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# PROCEEDINGS

**14 November 2024**
**Christchurch, New Zealand**

**SUSTAINABILITY IN THE DESIGN AND CONSTRUCTION OF THE CRL**
**TE WAIHOROTIU UNDERGROUND STATION**

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## SUMMARY

The City Rail Link (CRL) engaged Link Alliance to design and build all station structures on Auckland's $5.5Billion underground metro extension project - the largest public transport infrastructure project ever undertaken in New Zealand.

The structures include two new underground rail stations, a redeveloped existing surface station, and twin bored tunnels extending from Albert Street to Mt Eden where they bifurcate through a grade-separated interchange.

This paper focuses on the sustainability initiatives adopted during the design and construction of the Te Waihorotiu underground station, and review the lessons learnt that could direct the future of sustainability in major infrastructure projects.

## INTRODUCTION: SUSTAINABILITY AND THE CITY RAIL LINK

The City Rail Link (CRL) project is the most significant public transport infrastructure project ever undertaken in New Zealand, utilising resources, expertise, and materials from around the country and the world.

The project includes two mid-town underground rail stations – Te Waihorotiu and Karanga-a-Hape, the redevelopment of an existing surface rail station - Maungawhau, and two new 3.45km long rail tunnels extending from Maungawhau Station in Mount Eden to Waitematā Station (Britomart) in Auckland's city centre, as well as all rail and station systems.

Since its inception, the project has considered the sustainability and ecological impacts it has at every phase of its lifecycle – from its design, to construction, and operation.

Objectives for CRL's sustainability were included in the Link Alliance charter to ensure that a positive environmental and social contribution was achieved, along with more specific initiatives that match Aotearoa's unique history and landscape.

Being a project of such significant scale and importance, it was crucial that CRL created a benchmark for delivering sustainable civil infrastructure in New Zealand and could serve as an example for future major projects.

The CRL project's key sustainability objectives explored the following:

- Setting new benchmarks in sustainable and environmental performance by minimising disruption, reducing carbon, and minimising waste.

- Promoting positive social outcomes by engaging businesses, and promoting social development and understanding of diverse cultures and values.
- Leaving a great legacy by setting new benchmarks for safety, health, environmental and sustainability performance, and growing confidence in New Zealand's construction industry's ability to deliver mega-projects.

As a civil engineering project of such a grand scale, the CRL stations relied heavily on concrete and steel construction, which inherently create large carbon footprints. The structural forms of the stations and tunnels (see Figure 1) are as follows:

- The Te Waihorotiu mid-town underground station lies beneath Albert Street, extending between Wellesley and Victoria Streets, and is a 20m-deep, 300m long, single-island platform underground station constructed with a combination of top-down and bottom-up techniques, comprising reinforced concrete diaphragm or piled walls and three levels of reinforced concrete slabs. Entry to the station is provided from both Victoria and Wellesley Streets. The Wellesley Street entrance at the southern end of the underground station also has a four-storey reinforced concrete moment-frame building, designed and constructed to accommodate a future 24-storey tower.
- The Karanga-a-Hape up-town underground station extends beneath Pitt Street and Karangahape Road, between Beresford Square and Mercury Lane. This station is a 33m-deep mined underground, twin platform station, comprised of reinforced concrete diaphragm walls and up to seven levels of reinforced concrete slabs. Entry to the station is provided from Beresford Square which also supports a steel-framed entrance canopy structure, and Mercury Lane which has a three-storey steel braced-frame entrance building that will accommodate a future additional five-storey development.
- The existing Mt Eden surface station has been re-configured and upgraded to become Maungawhau Station and will provide connection and interchange between the CRL stations and the existing North Auckland Line (NAL). The station development includes a two-storey main entrance building constructed of reinforced concrete shear walls with a steel gravity structure on concrete piles extending up to 25m deep, steel-framed NAL and CRL overpass bridges extending to the platforms, and a four-storey reinforced concrete shear wall vent building. The new CRL platforms are located in a 7m deep open trench on the western connection and the refurbished NAL platforms are at ground level. Entry to the station is provided from Ruru Street.
- The reinforced concrete twin tunnels connecting the stations are 3.45km long and extend from Albert Street to Newton, and comprise a cut-and-cover section connecting the completed works beneath Albert Street to Te Waihorotiu Station and twin 7.5m diameter Tunnel Boring Machine (TBM) bored sections extending from Te Waihorotiu Station to Karanga-a-Hape Station, then further to Newton Junction.

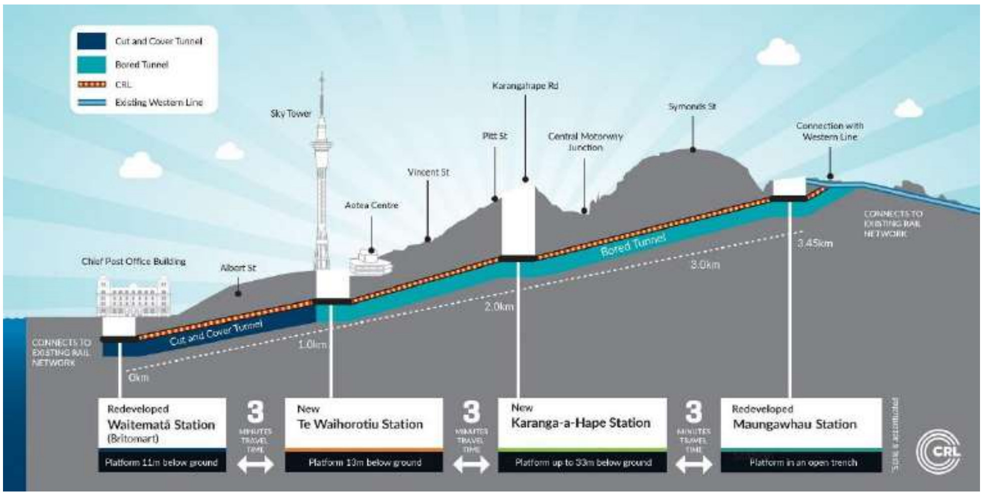


Figure 1: City Rail Link Stations.

Collectively, the CRL stations and tunnels are comprised of 185,000m$^3$ of concrete and 29,595tonnes of steel, producing 82,792 $tCO_2e$ and 40,234 $tCO_2e$ of embodied carbon respectively. A general breakdown of CRL's materials carbon footprint is shown in Figure 2 below.

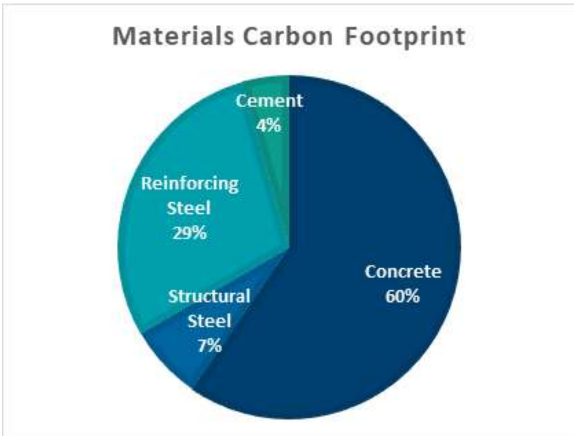


Figure 2: CRL materials carbon footprint.

As these quantities demonstrate, concrete contributes the majority of the project's embodied carbon emissions.

Over its 100-year design life, CRL's carbon footprint can be broken down into three sections:

- Materials' embodied carbon: Including materials needed to build the stations, tunnels, and rail systems, being predominantly concrete and steel.
- Construction energy: Including the electricity and fuel needed during construction for plant and equipment.
- Operational energy: Including electricity and fuel needed to power the stations, tunnels, and rail systems during normal service operation and maintenance.

The respective ratios of these sections as part of CRL's carbon footprint are shown in Figure 3 below.

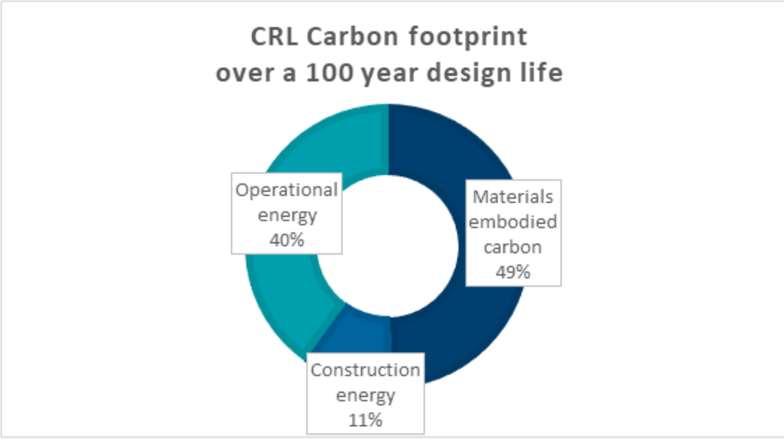


Figure 3: CRL 100 year design life carbon footprint breakdown.

Considering the contribution that material choice and construction techniques could have on the project's carbon footprint, positive measures toward sustainable solutions were incorporated during the design and construction phases of the CRL Stations.

**CRL SUSTAINABILITY OBJECTIVES**

Sustainability objectives incorporated in CRL's design and construction were established with the project's life cycle in mind, and included the following targets:

- 15% reduction in materials-related carbon emissions,
- 25% reduction in energy-related carbon emissions for construction and operation,
- Diversion of waste from landfills, i.e., 100% clean spoil diverted from landfill, 90% of construction and demolition waste diverted from landfill, 60% office waste diverted from landfill,
- 5% reduction in water consumption during construction and operation.

Key Performance Indicators were established based on the project's sustainability objectives, with a baseline against which to measure them established by calculating the projected whole-of-life footprint of the Reference Design used in the tender process (i.e. the "Base Case"). Initiatives to reduce the carbon footprint during the detailed design and construction planning were then implemented and compared against the Base Case. The BIM model was used to measure and track the impact of these initiatives on the project's projected carbon footprint as the design and construction progressed, helping to ensure the project's targets were met. The actual footprint of the materials and energy used during construction were recorded and tracked against the design projections (see Figure 4).

The establishment of the Base Case, and the calculations of savings, were externally verified by the Infrastructure Sustainability Council using their Infrastructure Sustainability rating tool. The project was recently awarded a "Leading" design rating, the highest available in the scheme.

In establishing sustainability initiatives for adoption in the design and construction of the stations, they could generally be divided into four main categories:

- Material choices and reduction of embodied carbon,
- Design optimisation,
- Reduction of direct emissions for construction and operation,
- Waste reduction.

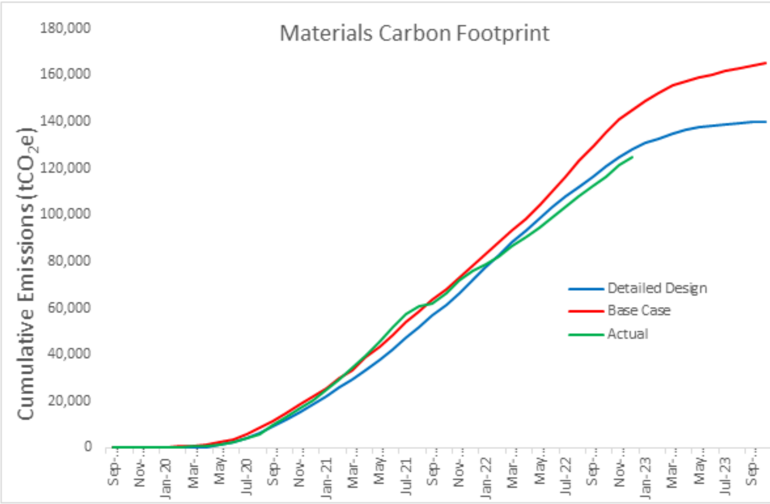


Figure 4: Link Alliance project-wide materials carbon footprint tracking.

## SUSTAINABILITY INITIATIVES IMPLEMENTS IN DESIGN AND CONSTRUCTION

Several key sustainability initiatives were implemented in the design and construction of Te Waihorotiu Station's underground structure. These initiatives included the following, with respective noted benefits:

- Optimisation of primary concrete elements
  - Reduce required volumes of concrete and reinforcement, associated costs, and embodied carbon.

- Review of concrete crack width reinforcement requirements
  - Reduce volumes of required reinforcement, associated costs, and embodied carbon.

- Use of fly-ash in concrete mixes
  - Reduce required cement and embodied carbon content,
  - Reduce temperature differentials in cast elements and minimise rework from crack repairs.

- Use of diaphragm walls instead of piled walls
  - Reduce required volumes of concrete and reinforcement, and associated embodied carbon,
  - Optimise construction equipment and reduce associated operational carbon emissions,
  - Avoid additional construction works for lining walls.

- Use of top-down construction for the underground station structure
  - Reduce required structural steel temporary prop elements,
  - Reduce construction time and better optimise staged construction principles,
  - Allow completed slabs to be used as working platforms,
  - Optimise construction equipment and reduce associated operational carbon emissions.

**RESULTS**

Results of these sustainability initiatives implemented in the design and construction of Te Waihorotiu Station are as follows:

<u>Optimisation of primary concrete elements</u>

The station's roof, concourse, and base slabs were optimised to use thinner central regions and construction void infill sections (see Figure 5), as well as improved reinforcement detailing. The edge regions were 1000-1200mm thick, while the central regions were 750-900mm thick, saving 1508m$^3$ of concrete. This optimisation resulted in a reduction of 686 $tCO_2e$ of embodied carbon in the station's slabs.

Approximately 28,000 reinforcement bar couplers were also used in lieu of lapping bars throughout Te Waihorotiu's construction, saving approximately 143,630kg of reinforcement and 177 $tCO_2e$ of embodied carbon. Additionally, the use of couplers allowed for more efficient staged construction and promoted a safer site by reducing reinforcement trip hazards in already busy and congested areas.

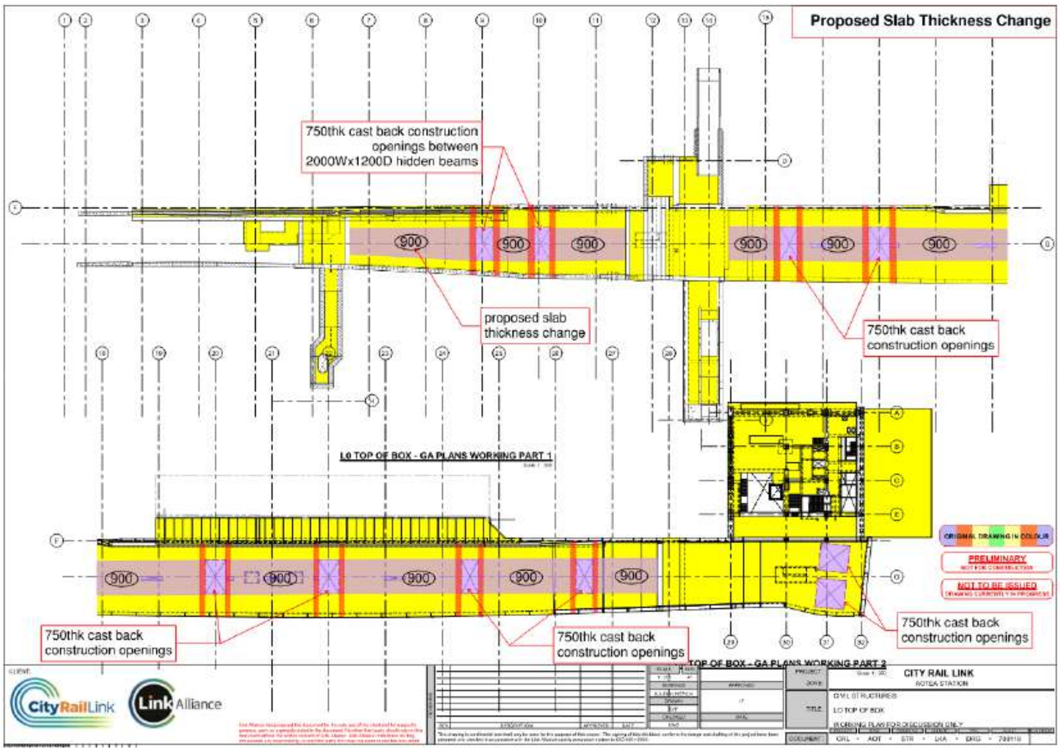


Figure 5: Te Waihorotiu L0 roof slab showing optimised central regions.

Review of concrete crack width reinforcement requirements

The project's minimum requirements stipulated a crack width limit of 0.2mm for all concrete elements adjacent to ground, but this was successfully challenged during the detailed design phase and replaced with a limit of 0.2mm for through- and flexural-cracks, and 0.3mm for all others. A minimum concrete compression zone depth of 50mm was maintained during design in accordance with NZS3101 and NZS3106 to ensure through- or flexural-cracks were avoided, resulting in more specific and optimised designs rather than blanket requirements for all concrete elements.

This review of crack width requirements led to a reduction of reinforcement of approximately 35% across Te Waihorotiu station's concrete elements, and an estimated reduction of approximately 938,580kg of reinforcement and 1158 $tCO_2e$ of embodied carbon in the station's D-walls alone. The difference in reinforcement content is indicated in Figures 6 and 7 below.

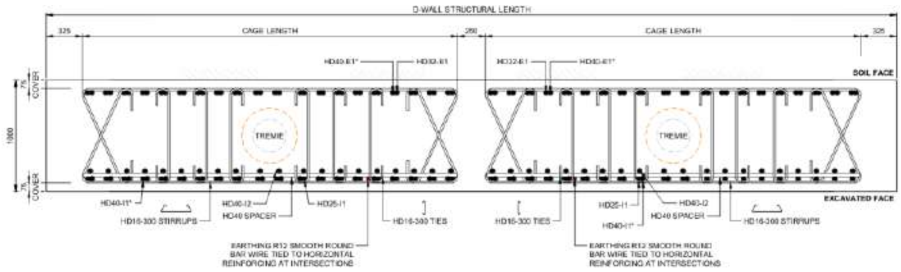


Figure 6: Cross-section of D-wall with typical reinforcement sufficient to meet 0.2mm crack width limit.

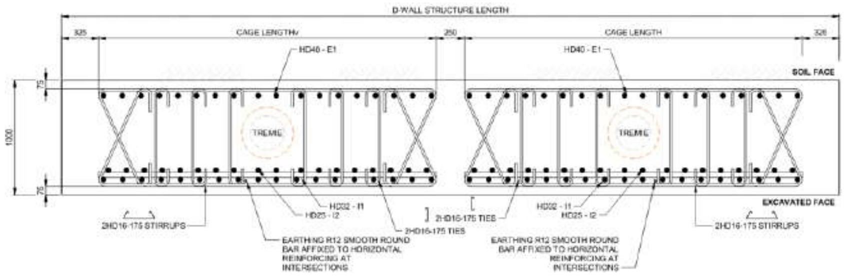


Figure 7: Cross-section of D-wall with typical reinforcement sufficient to meet 0.3mm crack width limit.

Use of fly-ash in concrete mixes

The project sought to maximise cement replacement with fly-ash, and the majority of the project's mix designs included 20-40% cement replacement. Data up to March 2024 shows that this has resulted in an average 21% cement replacement, reducing the project-wide embodied carbon footprint by almost 20,000 $tCO_2e$ (21%) and Te Waihorotiu Station's footprint by 7,472 $tCO_2e$ (23%).

Use of diaphragm walls instead of piled walls

During initial discussions between the design and construction teams, it was decided to use diaphragm walls (D-walls) instead of secant or contiguous piled walls with interior shotcrete linings. This approach was made possible due to the knowledge and expertise of Soletanche

Bachy[1] and Vinci Construction[2], who had successfully completed similar international projects. Reinforced concrete D-walls were adopted for much of Te Waihorotiu's underground station, constituting 65% of the station's total length (see Figure 8).

This decision resulted in a reduction of 3,600m$^3$ of concrete and approximately 331,400kg of reinforcement, equating to a total of 2,043 t$CO_2$e of embodied carbon.

Additionally, using D-walls meant that construction could be done in a single stage, avoiding the need for two-stage pile and lining-wall construction, ultimately reducing the labour required for full construction of the vertical elements and carbon emissions from associated construction machinery. However, using a D-Wall hydrofraise instead of a piling rig did slightly increase the construction energy emissions by 99 t$CO_2$e. Had a fully electric hydrofraise been available this increase could have been turned into an additional saving of 216 t$CO_2$e.

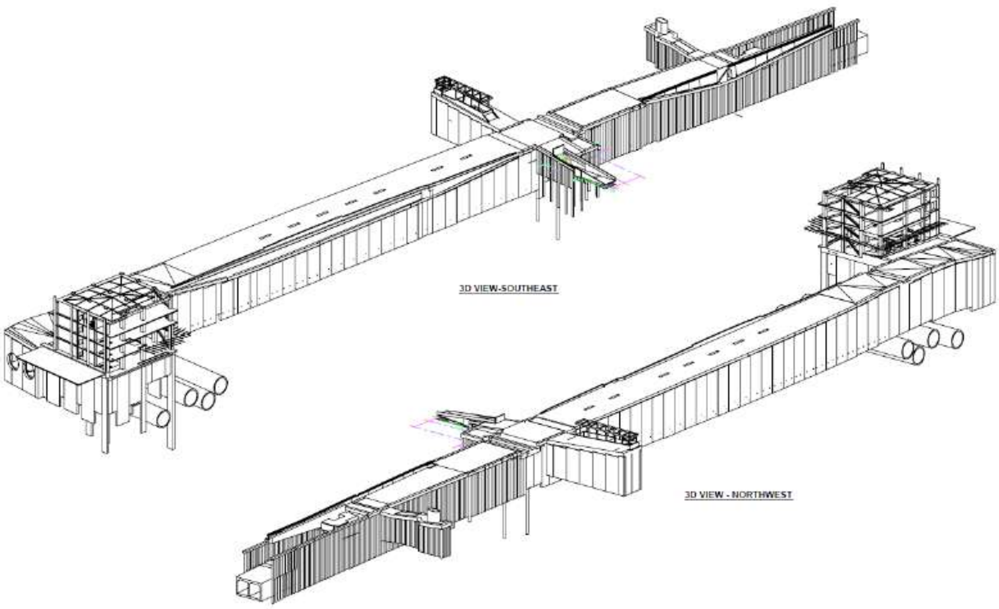


Figure 8: 3D render of Te Waihorotiu Station showing D-walls for much of the perimeter.

Use of top-down construction for the underground station structure

Te Waihorotiu station's construction methodology involved construction of the vertical wall elements first, then on-grade construction of the upper slabs, followed by excavation below to construct the lower slabs. This methodology was discussed between the design and construction teams, and the station's underground structure was designed to accommodate this staged construction and associated load conditions during both partial construction and full completion (see Figure 9).

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[1] *Soletanche Bachy is a France-based world leading construction firm specializing in foundations and soil technologies. They have completed projects in over 60 countries.*

[2] *Vinci Construction is a France-based global leader in construction of major building and civil engineering projects, with operations in over 100 countries.*

This top-down construction approach eliminated the need for lengthy excavation periods and reduced the reliance on structural steel temporary works had a traditional bottom-up methodology been used. The number of central-station plunge columns required was reduced from 40 to 7, resulting in a reduction of 100,259kg of structural steel and 163 $tCO_2e$ of embodied carbon; while the completed slabs formed the permanent structure to support the perimeter D-Wall and/or piled walls, reducing the need for temporary struts and walers, resulting in a further reduction of approximately 823,158kg of steel and 1,343 $tCO_2e$ of embodied carbon.

Additionally, the station used 85,000kg of the Albert Street tunnel's second-hand steel for temporary propping in the northern zone's cut-and-cover tunnel, while the temporary steel struts used in the station's central box construction were designed with bolted central splice sections for reuse along the station's length. The construction team has also returned nearly 70,000kg of temporary steel struts from across the station to the Link Alliance yard for potential reuse.

Finally, telescopic excavators fitted with clamshell arms were used during excavation to load directly into trucks, reducing double-handling; resulting in greater fuel efficiency, a more streamlined excavation programme, and reduced associated emissions by approximately 50% compared to more conventional excavation methodologies, saving approximately 900 $tCO_2e$.

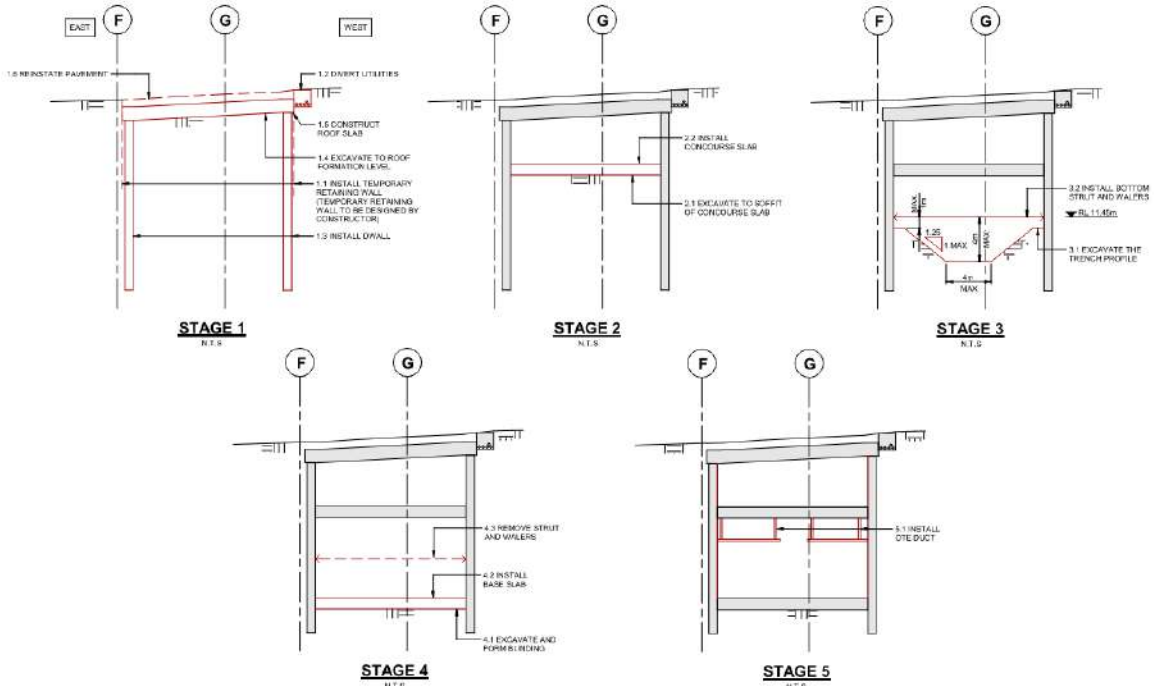

Figure 9: Typical top-down construction sequence of the Te Waihorotiu Station central box.

**PROJECT RESULTS TO DATE**

To date, as the City Rail Link's structural design has been completed and its construction concludes its phase of major civil construction and becomes that of architectural and services fit-out, the projections of the project's sustainability targets are as follows:

- Projected 16% reduction in materials carbon footprint.
- Projected 21% cement replacement with fly ash.
- Projected 19% reduction in construction energy footprint.
- 96% of construction and demolition waste diverted from landfills.
- 11% waste re-used.

**CONCLUSIONS AND LEGACY**

The value of sustainability in the design and construction of major civil engineering projects cannot be overstated. By incorporating sustainable practices, materials, and construction techniques projects could reduce their environmental impact, improve their efficiency, and contribute to more sustainable futures.

As shown by the initiatives implemented in the design and construction of CRL’s Te Waihorotiu Station, there are several key considerations to incorporate when undertaking major construction projects and the impacts they have on the development of sustainable infrastructure:

- Establishing communication and coordination between design and construction disciplines early is crucial to gaining support and cooperation when adopting innovative processes that could lead to significant benefits, both in terms of efficiency and reducing embodied carbon.
- Conducting quantitative analyses is critical to ensure sustainability initiatives are indeed beneficial.
- Drawing on previous or external knowledge and experience can bring valuable insights and inform construction and design processes.

By incorporating these considerations, major civil engineering projects could be completed more efficiently, sustainably, and with greater success.

Acknowledgements: To Bobby Bell, Neil Quayle, and Nick Braxton for their assistance in quantifying and enthusiasm in encouraging the sustainability initiatives adopted in the design and construction of the City Rail Link project.

*(Noted values are estimated values based on calculation of material quantities from verified IFC design data as measured by BIM model and reference design by Quantity Surveyor, except where stated from construction/suppliers).*

# PAP7, PARTICLE PACKING & PARETO PRINCIPLE IN CONCRETE TECHNOLOGY

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## SUMMARY

PAP7 was synonymous with high-quality manufactured sand blended with natural sands from marine or alluvial sources to produce the fine aggregate component of concrete. The assumption of quality of PAP7 has been questioned recently with some materials exhibiting poor shape, grading and/or deleterious fines, leading to poorer performance of concrete. This paper seeks to quantify the material and environmental cost of using manufactured sands that are below optimum and what alternatives exist. The research explains how manufactured sand affects both water demand and particle packing of concrete, that influence workability and compressive strength. It also questions current thinking in concrete technology where the cementitious paste content is assumed to control the properties of concrete (e.g. synonymous with the Pareto Principle). Various PAP7 sources were analysed and compared with modern processed sands in terms of their physical characteristics and the performance in mortar and concrete mixes. Testing was also undertaken to identify how particle shape of manufactured sand affects packing in concrete, with concrete microstructure being characterised using density and porosity. Research findings show that using lower quality PAP7 in concrete may result in a 10-15% increase in baseline cementitious content compared with concrete using high quality processed sand. This paper also discusses methods to control water demand, reducing adsorption effects and better particle packing to improve strength. Recommendations are also made about simple characterisation protocols for manufactured sands and their blends.

## INTRODUCTION

Manufactured sand for concrete is an increasing component of aggregates used in concrete mixes as natural sand resources become scarcer in New Zealand. Modern processed sands have the potential to reduce the need for blending significant quantities of natural sand that are often extracted from sensitive marine and coastal environments. This research was undertaken to compare the concrete properties of three different fine aggregates types used in construction:

- Natural sands from alluvial sources that are relatively clean, well-shaped and in cases where the grading allows, can be used without blending with other sand sources
- Manufactured sands often referred to as PAP7 (Premium All Passing 7mm) that typically are angular in shape, coarsely graded and require significant blending with fine sands (e.g. blending with fine sands at 40-60%)
- Modern processed sands produced by modern crushing equipment makes more cubical shaped particles with good grading that reduces blending of fine sand

The quality of aggregates has a direct bearing on the amount of cementitious material in concrete mixes. Unfortunately, in concrete technology the Pareto Principle holds sway

(i.e. where 20% of the causes affects 80% of the outcomes), which translates to the cementitious fraction have an outsized effect on concrete properties. Whilst this thinking may have some truth associated with it, ignoring aggregate quality can result in extra costs and associated carbon dioxide emissions.

Table 1 shows how these differences in fine aggregate quality can cause a significant difference in the cement efficiency of concrete mixes. Cement efficiency refers to the relative strength performance of concrete mixes.

Table 1: Quality influences of manufactured or processed sands on concrete properties

| Fine aggregate type (notation) | Grading | Particle shape | Cleanness | Concrete Properties |
|---|---|---|---|---|
| Natural sands (NS) | Continuous, well graded | Rounded to sub-rounded | Relatively clean | Low water demand and good cement efficiency |
| Manufactured sands (MS) | Poor grading, requires blending | Angular shaped | Washed to remove poor fines | High water demand, low cement efficiency |
| Modern processed sands (MP) | Well-graded with low to zero blending | Cubical shaped | Excess fines removed or controlled | Mod-low water demand and good cement efficiency |

This paper defines modern processed sand as being of sufficient quality in terms of grading, shape and cleanness to be used as 100% replacement of fine aggregates. This reduces the need to blend with fine sands from sensitive dune or marine deposits that are difficult to extract.

## RESEARCH SIGNIFICANCE

The quality of fine aggregate is known to influence the performance of concrete but marginal materials and poor processing are sometimes used in concrete production, affecting cement efficiency. This can be seen from the plot of compressive strength versus water/cement ratio (Abram's Law) shown in Figure 1 for NZ concrete mixes. The difference between worst and best performance represents over 20% extra cementitious material for some concrete mixes (e.g. a difference in cement content of 50-80 kg/m$^3$ in some cases).

It should be noted that these differences in cement efficiency are not a simple matter of higher water contents in concrete caused by sub-optimal fine aggregate combinations. If this was the case it would be possible to overcome this problem using chemical admixtures such as water reducers. These variations in cement efficiency have developed over many years and reducing the effect requires better understanding of the main influences affecting compressive strength of concrete.

