auckland	D	0.15	0.20
	C	0.17	0.23

In the following pages, floor diaphragm accelerations for the analysed buildings are presented in Figures 3 to 7 for both the irregular (Y-dir.) and regular (X-dir.) directions, covering all previously discussed scenarios. The pESA accelerations have been determined following the procedure outlined in Appendix A of the NZS 1170.5 Commentary. Furthermore, floor accelerations obtained from Response Spectrum Analysis (RSA), amplified by the overall building overstrength factor, and considered at each floor level. This approach provides a

reasonable and practical method for determining inertial demands, as RSA effectively captures the dynamic response of the structure under seismic loading. By comparing the RSA results with the pESA estimates, a more comprehensive understanding of diaphragm acceleration behaviour was achieved.

The key findings from the analysis, as illustrated in Figures 5 to 7, indicate that the pESA method provides a reasonable estimation of floor accelerations at all levels for 5, 7, and 9 storey buildings in the regular (X) direction across all soil subclasses. However, in the irregular (Y) direction, while pESA accurately estimates upper-level accelerations, it significantly (more than 50%) underestimates those in the lower half of the structure for all soil subclasses. This discrepancy arises from the lateral force component F_t (= 0.08V) in the Equivalent Static Analysis (ESA) load distribution prescribed by NZS 1170.5. Since this force is applied at the top of the structure, it compensates for higher-mode and irregularity effects in the uppermost stories but leaves the lower levels inadequately accounted for in the pESA approach.

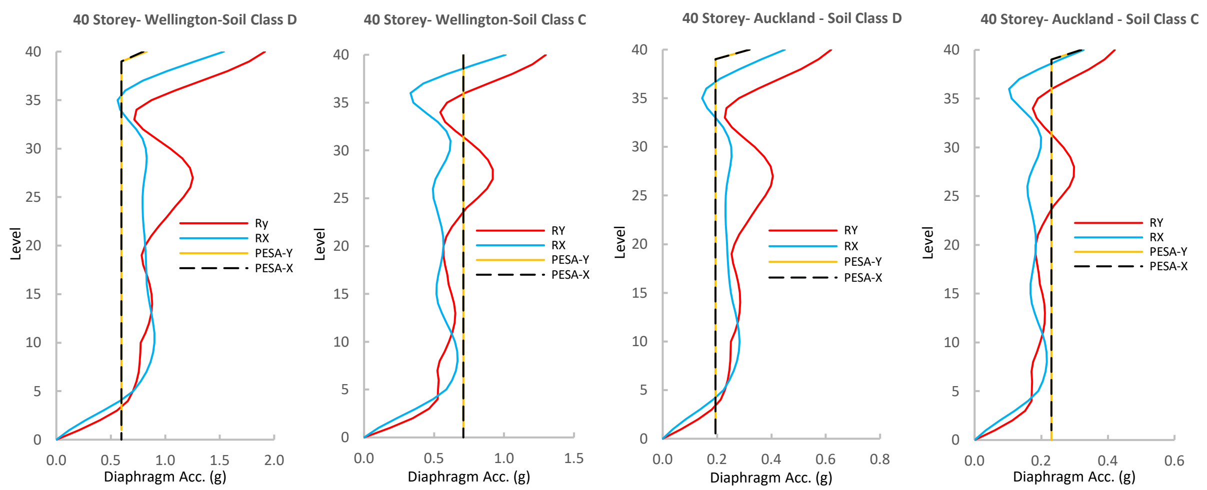

Figure 3: 40 storey building diaphragm acceleration

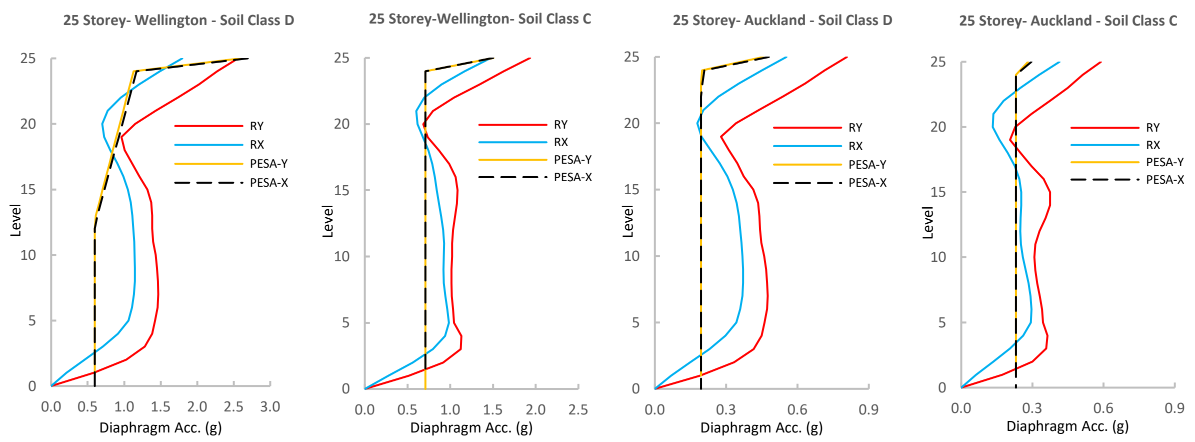

Figure 4: 25 storey building diaphragm acceleration

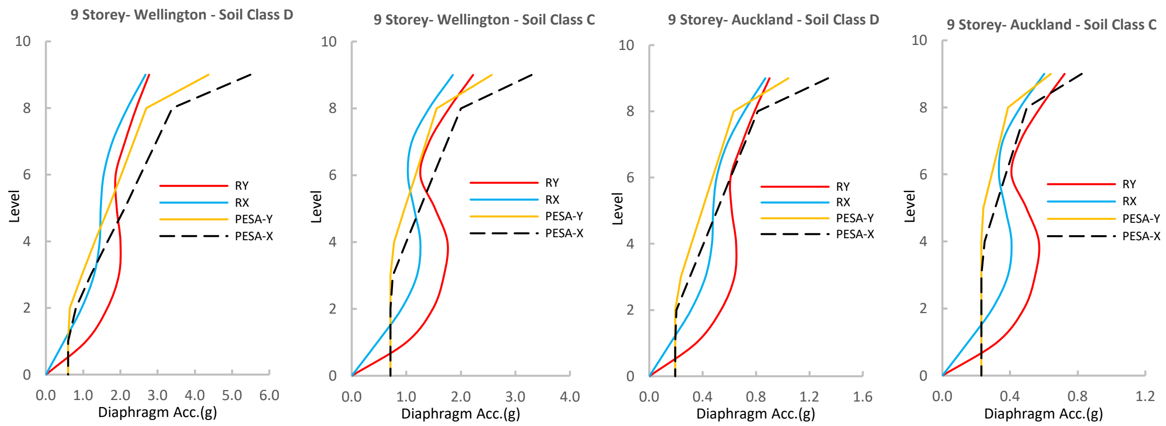

Figure 5: 9 storey building diaphragm acceleration

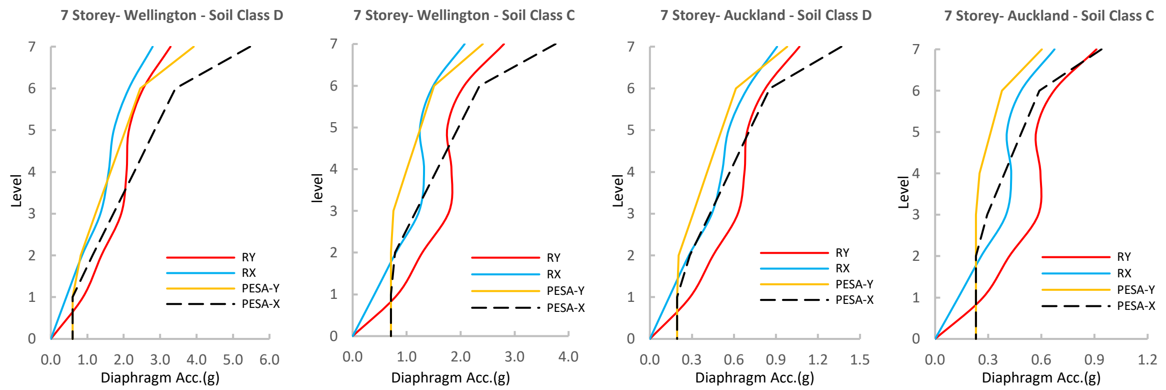

Figure 6: 7 storey building diaphragm acceleration

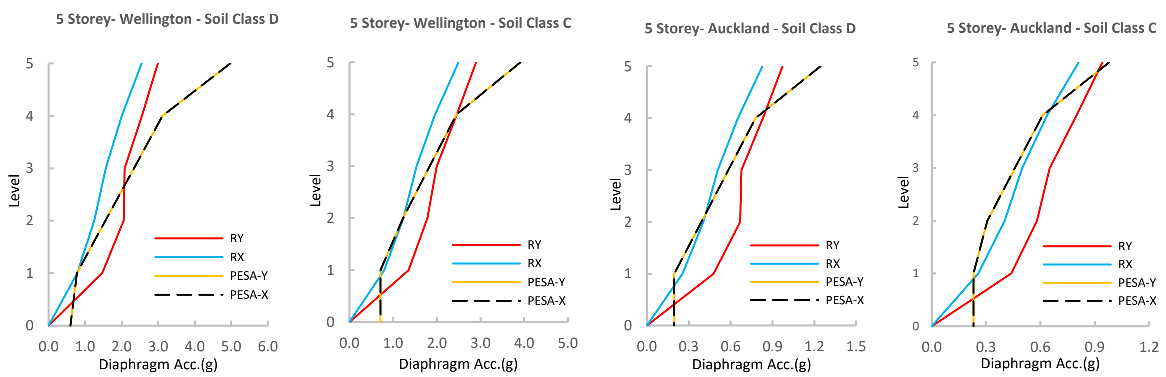

Figure 7: 5 storey building diaphragm acceleration

For 25 and 40 storey buildings with Soil Subclass C, the pESA method reasonably estimates floor accelerations at all levels in the regular direction. However, in the irregular direction with Soil Subclass C, pESA underestimates accelerations in the upper half of the structure. Additionally, when considering Soil Subclass D, pESA consistently underestimates floor accelerations at all levels in the irregular direction (Figures 3 and 4). These findings highlight pESA's limitations in capturing floor accelerations, particularly in taller buildings, irregular configurations, and soil conditions that influence prediction accuracy.

While the pESA method does not account for building irregularities, the study's findings highlight the significant impact of irregularity on floor diaphragm acceleration. In most buildings, the centre of mass (CM) and centre of rigidity (CR) are not perfectly aligned. Under lateral seismic forces, this misalignment induces torsional motion in addition to translational movement. As the structure rotates due to torsion, points farther from the center of rotation experience higher accelerations, combining both rotational and translational effects. The study indicates that the pESA method has limitations in predicting diaphragm response.

6 Conclusion

This study investigated the appropriateness of the pseudo-Equivalent Static Analysis (pESA) method recommended in the NZS 1170.5 Commentary, assessed the influence of building height, irregularity, and soil subclass on diaphragm accelerations through a series of response spectrum analyses. The findings highlighted key limitations of the pESA method, particularly in its ability to estimate diaphragm seismic forces accurately in mid- to high-rise buildings.

While the pESA method provides reasonable estimations for low-rise regular structures, its accuracy diminishes as building height increases and as irregularities and soft soil conditions become more pronounced. The analyses conducted on buildings with varying heights and soil conditions demonstrate that pESA significantly underestimates diaphragm forces in taller structures, particularly those with irregular configurations and soft soil conditions. This underestimation can lead to "unconservative" diaphragm forces, potentially compromising the seismic performance of the structure.

While the UOC-6378 report concluded that the pESA may be extended beyond the nine-story limitation in select cases, its applicability remains highly dependent on a building's dynamic characteristics. The RSAs results indicate the assumption that higher-mode effects primarily influence only the upper levels is inappropriate for mid- and high-rise buildings, where significant dynamic contributions can also occur at lower levels. This calls for a more rigorous methodology that better accounts for complex dynamic behaviours in diaphragm force estimations.

To improve the accuracy and reliability of diaphragm force predictions, this study suggests that floor accelerations obtained from response spectrum analysis ($\varnothing_{ob}$. Rx and $\varnothing_{ob}$. Ry) at a given floor level represent a reasonable approach to determining inertial demands at that floor. Additionally, enveloping these demands with pESA inertial demands would provide a more

comprehensive assessment, ensuring a conservative design approach that captures both static and dynamic effects. This approach can be considered as an “Alternative Solution” procedure in terms of the building code compliance requirements. It is important to note that floor accelerations obtained from response spectrum analysis should not be applied uniformly over the full building height when determining diaphragm transfer demands. Since unsigned modal combination response spectrum floor accelerations do not represent a consistent set of design actions, they cannot be directly used to derive diaphragm transfer forces.

Ultimately, these findings reinforce the necessity of refining seismic design methodologies for diaphragms in tall and irregular buildings. Future revisions of NZS 1170.5 should incorporate enhanced analytical frameworks to ensure that diaphragm forces are accurately captured, thereby improving structural resilience and seismic safety.

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Fire Protection Optimisation for Low Carbon Design

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ABSTRACT

The transition toward circular low-carbon design represents a fundamental shift in building practices, aiming to reduce resource consumption, minimise waste, and lower carbon emissions. A structured framework outlines key circular strategies and solutions for new low-rise buildings, including design for fire resilience, longevity, and durability. Fire resilience is critical to ensuring structural integrity during fire events, safeguarding both occupants and assets for a specified duration.

A performance-based fire design approach enables targeted fire safety solutions by integrating multiple strategies, including active fire protection, passive systems, and optimised design processes. This paper explores fire protection optimisation within a low-carbon framework by leveraging the inherent strength of structural steel to minimise reliance on intumescent coatings. By comparing different fire design scenarios, the study demonstrates how sustainable fire protection strategies can enhance material efficiency while maintaining structural performance.

Fire protection optimisation is examined through two key approaches: (1) incorporating active fire systems, such as sprinklers, to reduce fire resistance rating (FRR) requirements, and (2) refining intumescent coating application using the HERA Intumescent Tool (H.I.T.), which facilitates efficient material use while ensuring compliance with fire safety objectives. The findings highlight a pathway toward resource efficient fire design that aligns with circular economy principles, contributing to more sustainable and resilient buildings.

INTRODUCTION

Circular low-carbon design represents a fundamental shift from a traditional linear economy towards a regenerative model. This approach integrates fire resilience and protection strategies to ensure long-term material sustainability and structural safety, while prioritising resource efficiency, waste reduction, and carbon minimisation (Baker-Brown 2017, Cheshire 2021).

By focusing on the reuse, repurposing, and refurbishment of existing structures, circular design extends the service life of materials and reduces the demand for virgin resources (Cheshire 2021). A core principle is to minimise waste generation and pollution through designing buildings for durability, adaptability, and ease of deconstruction. This ensures materials can be efficiently reclaimed and reintegrated into new construction cycles, thus reducing both upfront and whole-life carbon emissions. Fire resilience forms a crucial component of this strategy, as fire-damaged materials often result in significant waste and carbon emissions (McNamee & Meacham 2023). Enhancing fire resistance in structures aligns with circular economy principles by extending material lifespans and reducing replacement needs.

A critical aspect of sustainable structural design lies in maximising material utilisation to enhance efficiency and mitigate environmental impacts. Studies have revealed inefficiencies in material use across various structural elements, including steel beams and concrete members. In these cases, conservative design practices often lead to excessive material consumption. By adopting strength-governed rather than serviceability-governed design approaches, it is possible to improve utilisation ratios. This reduces the need for surplus materials without compromising structural integrity. The approach involves refining strength and deflection criteria in structural modelling to prevent over-specification, ensuring materials are used optimally. Additionally, fire protection strategies can further enhance material efficiency by leveraging performance-based fire design to reduce reliance on thick passive fire protection coatings.

Beyond material efficiency, sustainable design involves holistic strategies that integrate resilience, longevity, and adaptability (Andisheh & ShahMohammadi 2024). Fire resilience plays a critical role in preserving structural integrity and minimising material waste due to fire damage. Fire safety considerations ensure that materials maintain their structural performance over extended lifespans, supporting the broader objectives of circular design. A performance-based fire design approach enables targeted fire safety solutions by integrating multiple strategies. These include active fire protection, passive systems, and optimised design processes. For example, incorporating active fire suppression systems such as sprinklers can reduce the required fire resistance rating (FRR) of structural elements, minimising the need for extensive fire protection measures.

Furthermore, tools like the HERA Intumescent Tool (H.I.T.) allow for precise optimisation of intumescent coatings. This ensures compliance with fire safety objectives while promoting material efficiency. As the construction industry moves towards a more sustainable future, adopting circular low-carbon design practices presents an opportunity to balance structural performance with environmental responsibility. Fire protection strategies play a key role in achieving this balance. Optimising intumescent coatings and incorporating active systems like sprinklers enhances fire resilience while minimising material use and carbon emissions. By prioritising material efficiency, durability, and adaptive reuse, the industry can significantly reduce its carbon footprint. This approach fosters long-term resilience and sustainability in the built environment. The integration of optimised fire protection strategies within this framework ensures that structures remain safe, resource-efficient, and aligned with circular economy principles. Ultimately, this contributes to more resilient and environmentally responsible

buildings. The reusable EBF from the design example of P4001 focuses on the frame for seismic analysis (HERA report P4001 2013). However, gravity loading must be considered for the fire design. To address this, a scenario is considered where the floor arrangement is based on a similar floor area as used in the seismic calculations (300 sqm).

DESIGN, FIRE PROTECTION AND OPTIMISATION PROCESS

Using the floor arrangement shown in Figure 1, gravity forces acting on the collector beam and column are calculated for the fire scenario. Since these members were already designed for seismic forces and optimised, the revised and optimised members were considered for fire design of the frame. The structural members (columns and collector beams) were designed for fire load conditions as per NZ standards (AS/NZS 1170.0:2002). NZS 3404 (NZS 3404 Part 1&2, 1997) is currently under revision, with some equations in the revised version proposed to change. These include equations for limiting temperature and time to reach limiting temperature. Therefore, in this paper, the proposed revised version of NZS 3404 is used for the design of structural steel members.

After verifying the members through fire design checks, fire protection optimisation is performed. In this paper, intumescent paint is used as a fire protection strategy. The optimisation of fire protection focuses on reducing the quantity of fire protection materials, specifically intumescent paint, without compromising the structural performance during fire conditions. In this case, two strategies are employed to achieve optimisation: 1. Incorporating active fire systems, such as sprinklers, to reduce the required fire resistance rating (FRR). 2. Refining the intumescent coating requirements using the HERA Intumescent Tool (H.I.T.), which ensures material efficiency while maintaining structural performance.

Intumescent Paint

Once the limiting temperature and section factor are known, the minimum required thickness of fire protection coatings can be obtained by using the loading tables provided by coating suppliers. In this paper, intumescent table data was obtained from Sherwin Williams. For instance, based on the loading table presented below, the maximum limiting temperature value in the table is 750 °C. However, the collector beam limiting temperature is 761 °C. Therefore, a conservative value of 750 °C is chosen. Applying 281 μm of thick intumescent coating could protect the steel collector beam from failing before 60 min under ISO-834 standard fire.

HERA Intumescent Tool

The HERA Intumescent Optimisation Tool (H.I.T.) is designed to help steel fabricators and specifiers reduce the cost of intumescent coatings. The tool combines key data from its library, including steel section properties and coating requirements. It accounts for factors such as: Section shape, Section factor, Fire exposure, Fire resistance rating (FRR), design temperature. Using a simplified 'maxi-calculator' approach, H.I.T. minimises the amount of input needed while providing clear and actionable recommendations. It can also suggest alternative steel sections with the same or greater capacity, helping users select cost-effective options.

Active Fire Protection System

Active fire protection systems play a crucial role in fire detection and early warning. They facilitate timely intervention to control and extinguish fires during their initial growth stage. Automatic sprinkler systems, in particular, are highly effective in reducing fire risks within buildings. For an office building requiring a 60-minute fire rating, Acceptable Solution C/AS2 Table 2.4 indicates that incorporating sprinklers can reduce the fire resistance requirement to 30 minutes for life safety. This demonstrates how integrating an appropriate active fire protection system, such as sprinklers, can optimise fire protection measures. Consequently, the structural fire design was revised to reflect the inclusion of the sprinkler system and an FRR of 30 minutes.

METHODOLOGY

The structural fire design of the Reusable Eccentrically Braced Frame (R-EBF) assumes: A predefined floor layout with column locations (Fig. 1), no secondary beams (i.e., floor load is directly transferred to collector beams), a roof load of 100 kN per EBF column. Office occupancy governs the imposed loading, i.e., 3.0 kPa. Although a moment connection exists between the collector beam and the column, it is conservatively assumed to be simply supported under fire conditions between the column and the brace support with a length of 4.2 m on either side of the shear link (Fig. 2).

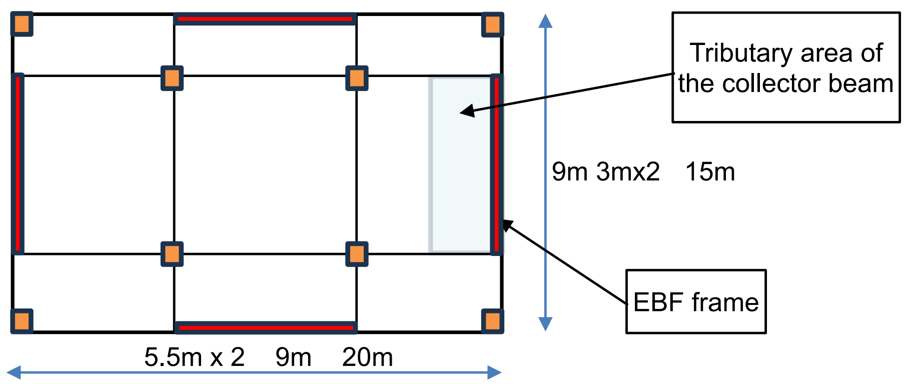

Figure 1: Floor layout

The design considers a total floor area of 330 m² with permanent load of 4 kPa. In the gravity system, the collector beam at level 1 or 8 will have the same loading. Therefore, we are adopting the least dimension of the collector beam, i.e., at level 8 for the design checking. However, a table is presented for collector beams at each level.

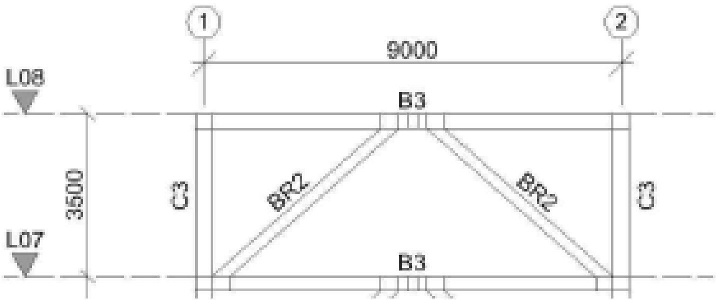

Figure 2: R-EBF elevation

ANALYSIS

The design of the beam is performed for fire load combination as per AS/NZS 1170.0:2002 i.e. $W^*_{u,pb,f} = G + 0.4Q$ 15.57 kN/m. Since the beam is simply supported between the column and the brace therefore the bending moment of the collector beam would be $M^*_{u,pb,f} = W^*_{u,pb,f}.L^2/8$ 34.33 kNm.

The capacity of steel beam at t 0 is read from the capacity result from ACI Structural Steel Capacity Design Table with proposed changes in Cl 11.6.5.3.2 of DR NZS 3404. This considers 1.5 times the design section capacity of the bare steel beam for positive moment as the section moment capacity of composite downstand beams supporting a concrete slab. The design capacity of the beam is $\phi M_{sx} = 133\, kNm\, x\, 1.5 = 199.5$. The load ratio of the beam is calculated as the ratio of demand and capacity without any capacity reduction factor, $r_{f,pb} = 0.155$. On using this load ratio in the limiting temperature equation, the limiting temperature is calculated following Clause 11.5 of NZS 3404:1997:

$$T_{l,sb} = 801e^{-\frac{r_{f,sb}}{1.24}} + 53.7 = 801 * e^{-\frac{0.16}{1.24}} + 53.7 = 761\ ℃;$$

Following Clause 11.6.3.1 as per DZ 3404 to calculate the temperature development of primary beams without protection in fire, the section factor of the beam is Am/V 170 m-1, the time for the unprotected collector beam to reach the limiting temperature is determined as:

$$t_r = -5.2 + 0.0221T_{l,cb} + \left(\frac{3.4\ T_{l,cb}}{A_m/V}\right) = -5.2 + 0.0221 * 761 + \left(\frac{3.4 * 761}{170}\right) = 26\ \text{min}$$

The collector beam does need to be protected if the FRR is 60 minutes.

Determine the required fire protection type and thickness

Given that the limiting temperature and section factor are obtained, engineers can determine the minimum required thickness of fire protection coatings using the loading tables provided by coating suppliers. For instance, based on the loading table, the table maximum limiting temperature value is 750 °C, however, collector beam limiting temperature is 761 °C. Therefore, a conservative value of 750 °C is chosen. Applying 281 μm thick intumescent could protect the steel collector beam from fail before 60min under ISO-834 standard fire.

Similar to Beam 200UC46.2, the calculations were performed for all the collector beams at each level and results are summarised in Table 1.

Table 1: Summary of structural fire design of collector beams at all levels.

Elements	Phi MSx (kNm)	Load Ratio	Limiting Temp (°C)	Am/V (m^{-1})	Failure time in Std Fire (min)	Paint thickness to reach limiting temp (μm)
200UC46.2	133	0.15	760.65	170	26.8	281.00
200UC52.2	154	0.13	772.80	151	29.3	258.00
200UC59.5	177	0.12	782.95	133	32.1	236.00
250UC72.9	266	0.08	806.21	134	33.1	237.00
250UC72.9	266	0.08	806.21	134	33.1	237.00
250UC89.5	309	0.07	812.78	111	37.7	210.00
250UC89.5	309	0.07	812.78	111	37.7	210.00
310UC118	494	0.04	828.21	122	36.2	223.00

DISCUSSION

Optimisation process

The optimisation of fire protection focuses on reducing the quantity of fire protection materials specifically intumescent paint without compromising the structural performance during fire conditions. In this case, two strategies are employed to achieve optimisation. When using the HERA Intumescent Tool for optimisation, we found that the limiting temperature of the collector beam is high at almost all levels. This is beyond the optimisation range of the HIT tool, as the intumescent certificate table doesn't have data for temperatures exceeding 750°C, Therefore, it is concluded that the beam sizes are optimum from the structural fire protection perspective on using HIT.

Initially, complete calculations were performed for an FRR of 60 minutes. However, by incorporating sprinklers in the building, the fire protection requirement reduces. This approach has been used in the fire design optimisation, reducing the requirement to an FRR of 30 minutes. After redesigning for 30 minutes, calculations were performed for all collector beams at each level, with results summarised in Table 2.

Table 2: Summary of structural fire design of collector beams at all levels.

Elements	Phi MSx (kNm)	Unprotected Beam time to Lim temp (min)	Fire protection required	Paint Thickness to reach limiting temp (µm)
200UC46.2	133	26.8	Yes	184
200UC52.2	154	29.3	Yes	184
200UC59.5	177	32.1	No	-
250UC72.9	266	33.1	No	-
250UC72.9	266	33.1	No	-
250UC89.5	309	37.7	No	-
250UC89.5	309	37.7	No	-
310UC118	494	36.2	No	-

The above table indicates that there is reduction in intumescent paint thickness requirement when the requirement reduced from 60 min to 30 min FRR. Although with the use of HIT tool the optimisation was possible but due to high limiting temperatures and limitations of the supplier's table for thickness calculations, the optimisation was not possible in this scenario. The below graph shows the percentage savings in the fire protection.

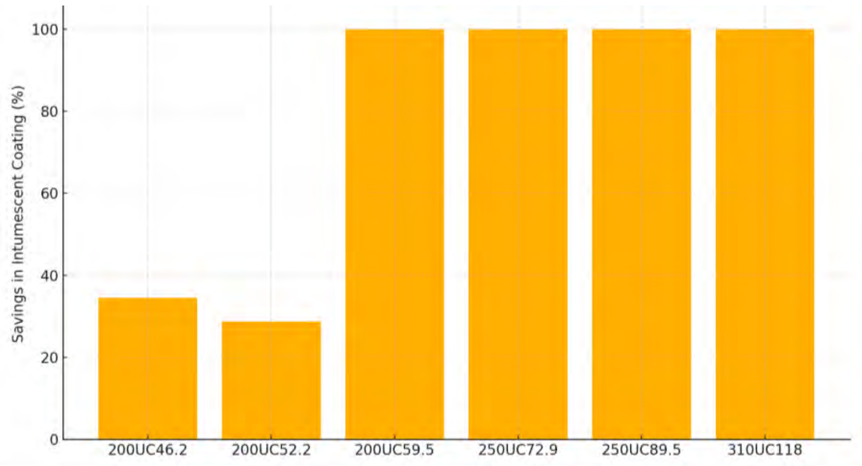

Figure 3: Percentage Savings in Intumescent Coating (Normal vs Optimised). The chart illustrates the substantial material savings achieved by reducing the fire resistance rating from 60 to 30 minutes.

Sustainability aspects

The optimisation of fire protection for steel structures not only enhances safety and reduces material usage but also contributes significantly to sustainability goals. There is a gap between the structural engineers' design and the fire engineer recommendations of the fire resistance rating requirement. In order to optimise the fire protection there is a several interactions involved between the different disciplines. Therefore, this HIT tool comes in handy where the specifiers (could be structural engineers, architects, fabricators and paint specifiers) can use to produce the cost-effective options by maintaining the structural integrity of the structure. This paper provides the clear information to the designers that with the use of the optimisation process there is a lot of saving in the fire protection material.

When steel structures are damaged beyond repair in fire events, the environmental cost of replacement is considerable. By implementing effective fire protection strategies that preserve structural integrity during fire events, buildings can potentially be restored rather than demolished and rebuilt, significantly reducing the carbon footprint associated with new steel production and construction. The life safety is the utmost priority, and this is engineers' responsibility to ensure that life safety is not compromised by providing the required fire resistance by the structural members and economising the design. In the process of cost optimisation, there is a certain reduction in material used in fire protection. The less the fire protection, there will be less carbon footprint at the end of life of the structure to recycle or reuse the steel, since steel is recyclable material.

In a separate study by the author, yet to be published, it is observed that, by using the HIT tool 65% of the times the results ended up saving carbon for the cost-effective options for the users for normal a\steel and 82% for the low carbon steel. This is a significant saving in the carbon footprint. The process of optimisation is in line with the low carbon design framework. If the steel is designed for less or no fire protection, then the material usage is optimised. At the end of life, during the recycling or reuse process, the steel requires less or no carbon footprint to make it ready for the next project which means there is less waste and faster recovery. This aligns with design for maximising material utilisation (steel is utilised maximum to reduce fire protection), design for longevity (no maintenance of fire protection required therefore longer life) and design for adaptability (steel can be reused/recycled quicker for next project).