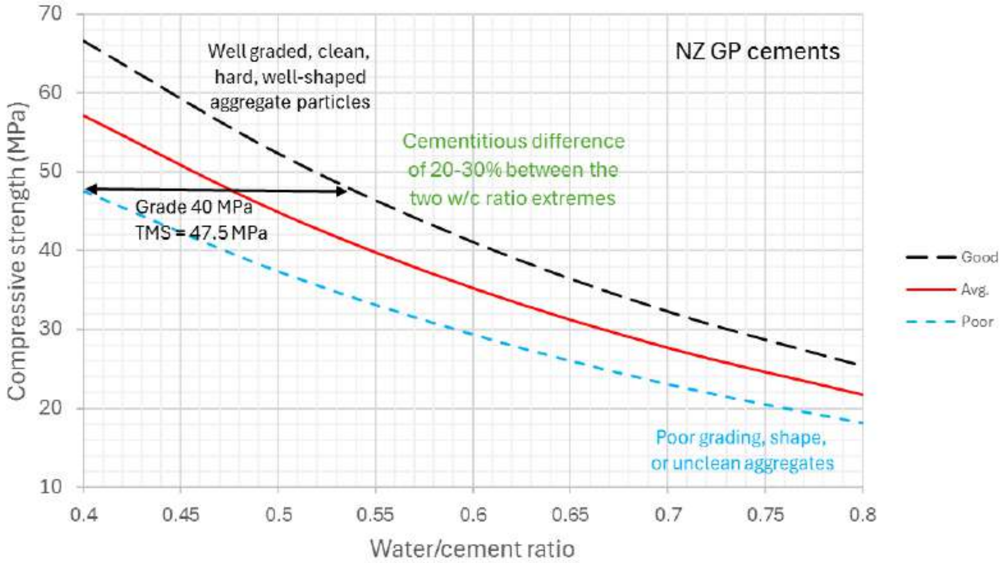


Figure 1: Variation in compressive strength with water/cement ratio

There has been significant research characterising manufactured sands used in concrete. Much of this research has either focused on aggregates characterisation without identifying the effects on concrete properties or where the benefits to concrete are shown without sufficient characterisation of the aggregate quality. Research also has started to quantify environmental advantages of moving from basic manufactured sand to higher quality processed sands produced by impact crushers.

**SAND QUALITY FACTORS**

Several factors affect the quality of fine aggregate blends used in concrete, which influence the efficiency of cementitious materials. Main factors are discussed below.

Grading

The particle size distribution of fine and coarse aggregate is important since this forms the matrix of particles from 20 microns to 20 millimetres and poor grading must be filled by more cement paste. Grading of fine aggregate is routinely done to ensure consistency and optimise packing of aggregate particles. Sieve analysis is relatively simple to perform and is routinely done for concrete aggregates. Wet sieving has become the standard in some countries and has the advantage that it allows finer fractions of sand to be accurately measured (e.g. 75 or even 45 microns).

Particle shape

Particle shape of fine aggregate particles affects fresh concrete performance since it affects water demand and workability of concrete. Manufactured sands that are more cubical in shape produce better concrete characteristics than angular materials and generally allow concrete to be designed with lower binder contents. Several image analysis techniques have recently been developed but these generally have poor resolution below 0.5 mm. An older test that provides useful information is the NZ sand flow test and is included in NZ standards NZS 3111 & NZS 3121. Note NZ sand test is generally not suitable with fine aggregate having particles much bigger than 5mm.

Cleanness

Deleterious fines are usually silts or clays that are extremely fine (i.e. less than 15 microns) and its plate form increases its surface area and activity. The influence of silts and clays on fresh and hardened concrete is not that predictable and its severity depends on geological type with swelling clay most deleterious in concrete. Strength is often less affected than secondary properties such as bleeding, setting and drying shrinkage. Three levels of analysis can be used; silt sedimentation testing, sand equivalent (SE) testing and more advanced methods such as clay index or methylene blue value (MBV) testing. The first method can be done at the concrete plant while other methods require specialist testing facilities.

<u>Overall assessment</u>

There is no simple measure available to provide an overall assessment of the quality of fine aggregate used in concrete production. Quarry suppliers attempt to keep the material within the stated limits of standards such as NZS 3121:2015 that are quite broad and when all three properties (e.g. grading, shape and cleanness) approach the limits there is a significant impact on concrete performance. This can be seen schematically in Figure 2 that attempts to provide a more holistic assessment of sand quality. It would be a logical approach and this paper will investigate whether it is appropriate for improving manufactured sand such as PAP7.

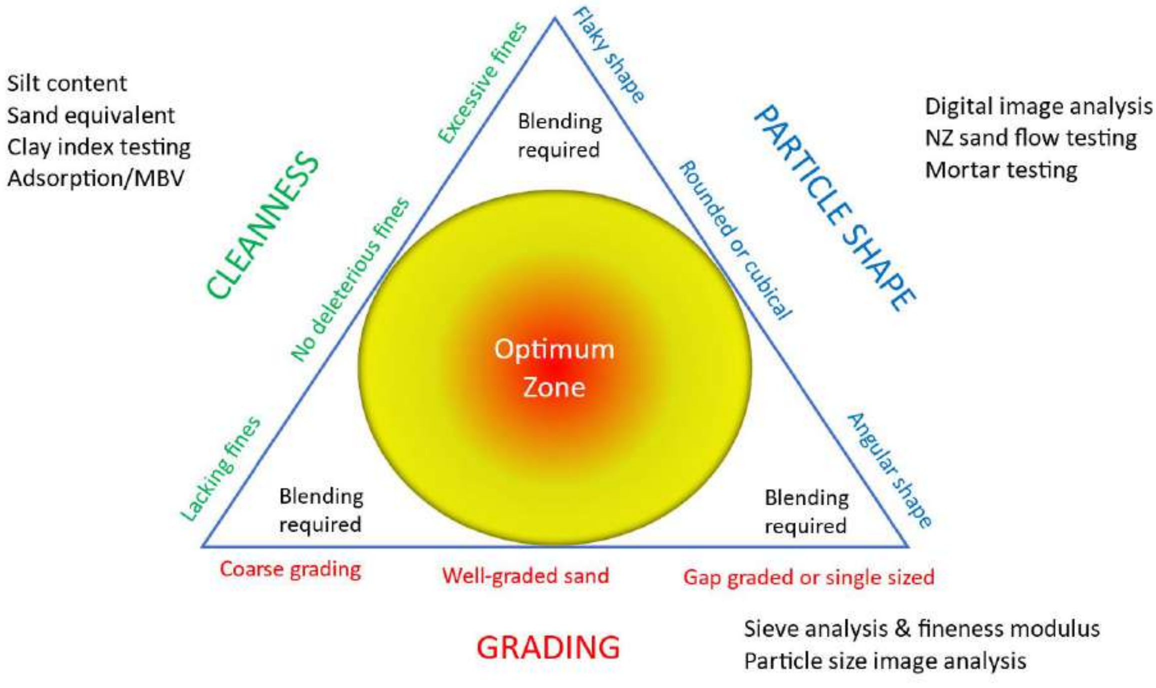


Figure 2: Optimum properties for manufactured sands used in concrete

Modern processed sands are produced using systems that are better able to optimise all three physical properties discussed above. The main advantage is an improved particle shape that is achieved using advanced crusher and air screening technology. There is no strict definition of a modern processed sand as distinct from a manufactured sand but it should be able to be used at high replacement levels for the fine aggregate fraction in concrete. Some characteristics include:

- Good moisture control due to air screening or limited washing with water
- Continuous grading that does not require significant blending with natural sands
- Good shape control of particles with cubical rather than angular particle shapes

- Low contamination levels of deleterious fines from sources such as silts, clays or organic materials

## EXPERIMENTAL PROGRAMME

A range of fine aggregates were compared in this investigation, representing typical materials used in concrete production. Fine aggregate resources were characterised in accordance with NZS 3111 in terms of grading, particle shape and cleanness (see Appendix 1 for grading analysis of these materials).

Details of these materials and combinations are shown in Table 2 and Appendices:

- NS represents a natural sand from alluvial source that is typically used without any blending of other fine aggregate (representing the ideal control)
- MP represents modern processed sand that is produced with better control of particle shape and grading
- MS represents manufactured sand (PAP7) that is produced from hard rock quarries in the North Island of New Zealand
- FS represents fine sand derived from coastal or marine deposits around New Zealand that is blended with manufactured sand to improve grading and shape
- PB is the notation for blends of MS & FS commonly used in concrete production in the North Island of New Zealand

Table 2: Fine aggregate combinations and properties

| Property | NS | MP1 | MP2 | PB1 | PB2 | PB3 | PB4 |
|---|---|---|---|---|---|---|---|
| Materials | Natural Sand - NS | Modern processed sand–MP1 | Modern processed sand–MP2 | 60% MS1<br>40& FS1 | 55% MS2<br>45% FS2 | 60% MS3<br>40% FS3 | 50% MS4<br>50% FS4 |
| Fines (< 0.075) | 0.8 | 1.3 | 4.8 | 1.1<br>0.2 | 1.3<br>0.3 | 1.3<br>0.1 | 3.5<br>1.8 |
| FM | 2.69 | 2.68 | 2.49 | 3.89<br>1.89 | 4.15<br>0.99 | 3.72<br>1.43 | 3.39<br>2.23 |
| Blend FM | 2.69 | 2.68 | 2.49 | 3.09 | 2.72 | 2.80 | 2.81 |
| SG | 2.65 | 2.68 | 2.65 | 2.68<br>2.63 | 2.68<br>2.65 | 2.68<br>2.65 | 2.62<br>2.62 |
| Silt (%) | 4.3 | 5.9 | 6.5 | 10.7<br>0.5 | 10.4<br>1.2 | 19.5<br>0.3 | 10.0<br>2.7 |
| SE (%) | 85 | 82 | 79 | 82<br>- | 66<br>- | 77<br>- | 87<br>- |
| Clay index (%) | 0.8 | 1.8 | 0.8 | 2.4<br>- | 1.9<br>- | 4.5<br>- | 2.4<br>- |
| Flow time (sec) | 22.3 | 26.4 | 23.5 | 36.5<br>22.2 | 36.0<br>21.3 | 33.0<br>20.1 | 36.0<br>24.5 |
| Voids | 38.5 | 45.5 | 41.9 | 47.3<br>45.6 | 43.5<br>48.0 | 45.8<br>44.4 | 48.0<br>45.6 |
| Location | Christ-church | South Auckland | Waikato | South Auckland | Rodney | Northland | Waikato |

Note: FS samples were not tested with SE or CI as these materials are naturally clean

Figure 3 shows the NZ sand flow results plotted against NZS 3121 limits for blended sands. Only PB3 (blend of MS3 and FS3) fell within the recommended blending limits. This was the result of the better shape and overall grading of these sands compared with the other blends. In contrast, natural sand and the two processed sands were well within NZS 3121 sand flow limits.

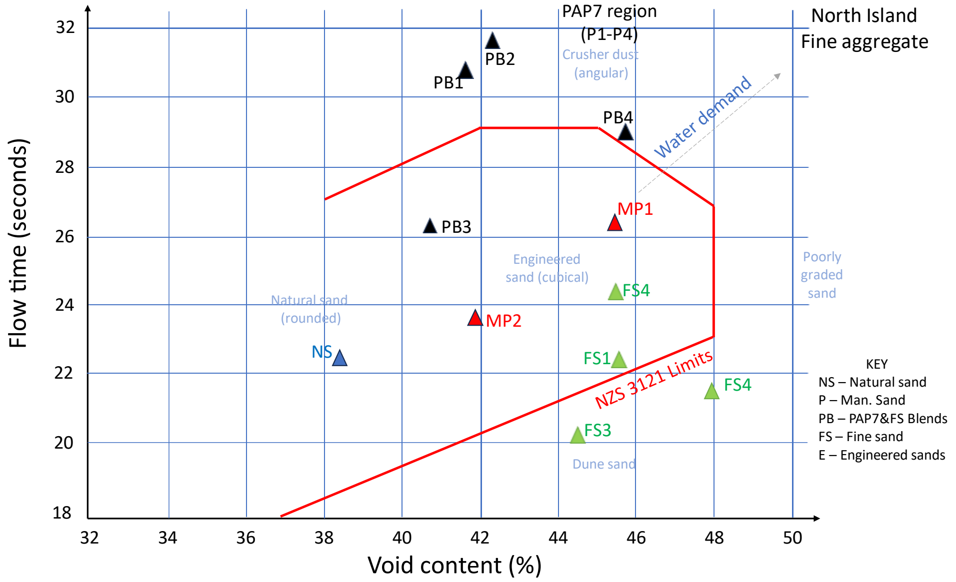


Figure 3: NZ sand flow results for NS, MP and MS blends

Mortar testing was conducted in accordance with AS/NZS 3583.6 with details shown in Table 3. Mortar containing either 100% natural sand or processed sands were compared with mortar containing the typical blend ratios of manufactured sand and fine sands. Mortars were mixed and tested at standard consistence of 150±15 mm (measured on a flow table apparatus). Porosity was measured by drying standard 50x50x50 mm cubes at 50 $^{0}$C until constant weight was achieved after 28 days.

Table 3: Mortar testing of aggregate combinations using AS/NZS 3583.6)

| Material | NS | MP1 | MP2 | PB1 | PB2 | PB3 | PB4 |
|---|---|---|---|---|---|---|---|
| Cement (g) | 333 | 333 | 333 | 333 | 333 | 333 | 333 |
| Sand 1 (g) | 1000 | 1000 | 1000 | 600 | 550 | 600 | 500 |
| Sand 2 (g) | 0 | 0 | 0 | 400 | 450 | 400 | 500 |
| Water (g) | 171 | 175 | 173 | 183 | 187 | 185 | 186 |
| w/c ratio | 0.514 | 0.526 | 0.520 | 0.550 | 0.562 | 0.556 | 0.559 |
| Flow Ø (mm) | 155 | 145 | 150 | 160 | 155 | 160 | 150 |
| 14D HD (kg/m$^3$) | 2228 | 2276 | 2254 | 2260 | 2250 | 2242 | 2218 |
| 14D $f_c$ (MPa) | 47.8 | 46.7 | 46.4 | 45.4 | 44.3 | 45.9 | 40.5 |
| 56D HD (kg/m$^3$) | 2231 | 2273 | 2262 | 2259 | 2260 | 2250 | 2210 |
| 56 $f_c$ (MPa) | 65.1 | 61.6 | 59.6 | 55.2 | 55.7 | 57.9 | 55.4 |
| Porosity (%) | 15.97 | 16.52 | 16.71 | 17.49 | 17.87 | 17.77 | 18.00 |

The higher water demand of mortar made with PAP7 predicted a lower strength compared with mortar using natural or processed sands. Figure 4 shows the relationship between strength and water/cement ratio, which confirms this prediction. This trend was more noticeable at 56 days probably due to the influence of micro-fines on initial hydration rates. The relationship between mortar strength and water/cement ratio was not well correlated

due to other influences such as packing efficiency (see porosity variations discussed below) and the possible influence of ultra fine particles.

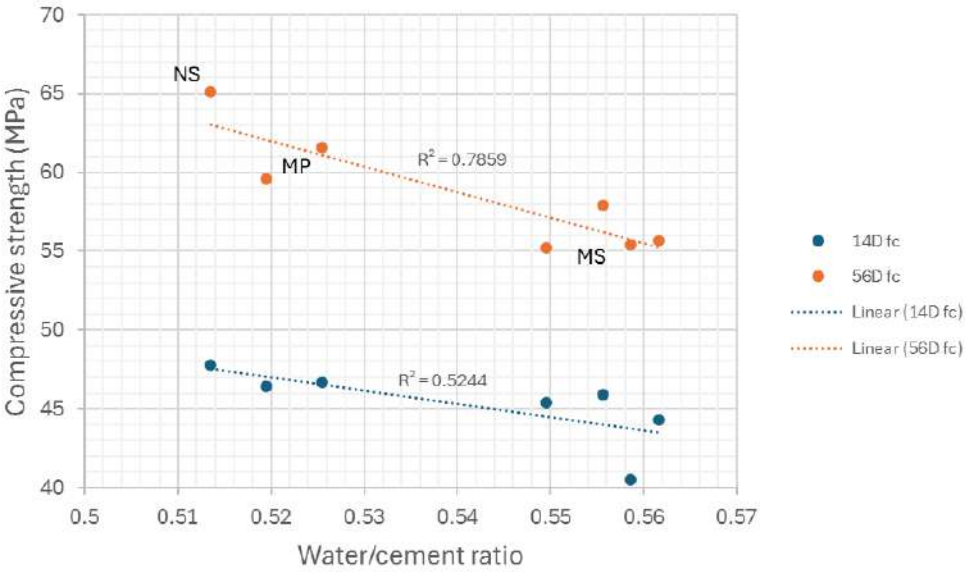


Figure 4: Compressive strength of mortar versus water/cement ratio

Porosity testing was undertaken on mortar samples to assess whether there was any correlation with predictions of particle packing based on grading and shape. Figure 5 shows the effective porosity for the seven mortar mixes that were reviewed, showing a consistent increase in porosity for mixes containing poorer shaped and graded sands. This was also consistent with an increase in water demand for these mortars.

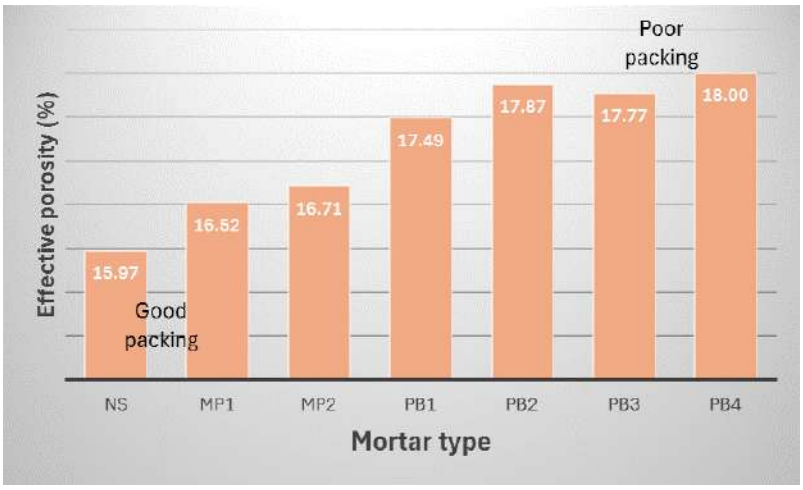


Figure 5: Effective porosity of mortar samples and relationship to packing efficiency

Concrete testing was undertaken to measure the influence of direct substitution of a processed sand (MP2) for a manufactured sand (MS1) in a 40 MPa concrete mix. Details are given below and in Appendix 2:

- Processed sand blends were varied from the standard 60/40 blend to 100%
- Concrete using 60/40 and 80/20 blends had cementitious contents varied from 100% to 90% of the control values to quantify potential cement savings

Figure 6 shows a summary of the strength results from this laboratory trial. These findings show that the replacement of processed sand in the standard concrete mix produced consistently higher strength due to lower water demand and better packing. Savings of cementitious material of more than 10% appear possible from these findings. Replacement of manufactured sand with more modern processed sands was therefore shown to significantly reduce embodied carbon dioxide contents in concrete.

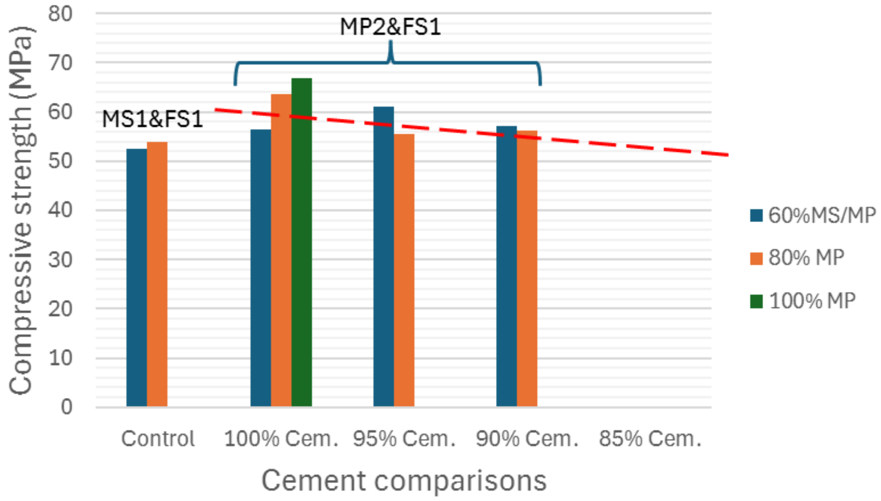


Figure 6: Comparison of cementitious contents for different fine aggregate combinations

**RECOMMENDATIONS**

Several characterisation methods for fine aggregate can be done at the concrete plant, providing quality assurance data; these include:

- Moisture content of fine aggregate that shows elevated moisture levels may sometimes be linked to dirty materials that drain more slowly
- Grading can be undertaken at the production point using standard sieve analysis in accordance with NZS 3111 Section 6
- Shape and texture can be assessed using the NZ sand flow test, which is easy to run with the correct equipment (see NZS 3111 Section 19)
- Cleanness can be assessed indirectly using the silt sedimentation test where high values can trigger further laboratory analysis (see AS1141.33:2015)

More advanced testing of fine aggregates is done either by specialist aggregate laboratories based at quarries or independent testing companies; including:

- Petrographic analysis to identify minerals and potential contamination of ultra fine materials such as clays and mica
- Physical property testing such as specific gravity, absorption and lightweight particles in fine aggregate
- More advanced methods of assessing cleanness using either sand equivalent (ASTM 3111 Section 18), clay index or methylene blue value testing

**CONCLUSIONS**

Laboratory trials of this nature need to be verified in full production since other factors have an influence in practice. Critics of this research might suggest compensation for poor manufactured sands can be made by increasing the dosage of water-reducing chemical admixtures. Poorer workability of concrete could be a significant issue and often

means that concrete mixes must be “softened” thereby reducing cementitious efficiency overall.

This paper shows the technical and economic advantage of investing in higher quality fine aggregate for concrete. Modern processed sands are available that not only reduce the amount of fine natural sand required in sand blends but also can lower cementitious content of concrete. Investing in quality aggregates should be considered as the first step in reducing the embodied carbon dioxide in concrete rather than the last. These improvements in manufactured sand have been shown to reduce overall cost of concrete and help lower environmental emissions.

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Appendix 1 – Grading analysis of fine aggregate materials (percentage passing)

| Sieve | NS | MP1 | MP2 | MS1 | FS1 | MS2 | FS2 | MS3 | FS3 | MS4 | FS4 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 4750 | 96.7 | 100.0 | 100.0 | 89.9 | 100.0 | 83.5 | 100.0 | 97.8 | 100.0 | 99.8 | 100.0 |
| 2360 | 72.9 | 94.1 | 96.2 | 56.1 | 100.0 | 50.4 | 99.9 | 65.0 | 100.0 | 71.0 | 99.4 |
| 1180 | 59.1 | 64.7 | 74.0 | 32.4 | 99.4 | 25.5 | 99.8 | 34.7 | 100.0 | 41.1 | 93.3 |
| 600 | 52.4 | 36.4 | 45.4 | 18.5 | 84.1 | 13.6 | 99.5 | 18.6 | 99.9 | 25.6 | 61.3 |
| 300 | 39.5 | 15.7 | 24.2 | 9.7 | 24.6 | 7.8 | 92.4 | 9.0 | 55.3 | 15.5 | 19.8 |
| 150 | 10.0 | 5.2 | 11.0 | 4.1 | 3.0 | 4.0 | 9.6 | 3.3 | 1.9 | 8.3 | 3.4 |
| 75 | 0.8 | 1.3 | 4.8 | 1.1 | 0.2 | 1.3 | 0.3 | 1.3 | 0.1 | 3.5 | 1.8 |
| 45 | 0.3 | 0.7 | 1.6 | 0.5 | 0.0 | 0.5 | 0.1 | 0.7 | 0.0 | 1.0 | 0.5 |
| Pan | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| FM | 2.69 | 2.68 | 2.49 | 3.89 | 1.89 | 4.15 | 0.99 | 3.72 | 1.43 | 3.39 | 2.23 |
| SG | 2.65 | 2.68 | 2.65 | 2.68 | 2.63 | 2.68 | 2.65 | 2.68 | 2.65 | 2.62 | 2.62 |

Notation: FM – fineness modulus, SG – specific gravity, wet sieving was done for fine fraction <150 micron

Appendix 2 – Concrete mix trials using manufactured vs modern processed sand

| Prop. | Unit | 60% MS1/40% FS1 Standard controls | | 60% MP2/40% FS1 | | | 80% MP2/20% FS1 | | | 100% MP2 |
|---|---|---|---|---|---|---|---|---|---|---|
| C. Agg. | kg/m$^3$ | 1025 | 1025 | 1025 | 1045 | 1060 | 1025 | 1045 | 1060 | 1025 |
| MS/MP | kg/m$^3$ | 485 | 485 | 485 | 485 | 485 | 485 | 485 | 485 | 485 |
| FS | kg/m$^3$ | 335 | 335 | 335 | 335 | 335 | 335 | 335 | 335 | 335 |
| Binder | kg/m$^3$ | 410 | 410 | 410 | 390 | 369 | 410 | 390 | 369 | 410 |
| WR | ml | 2900 | 2900 | 2900 | 2900 | 2900 | 2900 | 2900 | 2900 | 2900 |
| Water | L/m$^3$ | 187 | 190 | 173 | 177 | 175 | 170 | 178 | 175 | 172 |
| TW | kg/m$^3$ | 2442 | 2445 | 2428 | 2432 | 2424 | 2525 | 2433 | 2424 | 2427 |
| w/b | ratio | 0.456 | 0.463 | 0.422 | 0.454 | 0.474 | 0.415 | 0.456 | 0.474 | 0.420 |
| Slump | mm | 100 | 120 | 110 | 145 | 140 | 120 | 150 | 135 | 130 |
| HD | kg/m$^3$ | 2435 | 2440 | 2440 | 2450 | 2445 | 2455 | 2440 | 2440 | 2455 |
| $f_7$ | MPa | 43.3 | 41.5 | 51.9 | 49.6 | 43.9 | 51.6 | 42.7 | 44.1 | 52.4 |
| $f_{28}$ | MPa | 54.0 | 52.6 | 56.5 | 61.1 | 57.1 | 63.6 | 55.5 | 56.1 | 66.8 |

Appendix 3 – Comparison of concrete made with natural or modern processed sands

| Material / Property | Units | NS | MP1 | MP2 |
|---|---|---|---|---|
| 13 mm stone | kg/m$^3$ | 1100 | 1100 | 1100 |
| Sand | kg/m$^3$ | 835 [NS] | 835 [MP1] | 835 [MP2] |
| Cement | kg/m$^3$ | 350 | 350 | 350 |
| Water reducer | ml/m$^3$ | 1750 | 1750 | 1750 |
| Total water | L/m$^3$ | 158 | 172 | 166 |
| Water/cement | Ratio | 0.450 | 0.491 | 0.474 |
| Slump | mm | 120 | 100 | 100 |
| Workability | Subjective | Good | Slightly harsh | Ok |
| Hardened density | kg/m$^3$ | 2422 | 2395 | 2405 |
| 7D strength | MPa | 47.0 | 43.1 | 45.0 |
| 28D strength | MPa | 65.0 * | 56.0 | 59.5 |
| 28D Porosity | % | 8.61 | 10.31 | 9.57 |

Note * strength adjusted based on ISO-CEN strength comparison

# SILICOMANGANESE FUME USE AS AN SCM IN CEMENTITIOUS COMPOSITES: MECHANICAL BEHAVIOR AND LEACHING CHARACTERISTICS

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## SUMMARY

The construction industry continually seeks sustainable alternatives to improve concrete performance while minimizing environmental impact. One key strate-gy involves the use of supplementary cementitious materials (SCMs). SCMs are used in conjunction with Portland cement, resulting in improved concrete properties and helping to reduce carbon footprint. Different SCMs used in cementitious composites include silica fume, metakaolin, silicomanganese fume (SMF), fly ash, and slag. These substances are either pozzolanic or hydraulic, reacting with portlandite (a cement hydration product) to form secondary hydration products. The utilization of SCMs in concrete can improve strength, durability, and sustainability by modifying its microstructure, hydration products, and pore characteristics. SMF, a byproduct of the steel-making industry, is produced in millions of tons every year and has been used readily in the construction industry in recent years. However, there is potential harm to inhabitants due to the presence of heavy metals, like Manganese, in SMF. In this study, the leaching characteristics of Manganese from SMF-based cement paste samples have been evaluated. SMF was used to replace cement in different pro-portions (10%, 20%, & 30%), and the resulting mechanical properties and leaching characteristics were studied by compressive strength testing and ICP-MS analysis at different intervals over three months. The results show that there is a minimal amount of leaching within two days of leaching and no leaching over the long run. The compressive strength results show that there is a decrease in strength with increasing SMF content over 20wt%.

## INTRODUCTION

The foundation of the concrete industry is cement as a binder. Approximately four billion tons of cement are produced worldwide annually [1]. The current trends indicate that by the end of this decade, this figure is expected to surpass current levels and by the year 2050 the demand for concrete is expected to go beyond 18 billion tons per year [2,3]. It is responsible for huge CO2 emissions, posing environmental challenges. One of the methods to reduce associate carbon emissions is the use of supplementary cementitious materials (SCMs)[4]. SCMs are used either as a partial replacement of clinker in cement or a partial replacement of Portland cement PC while mixing the concrete [5]. A variety of materials are used as SCMs such as blast furnace slag, fly ash, silicomanganese fume, steel slag, biomass ash [6,7]. A significant proportion of the materials currently used as SCMs are by-products of other industries such as silica fume from the steel-making industry and fly ash from coal-fired power plants [8].

SMF is a manganese-rich by-product of the steel industry. SMF is generated during the production of ferroalloy which is used as a deoxidizing agent in the steel industry. The

Manganese (Mn) is used in the steel industry to remove sulfur and oxygen when iron ore is converted into iron and further iron to steel [9]. The steel industry uses almost 93% of the Manganese produced worldwide [10]. Approximately 100 kg of SMF is generated for the production of every ton of Mn alloy as per the recent reports [11,12]. In recent years SMF has been used as a partial cement substitute during concrete mixing and as the clinker because of its pozzolanic properties and ultrafine particle size in the range of micro to nanoscale [13–15]. The SMF is also being used in road construction, and drilling fluids as reported by researchers [12,16,17]. A few studies exist on the use of SMF as a SCM in the production of Self-compacting Concrete (SCC) [18], used alone or in conjunction with other Supplementary Cementitious Materials (SCMs) such as Ground Granulated Blast Furnace Slag (GGBFS) to produce sustainable Alkali-Activated Binders and alkaline-activated mortars[19,20]. The SMF has also been incorporated in concrete in different ratios to reduce concrete permeability and increase compressive strength[21].

In this study, the mechanical properties of concrete incorporating SMF were incorporated for various weight fractions. We explored the effect of various proportions of SMF on the microstructural characteristics and strength of concrete. These analyses were done before and after the leaching process to investigate the change in microstructure leading to a possible increase or decrease in strength with various proportions of SMF. The compressive strength of concrete is a critical factor for the integrity of structure and the addition of SMF has been assumed to contribute positively to this property. We measured the compressive strength of SMF-incorporated paste samples and compared them to controlled specimens.
Furthermore, the possible leaching of Mn could affect both the physical and chemical properties of the concrete. Therefore, this study assesses the extent to which leaching influences the strength and microstructure and highlights the SMF's performance as an SCM. The outcomes of this investigation are expected to yield significant insights into the optimization of concrete mix designs for enhanced durability and environmental safety, contributing to the broader field of sustainable construction materials.

## MATERIALS AND METHODS

A specific gravity of 3.15, ASTM C-150 Type I OPC manufactured by Saudi Cement Hofuf Plant was used to prepare the samples. SMF was obtained from the Jubail, Saudi Arabia-based alloy company SABAYEK. The particle size distribution of SMF and OPC with mean particle sizes 0.10456µm, and 30.349µm, and D50 median particle sizes 0.06213 µm, and 21.674 µm respectively. The chemical composition of SMF is shown in Table 1. SMF chemical composition.

MnO and $SiO_2$ as the major constituents. **Error! Reference source not found.** depicts an X-ray diffractogram of raw SMF highlighting the crystalline nature of SMF with Quartz ($SiO_2$: PDF#2009: 01-088-2488), Manganese ferric oxide ($MnFe_2O_4$: PDF#2009: 01-085-1202), Sylvite (KCl: PDF#2009: 00-004-0587) as the distinguished phases.

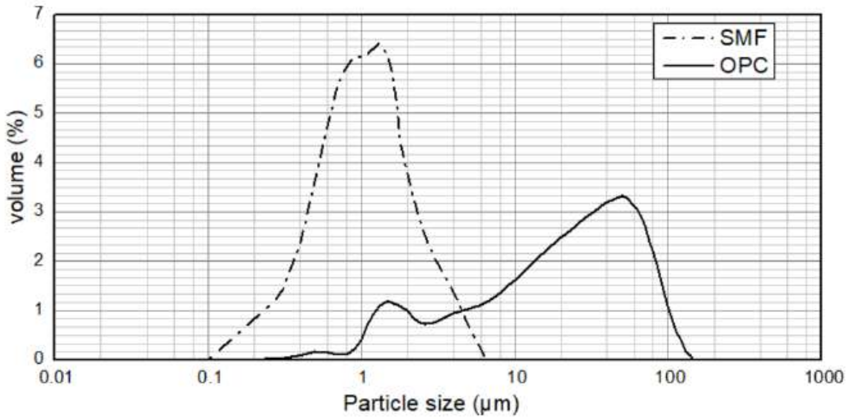


Figure 1.Particle size distribution of SMF and OPC

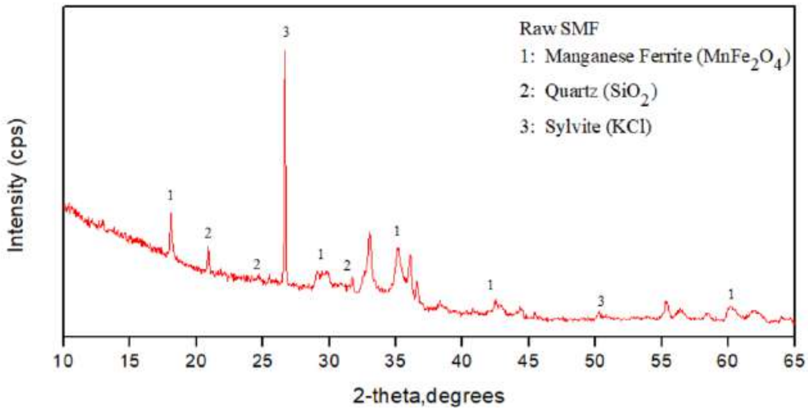


Figure 2. Raw SMF, X-ray diffractogram

Table 1. SMF chemical composition.

| **Chemical composition** | **$SiO_2$** | **$Al_2O_3$** | **$Fe_2O_3$** | **CaO** | **MnO** | **$K_2O$** | **$SO_3$** | **$P_2O_5$** | **Cl** |
|---|---|---|---|---|---|---|---|---|---|
| Wt (%) | 11.93 | 1.028 | 3.293 | 4.603 | 20.3 | 0.907 | 0.355 | 0.344 | 0.258 |

Four distinct paste samples were prepared with different ratios of OPC to SMF as 100:0, 90:10, 80:20, and 70:30. Based on the trial mixes, the W/B ratio used was 0.3.

## EXPERIMENTAL DETAILS AND RESULTS

Compressive strength testing

Digital CTM (compression testing machine) MATEST C55 with a loading rate of 1KN/s was used to test the compressive strength of the samples following ASTM C109/ C109M [22]. The compressive strength was determined at 28 days and after 90 days of the leaching test.

Compressive strength results

Figure 3 illustrates the average 28-day and 90-day strength with and without leaching of the samples prepared. The trend shows that the compressive strength at 28 days with partial replacement of cement up to 20% by SMF remains almost the same as that of the control. However, as the SMF content increased beyond 20%, a gradual drop was noticed from 77 MPa in 20% to 55 MPa in 30% replacement which can be observed from the

which aligns with the study reported [18]. This drop could be because of the slower rate of hydration of SMF resulting in an incomplete rate of reaction (un-hydrated SMF) at 28 days [20,23]. Furthermore, it can also be depicted that due to the high fineness of the SMF, the water demand increases, leading to a decrease in strength. In addition, due to the higher surface area of SMF, the cement paste needed to make a strong matrix is less, which could also cause a loss of strength in higher substitution of cement by SMF.

Figure 3 also depicts a comparison of compression test results at 90 days after curing with and without leaching samples. As expected, the compressive strength in concrete increased over time in samples without leaching due to the slower rate of hydration of SMF and the additional formation of CSH [23]. maximum strength obtained was 94 MPa in 20% partial cement replacement. However, the strength decreased in leaching samples compared to non-leaching samples in all the cases. This could be because there is a possibility of leaching Ca ions from the cement matrix leading to the non-formation of additional CSH gel which is otherwise responsible for the concrete strength gain.

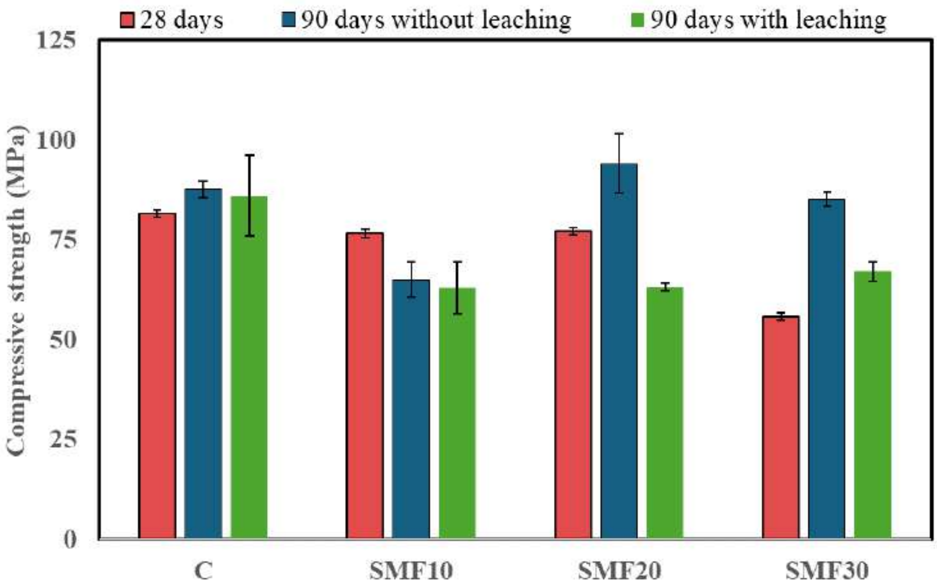


Figure 3 Compression test results 28 days and 90 days after curing with and without leaching

## CONCLUSIONS

In this study, the leaching characteristics of Manganese from SMF-based cement paste samples have been evaluated. SMF was used to replace cement in different proportions (10%, 20%, & 30%), and the resulting mechanical properties and leaching characteristics were studied by compressive strength testing and ICP-MS analysis at different intervals over three months. The results show that there is a minimal amount of leaching within two days of leaching and no leaching over the long run. The compressive strength results show that there is a decrease in strength with increasing SMF content over 20wt%.

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# Traceability and Compliant Reinforcing Steels

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## SUMMARY

The reinforcing steels placed in our concrete structures are essential in ensuring the safety and resilience of these structures. With an increasing supply of reinforcing products imported to supplement local manufacturing, there is an elevated necessity to ensure that all products meet the standard. The material standards AS/NZS 4671[1] outlines the minimum requirements for these steels, including chemical properties, material properties, geometric properties and requirements for product traceability. The standard also identifies what additional measures are required through the manufacturing of products and downstream processing.

The standard underlines the importance of traceability and ongoing testing during the manufacturing and processing of reinforcing products. Engineers and procurers must understand the critical requirements for determining if the reinforcing product delivered to the site is compliant. This paper will detail the requirements for reinforcing materials per New Zealand standards and how downstream actions like bending and cutting scheduling processes alter material properties. It will also guide engineers and procurers on what they need to do to ensure the delivery of compliant materials and what checks should be conducted upon product delivery.

The paper will also shed light on the latest global methods for digital traceability and product verification at a batch level. These methods offer a significant advantage, providing specifiers, designers, and certifiers with the confidence that the delivered product meets the specified requirements. This not only ensures compliance but also streamlines the verification process.

## INTRODUCTION

Reinforcing bars are supplied to building projects in several forms; these include straight bars with diameters from 10 mm to 50 mm, which are typically provided by manufacturers to steel processors in fixed lengths of up to 12 m. For the smaller bar sizes and up to 20 mm in diameter, the supply may be through continuous coils with weights up to 5 tonnes; the use of these coils by processors improves efficiency through automation and waste reduction. The other significant component of reinforcing material is fabric or reinforcing mesh; in New Zealand, both earthquake grade meshes (E Grade), with a pitch of 200 mm and non-structural mesh (L Grade), with a pitch of 150 mm are supplied.

The New Zealand building code recognises the importance of safety-critical materials and ensuring the use of compliant reinforcing materials in structures. Consequently, the New Zealand/Australian reinforcing material standard AS/NZS 4671 is referenced in both the New Zealand Building Code — B1 Structure [3] and NZS 3101 Concrete Structures standard [2]. In both documents there is a particular emphasis on achieving a level of ductility expected for structures in seismic regions.

The reinforcing material standard AS/NZS 4671 addresses several issues that impact the performance of reinforcing products within concrete structures. These include strength and ductility, which are critical, but other factors that contribute to the behaviour and

quality of the material, such as chemistry, geometry, surface profile and traceability are also identified with the standard specifying particular requirements. The reinforcing standard also recognises that through the manufacture and supply of the product to the industry, there are multiple stages of processing and different levels of compliance testing required.

**COMPLIANCE TO AS/NZS 4671**

Section 9 of the standard references the requirements for demonstrating Product conformity through the Normative Appendices A and B.

Appendix A outlines the criteria for testing along with a flow chart for testing showing the process required when using either Long Term Quality (LTQ) or through the testing of batches to demonstrate compliance. Appendix A also details the initial testing required to verify product compliance when products are first placed in the market, Type Testing. Appendix B details the type and frequency of testing for the various stages of supply. The standard requires the implementation of a Factory Production Control (FPC) system for the manufacturer or processor; these systems, while possibly based on an AS/NZS ISO 9001 system, have a particular focus on the products being produced, ensuring procedures are consistent, with the regular tests and assessments of the product, being used to control the input materials, ensuring that the product is consistently complying. The testing outlined in the appendices cover the complying requirements as detailed in Section 7 of the Standard. These include chemical composition, mechanical properties, geometric properties and surface geometry.

The standard outlines the minimum sampling and frequency of testing to demonstrate compliance. It should be noted that the method and rate of testing are dependent on the product being manufactured or the process being undertaken, whether manufacturing bar or coil, decoiling and straightening bar), or manufacturing mesh.

The standard also specifies that reinforcing products must be identifiable back to the steel producer through the use of roll marks unique to the bar manufacturer and site; additionally, labels placed on the product need to also identify the supply. As a result, the reinforcing material being delivered to the site is fully traceable.

With product being traceable and compliance testing mandatory, what does and how does the certifying engineer or the building official need to do to ensure the material's compliance with the standard? What test certificates are required? What are the minimum requirements? What other paperwork is required?

Fortunately, AS/NZS 4671 clearly sets out these requirements, which basically fall under five categories: Chemical Composition, Mechanical Properties, Geometric Properties, Surface Geometry, and Identification.

**Chemical Composition**

When the chemical composition of the reinforcing bar is compliant, the material is deemed to be weldable under the conditions outlined in the New Zealand welding standards (AS/NZS 1554.3). The compliant material has a carbon equivalent below a defined percentage which varies depending on the strength grades. The materials also have limitations on the percentages of Carbon, Phosphorus, and Sulphur.

The chemical composition of the material is defined at the point of manufacture with no changes occurring during hot rolling or processing. To demonstrate compliance a test certificate from the manufacturer of the material is required. This certificate should be checked to ensure the levels of chemistry as outlined in the standard are not exceeded.

**Mechanical Properties**

Two of the key requirements for the design of reinforced concrete structures are the strength and ductility of the reinforcing bar. The strength enables the determination of the ultimate capacities of structural elements, while the ductility is an important requirement for the structures post-ultimate behaviour. The ductility provides post-ultimate performance and prevents the brittle failure and collapse of structures when exposed to situations where the post-ultimate behaviour is critical, including earthquake zones.

The ductility and strength of steel are primarily defined through the chemical composition. However, further manufacturing processes of the steel from billet to the final bar can also have a significant effect. When a billet of steel is reheated and passed through a rolling mill to produce the plain or deformed bar, the process in particular the cooling alters the grain structure of the material, and in conjunction with the chemical composition can significantly impact both the ductility and the strength. These processes vary, using processes such as quenched and tempered used for Tempcore will produce a different material behaviour to the tempered microalloy bar used for straight lengths or spooled onto coils for later decoiling.

The stress-strain relationships for two hot rolled products shown in Figure 1, the first, lower graph, is a 300 E grade bar and the second a 500 E grade reinforcing bar. The stress-strain curve for the 300-grade product has similarities to mild steels with a defined yield plateau prior to strain hardening and significant elongation to the ultimate tensile strength. These steels typically have excellent ductility with a high elongation at maximum force ($A_{gt}$). The use of microalloying materials (for higher strengths) has a beneficial effect on the strength of the material through the addition of alloys. However, this increase in strength affects the ductility of the product. This variation is shown with the stress-strain for the 500E grade micro alloyed bar, upper graph. When compared to the 300-grade, the yield plateau is less pronounced, with yield strength typically determined as $f_{y0.2}$, being the measured stress at 0.2 % elongation. Additionally, the total elongation is smaller, with $A_{gt}$ dropping to around 15 %.

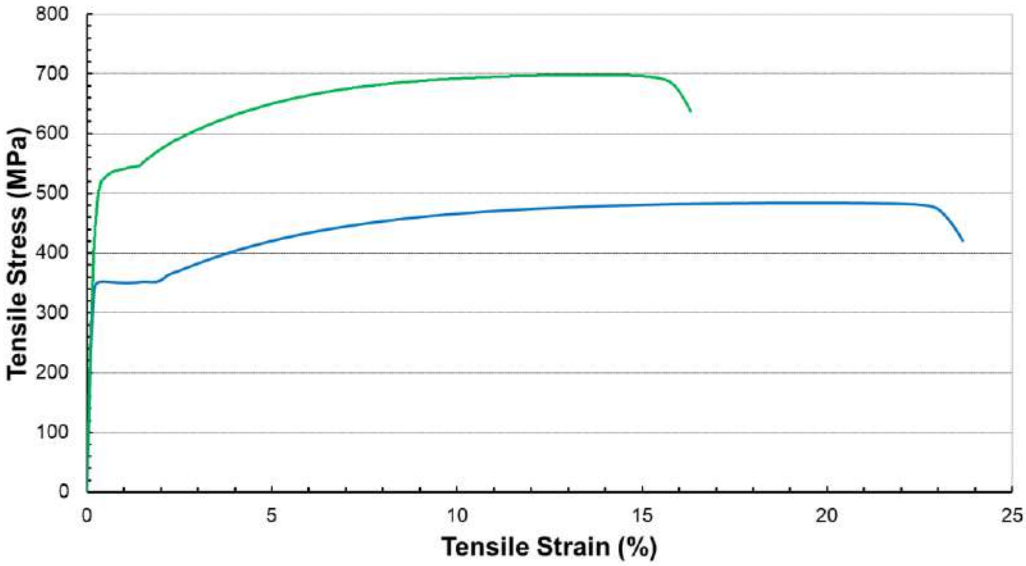



*Figure 1 Typical Stress-Strain Relationships*

AS/NZS4671 outlines the strength requirements and recognises the importance of the ductility requirements required for safe, durable structures. For New Zealand, where the default is the use of sesmic or E-grade reinforcement, higher minimum uniform elongation

requirements than for normal reinforcement are specified (greater than 15% and 10% for 300E and 500E, respectively). Additional requirements on the strength see the lower characteristic ratio of the tensile strength ($R_m$) to yield strength ($R_m$) being limited to at least 1.15, but also that it is no higher than 1.4 for the upper characteristic values for the 500E grade.

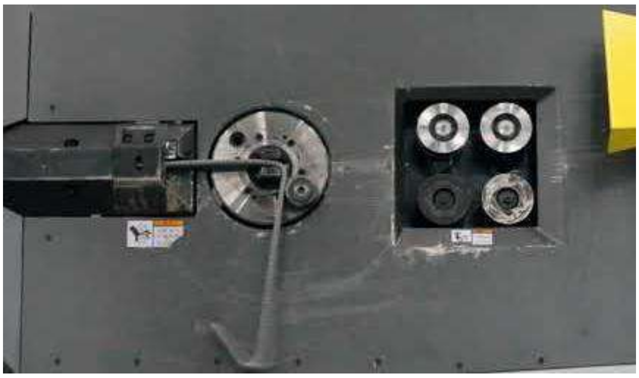
*Cold Working*

Another process utilised in the manufacture of reinforcing is cold working. This process involves taking a rod, and reducing the diameter by cold rolling or drawing through a die to achieve a desired strength and profile with minimal product heating. Raw materials (rods) are typically produced with strengths near 350 MPa with good elongation. However, the process involves taking the material beyond its nominal yield stresses and deforming it plastically. A prime example of this is the manufacture of wire for mesh; in this case, a rod with a nominal diameter of 10 mm is placed through several dies and drawn down to 6.3 mm wire. This cold work significantly increases the strength of the product but also reduces the ductility. In such cases, it is possible to have a 350 MPa steel increase to a nominal 500 MPa steel with the ductility ($A_{gt}$) reducing from in excess of 10% down to 1.5%.

While the example above is an extreme situation it illustrates how cold working significantly affects strength and ductility, and why 500L grade material is used for non-structural purposes. It is an example to highlight the effects of cold working steel products. Through the decoiling and straightening of coiled reinforcement the material is actually being cold worked and there is a measurable impact on the strength and ductility.

*(a) Coiled Reinforcing Product*

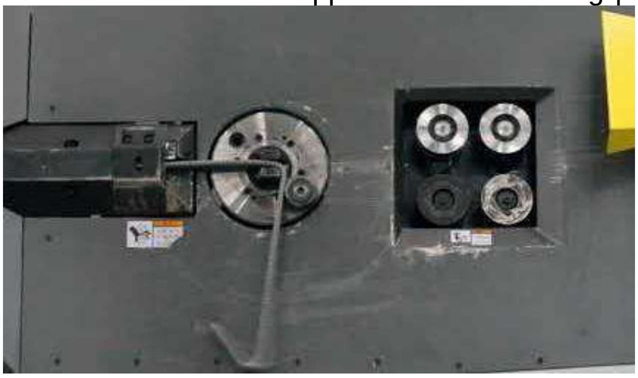
*(b) Typical bar straightening process*



*(c) Typical bar straightening process*

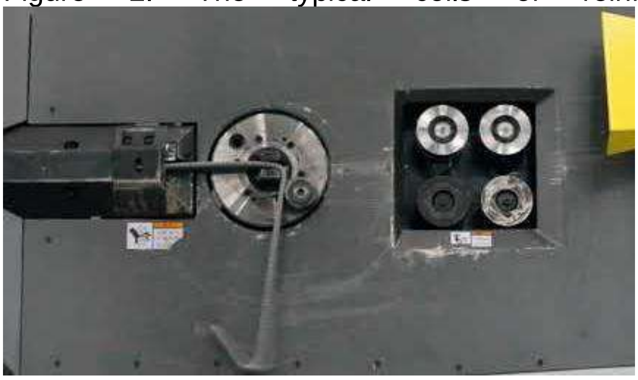
*Figure 2 Decoiling, straightening, bending*

To understand what happens in the decoiling process some of the steps are shown in



(c) Typical bar straightening process

Figure 2. The typical coils of reinforcing products are shown in



(c) Typical bar straightening process

Figure 2(a), the coil is then passed through a series of straightener rolls Figure 2(b) providing either straight bar or passed through to the bender Figure 2(c), that automates the bending of products to scheduled shapes. So this process is plastically deforming (cold working) the material; it is normal to see an increase in the product's strength, with the corresponding decrease in ductility. The magnitude of these strength and ductility changes is a function of the work carried out on the product; the quicker the bar is straightened, the more energy is exerted and the higher the impact on the material properties. For this reason, the standard requires that processors utilising decoilers to straighten bars and form shaped components have a Factory Production Control (FPC) system and the associated test program to monitor long-term data and demonstrate that the product remains compliant.

As ductility and strength are key in the behaviour of our structures, the standard specifies a minimum amount of testing that is required for all reinforcing products, looking at numerous aspects, including the yield stress ($R_e$), tensile stress ($R_m$), and elongation ($A_{gt}$) of the products. The Standard also recognises the effect of cold work on products resulting from processors and stipulates additional testing requirements for cold-worked products.

*Long term quality*

The standard outlines the frequency and testing requirements for the mechanical properties through the manufacturing control process. The ongoing assessment of these results using statistical principles is a proactive tool that a manufacturer or processor uses to monitor trends in the manufacturing process, and it is referred to as Long-term Quality (LTQ) in the standard. This process should form part of any factory control process as an

essential part of the quality system, but understanding the material inputs and processes is essential to determine compliance.

While the standards require the LTQ data to be supplied, it is important to understand that it is a statistical tool and not necessarily the best tool for determining the compliance of a batch of material. To this end, the Standard specifies requirements for both batch and LTQ compliance. Cases have been presented where the LTQ data has indicated that the process is non-compliant, but all testing has passed the thresholds nominated in the standard. Consequently, care should be taken when rejecting a product for non-compliant long-term quality.

**Geometric Properties**

The geometric properties required to demonstrate compliance involve checks on nominal diameters of the cross-sectional areas of the products being produced. These nominal diameters and areas are defined in the standard. With these parameters being set during the hot rolling of the product, the testing requirements of the standard are limited to the point of manufacture and not a requirement of the processing facilities. Additionally, the straightness of the product needs to meet a minimum requirement to be compliant, which needs to be demonstrated as the straightness of the bar.

**Surface Geometry**

Surface geometry for the deformed steel bars includes the geometry of the ribs or indentations required to achieve the bond with the concrete. The standard outlines specific tests to maintain a minimum profile and projected area of the ribs, thus ensuring the product supplied is compliant. As wear on the rollers in the roll former process directly impacts the rib profiles, regular testing for surface geometry is a requirement at the point of manufacture.

If not monitored closely the decoiling process while possibly making the ductility non-compliant it may also affect the rib profiles. Consequently, the standard requires the processor taking product from coils to conduct regular testing of rib profiles, thus ensuring the product supplied to the site is still compliant. Figure 3 shows a portion of a processed bar supplied to the site with significant damage to the rib profiles due to the straightening process, this product was being supplied to projects with the processor being unaware of the non-compliance.

*Figure 3 Damage to Rib profile due to decoiling*

**Mesh**

The standard also recognises different compliance requirements for manufactured mesh products and specifies some production tests and inspections for mesh products. These include assessing manufacturing methods, chemical composition for weldability, mechanical properties, and surface geometry, as outlined above. Additionally, the code requires the mesh manufacturer to undertake production testing of the shear strength of the welds in the mesh and ensure the number of bars and spacing of the product are compliant.

**Identification and Certificates**

The standard is very specific about the requirements for identifying reinforcing products and the required certificates for the associated product. The deformed bar must have a series of surface marks that identify the strength grade and ductility. Additionally, all reinforcing bars must have marks to identify the steel producer. This requirement is waived for plain 250 N and plain 500L grade products, typically drawn products.

In addition to the bar markings, each coil or bundle of steel supplied requires the attachment of durable labels providing details such as steel producers/processors' names, types of products being supplied, heat/batch numbers, and the number mass or quantity of any bundle.

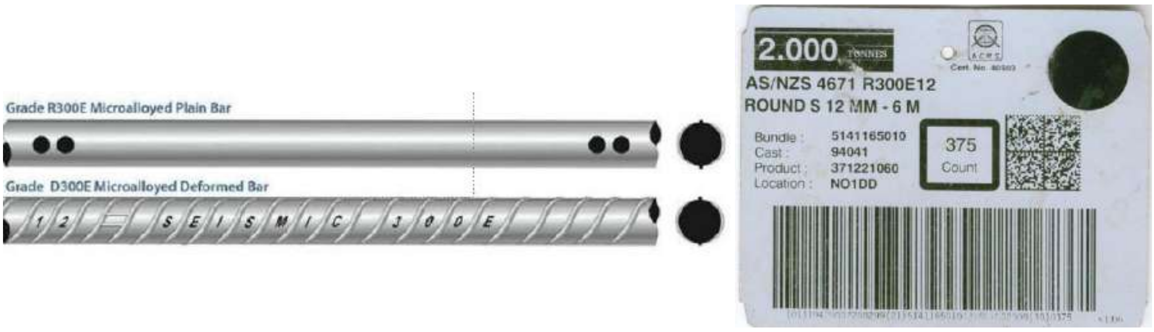



*Figure 4 Bar Markings and Labels*

Typical bar markings for a Local manufacturer are shown in Figure 4 along with the associated label attached to the product. Hence, when the product arrives, the receiver can use the tag to trace the material back to the original heat. In the above case, a round 300E 12 mm diameter bar is supplied with a given bundle number. The receiver should also check the markings on the bar and, in this case, should expect to see the dot-dot configuration on the product.

With the bundle number, there is an associated set of documentation that includes all the chemical and mechanical test results and the associated geometry and surface checks to demonstrate the product's compliance with the standards. The diligent receiver can now check that the product is, in fact, compliant.

**THIRD-PARTY INDEPENDENT CERTIFICATION**

It is recognised that the compliance checks required for each bundle of reinforcing may be onerous, and the staff in receipt of the delivery may not fully understand the intricacies of compliance and what should be checked. To assist with this and improve efficiency on site, the New Zealand Building Code allows for industry-based schemes to assess manufacturers/processors to the relevant standards to demonstrate compliance.

Schemes such as the Australasian Certification for Reinforcing and Structural Steels (ACRS) meet the requirements of being this type of industry body. Independent of the manufacturers, ACRS technical experts carry out annual audits at the manufacturing sites

for all certificate holders. These audits review the Factory Production Controls, observe steel-making and intermediate processing operations, undertake random independent testing, ensure and test the product traceability systems and review the ongoing submissions of the long-term test data. The scheme is also accredited by JASANZ and recognised internationally for its rigour and independence. Within this scheme, the certified manufacturers/processors are publicly listed with their associated scope of certification; the certificates include the bar marking and examples of labels provided on bundles.

By specifying the ACRS certification and ensuring the corresponding product is supplied and delivered, the designer, builder and end user can have confidence that the producer of the product has the quality and manufacturing systems in place that have been verified and continue to provide products that are compliant to the standard.

**DIGITAL TRACEABILITY**

While traceability has been a requirement of the reinforcing standard, it has been a paper-based system that can be cumbersome and difficult to collate documentation and maintain links from products to the associated certificates. This process is further complicated as more processors are introduced into the supply chain. Additionally, the rise of falsified documentation adversely affects confidence in paper-based certification systems.

In recent years there has been an increase in the use of digital systems for tracing products and ensuring safety, particularly in industries such as the automotive and food industries. With recent issues in building safety in the UK and other jurisdictions, coupled with work being carried out under the UN Centre for Trade Facilitation and eBusiness concerning traceability and international trading, there has been a significant push to introduce digital product certification into the construction industry. This need for traceability has been further enhanced by the need to supply environmental credentials with building products and the recognition that a material with an unknown source has unknown environmental credentials.

Consequently, we are seeing certification schemes such as ACRS move to a more digital platform to provide, firstly, confidence in the authenticity of the certification certificates, and secondly, the transition of the traceability systems from the paper-based system tried and tested digital medium.

**CONCLUSION**

The New Zealand reinforcing standard outlines the requirements for suppliers to demonstrate compliance to meet the New Zealand Building Code. These requirements include implementing factory production control systems along with the associated test programs to ensure that the chemical composition, mechanical properties, geometric properties and surface properties are all compliant. The checks required are detailed with a reasonable level of understanding of the industry required to verify all conditions have been met.

These requirements to demonstrate compliance of product are not only limited to the steel manufacturers, but also on the downstream steel processors to ensure that the compliance testing for mechanical properties and surface geometry are undertaken and that the traceability of product to the site is maintained. When ordering and importantly when receiving the product, the receiver or those responsible for compliance sign-off need to ensure that all testing and compliance checks have been conducted.

The building code allows for the use of accredited industry organisations to verify compliance, ACRS accredited by JASANZ is recognised as an international expert in the

certification of reinforcing and steel products to the Australasian and New Zealand standards. Consequently, ACRS is able to provide confidence to the specifiers and end users that the certified manufacturers/processors have the systems in place to produce compliant products.

While traceability has been a requirement of the standard to ensure that the product is traceable at a batch level, back to the point of manufacture. This traceability aspect is becoming an important compliance issue in the construction industry as traceability and compliance with building products are becoming new requirements coupled with ESG requirements.

Designers and specifiers should specify safety-critical components such as reinforcing steels with recognised accredited third-party certification, and on delivery, this accreditation should be demonstrated for the products. Systems exist for reinforcing steels.

**References**

[1] Standards Australia & Standards New Zealand. (2019), Steel for the reinforcement of concrete (AS/NZS 4671:2019) SAI Global.

[2] Standards New Zealand (2006). Concrete Structures Standard (NZS 3101.1:2006), Standards New Zealand

[3] Ministry of Business Innovation & employment. (2021) Acceptable Solutions and Verification Methods – B1Structure (Amendment 21)

# LOWER EMBODIED CARBON DESIGN WITH CONCRETE BRIDGES

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## SUMMARY

With carbon reduction being a major focus in infrastructure design work, finding efficient ways to design with lower embodied carbon has become a priority. With that in mind, a benchmarking exercise was undertaken to carry out life cycle assessments on a sample of bridges to find any observations that could assist efforts to reduce embodied carbon in design.

The sample comprised 32 bridges, primarily in the Auckland and Waikato regions, with two others in the Central North Island and two in the South Island. Structure types included concrete, steel only and steel-concrete composite bridges. Total structure lengths ranged from 18m to 305m, with span lengths between 12m to 60m. Life cycle assessments on each structure were conducted covering modules A to D, following the MBIE Whole of Life Embodied Carbon Assessment Technical Methodology Guidance (MBIE, 2022).

The benchmarking results demonstrated the benefits of using super tee and hollow core beams were not only economic but also resulted in less embodied carbon in general. This paper explores whether existing standard super tee and hollow core beams can be further optimised, and whether the use of alternative prestressed beams can lead to further reduced embodied carbon.

It was also observed that substructure arrangements had a significant impact on the embodied carbon, with multiple-column substructures tending to be more carbon-efficient than monopile substructures. Other ways to reduce embodied carbon were also explored, such as optimising the crosshead width in integral bridges, or the use of post-tensioning.

Reinforcing steel was also a significant factor increasing embodied carbon. The lack of recycled reinforcing steel available in New Zealand means a higher carbon intensity compared to imported steel. International guidance has recommended adding more reinforcing material to reduce embodied carbon (Hilton, 2022). In New Zealand this guidance is less applicable, with concrete being a less dominant carbon contributor at present.

## INTRODUCTION

This paper presents observations from a bridge Life Cycle Assessment benchmarking exercise which was conducted with the aim of identifying potential embodied carbon hotspots within bridge design and using these to improve future design. Life Cycle Assessments were conducted for each bridge, and data analysis has been performed to compare the embodied carbon performance between bridge superstructure and substructure types.

This exercise was not conducted for a specific project. This allowed the use of standardised assumptions to follow a principles-based approach meaning the findings can be applied to a range of future projects. Most of the bridges included within the study are

considered typical bridges which may be encountered on future Roads of National Significance type projects, or standalone highway bridge structures. The bridges were also designed following an optioneering process to determine the most cost effective and appropriate solution given the site constraints.

**ASSESSMENT METHODOLOGY**

The 32 bridges included within the benchmarking exercise sample were chosen from a sample pool of WSP projects. They have been constructed within the last 15 years, mostly within major roading infrastructure projects primarily in the Auckland and Waikato Regions, as well as individual bridges located in the North and South Islands. The bridge locations are shown in the location map in Figure 1 below.


Figure 1: Map of Sample Bridge Locations

The sample included a range of bridge types as summarised in Figure 2 below. These bridges range in total length from 18m to 305m, with spans between 12m and 60m. All except for one are road bridges spanning over road, water and North Island Main Truck Rail (NIMTR). The bridges include both piled and spread footing foundations. A range of bridge superstructure types were including in the exercise including concrete, steel-

concrete composite, steel only and one timber bridge.

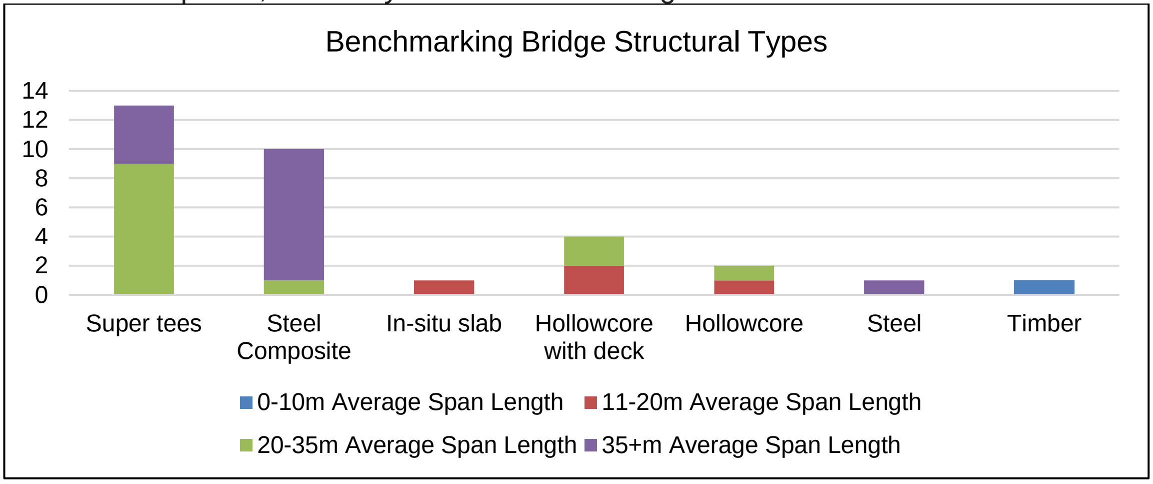



Figure 2: Benchmarking Bridge Structural Types

For each bridge, life cycle assessments were conducted in line with the MBIE "Whole-of-Life Embodied Carbon Assessment Technical Methodology (MBIE, 2022). A quantity take-off of the bridge materials was completed and collated into an in-house embodied carbon calculation spreadsheet. The in-house embodied carbon spreadsheet contains Environmental Product Declaration (EPD) data for the typical construction materials involved in the construction of bridges. Within the in-house spreadsheet the quantities were converted into embodied carbon by element, measured in kilograms of $CO_2$ equivalent.