CONCLUSION

For two 4.2m collector beams at each level of reusable EBF, it was observed that the incorporation of sprinklers in the building results in significant savings in intumescent coating requirements. 86% reduction achieved in the intumescent volume of the fire protection of the beams. This study demonstrates that fire protection optimization within a low-carbon design framework can be effectively achieved through two complementary approaches. First, by incorporating active fire systems such as sprinklers, the required fire resistance rating can be reduced, resulting in substantial material savings. Second, by utilizing specialized tools like the

HERA Intumescent Tool (H.I.T.), the application of intumescent coatings can be refined to ensure efficient material use while maintaining compliance with fire safety objectives.

The findings highlight a clear pathway toward resource-efficient fire design that aligns with circular economy principles. By leveraging the inherent strength of structural steel and optimizing fire protection strategies, it is possible to enhance material efficiency while maintaining structural performance during fire events. This approach not only contributes to more sustainable building practices but also demonstrates how performance-based fire design can be integrated into the broader framework of low-carbon construction. Future research could explore additional optimisation strategies, such as the use of alternative fire protection materials or innovative structural configurations that further reduce the need for protective coatings. Additionally, life cycle assessment studies could quantify the environmental benefits of these optimisation approaches in terms of carbon emissions and resource consumption. In conclusion, this study provides valuable insights for structural fire engineers, architects, and sustainability professionals seeking to balance fire safety requirements with environmental considerations in building design and construction.

ACKNOWLEDGEMENT

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Ivey West and Memorial Hall Revitalisation

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ABSTRACT

Ivey West and Memorial Hall are located at the Lincoln University campus approximately 20km south west of Christchurch. These buildings are assigned Category 1 heritage status and were damaged during the 2010-2011 Canterbury Earthquake sequence. The buildings were temporarily secured and sealed off post-earthquakes awaiting future repairs. The damage was extensive and included collapse of gable walls, dormers, cracking to URM walls and various decorative elements. Lincoln University as the owners of the buildings approved the revitalization of the earthquake damaged buildings around 2022. As part of this revitalisation the buildings were seismically strengthened and the earthquake damage repaired. This paper outlines the structural strengthening that was implemented, the repairs that were undertaken and challenges faced implementing the strengthening.

INTRODUCTION

The Canterbury Earthquake Sequence (CES) caused significant damage to Ivey West and Memorial Hall located at Lincoln University near Christchurch. These buildings were of unreinforced masonry (URM) construction dating from 1881 to 1920.

Ivey West is a two storey building of solid URM masonry construction. Memorial Hall is a single storey building with a cavity between the two inner brick whythes as the load bearing wall and a single outer brick whythe acting primarily as cladding.

Damage mapping was completed on these buildings post-quakes to identify the extent of damage and develop a high-level repair strategy. The buildings were temporarily secured and sealed off awaiting future repairs.

In 2022 Lincoln University approved the repairs and seismic strengthening of the damaged Ivey West and Memorial Hall. A contractor was appointed, repairs and seismic strengthening completed in November 2024. This paper outlines some of the construction stage challenges with repair and seismic strengthening of an earthquake damaged URM building.

EARTHQUAKE DAMAGE

The earthquake damage from the CES mostly attributed to the September 2010 event included:

- Collapse of Memorial Hall gable end walls.
- Collapse of the Memorial Hall kitchenette roof caused by the collapse of the gable wall above.
- Extensive and in cases wide cracking in URM walls.
- Collapse, partial collapse, or damage to a number of secondary structural elements, including chimneys, dormers, limestone capping stones, and balustrades.
- Extensive damage to internal lath and plaster wall and ceiling linings.

Figure 1: Memorial Hall north-east elevation before repairs and strengthening

Damage mapping was undertaken following each significant CES event supplemented with verticality surveys and laser scans to generate a point cloud for future reference in developing a structural drawing model.

SEISMIC ASSESSMENT

A Detailed Seismic Assessment (DSA) was completed to understand the building rating and scope of strengthening which would be completed at the same time as the earthquake repairs.

The assessment focused towards areas of the building that may be retained without significant alteration, such as in-plane loaded walls with only minor damage. Elements of the building that were majorly damaged, typically Secondary Structural Elements, were less rigorously assessed as some form of strengthening was clearly required. Assessments for these elements have therefore generally been qualitative. The building rating was less than 20%NBS (IL2) and the scores of selected elements were assessed as:

- Memorial Hall URM walls being an open plan structure scored 20%*NBS* (IL2) for URM walls in-plane and out-of-plane.

- Ivey West URM walls loaded out of plane scored 20*%NBS* (IL2) for out-of-plane loading above first floor and 60*%NBS* (IL2) for in-plane loads.
- Limestone capping stones scored <20*%NBS* (IL2) due to lack of any anchors.

EARTHQUAKE DAMAGE REPAIRS

The repairs and strengthening had to be considerate of the elements considered to be of heritage value. The facade in particular was of significant heritage value. Further the duration since the earthquakes meant additional deterioration had occurred by ingress of moisture through the damaged areas that could not be practically sealed off.

To assist with defining the façade repair scope, appreciating there would likely be hidden damage, a façade inspection was undertaken from the ground level and compared to post-earthquake damage observations. An indicative façade repair scope was marked up on the building elevations and ownership of this package of work was shared between the architect and engineer appreciating the façade at least for the Ivey West extent was the structural system.

Figure 2: Memorial Hall north-east elevation after repairs and strengthening

Damaged to masonry elements and the approach to repairs was agreed between the heritage architect, stone mason and engineer. This approach provided pragmatic solutions that maintained as much of the heritage fabric, took into consideration practical reconstruction consideration of the stone mason and engineering input on reinstating strength. Appropriate repairs were then scoped and developed.

CONSTRUCTION CHALLENGES

The post-quake damaged buildings meant that Health and Safety considerations limited the duration of and extent of access to the building. Areas of the building that were known to have suffered earthquake damage and posed an increased risk were clearly identified to the project team and contractor to allow for appropriate controls to be implemented till the damaged areas had received adequate strengthening.

Coupled with lack of original documentation of the buildings this meant there were various unknowns with the buildings that could not be appropriately confirmed while preparing strengthening designs. The intrusive investigations undertaken were limited to areas that posed the least risk and confirmed representative as-built details to progress strengthening schemes.

The proposed approach of working with a contractor to complete securing works and completing intrusive investigations prior to completing design was challenged as cost certainty was required by the client prior to giving approvals to proceed with the revitalisation. A strengthening design was prepared based on known information of the buildings and treating this as being representative of other areas that could not be safely accessed. Areas requiring further intrusive investigation to either confirm or complete design was outlined in the structural engineering documentation. Further details of the construction stage challenges follow.

INTRUSIVE INVESTIGATIONS AND UNEXPECTED FINDINGS

Ideally all the intrusive investigations would have been completed prior to completion of detailed design documentation to obtain all the required information. In this instance safety considerations with accessing earthquake damaged building and, likely presence of hazardous material limited the investigations to select areas to obtain representative samples of as-built information. Light touch intrusive investigations of the as-built construction were completed to provide strengthening designs that provided cost certainty for a large proportion of the works.

The remaining intrusive investigations were completed in stages based on the sequence of the contractor's securing work methodology. This resulted in full intrusive investigations findings arriving progressively through construction. The sequence of work was driven by the limited space to work with and temporary works installations.

The challenge of receiving intrusive investigation findings through construction had to be overcome by flexible resourcing of the engineering team to respond to site unknowns, designing appropriate solutions, obtaining builder inputs prior to formalising design.

FOUNDATION STRENGTHENING

The strengthening design for Memorial Hall and Ivey West required new foundations to support new reinforced masonry block walls. The existing Memorial Hall URM wall foundations could not be demolished in its entirety as it provided support to the outer brick wythe.

The strengthening scheme relied on the walls being unloaded off the existing foundations, the inner wythe of bricks carefully deconstructed before new foundations were constructed to support the new wall.

A round of light touch intrusive investigations were completed to confirm existing foundation dimensions by excavating outside the building and completing indicative foundation details noting the risk of unknown foundation profiles inside the building. Excavations inside the building were not possible due to asbestos contamination. As the decontamination progressed and foundations were exposed inside the building it became clear the existing foundations were not symmetrical, of inconsistent dimensions and depths, further compounded at the interface with Ivey West where old foundations were encountered. It became evident the existing foundations were more variable than allowed for.

A series of weekly workshops were held on site as excavations progressed to discuss the foundation solutions with the project team and client. These were challenging conversations and offered each party an opportunity to outline their preferences and for the project team to collectively brain-storm ideas. The time invested in these workshops resulted in solutions that had collective agreement from the project team and builder and allowed for construction to progress without unnecessary delays while keeping the client informed of these challenges.

REMOVAL OF URM LOAD BEARING WALLS

The heritage architect strongly preferred that Memorial Hall retain the exterior fabric of the building. A two brick inner wythe was originally provided as the load bearing wall for the roof and separate from the outer single brick wythe largely intended to act as the cladding. Even in an undamaged state the URM walls could not offer the required capacity to resist out of plane or in-plane loads to achieve an acceptable level of lateral load resistance. While the outer wythe was damaged a repair strategy was developed in conjunction with the stone mason to ensure the aesthetics of the building were largely unchanged.

Consequently, having understood the need to maintain the outer wythe of brick, the strengthening design progressed an option that relied on removal of the inner load bearing wythe and replacing it with a new reinforced concrete block wall. This approach provided a strengthening solution that maintained the inner clear space of Memorial Hall, allowed for a repair to be progressed from within Memorial Hall and did not require the removal of the outer wythe.

The removal of the load bearing inner wythe of URM walls posed temporary works challenges for the builder and required careful planning and sequencing to ensure work could progress in parallel on other aspects of Memorial Hall strengthening and repairs. The approach taken by the builder was to prop the roof and keep it in place through construction. The alternatives included bracing the outer brick wythe, removing the roof to relieve the load on the inner wythes before demolishing the load bearing walls. While keeping the roof in place offered a benefit of a shorter programme extensive planning was required to be able to work inside the propped space.

Figure 3: Temporary works Ivey West

Ivey West was originally constructed for student accommodation with rooms either side of a central passageway. This meant a lot of single and double skin URM walls were used as load bearing walls to support the first floor that the DSA scored low. The strengthening scheme allowed for removal of these internal URM walls to create larger rooms with new reinforced concrete block walls forming the new lateral load resisting structure. The temporary works installed to allow for removal of load bearing walls left the builder with tight spaces to implement the strengthening and construct new walls.

The project team had to work closely to ensure the temporary works engineer understood the structural system, the builder understood the limitations of the temporary works and the time at which these could be removed due to the permanent structural system being sufficiently in place to allow for removal of temporary works.

FLOOR DIAPHRAGM – IVEY WEST

Ivey West required a new floor diaphragm to tie the exterior URM walls and transfer forces to either existing URM or new reinforced block masonry walls.

The intent of the strengthening design appeared simple to implement at first and sensible in keeping the existing flooring by installing a new ply diaphragm on the underside of the first floor.

Figure 4: Ivey West diaphragm

Construction of new transverse direction reinforced concrete block walls in Ivey West meant sections of the existing flooring had to be replaced. The temporary propping also resulted in sections of the existing floor needing to be removed. Due to the lack of floor levelness additional timber furring and packing had to be installed to the underside of the existing floor joist. The result was significantly more existing flooring got removed than perhaps the structural design appreciated. While there was a chance to install a new diaphragm on top of the existing joists the connection of the diaphragm would have resulted in a reduced capacity installing connections to the thinner brick walls above first floor.

LEVELS AND VERTICALITY

The CES resulted in the walls of the building not being vertical and consistent with visual distress confirmed by point cloud survey of bulging and leaning. Around each of the dormers, bay windows and gables the walls were noted to be bulging or leaning outwards. Discussions with the heritage architect on the lack of verticality indicated a preference to limit demolishing and reconstructing existing walls unless required either structurally or based on the advice of a mason.

The URM walls were all tied back into the new reinforced concrete block walls, floor and roof diaphragms resolving the out of plane demands due to lack of verticality. Where the verticality was affecting installation of non-structural components or the resulting deformation had resulted in damage that could not be repaired in-situ the walls were deconstructed and reconstructed – not always plumb and vertical to allow to merge into the existing walls.

There were also instances where the scope of wall reconstruction increased as deconstruction progressed due to hidden damage. This was particularly the case for the west gable elevation of Memorial Hall that had to be entirely reconstructed. In this instance with the benefit of hindsight the wall deconstruction occurring first would have allowed for better access into Memorial Hall from the west end to undertake foundation construction. The foundation and

temporary securing works were completed from west to east and the subject of scrutiny due to perceived rate of progress.

FLEMISH GABLES, DECORATIVE ELEMENTS AND RECREATING FROM HISTORICAL PHOTOS

Figure 5: West elevation

Flemish gables with the sweeping arcs are a distinct feature of the gable ends of the buildings. All gables except for one on Ivey West collapsed during the earthquakes. To recreate the gables the architectural team relied on photos to recreate a similar geometry. The gable arcs were constructed in concrete to receive the wide limestone capping and finials. Concrete forms were created through exporting architectural gable elevations to a CNC machine to cut the appropriate profile.

Mock-ups were created for a dormer, concrete balusters and finials using historical photos to understand structural strengthening implications and aesthetics matched that of the undamaged pre-earthquake state prior to repeating for the remaining damaged items.

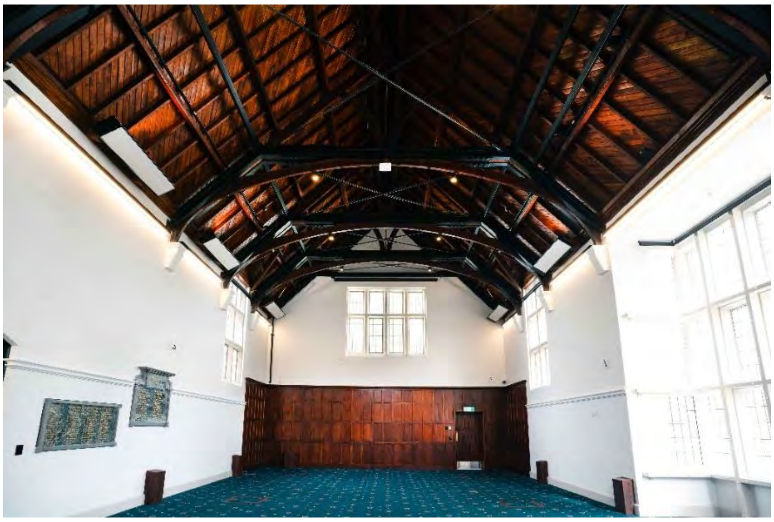

Figure 6: Memorial Hall post-repairs

CONCLUSIONS

In completing the seismic strengthening and repairs of Ivey West and Memorial Hall the construction stage outlined:

- The need to work closely with the builder from pre-selection to understand their works methodology and sequencing.
- Temporary works are not insignificant, and the appropriate scheme will require close collaboration between the temporary works engineer, the builder and the structural engineer.
- Foundation strengthening programme needs to be generous.
- Unexpected site findings are a given with historic buildings and the project team should have appropriate mechanisms in place to address the unexpected findings to avoid programme delays.
- Cost certainty can be provided for a large proportion of the works depending on available information backed by site verification of as-builts, however the contingency allocation needs to be appropriate to allow for the unexpected findings.
- Lack of verticality and levelness is to be expected and practical approaches to repair and strengthening can be applied instead of attempting to make the building vertical and level.

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Sustainability in Seismic Engineering

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ABSTRACT

The Climate emergency our world is facing is demanding a “paradigm shift” in design thinking in every branch of engineering; Structural engineering is not immune to this. Considerable efforts are already in place in this direction where minimizing upfront and embodied carbon is the primary focus. As most of these efforts are centered in the “non-seismic” part of the world, the methodologies developed are not directly applicable to “seismic” regions mainly because these methodologies do not explicitly acknowledge the significance of “seismic resilience”. In other words, solely focusing on minimization of embodied carbon without sufficient consideration of seismic resilience does not fulfil the goals of sustainability. In this paper, we outline a “smart design framework” called MOODD which explicitly considers the embodied carbon and seismic resilience as design parameters. The application of this framework is illustrated through some of the recent award-winning multi-story retrofits done in one of the highly seismic regions of the world, Wellington, New Zealand. It is shown that the application of MOODD results in high level of seismic performance with considerably less material (reductions in upfront embodied carbon) as compared to conventional retrofits.

Introduction

The conventional ductile design strategy relies on a structure absorbing seismic energy during a major earthquake by undergoing significant inelastic deformations. However, this inelastic deformation leads to substantial economic losses due to damage to both structural and non-structural elements. Past earthquakes have highlighted this issue, with the Mw 7.8 Kaikōura earthquake on 14th November 2016 in New Zealand serving as a recent example of the economic losses largely attributed to this design philosophy.

Although this earthquake resulted in only two fatalities, the damage to buildings and infrastructure was estimated to exceed NZ $15 billion. Similarly, the damage and business disruption from the 2010 and 2011 Canterbury earthquakes amounted to more than NZ $40 billion, which corresponds to approximately 20% of New Zealand’s Gross Domestic Product (Pampanin 2015). This figure does not account for the social disruption that could lead to significant migration and relocation of economic activities. Similar patterns have been observed in past events globally. For instance, the 1989 M 6.9 Loma Prieta earthquake caused over US $8 billion in direct damage (with several buildings and bridges suffering total or partial

collapse) despite no major loss of life occurring (Wada et al., 2004). Comparable observations were made following the 1995 Kobe earthquake (US $102.5 billion in damage, representing 2.5% of Japan's GDP at that time) and the 1999 Chi-Chi earthquake, which resulted in about US $10 billion worth of damage (Wada et al., 2004).

Four key factors drive the carbon discussion in New Zealand and the demand for sustainable design: Earthquake-prone buildings legislation, lessons from recent earthquakes, office use adaptations in our cities, and the push for sustainable practices.

Earthquake-Prone Building Legislation

The updated EPB framework, effective 1 July 2017, requires building owners to strengthen or demolish earthquake-prone buildings by specified deadlines based on seismic risk and priority classification.

Many buildings with EPB notices are often demolished due to economic considerations.

Lessons from Recent Earthquakes

Major earthquakes in New Zealand over the past 15 years revealed vulnerabilities in precast floor systems and non-structural elements. While code-designed buildings prevent loss of life, they often sustain severe damage, making repairs uneconomical. These events highlight the need for resilient retrofitting and sustainable design practices.

Path to Net Zero Carbon by 2050

New Zealand is advancing towards sustainable construction, with commitments to achieve carbon reduction targets by 2030 and net zero by 2050. Since the majority of today’s buildings will still be in use in 2050, retrofitting and reusing existing structures play a critical role in decarbonising the built environment.

Rethink of Buildings' Use

COVID-19 has shifted the demand for office spaces in town, encouraging a rethink of existing buildings' use. Refurbishment and retrofitting provide opportunities to repurpose and extend building lifespans sustainably.

Figure 1: Key factors driving NZ’s carbon discussions

In seismic regions, minimizing embodied carbon while ensuring seismic resilience presents a major challenge. Conventional ductile-design strategies often lead to significant damage and rebuilding after earthquakes, as seen in events like the Christchurch earthquake, where ~70% of the buildings in the Central Business District had to be demolished. To retain existing buildings in such high seismic areas, a completely different approach is required. Retrofitting existing structures with solutions like viscous dampers is yielding cost-effective retrofit solutions and significantly enhanced seismic resilience.

The study from Brown and Milke 2016 concluded that following the Canterbury earthquakes 2011, it was estimated that approximately 4 million tonnes of construction and demolition waste would be generated, equating to roughly 18 years of waste normally sent to landfill from the city. The demolition decision process following the impacted by Christchurch earthquakes was well summarized by Marquis et al 2015

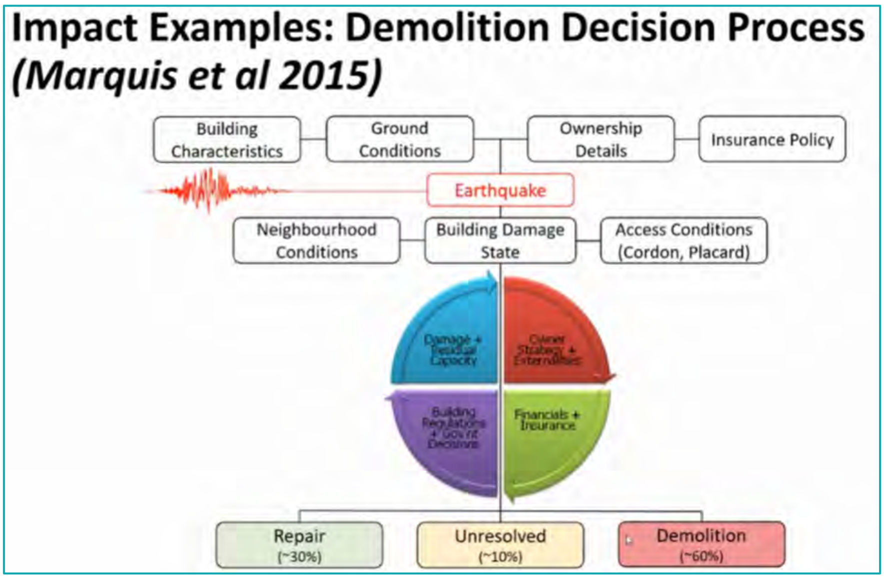


Figure 2: Demolition decision process following earthquake [Marquis et al 2015]

Viscous Dampers for Seismic Resilience

One effective way to reduce earthquake-induced damage and ensure life safety is by incorporating technologies such as viscous dampers into structures. Viscous dampers are highly effective in mitigating seismic responses, primarily because the damper force is linearly or nonlinearly proportional to velocity and operates out of phase with displacements, thereby reducing peak structural forces. Consequently, columns or foundations are not subjected to additional demand and may not require strengthening (Constantinou et al., 1993).

Viscous dampers effectively reduce earthquake impacts by minimising drift and floor acceleration, enhancing structural resilience and reducing downtime. For retrofits, they are more cost-efficient than base isolation systems, requiring less foundation and superstructure modification. However, their effectiveness depends on proper design.

MOODD Platform for Advanced Design Solutions

The proprietary MOODD platform addresses challenges in Performance-Based Design (PBSD), optimising dynamic interactions to improve seismic resilience while reducing carbon emissions. It also supports Performance-Based Wind Design (PBWD) for tall buildings. Two real-world applications of MOODD in seismic retrofits are highlighted.

8 Willis Street, Wellington, New Zealand

8 Willis Street is in the Central Business District of Wellington. The city rests on the meeting point of two tectonic plates and is prone to large earthquakes. Major fault lines very close to the city include the Wellington Fault, the Wairarapa Fault and the Ohariu Fault. The Wellington fault may be able to generate earthquakes larger than magnitude 8.0 and may generate ground velocities around or larger than 1.5 m/s and ground displacements greater than 2m. Obviously a retrofit design in Wellington should cater to these sorts of extreme seismic demands.

Figure 3: 1980's structure

Figure 4: 2022 retrofitted adopted reuse

The seismic rating of the existing structure built in the 1980s (Fig. 3) had fallen below what is acceptable for a commercial office space. To improve the commercial viability and future-proof of the existing asset, property developer Argosy, headquartered in Auckland, New Zealand, decided to retrofit the building to be 130% New Building Standard (NBS) along with increasing the floor area from 6500 m^2 to 11,750 m^2.

The primary structural system of the original building was comprised of Reinforced Concrete moment frames in one direction and shear walls in the other direction. The floor system was precast hollow-core concrete floors, and the foundation system was shallow pads.

Performance based seismic design using the smart design platform was applied for the retrofit design incorporating fluid viscous dampers. The primary retrofit scheme includes twelve viscous dampers arranged in the moment frame direction. The fluid viscous dampers are tuned in such a way that the overall building performance met the target requirement of the client with no additional foundation work for seismic loading. Seventy ground motions scaled to NZ 1170.5:2004 were used for the design. Aleatoric and epistemic uncertainties were explicitly accounted for in the design.

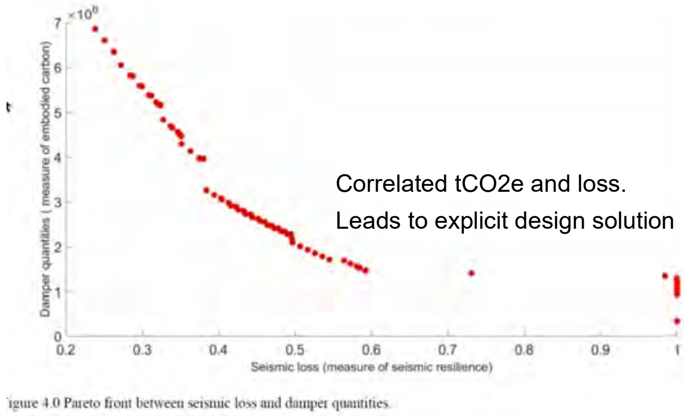


Figure 5: Pareto - Seismic loss and damping – 8 Willis Street

The final retrofit had more floors without dampers than with dampers and met the Architectural requirements of having uninhibited floor plates for maximum functionality. The adaptive reuse of the structure alone resulted in saving a whopping 1904 tons of carbon.

33 Bowen Street, Wellington

33 Bowen is a 12-story reinforced concrete moment-frame building designed and constructed in the 1980s, incorporating precast hollow-core floors. Beca's seismic assessment rated the building at 35*%NBS*, just above “earthquake prone” status, with the low rating due to hollow-core diaphragm issues and the frames nearing code drift limits. Coupled with shear failure concerns from beam overstrength not accounting for slab reinforcement, the deficiencies identified led to tenants vacating. The situation was further exacerbated by new GNS Science data indicating a 90% hazard increase for the site. The building owner decided to future proof

he building to 80% of new seismic hazard, a targeted strengthening to 150%*NBS* based on current hazard.

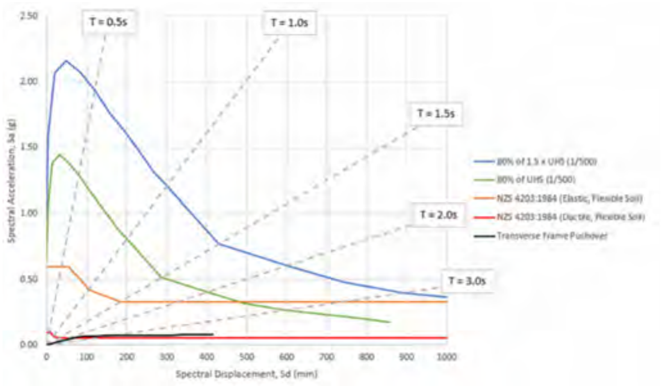
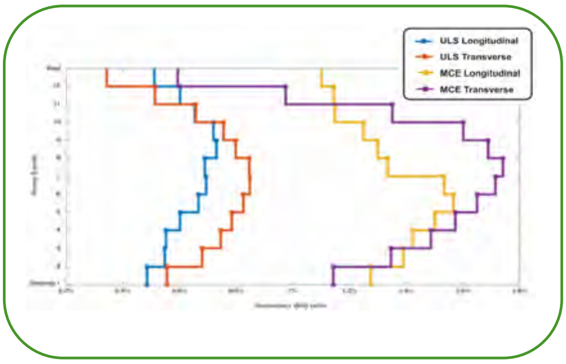


Figure 6: ADRS curve for 33 Bowen Street *Figure 7: 3D of 33 Bowen*

To achieve 150%NBS, a four-fold increase in seismic resistance was required, making conventional strengthening methods impractical due to high costs, carbon impact, and negative effects on functionality and aesthetics. A smart retrofit strategy using advanced PBSD principles was implemented, significantly reducing intervention while achieving cost-effective and sustainable outcomes.

The drift targets were set at less than 1% for DBE and under 2% for MCE—below NZ code of 2.5% for ULS. The final solution has inter-story drift to 0.8% (DBE) and <1.8% (MCE), addressing hollow-core issues effectively. Viscous dampers were strategically placed in perimeter frames to minimise disruption, eliminating the need for foundation work by reducing base shear. This approach delivered a sustainable solution with minimal carbon footprint compared to traditional methods.



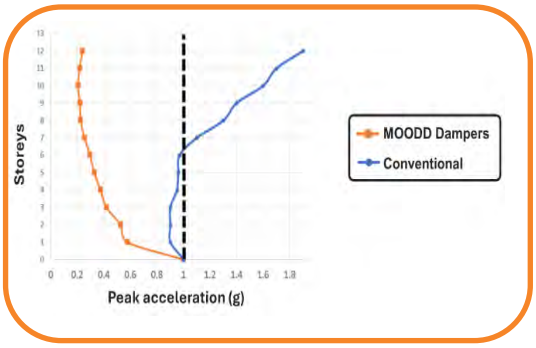


Figure 8: Acceleration Inter-story drift *Figure 9:: Floor acceleration*

Conclusions

The article clearly highlights the effectiveness of combining viscous dampers with advanced structural design technologies, such as the smart design platform, to identify optimized retrofit solutions. This innovative approach has proven highly successful in enabling cost-efficient seismic retrofitting of buildings with significant vulnerabilities (drift and acceleration related) in high seismic regions. The resultant retrofit solutions achieved a substantial reduction in total cost and embodied carbon compared to that possible using conventional analysis and design techniques and obviously even greater savings when compared to the alternative of the demolition and reconstruction of a replacement building.

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The challenges of designing an unconventionally shaped mass timber structure

H. Strange, J. Whitehead

Dunning Thornton, Wellington.

ABSTRACT

The new global headquarters for Fisher & Paykel Appliances merges complex site conditions with an unconventional architectural form. The almost complete use of mass timber aims to achieves a net-zero embodied carbon and has produced a structure that is eye-catching and truly unique. The timber diagrid wraps the entire curved perimeter of the 260-metre-long three-storey ribbon-shaped building.

The structural design needed to be deceptively simple, to achieve the challenges of the geometry. An innovative ‘product design’ approach with an overarching modular philosophy has been key to achieving this, with careful consideration of prefabrication and buildability leading to seamless and successful installation.

This paper focuses some of the more challenging highlights of the structural design; from the foundations suspending the building over the site’s overland flow paths, to designing in mass timber in New Zealand’s seismic setting which included full scale prototype testing of the ductile diagrid connections. Finally, it offers a summary of lessons learned that could be considered for future projects.

INTRODUCTION

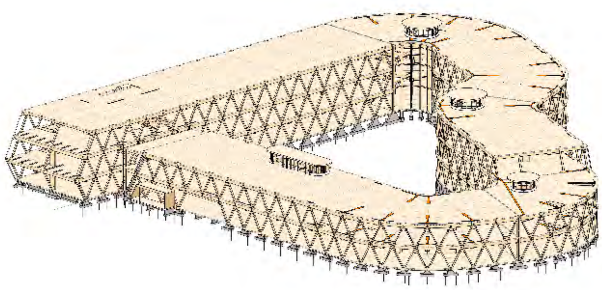

Figure 1: Structural Model of Fisher & Paykel Home

The new Fisher & Paykel Appliances headquarters, located in Auckland, New Zealand, prioritises net-zero carbon and architectural form. The “Home” mass timber building is one of three new buildings currently being constructed at the new campus. The structure is a 260m-long ribbon shape that is two and three stories high, with a floor area of over 12,000m², providing office space for nearly 1,000 staff and serving as the central hub that brings the company together. The structure is almost completely composed of mass timber, and the use of steel is minimal.