For the purposes of this benchmarking exercise, assumptions were established to assist with the standardisation of the results to make the findings more transferrable to future projects. The use of standard assumptions was preferred as the purpose of the benchmarking exercise was for identifying and opportunities for efficiency rather than collecting formal embodied carbon assessments or applying the benchmarking directly to a specific project.

The critical assumptions were as follows:

- Normal concrete used on all projects,
- Concrete and concrete beams locally sourced,
- Reinforcement steel locally sourced, manufactured in New Zealand,
- Medium carbon structural steel,
- Structural steel (plates, pile casing) imported overseas and transported from Napier to the project site,
- Reinforcement rates were used rather than a full reinforcement quantity take off to standardise and simplify the quantity take off.

## ASSESSMENT RESULTS AND TRENDS

The completed life cycle assessments were normalised in line with the PAS 2080 British Standard for carbon management in buildings and infrastructure (BSI,2023) for ease of comparison. For bridge structures, the normalisation is a rate of embodied carbon per square metre of functional area (i.e. the width between bridge barriers). This allowed the comparison of bridges of the range of span and total bridge lengths included within the benchmarking exercise. The Life Cycle Assessment included Modules A through D;

however, Modules B and C were found to be minor compared to Module A. Hence, the following results compared Module A results only.

The results have been organised into the superstructure and substructure types for clarity.

<u>Superstructure Observations</u>

The benchmarking exercise found Module A footprints ranged from 1,078kg$CO_2$-e/m$^2$ to 2,725kg$CO_2$-e/m$^2$, as shown in Table 1 below. These footprints included all bridge elements, so impact of the superstructure and substructure is reflected in the overall footprint.

Table 1: Average Module A Embodied Carbon by Superstructure Type

| Superstructure Type | Number of Structures | Mean average embodied carbon (kg$CO_2$e/m$^2$) | Median average embodied carbon (kg$CO_2$e/m$^2$) | Lowest embodied carbon (kg$CO_2$e/m$^2$) | Highest embodied carbon (kg$CO_2$e/m$^2$) |
|---|---|---|---|---|---|
| **Super tees** | 13 | 1,298 | 1,191 | 1,078 | 2,487 |
| **Steel-concrete Composite** | 10 | 1,849 | 1,659 | 1,306 | 2,725 |
| **In-situ slab** | 1 | 2,333 | 2,333 | 2,333 | 2,333 |
| **Hollowcore with deck** | 4 | 1,570 | 1,502 | 1,412 | 1,863 |
| **Hollowcore** | 2 | 1,457 | 1,457 | 1,200 | 1,714 |
| **Steel only (footbridge)** | 1 | 1,994 | 1,994 | 1,994 | 1,994 |
| **Timber** | 1 | 149 | 149 | 149 | 149 |

The superstructure types which had more than one example included within the benchmarking exercise are shown in Figure 3 below. As shown below, super tees and hollowcore (with no deck) superstructures have lower embodied carbon by functional area. An in-situ concrete deck tends to increase the embodied carbon values of hollowcore bridges, compared those tied together with transverse post-tensioning. But that is sometimes necessary to accommodate barrier loading while keeping structural depth low.

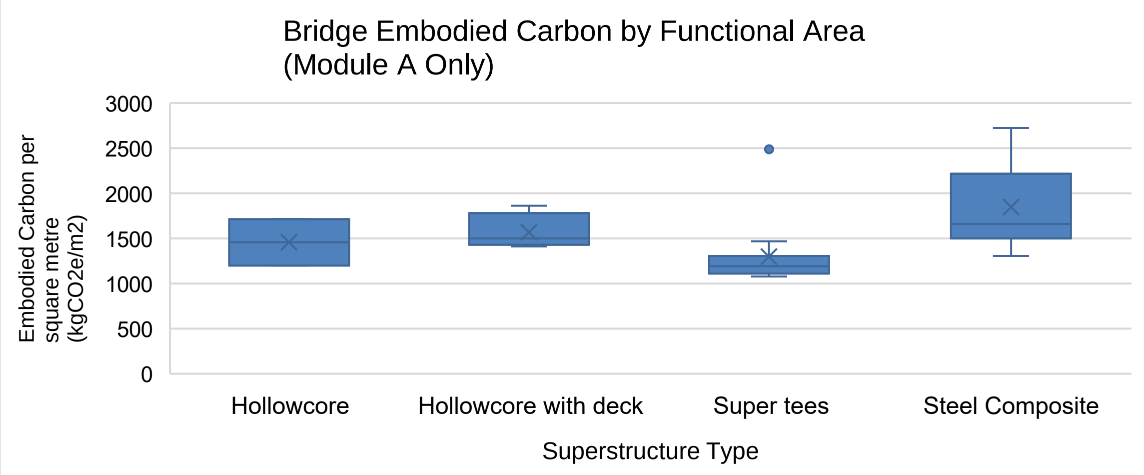



Figure 3: Bridge Embodied Carbon by Functional Area

From the bridges assessed within this exercise, bridges with steel-concrete composite superstructures had higher embodied carbon when compared to the super tee or hollowcore superstructures. When the Life Cycle Assessment Boundaries were extended to include the end of life and recycling of elements (Module D), the embodied carbon in the steel-concrete composite superstructures is reduced; however, this remains higher than the super tee and hollowcore alternatives. It should be noted than in all of the bridges included within this exercise, the steel-concrete composite bridges were only used when the span lengths or site constraints required it, such as long clear span over waterway or ecological sensitive area to satisfy consent conditions, or over a railway line where it's undesirable to have an intermediate support.

The results show there is a wider range of carbon performance when designing steel-concrete composite bridges depending on the efficiency of the design. Continuous steel bridges were found to have lower embodied carbon than multi-span simply supported steel bridge options.

Substructure Observations

The foundation solution greatly affects the bridge's total carbon values, as shown in Table 2 below. In theory, a lighter superstructure should result in a smaller substructure, the reality is that the substructure is greatly influenced by the foundation type. For example, the founding level of the piles are similar regardless of the foundation load being considered and are more governed by the need to reach a suitable founding layer.

Table 2: Average Module A Embodied Carbon by Substructure Type

| Substructure Type | Number of Structures | Mean average embodied carbon ($kgCO_2e/m^2$) | Median average embodied carbon ($kgCO_2e/m^2$) | Lowest embodied carbon ($kgCO_2e/m^2$) | Highest embodied carbon ($kgCO_2e/m^2$) |
|---|---|---|---|---|---|
| **Shallow foundation** | 4 | 1,104 | 1,214 | 149 | 1,841 |
| **Top driven pile** | 4 | 2,339 | 2,561 | 1,510 | 2,725 |
| **Bored pile** | 17 | 1,412 | 1,306 | 1,078 | 2,080 |

| | | | | | |
|---|---|---|---|---|---|
| **Bored pile (secant pile wall)** | 1 | 2,333 | 2,333 | 2,333 | 2,333 |
| **Bottom driven piles** | 6 | 1,487 | 1,534 | 1,095 | 1,714 |

This was particularly apparent for bridges in the Waikato Expressway, where 40m long driven piles were used. The use of permanent steel pile casings contributes greatly to Module A, and as the casing cannot be retrieved for recycling, the negative carbon values from recycling (Module D) cannot be taken into account.

In Auckland, bridges tended to have shorter bored piles as the rock is located at a higher level. This reduced the embodied carbon in the foundations for the bridges located in Auckland when compared to the Waikato region. Similarly, where shallow foundations were feasible, these structures had significantly lower overall embodied carbon compared to those with deep piled foundations.

Limitations

This benchmarking exercise did not include many bridges located within a high seismic zone. Further research would be required to verify whether these observations hold in high seismicity locations, in particular when considering the additional requirements with the updated seismic hazard model information.

**DEEP DIVE INTO DATA**

The benchmarking results demonstrated the impact of steel on the embodied carbon in a bridge structure. Whether the superstructure was a prestressed concrete girder type or a steel-composite girder type and concrete substructure, the dominant contributor to carbon emissions was the steel from reinforcement or the structural steel in the girders. The steel and concrete contributions (by volume) for the most common bridge types within the sample, super tee and steel-concrete composite, and their embodied carbon contribution are summarised in Figures 4 to 7 below.

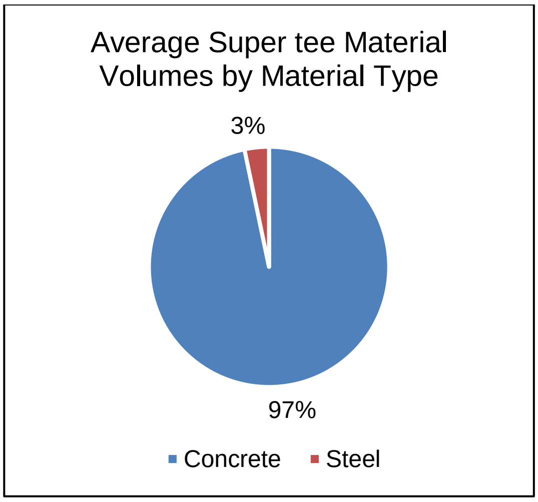



Figure 4: Super tee Volume Comparison by Material

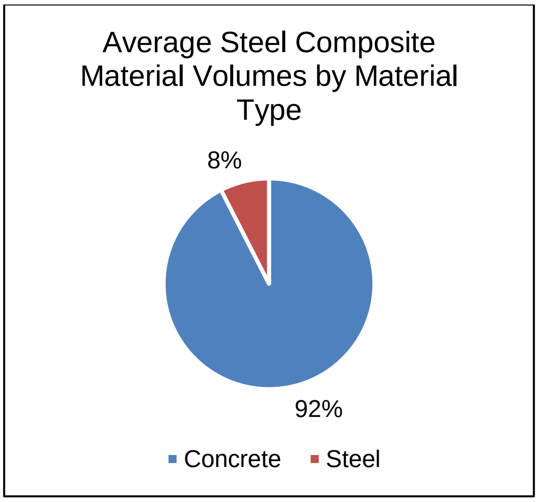
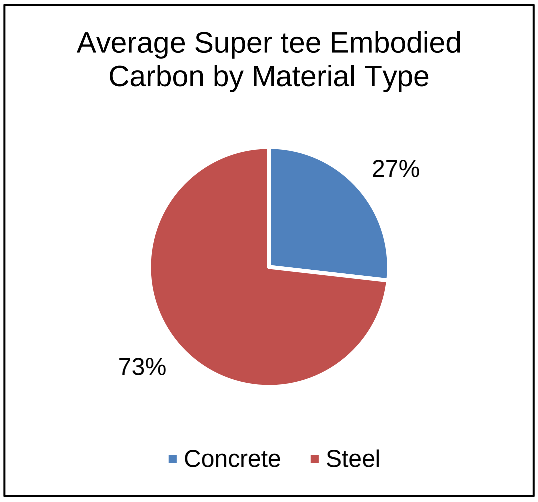



Figure 5: Steel-Concrete Composite Volume Comparison by Material



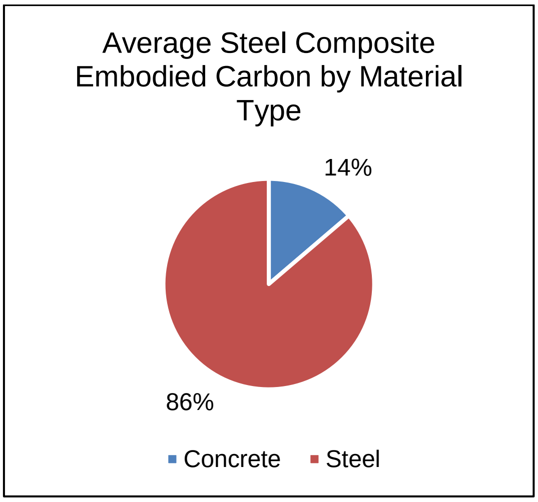



Figure 6: Super tee Embodied Carbon Comparison by Material

Figure 7: Steel-Concrete Embodied Carbon Comparison by Material

Across the 13 super tee bridges included in the study, the average volumetric contribution of concrete was 97% with the remainder comprising steel reinforcement, and steel pile casings in some cases. When considering the embodied carbon from these structures, however, 73% of the embodied carbon was provided by the steel with the remaining 27% coming from the concrete.

Similarly, for a steel-concrete composite superstructure, the average volume of concrete within the structure remained high, at 92%, with the remainder a combination of structural steel and reinforcing steel. For these structures, an average of 86% of the embodied carbon was contributed by the steel components.

These results demonstrate the influence of steel reinforcement in the embodied carbon in concrete bridge girders. The steel girders are the primary contributor to the embodied carbon in steel girder bridges.

Reinforcing steel in New Zealand is manufactured from virgin material meaning higher carbon intensity than imported steel which uses recycled scrap steel (NZ Steel, 2023). It should be noted, international guidance (Hilton, 2022) has recommended adding more reinforcing material to reduce embodied carbon due to higher carbon concrete abroad. In New Zealand, this guidance is less applicable, with concrete being a less dominant carbon contributor at present. Concrete in New Zealand is also generally ahead of international standards when considering Global Warming Potential across a range of mixes and locations (Firth, 2020, Allied, 2019) against the Infrastructure Sustainability Council Materials Calculator Base Case (IS Council, 2023).

The impact of the carbon intensity in materials was also demonstrated when comparing the embodied carbon by element type. These comparisons and illustrated in Figures 8 to 11. When considering a super tee superstructure type, the contribution of the superstructure, substructure, foundation and miscellaneous elements was similar when comparing the material volumes and embodied carbon. For a steel-concrete composite structure, however, when compared by element, the superstructure (deck and girders) contributed less volumetrically, and more through embodied carbon.

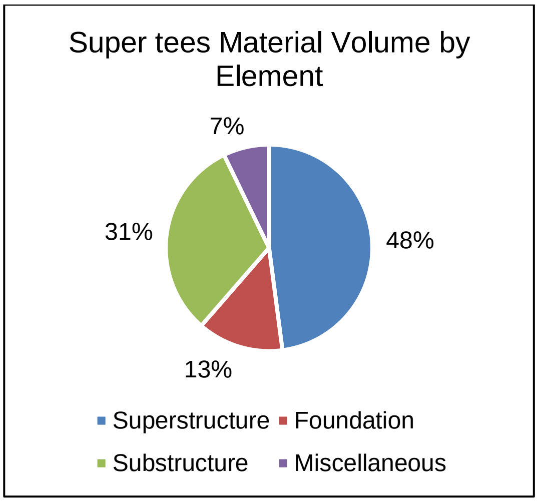



Figure 8: Super tee Volume Comparison by Element

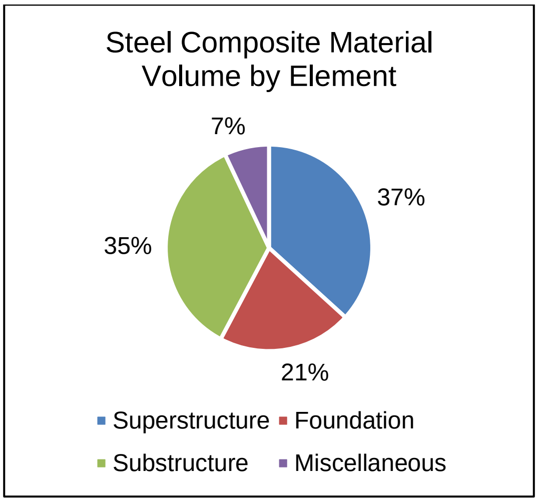



Figure 9: Steel-Concrete Composite Volume Comparison by Element

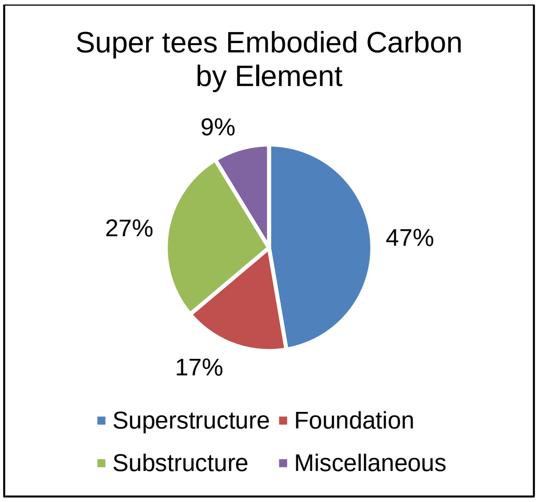



Figure 10: Super tee Embodied Carbon Comparison by Element

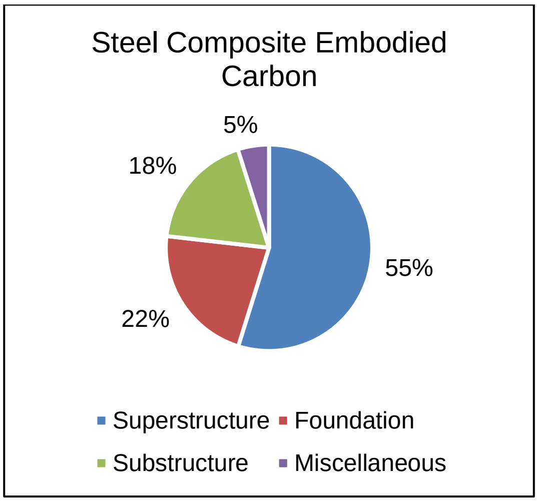



Figure 11: Steel-Concrete Composite Embodied Carbon Comparison by Element

This observation could assist with how carbon reductions should be targeted differently depending on the bridge type. In a typical concrete bridge, the element's carbon contributions correspond to their size, whereas structural steel elements are likely to dominate the embodied carbon of a steel-composite bridge. Inefficiently designed steel girders will have a significant impact on the embodied carbon of a steel-concrete composite bridge.

In substructure and foundation elements, the driving factors were the depth and type of foundation. In deep piled foundations, it is unavoidable to have increased material volumes which increase the embodied carbon regardless of the superstructure type. In structures where these piled foundations are top or bottom driven closed end steel shell piles, the embodied carbon in the structure is greater due to the need for permanent steel pile casings.

It was observed that the arrangements also had a significant impact on the embodied carbon, with multiple-column substructures tending to be more carbon-efficient than monopile substructures through smaller crosshead and smaller pile area. For example, two 900mm diameter piles have a cross-sectional area and volume, 12% greater than one 1200mm diameter pile; however, the pile circumference is 50% greater providing significantly more soil skin friction.

An additional observation in relation to the substructure elements was in regard to the crossheads. While the use of integral connections at the piers allows the superstructure to be designed more efficiently by allowing continuity and reduce maintenance by eliminating bearings and deck joint, the need to lap the continuity reinforcement from the girder does often lead to a much wider crosshead (~2.6m) than necessary which can contribute to the carbon footprint by a significant amount. Alternative method of lapping continuity reinforcement used overseas only require 2 to 4 feet (0.61-1.22m) as shown in Figure 12 below, this should be explored to optimise this.

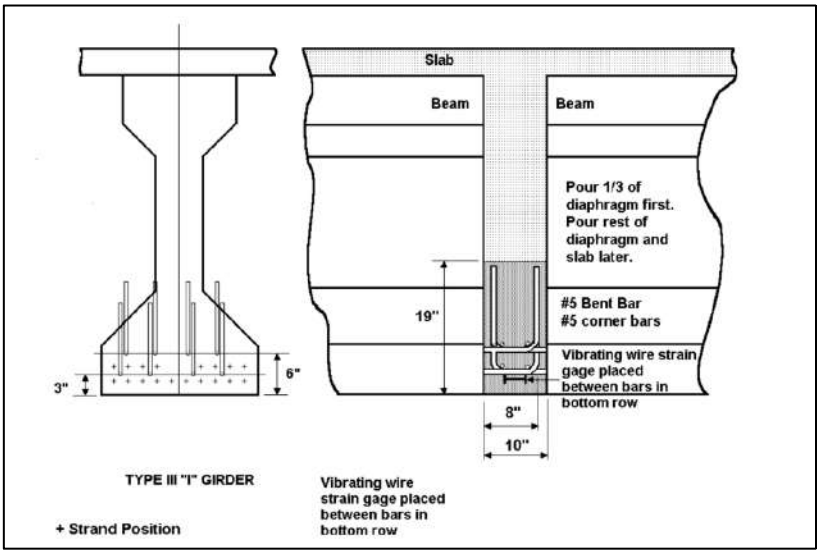


Figure 12: Bent Bar Diaphragm Example (Miller et al., 2004)

## FUTURE IMPROVEMENT

The purpose of this exercise was to identify what the main elements contributing to embodied carbon in bridge design and use this to guide future improvements to bridge engineering design principles which would embed low carbon in the design approach. The following observations have been considered as potential applications of the research findings.

### Supplementary Cementitious Materials

- A common strategy for reducing carbon within structures is the use of Supplementary Cementitious Materials (SCMs), in place of Ordinary Portland Cement (OPC) (Concrete NZ, n.d.). The results of this exercise demonstrate this won't be the silver bullet in low carbon bridge design. For common bridge types, where the contribution of embodied carbon from concrete was around 30%, focusing reduction efforts on the 30% will not be sufficient alone to achieve carbon emission reduction targets.

**Electric Arc Furnace Installation**

- Planned improvements to steel production in New Zealand are expected to positively impact the embodied carbon of bridge structures. The construction of an Electric Arc Furnace (EAF) by New Zealand Steel will significantly reduce the embodied carbon of reinforcing steel (NZ Steel, 2023). It would also increase the amount of steel which can be recycled on shore, further reducing the embodied carbon of future structures. Once the EAF is operational, updated guidance and assessment of the embodied carbon in bridge structures will need to be conducted with updated EPDs. This is likely to further improve the embodied carbon performance of prestressed concrete bridge types.

**Alternative Girder Types**

- There are alternative girder types, such as the NU and U girders which were not included within the benchmarking exercise as they have not yet been constructed in New Zealand. These are commonly used in North America and are starting to be used in New Zealand projects.

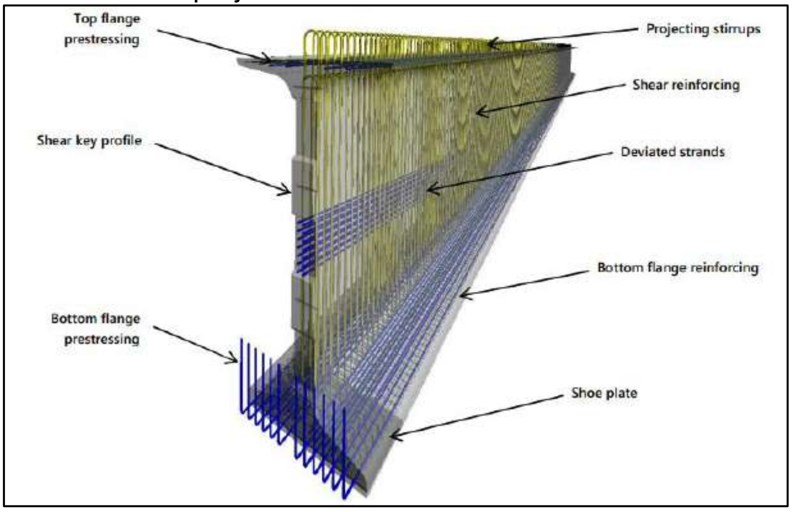



Figure 13: Prestressed NU Girder Features (Alberta Transportation Technical Services Branch, 2018)

These are more structurally efficient than super tee by having lower self-weight and through the combined use of pre-tensioning and post-tensioning they can span further than super tees (50m+), as shown in Figure 14.

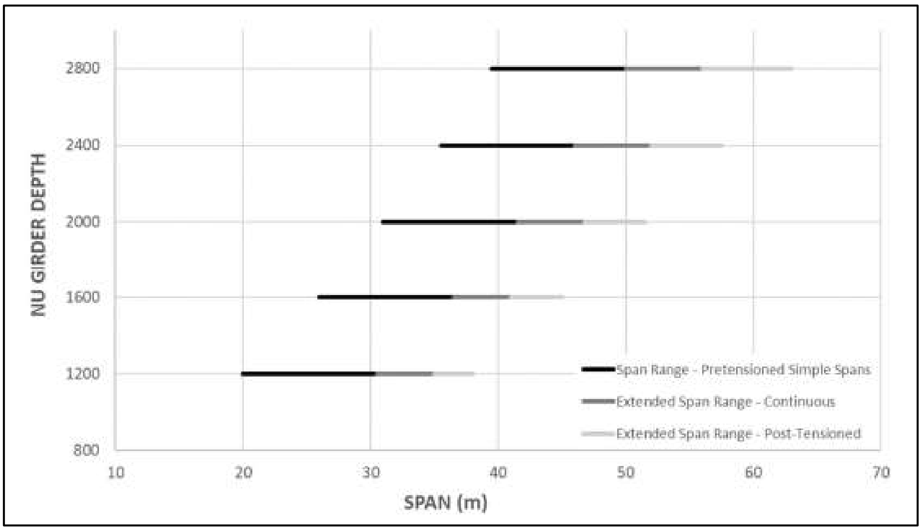



Figure 14: Typical NU Girder Span Range (Alberta Transportation Technical Services Branch, 2018)

The findings of this benchmarking exercise indicate these girders could be a low carbon alternative to steel composite girders for longer spans, on the basis of the rate of embodied carbon for prestressed concrete girders corresponding to their volume. However, they will require an upfront investment to create the mould, they have the trade-off of being more effort during construction to ensure stability of the girders and more temporary formwork needed to form the deck. These factors may decrease the uptake by contractors.

## CONCLUSIONS

This benchmarking exercise demonstrates the range of embodied carbon performance across a range of common bridge structure types. The purpose of the exercise was to identify design principles which could be applied to future projects to start with an informed perspective of the embodied carbon in typical bridge options. The exercise has found super tees to be particularly carbon efficient. It also found the impact of the foundation and substructure elements can outweigh even carbon efficient superstructures.

Future development opportunities within the bridge industry have also been identified. The current progress within the New Zealand steel industry to establish Electric Arc Furnaces is expected to significantly affect the carbon intensity of our future structures. The use of alternative girders is also expected to further improve the carbon efficiency of prestressed concrete bridges. The future design and construction of these should be encouraged as a low-carbon alternative to steel girders.

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# CONNECTING COMMUNITIES VIA POST TENSIONED CONCRETE - TE PAPA ŌTĀKARO AVON RIVER PRECINCT, NORTH FRAME PEDESTRIAN BRIDGE

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[1] Beca Ltd

## SUMMARY

As part of the Te Papa Ōtākaro / Avon River Precinct upgrades and beautification, Ōtākaro Limited had an objective to increase the walking and cycling connections in the central Christchurch riverfront area – one of the connections was a new bridge over the Ōtākaro Avon River linking the North Frame(Cambridge Terrace) to Victoria Square(Oxford Terrace).


Figure 1. Bridge location, Avon River Precinct

To assist in achieving these goals, Beca was engaged to complete the design of the North Frame Pedestrian Bridge from concept options through to Issue for Construction. Concept options were prepared with multiple material types and span arrangements. Concrete was selected by the client and stakeholders as the preferred material type for its limited maintenance, reduced operational costs and resilient design.

This paper will focus on the concrete asymmetric span arrangement and how a varying depth post-tensioned concrete section was specifically designed to reduce the overall concrete quantity, while achieving the Principal Requirements.

## INTRODUCTION

One of the Principal Requirements was to connect the new bridge path level to the existing path levels at Oxford Terrace and Cambridge Terrace whilst providing the flood/freeboard levels under the bridge and accessibility standards on the bridge and approaches. Figure 2 below shows the site prior to the bridge construction and the existing path on Oxford terrace. This requirement limited the available structural depth of the bridge as the maximum accessible grades limited the bridge height.

Figure 2. Bridge site

The selected bridge form is a 32m-long concrete post-tensioned 2 span bridge providing a 3.5m-wide walking and cycling crossing over the Ōtākaro Avon River and is supported on bored concrete piles.

**BRIDGE DESIGN OVERVIEW**

The 32m long bridge consists of 24m and 8m spans with a central pier on the edge of the river. The cast insitu concrete bridge deck is 4m wide but varies in depth from 900mm at the pier to 600mm at the abutments, as shown in Figure 3 and 4 below. The arched geometry was setout to provide the minimum freeboard requirements directly over the main Avon Channel.
Post tensioned concrete was selected to minimize the concrete bridge deck thickness allowing for an elegant asymmetrical span arrangement.

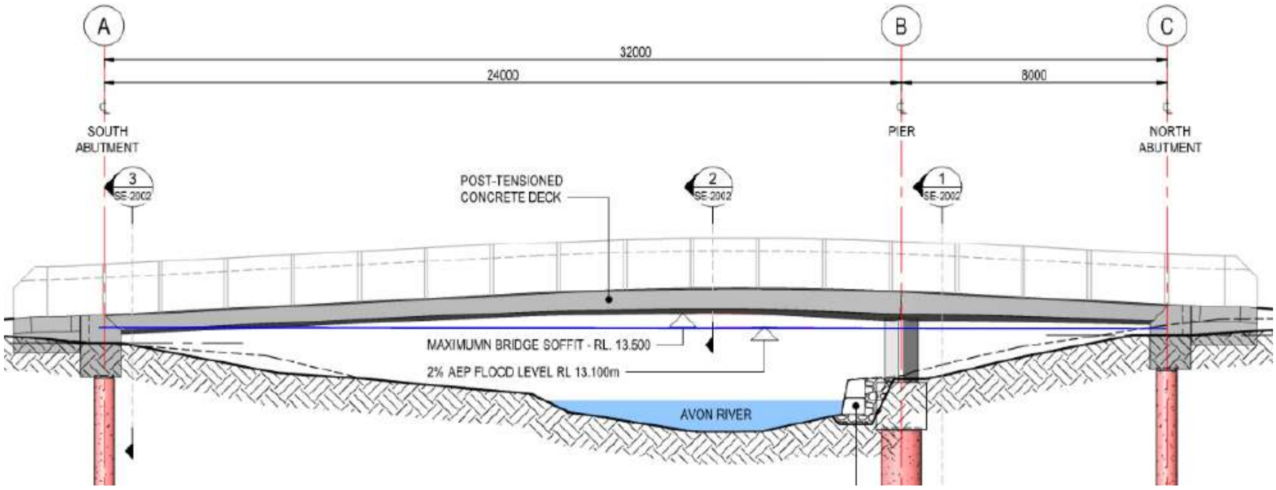


Figure 3. Bridge long section.

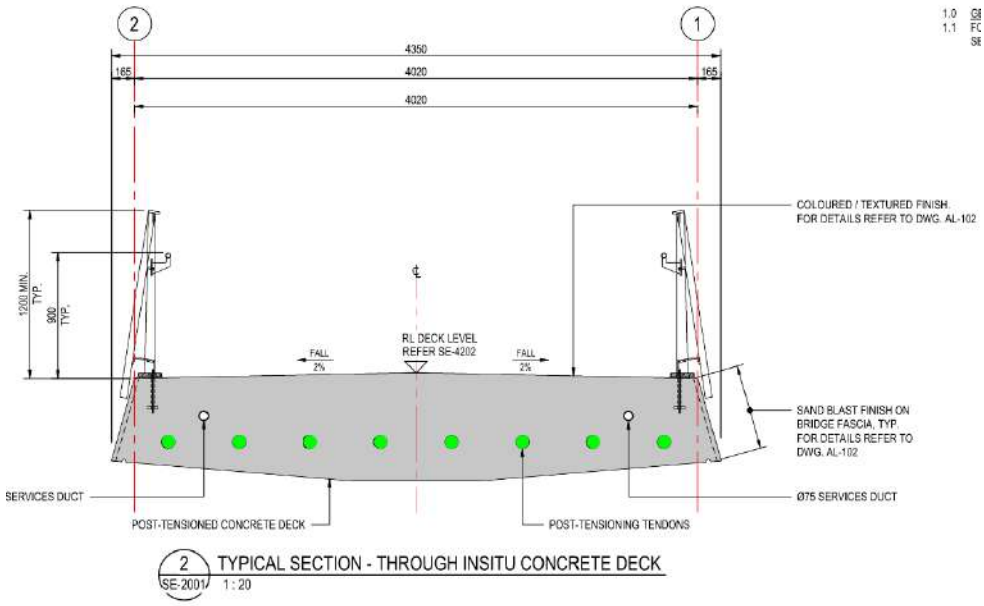


Figure 4. Typical bridge cross section.

## POST-TENSIONING SYSTEM

High-strength steel tendons stressed after curing put the variable depth concrete slab into compression, increasing stiffness and allowing the long, slender span. The 12No. 15.2mm low relaxation Super Grade 7 tendons run longitudinally through 8No. ducts cast in the box girder. Once stressed against the anchors, the post-tensioning resists bending, shear and live loads. Refer Figures 5 and 6 for the tendon profiles. The slight curve of the bridge deck and asymmetrical pier arrangement was designed to allow the tendon arrangement to be straight over the 24m span and curved over the pier. This tendon arrangement maximised

the positive bending capacity with the 24m span and the negative moments over the pier where the tendons were near the top of the concrete deck.

Figure 7 shows the tendon stressing underway.

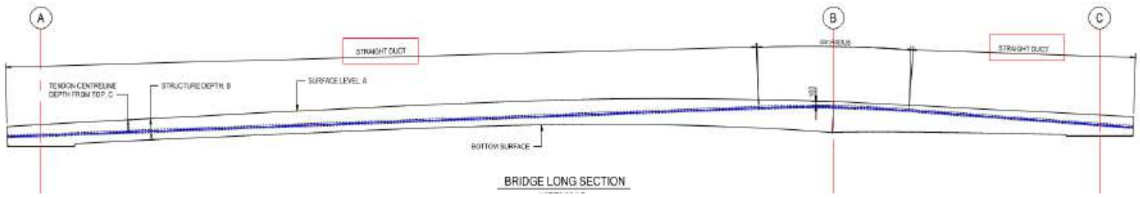


Figure 5. Overall tendon profile for the bridge.

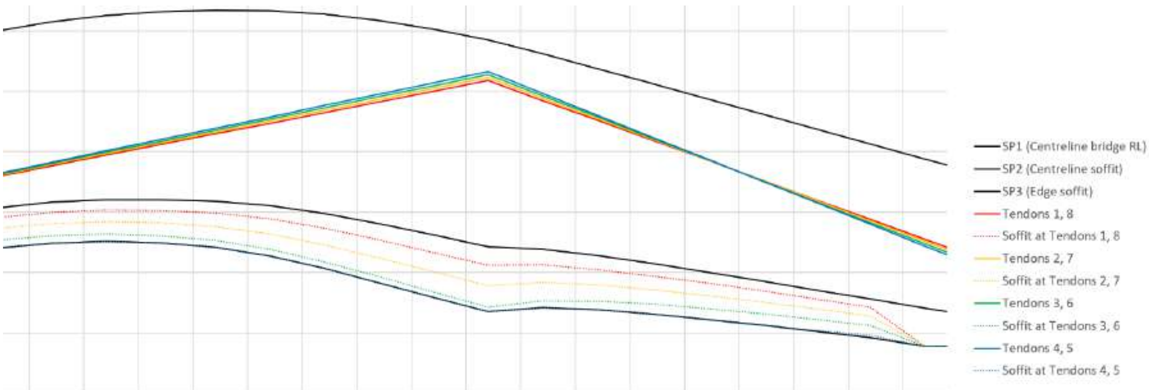


Figure 6. Partial long section showing tendon arrangement with respect to section depth (vertical axis exaggerated).

Figure 7. Tendon stressing.

## UNIQUE CONCRETE ARCHITECTURAL DESIGN FEATURES

The bridge incorporates cultural narratives through varied colored concrete textures and basalt stone disks designed in collaboration with Māori artists.

The bridge deck surface features two different concrete textured bush hammered finish representing the movement of the river beneath. Set within the textured walking surface of the bridge deck are 28 basalt stone disks expressing the cultural narrative of the Ōtākaro Avon river. The granite disks are etched with images of native plants and animals created by artist Piri Cowie (Kāi Tahu, Ngāpuhi, Ngāti Kahu) working with Matapopore Charitable Trust to share Ngāi Tūāhuriri pūrākau (stories). Figure 8 and 9 below show the textured surface and etched granite disks.

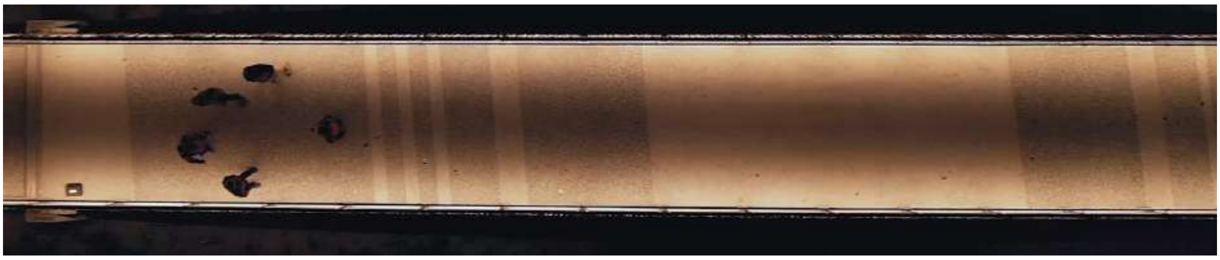

Figure 8. Bridge deck bush hammered surface

Figure 9. Etched granite disks inset

**Conclusion**

The post-tensioning and optimized asymmetrical form enabled an efficient and slim concrete bridge form. The bridge showcases innovative engineering combined with art to create a meaningful community asset.

The bridge is open and used by pedestrians and cyclists. It serves as a valuable asset for the city of Christchurch, acting as a unifying force that connects communities. Additionally, it serves as a stunning testament to a natural environment and the use of post tensioned concrete whilst encapsulating beauty and storytelling. Refer to Figure 10 for a picture of the completed bridge.

Figure 10. finished bridge

**ACKNOWLEDGEMENTS**

Rau Paenga (formally Ōtākaro) for enabling this project

# CONCRETE INDUSTRY APPRENTICESHIPS – STRATEGIES FOR SUCCESSFUL RETENTION AND COMPLETION

A BARKER[1] - G DURKIN[2] - P GILL[1] - C CAMERON[1]

[1] Scarlatti Ltd [2] BCITO

## SUMMARY

This paper provides early insight into research to answer whether withdrawal rates within trades apprenticeships (including those within the concrete industry) have changed in recent years; if so, by how much, and why. We provide a summary of the reasons for withdrawal, taken from both the perspective of apprentices and employers, and employer insights into recent events that may have changed the landscape. Finally, we offer an early glimpse into the opportunities that both training providers and employers can seize to obtain better outcomes for apprentices and the industry as a whole.

## INTRODUCTION

Quality training is vital to the concrete industry. Previous research has shown that, despite the initial time and financial investment required, training an apprentice returns a net positive value for employers due to the added knowledge capital and productivity that they bring (Hogarth and Kestle, 2014). However, a concerning trend has been identified by BCITO (the largest training organisation in New Zealand's concrete sector). In line with industry growth, BCITO's apprenticeship enrolment numbers surged from approximately 11,000 in 2016 to over 23,000 in 2022, along with increased numbers of apprentice withdrawals (non-completion of training).

A large percentage of apprentices complete their training with corresponding expressions of support for the positive actions of employers and BCITO. However, withdrawals from apprenticeships do occur. These can be for the right reason but can also be for reasons that were avoidable. If withdrawals have increased at a faster rate than enrolments in recent years, this is problematic for both BCITO and the industries it serves. Accordingly, BCITO undertook research between December 2023 and July 2024 to investigate this. The research uses a mixed methods approach to confirm whether the increase in withdrawal rate was more than the increase in enrolments, to investigate the reasons why apprentices withdraw, and to understand what events or changes in recent years may have caused an increase.

Previous literature indicates the reasons apprentices withdraw are complex. For example, Chan (2014) proposes that apprenticeships are, "*a socio-personal process requiring apprentices' agency and support from the social milieu (i.e., workplace, support agencies including ITOs, family, peers, etc.) within which apprenticeship learning occurs*." This implies that if an apprentice is to withdraw, it can be due to a host of reasons – some in control of the apprentice, the family, the education provider, and some the workplace. This is supported by a similar project carried out by Scarlatti for Te Pūkenga (New Zealand's largest vocational education provider) in 2023, which found that reasons for withdrawal were highly varied.

Several authors emphasize the role of the workplace. For example, Chankseliani et al. (2017) refer to the concept of 'occupational socialisation' in apprenticeships and argue

that their 'reliance on the active participation and support of employers' makes apprenticeships 'more vulnerable' than non-work-based learning methods. In another piece of research by Australia's National Centre for Vocational Education Research (NCVER) it was found that apprentices were most likely to withdraw for reasons relating to the employment component of their apprenticeship, such as 'experiencing interpersonal difficulties with employers or colleagues' or 'being made redundant' (Bednarz, 2014).

In this paper, we explore the early findings of this BCITO research, with a specific focus on the role of employers and actions they could take to improve outcomes for apprentices. We provide a brief overview of the methodology used in this work and then discuss the results. We will conclude with recommendations on how BCITO and employers can improve the retention of apprentices.

### Notes on Interpretation

A few notes should be considered in interpreting this paper:
1. This paper does not capture the entire scope of the research but rather focuses on the findings most relevant to employers.
2. The research was still in progress when this paper was written. Data and results should be taken as preliminary.
3. The definition of withdrawal used here is that an apprentice unenrolls from BCITO. This may or may not coincide with the apprentice leaving the employer. It also means that some apprentices considered withdrawn in this paper could have continued their apprenticeship with a different training provider.
4. This research has been undertaken across all BCITO trades, not just concrete. However, the findings are relevant to concrete and other construction industries

## METHODOLOGY

### Research Questions

This research aimed to answer:
1. Have BCITO withdrawal rates changed and, if so, by how much?
2. Who is most at risk of withdrawing?
3. Where do apprentices go after withdrawing?
4. Why do apprentices withdraw?
5. What may have caused a change in withdrawal rates in recent years?
6. What can be done to obtain better apprentice outcomes?

A mixed methods approach was used. Quantitative analysis was undertaken of data retrieved from Stats New Zealand's Integrated Data Infrastructure (IDI), as well as internal data provided by BCITO. Modelling of this data was undertaken to identify overall trends. This was followed by qualitative interviews conducted with BCITO employers and apprentices.

### Data Analysis and Modelling

Analysis of Stats NZ's Integrated Data Infrastructure (IDI) data and internal BCITO data was undertaken to better understand the types of BCITO apprentices that are

withdrawing, and where they go after withdrawing.[1]

For the IDI data retrieval, a new entrant is defined as a learner who started an apprenticeship with BCITO for the first time, and a withdrawn apprentice is defined as an apprentice who leaves BCITO without re-enrolling at a later date.

Neither IDI nor BCITO data provide a complete, current, and consistent record of withdrawals. To address these gaps, IDI data from 2015 to 2021 and BCITO data from 2021 to 2023 were combined to act as calibration data for a withdrawal rate model. The model brings out the overall trend from the data by extrapolating data gaps and smoothing noisy data points.

<u>Interviews</u>

Interviews were conducted with 80 apprentices and 40 employers between April and July 2024. Of the 80 apprentices, 40 had withdrawn, and 40 were either still in their apprenticeship or had completed. Within employer interviews, we discussed a total of 107 apprentices. We chose to include this range of interviewees to ensure a balanced view of apprenticeships.

Random sampling was used to contact potential interviewees, with an incentive provided for participation. The resulting interviewees were roughly representative of the actual distribution of learners across BCITO, in terms of trade, ethnicity, and age. However, in a few cases, purposive sampling was used.[2]

The interviews followed semi-structured interview guidelines, exploring what people enjoyed about apprenticeships, what they disliked, and why apprentices withdrew. Employers were additionally asked about changes they had seen over time that could impact withdrawal rates. The interviews were typically 30 minutes long and conducted by phone. An exception to this were interviews with Pasifika apprentices and employers, that used the talanoa method.[3]

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[1] Access to the data used in this study was provided by Stats NZ under conditions designed to give effect to the security and confidentiality provisions of the Data and Statistics Act 2022. The results presented in this study are the work of the authors, not Stats NZ or individual data suppliers.

These results are not official statistics. They have been created for research purposes from the Integrated Data Infrastructure (IDI) and Longitudinal Business Database (LBD) which are carefully managed by Stats NZ. For more information about the IDI and LBD please visit https://www.stats.govt.nz/integrated-data/

The results are based in part on tax data supplied by Inland Revenue to Stats NZ under the Tax Administration Act 1994 for statistical purposes. Any discussion of data limitations or weaknesses is in the context of using the IDI for statistical purposes, and is not related to the data's ability to support Inland Revenue's core operational requirements.

[2] This was done in some cases to increase the presence of certain groups when there was a hypothesis that a certain demographic would likely be more capable of commenting on. Specifically, to increase the proportion of withdrawn apprentices who were still with the same employer, withdrawn apprentices who were in the same industry but with another employer, and employers who hired their first apprentice after the start of TTAF.

[3] The talanoa method is an unstructured, typically face-to-face discussion process used mainly in the Samoan, Tongan and Fijian cultures. The focus is on developing relationships between people as part of a process "where people story their issues, their realities and aspirations, [that] allows more mo'oni (pure, real, authentic) information to be available for Pacific research than data derived from other research methods" (Vaioleti 2006).

To analyse the reasons, we primarily used constructivist grounded theory (Charmaz, 2017). For this research, this meant categorising themes where possible into existing frameworks (specifically, the reasons framework adapted from the previous Te Pūkenga work) or using existing hypotheses, but also identifying themes inductively (Scarlatti, 2023). Interviewers then estimated the relative importance of each theme based on the interviewees' comments, and estimated the effectiveness of each proposed intervention, depending on the analysis being undertaken.

## RESULTS

### Change in Withdrawal Rates

Modelling of IDI data and internal BCITO data was used to confirm whether the withdrawal rate had increased in recent years. This research found that the withdrawal rate doubled from 12% in 2020 to 24% in 2023. Prior to 2020, the withdrawal rate was relatively stable.

### Learner and Employer Attributes Associated with Withdrawing

Figure 1 and Figure 2 explore whether certain learners or employers see higher withdrawal rates than others. A surprising finding here is that there are few differences between groups, despite the widespread belief that factors such as gender and ethnicity make a difference. Where differences are seen, they are not large enough to suggest that a particular focus needs to be placed on learner or employer segments. A partial exception is the relationship between withdrawal rate and learner income. In this case, we suspect that the causality is more complex than saying low incomes drive withdrawals although this may play a role. For example, new apprentices are both more likely to withdraw and more likely to earn less.

The attribute found to be most strongly associated with withdrawals was whether an employer had previous apprentice completions.[4] Of all apprentices in 2021, 59% were with an employer who had no previous completions, while 68% of withdrawals in 2021 were with an employer who had no previous completions. This suggests that there may be other attributes (not captured here) that make some employers more conducive to completion than others.

[4] We do not comment here on income. This is because while income also appears to correlate with withdrawal, the direction of causality is less clear. It could be any or all of: people are more likely to withdraw early in their apprenticeship when pay is low; timing gaps between apprentices leaving jobs and showing up as withdrawn in BCITO data may show income as being artificially low; or low wages (relative to their peers) causes apprentices to withdraw. This will be explored in the remainder of this research.

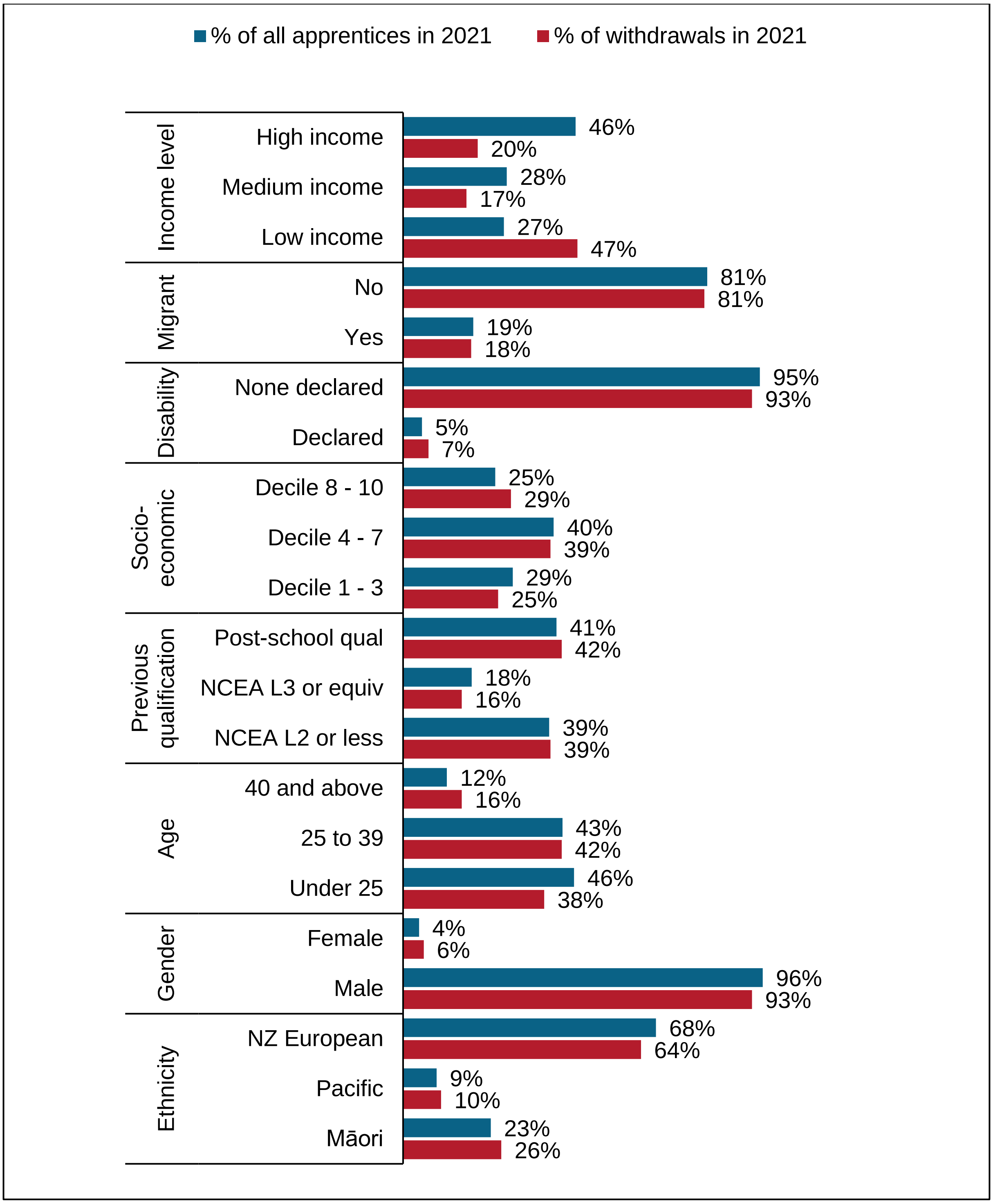


Figure 1. Learner Attributes Over-Represented Among Withdrawals

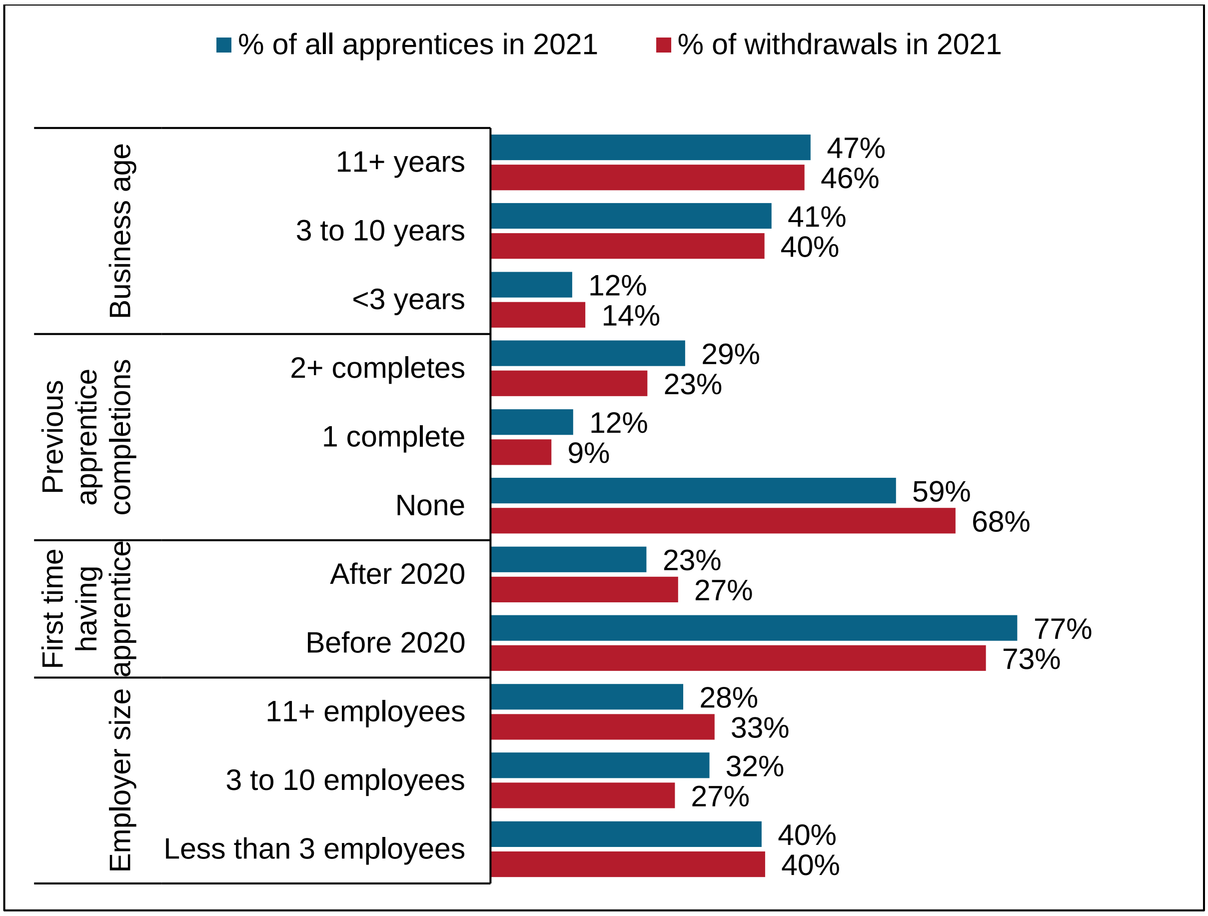


Figure 2. Employer Attributes Over-Represented Among Withdrawals

<u>The Reasons Apprentices Withdraw</u>

We spoke with apprentices and employers to understand their perspectives on what was going well, what was challenging, and why apprentices withdrew. Our first analysis was to categorise interviewees' reasons for withdrawal using a framework adapted from the Te Pūkenga project (Scarlatti, 2023). This original framework included four high-level categories (system, provider, employer, and personal factors). A few additional subcategories were identified during the interview process and added to this framework, as per the constructivist grounded theory approach.

Figure 3 shows that the reasons for withdrawal are diverse, aligning with previous research (Scarlatti, 2023). Several reasons cannot be actioned, either because they are in the control of the apprentice (including the one with the highest relative importance – personal circumstances), or they are outside of the apprentice, employer, or BCITO's control (typically those related to system factors).

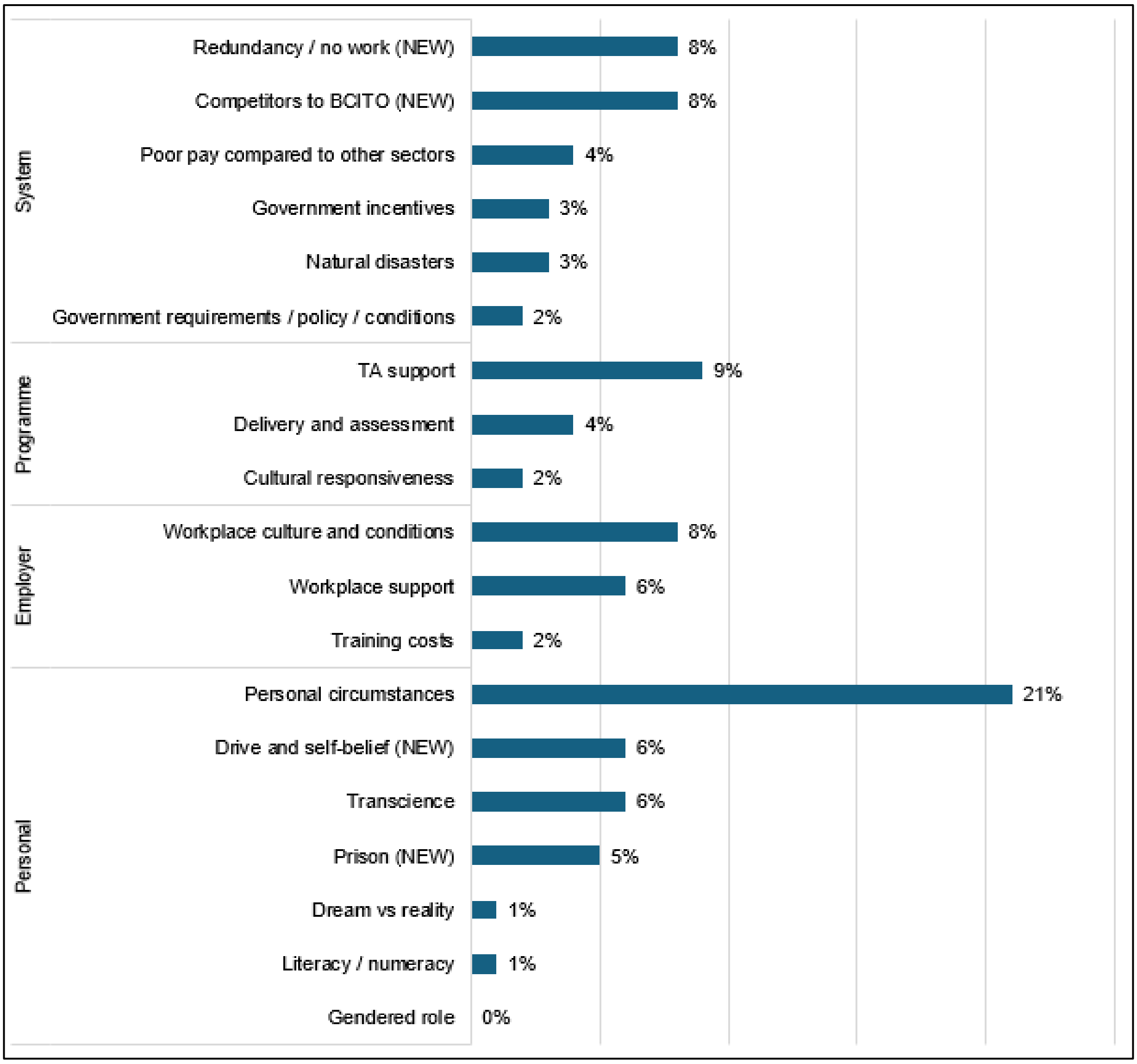


Figure 3. Overall Relative Importance of Reasons for Withdrawal (n=79)

<u>Deep-Dive into Employer Support</u>

For withdrawals that were caused at least in part by employer support, workplace culture/conditions, or training cost, the most common reasons for withdrawal were:

1. **Training cost**
   Several interviewees explained that when they or an apprentice had to pay for their apprenticeship, they quickly withdrew. In some cases, they attributed this to a funding scheme ending. In others, there was no mention of government funding.

   *The company originally said they were going to pay but then seemed to change their mind, and I had just had twins so I couldn't afford it. (Withdrawn apprentice, construction management, withdrew 2022)*

*Suddenly out of the blue, I got a big bill and found out that free fees had stopped. My employer wasn't prepared to pay my fees, and we were really busy in the shop, so I decided to give up. (Withdrawn apprentice, flooring, withdrew 2023)*

2. **Range of tasks**
A common theme mentioned by apprentices was the challenge of getting the right range of tasks at the workplace to get things signed off. Typically, this was because the workplace simply did not do that type of work, or the business was under too much pressure to focus on the apprentice's needs. In some less common cases, apprentices felt that their employer did not care enough to give them the right work. Apprentices noted that this challenge meant they had to "pester" their employer, which could create tension or lead to the apprentice giving up.

*I was with an owner-operator business. The scope of work was a real barrier to getting things signed off. I was 2 years into my apprenticeship and only 35% through. My employer didn't have time/motivation to seek out opportunities for me to get signed off on things outside the work we did like concreting, gib placing, roofing, etc. (Withdrawn apprentice, carpentry, withdrew 2022)*

On the other hand, there were several examples of employers who went out of their way to find apprentices the tasks needed for their apprenticeship, either by booking jobs on purpose to get the right tasks, by simulating tasks, or by finding other employers the apprentice could work for temporarily.[5] This was highly appreciated by apprentices.

*I feel like my employer will go out of their way to try help me finish off the last few modules. He might simulate tasks for me that I can't do in the job. (Current apprentice, carpentry)*

*My employer made connections with other companies [in the area], so if I needed work, I could go to other companies to tick stuff off. (Completed apprentice, carpentry)*

3. **Not enough time to study or for visits**
The expectation within apprenticeships is that apprentices collect evidence of their work as they go, that they receive visits from their BCITO Training Advisor (TA) on-site, and that they complete bookwork outside of work hours. However, apprentices note that a lack of time makes these difficult. During work hours, it may not be encouraged to take time "off" work for these activities, or even if it is, apprentices note that the fact that it is done on a phone can give off a bad impression. Aside from this, at the end of a physical day, it is difficult for apprentices to have the energy or self-motivation to do bookwork in the evenings.

*If I am being honest, I am planning to leave. Withdrawing has been on my mind for about a year now. I am way behind... My employer isn't doing his part… I don't think he wants us to progress. He doesn't allow us to upload anything during work hours. (Current apprentice, stonemasonry)*

*My employer wasn't even really open to the advisor coming to visit me and use up that time. (Withdrawn apprentice, glass and glazing, withdrew 2024)*

[5] Although some employers expressed hesitancy about this, as there was a concern that they could "lose" their apprentice to the other employer.

Several exemplary workplaces had tackled this issue by offering set time during the day for apprentices to complete their bookwork, whether it being during set up in the morning, or one day a week during a set “office hour” where apprentices work on their bookwork together.

*I got an hour a day at work paid to sort BCITO stuff, like uploading photos and doing bookwork. This helped me a lot. Then later on, when things got easier, I didn’t always use it. (Completed apprentice, carpentry)*

*We… [attribute our apprentice completion rate to] providing a safe space for them to work on their course materials after work. (Employer, carpentry)*

4. **Capability, capacity, and/or interest to train**
   Finally, some apprentices felt their employer did not have interest in their apprenticeship or recognise their role as their trainer, instead seeing the apprentice only as “cheap labour”. In rare cases, some even felt that employers did not wish for them to progress, to keep salaries low. This again put apprentices in a difficult position where they had to “pester” employers to provide them with training.

   *[The] boss did not want me to progress, just didn’t help me with apprenticeship at all, just chucked me on jobs and told me to do it even though I [had] never done it before… I was just watching YouTube videos to learn… I wouldn’t be surprised if the employer was purposely getting in the way of the Training Advisor, to stop them talking to me, and me progressing. (Withdrawn apprentice, interior systems, withdrew 2022)*

   *I didn’t have a supportive employer, [they] didn’t support me at all, didn’t back me to finish or be by myself – they just wanted a labourer, as [they’re] cheaper to have than qualified builders. (Completed apprentice, carpentry)*

   When employers did care about training, it made a significant difference to apprentices. In fact, when apprentices mentioned something positive about their employer, it tended to be about the general sense of support, care, and guidance they had received.

   *We have development plans for all our staff. It would be good to set these up alongside BCITO, do a joint training plan and then apprentices can see the long-term pathway. (Employer, concrete)*

   *My employer is good… Me and my boss go through [everything] together after meeting with the Training Advisor and we make plans. When I need to learn something, he will try book jobs with stuff that I need to tick off. Communication was daily. (Completed apprentice, brick and blocklaying)*

   *A lot of the good parts of the apprenticeship were to do with my boss. He was my main go-to, he’s been a champ. (Completed apprentice, painting and decorating)*

In general, these examples appear to align with one of the findings from Bednarz (2014) – that there is a large difference in completers and non-completers satisfaction with their employment experience”, with 80% of completers satisfied compared to 42% of non-completers.

<u>Hypotheses for the Recent Change in Withdrawal Rate</u>

Next, we explored what employers thought might have changed in recent years to negatively impact withdrawal rates. This involved employers thinking about the problem at a more macro level, and drawing upon their perspectives from beyond their most recent apprentices.

Employer perspectives were categorised into existing hypotheses, or otherwise, new categories were created as per constructivist grounded theory. Each hypothesis was then given relative importance based on what the employer had implied.

It is important to clarify that these hypotheses are independent of (and do not replace) the pre-existing reasons why apprentices withdraw, such as a loss of interest or a need to relocate. Instead, they revolve around the timing of significant events or changes in recent years.

The theme with the most relative importance according to employers was found to be generational differences, at 36%. Many employers mentioned that younger staff have unrealistic expectations of the workplace, require a different style of pastoral care, always think that the grass may be “greener” elsewhere, and/or have higher expectations of communication.