BREIF & SITE CONDITIONS

The project aims to capture Fisher & Paykel's main values: curiosity, innovation, sustainability, trust, and generosity. This brief has resulted in a low-carbon, architecturally ambitious, and inventive design. The structure pushes the boundaries of typical mass timber construction, from its unusual shape, the almost complete use of timber and screws, and its innovative connections. The project would not have been achievable without early collaboration from the entire project team, setting out key parameters for each consultant to consider and work with throughout the project.

The site is located at 830 Great South Road, Penrose, with its boundary along State Highway 1, forming the natural edge of the Auckland Volcanic Fields. Basalt varies in quality, level, and has volcanic voids as well as non-uniform flow paths of lava. The Home is situated around the most significant area of depreciation, which has been identified as prone to flooding, requiring the surrounding area to allow for overland water flow (refer Figure 2).

Reduction of carbon was a major constraint in achieving the project's strict sustainability goals. The project employed the following key strategies to meet this target:

- Use of mass timber where possible, limiting steel as much as possible
- Integration of a significant number of solar panels
- Natural ventilation for both reducing services and electrical demand (operational carbon)
- Localised foundation plinths cantilevering out of the basalt to elevating the building above flood level
- Use of low-carbon concrete wherever possible

Figure 2: Structure and overland flooding path

FOUNDATIONS

The overland flow path requirement lends itself to an elevated ground floor. The added complexity of the hard basaltic ground conditions meant mass excavation was costly and would not achieve the goal of reducing the embodied carbon of the project. As a result, discrete foundations were chosen to support the ground floor, which consists of a 166mm-thick Cross Laminated Timber (CLT) slab sitting between 900mm and 2,300mm above the finished ground level at its lowest and highest points, respectively. There are three main foundation types in this project: diagrid plinths, moment frame plinths, and blockwork walls.

All foundations provide gravity support for the ground floor, with plinths below the diagrid and moment frames also resisting lateral load demands. Minimising plinth sizes both reduced embodied carbon and also allowed for underground services to be direct fixed to the CLT with clear pathways for services to run under the building. This limits onsite coring or setting out penetrations. 1400mm wide gaps were provided at four locations for services (3 left**Error! Reference source not found.**). All precast elements are comprised of low carbon concrete with a target 40MPa 58-day strength.

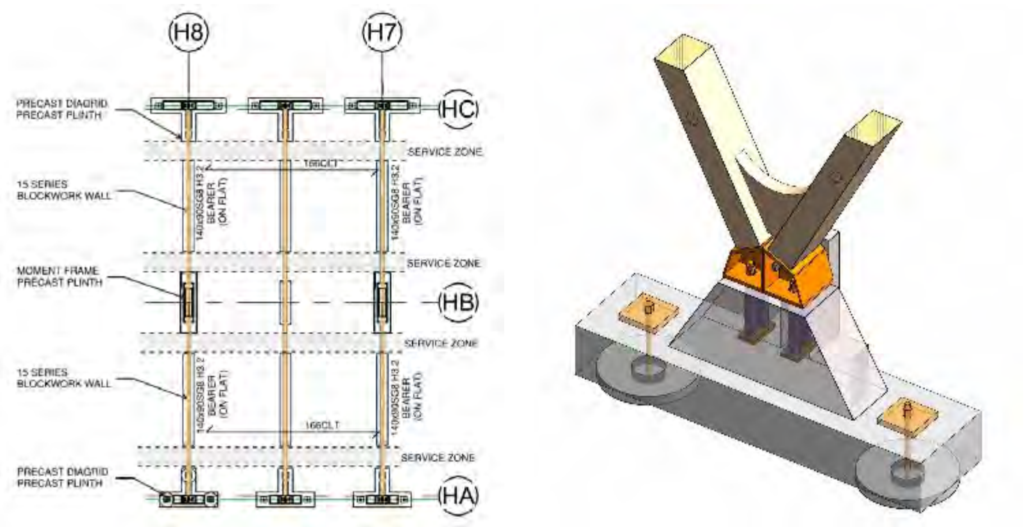

Figure 3 (Left) Typical Foundation bay, (Right) Diagrid Plinth

Architecturally sympathetic, the diagrid plinths, are triangular to follow the natural slope of the diagrid above (Figure 3, right**Error! Reference source not found.**). These plinths are the most heavily loaded foundation type. The high tension and compression demands from the diagrid are transferred into the ground via ground anchors with the clamping force providing a shear friction interface. This was in part driven by the ground conditions being fractured and solid basalt. The foundations were designed using concrete strut and tie models due to the unusual form. The back leg of the plinth is to support the 140x90SG8 bearers, provide stabilisation to the diagrid and transfer base shear from the ground floor inertia into the ground.

Internally, the moment frames sit on a large square precast plinth which are placed inside a secondary socket plinth. The plinths have been designed for the overstrength of the moment frame above, but rocking of the plinth in the socket has been allowed beyond this. The ground floor diaphragm, although vertically supported by the block walls, is braced by the diagrid plinths as these are the stiffest load path as the block walls are founded on fill.

SUPERSTRUCTURE

The superstructure consists of two perimeter diagrid lines that wrap around the structure, both on the exterior and interior of the ribbon-shaped form. The building is separated into four buildings and are seismically separated as shown in Figure 5. The diagrid consists of 295x180 Laminated Veneer Lumber (LVL13), braces at a 2:1 angle on a 4-metre module (Figure 4, left). The diagrid's horizontal element is a 590x218 LVL13 perimeter beam, which provides gravity support and transfers tie forces. The diagrids are 16 metres apart, with a central 1180x295 moment-resisting frame column. The column supports two 590x236 LVL13 primary beams, which in turn support 590x177 LVL13 secondary beams at 3.2 metre centres. The secondary and primary beams support 210mm CLT for the office floor and 166mm CLT at the roof (Figure 4, right)

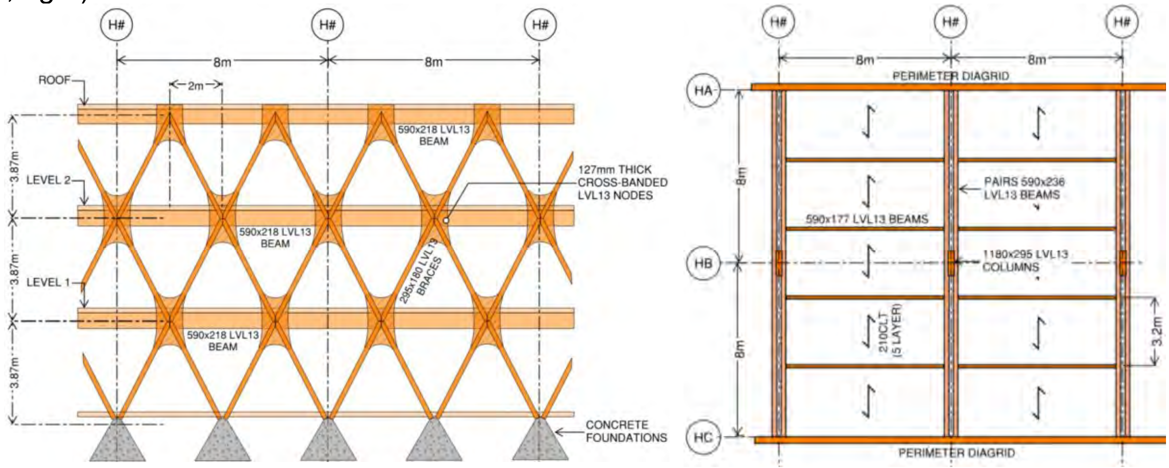

Figure 4: (Left) Diagrid, (Right) Typical superstructure bay

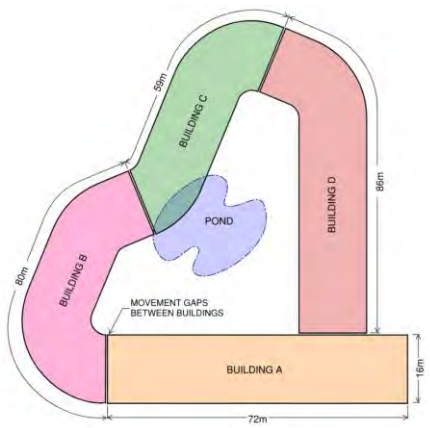

Figure 5: Seismically seperated buildings forming the foot print of 'Home'

One of the most unique things about the structure, is the lack of visible fixings in the structure. Most screws being used are hidden with timber dowels both for architectural reasons but also for fire protection. The building's fire rating is 40 minutes which was derived by the fire engineer, Crossfire, using the Brandon method. The combustible load is reduced in the structure by a magnesium board raised floor which encapsulates the floor and the use of localised fire rated plasterboard direct-fixed to the CLT and as part of some ceilings.

Gravity Structure

At concept design, the service integration was optimised by setting the primary beams 300mm below the underside of the CLT. This allows for clear service paths between bays, functioning similarly to the service pathways within the foundations, and minimises holes through the LVL beams. Additionally, this configuration enables the secondary beams to be notched and directly landed on top of the primary beam, transferring load via bearing (Figure 6 left). On-site, the only installation required is the Rothoblaas HBS8x400 restraint screws, while all other reinforcing screws (for the notch) can be installed off-site. All notch connections were designed using the guidance noted in Chapter 12.6 of the NZ Wood Guide (Daniel Moroder & Tobias Smith, 2020). Design of the screws were to the Rothoblaas ETA which considers k_{β} which reduces the withdrawal capacity of screws parallel to glue lines in the LVL (ETA-Danmark, 2022).

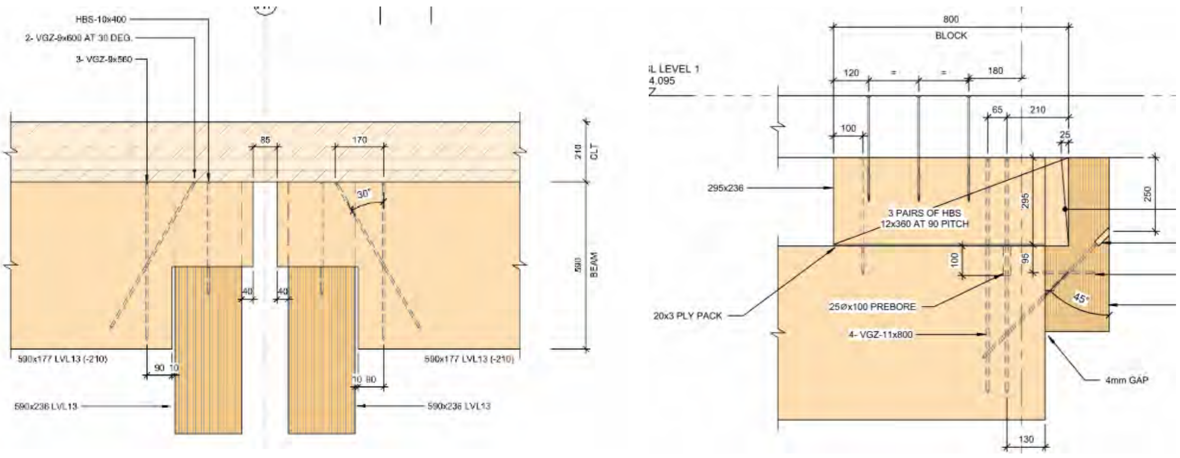

Figure 6: (Left) Typical Secondary beam, (Right) Typical primary beam connection

At the perimeter diagrid beam, the primary beams sit 300mm below the top of the LVL. Initially, this complicated the detailing, however, after drawing inspiration from double tee Cazaly hangers, a solution was developed: notching the perimeter beam and installing a "hanger block" off-site above the primary beam. This solution ensured that the primary beams could be easily positioned onsite without requiring additional screwing to achieve gravity support (refer Figure 6 right). The hanger block is oriented with its wide face facing upwards — an intentional

design choice to utilise the favourable load sharing between layers therefore removing the k_β factor outlined in the Rothoblaas ETA.

Lateral resisting system

The main lateral resisting element of the structure is the diagrid. There are two diagrid lines, one on the interior and one on the exterior of the building. The timber diagrid achieves its required ductility with specialist Rothoblaas fully threaded screws, which act as the "potential ductile elements" (PDE) for the structure which is nominally ductile. All other elements have been designed to be capacity protected (CPE). The diagrid consists of four 180x295 LVL braces, which are knifed over a 127mm-thick cross-banded LVL node and screwed with VGZ9x220 screws in various quantities between 8 to 24 screws. The cross-banded node also provides gravity bearing support for the perimeter 590x218 LVL beams. The perimeter beams are also CNC-machined to allow them to run past and be face fixed into the cross-banded node with VGZ11x150 fully-threaded screws. This configuration allows the shear loads from the diaphragm to be transferred into the node (refer Figure 7).

The moment frames are 1180x295 LVL13 or 790x295 LVL13 central columns with 590x236 LVL13 beams each side. These are screwed together with a rectangular pattern of VGZ11x550 fully-threaded screws. The moment frames provide lateral load resistance primarily in the straight sections of the structure. At the curves, the moment frames provide little lateral resistance due to the highly stiff perimeter diagrid so these locations have smaller columns.

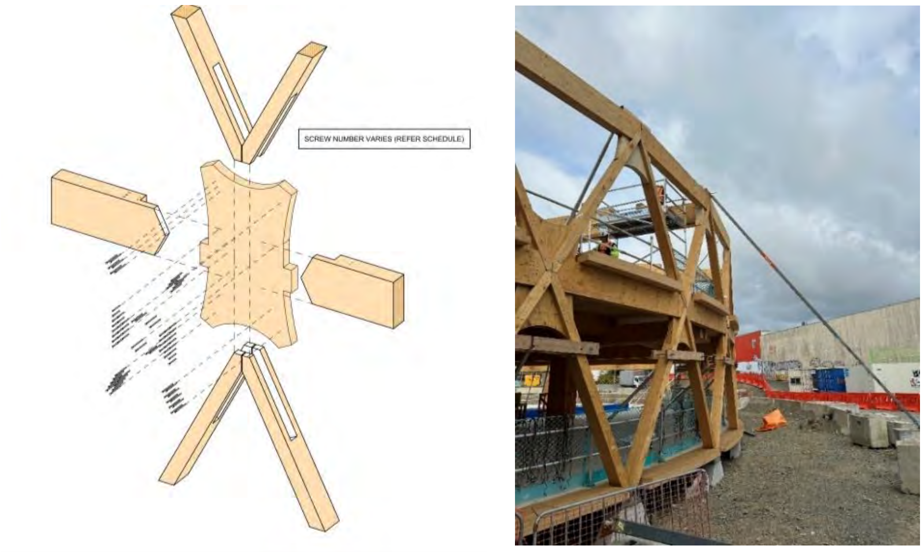

Figure 7: (Left) 3D Diagrid Model, (Right) In-Situ Install at curve

Again, repeatability was a key consideration for the application of the diagrid around its meandering footprint. Broadly speaking, there are only three unique diagrid arrangements for the entire structure: at the straights, at the bends, and at the entry atria. It should be noted that, of the three different geometrical arrangements, there are multiple variations for example whether the primary beam is present. However, the geometry and the rules associated with the diagrid arrangement remain consistent across all scenarios. To achieve success in installation the nodes at the curves were detailed with a flat vertical surface with the braces CNC'd to match the curve and shape. The vertically true node allows for diagrid "diamonds" and "M" to be pre-fabrication and easily slotted into place on site.

NZS AS 1720.1:2022: Timber Structures (Standards New Zealand, 2022) was recently released at the start of the initial concept phase of the project. It was therefore elected that the building would be designed to NZS AS 1720.1:2022 with some cross checking against Eurocode 5, Design of Timber Structures (CEN, 2004). The structure was analysed using ETABS using non-linear static analysis. The complex curved buildings were analysed with a flexible shell diaphragm as opposed to a rigid diaphragm to account for the screw slip and reduced stiffness of a CLT diaphragm, so that forces in the curved portions of the diagrid were not over-estimated.

Full scale proof testing of the diagrid node was required due its complex geometry, the use of NZS AS 1720.1:2022 (brittle failure modes), the structural shape, and the innovative use of the materials. This secondly, provided an opportunity to verify some of the main buildability questions. Two full test was carried out by BRANZ New Zealand at their testing facility located in Wellington (Figure 8 left). While successful, the first test provided insight into an unforeseen failure mechanism - splitting in the brace along the screw rows. The second test was reinforced perpendicular-to-grain and had no splitting, with the connections performing to the predicted overstrength of 1.6 times the nominal capacity of 235kN. It was found that the screw stiffness was generally a lot lower than noted in Eurocode 5 and NZS AS1720.1:2022 (Figure 8 right). We note that this did not impact the design for this project as the structures had a low period and low drift so an increase in period and displacement was favourable. Further learnings from the tests is being published to the Timber Design Society journal.

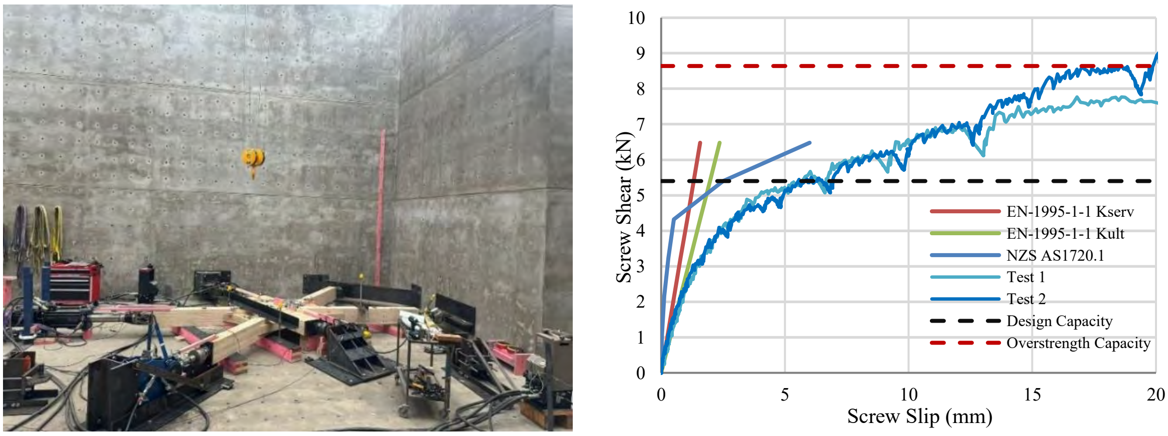

Figure 8: (Left) Test set-up, (Right) Test results

CONSTRUCTION

On-site installation was considered early in the design process with the development of an erection methodology: erecting the building to full height and working around the floorplate to minimise the exposure to wetting. This methodology became part of the building specification, communicating to the Naylor Love and mass timber specialist installer Kobe Construction, on how the building was intended to be constructed. Once this methodology was established, it was ensured throughout the design phase that the design aligned with the proposed sequence. Furthermore, there was an expectation that most elements could be prefabricated in a controlled environment and delivered to site as needed. In practice, this approach has proven successful during the construction phase, with both beam and diagrid elements arriving onsite with most screws pre-installed, requiring only a small number to be installed in-situ.

Figure 9: (Left) Diagrid prefabrication, (Right) Diagrid loaded for delivery

Successful mass timber projects require effective water management during the construction phase. It was essential that both Naylor Love and ourselves worked together collaboratively to address concerns associated with moisture management. Early in the construction process, Naylor Love and our team attended multiple pre-start moisture management meetings to outline preventive measures. In general, moisture management is difficult to correct once it becomes a significant issue on site. The corrective measures, when the problem worsens, are labour and electricity intensive. There were three key elements that we believe worked well in reducing these issues on site, which are noted as follows:

- The building design was intended to be constructed horizontally along the ribbon shape, rather than vertically upwards. This allowed the façade to follow the building, effectively ensuring the tail of the “ribbon” remains dry (refer Figure 10 left).

- Early on, Naylor Love invested in having pre-installed membranes on the roof CLT and applied an in-situ breathable Rothoblass Monolithic membrane on the trafficable floors. These membranes were beneficial as they limited water exposure to the CLT while also preventing water from penetrating between floors (refer Figure 10 right).
- Naylor Love installed temporary tarps during the intermittent stages between erection and façade installation. The tarps were fixed loosely, allowing airflow internally, which further aided in drying and significantly reduced water ingress from the horizontal direction. Furthermore, this had the added benefit of limiting UV bleaching of the timber, which can be time-consuming to remediate.

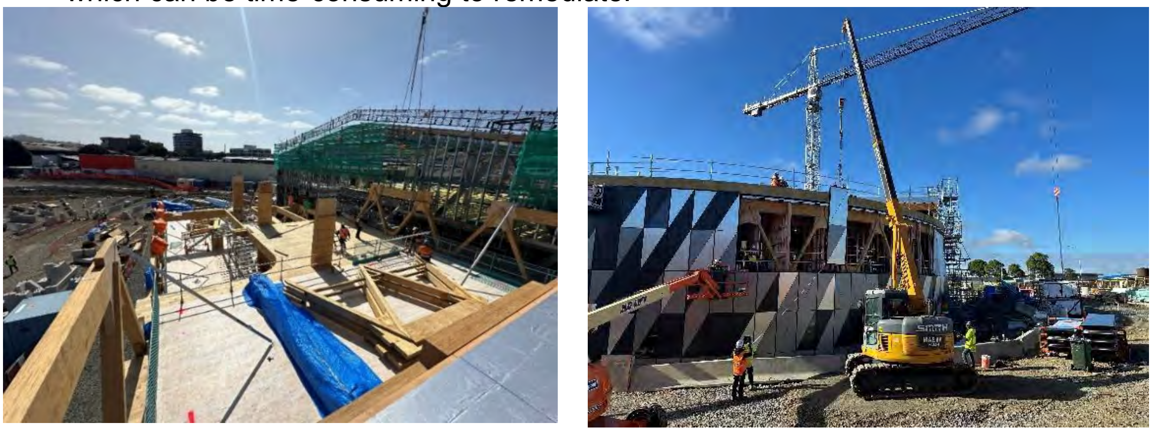

Figure 10: (Left) Lineal building constructon and installed membranes, (Right) Facade following behind structural erection

LESSONS LEARNT

A building of this scale and shape has pushed the limits of mass timber construction. No project is perfect, and there are many lessons to be learned from this project that could be applied to future developments. A summary of some of the main lessons is presented below:

Figure 11: Unexpected voids in basalt

- The method by which we drafted the structure was carefully considered well before lines were placed on paper. Communicating this knowledge during the shop drawing phase could have been incredibly beneficial in speeding up the associated work by the LVL supplier. The shape of the structure inherently made shop drawing difficult resulting in a process that was longer than initially anticipated. More face-to-face and one-on-one collaboration with the shop drawing team would have greatly broadened the understanding of how the building works, resolve detailing issues and how to effectively draw it.

- In general, keeping screw lengths below 800mm will benefit those installing the screws onsite. Drill bits of these lengths are not typically available and had to be custom-ordered. Furthermore, due to the nature of LVL, the drill bits need to be replaced frequently as they overheat. The limited supply and frequent replacement can lead to stock issue and delays on site.
- Basalt varies significantly, and the team did not anticipate the extent of this until work began onsite. Large voids and surface irregularity led to some creative solutions such as bridging voids that appeared from detail excavation. Pro-active and quick turnaround times was critical to not delaying things on site.

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Seismic Carbon Risk in Context

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ABSTRACT

With no defined timeframe for regulation on embodied carbon for the built environment, the shift to integrating carbon into our design process and advocating for low carbon structures as the new norm will have to be sector-led to achieve the net zero target by 2050. Whether looking at existing buildings or new design, the current market values seismic resilience over low carbon. This is guided in part by the structural engineering industry and the wider property sector's focus on addressing the seismic resilience of our buildings following the Christchurch earthquake sequence.

The relationship between embodied carbon and seismic resilience will be explored across both upfront and lifecycle carbon. Does an increased initial carbon investment make sense for better seismic performance? How might this balance shift across different seismic zones? While these two factors are often seen as competing, early and ongoing consideration throughout design can enable both priorities to work together. An argument is made for considering seismic risk in the context of the climate crisis.

INTRODUCTION

As engineers we are at home with critically evaluating design options, making tradeoffs, weighing each design decision against a spectrum of needs.

But are we correctly accounting for the weight of each of those needs? Are we giving enough weight to the very real and pressing need to drive construction emissions to zero and minimise the impact of climate change, or is it time for carbon to step out from the shadows and take its place alongside resilience as one of our primary responsibilities.

Improved seismic resilience, a drive for above code performance, has been the mindset for sector-leading structural engineers for the past decade. Through the lens of sustainability, the question of how we are achieving this resilience is asked, and if we are applying the goal of 'better than code minimum' to the right features? Structural failures as a result of the Christchurch and Kaikoura earthquakes were largely contained to historic construction forms (URM), isolated instances of poor practice, and specific detailing issues in modern buildings, rather than systemic underperformance (J. J. Kim et al., 2017; Marquis et al., 2017). The huge volume of post-event demolitions were predominantly driven by ground conditions and our insurance landscape—not by structural failure (Drayton & Verdon, 2013). Figure 1 from Kim (2015) shows the demolition data from Christchurch against damage ratio, with more than half of the buildings having a damage ratio less than 10%. Gonzalez et al. (2021) quantified the carbon emissions from demolition just of the reinforced concrete buildings, again finding that over 50% of the carbon emissions could be attributable to buildings with less than 10% damage. The images showing the patchwork city centre left behind following the Canterbury earthquakes spurred the call for resilient construction, but in driving to improve resilience have we reached for the hammer (increasing design loads, base isolation, more material) instead of applying the key techniques that we know give better certainty in performance - regularity, capacity design, ductile detailing, direct load paths etc.

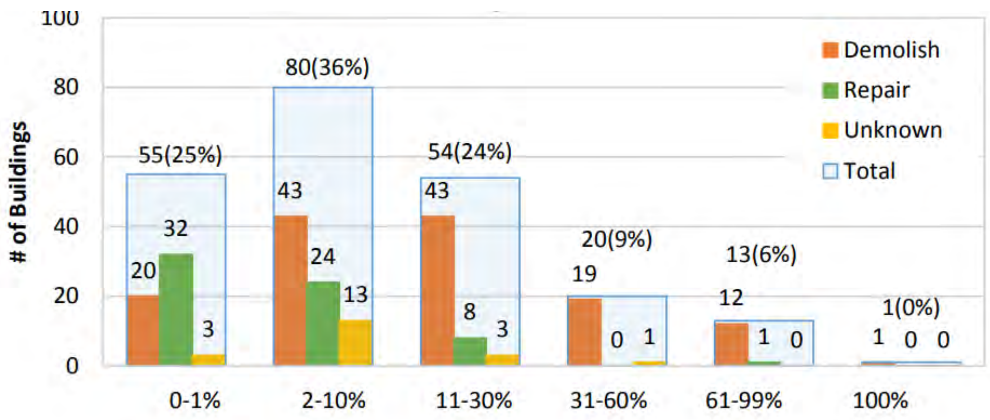

Figure 1 Demolition data by damage ratio (figure from (J. H. Kim, 2015)

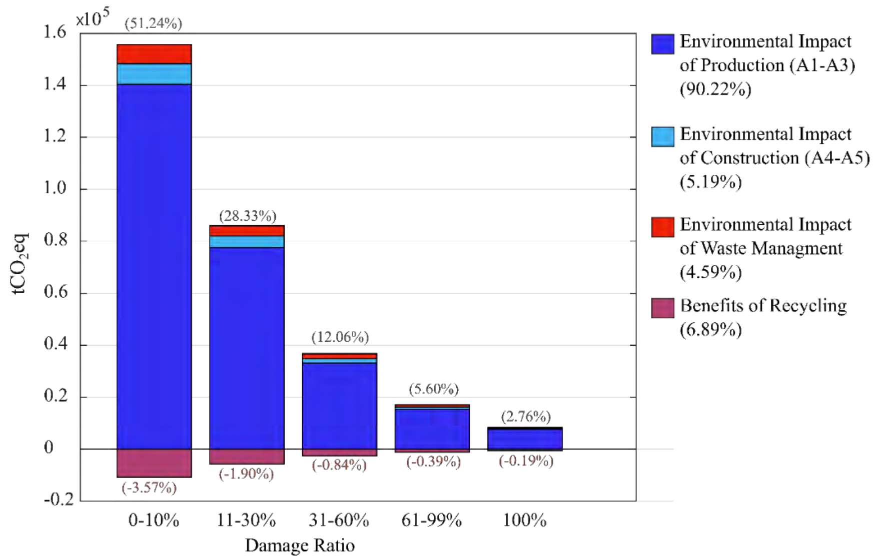

Figure 2 GWP Environmental impact from the demolition of reinforced concrete buildings in Christchurch CBD following the 2011 earthquakes (Gonzalez et al., 2021)

Contrastingly, in the face of overwhelming scientific evidence on the existence, cause, and physical impacts of anthropogenic climate change, behavioural change from our structural engineering industry and the wider building industry, to address our emissions contribution has been slow. Whilst we are taught that as engineers we are uniquely positioned to have a positive impact on the world around us, and address the most challenging problems of the day, there has been debate on whether addressing climate change is part of the core role of structural engineers. We have seen a pushback on how to integrate carbon thinking into the design process when it is not asked for or valued by the client and wider design team.

The general approach to improved seismic performance following the Christchurch earthquakes has been to 'raise the bar' through industry-led action. Can we apply a similar approach to a collective position on sustainable structural design?

The chicken or the egg? Government regulation versus industry-led change

New Zealand's second emissions reduction plan (ERP) (Ministry for the Environment, 2025) recently released, sets a target of 51-55% below 2005's gross emissions for 2030. This target has been called out as using creative accounting to under commit - a target far from what was expected internationally, and is far from being considered our 'highest possible ambition' as required by the Paris Agreement. We can see in the latest ERP that the direction of this government is to lean on industry-led initiatives, stepping back from regulation in the Building and Construction sector.

The strength of position and support from industry for transforming the way we design is more unified. Engineering New Zealand's Climate Change Position Statement, the joint letter to the Minister by New Zealand's largest property, construction and business organisations urging the government to commit to the Building For Climate Change Programme, and SESOC's own Sustainable Design Position Statement all set a clear tone of action. If the government is quietly shedding the mantle of responsibility, the opportunity is there for engineers to pick it up and show the necessary leadership.

A BALANCED FUTURE: SEISMIC RESILIENCE AND CARBON REDUCTION

The current market priority of seismic resilience is likely to shift to a balance of 'seismic resilience+low-carbon' as the Canterbury earthquake sequence fades in our collective memory while climate threat, market demand, and eventual regulation prioritises low-carbon. What is clear is that we will not achieve community resilience or sustainability by focusing efforts on a few 'best in class' structures.