*Young guys are not so motivated. They take sick leave more than we ever did when we were coming through the system. (Employer, concrete).*

*Young people are different these days - they are impatient, they want things to happen quickly. [But also,] at that age we were more thinking about the long term, having our own business, being our own boss, that [i.e., your career] doesn’t seem to be as much of a driver these days. (Employer, carpentry).*

*I think young people don’t want to work too hard. I think the whole Covid thing has changed people. They are a little bit less caring about other people’s expectations. People are more insular and focused on themselves. (Employer, carpentry).*

*There are high standards, and sometimes if you are an employer you get frustrated with [them]. Apprentices need to be more resilient to making mistakes and being told about it. (Employer, carpentry).*

The next most important hypotheses, according to employers, were changes relating to the BCITO Training Advisor (TA), and the appearance of new competitors (each with 20% relative importance). The first hypothesis relates to changes in the capacity or quality of TAs, as well as to an increase in TA turnover. This could be linked to the construction boom of 2020-2021, which increased enrolments and in turn, impacted the ratio of TAs to apprentices. The second hypothesis relates to apprentices moving to new education providers, in some cases due to wanting to follow a TA who was changing jobs.

These results show that there are opportunities for BCITO to address challenges with TAs. However, there are also opportunities for both BCITO and employers alike to improve their engagement of Gen Z apprentices.

<u>Intervention-Centric Analysis</u>

Finally, we explored the issue through an intervention-centric lens. Instead of counting categorised reasons for withdrawing or reasons for a change in withdrawal rate, we reviewed each of the 40 withdrawn apprentices methodically and suggested one

intervention BCITO could have actioned, and one action the employer could have actioned, that each had the best chance of retaining the apprentice. We then estimated the overall effectiveness of each intervention. This provided us with an estimate of how many of the 40 apprentices each intervention could potentially have retained. This is different from counting reasons, as it takes into account the specific characteristics and situation of each apprentice in deciding which intervention would work best. It is also different in that one intervention can potentially be used to address more than one reason for withdrawal, and it does not assume as quickly that a reason cannot be addressed by either BCITO or employers. Again, we focus again here on interventions relevant to employers.

The top preliminary interventions drawn for employers from this analysis are:

1. **Job search support (estimated to retain 3 or 4 of 40)**
   Redundancy is a key cause of apprentice withdrawals. In many cases, the apprentices who are made redundant struggle to then find another role, despite a desire to finish their apprenticeship. While this is something BCITO can assist with, there are likely also opportunities for employers to smooth the transition into another role. For example, an employer may know of other companies in the area or may be able to provide positive references.

2. **Work culture / pastoral care (estimated to retain 2 of 40)**
   In some cases, apprentices explained that there had been poor support from the workplace in terms of culture and general pastoral care. For example, this was seen in cases where the apprentice had experienced a significant family event (e.g., a birth or death), and where the apprentice was struggling with mental health. This may mean actively working to create and maintain a culture of support and kindness; adjusting work where possible to support apprentices through difficult times; or referring apprentices to mental health providers where appropriate.

3. **Alternative funding guidance (estimated to retain 1 or 2 of 40)**
   There were a number of apprentices who wanted to complete their apprenticeship but could not due to the cost. For trades where a qualification is not required, this is even more likely to cause withdrawal as there is little value proposition to continue. Offering support to find alternative funding could help such apprentices to continue. While BCITO may be able to let apprentices (and employers for that matter) know about funding options, employers could also communicate these options to apprentices.

4. **Interest (and recognition of role) in training (estimated to retain up to 1 of 40)**
   Some apprentices feel that their employer is not interested in training them, which puts apprentices in difficult position. Providing apprentices with a safe space to advocate for their needs, in balance with the businesses' needs, could help such apprentices to not only complete their apprenticeship but develop self-confidence and longer-term skills such as communication.

We estimate that these interventions together could reduce withdrawal rates by somewhere in the range of 10 to 20%.

## DISCUSSION

This research, like research before it, has found that apprentices withdraw for a diverse range of reasons. In many cases, these reasons are likely the same as those that existed prior to the recent increase in withdrawal rate.

We sought to answer why the withdrawal rate has changed in recent years. Two themes that come through strongly are an increase in redundancies and the emergence of a competing provider. In the case of the second, this will mean that a small portion are not truly withdrawing, but instead completing their apprenticeship elsewhere. However, these two themes do not explain the change in withdrawal rate on their own. This is because they appear to be common in 2023 and 2024 withdrawals but do not necessarily explain a change in withdrawal rate *before* these years – particularly given there was a construction boom prior to this, and the competitor landscape for BCITO shifted in 2023.

Many employers point to generational change as the key factor in the change in withdrawal rates in recent years. While this likely is true to some degree, it too provides only limited explanation, given that any generational change would have been gradual, and the change in withdrawal rates was sudden.[6]

A second theme pointed to by employers is BCITO Training Advisor changes (capacity, quality, and turnover). These would likely have happened quite suddenly when enrolments skyrocketed in 2022. This could be one of the more plausible drivers of the sudden increase in the withdrawal rate.

A final possibility mentioned by employers, albeit less often, is that funding and the construction boom both encouraged employers and apprentices into the system who would not have been there otherwise, and who likely had less propensity to stay until completion.

**CONCLUSION**

<u>The 'So What' points for all</u>

In this paper, we have undertaken various types of data collection and analysis, to approach this problem from different angles. The last of these used an intervention-centric approach to identify interventions that BCITO and employers could have each taken to best retain withdrawn apprentices.

While this approach provides us with highly actionable interventions, when we look at this research as a whole, it would be remiss to not comment on wider systematic issues. These include:

- The reliance of the apprenticeship system on employers for teaching and pastoral care, when employers do not feel incentivised (other than intrinsically) to do this well.
- The use of a pastoral care system that was built for previous generations, who had a different set of values to the generations coming through today, and who were also less multicultural.

This macro view will be considered carefully during the remainder of this research.

<u>The 'So What' points for employers</u>

Some of the reasons for withdrawal and for the change in withdrawal rates can be addressed by BCITO. However, many others can only be addressed by employers. Combined action from both sides would likely have the best chance of retaining

[6] Although, some note that COVID-19 likely heightened the rate of generational change.

apprentices. This research highlights the ongoing need to work together to improve apprentice outcomes.

Despite industry challenges, overall apprentice numbers are still high from the earlier construction boom. This suggests that there is an opportunity for both BCITO and employers to take action - both in terms of improving their support for current apprentices and supporting them in their job search post-redundancy - to retain the number that will be needed not just post-recession but, in the years to come.

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# GREEN AND SUSTAINABLE CONSTRUCTION WITH RECYCLED AND HIGH STRENGTH STEEL

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## SUMMARY

Steel Reinforcement is one of the key building resources which have evolved since its inception in Reinforced Concrete from mild steel to 460MPa steel, which is commonly used. However, the demand for higher Construction Productivity and Green Buildings has set a new stage for optimal use of resources and this warrant for High strength materials which has to be Productive, Sustainable and Green. High Strength Steel Reinforcement Grade 600 is one such material which can reduce the steel usage by up to 30% while the strength and ductility of the structures remains unaffected. Reduction in steel usage not only translates into proportional reduction in the carbon footprint of the building it also translates into reduced member sizes which leads to reduced dead loads, lighter foundations, reduced formwork, more usable space and reduced manpower. This eventually brings in higher Construction Productivity, Cost Effectiveness and reduces Environmental damage.

## INTRODUCTION

Carbon emissions are harmful to global warming and it’s hitting new highs. Built environment is estimated to contribute about 37% to the global warming[4]. For many years’ major focus was placed on operational carbon however up to 40% of the carbon emission in a building is estimated to be from the embodied carbon or the carbon footprint of the building and this number is expected to rise as operational carbon is being optimised. Steel, Concrete and Glass are estimated to be the major contributors of the carbon footprint of a building of which steel is estimated to contribute a major share. While majority of the steel production in the world is based on burning coal the focus is now shifting to greener energy and recycled steel. Using recycled steel products and high strength steel such as Grade 600 reduces the carbon footprint of the building considerably. To be effective, these recycled products needs to considered upfront in the concept / design stages. Various tools are available to the Engineers to make an informed decision while choosing building materials for a project upfront, before construction. This paper delves on the use of Grade 600 steel reinforcement and Sustainable and Green Construction with a focus on Embodied carbon.

## HIGH STRENGTH GRADE 600 STEEL REINFORCEMENT

There is a growing interest in the use of high strength steel reinforcement to reduce member sizes and rebar congestion while not compromising on strength and ductility requirements. With the adoption of Eurocode 2 standards[1], Grade 600 can be used for the construction of Residential, Industrial, Commercial, Institutional, Infrastructure projects etc. Compared with the Grade 460 or Grade 500 steel the Grade 600 steel is 140 or 100 MPa or about 20% to 30% higher in strength. This reduces the overall steel consumption of a project by up to 30%, under ideal scenarios, while preserving strength

and serviceability requirements. The following section defines the properties of Grade 600, weldable steel reinforcement.

## MATERIAL PROPERTIES

The characteristics of reinforcement steel are defined by its Chemical and Mechanical properties. EN 10080[2] defines the chemical properties required for Grade 600 steel as shown in Table1 below.

Table 1 – Chemical Composition (maximum % by mass)

| | **Carbon** | **Sulphur** | **Phosphorus** | **Nitrogen** | **Copper** | **Carbon equivalent** |
|---|---|---|---|---|---|---|
| Cast analysis | 0.22 | 0.05 | 0.05 | 0.012 | 0.80 | 0.50 |
| Product analysis | 0.24 | 0.055 | 0.055 | 0.014 | 0.85 | 0.52 |

To maintain weldability, the carbon equivalent value, $C_{eq}$, shall not exceed 0.50 for cast analysis. However it could be exceeded by up to 0.02 for product analysis. The $C_{eq}$ value can be computed by the following equation,

$$C_{eq} = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15} \qquad (1)$$

where,

*Mn* is the percentage manganese content;
*Cr* is the percentage chromium content;
*V* is the percentage vanadium content;
*Mo* is the percentage molybdenum content;
*Cu* is the percentage copper content;
*Ni* is the percentage nickel content.

EN10080[2] does not define the Mechanical properties; however the Mechanical properties of Grade 600 steel can be obtained from the SS EN 1992-1-1[1] or commonly known as Eurocode 2[1], which is the Reinforced Concrete design standard in Singapore. The Tensile properties as defined by the Eurocode 2[1] are furnished in Table 2.

Table 2 – Characteristic Tensile Properties

| | **Yield strength, $R_e$** MPa | **Tensile/Yield strength ratio, $R_m/R_e$** | **Total elongation at maximum force, $A_{gt}$** % |
|---|---|---|---|
| B600A | 600 | 1.05 | 2.5 |
| B600B | 600 | 1.08 | 5.0 |
| B600C | 600 | ≥ 1.15, < 1.35 | 7.5 |

It could be noted that there are 3 ductility classes of Grade 600 steel specified, namely, B600A, B600B and B600C. It is generally expected that B600B could be used for normal construction while B600C could be used for earthquake resistant structures. It could be noted that the ductility requirement, $A_{gt}$, is now defined as total elongation at maximum force. There is also a cap on the absolute maximum permissible value of yield strength at

780MPa, which requires the steel manufacturer to maintain a consistent product quality over the long term.

**DESIGN CONSIDERATIONS**

Design should be based on the nominal cross-section area of the reinforcement. Design yield strength of reinforcement ($f_{yd}$) shall be taken as the characteristic yield strength ($f_{yk}$) of 600 MPa divided by a material safety factor ($\gamma_s$) of 1.15, that is, $f_{yd}$ = 600 / 1.15 = 522 MPa.

For normal design, the design stress-strain diagram for both tension and compression may be assumed to be:

- **(A)** bi-linear with a horizontal top branch without strain limit; or
- **(B)** bi-linear, with an inclined top with a strain limit of $\varepsilon_{ud}$= 0.045 and corresponding stress of 565 MPa

as shown in Figure 1.

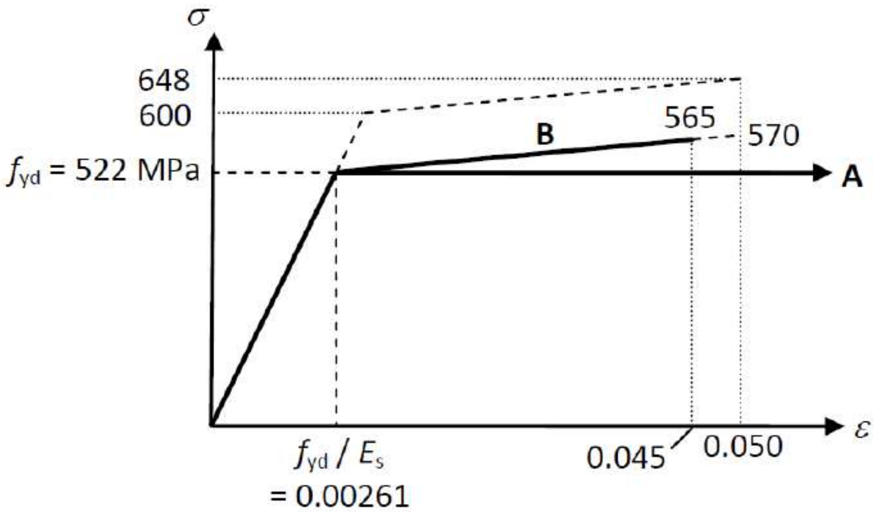


Figure 1 - Design Stress – Strain Diagram for Grade 600 Steel

Generally, one may think that high strength steel must be used together with high strength concrete, however it's not factual. Normal strength concrete, such as Grade 40 (cube), which is commonly used in many projects, can be used together with Grade 600 steel. However for members which are subjected to pure compression, such as struts / short columns, the Eurocode 2[1] places a compression strain limit of 0.2% for concrete grades up to Grade 60 (cube) – to prevent concrete from failing before the steel. Under such circumstances the full yield strength of the Grade 600 steel cannot be mobilised. Nevertheless, the Eurocode 2[1] also provides a solution for such scenario which is in the form of confinement reinforcement. One can improve the ductility of concrete by providing this confinement reinforcement – commonly used in Japan – in the form of additional links, and thereby utilising the full strength of the Grade 600 steel when used with normal strength concrete such as Grade 40 (cube). The overall steel savings under such scenario will still be considerable, even after allowing for the additional links / confinement reinforcement, as these links only form a minor percentage when compared to the main reinforcement.

**CASE STUDIES**

To estimate the actual steel savings that one could possibly achieve under practical construction scenarios a couple of ongoing projects were chosen for the study. To have diversity in the projects one high-rise residential building and one institutional building was chosen. Detailed designs were carried out for both the projects using Grade 600 steel reinforcement. The design was repeated again using Grade 460 steel to estimate the difference in the steel quantity. Eurocode 2[1] was adopted for the design along with the Singapore National Annex for Eurocode 2[1]. Grade 40 (cube) concrete was used for both the projects.

In the case of the high-rise residential building, the columns were in predominant compression and no confinement links – apart from nominal links – were provided. Conversely, confinement links were provided to the columns of the Institutional building to enhance the concrete strain. The steel savings achieved for the High-rise residential building and the Institutional building are presented as below.

<u>High-rise Residential Building – Project Info:</u>

- No. of storey: 20
- Design code: Eurocode 2[1]
- Steel grade: Grade 460 / Grade 600
- Concrete grade: Grade 40 (cube)
- Slab thickness: 125mm
- Beam size: 250 x 500mm (Typical)
- Column size: 300 x 1500mm (Typical)

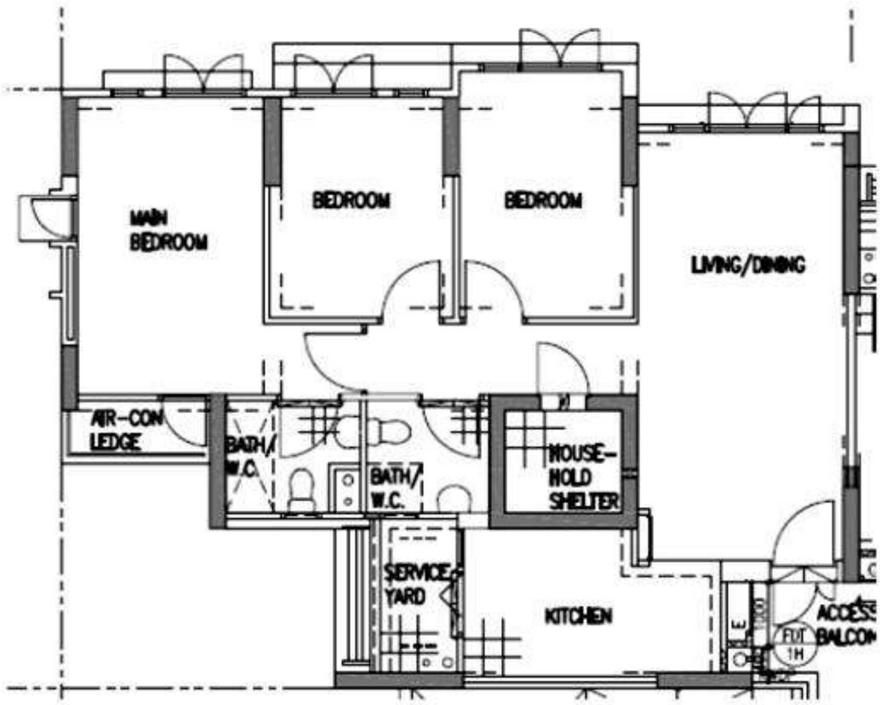


Figure 2 – A Typical Unit of a High-rise Residential Building

Table 3 – Estimated Steel Savings – High-rise Residential Building

| Element | % Savings |
|---|---|
| Slab | 30.4% |
| Beam | 21.8% (Some beams with min. steel) |
| Column | 5.2% (W/Nominal links) |
| **Overall Savings** | **20.26%** |

Institutional Building – Project Info:

- No. of storey: 8
- Design code: Eurocode 2[1]
- Steel grade: Grade 460 / Grade 600
- Concrete grade: Grade 40 (cube)
- Slab thickness: 300mm (Flat slab)
- Column size: 400 to 600mm dia.

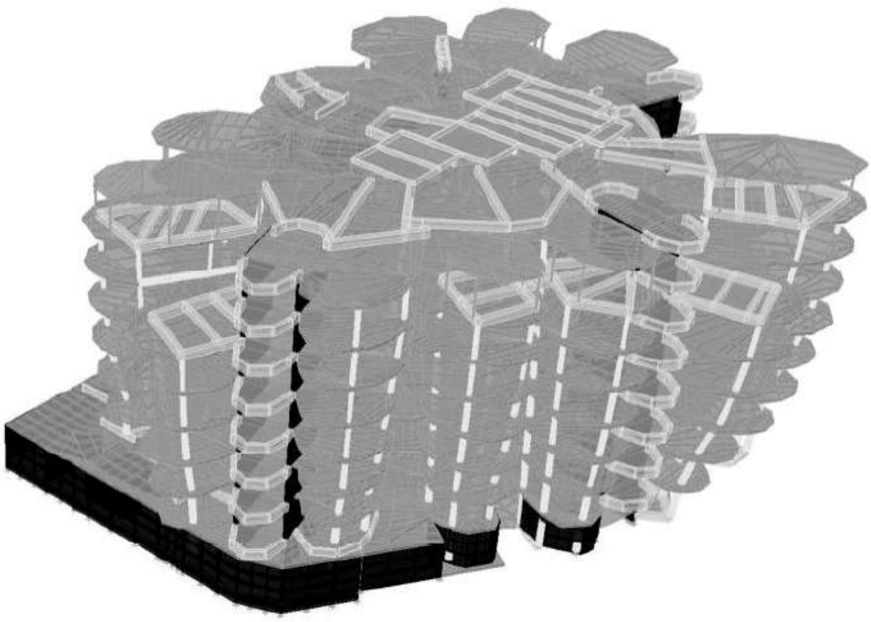

Figure 3 – 3D View of an Institutional Building

Table 4 – Estimated Steel Savings – Institutional Building

| Element | % Savings |
|---|---|
| Flat Slab | 26% |
| Column | 23.6% (W/Confinement links) |
| **Overall Savings** | **25.04%** |

The two case studies show that the steel savings varies with project type and structural element. Slab and Beam, which are predominantly subjected to bending, contribute the

maximum savings which are up to 30.4%, under ideal scenarios. The steel savings in Columns are dependent on the loading scenario – the savings could be lower when compression is predominant; however with the confinement reinforcement the savings could be as high as 23.6%. Overall, one may achieve up to 25% of steel savings for a project by adopting appropriate design methods.

## BENEFITS OF HIGH STRENGTH GRADE 600 STEEL REINFORCEMENT

High strength steel has multiple benefits, some of which are tangible and some are intangible. The key benefits are illustrated in Table – 5 below.

Table 5 – Benefits of High Strength Grade 600 Steel Reinforcement

| Item | Description |
|---|---|
| Steel Savings | Potential to reduce steel reinforcement by up to 30% compared to Grade 460 steel |
| Environment | Reduced Carbon Footprint, Construction noise, Fuel consumption etc. |
| Steel fabrication | Up to 30% less workers are needed |
| Manpower - Steel fixing / Installation | Up to 30% less onsite workers are required to install / fix steel reinforcement onsite |
| Logistics | Less trucks carrying steel reinforcement on the roads – up to 30% less |
| Site Crane | Handles up to 30% less steel and frees up crane time for other construction activities thereby speeding up construction. |
| Concrete Savings | Reduction in structural element size is possible when used with appropriate grade of concrete and results in overall dead load being reduced |
| More usable space | More floor space is usable with column size reduction |
| Less formwork | Possible to reduce formwork needed for Columns and Beams due to member size reduction |
| Lighter foundations | Due to reduction in members size resulting in lighter super structure, foundation loads and cost can be reduced |
| Storage space | Space required for site storage of steel reinforcement can be reduced by up to 30% |
| Improved Safety | Site safety will be improved due to less material handling, steel fixing etc. |
| Time reduction | Overall time savings can be accomplished by factoring in the earlier stated benefits |
| Cost reduction | Overall cost reduction can be achieved from reduced Material, Manpower, Construction Time etc. |

Superior construction productivity can be achieved with the reduction in construction time and manpower. Furthermore, one could complete the project ahead of time if time savings arising from productivity improvements were factored in during the planning stage. Early completion of a project could lead to considerable cost savings for the contractor apart from prospecting for the next project. Moreover, it could be beneficial to the developer as the overall cost of the development could be lower and early sales / renting of the building could lead to improved returns on the investment made.

**GREEN & SUSTAINABILITY**

Steel can be produced either from iron ore or by recycling steel scrap. However, by either way, carbon is being emitted into the atmosphere. It has been estimated that, on an average, about 1.91 Kg of carbon is emitted to produce 1Kg of steel[3]. Blast Furnace (BF) emitted about 2.33 Kg of carbon to produce 1Kg of steel[3] while Electric Arc Furnace (EAF) emitted 0.68 Kg[3]. The current level of steel production in the world is reported to be 1,892 million tonnes thereby the steel industry's carbon emission contribution stands at up to 9% of the global $CO_2$ emissions[3]. However it is estimated that, to meet the demands of the growing population, the steel use is projected to increase from current levels.

The building and infrastructure sector is the largest consumer of steel today, consuming about 52% of steel produced[3]. With the adoption of high strength steel Grade 600 or higher, one can reduce the carbon footprint of the building by up to 30% or more (depending on the steel used), with respect to the steel's carbon footprint share.

To produce 1 tonne of steel in a blast furnace – about 71% of the world's steel is produced by this route[3] – one would need about 1400 Kg of iron ore, 800 Kg of coal, 300 Kg of limestone and 120 Kg of steel scrap. On the sustainability front, the use of high strength Grade 600 steel reduces the consumption of these natural resources such as iron ore, coal etc. by up to 30%, and thereby reserving it for the future generations to come.

**UNDERSTANDING AND REDUCING CARBON IN BUILDINGS**

Globally, buildings are responsible for 37% of the global carbon emissions[4]. These emissions are broadly classified under two types, Operational Carbon and Embodied Carbon. Operational carbon is contributed when the completed building is in operation and consumes energy for lights, air conditioning, elevators etc. Embodied carbon is contributed during the construction phase of the building, primarily by the construction products used in the building construction, such as Steel, Concrete, Glass, Aluminium etc. The total carbon emitted during the product manufacturing phase gets accounted as embodied carbon in the respective construction products.


Figure 4 – Embodied & Operational Carbon (Picture credit: architectmagazine.com)

Operational carbon can be optimised after the building has been completed by the use of energy efficient devices, renewable energy etc. However, Embodied carbon cannot be reduced after the building has been completed. Therefore, one needs to consider them upfront during the building concept planning stage or during the design stage. Over the years much focus was given to optimise Operational carbon which resulted in the greater reduction of operational carbon emissions. The Embodied carbon component in a building now stands tall which needs focus.

Environmental Product Declaration (EPD) of a product plays a key role in understanding the Global Warming Potential (GWP) of a construction product. Stakeholders in the construction value chain need to understand the carbon budget they have set aside for the building construction and choose products based on GWP to be within that budget. One could leverage on the various GWP based Carbon calculation tools that are available to estimate the Embodied carbon before construction begins.

Generally, Steel reinforcement made by recycling metal scrap through EAF route consist of lower GWP (average of 0.68 Kg $CO_2e$ / Kg[3]) vs. higher GWP (average of 2.33 Kg $CO_2e$ / Kg[3]) of Steel reinforcement made from BF route. It is environmentally friendly to choose recycled steel and stay below the carbon budget. However only about 29% of the steel made globally are through EAF route which might create supply / cost concerns. However, one could mitigate these with proper planning.

Table 6 below illustrates a typical construction project where the steel reinforcement demand is 5,000T. Three steel options are presented with their respective GWP values. Up to 20% reduction in steel consumption will result when Grade 600 steel is used. It can be seen that the Grade 600, EAF option gives the lowest Embodied carbon for this illustrated project.

Table 6 – Embodied Carbon Calculation Illustration

| **Steel options** | **GWP, Kg CO2e/Kg** | **Project Steel Tonnage** | **Total Embodied Carbon, Tonne** |
|---|---|---|---|
| Grade 500, BF | 2.33 | 5,000 | 11,650 |
| Grade 500, EAF | 0.68 | 5,000 | 3,400 |
| Grade 600, EAF | 0.68 | 5,000 x 0.8 = 4,000 | 2,720 |

**CONCLUSION**

The benefits of using high strength steel are multifaceted. It not only saves steel, it also brings in superior construction productivity which saves construction time and manpower. With appropriate design techniques, one could reduce the structural member sizes thereby brining in attractive benefits like, increased usable space, lower foundation cost and concrete usage. Moreover, high strength steel helps to lower the carbon footprint of a building project and promotes sustainability. Around the world, high strength steel reinforcement usage is gaining momentum in many countries. USA and Japan have reached greater heights in adopting super high strength steels and are already deriving the benefits from it. It’s imperative that countries that endeavour to progress to the next level embrace high strength steel and thereby begin a new chapter in smart construction.

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# SEISMIC DESIGN OF POST-INSTALLED FASTENERS IN NEW ZEALAND

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## SUMMARY

The fastener assessment method known in New Zealand as “C1” has been omitted from the ACI 355 codes and the fastener assessment method known in New Zealand as “C2” has been suggested instead. This paper critically analyzes the “C2” method from a design perspective and in the context of the DZ TS 1170.5-2024 Public Comment Draft. It is demonstrated that the “C2” characteristic seismic capacity of fasteners (listed in their European Technical Assessment, ETA) has very limited practical significance in the seismic design of fasteners, and designers could rely on those derived fastener capacities in their actual design in rare cases, if ever. Potential directions towards the development of fastener seismic assessment and design in New Zealand are provided in this paper.

## INTRODUCTION

The majority of the NZS 3101 Concrete structures standard is based on ACI 318, however, as of today (2024) and since 2017, it recommends European standards (EN) and other documents (EOTA TR) for design and assessment of post-installed fasteners. The topic relevant ACI codes (ACI-CODE 355.2 and ACI-CODE 355.4) have been substantially developed in 2023-2024. The most important update is that the fastener assessment method known in New Zealand as “C1” has been omitted from the ACI 355 codes and the fastener assessment method known in New Zealand as “C2” has been suggested instead of it. This improvement is welcome since the method “C1” has been proven to be inferior for seismic assessment, demonstrated by multiple scholars in the literature (Silva, 2001; Hoehler and Eligehausen, 2008; Mahrenholtz, 2012). NZS 3101 should follow this good example and the current references that include an allowance for the assessment method “C1” need urgent replacement in NZS 3101, especially since performance category “C1” currently does not apply to New Zealand in accordance with EN 1992-4:2018.
Despite the more than 20 years active research in the topic, seismic design of fasteners is still an underdeveloped practice both in ACI and EN codes. The fastener capacities available from product assessment are very loosely connected with the design seismic demands. Seismic fastener design can be either overly conservative or unsafe due to the disconnection between fastener assessment and design. This paper provides an analysis of these details and introduces the fastener assessment method currently known in New Zealand as “C2” in the context of the DZ TS 1170.5-2024 Public Comment Draft. An analysis for possible directions in the improvement of seismic fastener design in New Zealand is also provided.

## THE CURRENT SEISMIC DESIGN OF FASTENERS IN NEW ZEALAND

Chapter 17.5.5 of NZS 3101 recommends that “post-installed mechanical anchors and post-installed adhesive anchors shall pass the prequalification testing stipulated in ETAG 001, Annex E and be designed in accordance with EOTA TR 045.” NZS 3101 is a primary reference document in NZBC B1 VM/AS, and for the purposes of New Zealand Building Code compliance, B1 VM/AS gives the following guidance: where the primary reference documents refer to other standards or other documents (secondary reference

documents), which in turn may also refer to other standards or other documents, and so on (lower order reference documents), then the applicable version of these secondary and lower order reference documents shall be the version in effect at the date the B1 VM/AS was published. The ETAG 001, Annex E has been superseded by EOTA TR 049 (https://www.eota.eu/etags-archive), and the EOTA TR 045 has been superseded by EN 1992-4:2018 (https://www.eota.eu/technical-reports). Consequently, Chapter 17.5.5 of NZS 3101 translates to the following: post-installed mechanical anchors and post-installed adhesive anchors shall pass the prequalification testing stipulated in EOTA TR 049 and be designed in accordance with EN 1992-4:2018.

## DESIGNERS' CHALLENGES IN NEW ZEALAND

As explained above, EN 1992-4:2018 is currently an NZBC B1 Verification Method, and therefore designers in New Zealand are expected to perform the seismic design of fasteners accordingly. In the followings, certain specifics of the EN 1992-4:2018 fastener design are highlighted, which may be challenging for practitioners in New Zealand. Further aspects of these topics can be found in the literature (Borosnyoi-Crawley, 2024a).

### Consequences of failure and reliability level

The scope of EN 1992-4:2018 is given in Clause 1, and in particular, Clause 1.1(2) explains that the standard "is intended for safety related applications in which the failure of fastenings may result in collapse or partial collapse of the structure, cause risk to human life or lead to significant economic loss. In this context it also covers non-structural elements." Clause 1.1(4) adds that the standard "is valid for applications which fall within the scope of the EN 1992 series."
It is apparent that designers in New Zealand must have in-depth knowledge about the applications that fall within the scope of the Eurocode 2 (EN 1992) series in comparison to those of NZS 3101 to be able to judge if a given application covered by NZS 3101 is in the scope of EN 1992-4:2018, or not.
EN 1992-4:2018 Clause 4.1(2) explains that fastenings (see Appendix of this paper for terms and definitions) "shall be designed according to the same principles and requirements valid for structures given in EN 1990 including load combinations and EN 1992-1-1." In a note it is added that design using the partial factors given in EN 1992-4:2018 and the partial factors given in the EN 1990 Annexes are considered to lead to a structure associated with reliability class RC2 (consequence class CC2), with a $\beta$-value of 3.8 for a 50 year reference period (i.e., $p_f = 1.1\times10^{-4}$ probability of failure).
It is apparent that the level of reliability is different in NZS 3101 and in EN 1992-4:2018. Clause C2.3.2.2 of NZS 3101 explains that the basis for the selected values of strength reduction factors is detailed in the study by MacGregor (1983), which ascertained that for the values of $\phi$ similar to those in Clause C2.3.2.2 of NZS 3101 and load factors corresponding to AS/NZS 1170, the target values of the reliability index, $\beta$ (that is called as *safety index ($\beta$)* in NZS 3101) of 3.0 for dead and live load, 2.5 for dead and live and wind forces and 2.0 for dead and live and earthquake forces applied. NZS 3101 also adds that these values for the safety index are within the range implicit in AS/NZS 1170. The target reliability indices set by NZS 3101 would mean $p_f = 1.35\times10^{-3}$ (for $\beta = 3.0$), $p_f = 6.21\times10^{-3}$ (for $\beta = 2.5$), and $p_f = 2.28\times10^{-2}$ (for $\beta = 2.0$) probabilities of failure. The length of reference period corresponding to the given target reliability indices is not detailed in NZS 3101, however, it can be noted that the reliability indices in those early studies (e.g., the paper by MacGregor (1983), based on the NBS Special Publication 577, June 1980) were determined for structural members based on a service period of 50 years.
The existing reliability mismatch between NZS 3101 and EN 1992-4:2018 needs engineering judgement from the designer. It is also noted that the EN 1992-4:2018

fastener design (as explained above) does not allow for different levels in the assumed consequences of failure and the target reliability class is RC2 (consequence class CC2) when the default partial safety factors are used.