The discussion below focuses on the key questions we need to ask when considering carbon expenditure and seismic risk in a combined context— and starts to think about how this might change our approach to design and the way we evaluate overall risk.

What takes precedence – seismic resilience or upfront carbon expenditure?

To date, building resilience decisions have typically been informed by perceived seismic risk and cost. However, in today's market, we also need to overlay a sustainability lens—considering how carbon expenditure is balanced alongside seismic resilience in the context of building use and geographical location.

As a society, we aren't always good at accurately assessing and ranking risks comparatively. Our decisions rarely reflect an equal risk assessment—we tend to react to dramatic, visible dangers while underestimating chronic or distant threats and this is further impacted by our inbuilt bias as individuals. As structural engineers working in a seismic region, we spend our days thinking and talking about earthquakes and we are specifically trained to design with them in mind. We can easily become skewed towards perceiving earthquakes as a common occurrence. In contrast, climate change presents a different challenge—we contribute to the problem through building-related emissions yet lack expertise in protecting against its consequences. We instead need to address the root cause by reducing the emissions driving these climate events.

Comparative statistical assessment shows that our overall risk of death or personal injury from extreme climate events in Aotearoa is significantly higher than that posed by seismically deficient buildings. A new building designed to the current standard will have a 1 in 1,000,000 annual fatality risk due to earthquakes, an earthquake prone building with a score close to 34%NBS increases that risk to between 1 in 40,000 to 1 in 100,000. Putting both those in another context, the annual fatality risk from flying in an aeroplane is 1 in 700,000, having a fatal car accident in New Zealand is 1 in 15,000. Climate change is resulting in an ever increasing risk to human health, both directly through flooding, extreme heat, wildfires, and drought, and indirectly through air pollution, water quality and food availability and ecological changes (Ministry for the Environment, 2020; Swiss RE Institute, 2023).

For normal buildings in locations with low seismicity shouldn't we logically be placing our primary focus on this larger, and ever-increasing, climate-related risk? Recognising that this can exist alongside achieving code-compliant performance with good detailing. Perhaps it's time to step back and consider a more holistic approach that better reflects each region's unique hazard profile in the context of the other environmental and societal challenges. This could enable us to focus our design efforts - and carbon investment - where they'll have the most impact in response to pressing hazards.

Does seismic performance need to come at a carbon cost?

Embodied carbon is intrinsically linked to the quantity of materials used for a building, and therefore has a relationship with the structural design philosophy and required performance objectives. There is a perceived tension between seismic resilience and low carbon design but the two don't need to be mutually exclusive. Building damage during seismic events is commonly linked to structural drift, with economic losses often stemming from damage to non-structural components such as building services and fit out (Gonzalez et al., 2023). However, the solution shouldn't be simply adding more material. Rather than defaulting to material-intensive solutions, effective seismic design requires consideration of the complete structural system. Strategic choices, such as selecting braced frames over moment resisting frames early in design, can achieve both drift reduction and material efficiency.

The figures below present results from a recent study of a multi-unit residential building in Auckland. Two preliminary design structural solutions were analysed: a moment resisting frame (MRF) and an eccentrically braced frame (EBF) system. The EBF solution offers reduced A1-A3 embodied carbon within the structural components and lower monetary costs for a comparable (in fact slightly lower) interstorey drift performance than the moment resisting frame alternative.

This combination of advantages led to the EBF being selected for implementation, illustrating how carbon reduction, structural performance and cost efficiency can successfully align in practice.

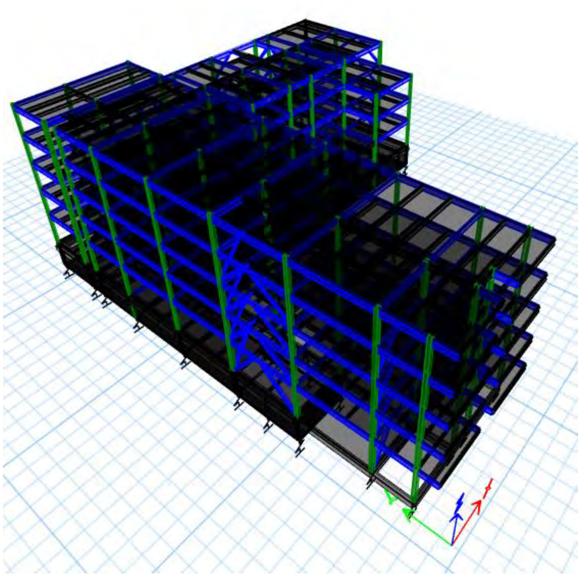

Figure 3: ETABS model - EBF system

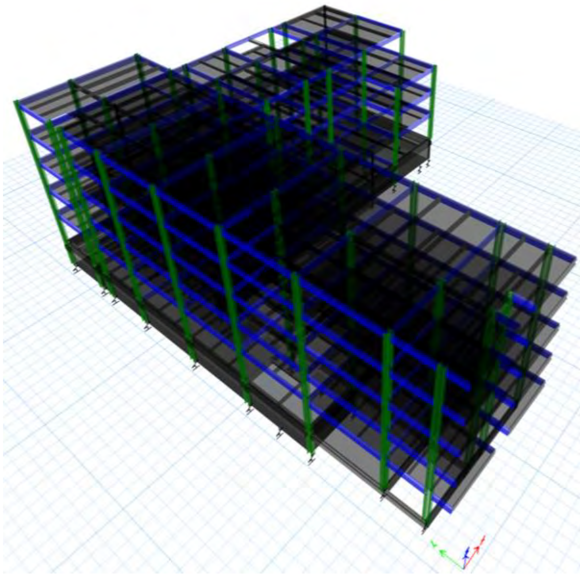

Figure 4: ETABS model - MRF system

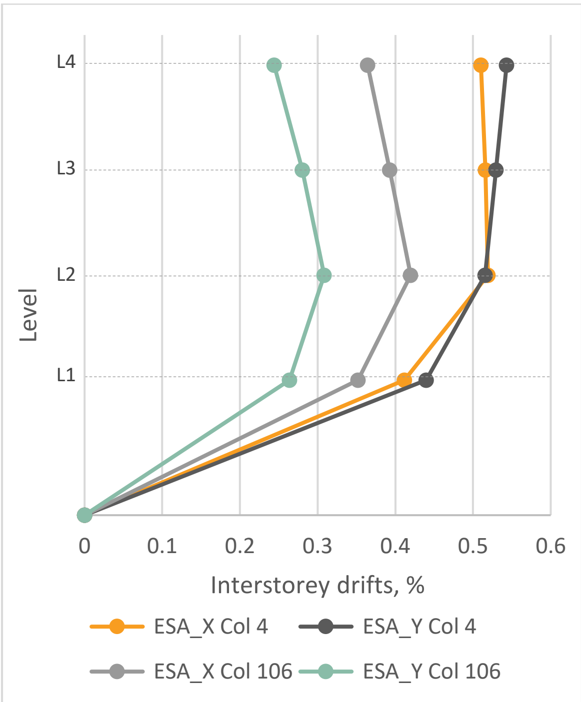

Figure 5: Interstorey drifts at corner column lines under ESA X & ESA Y – EBF system

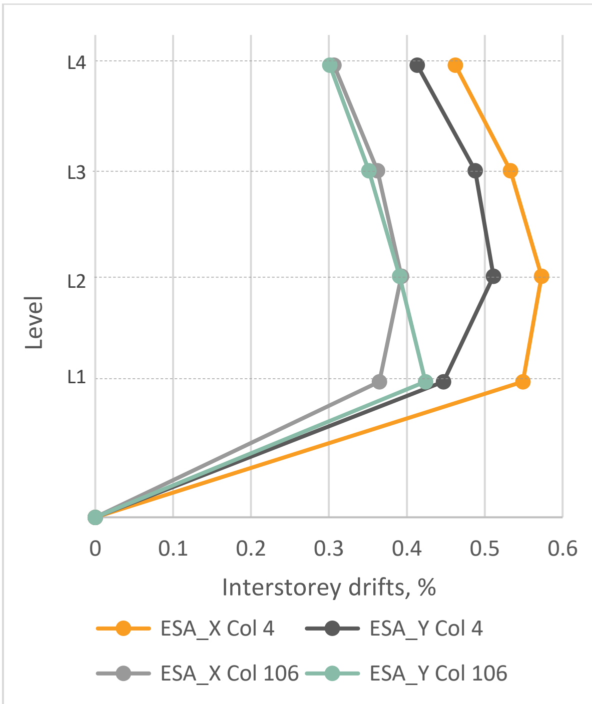

Figure 6: Interstorey drifts at corner column lines under ESA X & ESA Y – MRF system

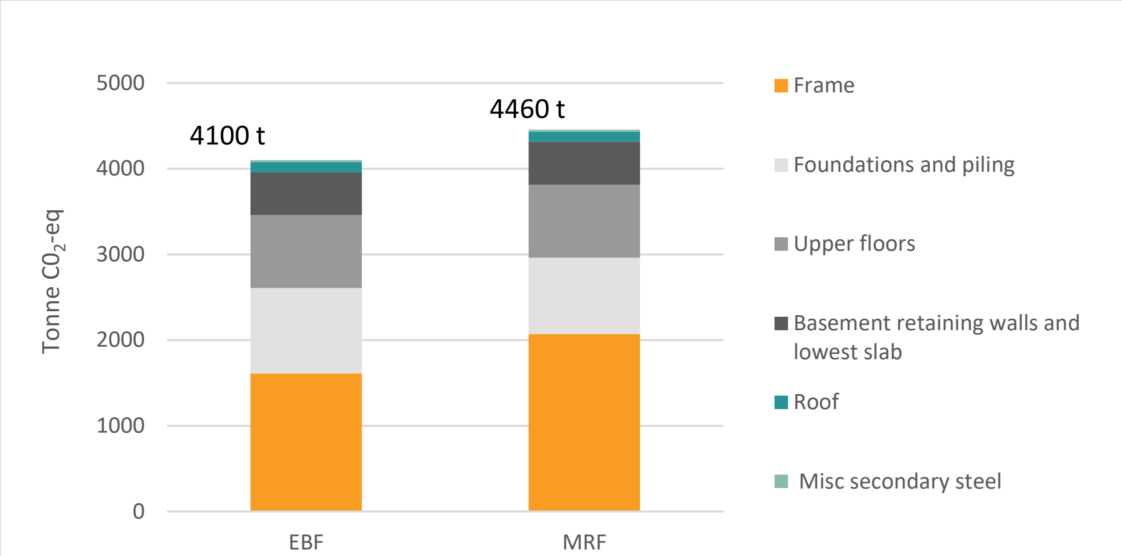

Figure 7: Comparison of total A1-A3 CO2-eq - Primary structure only

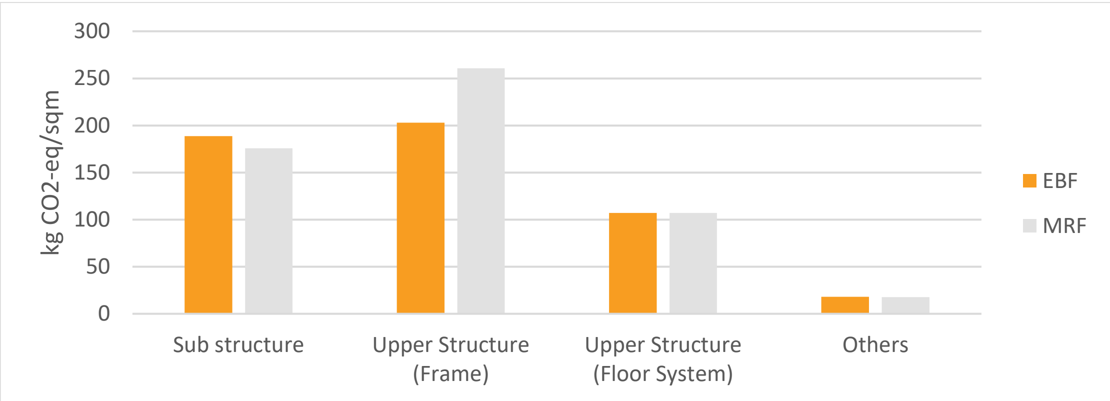

Figure 8: Comparison of A1-A3 CO2-eq by structural element

The key lies in fundamental design principles: regular buildings with clear load paths perform better seismically while also using materials more efficiently. Attention to structural hierarchy, system optioneering and connection detailing can enhance seismic performance without additional material use. In recent earthquakes, seemingly minor issues led to significant consequential damage. Some buildings remained un-occupiable for months despite minimal structural damage due to dislodged ceiling tiles or damaged sprinkler systems causing extensive flooding. The fixes to these issues are simple and well understood. Taping ceiling tiles and providing adequate seismic restraint are non-carbon-intensive interventions that could have dramatically improved the seismic performance of these buildings.

Moreover, we tend to think about seismic resilience primarily in terms of damage prevention and asset protection. However, equally crucial is recovery time. Structural systems that strategically concentrate deformations to specific zones can minimise damage to the primary frame while reducing the need for extensive removal of finishes and cladding during repairs. This approach enables faster building reoccupation and lowers the seismic carbon footprint of both the structural frame and interior elements.

In essence, the path to both seismic resilience and sustainability isn't through excess, but through smarter, more integrated design-thinking implemented early and continuously interrogated throughout the project life cycle (Loasby et al., 2025).

Does higher initial carbon for greater seismic performance pay-off over a building's life?

The short answer appears to be no — for a normal building, when considering the probability of a seismic event severe enough to demand major repairs or replacement, coupled with the associated carbon costs, the data suggests otherwise. When viewed across a building's expected lifespan, the potential carbon savings from prevented future repairs doesn't justify an additional upfront carbon expenditure (Toma, 2024). Within this, it is important to understand the multiple layers of uncertainty tied to potential future emissions. We must make inherent assumptions about the materials which may be used and the construction and demolition practices that might become standard in the future. We also need to estimate emissions associated with those future materials and practices, which are likely to be lower than today's levels as we drive toward a net zero economy (Symons & Moses, 2024).

Consider an Importance Level 3 building. While theoretically designed for a 50-year life, its primary structure could serve 75 years or more. The Ultimate Limit State (ULS) seismic design event has an annual exceedance probability of 1/1000, translating to a 7.5% chance of occurrence during that 75 year period. Code-minimum design should protect life safety in such an event, but does not intend to prevent the need for significant structural repair, or replacement. However, there is considerable uncertainty in predicted seismic loading and, in reality, our conventional design approaches typically deliver structures with capacity well beyond the minimum requirements. When we factor in the combined probability of both a ULS (or greater) event occurring and that event actually causing damage warranting major repair, the likelihood becomes remarkably low. This makes it very difficult to justify increased upfront carbon expenditure in the name of reduced future carbon due to enhanced seismic resilience.

For secondary and non-structural components, a similar probability analysis can be applied. With a Damage Control Limit State (DCLS) scenario at 1/250 annual exceedance probability and a typical 15-year fit-out cycle, there would be roughly a 6% chance of a DCLS event during the life of that fit-out. While this analysis may not make sense for buildings with post-disaster functionality requirements, it may provide some useful insight when considering carbon investment in low-damage design fit-out detailing for typical buildings.

The certainty of today's carbon versus the uncertainty of tomorrow's

While probabilistic analysis can help estimate potential future carbon emissions from seismic damage and repairs, these remain theoretical emissions that may never materialise. In contrast, the carbon we expend in construction today is an absolute certainty (Symons & Moses, 2024).

As our industry progresses toward decarbonisation, it's unrealistic to assume that future repair or replacement work would carry the same carbon footprint as current construction methods. New technologies, materials, and processes are continuously reducing the carbon intensity of construction. We are seeing clear moves towards this across Aotearoa's supply chain with initiatives like the release of Concrete NZ's Roadmap to Net Zero and the new electric arc furnace being installed at New Zealand steel. Moreover, emissions released today immediately begin warming the planet and persist in the atmosphere for decades, resulting in a greater cumulative warming effect than the same amount of emissions released in say 30 years time (ARUP, 2024).

Given this trajectory, and the urgent need to address climate change, our priority must be reducing emissions in the present rather than overbuilding for hypothetical future scenarios. This means schemes to decarbonise end-of-life processes, while beneficial, are less valuable than strategies that directly reduce upfront carbon emissions.

An alternative design hierarchy for the future

What if we could agree to fundamentally shift our priorities to place carbon reduction at the forefront of design? This might mean accepting higher ductility, emphasising building regularity, designing to minimise material waste, and leverage prefabrication.

What might it look like if we committed to this approach for say the next two decades, acknowledging that future rebuilding would likely have a significantly lower carbon footprint—how would our designs evolve? Could we meaningfully shift the dial on a climate target that we are currently falling well short of?

The devastation and the rebuild of Christchurch brought the profession into the spotlight, and required us to step further into the role and responsibility of civil leadership and advocacy than we perhaps imagined. The climate emergency is here now, and requires us to again step into this role. The reductions needed to meet our carbon budgets require more than the reductions which can be achieved solely through low-carbon material selection. Our design processes and thinking need to incorporate carbon from the outset. What we don't have is time. Incremental improvement will not achieve the reductions we need.

As an individual engineer, you may not feel you have the mandate to make decisions based on a low carbon remit, but engineers as a collective, all pulling in the same direction can.

Conclusions

- The structural engineering industry must lead the shift toward integrating carbon considerations into design processes, as current government regulation appears insufficient to drive the change needed and meet 2050 net-zero targets.
- Seismic resilience and low-carbon design need not be mutually exclusive. Smart design principles (regularity, clear load paths, strategic system selection) can achieve both goals without additional material use.
- For typical buildings, the data suggests that increased upfront carbon expenditure for enhanced seismic performance doesn't pay off over a building's lifetime, considering the low probability of major seismic events requiring repairs and uncertainty of future practices.
- Today's certain carbon emissions have greater climate impact than potential future emissions from repairs, especially as construction methods continue to decarbonise.
- The industry needs a priority shift that places carbon reduction at the forefront of design alongside other key drivers.
- Collective action by engineers is essential. Incremental improvements will not achieve the significant carbon reductions needed to address the climate emergency.

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Wind Engineering of Structures in New Zealand: Some Observations

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ABSTRACT

Designing structures for wind effects in New Zealand presents its own considerations compared to other international practice, which may not be immediately apparent when using the wind Standard. The high annual rate of strong oceanic storms and the probabilistic distribution of winds affects the risk profile, which differs from other areas in the world. Other aspects remain less understood, such as the wind-induced fatigue check, which application remains notoriously unpopular. This method applies well in the context of New Zealand climate but might benefit from some explanation and further simplification, both of which are provided in this publication. The revised 2021 version of the wind Standard is still not cited in Clause B1 Structure, with risks ranging from significant un-conservatism of new builds in parts of the South Island to potentially erroneous designs of dome-shaped structures. Wind tunnel tests are regularly used for design. If completed with relevant inputs from structural designers, this method can be a valuable tool to refine the design. This is more necessary for tall buildings due to the increased sensitivity to wind dynamic effects, but medium-rise façade design or unusual geometries could also benefit.

INTRODUCTION

Designing structures for wind in New Zealand presents unique considerations due to its distinct climate dominated by frequent, strong oceanic storms. This influences the probability of extreme winds and the associated risk profile differently compared to other regions. This paper offers observations on key aspects of NZ wind engineering practice, examining the local climate's effect on design parameters and discussing the implications of the uncited AS/NZS 1170.2:2021 standard Ref [1] – highlighting changes affecting specific regions and arched buildings. It also explores the practical benefits of wind tunnel testing beyond conventional uses and addresses wind-induced fatigue, outlining a simplified assessment method suitable for New Zealand standards.

NEW ZEALAND'S WIND CLIMATE AND EXTREME WINDS

Irrespective of the sensitivity of a structure to wind effects, all wind loading assessments, and for all methods method (code-based or wind tunnel testing) rely on the basic or regional wind speed V_R for New Zealand, typically defined for a range of yearly risk of exceedance or Return Period (RP). These basic speeds are fundamental for any wind design and are normally derived by analysing long-term wind data measured by anemometers at weather stations, then extracting the extremes and fitting those to statistical models. This statistical exercise is needed to extrapolate to the levels of risk consistent with engineering practice (i.e. 500-year RP for normal structures) and as it is rare to have more than 50 years of good quality wind data. There is still no agreement internationally about the statistical models to be used, not forgetting the frequent wind measurement errors, the various corrections for terrain and obstacles…, all these factors making this exercise notoriously perilous. Fortunately, some models are relatively simple and are based on mathematical parameters easier to check or compare with those of other areas. As shown in Figure 1, plotting the wind speed squared versus the RP on a log scale reveals a straight line defined by two key parameters, the mode (defined as the most likely maximum extreme wind speed) and the slope. A review of these two parameters is critical but the slope is of particular interest for structural engineers as it defines the variability between the extremes, or more explicitly, the ratio between the SLS and ULS wind loads. The slope is inherently linked to the nature of the wind mechanisms governing the wind climate, which could range from thunderstorms to cyclones, depending on the location. In New Zealand, while there are incursions of cyclones in the north mainly, the extremes are generally governed by the strong large-scale oceanic storms. In mathematical terms and using compound statistics for independent wind events:

$$P_{Vmax} = (P_V)^N$$

Where P_{Vmax} is probability of the extremes, N is the number of independent wind events and Pv the probability of all winds, ie. Parent wind data (usually Weibull distribution). The wind climate in NZ differs from other region worldwide in that it is subject to a relatively high number of independent wind storms. The number N is found to be well over 200, which is higher than the normal 160 events generally accepted for northern Europe. It also features an unusual shape of the Weibull distribution, with a larger spread, Weibull shape parameter significantly higher than 2. Following Equation 1, these two differences have for direct effect to flatten the slope of the curve discussed above and reduce the variability of the extreme. Note that the revised wind Standard 2021 correctly adjusted the slope of the curve resulting in a ratio of the ULS over SLS wind loads equal to 1.33 instead of 1.5 in Northern Europe for example. Ref [2], this might be even more pronounced using alternative fits. In any cases, the variation between extremes in New Zealand is less than in northern Europe and significantly less than for Cyclonic regions, which is a direct result of the wind climate characteristics.

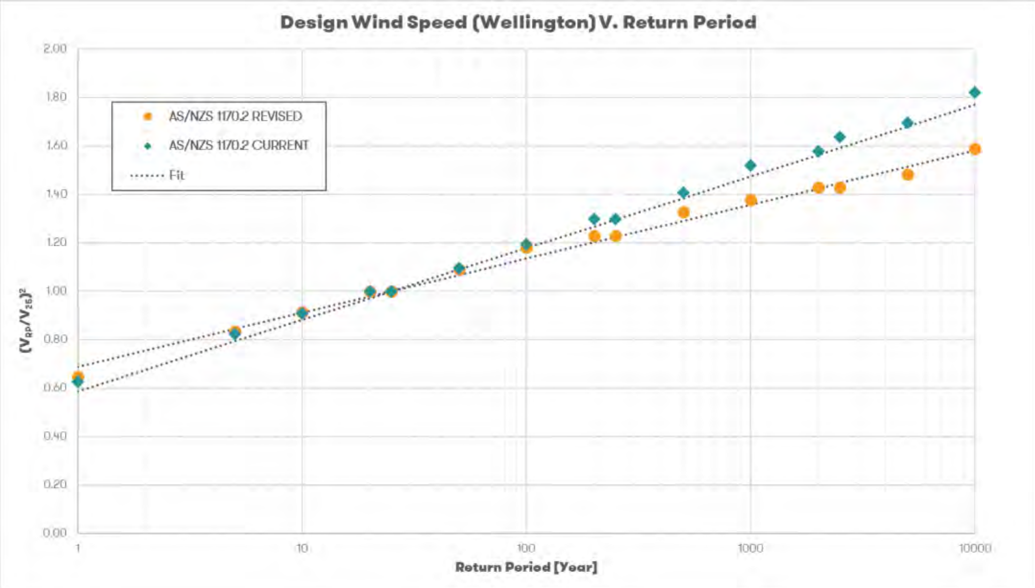

Figure 1: Variation of Design Wind Speeds with Return Pariod (Wellington)

CHANGES TO AS / NZS 1170.2:2011

Background

The revised 2021version of the Standard for Wind Loading is still not cited in Clause B1 Structure, leaving the 2011 version as the primary reference for designers. Unfortunately, this results at best in inefficiencies in the design process, with designers having to repeat the wind loading assessment using the two standards and at worse in unconservative modern design, especially as the revised standard introduces several increases in the design wind loads including (but not limited to):

- New region at the south of the South Island, up to 50% increase in all wind loads. Directional multipliers and new lee zones
- Increase in local pressures K_ℓ for steep duo-pitched roof
- Increase in internal pressure for dominant openings
- Circular structures, solar panels and tall buildings in broad term

Arched Buildings

The change to the design data applicable to arched building has become particularly critical for structural engineers, in particular the introduction of the asymmetric load case. Arched structures tend to be less efficient at resisting asymmetric loads, making this update significant for design. On a real-case example, wind tunnel tests were completed on a large arched

building to optimise the wind loads for this asymmetric load case. The process involved measuring local pressures around the structure, then integrating these measurements using influence surfaces to capture the critical structural forces—including maximum uplift, downforce, and lateral forces. This integration process required close collaboration between wind and structural engineers to ensure raw data were processed to capture the relevant structural actions. Results revealed that lateral loads may be reduced slightly only from that of the revised in the Standard, which may be partly due to sheltering effect and / or various reduction in wind speeds. The critical direction for the lateral wind load case was found to be that attacking the second arch from the corner on a skewed angle, increasing the pressure on the windward side, while the downwind side is subject to wind accelerating around the downwind corner. Pressures on the other arches were found to be notably lower. In any case, the revised standard's design data for such geometries aligns better with international guidance and it is strongly recommended designers adopt these updated provisions for arched structures, even though the 2021 Standard has not yet been formally cited.

OTHER CASES OF WIND TUNNEL TESTING

Wind tunnel testing is a classic method to assess wind loads and is typically needed and clearly valuable for tall or dynamic structures. It was however recently found that this methodology may be valuable for medium-rise façade and secondary structural design including in the following cases:

- For narrow building, i.e. building slightly taller than wider, the wind Standard forces a 50% increase in local pressures through the increase in K_ℓ from 2 to 3. As for any step changes, this might be considered as severe in certain context, and with associated significant impact on quantity and costs of glass, especially for buildings with a second long face.
- Light timber buildings potentially on base isolation system also have the potential to become wind sensitive. Wind tunnel tests can be valuable to assess in more details the performance of the building under cyclic wind loads.
- However, in such cases, the results of the wind tunnel tests typically require a level of rationalisation to manage risks and potential under-conservatism in complex and rapidly changing urban areas.

WIND FATIGUE ASSESSMENT

Background

Wind fatigue is the mechanism by which successive cycles of wind loads inflict an increment of damage to the structure. Typical structures subject to wind fatigue are light poles, sculptures or road signs although light roofs and stadia may be subject to fatigue issues too. The response of a structure to the wind gusts generally includes a quasi-static broad-band and a dynamic narrow-band components, both typically acting in the same direction as the winds. As such, a time-history of wind-induced stresses typically exhibits a narrow band component at the natural frequency of the structure superimposed on the wide-band random process of the gust-induced stresses. Figure 2 below shows the fatigue damage separately for the different components and for a moderately dynamic structure. The broad-band component alone (shown in blue) would cause considerably less fatigue damage than the narrow-band component (shown in green) if each were considered separately. Interestingly, looking at the total response (in red), there is also a redistribution of the cycles from the lower to higher stress ranges, as the narrow-band component is superimposed on that of the broad-band. This mechanism increases the damage from that of the narrow-band in isolation.

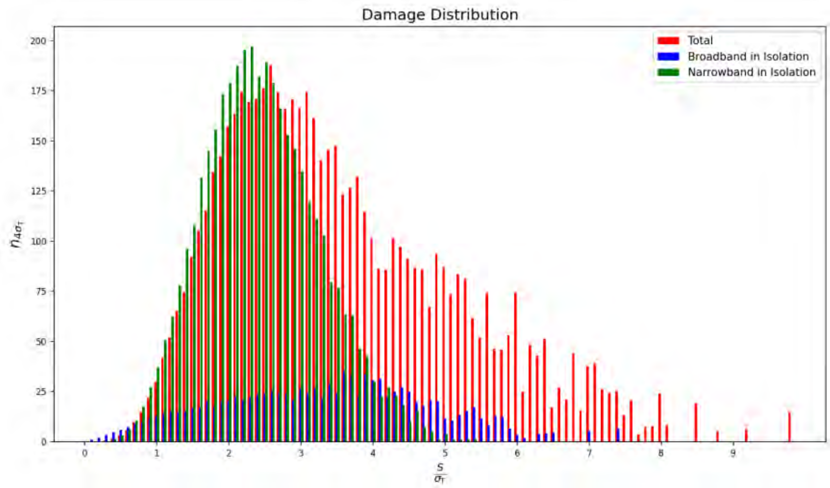

Figure 2: Wind Fatigue Damage Distribution (Moderately dynamic line-like structure)

The additional following results may be found informative for structural designers. More details can be found in Ref [3] and other essential reading on this subject can be found in Ref [4].

- Structural damage is more sensitive to the stress level than the number of cycles. This is due to the fatigue damage varies with the cube of the stress. Fatigue assessment should therefore be done based on stress rather than fatigue life.
- For a given hour of wind, the most efficient stress level to induce fatigue is relatively high, approximately 2-4 times the standard deviation of the stress time-history.
- Over a year, the most efficient wind event to induce wind fatigue damage is for wind events of mean speed equal to 2-3 times the yearly mean speed, relatively rare.
- The wind spectrum shown in the wind Standard is not quite correct for non or moderately dynamic structures. However, this does not affect significantly the fatigue assessment, which can still be used in a conservative manner.

Simplified Fatigue Check

This check is typically done by using the Miner's rule, based on the fatigue design spectrum of the win standard and the S-N curves from the relevant structural Standard, such as the NZS 3404 Ref [5]. This calculation is notoriously confusing and prone to mathematical errors. A useful note introduced in the revised Standard concerns the fatigue assessment and form the basis of a significant simplification. The issue is that the note is only for use with the S-N curves Australian steel Standards Ref [6]. The exercise was repeated by the author for the S-N curves of NZS 3404 Ref [5] and NZS 5100.6 Ref [7]. The same result was found to apply as follows:

$S_{max} < R_{Fatigue} \times \Phi \times$ Detail Category

- S_{max} is the maximum stress induced by wind in the Design Life of the structure. If the Design Life is equal to 50 years, it would be the stress calculated for a 50-year Return Period wind event.
- $R_{Fatigue}$ is a factor equal to 3.59 for the S-N curves of NZS 3404 and 3.65 for the S-N curves of NZS 5100.6. The design rule above is exact for the S-N curves of NZS 5100.6 and up to 4% conservative for the S-N curves of NZS 3404 S-N, depending on the detail category.
- Φ is the strength reduction factor as defined in the two standards.

CONCLUSION

This paper highlighted several key observations for wind engineering in New Zealand. The specific wind climate affects the statistical distribution of extremes and the ratio between serviceability and ultimate loads. The uncited AS/NZS 1170.2:2021 standard creates practical challenges, with notable change for arched structures. The value of wind tunnel tests was recently found to extend beyond tall buildings to optimize medium-rise façades and secondary structures in specific cases only. The relevance of wind-induced fatigue was discussed, alongside a simplified check compatible with NZS 3404 and NZS 5100.6.

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Advancing Structural Engineering with AI

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Abstract

The rapid transformation of structural engineering through Artificial Intelligence (AI) is becoming increasingly evident, as AI enables unprecedented automation, computational efficiency, and accuracy in preliminary and detailed design.

AI-driven methods, particularly through reasoning-enabled language models, have evolved significantly to handle complex structural engineering computations reliably. The ability to fine-tune base AI models, using customized training data, allows structural engineers to incorporate specialized engineering knowledge into AI-driven workflows.

This paper outlines technical methodologies leveraging AI, detailing numerical representations of 3D structural geometry, finite element analysis considerations, and structural design optimizations.

Through examples related to the leveraging of structured input-output protocols and advanced prompt engineering, we demonstrate how AI can systematically simplify, analyse, and validate structural engineering design tasks. The paper underscores the necessity of human oversight to validate and finalize AI-generated designs, addressing common concerns related to intellectual property and data protection in AI workflows.

Introduction

AI has significantly evolved, becoming an indispensable tool within most professions. Initially limited to basic computational tasks, recent advancements have enabled AI to process complex mathematical operations and incorporate reasoning models that enhance reliability and accuracy. Modern language models now not only solve intricate equations but also provide logical reasoning for results, significantly improving decision-making processes.

Accessibility to advanced AI has dramatically increased, allowing structural engineers to either:

- Employ commercialized model endpoints; or
- Train base models with custom data specific to their projects.

Fine-tuned models can now adeptly address specialized structural engineering challenges, enabling efficient preliminary designs and validation processes that complement existing structural engineering workflows, throughout the following stages of the process:

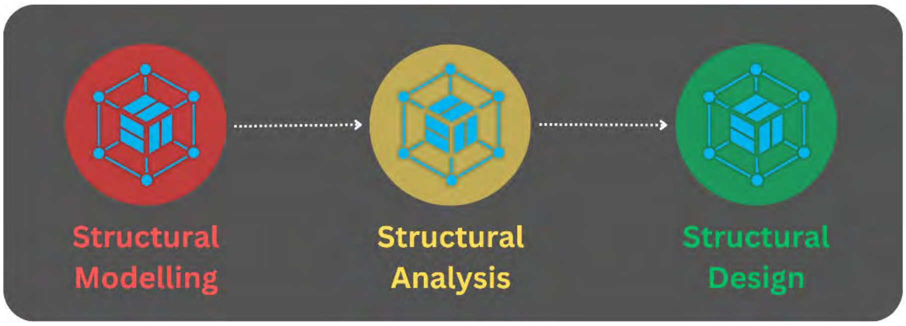

Figure 1: Structural Engineering Workflow (Vorster 2024)

3D Geometrical Problem Definition

Accurate numerical representation of structural models is crucial in structural analysis workflows. Spreadsheet-based input systems (or any structured table data) provide organized, compartmentalized data management for defining structural models. This systematic approach simplifies complex problems by clearly delineating material definitions; sectional properties; node coordinates; framing elements; element connectivity; design groups; load cases; loads; and load combinations as follows:

- **Material** definitions are numerically encoded, including mechanical properties such as modulus of elasticity, shear modulus, Poisson's ratio, and characteristic strengths.
- **Section** definitions specify geometric and sectional properties like cross-sectional areas, moments of inertia, and torsional constants, critical for structural analyses.
- **Node** definitions detail coordinates within three-dimensional space (X, Y, Z), forming essential reference points for structural connectivity.
- **Framing** elements, clearly linked to nodes, are defined by their stiffness and connectivity, essential for accurate load transfer and structural response evaluation.
- **Element connectivity** ensures precise spatial relationships, crucial for assembling accurate stiffness matrices. Design groups aggregate components based on various design or loading criteria, allowing targeted analyses.
- **Load cases** define specific loading scenarios such as dead, live, wind, or seismic loads, each precisely defined to ensure clarity in structural responses. Individual loads are detailed, including their magnitudes, directions, and locations. Load combinations systematically integrate these load cases, applying appropriate factors to simulate realistic design conditions.

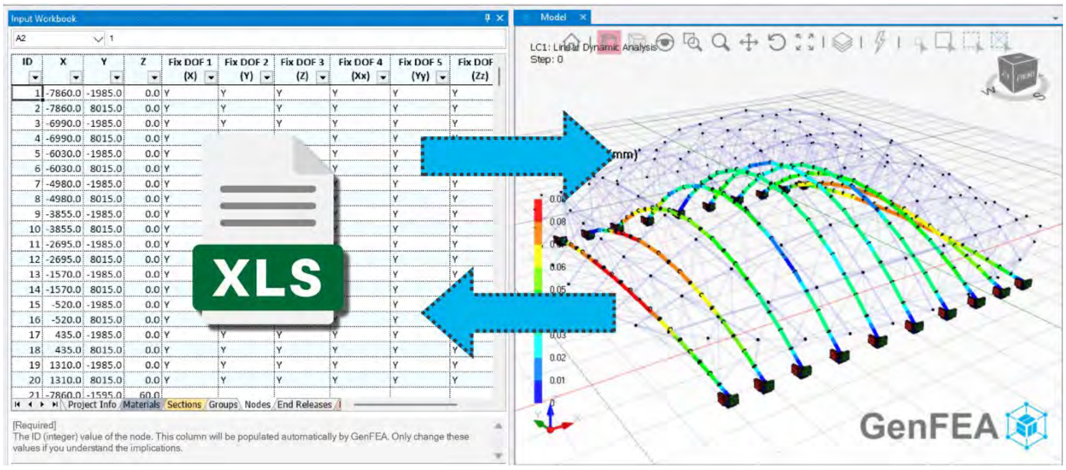

Figure 2: Data and Graphical Model relationship

Spreadsheet input workbooks (a form of structured table data) facilitate bi-directional communication with graphical 3D modeling environments, ensuring data consistency between numerical definitions and visual representations.

These clearly structured tables significantly aid language models (LLMs) in accurately interpreting and generating structural models. With this simplification, LLMs can effectively create basic to intermediate complexity structural systems such as portal frames, crane gantries, and rectangular concrete building frames - autonomously.

Our testing also shows that high performing LLMs, such as the latest o3-mini (OpenAI), R1 (DeepSeek), and Claude 3.7 (Anthropic) models, are even capable of parsing input model data using this format to generate wind loads on structures to medium complexity – fully automated and based on project geographic location.

Finite Element Analysis (FEA)

Numerical methods currently remain optimal for most structural analyses. Sparse matrix solvers, essential to finite element methods, are computationally efficient, reliable, and well-established, outperforming AI-driven methods in general scenarios due to their consistent computational reliability.

Machine Learning (ML), despite advancements, still requires substantial training time and extensive datasets, limiting its applicability primarily to repetitive, well-defined structural scenarios. The effectiveness of ML in generalized structural analysis is currently limited, reinforcing numerical approaches as the preferred method.

Conversely, Computational Fluid Dynamics (CFD), which requires significant computational resources, presents opportunities for AI integration. AI-driven techniques can potentially

accelerate iterative solution processes, reducing computational overhead in complex fluid-structural interaction scenarios.

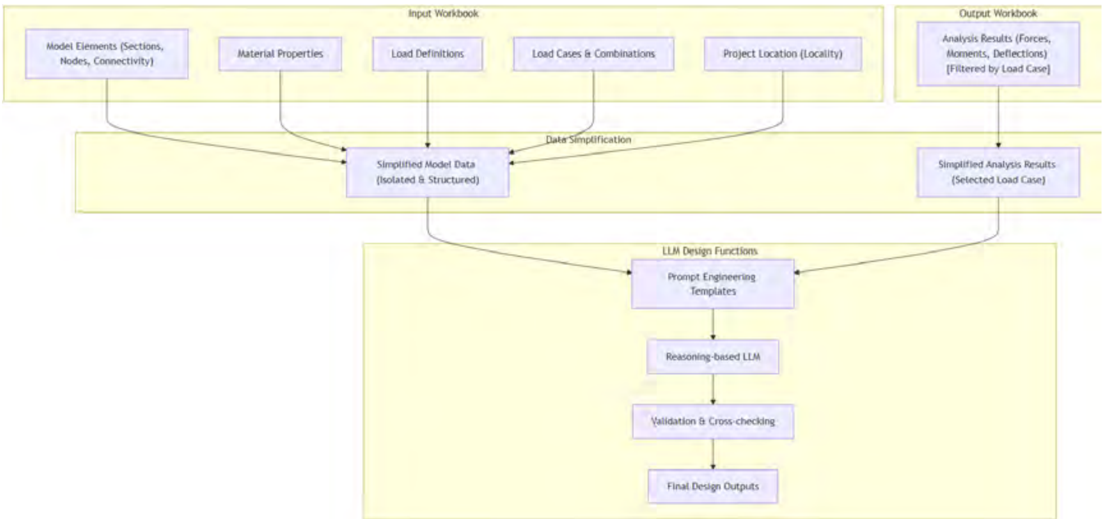

Figure 3: Extracting Critical Information from Larger Structural Datasets

Structural Design

AI has the potential to significantly aid structural design processes, but large datasets (3D model definitions, analytical results, etc.) may prove challenging for LLMs to digest – due to limitations in context window sizes. By simplifying complex design problems and isolating distinct subsets (such as **load** cases / combinations, and **design groups**), we can fit element design data into context window sizes.

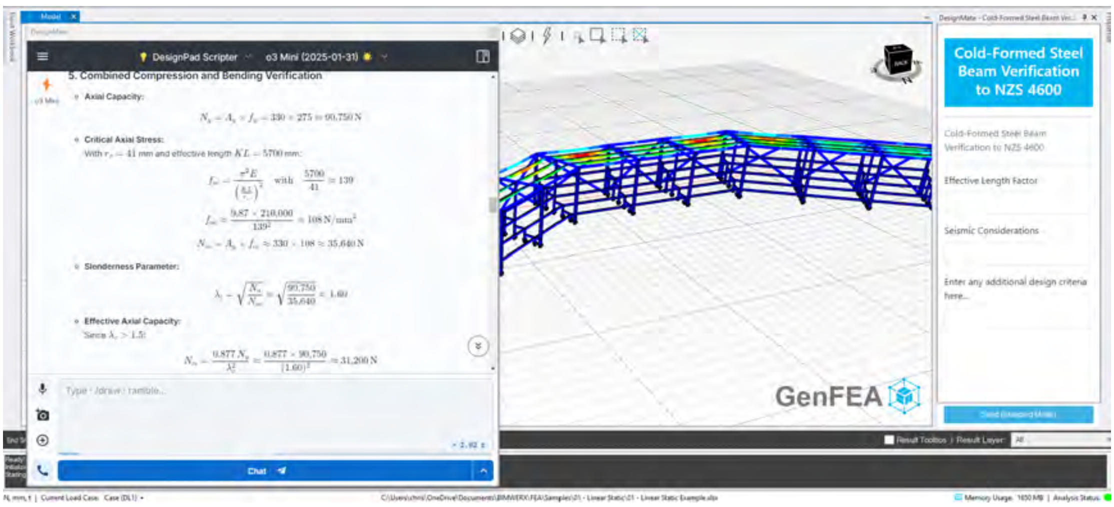

Figure 4: Example LLM Structural Design Output and Tool Use

Ensuring consistency in AI output involves structured prompt engineering techniques. Detailed, clear instructions and prompt templates guide language models, establishing reliable, repeatable results. Consistency is further enhanced by controlling AI parameters such as temperature settings, typically set to zero for lower ‘creativity’ and more concise responses, and by utilizing reasoning-enabled AI models.

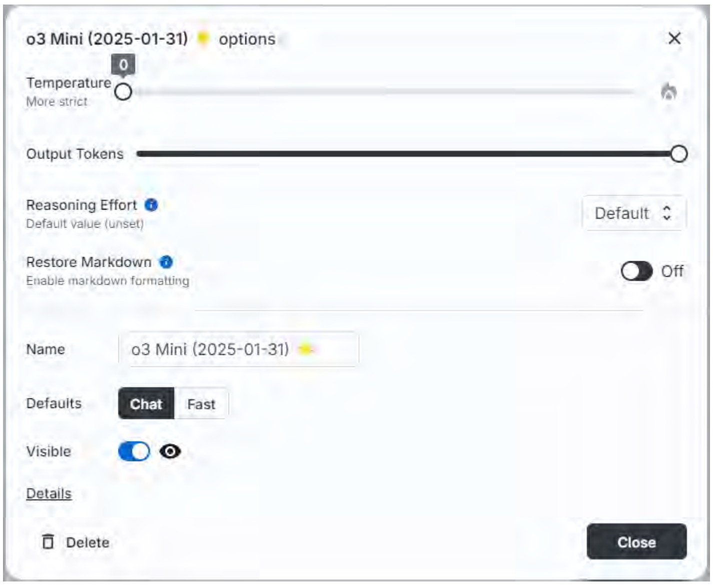

Figure 5: Example of LLM Temperature Setting

Fine-tuning AI base models involve training them with extensive, project-specific problem-answer datasets (ideally 1000+ pairs). Rapid advancements in AI training technologies enable structural engineers to fine-tune and deploy these specialized models locally using affordable GPU hardware, increasing accessibility and customization.

Validation of AI-generated designs relies on systematic reasoning models, ensuring logical coherence and computational correctness. These outputs undergo further cross-validation via established calculation scripting engines or dedicated computational tools, enhancing confidence in the results.

Intellectual Property

Most intellectual property (IP) concerns are addressed through AI service providers’ data sharing policies, which generally allow end-users to restrict proprietary data sharing. Additionally, neural network methodologies ensure data interpolation rather than direct IP sharing, significantly mitigating IP risks.

Locally run LLMs ensure 100% control over data sharing, while reducing carbon footprint due to lowered resource requirements for running smaller, more specialized models. Locally trained models can perform on par with commercial LLMs on specific tasks if trained correctly.

Conclusions

AI is rapidly reshaping structural engineering practices, driving efficiencies and innovation that significantly enhance productivity. Its continued development represents a profound shift in industry methodologies, compelling engineers to integrate AI-driven solutions to maintain competitive advantage.

Nevertheless, structural engineers retain ultimate responsibility for final design decisions. AI serves effectively as an advanced automation tool, facilitating preliminary designs, validating outcomes, and streamlining detailed reporting, while human oversight ensures the reliability and integrity of structural engineering outcomes.

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Embedded Retaining Walls – methodology selection

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ABSTRACT

For structures with basement the selection of the correct embedded retaining wall can have significant impact during construction and over the life of the structure. In recent years installation technology has developed, and the number of possible options has increased making the selection of the most appropriate methodology given project constraints and functional requirements has become more complicated.

This paper will provide guidance on the embedded retaining wall selection process looking at the primary considerations:

- Requirements over the life of the structure
- Buildability of the wall given project constraints and ground conditions
- Design efficiency and associated GWP / CO2
- Environmental and resource consent compliance

Case studies will be used as examples to highlight the selection process, challenges encountered, outcomes and learnings. The use of appropriate specifications and guidance documents will be covered as background to the development of the NZ specification.

Background

In recent years the development of technology and the nature of NZ projects has resulted in the adoption of a wider range of embedded retaining wall types. Like most specialized technologies, one solution does not fit all, and it is important to be aware of the various options available; figure 1 provides an outline of the structural forms available. To select the most appropriate option for an embedded retaining wall the benefits and limitations of each must be understood in the context of the specific project. Importantly, the need to adequately specify the methodology to ensure the expected performance objectives can be and are ultimately achieved.

Concept design

The purpose of the embedded retaining wall in the long-term needs to be resolved at an early stage. The Client expectations of basement walls can require difficult conversations regarding the space taken up by the wall structure and the watertightness.

In the context of a basement subject to groundwater pressures the ICE "Reducing the risk of leaking substructure, a Clients guide" (2009) is a useful reference in conjunction with BS8102:2009 to agree grading based upon the usage of the basement and thus determine acceptable performance level.

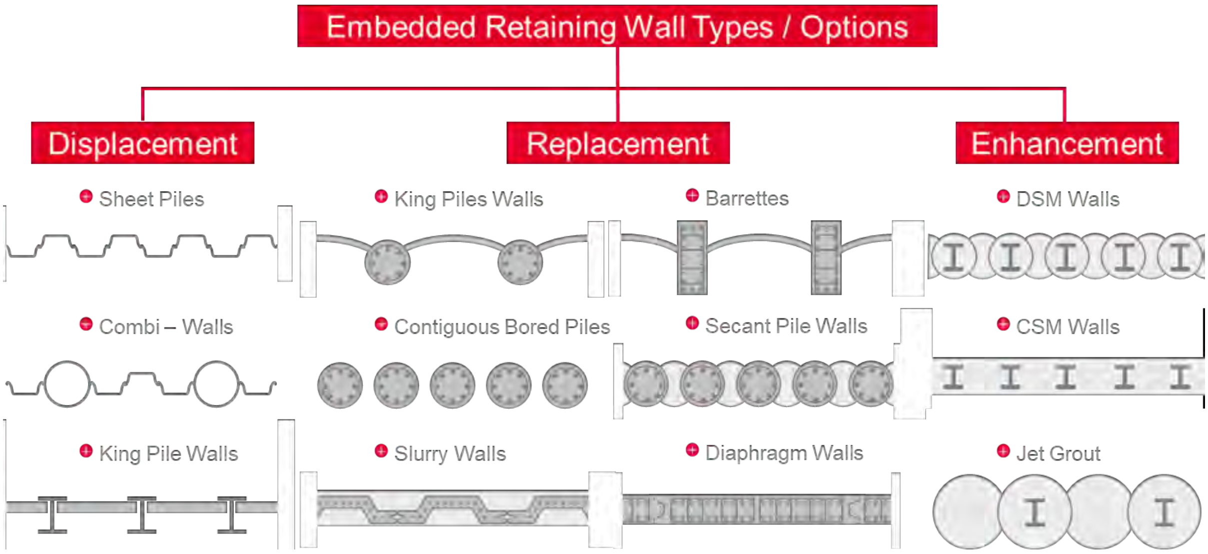

Figure 1: Embedded retaining wall options summary

With the performance level and ground water levels defined; the nature of the soils to be retained will determine whether an open structure with gaps between structural wall elements are sufficient. For example, in firm to stiff clays king pile or contiguous bored piles can provide the necessary temporary support to allow excavation and sequential completion of the exposed retaining wall face. However, in sands or gravels below the water table the soil would flow between wall elements into the excavation causing instability and settlement; in this situation the wall needs to be continuous like sheet piles, secant piles or diaphragm walls. Particularly in urban areas, embedded retaining walls play a significant role in managing ground water inflows into the excavation and thus groundwater drawdown outside the basement that can cause settlement.

Where temporary ground support and groundwater control is achieved using an embedded retaining wall the basement grade and durability are primary factors to determine the nature of the wall and any associated waterproofing protection that may be required. BS8102:2009 defines three types of waterproofing protection:

Type.A - Type A barrier protection comprising the installation of a membrane. However, for embedded retaining walls this can only be applied to the exposed internal face and thus generally impractical as it must be sufficiently bonded to the face to resist water pressures.

Type.B - Type B structurally integral protection considers the structure resists water ingress. In the context of an embedded retaining wall the structural design considers crack widths, waterbars at cold joints, etc. For diaphragm walls the panels can be designed and detailed to achieve this, however, secant bored pile walls will require an internal lining wall designed to resist water pressures.

Type.C - Type C drained protection comprises the construction of a separate inner wall which creates a cavity that can manage groundwater ingress through the retaining wall. This can be applied to all embedded retaining wall types above the basement slab. Maintenance of the space and pumps are essential to maintain effectiveness.

It is useful to align the above requirements with the prevailing ground conditions and site constraints such as geometry, affected stakeholders, sensitive structures, etc. in a formal comparison approach as presented in figure 2.

Methodology Selection Wall Type	Typical Retained Height (m)	Wall Layout	Stable / Cohesive Soils	Unstable / Granular Soils	Water Retention	Durability	Liner wall	Access / Space	Noise / Vibration	Cost	Sustainability
Sheet Pile	3 – 10	✓✓	✓✓✓	✓✓✓	✓	✓×	Y / N	✓✓	××	$	✓✓
Combi Wall	10 – 15	✓	✓✓✓	✓✓✓	✓	✓×	Y / N	✓✓	××	$$	✓✓
King Piles Walls	3 – 6	✓✓	✓✓✓	××	×××	✓✓✓	Y	✓✓	✓✓××	$	✓✓✓
Contig. Bored Piles	3 – 10	✓✓✓	✓✓✓	✓✓	×××	✓✓✓	Y	✓✓	✓✓	$$	✓✓
Slurry Walls	3 – 6	✓×	✓✓✓	✓✓✓	✓✓✓	✓✓	Y / N	✓	✓✓✓	$$	✓✓
Barrettes	8 – 20	✓✓	✓✓✓	××	×××	✓✓✓	Y	✓✓	✓✓✓	$$	✓✓
Secant Pile Walls	5 – 7 / 10 7 – 15 (c)	✓✓✓	✓✓✓	✓✓✓	✓✓	✓✓	Y / N	✓✓	✓✓	$$$	✓
Diaphragm Walls	7 – 20 +	✓	✓✓✓	✓✓✓	✓✓✓	✓✓✓	N	✓	✓✓✓	$$$	✓✓
DSM Walls	3 – 10 m	✓✓	✓✓	✓✓✓	✓	✓✓✓	Y	✓✓	✓✓✓	$$	✓
CSM Walls	3 – 10 m	✓	✓✓	✓✓✓	✓✓	✓✓✓	Y	✓✓	✓✓✓	$$	✓✓
Jet Grout	3 – 6m	✓✓✓	✓✓	✓✓✓	✓✓	✓✓✓	Y	✓✓✓	✓✓✓	$$$$	×

Figure 2: Methodology selection considerations and approach

The table above provides an example of a qualitative assessment of different embedded retaining wall options that, with the input of a specialist contractor experienced in the different methodologies, would improve the selection process at an early stage.

The overall construction thickness is often a parameter derived at concept stage as this impacts the effective useable space in the basement. A project example comparison for a 10m deep basement wall is provided in Table 1. In this case the 35m x 50m urban project site was large enough to establish the diaphragm walling plant and the selection provided a useable basement area of 1584 sq.m which was 80sq.m greater than that using a secant pile option.

Table 1: Example comparison of overall wall thickness.

Aspect	Secant Pile Wall	Diaphragm Wall
Guidewall / building clearance (mm)	200	200
Structural thickness of retaining wall (mm)	900	600
Construction tolerances (mm as SPERWall)	25 + 100	25 + 50
Levelling layer & waterproofing (mm)	50	0
Internal lining wall (mm)	200	0
Total offset of inside face from boundary (mm)	1475	875

The need for an internal lining wall or cavity wall is largely due to the cold joints between piles and the durability of the low strength concrete used in alternate unreinforced piles. As described (Wharmby 2010) and (Gannon 2016) the ability to drill the low strength concrete piles at early age can be in conflict with long-term durability of the piles. It noted that specifying higher strengths can lead to lack of interlock between piles and poor overall secant pile wall performance.

Detailed design

The detailed design of the retaining wall methodology needs to be aligned with the specific construction method appropriate to the installation. Taking secant piles as an example, the piles can be installed using traditional single length casing, segmental casing or continuous flight auger (CFA) techniques. The selection of the technique will generally depend upon the ground conditions and cost; however, the size and achievable verticality tolerances vary considerably which will affect both the basement space and pile layout / interlock depth. The ICE Specification for piles and embedded retaining walls (SPERWall 3rd Edition 2017) provides well researched guidance on the achievable construction tolerances in table B1.4 and guidance on the depth of reliable interlock for secant piles. For secant piles the use of a guidewall / system is required to locate the piles at a positional tolerance of +/- 25mm but as can be seen in Figure 3 the verticality tolerances vary from 1 in 75 and 1 in 200 for bored piles and the reliable interlock depth range from 5m with conventional bored piles using short casing to 20m where full depth segmental casing is used.

Method	Technique	Tolerance	Depth of pile interlock
Contiguous (C) or Secant (S) piled walls	Minimum tolerance	1 : 75 (C) 1 : 75 (S)	N/A 0-5 m
	CFA using extra heavy duty augers*	1 : 125 (C) 1 : 125 (S) 1 : 75 (S)	N/A 0–7 m 7–10 m
	Cased CFA	1 : 150 (C) 1 : 150 (S)	N/A 0–15 m
	Bored cast-in-place using standard tools	1 : 100 (C) 1 : 100 (S)	N/A 0–7 m
	Bored cast-in-place using stiffened casings with cutting teeth	1 : 200 (C) 1 : 200 (S)	N/A 0–20 m
Diaphragm walls	Cable grab	1 : 100	N/A
	Hydraulic grab	1 : 150	N/A
	Reverse circulation mill	1 : 250	N/A

Figure 3: Extract from SPERWall Table B1.4

SPERWall is a specification used and developed for over 20 years with input from Clients, Consultants and Contractors. It provides specification and guidance for all embedded retaining walls methods; an NZ addendum has been developed jointly by SESOC and NZGS technical groups to facilitate adequate specification of the different methods now available in NZ and raise quality standards. This has been issued for comment and will be published shortly.

The standard sizes available for each pile installation method are available on the Federation of Piling Specialist (NZ) website (www.federationofpiling.co.nz/resources). Adopting standardized pile sizes provides programme and cost benefits related to tooling and casing. With segmental casing the diameters are adopted globally due to the cost of fabrication of the thick wall, double skin casing and specifically machined jointing system.

Consideration of the climate change impact is now a routine process that can provide some surprising results when the CO2 equivalent of different methodologies is calculated. A specific project example is provided in figure 4 for a basement structure.

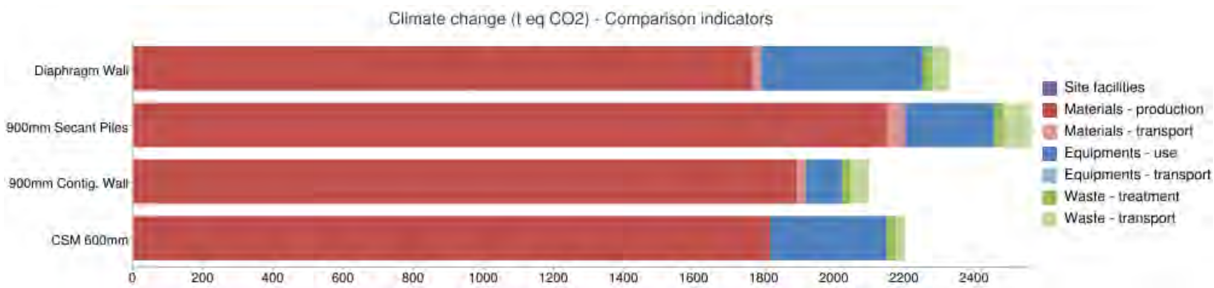

Figure 4: Prism CO2 calculation for basement wall methodologies

This indicates that the least impact is associated with a contiguous bored pile wall, however, this was rejected as the lack of groundwater control represented an unacceptable risk of large groundwater discharges and dewatering settlement of adjacent structures. The 18m depth limit of the CSM machine meant it was unable construct the 22m deep embedded retaining wall elements. Diaphragm wall and secant pile wall options are both suitable at the site; the diaphragm wall has less impact due to the more efficient structural profile and because the secant pile wall is cutting back into firm piles / removing concrete and generating more spoil. This was also confirmed (Page 2023) on the City Rail Link project where diaphragm walls were adopted for the underground station boxes.

When detailing the structural elements that make up the embedded retaining wall this needs to be aligned with the construction method adopted. Undoubtedly the most significant quality issues that arise are associated with congestion of reinforcement. Most reinforced concrete elements cast in formwork using low slump vibrator-compacted concrete, embedded retaining wall elements are cast "blind", against the ground, using high slump concrete. The concrete is placed using a hopper and tremie pipe that extends to the full depth of piles that are wet or contains drilling support fluid; the concrete displaces the water or drilling fluid and must flow freely around the reinforcement to ensure structural integrity. The importance of the material and placement method has led to global research and the development of joint guidance by European Federation of Foundation Contractors (EFFC) and Deep Foundations Institute (DFI); Guide to Tremie Concrete for Deep Foundations (EFFC/DFI 2024). From a reinforcement detailing perspective,

the **<u>clear</u> spacing between bars = 100mm**
or 5 x the maximum aggregate size in the concrete mix.

Construction example observations

For Secant pile walls, projects have shown the layout, low strength concrete and associate pile construction sequence are key technical aspects to successful installation of secant pile wall elements. In figure 5 working through the ECI phase we were able to rationalize the layout of the piles to optimize the achievable installation tolerances using the CFA pile installation method.

Following construction of the secant piles the internal lining wall type and construction can impact the waterproofing performance.

Figure 5 Secant pile wall

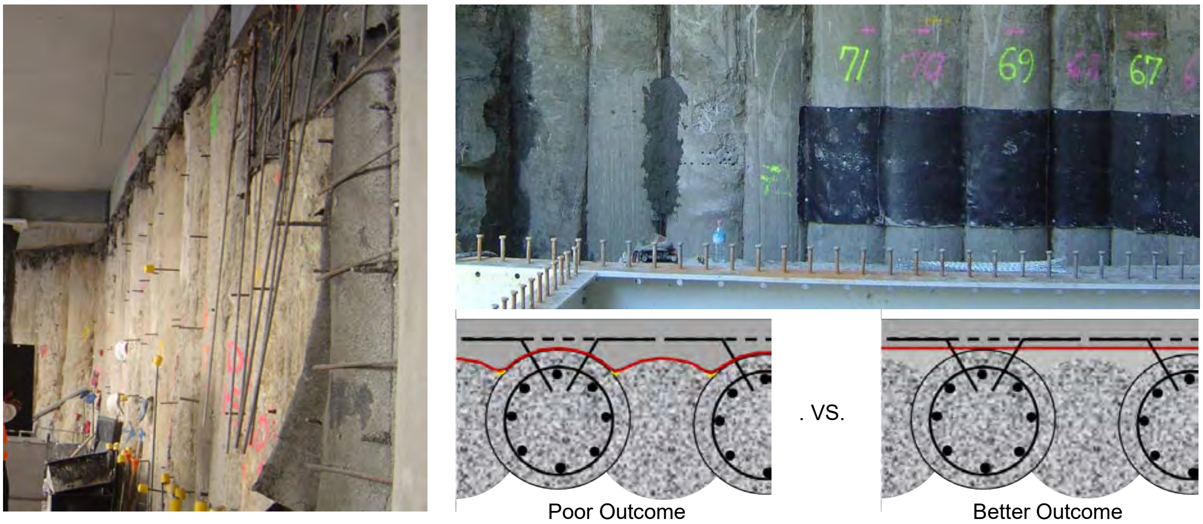

Figure 6: Secant pile wall linning wall

In figure 6 a bentonite waterproofing membrane and cast in-situ lining was specified. However, the application of the membrane to the irregular form of the wall and use of sprayed concrete did not provide the necessary confinement. A levelling layer to provide a good substrate for the waterproofing membrane and an in-situ concrete pour to apply pressures greater than the external water pressure gives a better outcome.