<u>Demands and capacities</u>

The design method in EN 1992-4:2018 uses physical models which are based on a combination of tests and numerical analysis consistent with EN 1990 (Eurocode 0). Clause 4.1(4) of EN 1992-4:2018 requires the values of actions to be obtained from the relevant parts of the EN 1991 series and EN 1998 series in the case of seismic actions. It means that the design value of the effect of seismic actions $E_{Ed}$ acting on the fixture is to be determined according to EN 1998-1 (Eurocode 8) and its additional parts. The seismic characteristic resistance $R_{k,eq}$ of a fastener is to be determined by the selection of the relevant seismic performance category given in EN 1992-4:2018 and the characteristic resistances and distances of fasteners are to be taken from a European Technical Product Specification (e.g., ETA).

It is apparent, in summary, that NZBC compliant fastener design is based on Eurocodes and not on AS/NZS & NZS standards, and if fastener loads are calculated based on AS/NZS & NZS standards then EN 1992-4:2018 cannot be directly used for design. It is also noted that certain load combination factors are not the same in AS/NZS 1170.0 and in the Eurocodes, therefore, the designer’s engineering judgement is needed when translating the calculated fastener loads based on AS/NZS & NZS standards to the equivalent of those in accordance with the Eurocodes.

<u>Fasteners for redundant non-structural systems</u>

EN 1992-4:2018 Clause 1.2(4) explains that the standard applies to single fasteners and groups of fasteners, and in a group of fasteners the loads are applied to the individual fasteners of the group by means of a common fixture. If fasteners in redundant non-structural systems (e.g., suspended ceilings, fire sprinkler systems or other engineering systems) do not have a common fixture, they cannot be designed as a group of fasteners in accordance with EN 1992-4:2018. Clause 7.2(2) explains that design of fasteners for redundant non-structural systems can be found in CEN/TR 17079, which is limited to static and quasi-static loads.

It is apparent that the seismic design of fasteners for redundant non-structural systems is outside the scope of NZS 3101 since it is outside the scope of EN 1992-4:2018. It is also noted that no standardized method is currently available for such design internationally.

## NEXT STEPS IN THE SEISMIC DESIGN OF FASTENERS IN NEW ZEALAND

As it was demonstrated above, the seismic design of post-installed fasteners in New Zealand is challenging, despite being well regulated. It can be assumed that many designers overlook the specifics highlighted in this paper and perform their fastener design with questionable code compliance. It can also be assumed that majority of the designers use software provided by overseas fastener manufacturers that also includes the risk of overlooking certain details of their design.

The ideal solution for New Zealand would be to develop (or adopt) a design method that is tailor-fit to the local conditions and fully compliant with the NZBC in a transparent way, optimally being part of an already existing primary reference NZS standard in B1 VM/AS (e.g., NZS 3101). For this development, either ACI 318 or EN 1992-4 could be used as a basis, since the technical background of those codes are essentially the same, however, the New Zealand specific strength reduction factors must be carefully calibrated.

Regarding the fastener seismic assessment methods, the current international situation

is quite uncertain, especially in the EU. On 10th April 2024, the European Commission (EC) published the result of the trilogue process (EU parliament, EU Council and EC) of a new Construction Product Regulation (CPR) with a date of application in November 2025. In the new CPR the route to CE marking based on European Technical Assessments (ETA) is maintained, however, EADs that were published within the current legal framework will become invalid after approximately 2030. This change may paralyze the complete EOTA/EAD/ETA system, and there are industry voices already heard that manufacturers might choose to ignore the voluntary EAD route and would go back to Member States assessments for product qualification (Bourgund et al, 2024). Such a move from the European manufacturers would make significant impact on the current New Zealand practice in fastener seismic assessment.
New Zealand professionals, and especially the NZS 3101 Development Group (within the Concrete NZ Learned Society), need to start making early plans and proactive steps to prevent a situation that may end up in dead-end provisions frozen for decades in NZS standards if the new CPR results in unpredictable outcomes in the industry.
One stable, conservative approach could be to rely more closely on the content of ACI codes and other ACI publications, regarding the fact that those documents seem to be less prone to sudden, unpredictable changes similar to the EOTA/EAD/ETA system in the EU. Further reasons towards this direction are the recent, major developments in ACI-CODE 355.2 and ACI-CODE 355.4. These drafts were open for public discussion from December 20, 2023 to February 3, 2024. In accordance with the public discussion drafts, the fastener assessment method known in New Zealand as "C1" has been omitted from the ACI 355 codes and the fastener assessment method known in New Zealand as "C2" has been suggested instead of it.
The rest of this paper provides an analysis of the fastener assessment method currently known in New Zealand as "C2" in the context of the DZ TS 1170.5-2024 Public Comment Draft. An analysis for possible directions in the improvement of seismic fastener assessment and design in New Zealand is also provided.

## FASTENER SEISMIC PERFORMANCE CATEGORY KNOWN AS "C2"

EN 1992-4:2018 recognizes two different seismic performance categories for fasteners, referred as "C1" and "C2", respectively. Performance category "C1" currently does not apply to New Zealand. Also, the "C1" performance category methods have been proven to be inferior for seismic assessment, demonstrated by multiple scholars in the literature (Silva, 2001; Hoehler and Eligehausen, 2008; Mahrenholtz, 2012), therefore, it is not discussed any further in this paper.
The assessment methodology for the performance category "C2" has been developed by Mahrenholtz (2012), based on the theoretical research of Wood et al (2010), and has been first published in ETAG 001, Annex E (2013) and later adopted without any change in EOTA TR 049 (2016), as well as in EAD 330232 (2021) and EAD 330499 (2020). The most recent adoption of the method, as mentioned above, has happened in December 2023, in the ACI-CODE 355.2 and ACI-CODE 355.4 public discussion drafts.
Essentials of the "C2" methodology (Borosnyoi-Crawley, 2024a):

- It consists of three separate testing protocols:
  1) Load cycling under pulsating tension load;
  2) Load cycling under alternating shear load;
  3) Crack cycling with tension load under varying crack width.
- The basis of the developed testing protocols is a theoretical study; building response nonlinear simulations performed on five perimeter concrete Special Moment Resisting Frames (SMRF) of 2, 4, 8, 12 and 20 stories, and two dual lateral load resisting systems (Ordinary Moment Resisting Frames (OMRF) coupled with a structural concrete shear wall) of 4 and 8 stories, presented in the

work of Wood et al (2010).

- Building prototypes were designed and detailed by IBC-2006, ASCE 7-05 and ACI 318-08 in the simulations presented in the work of Wood et al (2010).
- Earthquake motion selection was based on a PSHA conducted for Charter Oak (Los Angeles County, California, USA). The selected return period for the "C2" protocols was 475 years, and 21 strong motion records from the literature were used in the numerical studies, scaled to achieve a design spectral acceleration of 2.01 g at short periods ($S_s$) and 0.61 g at a period of one second ($S_1$).
- The load cycling protocols are based on the acceleration response time history analysis of elastically responding single-degree-of-freedom (SDOF) oscillators of 5, 10, 15 and 20 Hertz frequencies, placed on each floor level and in an uncoupled fashion, subjected to the floor level accelerations in the numerical studies.
- In the load cycling protocols, the maximum loads for the stepwise increasing load cycles are set to arbitrarily selected values of 0.375 $N_{u,cr,m}$ and 0.425 $V_{u,cr,m}$ for serviceability level and 0.75 $N_{u,cr,m}$ and 0.85 $V_{u,cr,m}$ for suitability level, based on the work of Mahrenholtz (2012); where $N_{u,cr,m}$ and $V_{u,cr,m}$ are the mean ultimate tensile and shear capacity in cracked concrete under static loading at w = 0.8 mm static crack width, respectively.
- The crack cycling protocol is based on the curvature response time history analysis of the beams, just outside the plastic hinge zones, where the beams remain linear elastic, and the relationship between curvature and strain can be assumed to be linear. No actual crack width assessment was performed with regards to the earthquake loads. It was assumed that the crack width opening-closing amplitudes are equivalent to the curvature amplitudes.
- In the crack cycling protocol, the maximum crack widths (and the corresponding permanent anchor tensile loads) for the stepwise increasing crack width cycles are set to arbitrarily selected values of 0.5 mm (and 0.4 $N_{u,cr,m}$) for serviceability level and 0.8 mm (and 0.5 $N_{u,cr,m}$) for suitability level, based on the work of Mahrenholtz (2012); where $N_{u,cr,m}$ is the mean ultimate tensile capacity in cracked concrete under static loading at w = 0.8 mm static crack width.
- During the fastener assessment procedure, fasteners must withstand the complete load cycles (75 repetitions in a stepwise increasing fashion) and the complete crack opening-closing cycles (59 repetitions in a stepwise increasing fashion), before performing an ultimate loading test on them under static loading. If a fastener cannot withstand the complete cycles, then the applied maximum loads introduced above are reduced until the complete cycles can be finished.
- The "C2" characteristic seismic capacity of the fastener in essence is given as its characteristic static capacity in cracked concrete (with static crack width of w = 0.5 mm) multiplied by the ratio of the reduced maximum load needed to achieve the complete cycle of the protocols related to the original maximum load defined above for a given protocol. Table 1 summarizes the details for tension capacity.

It can be observed that the "C2" characteristic seismic capacity of fasteners (listed in their European Technical Assessment, ETA) has very limited practical significance in the seismic design of fasteners, and designers could rely on those fastener capacities in their actual design in very rare cases, if ever. The reasons are as follows:

- Fastener seismic capacity depends on the actual damage of the concrete substrate during an earthquake event. This damage can be characterized by the actual width of the crack formed at the location of the fastener and the actual number of crack opening-closing cycles acting on the fastener.
- The actual crack anatomy described above depends on the intensity and duration of the actual ground motion, the actual building response with all of its non-

linearities, the actual seismic weight (or seismic mass) incorporated with the fastener or group of fasteners, the exact actual location of the connection, and the geometry, detailing and structural response of the member at the exact actual location of the connection.

- It can be expected in very rare cases that the actual fastener or group of fasteners of interest are located in beams of specific moment resisting frames designed and detailed as per the work of Wood et al (2010) and exposed to strong motions applied to those building prototypes in the work of Wood et al (2010).
- Consequently, the cycle numbers in the assessment protocols of the "C2" seismic performance category are either too low or too high for actual seismic design of fasteners.
- Considering different Importance Levels and the corresponding design return periods of events it can be observed that the 475-year return period and the design acceleration spectrum selected for the "C2" assessment is not suitable for the determination of fastener capacity in any other loading scenario, not even being applied on the same prototype buildings used in the work of Wood et al (2010).
- If fasteners are located in walls, diaphragms, columns, or slabs in actual design, then the "C2" characteristic seismic capacity values cannot be used since those correspond to fasteners located in specific beams of moment resisting frames loaded with specific ground motions.
- There are well known discrepancies and simplifications in the development of the "C2" assessment protocols, which are not always demonstrated to be evidence based or conservative. One of these known discrepancies is the arbitrarily selected 0.5 mm and 0.8 mm crack widths, that was demonstrated to lack well-established scientific evidence (Borosnyoi-Crawley, 2024a, 2024b).

Table 1. Determination of the maximum theoretical characteristic "C2" seismic tension load capacity ($N_{Rk,C2}$) in accordance with EOTA TR 049

| |
|---|
| ***Reference test 1:*** Static pull-out tests with w = 0.5 mm crack width in low and high strength concrete (mean ultimate tensile capacities are $N_{u,m,3}$ and $N_{u,m,4}$ respectively)<br>Determination of reference characteristic capacity → $N_{Rk,0} = \min(N_{Rk,3}\ ;\ N_{Rk,4})$ |
| ***Reference test 2 (C2.1a and C2.1b):*** Static pull-out tests with w = 0.8 mm crack width in low and high strength concrete (mean ultimate tensile capacities are $N_{u,m,C2.1a}$ and $N_{u,m,C2.b}$ respectively)<br>Check → If $N_{u,m,C2.1a} \geq 0.8 \cdot N_{u,m,3}$ and $N_{u,m,C2.b} \geq 0.8 \cdot N_{u,m,4}$ then $\alpha_{C2.1} = 1.0$ |
| ***Pulsating tension load tests (C2.3)*** with $N_{max} = 0.75 \cdot N_{u,m,C2.1a}$<br>Check → If the full cycle can be completed and $N_{u,m,C2.3} \geq 0.9 \cdot N_{u,m,C2.1a}$ and $\delta_m(0.5 \cdot N/N_{max}) \leq 7$ mm then $\alpha_{C2.3} = 1.0$ |
| ***Varying crack width tests (C2.5)*** with $N_{w1} = 0.4 \cdot N_{u,m,C2.1a}$ and $N_{w2} = 0.5 \cdot N_{u,m,C2.1a}$<br>Check → If the full cycle can be completed and $N_{u,m,C2.5} \geq 0.9 \cdot N_{u,m,C2.1a}$ and $\delta_m(\Delta w=0.5) \leq 7$ mm then $\alpha_{C2.5} = 1.0$ |
| ***Maximum theoretical characteristic "C2" seismic tension load capacity*** is taken as the static tension capacity with w = 0.5 mm crack width:<br>$N_{Rk,C2} = N_{Rk,0}$ (since $N_{Rk,C2} = \alpha_{C2} \cdot N_{Rk,0}$ where $\alpha_{C2} = \alpha_{C2.1} \cdot \min(\alpha_{C2.3};\alpha_{C2.5}) = 1.0$) |

## HOW TO IMPROVE FASTENER SEISMIC DESIGN IN NEW ZEALAND?

It is emphasized that the general design methods for fasteners in concrete in accordance with EN 1992-4:2018 (or ACI 318) are long-established, refined methods. The major gap in the seismic design of fasteners is the lack of capacity information that designers could

safely and economically use in their actual calculations.

The currently available state-of-the-art in fastener seismic assessment (i.e. the “C2” test protocols) was not developed with the aim of providing fastener capacity information for general seismic design and, therefore, it cannot provide a basis for that. But, without a doubt, the “C2” assessment method is currently the most sophisticated way of fastener seismic assessment.

As a representative, New Zealand specific illustration, Figure 1 indicates the design target acceleration spectrum of the original research by Wood et al (2010) developed for the seismic demand corresponding to a 475-year return period event in Charter Oak (Los Angeles County, California, USA), in comparison to four DZ TS 1170.5:2024 spectra for New Zealand locations. It can be observed that assuming the fasteners being located in the beams just outside the plastic hinge zone in the same prototype buildings used in the original research, the usability of the developed fastener assessment protocol “C2” would be very limited in New Zealand, even in Importance Level 2 buildings. Importance levels other than IL2 or other structural members than beams of specific moment resisting frames, as highlighted above, cannot be designed based on the “C2” characteristic seismic capacity of fasteners.

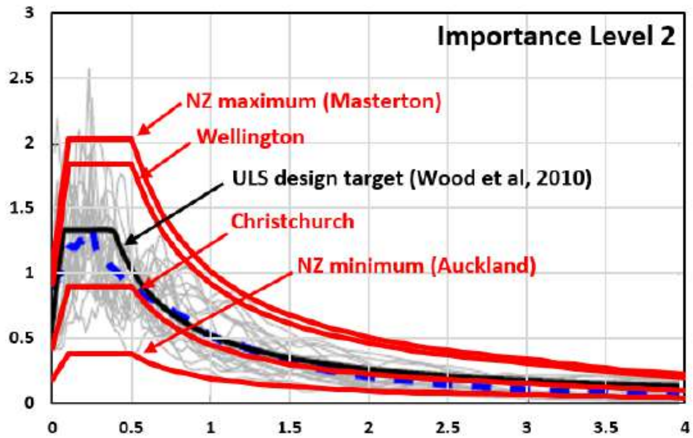


Figure 1. Design target acceleration spectrum of the original research of Wood et al (2010), in comparison to four DZ TS 1170.5:2024 spectra for New Zealand locations.

It is apparent that the output of fastener seismic assessment cannot be one single number for the fastener capacity. Development of a more refined approach is needed. The approach must reflect on the fact that a fastener’s actual seismic capacity depends on the actual damage of the concrete substrate during an earthquake event. Figure 2 schematically indicates such dependence.

The New Zealand engineering community, and especially the NZS 3101 Development Group (within Concrete NZ Learned Society) must consider developing the framework of a holistic design and assessment for fasteners. The framework must allow:

- Seismic assessment and design of fasteners for different return period events.
- Determination of characteristic fastener capacity that considers different damage levels of the substrate.
- On the assessment side it requires multiple load cycling protocols with different cycle counts and fastener loads, as well as multiple crack cycling protocols with different cycle counts and crack widths.
- On the design side it requires appropriately calibrated strength reduction factors for the different levels of consequences of failure.

Further details of the above are discussed elsewhere (Borosnyoi-Crawley, 2024c).

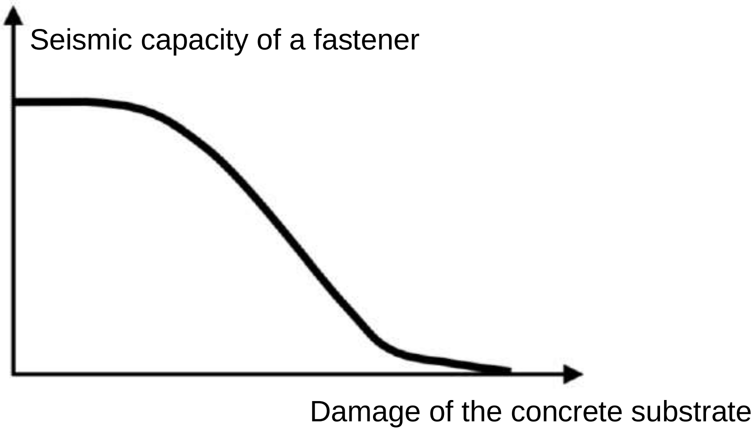


Figure 2. Schematic representation of the seismic capacity of a fastener related to the actual damage of the concrete substrate during an earthquake event.

## CONCLUSIONS

The fastener assessment method known in New Zealand as “C1” has been omitted from the ACI 355 codes in the ACI-CODE 355.2 and ACI-CODE 355.4 drafts versions open for public discussion from December 20, 2023 to February 3, 2024, and the fastener assessment method known in New Zealand as “C2” has been suggested instead of it. This improvement is welcome since the method “C1” has been proven to be inferior for seismic assessment, demonstrated by multiple scholars in the literature. NZS 3101 should follow this good example and the current references that include an allowance for the assessment method “C1” need urgent replacement in NZS 3101, especially since performance category “C1” currently does not apply to New Zealand in accordance with EN 1992-4:2018.

NZS 3101 is a primary reference document in NZBC B1 VM/AS, and EN 1992-4:2018 is the superseding document in effect for a secondary reference document in it. Consequently, EN 1992-4:2018 is currently an NZBC B1 Verification Method, and therefore designers in New Zealand are expected to perform the seismic design of fasteners accordingly. This paper highlighted certain specifics of the EN 1992-4:2018 fastener design, which may be challenging for practitioners in New Zealand.

It was demonstrated that the “C2” characteristic seismic capacity of fasteners (listed in their European Technical Assessment, ETA) has very limited practical significance in the seismic design of fasteners, and designers could rely on those fastener capacities in their actual design in very rare cases, if ever. It is emphasized that the general design methods for fasteners in concrete in accordance with EN 1992-4:2018 (or ACI 318) are long-established, refined methods. The major gap in the seismic design of fasteners is the lack of capacity information that designers could safely and economically use in their actual calculations. Appropriate directions for the development of fastener seismic assessment and design in New Zealand have been provided in this paper.

## APPENDIX

Figure A.1 explains the terms used in this paper through the example of the restraint of an engineering system component (where the fastening system is a connection of non-structural elements to the concrete of the supporting structure). The terms defined in Figure A.1 (whichever is relevant) can be extended for the use in relation to fastening

systems that are connections of structural elements to the concrete of the supporting structure.

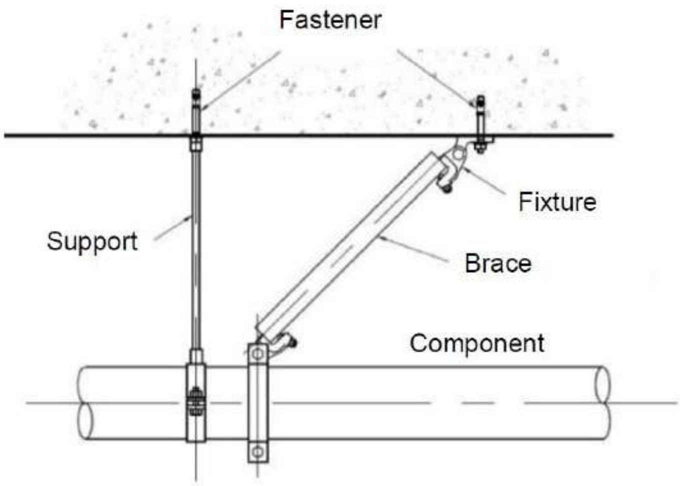


Figure A.1. Elements of a fastening system

**Fastening (system)** – Connection (system) which transmits actions to the concrete of the supporting structure, between elements of structural components or between elements of non-structural and structural components.

**Component** – An individual part of a structural system or an engineering system; an item that can be considered separately/independently for the purposes of assessing the load transfer and/or load paths.

**Supporting structure** – The primary earthquake resisting part of the structure.

**Fixture** – A structural attachment between a component and a fastener, or between a brace and a fastener, or between a support and a fastener that transfers loads to the supporting structure; an item that is being secured to the supporting structure via fasteners.

**Brace** – An item of the seismic restraint (system) used to transfer seismic loads from a component to a fixture.

**Restraint (system)** – A structural assemblage of items to transfer seismic loads from a component to multiple fixtures.

**Support** – A structural assemblage of items to transfer loads from a component to a fixture.

**Fastener** – Device for load transfer that is a) for concrete: either post-installed into hardened concrete or installed into position prior to the casting of concrete (known as cast-in fastener) and is conform to the design and testing requirements of NZS 3101; b) for concrete filled metal deck: either post-installed into hardened concrete or installed into position prior to the casting of concrete (known as cast-in fastener) and is designed and tested by appropriate seismic methods and protocols; c) for steel: prefabricated item that

is conform to the design and testing requirements of NZS 3404 or AS/NZS 4600. Note: Type b) and c) fasteners are outside of the scope of this paper.

**Post-installed mechanical anchor –** Fastener for load transfer in hardened concrete that utilizes interlocking or frictional force transfer or the combination of the two. Examples are undercut anchors, screw anchors, expansion anchors.

**Post-installed adhesive anchor** – Fastener system with the combination of adhesive and anchor items for load transfer in hardened concrete, with an anchor hole diameter not greater than 1.5 times the anchor diameter, that transfers loads to the concrete by bond between the anchor and the adhesive, and bond between the adhesive and the concrete. Examples are injection anchor systems, capsule anchor systems. Bonded expansion anchors are also part of this anchor category.

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# SEISMIC TESTING OF CAST-IN AND POST-INSTALLED ANCHORS WITH A COMMERCIALLY AVAILABLE TOLERANCE DEVICE

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## SUMMARY

It is common that post-installed anchors encounter locations in the concrete where reinforcement interferes with their installation. In some cases, the initial hole can be discarded, and the anchor re-installed where reinforcement does not interfere. On the other hand, there are many cases where the prefabricated fixtures do not provide this flexibility and costly modifications are required. Furthermore, it is common that cast-in anchors encounter location tolerance challengers and are often discarded for a post-installed solution. This paper will investigate the seismic shear testing of both post-installed and cast-in anchors when mitigating the tolerances issues using a commercially available device.

## INTRODUCTION

Regulations are constantly changing to keep up with the dynamic nature of international standards. In New Zealand, seismic testing requirements for post-installed anchors are stipulated in NZS3101.1 Chapter 17 *(Standards New Zealand 2006)* which references ETAG 001 Annex E *(EOTA April 2013)* and TR 45 *(EOTA February 2013)*. From the same chapter, the design requirements for both post-installed and cast-in headed stud anchors follow the Concrete Capacity Design (CCD) method, like Eurocode 2 BS EN1992-4 *(British Standards 2018)* & ACI 318 Chapter 17 *(American Concrete Institute 2019)*. Furthermore, it is important that the systems used for connections are qualified to ensure they perform accordingly. Post-installed anchors are required to undergo a vigorous prequalification requirement by testing in accordance with the European Assessment Document (EAD). Once testing is complete, a European Technical Assessment (ETA) is issued which demonstrates compliance with the prequalification. Cast-in headed stud anchors do not need the same prequalification as post-installed anchors, however they need to follow the design requirements in NZS3101.1 Chapter 17 *(Standards New Zealand 2006)* which are aligned with the European and American Standards mentioned above.

When considering a commercially available tolerance device such as the OrBiPlate™, a serrated orbital plate with toothed washer and serrated flange head bolt, the testing required is focused on the performance in the system that has not been explored previously. Therefore, the testing regime only investigates the shear performance of the tolerance enabling device for both cast-in and post-installed systems, as the tensile performance remains unaffected. It includes shear performance for category C1 and C2 seismic test regimes stipulated in TR049 *(EOTA 2016)*. It also includes static shear testing in concrete and all results are compared with the existing published data.

The results of the testing will provide designers with a performance-based solution for both post-installed and cast-in headed stud anchors when used with the commercially available tolerance enabling device.

**BOLTED/CAST-IN CONCRETE CONNECTIONS**

Typical bolted/cast-in systems

Typically bolted/cast-in systems for concrete and steel connections comprise of a cast-in element (also referred to as a ferrule) and a connection plate or fixture, typically made of steel. These elements are used to connect steel-to-concrete or concrete-to-concrete as shown in the figure 1.

(a) Steel to concrete connection

(b) Concrete to concrete connection

Figure 1. Typical Bolted/cast-in connections

For a typical connection, a standard fixture hole is designed into the steel fixture plate, so that the specified bolt can pass through and connect into a cast-in ferrule. With this set-up, cumulative construction tolerances may cause problems with lining up the connection or steel fixture plates.

Tolerance enabling device – cast-in application

This article discusses a commercially available tolerance enabling device which overcomes the cumulative construction tolerances using a serrated orbital plate, toothed washer and serrated flange head bolt. This system relies on a specific hole size in the connection or steel fixture plates to suit the system which will connect into the cast-in ferrule. Figure 2 depicts the components for the bolted connection system consisting of the commercially available tolerance device and Figure 3 depicts its connection to a cast-in ferrule.


Figure 2. Components for commercially available tolerance device


Figure 3. Connection of tolerance device to a cast-in ferrule

Review of qualification processes for bolted/cast-in systems

NZS 3101.1 Chapter 17 *(Standards New Zealand 2006)* follows the CCD method for cast-in headed stud anchors. From this, it is clear the most appropriate standard to use for the seismic calculation of the cast-in ferrule system would be the ACI 318-19 Chapter 17 *(American Concrete Institute 2019)*.

Tensile Capacity – Seismic Design & Qualification – cast-in system

With respect to tensile capacity, ACI 318-19 clause 17.7 *(American Concrete Institute 2019)* provides theoretical calculation methods for steel, concrete and pullout failure modes and provides further reductions factors in clause 17.10 for each failure mode to allow for seismic conditions.

Shear Capacity – Seismic Design & Qualification – cast-in system

With respect to shear capacity, ACI 318M-19 clause 17.7 *(American Concrete Institute 2019)* provides theoretical calculation methods for steel, concrete and pryout failure modes but does not require any further reduction factors for seismic conditions according to clause 17.10. Having said this, when considering the unique nature of the bolted connection system incorporating a tolerance enabling device which requires steel interaction between the serrated orbital plate and the toothed washer, it warrants an investigation to explore how seismic conditions would affect this potential failure mode. As such, the most suited seismic testing for shear steel performance is detailed in ACI 355.2 *(American Concrete Institute 2022)*, which is referenced in ACI 318M-19 Chapter 17 *(American Concrete Institute 2019)* for post-installed anchors. ACI 355.2 clause 9.6 *(American Concrete Institute 2022)* is equivalent to the seismic shear testing protocol of TR049 clause 2.3 category C1 shear *(EOTA 2016)*. Therefore, it was decided to organise seismic shear testing for the commercially available tolerance enabling device consisting of the bolt, orbital serrated plate and toothed washer interaction based on TR049 clause 2.3 category C1 shear *(EOTA 2016).*

**CAST-IN TESTING PROGRAMME**

The tolerance enabling device (referred as orbital washer from here onwards) was tested according to C1 seismic shear testing protocol in TR049 *(EOTA 2016)* with different sized bolts (M16 and M20) and plate thicknesses in the Swinburne SMART Structures Laboratory. Images of the typical test setup are shown in Figure 4. The system was also tested in two different orientations as shown in Figures 5 and Figure 6.

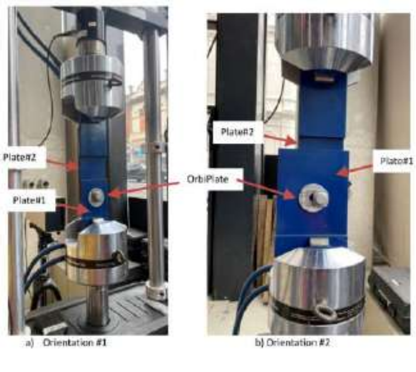


Figure 4. Shear test setup in MTS 250 kN actuator with 2 different orientations

Figure 5. Orientation #1

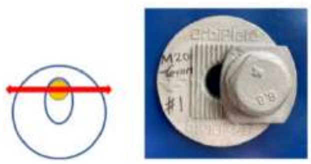

Figure 6. Orientation #2

For Orientation #1, depicted in Figure 5, the bolt was positioned 10mm off centre (half-way of the slot) and the load applied parallel to the slot for the M16 (ORB2016BGH) with 12mm plate and M20 (ORB2020BGH) with 16mm plate. This set-up was used to allow for a more challenging condition on the bolts in bending and on the teeth between the two washers (i.e., between the serrated orbital plate and tooth washer).

For Orientation #2, depicted in Figure 6, the bolt was positioned 20mm off centre (outer-most position on the slot) and the load applied perpendicular to the slot for M20 (ORB2020BGH) with a 6mm plate. This set-up was used to allow for a more challenging condition on the ply in bearing and on the teeth around the big washer (serrated orbital plate).

The orbital washer was subjected to alternating shear load protocol as per Figure 7 and the load levels indicated in Table 1.

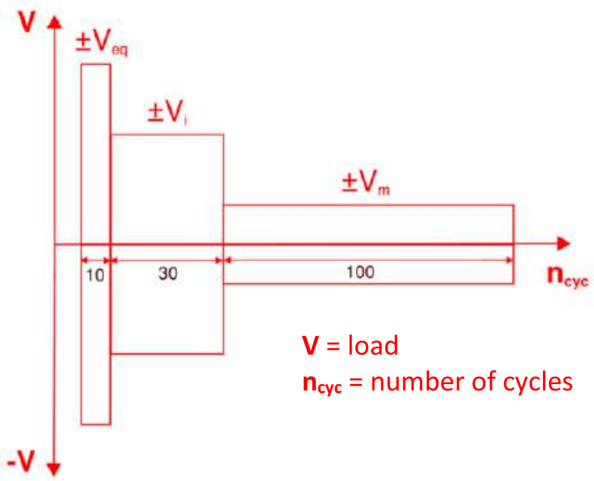


Figure 7. Cyclic shear loading protocol - Figure 2.8 from TR049

The derivation of the load levels in Table 1 were based on equations 2.9 to 2.11 from TR049 *(EOTA 2016)* as follows.

$V_{eq} = 0.35*A_s*f_{uk}$ equation (1)

Where

$A_s$ = [mm$^2$] – effective stressed cross section area of steel element in the shear plane;

$f_{uk}$ = [N/mm$^2$] – characteristic steel ulitmate tensile strength (nominal value) iof the finished product;

$V_i = 0.75*V_{eq}$ [N] equation (2)

$V_m = 0.5*V_{eq}$ [N] equation (3)

Note: The effective stressed cross section area ($A_s$) and the characteristic steel ultimate tensile strength ($f_{uk}$) were both based on the cast-in ferrule.

Table 1. C1 alternating shear loading level for orbital washer with two different bolt sizes.

| | | | Load level (kN) | | |
|---|---|---|---|---|---|
| Specimen | $A_s$ | $f_{uk}$ | $\pm V_{eq}$ | $\pm V_i$ | $\pm V_m$ |
| ORB2020BGH - M20 bolts | 263.4 | 400 | 36.9 | 27.7 | 18.5 |
| ORB2016BGH - M16 bolts | 158 | 500 | 27.6 | 20.7 | 13.8 |

Table 1 Reference: OrbiPlate test report *(Swinburne University 2021)*

Test Results – cast-in system

Fifteen test specimens were subjected to a complete program of alternating shear protocol depicted in Figure 7. Examples of the load-displacement curves for this test protocol are shown in Figures 8 and 9.