The construction of diaphragm walls is centered around the panel layout with panel lengths typically between 2.8m and 7.0m impacting excavation method, inter-panel joint type, reinforcement cages and concrete pours; this requires specialist contractor input.

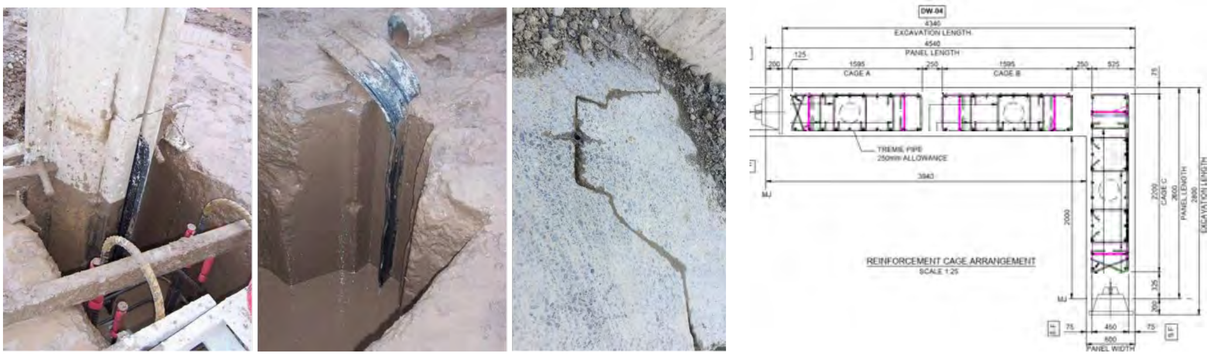

Figure 7: Diaphram wall inter-panel joint & cage layout

The connection to floor slabs can be achieved using couplers embedded in the panel and, with a box-out, can be readily located as the internal face of the wall is exposed.

The SPERWall document provides a good specification for diaphragm walling that includes rebar spacing requirements. However, these need to be followed as it is still the case that the greatest cause of quality issues is congestion of rebar; a case study (Wharmby 2021) highlighted such issues on a project. Actual rebar clear spacing is less than the theoretical clear spacing due to the unreinforced inter-panel joint and space required between cages. It is noted that additional rebar is required for cage fabrication, lifting of cages from horizontal to vertical and suspending the cage in the trench.

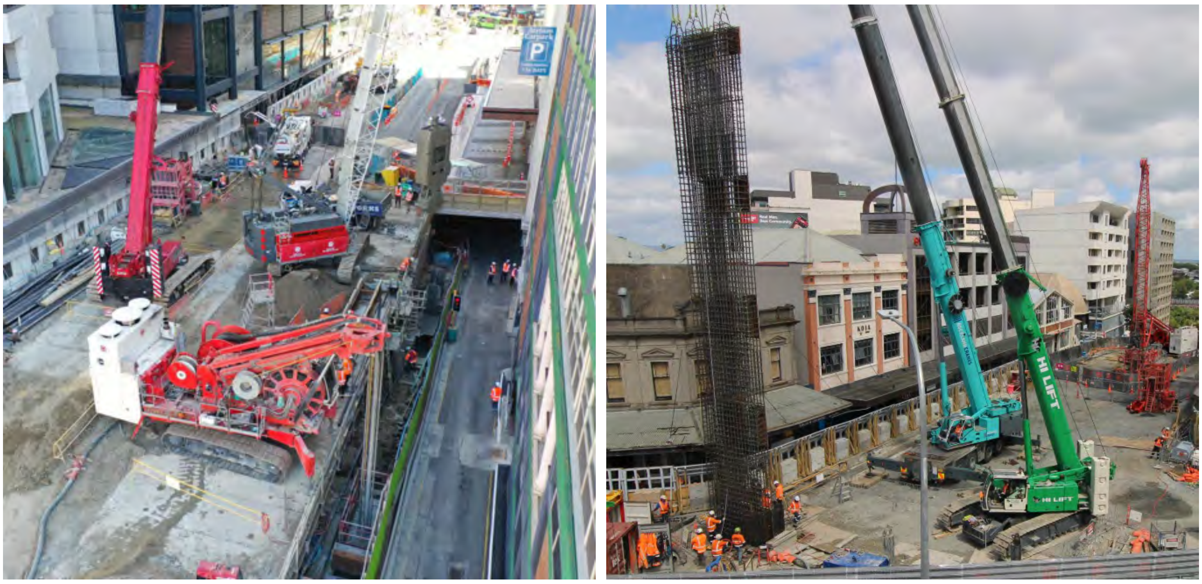

Figure 8: Diaphragm wall digging (using hydrofraise infront & grab behind) and cage lifting

Conclusion

In recent years the retaining wall options available and associated experience in NZ has increased which can lead to better overall project solutions given appropriate selection, design, specialist contractor input and the necessary quality controls. The development of the NZGS SESOC addendum to the ICE SPERWall has resulted in good industry discussion and will disseminate best international practice. This is further enhanced by the formation of the industry Federation of Piling Specialist (NZ) to promote this and associated quality and safety improvements.

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Quantification of Embodied Carbon Benefits of Base Isolation with Supplemental Damping

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ABSTRACT

This paper will present a study of a building life cycle assessment to quantify the embodied carbon benefits of using a base isolation system with supplemental damping. The study considers the comparison of two benchmark buildings on a conventional base isolation system to one with supplemental damping, keeping all other design criteria the same. The study uses a life cycle assessment to quantify impact of the inclusion of supplemental damping from an embodied carbon perspective.

INTRODUCTION

Goal and Scope

This case study uses a 6-storey office building in a high seismic region to quantify the embodied carbon savings in the structure by using supplemental damping at the isolation plane. The assessment compares the embodied carbon of two benchmark buildings (concrete and steel with no supplemental damping) with a comparison building (timber with supplemental damping). The assessment is limited to the structural components (substructure and superstructure) and for life-cycle stages A1-A3. Architectural fit-out, façade and services are not included in the assessment unless specifically noted, but are assumed to be consistent across all three scenarios.

Design Criteria

The building is designed to Importance Level 4 (IL4) loads using a site-specific Probabilistic Seismic Hazard Analysis (PSHA) spectrum, in keeping with industry best practices. A drift limit of 0.5% at Serviceability Limit State 2 (500-year return period) is used.

The proposed use of the building requires future flexibility in the floor plans, which precludes the use of shear walls or braced frames internally to the structure. Therefore, the building is designed with braces in the longitudinal direction and moment frames in the transverse direction.

The main building footprint is approximately 70m by 20m with six storeys above grade. The isolation plane is located just below ground level. There is a one-storey basement below the isolation plane across the full footprint of the building.

Isolation System

When designing base isolation systems, the designer must consider the inverse relationship between accelerations and displacement. Lower accelerations transmitted to the superstructure require larger displacements at the isolation plane, and vice versa. The decision was made that for practical reasons, the design of the isolation system should limit displacements at the isolation plane to a maximum of 1000mm. Larger displacements, while possible to accommodate, become increasingly difficult and expensive to manage with non-structural elements such as rattle space moat lid covers, services connections, and façade joints.

The use of supplemental damping at the isolation plane reduces accelerations transmitted to the building while maintaining the same displacement at the isolation plane. This reduction in accelerations allows the use of a timber superstructure and reduces carbon compared to a conventional isolation system necessitating a steel or concrete solution. Three designs were considered:

- Conventional isolation system, concrete superstructure
- Conventional isolation system, steel superstructure
- Supplementally damped isolation system, timber superstructure

While beyond the scope of this paper, future studies may compare the conventionally isolated buildings with steel and concrete superstructures to supplementally damped versions.

BASELINE ASSUMPTIONS ACROSS ALL THREE DESIGNS

Certain elements of the design were assumed to be similar or the same between all three designs. These include:

- Concrete basement: the basement is assumed to be generally a 450mm thick in situ walls over a 1000mm thick in situ raft slab. A two-bay lift pit is located in the centre of the basement. The reinforcement was assumed to be 180 kg/m^3. The concrete strength was assumed to be 40 MPa.
- Stairs: the stairs are assumed to be steel stringers with cross-laminated timber (CLT) treads and landings. This includes the two main full-height circulation stairs, a feature entry stair, and a basement stair.

- Lift structure: the lift structure is primarily steel hollow sections.
- Façade connections: the façade connections are fabricated steel bracketry.

While the isolators have not been designed in detail and would likely vary in minor ways, they are assumed to have same carbon footprint in all three designs. Associated plate and grout fixings have also been included.

BENCHMARK BUILDINGS

As noted above, the isolation system is designed to maintain a maximum displacement of 1000mm at the isolation plane. Practical limitations on the isolation devices have been ignored for purposes of this study and it has been assumed that a device exists that can satisfy the design requirements. This allows comparisons to be made which satisfy the majority of the intent of a benchmark building while maintaining a consistent reference point.

Benchmark Building – Concrete

The superstructure lateral structure is a concrete moment frame in the transverse direction and a concrete frame with steel braces in the longitudinal direction. The floor is precast concrete rib and infill. The concrete strength for in situ concrete is 35 MPa (columns, beams, concrete capitals), and for precast concrete is 45MPa (rib and infill). The typical floor beams were assumed to be of two sizes (L1-L2, L3-Roof). The braces were assumed to be of two sizes (ground floor-L3, L3-roof). The isolation grillage was assumed to be post-tensioned beams.

Sawn timber for the infill was ignored and there was not a volume reduction on concrete elements for the reinforcing steel. Connections were not modelled but allowances were made in the reinforcing rates. The roof was assumed to be similar to the typical floors. Refer Figure 1 for a schematic representation of this building.

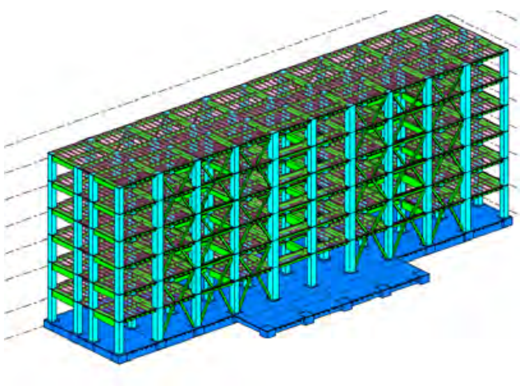

Figure 1. Schematic model of concrete benchmark builiding superstructure

Table 1 summarises the elements used in this design:

Table 1. Elements of concrete benchmark building

Elements	Description	Reinforcing	f'c
Floor System	100mm rib and infill (900crs) with 100mm topping	5kg/m^3 (ribs) 120kg/m^3 (topping)	ribs (45MPa) topping (35MPa)
Columns (Corners)	900x900	250kg/m^3	35MPa
Columns (Interior)	1100x1100	160kg/m^3	35MPa
Ground Floor Beams	1500x800 post tensioned	200kg/m^3	35MPa
Ground Floor Beams	1000x800	200kg/m^3	35MPa
Primary beams – Upper Floors	1200x600 (L1-L2) 900x600 (L3-roof)	250kg/m^3	35MPa
Secondary beams	600x250	200kg/m^3	35MPa
Braces	310UC97 (ground floor-L3) 250UC73 (L3-roof)	-	-
Concrete Capitals	1200dx1600mm SQ concrete capitals (typ) at isolator locations	250 kg/m^3	35MPa

Benchmark Building – Steel

The superstructure lateral structure is a steel moment frame in the transverse direction and a steel braced frame in the longitudinal direction. The floor is slab on metal deck. The typical floor beams were assumed to be the same up the height of the building. The braces were assumed to be of two sizes (ground floor-L3, L3-roof). The isolation grillage was assumed to be steel beams with slab on metal deck.

There was not a volume reduction on concrete elements for the reinforcing steel. Connections were not modelled but a 20% allowance on the steel tonnage was made. The roof was assumed to be similar to the typical floors. Refer Figure 2 for a schematic representation of this building.

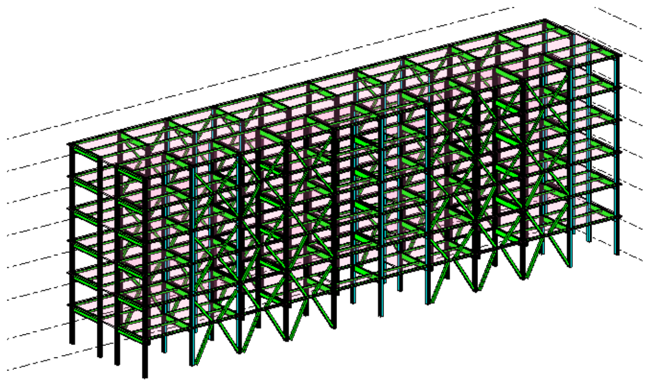

Figure 2. Schematic model of steel benchmark builiding superstructure

Table 2 summarises the elements used in this design:

Table 2. Elements of steel benchmark building

Elements	Description	Reinforcing	f'c
Floor System	Comflor 60, 0.9mm thickness Overall slab thickness 150mm	75kg/m^3	35MPa
Columns	400WC303	-	-
Primary Beams	800WB146	-	-
Secondary Beams	250UB37.3	-	-
Braces	310UC97 (ground-L3) 250UC73 (L3-roof)	-	-

COMPARISON BUILDING

The comparison building is designed with supplemental damping system to reduce accelerations while maintaining the same displacement as the conventionally isolated benchmark buildings. This was assumed to be in the form of eight fluid viscous dampers connecting the ground floor grillage to the basement raft, four in each direction, located at the edges of the building for maximum efficiency.

The reduction in accelerations allowed a timber superstructure to be viable for both lateral and gravity structure. The superstructure lateral structure is a timber post-tensioned moment frame in the transverse direction and a timber braced frame in the longitudinal direction. The floor is un-topped CLT. The typical floor beams were assumed to be the same up the height of the building. The braces were assumed to be of three sizes (ground floor-L2, L2-L4, L4-roof). The isolation grillage was assumed to be steel beams with slab on metal deck.

There was not a volume reduction on concrete elements for the reinforcing steel. Brace and moment frame connections were modelled. The roof was assumed to be plywood over timber beams. Refer Figure 3 for a schematic representation of this building.

No information on the carbon footprint of the fluid viscous dampers was readily available. Therefore, the carbon footprint was assumed to be the same as an equivalent steel tube (S355) sourced from Germany (not necessarily recycled steel).

Due to the acoustic properties of an un-topped CLT floor, allowance was made for a raised acoustic floor across all occupied floors.

The fire protection strategy for the timber elements was assumed to be through an allowance for charring (included in the sizing of the beams, column, and floor for this study), and any additional treatment for stairs or other structure is assumed to be equivalent to the benchmark buildings.

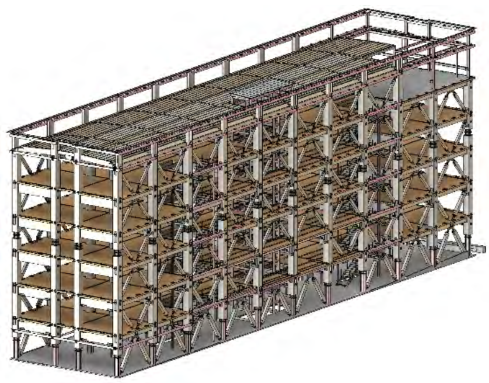

Figure 3. Schematic model of timber comparison building superstructure

Table 3 summarises the elements used in this design:

Table 3. Elements of timber comparison building

Elements	Description	Reinforcing	f'c
Floor System – Ground floor	Comflor 80, 0.9mm thickness Overall slab thickness 195mm	150kg/m^3	40MPa
Floor System – Upper floors	5/170 thk CLT	-	-
Roof	225x660 LVL13 (transverse) 80x360LVL11 (longitudinal) 135x400 LVL11 purlins 19mm plywood	-	-
Columns – ground floor	500WC290	-	-
Columns – upper floors	810x810 GL8	-	-
Primary Beams – ground floor typ	900WB218 (transverse) 900WB175 (longitudinal)	-	-
Secondary Beams – ground floor typ	310UC118	-	-
Primary Beams – upper floors	400x1200 LVL13 (transverse) 400x610 LVL13 (longitudinal)	-	-
Secondary Beams – upper floors	190x475 LVL11 (transverse) 300x475 LVL11 (longitudinal)	-	-
Braces	360x405GL8 (GF-L2) 270x360GL8 (L2-L4) 225x360GL8 (L4-Roof)	-	-

LIFE CYCLE ASSESSMENT

Carbon Quantification Assumptions

Embodied carbon was quantified using available Environmental Product Declarations (EPD) or other rational methods. Phases A1-A3 (cradle-to-gate) were considered. Only primary and secondary structural elements were included in this assessment. Architectural fit-out, façade and services are not included in the assessment but are assumed to be consistent across all three scenarios unless specifically noted otherwise. The functional unit is the building's floor area in meters squared. Table 4 summarises the key elements and their assumed carbon density for both Global Warming Potential (GWP) and Biogenic (carbon stored in the timber).

Table 4. Material carbon intensity assumptions

Material	GWP ($kgCO_2$-eq per kg)	Biogenic carbon ($kgCO_2$-eq per kg)	Source
Concrete (in situ) 30 MPa	0.11	0	Industry average for Ordinary Portland Cement only, no cement replacement considered
Concrete (in situ) 35 MPa	0.12	0	S-P-02050 FIRTH CERTIFIED READY-MIXED CONCRETE - Plant Wellington Belmont
Concrete (in situ) 40 MPa	0.10	0	Higgins Datasheet
Concrete (in situ) 40 MPa - Columns	0.12	0	Higgins Datasheet
Concrete (precast) 45 MPa	0.20	0	(NZGBC material) PR_20_31_16_7_3_1_1 Reinforced concrete, 45 MPa, precast, inc. 50 kg/m³ steel reinforcing, (OPC) with reinforcement removed *70% GWP_ff
Grout	0.49	0	Sika Grout 212 Datasheet
Reinforcement	3.95	0	S-P-01002 PACIFIC STEEL™ Steel Reinforcing Bar, Coil, Rod and Wire (EN 15804:2012+A2:2019)
Structural steel Welded sections	2.86	0	S-P-00559 Steel – Welded Beams and Columns (EN 15804:2012+A2:2019)
Structural steel Open sections	3.32	0	S-P-01547 Version 1.2 Hot Rolled Structural and Rail
Structural Steel Hollow steel section	2.64	0	Default based on BlueScope XLER plate EPD.
Steel deck	3.19	0	(NZGBC material) PR_25_71_51_89_1_3_5 Steel, primary (galvanised finish, coating class Z275), cold rolled profile metal sheet, trough section 56mm deep at 305mm ctrs, 0.95 BMT
Engineered Timber CLT	0.15	-1.54	Red Stag H1.2 treated
Engineered Timber Glulam	0.28	-1.63	WPMA glulam H1.2 treated
Engineered Timber LVL	0.17	-1.51	CHH LVL H1.2 treated
Engineered Timber Plywood	0.36	-1.60	CHH untreated plywood
Sawn Timber	0.15	-1.65	Assumes an average of industry values for sawn timber, ungauged, kiln dried and treated to H1.2.

An allowance of 52 $kgCO_2$-eq /m^2 was made for the raised acoustic flooring, assumed to be 600x600mm raised floor panels (EPDITALY0673 - Twin Floor raised floor from Nesite). An allowance of 4,604 $kgCO_2$-eq/unit was made for the fluid viscous dampers.An allowance of 8,130 kg CO_2-eq/unit was made for the base isolators based on an average weight of 2,070 kg/unit and a spherical bearing EPD (HUB, HUB-2481 - RESTON SPHERICAL bearing from Mageba Services & Technologies AG).

Results

For the following charts, the whole building includes the basement and is 8,200 m^2 gross floor area. The superstructure includes the structure above the isolation system (ground floor and elevated floors) and is 7,300 m^2 gross floor area.

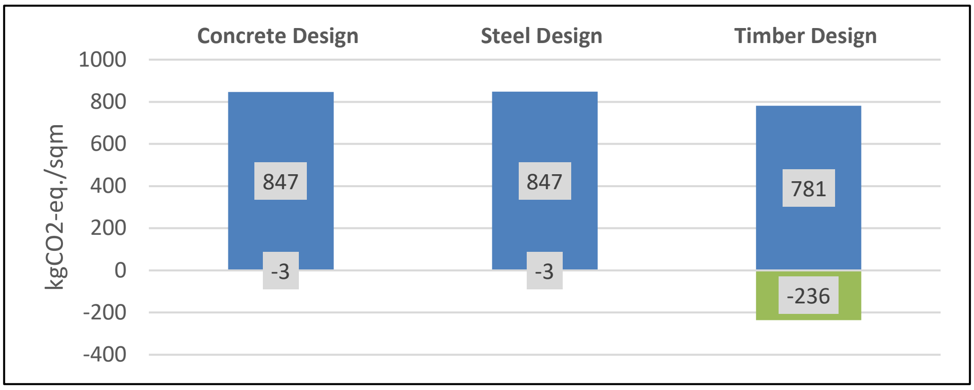

Figure 4. Whole Building Carbon Emissions (biogenic emissions indicated in green)

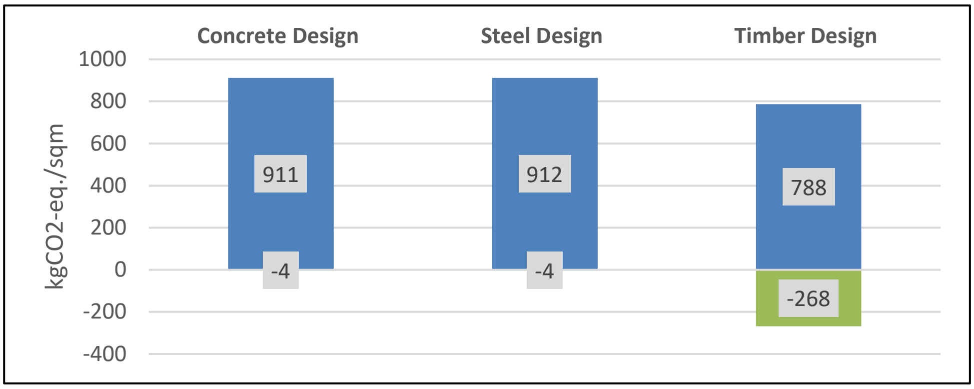

Figure 5. Superstructure-only Carbon Emissions (biogenic emissions indicated in green)

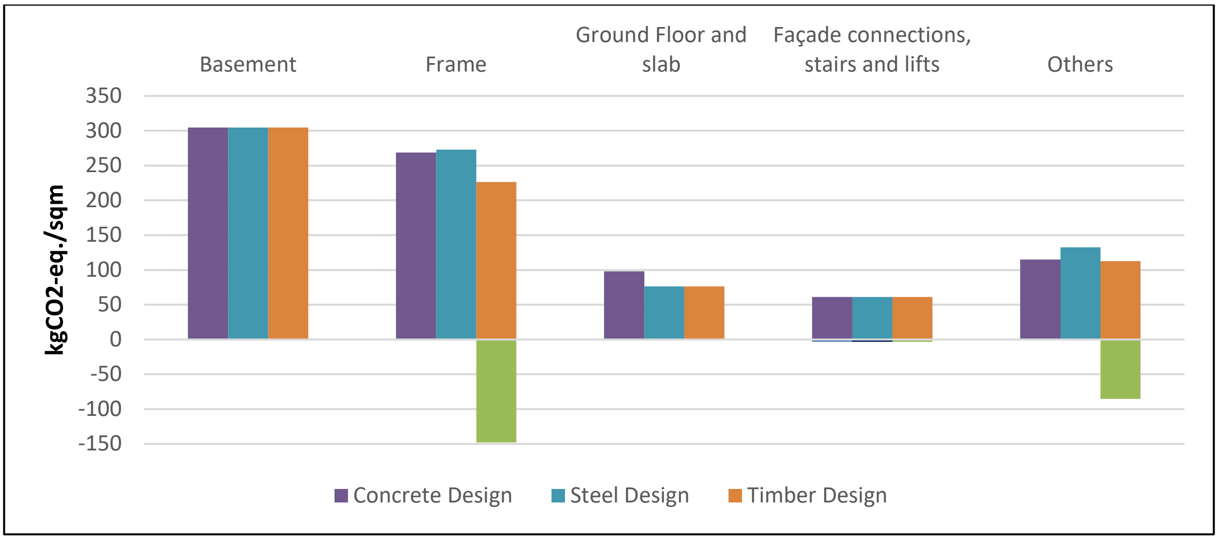

Figure 6. Top 4 Categories (Whole Building), plus rest included in “Others” (biogenic emissions indicated in green)

“Others” includes all other measured items such as the upper floors, isolators, acoustic floor, the dampers and the roof (listed here in order of relative contribution).

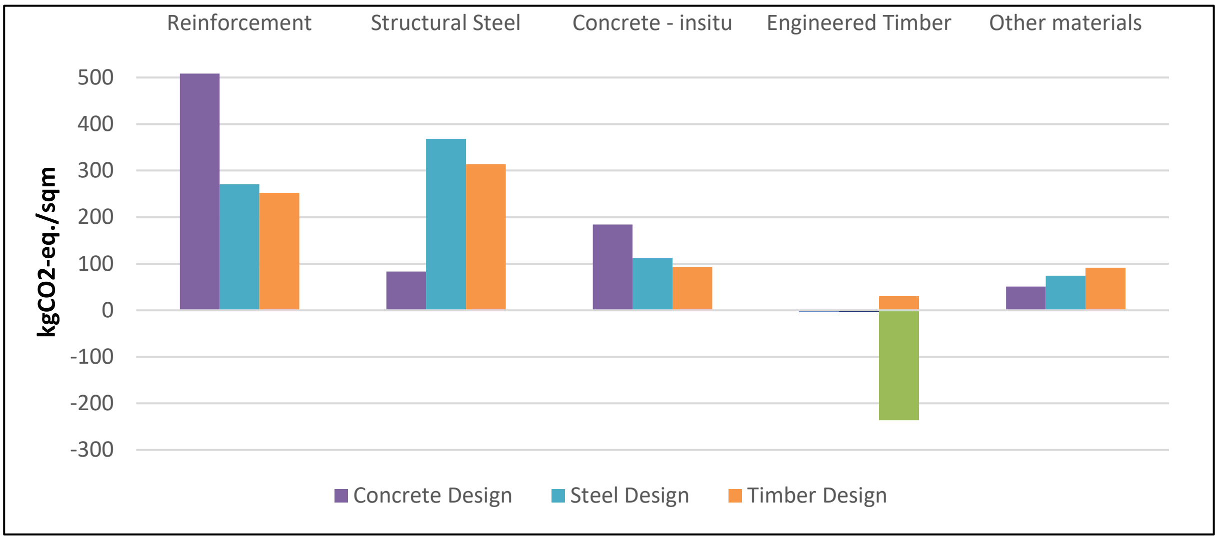

Figure 7. Relative Material Contribution (Whole Building) (biogenic emissions indicated in green)

Based on the RICS methodology (Sturgis et al 2023), contingency should be applied to the calculated GWP based on the stage of design and level of accuracy of the material and carbon

data. Based on this, we have estimated the contingency factor of 20% for the Benchmark buildings (concrete and steel) and 11% for the Comparison building (timber).

CONCLUSIONS

Reviewing the overall GWP carbon, the supplementally damped structure achieves approximately 10-15% reduction in carbon emissions compared to both benchmark buildings before biogenic carbon is allowed for, refer to Table 5. With biogenic carbon emissions included, the reduction in GWP is approximately 35-45% compared to the benchmark buildings.

Table 5. Reduction in carbon

Benchmark building	Reduction in GWP with Supplemental Damping (excl biogenic offset)		Reduction in GWP with Supplemental Damping (incl biogenic offset)	
	Whole building	Superstructure	Whole building	Superstructure
Concrete	8%	14%	35%	43%
Steel	8%	14%	35%	43%

The assumed acoustic floor treatment is a substantial contributor to the timber building's embodied carbon. If this is removed, the timber building superstructure achieves approximately 20% reduction compared to the benchmark buildings. However, this would potentially result in a lower quality space and would need to be accepted by stakeholders.

Figure 7 shows that even in a "timber" building, a large quantity of steel is still required for connections and lateral system ductility, only approximately 15% less than an entirely steel frame. This may be partially due to the relative level of development of the models used to quantify the elements, however effort was made to account for this.

The carbon footprint of these designs is relatively high, however this is largely due to the high seismicity of the site and the IL4 requirements for the building.

Future studies of this building may include a comparison of a concrete and/or steel superstructure on the supplementally damped isolation system, however this comparison was beyond the scope of this study.

ACKNOWLEDGMENTS

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Utilising a dFMA approach to deliver 42 teaching spaces, across Auckland in six months

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ABSTRACT

The Short Term Roll Growth (STRG) project for the Ministry of Education (MoE) aims to address the need for teaching spaces as student roll sizes grow across the country over the next one to five years.

This paper presents the structural design approach Preformance undertook in collaboration with its partners, Crown Infrastructure Delivery (CID), Southbase Construction, Builtsmart, Portacom and A-line to masterplan, design and deliver 42 teaching spaces across nine different schools within six months for the Budget 25 (B25) program for Auckland. With an approach centred on speed and efficiency, the project utilised a repeatable, modular design. A design for manufacture and assembly (dFMA) approach was adopted by the project team, exploring what work could be done prior to the buildings being delivered via truck or crane.

Circularity was a key driver for CID, thus the design solution needed to be adaptable to allow for future relocation. Through market research, Preformance optioneered different foundation solutions that focused on transport, the environment, and speed of construction, without compromising on health and safety. A steel micro piled solution was used.

Preformance's approach on the Short-Term Rapid Roll Growth (STRG) project reflects a commitment to pragmatic and collaborative delivery and optimising resources while maintaining quality and safety. This project underscores the vital role of structural engineers in collaborating with all stakeholders to deliver successful results for Aotearoa.

INTRODUCTION

As part of MoE's Roll Growth program, the STRG project delivered 42 teaching spaces across nine school sites in Auckland in six months from project inception to handover. Delivering this volume of buildings within MoE's timeframe was achieved through several strategies described in this paper, including use of offsite manufactured buildings (OMBs), dFMA approach, repeatable designs, taking a programmed approach, and early contractor involvement under a design build model.

MoE has highlighted the importance of utilising OMBs for “achieving better value for money” as outlined in their ‘Value-for-money reviews of school property’ (*MoE, 2024*). The traditional approach for these OMB classrooms is for a school to independently engage with an MoE appointed OMB supplier. However, by taking a programmed approach with multiple sites under a single contract, this allowed for the project team to remain consistent, enabling greater quality and speed on the project.

As project managers, CID engaged the modular suppliers Builtsmart, Portacom, and A-line to either provide newly manufactured, or relocate existing, teaching spaces. Southbase group provided Structural Engineering, Architectural, and Design Management services on the project with dFMA principles in mind. This ensured the design considered OMB delivery specifics for building delivery to site and was adaptable to different OMB suppliers and their logistics. Southbase Construction were engaged as design build contractors and were involved early in the project, enabling site planning for OMB delivery methods, early procurement, and overall de-risking the project to deliver within the timeframe.

PROJECT AND CLIENT’S REQUIREMENTS

Efficiency and quality were key drivers to deliver the 42 teaching spaces in the given time frame. Additionally, MoE’s specific project requirements included circular principles including designing for disassembly and allowing for flexibility in relocating classrooms to different school sites in response to future roll changes. Furthermore, aspects of construction would take place during live school environments, thus minimal time on site and health and safety were key factors in the design approach.

PROJECT SOLUTIONS AND APPROACH

dFMA approach

The use of modular classrooms and OMB’s steered the design approach towards dFMA. The project program was coordinated with the OMB suppliers so that the buildings were delivered to site as soon as they were manufactured, avoiding the need for temporary storage. Furthermore, the design considered how these modular buildings would be delivered and lowered into place and how they may be removed and relocated in future, which drove the decision to use micro steel micropile foundations.

Micro Piles

Prior to the selection of proprietary micro steel pile foundations (Surefoot steel micropiles), several piling solutions were optioneered including screw piles, driven timber piles and bored timber piles. The alternatives required installing piles after the building was delivered, which involved operating a small drilling rig under the building. This added time to the project and increased health and safety risks due to working under a lifted load for an extended period.

Screw piles required pile testing, which had logistically challenges getting a piling rig onsite, across live environment schools prior to site establishment by the construction team.

Using Surefoot proprietary footings permitted the micro piling before the delivery of the OMB buildings. Firstly, the OMB buildings are delivered on temporary supports. Then, the posts and subfloor connections are installed by hand prior to the building being lowered on the foundations, thus avoiding the need for heavy machinery under the building. Additionally, the pile placement considered OMB logistics for buildings that were delivered via vehicle with pile spacing to avoid OMB truck axles as shown in Figure 1 below.

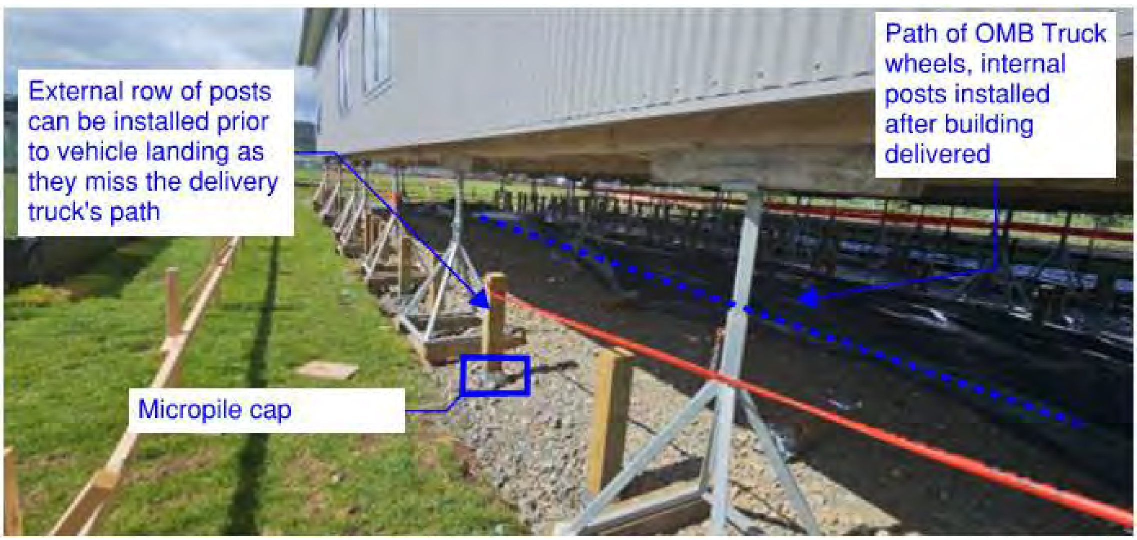

Figure 1 Modular building landing on temporary supports with micro steel pile and pile posts underneath

Consistency

The dFMA approach also extended to other design components of the project, including modular timber decks, proprietary barriers and handrails, and timber canopies. These modules can be tailored to meet different site or OMB requirements. The modularity of design components resulted in the design and construction team staying consistent throughout the nine sites.

Figure 2 - Proprietary Moddex barrier and handrails installed on stair

Figure 3 - Deck Module constructed off site prior to installation

Managing Contractor Model

Southbase Construction employed a ‘managing contractor model’ which meant as contractors, they led the delivery of the project from the outset, with CID as project managers. Their input included verifying that building placement locations were feasible for the OMB delivery method, modular components such as the handrails could be procured, and gave program certainty.

SOLUTION IMPLICATIONS

Project Efficiencies

The dFMA approach and consistent team enabled various project efficiencies to take place and was a crucial aspect in successfully delivering against the client’s timeframe and budget.

With a repeatable modular design, the design team were able to minimise design errors and turn over their design quicker with each site, iterating the design with adjustments for certain site specifics. Consistency of the design team also developed synergies as the project progressed, with roles and scopes staying consistent across the project. All the designs occurred concurrently, adopting a three-phase staggered approach which meant that from start of design to completion took a total of six weeks to complete nine sites. This tight timeline was only achievable through adopting a repeatable design.

The efficiencies of repeating modular design similarly paid off with the construction and procurement team, which were able to learn and adapt from each site. This meant less time on site and disruption to school activities and personnel. Supplier engagement was streamlined with each site, with faster and more accurate pricing being delivered. Due to the modular design, only minor adjustments were needed after initial pricing. The construction team were also able to learn and adapt with each site, taking lessons learnt from each and applying to the next site. As a testament to this, the design team received an average of 13 RFIs per site, mostly being related to items that required to be confirmed onsite.

This efficient approach is evident on site, where OMBs from the 2024 MoE STRG project were still being installed, while OMBs from this B25 project were completed and handed over in an average of 62 days.

Circular and Sustainable Principles

At three out of the nine sites, the design team were able to accommodate relocation of existing modular buildings to be moved from different schools. This recognises adaptability and resilience of the project approach, with significant carbon and cost savings realised by reusing existing buildings instead of demolition and building new when not required.

Figure 4 - Existing A-line classroom at Waiheke High School being prepared for relocation to Massey High School

Potential carbon savings from relocation of all other new OMB buildings is also achieved through the design, as disassembly and reassembly of modular timber deck, handrails, and canopy framing is simpler, faster, easier to transport, and results in less construction and demolition waste. The OMB buildings and their modular components can be removed from the pile posts and relocated to other school sites to meet changing school enrollment needs as required by MoE.

LESSONS LEARNT

Existing Underground Services

Greater emphasis and program allowance for early site information such as underground services may have been beneficial to the project. At two locations, underground services were identified during piling installation that differed from the existing site information. This meant pile bridging and building placement decisions were made with input from the contractor informing what was found on site. The design build model of Southbase Construction enabled this process to be much more efficient and flexible with solutions being developed onsite at real time, between the surveyor, main contractor, sub-contractor and structural engineers.

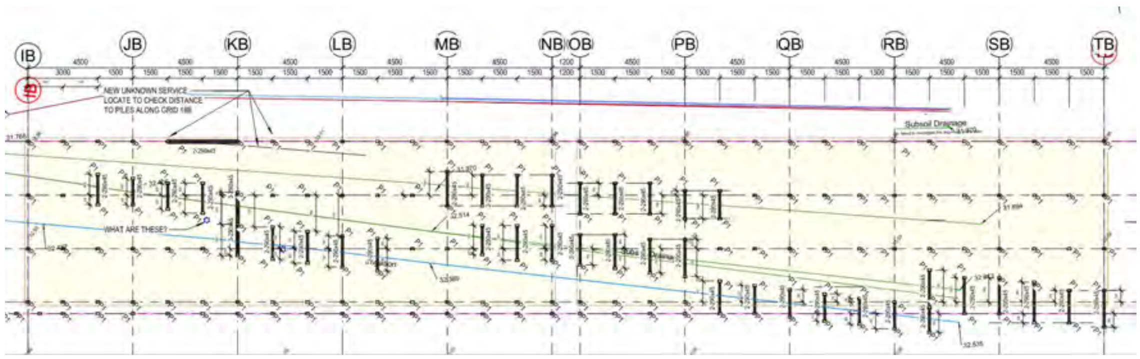

Figure 5 - Example of extensive pile bridging required at one site to avoid existing underground services

Topography surveys

To further streamline the project, topography surveys and existing design data from the previous years' (B24) MoE STRG project was used at the first school sites. However, construction on site revealed inaccuracies, causing delays to fix level misalignments. Following this, a topographic survey was always conducted for all subsequent sites following the initial site walkover. This practice ensured more accurate design and reduced construction time.

Misalignment and set out

There were issues with accuracy of grid set out and truck delivery of OMB modules at two of the sites, which meant that when the OMB buildings landed, the building was not fully flush with the pile posts. The contractor had to adapt on site and install additional timber framing as instructed by the structural team to accommodate for these. A lesson learnt is to program and design for greater tolerance around OMB delivery, particularly with truck delivery where the accuracy of building alignment can be limited by the vehicle.

CONCLUSIONS

In conclusion we believe that the B25 STRG program of works (42 teaching spaces across nine sites in Auckland) could only have been delivered on-time, within six months through the structural engineer working collaboratively with the contractor and their subcontractors through utilising a dFMA approach, making components modular and repeatable and allowing for transport, delivery and disassembly (if required).

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Seismic Performance of Concentrically Braced Frame with Friction Sliding Connections: Learning from Subassembly Component Tests, Numerical Simulations and Shake Table Tests

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ABSTRACT

Concentrically braced frames (CBFs) are one of the common systems to provide load resistance for structures under lateral loads. However, when loaded beyond the elastic range, the brace itself is prone to buckling under compressive axial force and will result in unequal forces in the braces under tension and compression. This is incorporated into current design procedures, but results in reduced performance in severe earthquakes. Several low damage design concepts have been developed and implemented in the recent years to overcome this brace buckling issue. One of these is the use of friction-sliding connections (FSCs) as the dissipative component. CBFs with friction sliding dissipative mechanisms can exhibit inelastic deformation capacity through friction sliding in both tension and compression along the line of the brace, with forces limited to avoid section or member buckling and non-ductile fracture of braces. In a research programme to determine the brace and system behaviour, quasi-static testing on a subassembly of brace-to-gusset plate connection with FSCs was first undertaken to verify its adequate deformation and resistance. The obtained hysteretic characteristics were then adopted in SAP 2000 for numerical simulation of a three-storey CBF. Finally, shake table testing of the three-storey CBF was conducted, including V-bracing and diagonal bracing configurations. This paper reports the key findings from the preliminary design, numerical

simulation, and experimental testing and provides guidance on how to design these systems using current CBF design procedures with appropriate modifications.

Concentrically braced frames with friction-sliding connections

In a concentrically braced frame (CBF), the members are subjected primarily to axial loads. When a conventional CBF responds inelastically to seismic loading, the inelastic demand is concentrated into the braces. Therefore, the inelastic behaviour of the CBF system is very dependent on the effect of inelastic demand on the braces. Inspired by the principle of friction brake pads, in the late-1970's and early-1980's, Pall friction dampers (PFDs) were developed as the pioneering system for controlling the seismic response of buildings through friction devices (Pall, 1979) reducing the initial cost of construction while significantly increasing the earthquake resistance against damage (Pall, 2004). More recently, for enhanced seismic resilience and increased focus on sustainable design, friction devices in different forms have been introduced into braced frames. These devices are designed to be activated before other elements of the structural system, dissipating energy and providing deformation capability while limiting the forces in the protected elements. Some examples used in industry include yielding restrained brace (YRB) equipped with Ten-Co seismic brake from QUAKETEK (2020), DMAX brace with damage free friction joint (DFFJ) from TECTONUS (2023) and the brace installed with a rotational friction damper from DAMPTECH (2021), as shown in Figure 2.

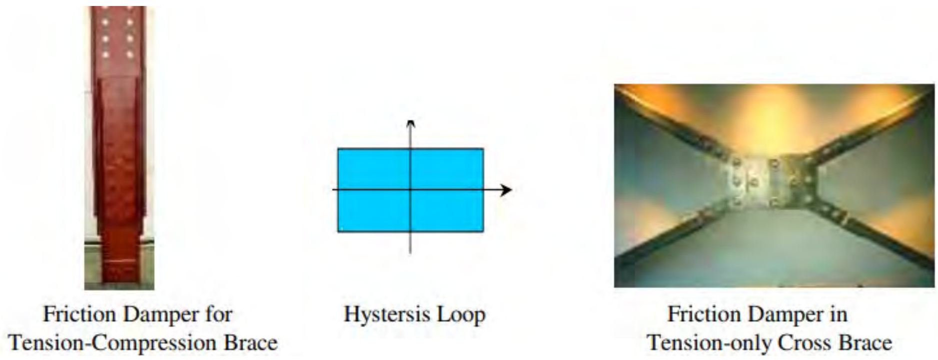


Figure 1: PFDs (Pall, 2004)

(a) (b) (c)

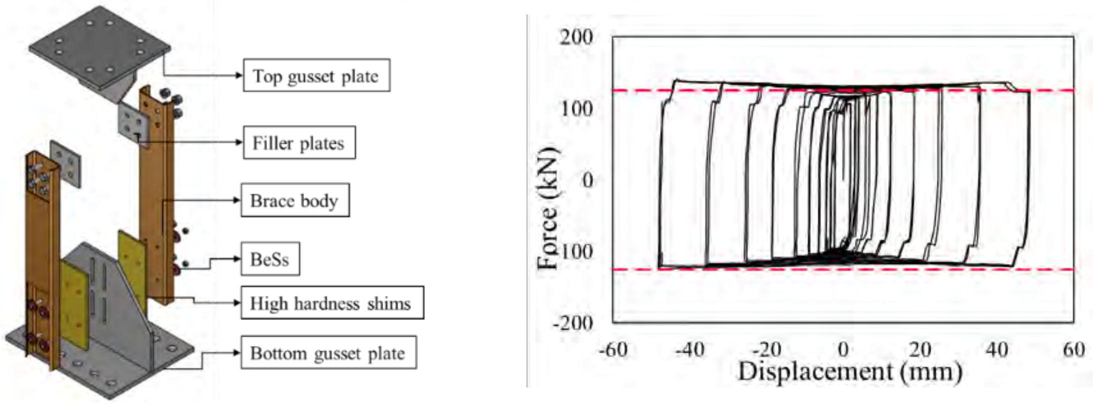
Figure 2: Products from (a) QUAKTECH, (b) TECTONUS and (c) DAMPTECH

General guidance for design and implementing CBFs with friction sliding connections (FSCs) is being included in the next revision of New Zealand Steel Standard (NZS3404), due for public comment release around mid-2025. Currently, the suite of New Zealand standards does not contain end-to-end design methods for CBF with friction dissipative mechanisms that are sufficiently prescribed to allow their inclusion as part of a verification method.

As part of the New Zealand China international collaborative project, RObust BUilding SysTem (ROBUST) Programme (MacRae et al., 2024), experimental testing on FSCs has been undertaken at component level as well as whole of building level on different structural systems including CBFs. This paper reports the key findings and issues from the preliminary design, numerical simulation, and experimental testing.

Component test at brace-to-gusset plate assembly level

The examples of the friction devices mentioned in the previous section include the FSCs as a separate device installed in line with brace. In contrast, in this study, the inclusion of the FSCs in a brace is accommodated in the brace to gusset plate connection, replacing the traditional non-sliding brace to gusset plate connection. Adapting from previous research on friction sliding braces (Ramhormozian et al., 2017), a symmetric friction connection (SFC), a type of FSC, was designed, detailed and installed at the brace to gusset plate connection with partially deflected Belleville Springs (BeSs) and the bolts installed in the elastic range, noted as SFCBeSs. This means long slotted holes are provided on the gusset plate at one end of the brace-to-gusset plate assembly, designated as the sliding end, while the other end with standard holes is the non-sliding end (see Figure 3(a)). Full-scale component tests were conducted to obtain the design strength reduction factor and the overstrength factor. The hysteretic response is shown in Figure 3(b), exhibiting a stable rectangular hysteretic shape, where the red dashed lines indicate the nominal strength. Directly from these component tests, the strength reduction factor is determined as 0.85 and the overstrength factor is 1.15. Due to the limited number of component tests, it is recommended that the design strength reduction and overstrength factors should be 0.80 and 1.20, respectively. More details and results from this testing are reported by Yan et al. (2025a and 2025b).



(a) (b)

Figure 3: Illustration of (a) brace-to-gusset plate assembly and (b) hysteresis loop (Yan et al., 2025a)

Numerical simulation at system level

The V-braced CBF system with bracing effective in tension and compression was designed and detailed in the transverse direction of a three-storey building, as shown in Figure 4(a). SFCBeSs was used at brace-to-gusset plate connection. More details of the three-storey building are reported by Yan et al. (2023). A capacity design procedure was followed, with SFCBeSs being the dissipating system accordance with HERA R4-76 (Feeney and Clifton, 1995/2001) and Cl. 12.12 of NZS 3404 (2007). The stiffness and strength of the brace was decoupled with inelastic behaviour occurring at the joint and the brace remaining elastic. Therefore, the C_s factor, taking account for less satisfactory inelastic behaviour of the CBF systems due to compression brace buckling, was be taken as 1.0. The brace size was first selected to control the required seismic lateral deflection, then to check against the overstrength action of the SFCBeSs. In addition, as the brace shall not buckle with the presence of SFCBeSs, the difference between overstrength capacity and the design capacity of the SFCBeSs was used to determine the net vertical seismic force applied to the collector beam for each considered level at the brace/ collector beam joint. This is an important requirement to suppress collector beam plastic hinging.

Numerical simulation in 3D was conducted by Yan et al. (2023), taking the brace with SFCBeSs as a two-joint link element with uniaxial plasticity using plastic (Wen) model in SAP 2000. Three records were selected and scaled as recommended by Oyarzo-Vera et al. (2012) for the North Island of New Zealand. The damping type considered for the nonlinear time-history analysis is the Rayleigh-type proportional damping, considering 5% of critical viscous damping for the first two modes. Results from non-linear time-history analysis showed a ratio of average base shear under maximum considered earthquake (MCE) excitation to that under design level ultimate limit state (ULS) excitation was 1.25. Moreover, the MCE to ULS drift is 1.5. The average maximum residual drifts measured from roof level under MCE (0.07%) was found to be less than that under ULS (0.11%), below the 0.14% limit reported by Clifton (2013).

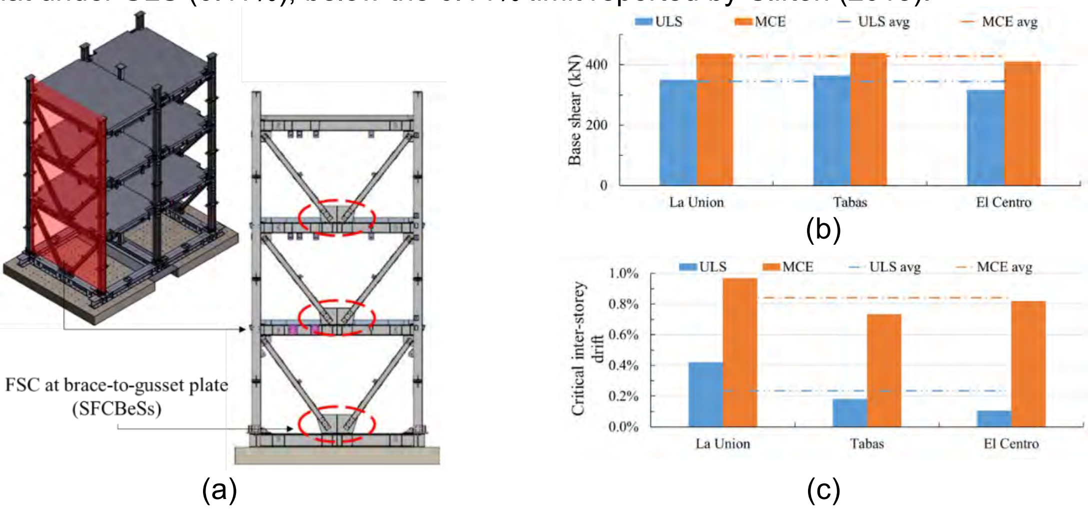
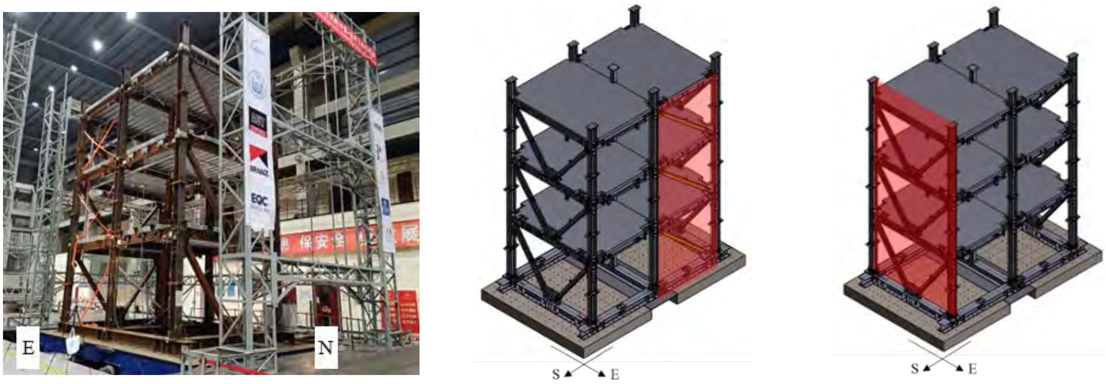


Figure 4: Illustration of (a) consdiered structure, maximum and average of maximum (b) base shear and (c) critical inter-storey drift (Yan et al., 2023)

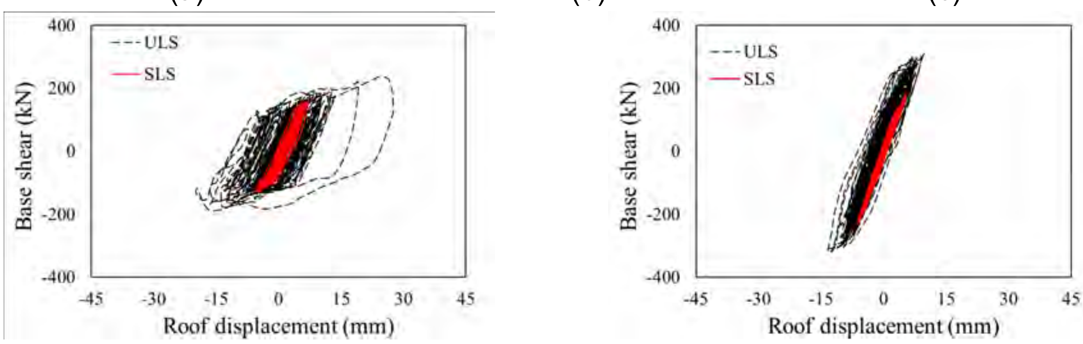
Shake table testing at system level

Shaking table testing was conducted of a near full scale three-storey steel frame composite floor building, with 3 m inter-storey heights and a centreline plan dimension of 7.25 m by 4.75 m. The isometric view of the tested building is shown in Figure 5(a). Two bracing configurations were tested using single diagonal bracing as shown in Figure 5(b) and V-bracing as shown in Figure 5(c). Both configurations were designed with SFCBeSs at the brace-to-gusset plate connection, dissipating energy through frictional sliding under design level ULS events.

The base shear response under design level service limit state (SLS) and ULS are shown in Figure 5(d) and Figure 5(e) for diagonal bracing and V-bracing configurations, respectively. For the V-bracing configuration, the recorded ULS base shear was 321 kN very close to the average base shear obtained from numerical simulation of 344 kN in Figure 4(b). The roof residual drift was found to be 0.02%, less than that predicted in the numerical model of 0.11%. The diagonal bracing configuration experienced more displacement in the positive direction (South), as shown in Figure 5(d). However, this didn't result in a high residual drift, as the roof residual drift under ULS shaking for the diagonal brace configuration was almost zero, 0.005%. For both bracing configurations, there was no change in fundamental period and no visible damage observed at the conclusion of the tests.



(a) (b) (c)



(d) (e)

Figure 5: Illustration of (a) tested structure and (b) diagonal bracing configuration, (c) V-bracing configuration, base shear response of (d) diagonal bracing and (e) V-bracing configuration (Yan et al., 2025a)

Summary

This paper presented the key findings from the preliminary design, numerical simulation, and experimental testing of CBF with FSCs. Overall, the performance of the tested three storey CBF was satisfying, providing reliable and adequate deformation capacity and resistance at both component and system levels, showing the low-damage characteristics. The non-linear deformation of the system occurred through friction sliding along the line of the brace, with the brace being designed for the overstrength actions from the FSCs, keeping the brace elastic and non-buckling. The experimental recorded base shear was found to be close to the average base shear from numerical simulation under design level. However, it was found that the residual drift recorded from shake table test was much less than that estimated from numerical model (0.02% comparing to 0.11%). FSCs can be incorporated into CBFs based on the current design provisions with careful consideration on load path and connection detailing. A detailed worked example has also been prepared and is presented in an international journal article (Yan et al., 2025b) that is currently under review at the time of writing this paper. Interested readers are encouraged to consult the article once it becomes available.

Parametric studies are underway to study and quantify other key design parameters, for example distribution of lateral resistance, height restriction and bracing configuration, which targets to relax the current limit for applications in height and number of storeys.

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Aspects of Design and Construction for the Skyline Queenstown Redevelopment

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Abstract

To ensure the longevity of the Skyline Queenstown Gondola tourist attraction and provide a world class experience, Skyline Enterprise have committed to a complete replacement of the gondola and associated buildings. Considerable planning has been done on the positioning and design of the new buildings to be sympathetic to the ongoing operation of the existing facility.

This paper presents the unique engineering challenges and solutions implemented across the three major project phases: new bottom and top terminals, a building extension, and replacement of the main building. As this project is still under construction, this paper covers the overall design philosophy and the construction to date.

Introduction and Project Overview

Skyline Enterprise operates the tourist attraction on Bob's peak to the northwest of the Queenstown township. The existing facilities need upgrading to satisfy the requirements of the growing tourism attraction. An increased capacity gondola is also required to support future business growth.

The project includes:

- Replacement of the existing gondola with a new higher capacity gondola (by Doppelmayr)
- A new Bottom Terminal building
- A replacement and extension of the Top Building, including the upper gondola terminal.

The design and construction of these buildings have had to deal with some interesting and unique constraints. This includes extreme wind loading, amplified seismic loading, a steep and confined construction site and building around a live tourist operation.

Figure 1: Skyline Redevelopment site plan at Bob's Peak, northwest of Queenstown

Key Project Drivers

This is an owner-occupied facility, and therefore there was a strong future focus, delivering a well thought out facility to stand the test of time. To do this, some of the key objectives were:

- Upgrade of the current facilities to provide a world-class tourism experience.
- Minimise business disruption by staging construction work around the existing operation. This included staged construction programmes for both the Top and Bottom Buildings, and concurrent construction of the new gondola towers around the operational gondola line.
- Build a robust, low-damage facility to minimise business disruption in the longer term, in recognition of the high likelihood of the Alpine Fault rupturing during the building's life.
- Create large open-plan spaces and form a terraced restaurant floor to maximise the views.

Adoption of Seismic Topographic Amplification Factor

The phenomenon of topographic amplification is when the ground profile results in focusing and polarisation of seismic waves at the location of ridges or cliff crests. The top building is located on a steep hillside and a ridgeline. Being positioned high above the lowland of Queenstown, higher seismic accelerations are expected.

This topographic seismic amplification was recognised by the geotechnical engineer to apply to this site and recommended it was incorporated into the seismic loading for structural design of the Top Building. The amplification is not a recognised requirement of the Building Code (NZ

BC 2011), but Skyline Enterprise elected to adopt this additional factor in response to their desire to operate a robust building and reduce the magnitude of business disruption after a major earthquake. A topographic amplification factor of 1.7 for seismic loading was proposed by the geotechnical engineer and adopted for the structural design.

A Very Exposed Wind Site

Due to the exposed nature of the site on a prominent and exposed ridge, the design wind loads for the Top Building are significant. The topographic multiplier (M_t) results in wind loads considerably higher than an average building on the flat in urban Queenstown. High wind pressures require a stronger building envelope. The high wind loads governed the design of the exterior elements including the roof structure, external walls and lateral design of the terminal portal frames. The glazing also needs to be capable of resisting the high wind pressures. Given the cost implication of designing for excessive wind pressures and the exponential increase in pressure relative to wind speed, we carried out additional investigations early in design to ensure we were designing for the appropriate level of risk for the new building.

The design wind loads calculated in accordance with AS/NZS1170.2 (Standards New Zealand, 2011 – current at the time of design), resulted in an Ultimate Limit State design wind pressure of just under 3kPa, equating to a wind speed of 250 km/h. This is on par with the highest recorded wind gusts in NZ which are 270 km/h in Wellington, 1968 (NIWA 2008) (Resulting in the Wahine Disaster). Followed by 250 km/h in Mt John, Canterbury, 1970 (NIWA n.d.). In 2016 the brand new Remarkables Ski Field Base Building was subjected to alleged ~200km/h wind gusts, smashing a number of the large glazing panels (Williams 2016)

Technical research papers were explored relating to assessment of wind loads over complex topography (Ruel, 1998) (Walmsley, 1988). A 'steep terrain' multiplier is typically in the order of 1.75 - 2.1 for sites located near the crest of a hill. This is generally in agreement with the calculated terrain multiplier of 1.79 calculated for this site using AS/NZS1170.2.

Three years' worth of site recorded wind speed was available from the Skyline operations team and a nearby weather station at G-Force paragliding. The highest recorded wind gust over the three-year period was 108 km/h. We engaged NIWA (National Institute of Water and Atmospheric Research) for a second opinion. Unfortunately, the available data was insufficient to correlate against other long-term records in the Queenstown region to produce reliable long-term predictions. Further options included computation fluid dynamic modelling, and installation of measuring equipment at the site, to expand on the data set. NIWA confirmed the design pressures calculated using AS/NZS1170.2 were appropriate.

The investigations outlined above did not provide a conclusive outcome on the potential conservatism of the code derived wind pressures, but indicated that any significant reduction to the code load was unlikely. It was decided to not pursue any further refinements in the design wind loading such as wind modelling, therefore code derived actions were adopted.

Top Building

Building Description and Design Philosophy

The new Top Building will replace the existing building at the top of the gondola. The new building will be a four-storey structure, with a top floor plate of approximately 4000m2. The building is stepped into the hillside to optimise the floor area against the existing topography.

As each floor slab butts up against the hillside, each level is able to be supported laterally, by a direct connection into the hillside, rather than requiring a more traditional vertical bracing system. Each floor plate acts independently to transfer lateral loads to a rigid rock connection, along the north side of the building. The building's foundations are anchored into the schist rock hillside with high strength steel rock anchors as shown in Figure 2.

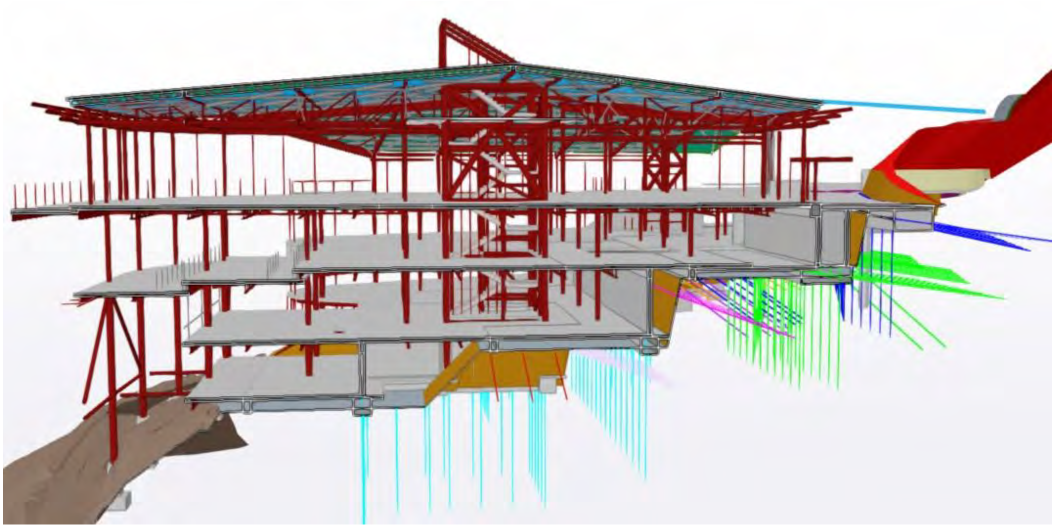

Figure 2: Section through the new Top Building showing rock anchors

The Top Terminal is part of the Top Building and is located at the west end, behind the original Top Terminal. The Top Terminal varies in structural form to the main building as it includes a double height open space housing the Doppelmayr gondola system.

Construction Staging

The construction of the entire redevelopment is staged to minimise disruption to the operation of the existing facility. The construction of the Top Building is split into three main phases as shown in Figure 3.

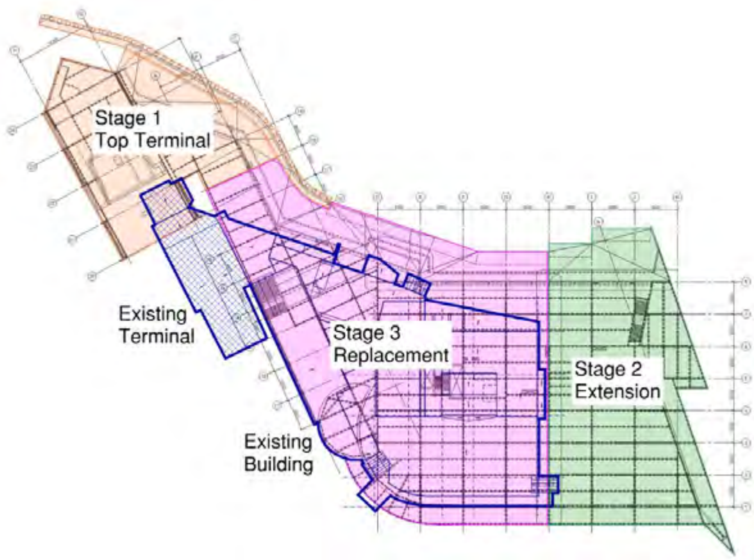

Figure 3: Construction staging of the new Top Building

The Top Terminal (Stage 1) was strategically located behind the existing gondola terminal which allowed it to be constructed in full, with limited impact on the existing operations. Once commissioned, the Top Terminal was linked up to the existing building ahead of the gondola system changeover.

Figure 4: Construction of the new Top Terminal behind the existing Top Terminal

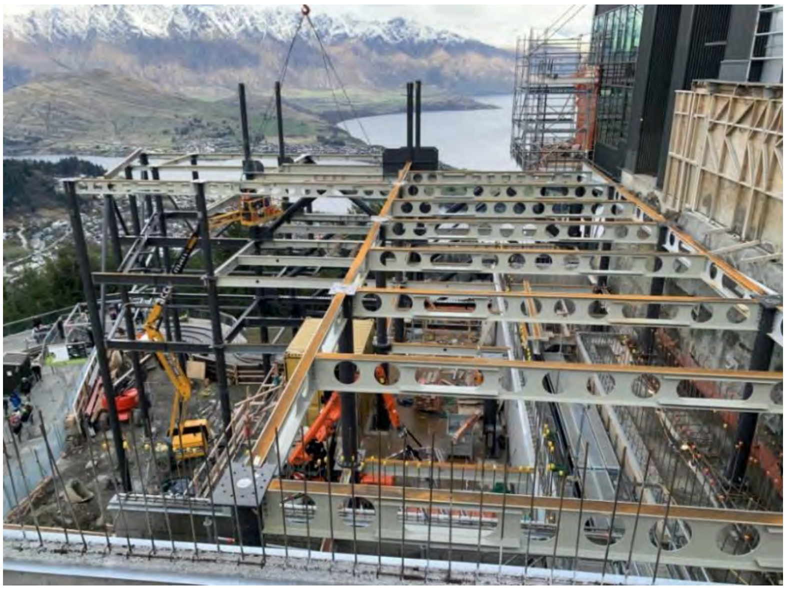

Figure 5: Construction of the new Top Building - Extension to the east of the existing building

The Top Building (Stage 2 - Extension) is currently under construction to the east of the existing building. This stage is located within 1m of the existing restaurant building and special consideration has been given to the construction staging and interface detailing resulting from the varying floor heights. Key construction challenges for this stage include:

- A constrained site with tight working space between the existing building and luge track, limited access and lay-down space.
- Partial undermining of the existing building to construct the lower levels of the new building.
- Requirement to maintain the ongoing operations of the site. Seven foundation pads adjacent to the luge track were installed during the luge track re-alignment work in 2017 to facilitate this.

The individual building stages needed to be constructable and self-supporting until the final stage connects them together. At this point the overall building realises its full-strength capacity. In the interim a temporary vertical bracing system was introduced into the Extension stage construction, as the partial building did not have the capacity to cantilever off the hillside.

Upon completion of Replacement stage, the floor plates of the first two stages will be stitched together and the temporary bracing removed.

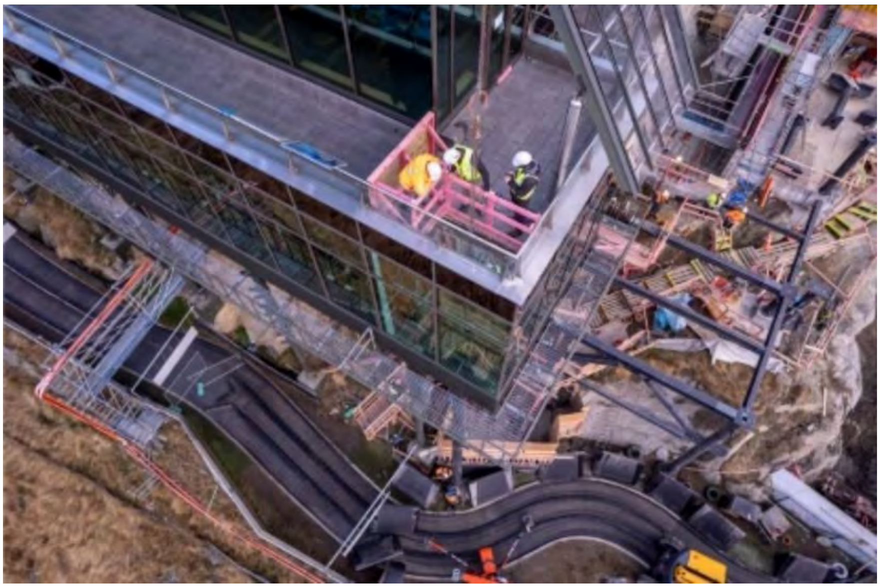

Figure 6: New Column being installed through existing live restaurant for the Extension structure

Bottom Terminal Building

Building Description and Design Philosophy

The Bottom Terminal is a massive 18m high, three-storey structure with a ground floor area of approximately 1300 m2. The building form is driven by its primary function as the base station for the gondola and to house the gondola cabins when they are removed from the line at night. The building also includes queuing and ticketing functions, and maintenance, storage and servicing space for the gondolas.

The integration and support of the 36 gondola cabins from the roof, each weighing 800kg, was a significant design feature for the Bottom Terminal. The detachable gondola cabins needed to travel through to the side workshop for servicing and maintenance, and up to the gondola parking/storage space, suspended above the gondola bull wheel.

Construction Staging

The new Bottom Terminal was built in two stages as shown in Figure 8. Two thirds of the new building structure (shown in red below) was required to be built whilst the existing bottom terminal building remained in use. Temporary CHS columns were installed through the existing building to support the new structure.

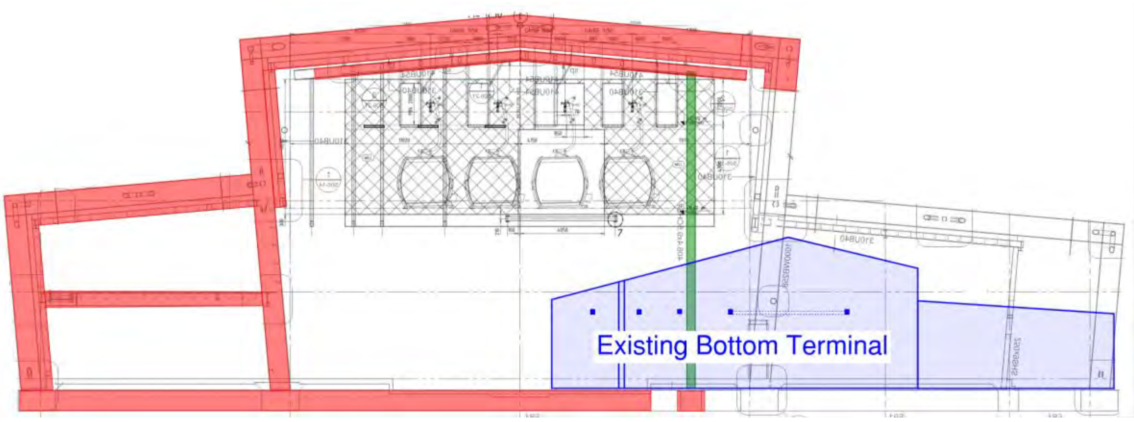

Figure 7: Cross section through the Bottom Terminal showing the scale of the new building

The gondola line was slightly offset to the existing to allow for partial construction of the new building and also construction of the gondola towers without obstructing the existing gondola line.

A key project driver was to minimise disruption during construction and keep the shutdown for transitioning to the new gondola system to an intensive 3-month period. The contractor worked double shifts, 7 days a week to complete the transition.

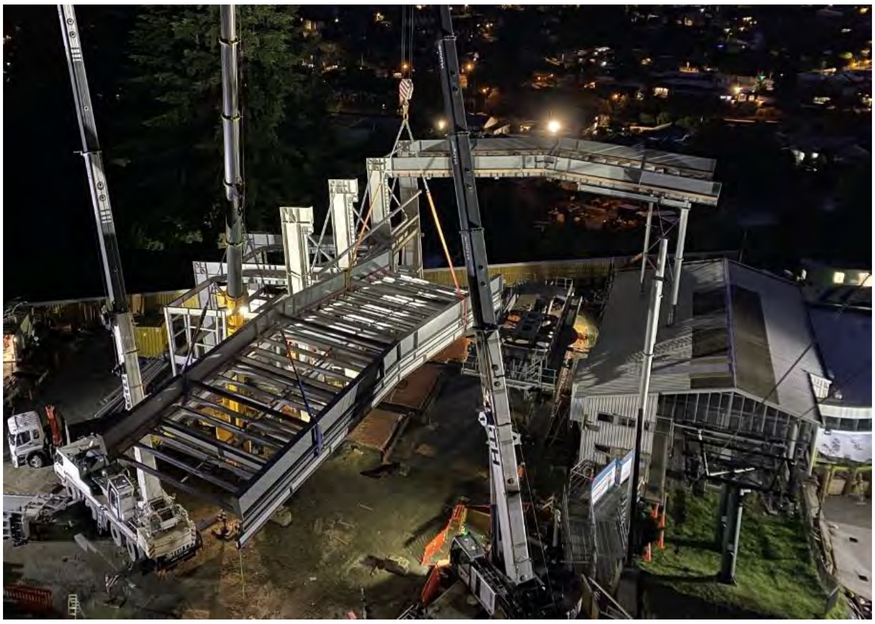

Figure 8: Construction of the new Bottom Terminal Building over the existing Bottom Terminal.

Summary of Construction Work to date

The Redevelopment Project is approximately halfway through the overall construction programme. The new Top and Bottom Terminals are complete and are in commission. The structure for the Top Building - Extension is nearly complete as shown in Figure 9. Following the commissioning of the Extension and temporary relocation of operational facilities, focus with shift to the demolition of the existing building and construction of the final Replacement stage of the project, for which planning is well underway.

Figure 9: Construction Progress on the Top Building (Extension) - February 2025

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The Seismic Performance of Concrete-Encased Concrete-Filled Steel Tubular Column to RC Wide Flat Beam Joint with Different Connection Details

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ABSTRACT

The application of composite column joints made of concrete-encased concrete-filled steel tubular (CFST) columns with reinforced concrete (RC) wide flat beam framework joints in underground structures is increasingly prevalent due to their superior performance. However, previous research on the structural forms and seismic performance of this type of joint is still insufficient. Therefore, three types of specimens were designed in this paper: the concrete-filled steel tubular column-RC wide flat beam joint (CFSTJ), the welded concrete-encased concrete-filled steel tubular column-RC wide flat beam joint (WCJ), and the anchored concrete-encased concrete-filled steel tubular column-RC wide flat beam joint (ACJ). Quasi-static testing was conducted on these specimens. The results indicated that the hysteresis curves of the three types of joints demonstrate good hysteretic performance and energy dissipation capacity. The stiffness degradation and strength degradation of the concrete-encased concrete-filled steel tubular column joint is smoother than that of the concrete-filled steel tubular column joint.
Keywords: Concrete-encased Concrete-filled Steel Tubular Column, Wide Flat Beam, Joint; Seismic Performance, Quasi-Static Test

INTRODUCTION

With the acceleration of global urbanization process and the gradual depletion of land resources, the efficient utilization of underground space has become an important trend to expand urban functions and solve the problem of space shortage. However, the challenges faced by underground structures are becoming increasingly severe. Earthquakes, as a common natural disaster, can bring significant damage and threats to underground structures. In order to improve the overall seismic performance of underground structures, it is necessary to conduct in-depth research and optimize design. As a critical component connecting beams and columns in a structure, joints play a vital role in determining the overall stability and safety of the structures. Therefore, in-depth research on the seismic performance of joints is crucial for optimizing their design and enhancing the overall seismic capacity of the structure.

In terms of seismic performance of concrete-encased concrete-filled steel tubular column joints, experimental studies have been undertaken. Ma et al. (2019) conducted experimental studies on the seismic performance of joints in 13 concrete-encased concrete-filled steel tubular column-RC beam connections with six different types of joints. The experimental results indicated the influence of various test parameters on the seismic performance of the joint specimens. Four typical failure modes of the joints and their hysteretic relationships were analyzed. Through comparisons with different types of joints, an evaluation of the seismic performance of the concrete-encased concrete-filled steel tubular column joints was provided. Pan et al. (2013) conducted a hysteretic test study on a new type of concrete-encased concrete-filled steel tubular column-RC ring beam joint and analyzed the load transfer mechanism of this type of joint. Aravind et al. (2014) introduced the load-bearing performance of composite column-reinforced concrete beam joint with an improved connection system.

In terms of the wide flat beam joint structure，experimental studies and finite element analyses have been undertaken. Stehle et al. (2001) found that suitable connection forms can reduce concrete cracking in the core area of joints. Huang et al. (2019) conducted hysteretic tests on six full-scale RC wide flat beam-column joints. The results indicated that as the reinforcement ratio of the longitudinal steel bars in the beam increases, the failure mode of the joint specimens shifts from bending failure at the beam ends to shear failure at the joint. Alipakzad et al. (2009) found that the joint specimens with square column cross-sections have better energy dissipation capabilities but poorer ductility compared to those with circular cross-sections. Li et al. (2015) conducted experiments and finite element analyses on RC wide flat beam-column joints. The study found that the axial load on the column significantly affects the seismic performance of wide flat beam-column joints, and increasing the anchorage ratio of the longitudinal reinforcement in the beam can enhance the shear bearing capacity of the joint.

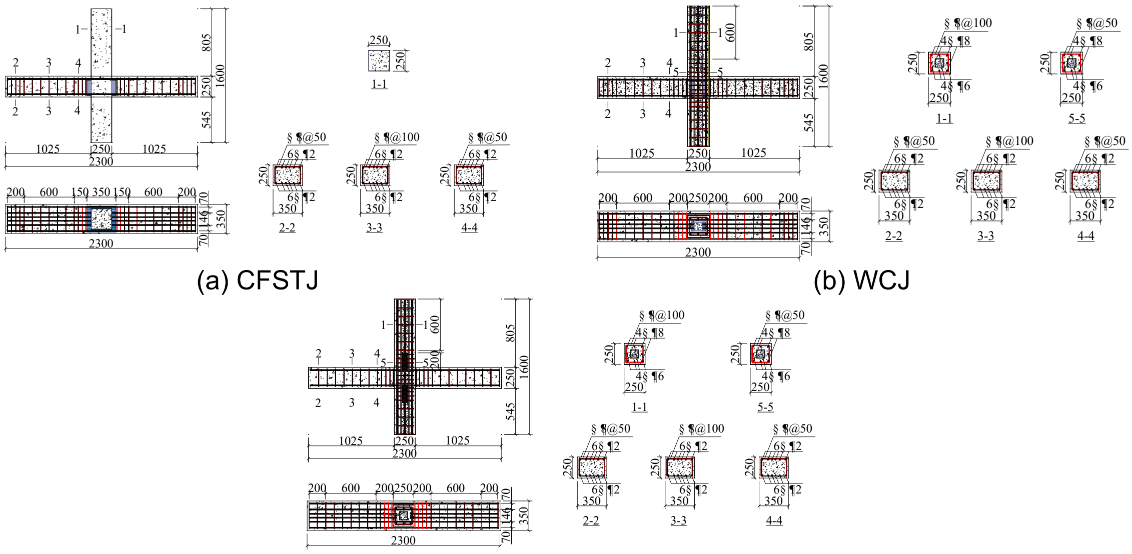
Based on the existing on the concrete-encased concrete-filled steel tubular column-RC beam joints, it has been found that the investigation of RC ring beam joints is relatively comprehensive, and it has been proven that this type of joint exhibits good seismic performance. However, due to the specificity of the ring beam joint region, the actual construction in engineering is relatively complex and inefficient. In contrast, RC wide flat beams demonstrate higher operability and controllability during the construction process, while reducing costs and construction periods. Nevertheless, there has been relatively limited research on concrete-encased concrete-filled steel tubular column-RC wide flat beam joints. Considering the significant advantages of these two types of structures in terms of seismic resistance, this paper will focus on exploring and analyzing the seismic performance of such joints. This aims to fill the existing research gap and provide a theoretical basis for further engineering applications.

EXPERIMENTAL DESIGN

Design and Overview of the Specimen

To intuitively compare the seismic performance of the three joints, after determining the joint forms and the loading method at the column ends, the design of the test specimens was based on two fundamental principles: first, the bending stiffness of the column sections must be equal; and second, the shape and size of the sections must be equal.

All three joint specimens were designed at a scale ratio of 1:2 to ensure consistency in geometric dimensions and beam reinforcement. Among them, the longitudinal reinforcement of the beams uses HRB400 grade steel bars with a diameter of 12 mm. The height of the square column is 1600 mm, with a cross-sectional side length of 250 mm. The length of the wide flat beam is 2300 mm, with a cross-sectional width of 350 mm and a height of 250 mm. For the concrete filled steel tubular column, the steel tube with thickness of 4 mm is made of Q235 grade steel, while the concrete in the core area is of C30 grade. For the concrete-encased concrete-filled steel tubular column, the thickness of the protective layer is set at 20 mm. The internal steel tube selected has an outer diameter of 100 mm and a wall thickness of 6 mm, made of Q235 grade steel. The outer layer of concrete uses C40 grade. The dimensions and reinforcement layout of the joint specimen are shown in Figure 1.



(a) CFSTJ (b) WCJ

(c) ACJ

Figure 1: Joint Dimensions and Reinforcement Illustration (Unit: mm)

Testing Load Device and Loading Scheme

The configuration of the loading equipment used in the experiment is shown in Figure 2. Fixed hinge supports are connected at the bottom of the column and at the bottoms of both ends of the beam, while the top of the columns is equipped with horizontally movable hinge support, along with lateral restraint devices to prevent lateral displacement of the specimen. In this experiment, the vertical axial force applied at the top of the column was controlled through a force control method, ensuring both the accuracy and controllability of the loading process. The axial load was incrementally applied in three stages until the design value was reached. At each stage, the load was maintained for five minutes to stabilize the condition of the joint specimen. Once the axial force was fully applied, horizontal cyclic loading was initiated, employing a force-displacement mixed control approach. The horizontal cyclic loading scheme that was adopted in this test is shown in Figure 3.

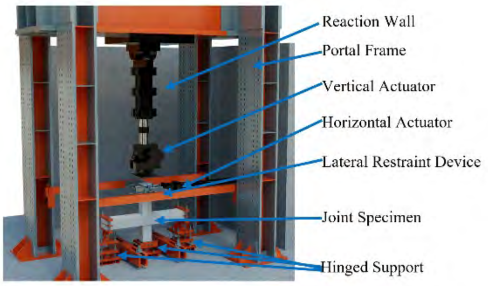


Figure 2: Schematic Diagram of the Test Loading Device

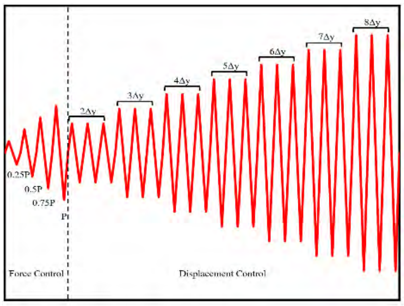
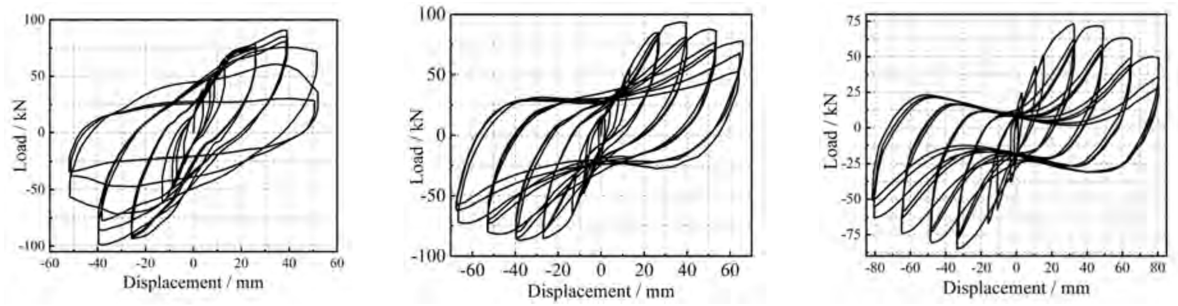


Figure 3: Schematic Diagram of the Loading Scheme

EXPERIMENTAL DESIGN

Hysteresis Curve Analysis

The hysteresis curves for the three joint specimens are shown in Figure 4. The hysteresis curves of the three joint specimens illustrate that all specimens exhibit relatively full hysteresis curves, indicating that the three types of joints possess good hysteretic performance and energy dissipation capabilities. The curves are shaped like an inverted S, suggesting that there is a certain degree of slip between the internal reinforcement and concrete of the specimens. In comparison to the CFSTJ specimens, the hysteresis curves of the two composite column joint specimens show more pronounced pinching behavior. This phenomenon is primarily due to the severe shear deformation in the core area of the composite column joints, as well as more significant bond slip between the reinforcement and the concrete. Furthermore, the reduction in bearing capacity of the CFSTJ is greater than that of the composite column joints. This is mainly attributed to the poor connection performance at the beam-column junction of the CFSTJ, resulting in concrete crushing at the beam ends and delamination at the beam-column interface, which leads to considerable stress concentration in the joint region.



(a) CFSTJ (b) WCJ (c) ACJ

Figure 4 Load-Displacement Hysteresis Curve

Skeleton Curve Analysis

The comparison of the skeleton curves for the three joint specimens is shown in Figure 5. The failure displacements of the WCJ specimen and the ACJ specimen both exceeded 60 mm, indicating that these two types of composite column joint specimens possess good deformability. The trends observed in the three joint specimens are fundamentally similar, with an asymmetry phenomenon in loads applied in the positive and negative loading directions. Experimental analysis suggests that the primary reasons for this asymmetry may be due to unavoidable gaps between the loading equipment and inevitable errors during the fabrication of the specimens. The average values of the ultimate loads in the positive and negative loading directions are 93.73 kN, 89.10 kN, and 73.33 kN, respectively. The load-bearing capacity of the CFSTJ is slightly higher than that of the WCJ, while the ACJ exhibits the lowest capacity. This indicates that the load-bearing capacities of the WCJ and the CFSTJ are relatively similar, whereas the ACJ experiences a notable reduction in load-bearing capacity. The average values of the failure displacements in the positive and negative loading directions are 46.08 mm, 63.51 mm, and 70.74 mm, respectively. The failure displacement of the ACJ is slightly greater than that of the WCJ, suggesting that the deformability of these two types of composite column joints is comparable, while the CFSTJ has the smallest failure displacement. This can be attributed to the sudden drop in the skeleton curve of the CFSTJ after reaching its peak, which may be related to poor connections between the concrete filled steel tubular column and the wide flat beam. This poor connection can lead to interface delamination, resulting in stress concentration at the welds between the truncated reinforcements and the tensile steel plates, causing damage or even failure at the welds.

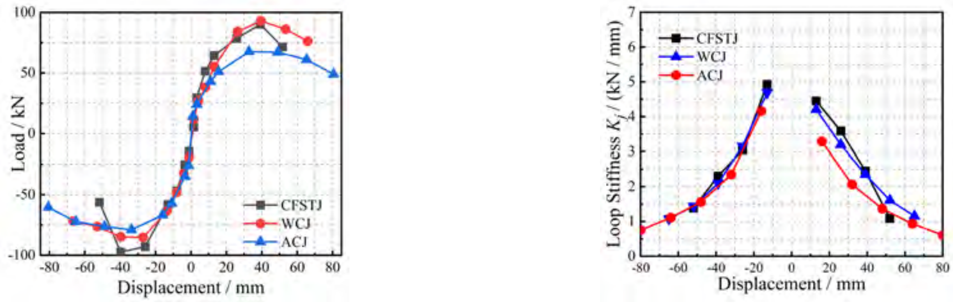


Figure 5: Skeleton Curve Comparison Chart

Figure 6: Stiffness Degradation Curve Comparison Chart

Stiffness Degradation Analysis

The loop stiffness of the three joint specimens decreases while the displacement levels increase. The stiffness degradation trends of the two composite column joints are essentially the same. All joint specimens exhibit asymmetry in both initial stiffness and degradation trends, with the loop stiffness under negative loading generally greater than that under positive loading. Under negative displacement loading, the stiffness degradation processes of the three joint specimens are relatively close; however, under positive displacement loading, there are significant differences among them, measuring 4.45 kN/mm, 4.20 kN/mm, and 3.29 kN/mm, respectively. The initial stiffness of the CFSTJ is slightly greater than that of the WCJ, while the ACJ has the lowest initial stiffness.

Analysis of Energy Dissipation Capacity

The curves showing the variation of cumulative energy dissipation and the equivalent viscous damping coefficient for the three-joint test specimens with increasing displacement levels are shown in Figure 7

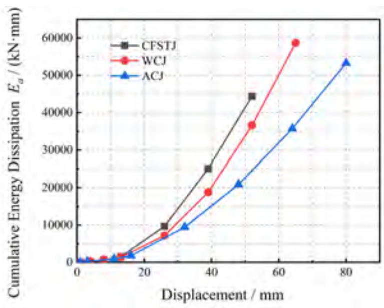


Figure 7: Comparison Chart of Energy Dissipation Capacity Indicators

The cumulative energy dissipation of the CFSTJ, the WCJ, and the ACJ are 44368.53 kN·mm, 58708.59 kN·mm, and 53277.28 kN·mm, respectively. Compared to the CFSTJ, the cumulative energy dissipation of the two composite column joints is improved by 32.32% and 20.08%, respectively. The WCJ exhibits the strongest energy dissipation capacity, while the ACJ has a slightly lower energy dissipation capacity than the WCJ, making the CFSTJ the weakest in terms of energy dissipation capacity. Throughout the entire loading process, the cumulative energy dissipation of the CFSTJ is higher than that of the WCJ and the ACJ at the same displacement level. However, due to the rapid degradation of strength and stiffness, as well as poor ductility characteristics, the CFSTJ is unable to sustain energy dissipation.

CONCLUSION

This study combines experimental methods to compare the seismic performance of three types of connections: concrete filled steel tubular column - RC wide flat beam joint connection (reference specimens), welded concrete-encased concrete-filled steel tubular column - RC wide flat beam joint connection, and anchored concrete-encased concrete-filled steel tubular column - RC wide flat beam joint connection. The main conclusions are as follows:

(1) The hysteresis curves of the three types of connections are overall relatively full, demonstrating good hysteretic behavior and energy dissipation capacity. The hysteresis curve of the composite column joints is relatively narrowed due to severe shear deformation in the outer core area and bond slip between the steel reinforcement and concrete. However, the connection at the beam-column interface of the composite column joints has better connectivity, resulting in a significantly slower decline in load-bearing capacity compared to the CFSTJ.

(2) The final strength degradation coefficients for the two types of composite column joints are 46.94% and 38.78% higher than that of the CFSTJ, respectively;

(3) The failure displacement and displacement ductility coefficients increase by 34.09% and 48.83%, respectively; and the accumulated energy dissipation increases by 32.32% and 20.08%, respectively. This indicates that the composite column joints exhibit better damage resistance, ductility, and energy dissipation capabilities.

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