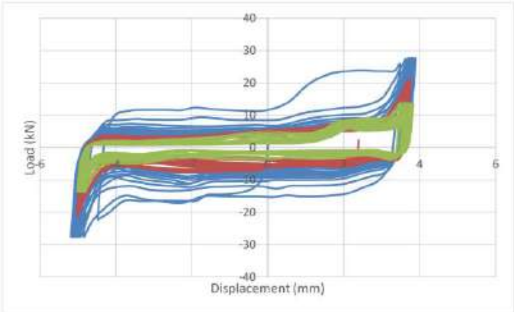


Figure 8. Cyclic shear M16 bolts, 12mm plt.

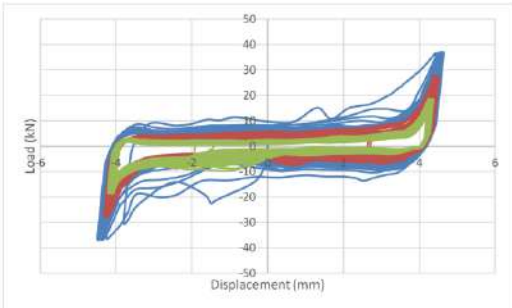


Figure 9. Cyclic shear M20 bolts, 16mm plt

Reference: OrbiPlate test report *(Swinburne University 2021)*

***Legend:*** *= 10 cycles at $V_{eq}$* *= 30 cycles at $V_i$* *= 100 cycles at $V_m$*

Following completion of the alternating shear protocol, the tolerance enabling device system was tested to failure in shear to determine the residual shear capacity. Table 2 provides a summary of the residual shear test results tabulating the number of tests, average residual shear load ($V_{res,m}$) and the mode of failure (MoF) as follows.

Table 2. Summary of residual shear test results for cast-in system

| **Test Set-up** | **No. of tests** | **Mode of failure (MoF)** | **Average Residual shear load, $V_{res,m}$ (kN)** |
|---|---|---|---|
| Orientation #1 – M16 bolts with 12mm plates | 5 | Tooth washer split | 71.1 |
| Orientation #1 – M20 bolts with 16mm plates | 5 | Tooth washer split | 91.8 |
| Orientation #2 – M20 bolts with 6mm plates | 5 | Bolt shear | 150.8 |

As stipulated by Cl 3.1.2 TR049 *(EOTA 2016)*, the reduction factor on characteristic shear capacity from static loading is calculated as shown in Table 3.

Table 3. Reduction factor calculation on characteristic shear capacity per Cl 3.1.2 TR049

| **Test Set-up** | **$V_{res,m}$ (kN) (from table 2)** | **$V_{eq}$ (kN) (from table 1)** | **Ratio of $V_{res,m}/V_{eq}$** | **$\alpha_{v,C1}$ (reduction factor)** |
|---|---|---|---|---|
| Orientation #1 – M16 bolts with 12mm plates | 71.1 | 27.9 | 254.84% > 160 % | 1 |
| Orientation #1 – M20 bolts with 16mm plates | 91.8 | 36.9 | 248.78% > 160 % | 1 |
| Orientation #2 – M20 bolts with 6mm plates | 150.8 | 36.9 | 408.67% > 160 % | 1 |

Tables 2 and 3 Reference: OrbiPlate test report *(Swinburne University 2021)*

All specimens completed the C1 alternating shear protocol in TR049 *(EOTA 2016)* with the average residual shear load for 5 specimens in each test series exceeding more than 1.6 times the applied $V_{eq}$ for cyclic protocol, demonstrating the suitability of the serrated orbital plate and toothed washer interaction for the orbital washer system for seismic shear C1 category performance defined in TR049 *(EOTA 2016)*. According to Clause 3.1.2 in TR049 *(EOTA 2016)*, no reduction to static shear load is required if $a_{v,C1} = 1$. As such, Table 4 provides the seismic design table which reflects the proprietary connection system shear performance.

Table 4. Seismic Design Table $\phi V_{usc,seis}$ (kN) where $\phi = 0.75$ and based on f'c $\geq$ 32MPa

| Ferrule | OrbiPlate™ | Seismic Cracked $\varphi V_{usc,seis}$ Fixture Thickness (mm) 6-12 | Seismic Cracked $\varphi V_{usc,seis}$ Fixture Thickness (mm) 16 |
|---|---|---|---|
| FE16095GH | ORB2016BGH | 20.7 | - |
| FE20095GH | ORB2020BGH | 27.7 | 27.7 |
| TIM20x75G (NZ only) | ORB2020BGH | | |

Note: Seismic steel shear data is based on testing in accordance with ACI 355.2

Table 4, Reference: OrbiPlate Design Guide *(Ramset 2023)*

**POST-INSTALLED CONCRETE CONNECTIONS**

<u>Typical post-installed systems</u>

Post-installed connections can be either Mechanical or Chemical. With respect to this paper, we will be focusing on Chemical post installed anchors which comprised of a steel element (anchor stud) and the adhesive element (chemical anchor). These elements are typically used to connect steel to concrete as shown in Figure 10.

Like cast-in anchors, the standard fixture hole is designed into the steel fixture plate so that the head of the chemically bonded anchor stud can pass through and be torqued. On the concrete side, a hole needs to be drilled into the concrete at a diameter and depth which suits the chemical post installed anchor as per the engineer's design. A challenge in this process is the interference caused by steel reinforcement at the hole's location. When this happens, the hole needs to be discarded and then filled with cementitious grout. Another hole then needs to be drilled into the concrete at another location where there is no interference caused by steel reinforcement.

<u>Tolerance enabling device – post-installed application</u>

When the above occurs, the orbital washer system can be used to ensure the intended fixture is still usable, thus minimising potentially costly remedial works. Figure 11 depicts the components used for the post-installed anchor utilising the orbital washer.

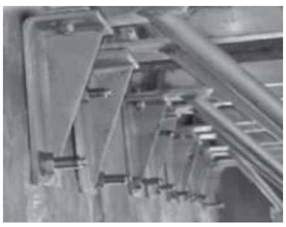

Figure 10. Chemical anchoring with anchor studs

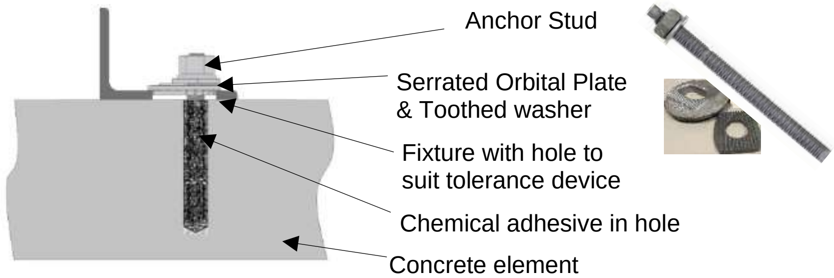


Figure 11. Post-installed Chemical Anchor components utilizing tolerance device

Seismic design for post-installed systems.

NZS3101 (A3) Chapter 17 clause 17.5.5 *(Standards New Zealand 2006)* states that 'post-installed adhesive anchor shall be designed in accordance with TR045'. This is a European Standard reference which has been superseded by Eurocode 2 BS EN1992-4 Annex C – Design of fastening under seismic action *(British Standards 2018)*.

Seismic qualification for post-installed systems

NZS 3101 (A3) Chapter 17 clause 17.5.5 *(Standards New Zealand 2006)* states that 'post-installed adhesive anchors shall pass the prequalification testing stipulated in ETAG 001 Annex E'. This is a European Standard reference which has been superseded by TR049 *(EOTA 2016)*. Therefore, the most appropriate standard to use for the seismic prequalification testing of the post-installed system would be the European Standard TR049 *(EOTA 2016)*. This can also be found in the EAD for bonded fasteners EAD 330499-02-0601 *(EOTA 2022).*

With respect to tensile capacity for post-installed chemical anchors using the orbital washer, no further testing for seismic actions is required given it will not influence the performance of the post-installed anchor in accordance with the ETA.

With respect to shear capacity for post-installed chemical anchors, further testing for seismic action will be required given the unknown performance of the potential stand-off geometry along with steel interaction between the serrated orbital plate and the toothed washer. As such, the most suited seismic testing for this would be the seismic shear testing protocol of TR049 *(EOTA 2016)* clause 2.3 category C1 shear and clause 2.4 category C2 shear. Therefore, it was decided to organize seismic shear testing (C1 & C2) on the orbital washer system interaction connected with a steel anchor stud used for the post-installed chemical anchors. Furthermore, to ensure monotonic shear behaviour remained consistent with the ETA, a static shear test programme in concrete for different fixture thicknesses was also actioned. The results of the static shear test programme is used to complement the seismic shear testing performed at SWUT.

**POST INSTALLED TESTING PROGRAMME**

Test Setup – post-installed system – in-air testing

The post-installed chemical anchor stud with the orbital washer system was tested for seismic shear performance with an M20 anchor stud (CS20260GH), fixed with ChemSet™ EPCON™ C6 Plus chemical anchor, and different plate thicknesses in the Swinburne University of Technology SMART Structures Laboratory. The typical test setup was identical to the bolted/cast-in system noted earlier in the paper (refer Figure 4). Both C1 and C2 seismic shear testing was performed on this set-up and the plate thicknesses combination versus orbital serrated plate orientation is shown in Figures 12 and 13.

Figure 12. Orientation #1 with 16mm & 32mm plate

Figure 13. Orientation #2 with 6mm, 16mm & 32mm plate

For C1 seismic testing, the post-installed chemical anchor connection system was subject to alternating shear load protocol noted earlier in the paper (refer Figure 7) and the load levels indicated in Table 5.

The derivation of the load levels in Table 5 were based on equations 2.9 to 2.11 from TR049 *(EOTA 2016)* for the 6mm and 16mm plate. Refer equations 1,2 and 3 noted earlier in the paper.

For the 32mm plate, the $V_{eq}$ value was based on EAD testing data on grade 10.9 stud for ETA 18/0675 *(ZUS 2024)* and converted to grade 5.8 stud as follows,

For 10.9 grade stud, $V_{eq}$ = 52.3 kN
For 10.9 grade stud $f_{uk}$ = 937 kN and for 5.8 grade stud $f_{uk}$ = 520 kN

Reduction factor = 520/937 = 0.55 equation (4)
$V_{eq}$ = Reduction factor * $V_{eq,grade\ 10.9}$ equation (5)
$V_{eq}$ = 28.8 kN,

Applying equation (2) $V_i$ = 21.6 kN and applying equation (3) $V_m$ = 14.4 kN

Table 5. C1 cyclic loading level for post-installed

| Specimen | $A_s$ | $f_{uk}$ | Plate thk | Load level (kN) | | |
|---|---|---|---|---|---|---|
| | | | | $\pm V_{eq}$ | $\pm V_i$ | $\pm V_m$ |
| ORB2020BGH - M20 ChemSet Anchor Stud | 232.4 | 520 | 6mm & 16mm | 42.3 | 31.73 | 21.15 |
| ORB2020BGH - M20 ChemSet Anchor Stud | * | * | 32mm | 28.8 | 21.6 | 14.4 |

*Note: The $V_{eq}$ value for 32mm plate was based on EAD testing data on 10.9gr stud converted to 5.8gr stud

Table 5 Reference: OrbiPlate in-air testing report *(Swinburne University 2023)*

For C2 seismic testing, the post-installed chemical anchor stud with orbital washer system was subjected to alternating shear load protocol as per figure 14 and the load levels indicated in Table 6.

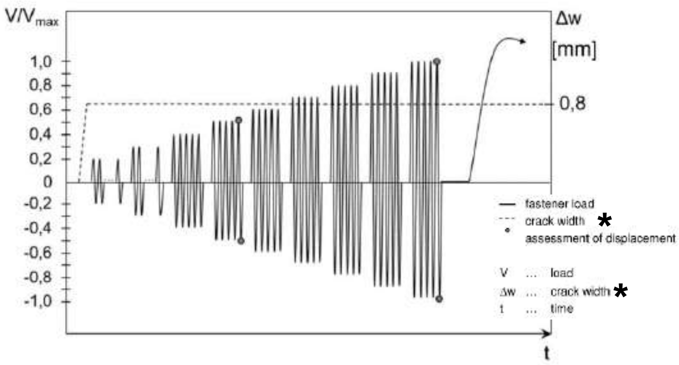


Figure 14. Cyclic C2 shear loading TR049 protocol *(EOTA 2016)*

Table 6. C2 cyclic loading level for post-installed connection system *(EOTA 2016)*

| $\pm V/V_{max}$ | Number of cycles | Crack width $\Delta w$ [mm]* |
|---|---|---|
| 0.2 | 25 | 0.8 |
| 0.3 | 15 | 0.8 |
| 0.4 | 5 | 0.8 |
| 0.5 | 5 | 0.8 |
| 0.6 | 5 | 0.8 |
| 0.7 | 5 | 0.8 |
| 0.8 | 5 | 0.8 |
| 0.9 | 5 | 0.8 |
| 1 | 5 | 0.8 |
| SUM | 75 | |

*Note: the cracked width is not applicable for in-air testing

$V_{max}$ is calculated as per the following equation:
$V_{max} = 0.85 * V_{um} * \alpha_{c2.4a}$ equation (6)
$V_{um}$ = 62.5 kN from EAD testing for European Technical Assessment ETA 18/0675 *(ZUS 2024)*
$\alpha_{c2.4a}$ = 0.6 from EAD testing for European Technical Assessment ETA 18/0675 *(ZUS 2024)*

$V_{max}$ = 31.9

Test Results – post-installed system – in-air testing

For C1 shear, 25 test specimens were subjected to a complete program of alternating shear protocol depicted in Figure 7. An example of the load-displacement curves for the C1 cyclic shear tests on the test specimens are shown in Figure 15. Furthermore, the average residual shear load following the cyclic testing is shown in Table 7 and the

reduction factor calculation on characteristic shear is shown in Table 8. Typical bolt shear Mode of Failure is shown in Figure 16.

Table 7. Summary of residual C1 shear test results for post-installed system

| Test Set-up | No. of tests | Mode of failure (MoF) | Average Residual shear load, $V_{res,m}$ (kN) |
|---|---|---|---|
| Orientation #2 – M20 Anchor Stud with 6mm plates | 5 | Bolt shear | 108.6 |
| Orientation #1 – M20 Anchor Stud with 16mm plates | 5 | Tooth washer split | 72.8 |
| Orientation #2 – M20 Anchor Stud with 16mm plates | 5 | Bolt shear | 98.3 |
| Orientation #1 – M20 Anchor Stud with 32mm plates | 5 | Bolt shear (4/5)* | 73.8 |
| Orientation #2 – M20 Anchor Stud with 32mm plates | 5 | Bolt shear | 78.3 |

*Note: Average of 4 test showing bolt shear failure

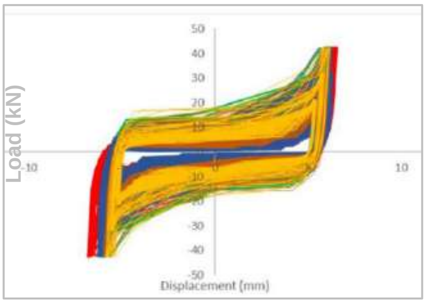


Figure 15. Cyclic C1 shear M20 Anchor Stud 16mm Plt – Orientation 1

Table 8. Reduction factor calculation for C1 shear on characteristic shear capacity per Cl 3.1.2 TR049 *(EOTA 2016)* for post -installed system

| Test Set-up | $V_{res,m}$ (kN) (from table 5) | $V_{eq}$ (kN) (from table 5) | Ratio of $V_{res,m}/V_{eq}$ | | $\alpha_{v,C1}$ (reduction factor) |
|---|---|---|---|---|---|
| Orientation #2 – M20 Anchor Stud with 6mm plates | 108.6 | 42.3 | 257.00% | > 160 % | 1 |
| Orientation #1 – M20 Anchor Stud with 16mm plates | 72.8 | 42.3 | 172.00% | > 160 % | 1 |
| Orientation #2 – M20 Anchor Stud with 16mm plates | 98.3 | 42.3 | 232.00% | > 160 % | 1 |
| Orientation #1 – M20 Anchor Stud with 32mm plates | 73.8 | 28.8 | 255.00% | > 160 % | 1 |
| Orientation #2 – M20 Anchor Stud with 32mm plates | 78.3 | 28.8 | 272.00% | > 160 % | 1 |

Figure 16. Typical bolt shear failure mode C1 test M20 Anchor Stud 32mm Plt – Orientation 2

For C2 shear, 25 test specimens were subjected to a complete program of alternating shear protocol depicted in figure 14. An example of the load-displacement curves for the C2 cyclic shear tests on the test specimens are shown in Figure 17. Furthermore, the average residual shear load following the C2 cyclic testing is shown in Table 9. Typical bolt shear & toothed washer splitting Mode of Failure in C2 testing is shown in Figure 18.

Table 9. Summary of residual C2 shear test results for post-installed system

| S.No. | Anchor | Orientation | Plate Thicknesss (mm) | Average Residual Load (kN) ($V_{res}$) | Failure Mode | $V_{res}/V_{max}$ |
|---|---|---|---|---|---|---|
| 1 | ChemSet Anchor Studs | 1 | 16 | 78.3 | Toothed washer splitting | 2.5 |
| 2 | ChemSet Anchor Studs | 1 | 32 | 91.2 | Bolt Shear | 2.9 |
| 3 | ChemSet Anchor Studs | 2 | 6 | 111.0 | Bolt Shear | 3.5 |
| 4 | ChemSet Anchor Studs | 2 | 16 | 99.8 | Bolt Shear | 3.1 |
| 5 | ChemSet Anchor Studs | 2 | 32 | 76.4 | Bolt Shear | 2.4 |

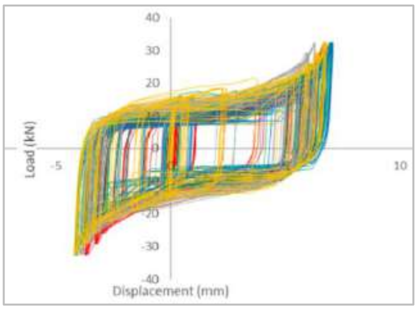


Figure 17. Cyclic C2 shear M20 Anchor Stud 16mm Plt – Orientation 1

Tables 7, 8 and 9 Reference: OrbiPlate in-air testing report *(Swinburne University 2023)*

As mentioned earlier, the $V_{u,m}$ for the M20 Anchor Stud as derived from EAD testing for European Technical Assessment ETA 18/0675 *(ZUS 2024)* was 62.5 kN. As the average residual load from Table 9 exceeds the $V_{u,m}$ value, this would suggest that $\alpha_{c2.4}$ in the EAD testing data will not require further adjustment.

Figure 18. Typical bolt shear and toothed washer splitting failure mode C2 test M20 Anchor Stud 16mm Plt – Orientation 1

Test Setup – post-installed system – in-concrete testing

The post-installed chemical anchor stud with orbital washer system was assessed for monotonic shear performance with an M20 anchor stud and different plate thicknesses installed in 30 MPa concrete at the ITW Product Engineering Laboratory (PEL). The typical test setup is shown in Figure 19.

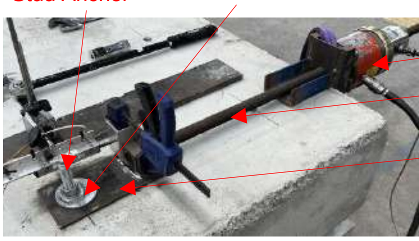


Figure 19. Monotonic Shear test setup in concrete

Test Results – post-installed system – in-concrete testing

The monotonic shear tests were conducted at the PEL according to the orbital plate orientations discussed in Figures 12 and 13. The fixture plate thicknesses assessed were 6mm, 12mm, 16mm and 32mm. The Mode of Failure for all the tests was the steel element. PEL Registered Engineers derived the Lower Characteristic capacity, compared the data against ETA 18/0675 *(ZUS 2024)* and subsequently derived the appropriate reduction factors as presented in Table 10. In general, no reduction factor was required except for a fixture thickness of 32mm.

Table 10. Summary of monotonic shear test results

| Test Set-up | No. of Tests | Mode of Failure (MoF) | Lwr. Char. from Test Results $R_u$ (kN) | ETA-18/0675 $V_{Rks}$ (kN) | $\phi_{orb}$ (reduction factor) |
|---|---|---|---|---|---|
| Orientation #2 – M20 Anchor Stud - 6mm Plt. | 5 | Steel Element | 100.4 | 61.0 | 1 |
| Orientation #1 – M20 Anchor Stud - 16mm Plt. | 5 | Steel Element | 84.0 | 61.0 | 1 |
| Orientation #2 – M20 Anchor Stud - 16mm Plt. | 5 | Steel Element | 77.9 | 61.0 | 1 |
| Orientation #1 – M20 Anchor Stud - 12mm Plt. | 5 | Steel Element | 84.3 | 61.0 | 1 |
| Orientation #2 – M20 Anchor Stud - 12mm Plt. | 5 | Steel Element | 81.2 | 61.0 | 1 |
| Orientation #1 – M20 Anchor Stud - 32mm Plt. | 5 | Steel Element | 76.5 | 61.0 | 1 |
| Orientation #2 – M20 Anchor Stud - 32mm Plt. | 5 | Steel Element | 53.0 | 61.0 | 0.87 |

**CONCLUSION**

The assessment & testing of cast-in/bolted connection system using the orbital washer system proved the suitability of the system to accomodate cyclic/seismic shear loading in accordance to TR049 *(EOTA 2016)* clause 2.3 category C1 shear. The testing confirmed that the tooth washer interaction with the orbital serrated plate is suitable for the seismic shear loads it was measured against, which have been converted to a shear design capacity of the system in the corresponding OrbiPlate™ Design Guide *(Ramset 2023).*

The design of the post-installed connection system using the orbital washer also proved that the system is capable of withstanding in-air cyclic/seismic shear loading in accordance with TR049 *(EOTA 2016)* clause 2.3 category C1 shear and clause 2.4 category C2 shear. The in-concrete monotonic shear testing in concrete shows that when fixture thicknesses of 6mm, 12mm and 16mm are used, the system also met the performance requirements of the post-installed anchor according to ETA-18/0675 *(ZUS 2024)*. It was found that for a fixture thickness of 32mm, the post-installed system with orbital washer would need to have a reduction factor of 0.87 applied to the steel shear capacity published in the European Technical Assessment ETA 18/0675 *(ZUS 2024)* for ChemSet EPCON C6 Plus.

**ACKNOWLEDGMENT**

Swinburne University of Technology, Reid Construction Systems, Ramset.

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# COUNTERFORT WALLS AND MAXIMISING PEDESTRIAN SPACES IN QUEENSTOWN

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## SUMMARY

The St Josephs Wall is a reinforced concrete counterfort wall constructed in Queenstown as part of the Arterial Road. A description of the wall design and some construction challenges are presented.

## INTRODUCTION

In recent years, rapid population growth and increased visitor numbers to the Queenstown area have imposed significant demand on the local road transport network. This has resulted in transport issues on several roads in the Whakatipu Basin, with an acute congestion problem in the Queenstown town centre. The congestion affects residents, workers and tourists, as well as travellers to the township of Glenorchy, as the only road to this town passes through the Queenstown CBD.

The Queenstown Integrated Transport Business Case, as well as previous transport studies, have identified the Queenstown Town Centre Arterial Road (Arterial Road) as a key component of the overall Whakatipu Basin transport solution. The Arterial Road is expected to provide improved access to and through the town, increased economic performance for Queenstown, and improved liveability and visitor experience in the town centre. This is achieved by reassigning the traffic from the current route through the historic core to a new corridor, thus freeing the town centre for pedestrians, cyclists and other road users.

The Arterial Road corridor extends from the Frankton Road (State Highway 6A) and Suburb Street intersection on the eastern side of Queenstown, through to the One Mile Creek roundabout on the Glenorchy-Queenstown Road on the western side of the town centre. The Arterial Road project has been split into three separable portions. Separable portion 1 extends from the Suburb Street intersection through to Gorge Road and has been termed Arterial Stage 1. Arterial Stage 1 is currently under construction and is being delivered by the Kā huanui a Tāhuna alliance. The Kā huanui a Tāhuna alliance is a programme alliance comprising NZ Transport Agency Waka Kotahi and Queenstown Lakes District Council as owner participants, and Beca Limited, Downer Group, Fulton Hogan and WSP as the non-owner participants. Construction of stages 2 and 3 of the Arterial Road has been deferred to after the completion of Stage 1.

The Arterial Stage 1 route generally follows existing roads (Frankton Road SH6A, Melbourne Street and Henry Street), however a portion of new road is required to connect Melbourne Street and Henry Street, between intersecting roads Beetham Street and Ballarat Street. Due to the steep topography of Queenstown, the increased width required for the new road alignment and pedestrian space cuts into the uphill properties and fills in the downslope properties, resulting in several new retaining structures being required above and below the

road. Along the 320m stretch of the Arterial Stage 1 road that requires retaining structures, over 800m length of new walls has been constructed. For most of the taller walls, an anchored kingpost design has been adopted. However, for a portion of the alignment near the St Josephs Church, an anchored design was not a viable solution, and a counterfort wall was designed and constructed. The St Josephs Wall is understood by the design team to be one of the largest counterfort walls in New Zealand and this paper will describe the chosen design solution, challenges encountered during construction and how the resilience of a concrete structure achieved the desired project outcomes.

## SITUATION

Prior to construction, the existing layout of Melbourne Street outside the St Josephs Church and School grounds was narrow and dropped steeply to meet Beetham Street. The school grounds were retained by a series of structures and batters: a dry stacked rock wall up to 3.5m in height, a 1m tall concrete wall above, and steep, vegetated slopes. The dry stacked rock wall also continued up Beetham Street to meet the concrete wall from above.

The existing walls were considered to be in poor condition. The dry stacked stone wall had portions of stonework missing, leaving unretained steep earth slopes exposed (refer Figure 1). There was also evidence of movement of the concrete wall along Beetham Street, with the wall leaning over towards the road and large cracks through it.

Figure 1. Original dry stacked stone wall in front of St Josephs Church at the commencement of works.

The new alignment of the Arterial Stage 1 involves extending Melbourne Street past the Beetham Street intersection and through to the Henry Street and Ballarat Street intersection. The steep gradient of Melbourne Street as it approached Beetham Street was reduced with the new road alignment and the crest of the road moved further east to provide a gentler

gradient for the new road. This resulted in undercutting the existing stacked stone wall as well as creating a new steep slope in front of the St Josephs Church.

Furthermore, the Arterial Stage 1 alignment is a two-lane road with raised central median along a large portion of its length, with a 2.5m wide footpath provided on either side for pedestrians, which is considerably wider than the original road. This increase in road width required further modification to the soil retaining system. Due to the condition and construction methods of the existing walls, modifiction of these to achieve the new road alignment requirements would not be possible. Therefore, a new retaining wall as identified to be required if land purchase was to be avoided.

The total height to be retained at the tallest part of the slope is approximately 7m, while the length of slope to be retained is roughly 85m long. Above the majority of the slope to be retained is the St Josephs School field with no structures present (though it is intermittently used as an access drive and sees the occasional vehicle surcharge). However, located at the eastern end of the slope is the St Josephs Church building. The St Josephs Church is a Catgeory 2 listed Historic Place and could not be disturbed by construction activities. A schematic plan of the site is given in Figure 2, showing the features of interest.

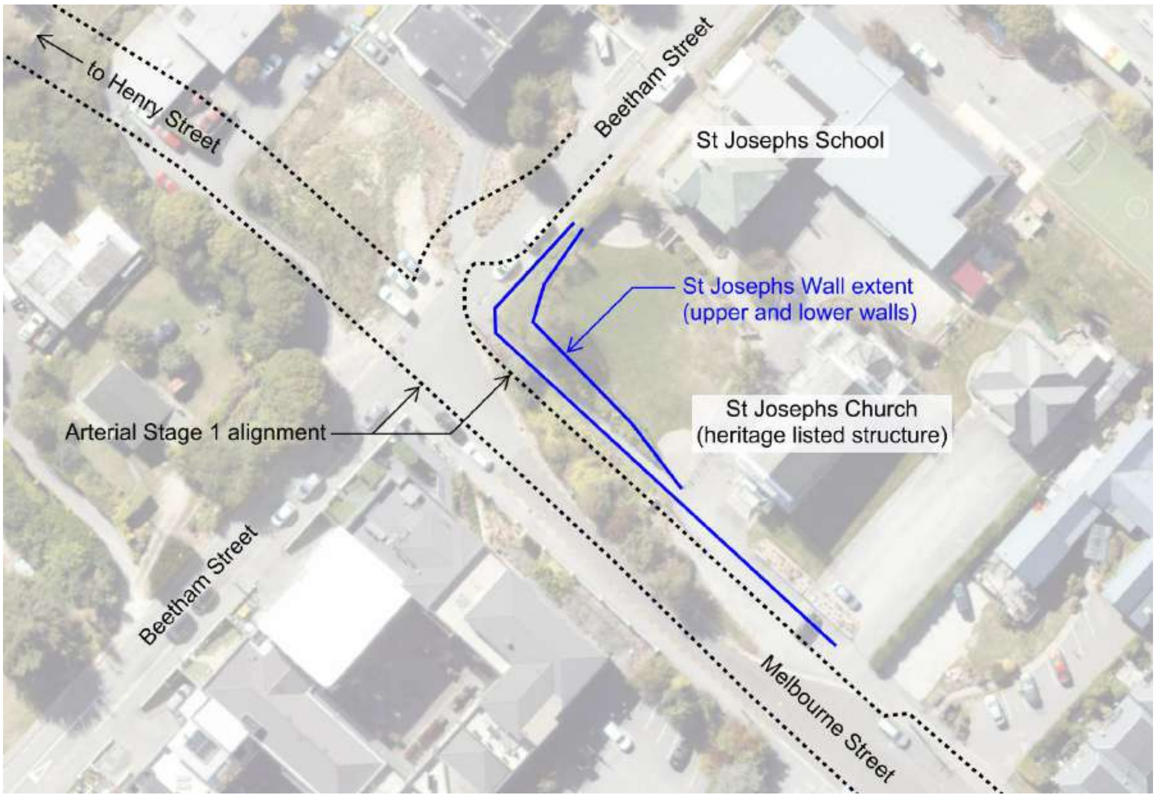


Figure 2. Schematic plan of the St Josephs Wall site.

## SELECTION OF DESIGN SOLUTION

The geology under the Arterial Stage 1 alignment is understood to be glacially formed. The overall site generally comprises beach material deposited when Lake Whakatipu was at higher

levels, overlying glacial deposits incorporating a variety of materials sands, gravels and occasional boulders.

For most of the retaining walls required for the Arterial Stage 1 alignment, an anchored kingpost wall system was identified as the preferred solution to reduce pile sizes and overall wall structure. However, for the walls on the upslope side of the Arterial Stage 1, an anchored solution would require permanent construction outside of the road designation. This could limit the options for further development of the adjacent private land, particularly any construction requiring deep foundations. For the St Josephs Church and School site, an agreement was not able to be reached with the owners to install anchors into the private property. Therefore, the anchored kingpost system typical of elsewhere on the Arterial Stage 1 could not be used.

Through the design optioneering phase, alternative retaining wall types were considered that did not require permanent encroachment onto private property, including secant pile type walls and a counterfort concrete wall. A concrete counterfort wall was selected as the preferred solution as it would minimise the land required for the permanent works whilst maximising the usable space for the roadway and pedestrian footpath in front of the wall. Temporary access to the St Josephs land was granted for removal of the existing retaining wall and temporary cut batters into a portion of the playing fields, to enable the counterfort wall to be constructed.

## DESCRIPTION OF DESIGN SOLUTION

The decision was made to split the retaining system into an upper and a lower wall as existed previously on the St Josephs site. This was done to reduce the retained height for the lower wall and minimise construction costs. The two walls are separated by a gently sloping planted area. The upper wall is a reinforced concrete low wall with a retained height of up to 0.75m, which roughly follows the alignment of the original upper wall. The upper wall provides the boundary for the school playing field and supports the school boundary fence. The lower wall is the main retaining structure. The design solution for this wall is to construct a counterfort wall to retain the slope and use a pile to resist seismic induced sliding and overturning demands.

As the retained height varies considerably along its length, the counterfort wall was separated into two distinct portions:

- Type A: Maximum 2.5m high counterfort wall with a 0.6m high slope/retaining wall above the counterfort wall to support the playing field/site access. The wall has a 4.7m wide base slab and does not require a pile.
- Type B: Maximum 5.0m high counterfort wall with a 2m high slope/retaining wall above the counterfort wall to support the school playing field. The wall has a 4.7m wide base slab. The wall requires a 7m deep, 0.6m diameter pile for each counterfort to resist sliding shear and tension in a major earthquake. Counterforts are spaced at approximately 1.8m centres.

Refer to Figure 3 for a typical cross section and Figure 4 for a photograph of the typical wall construction.

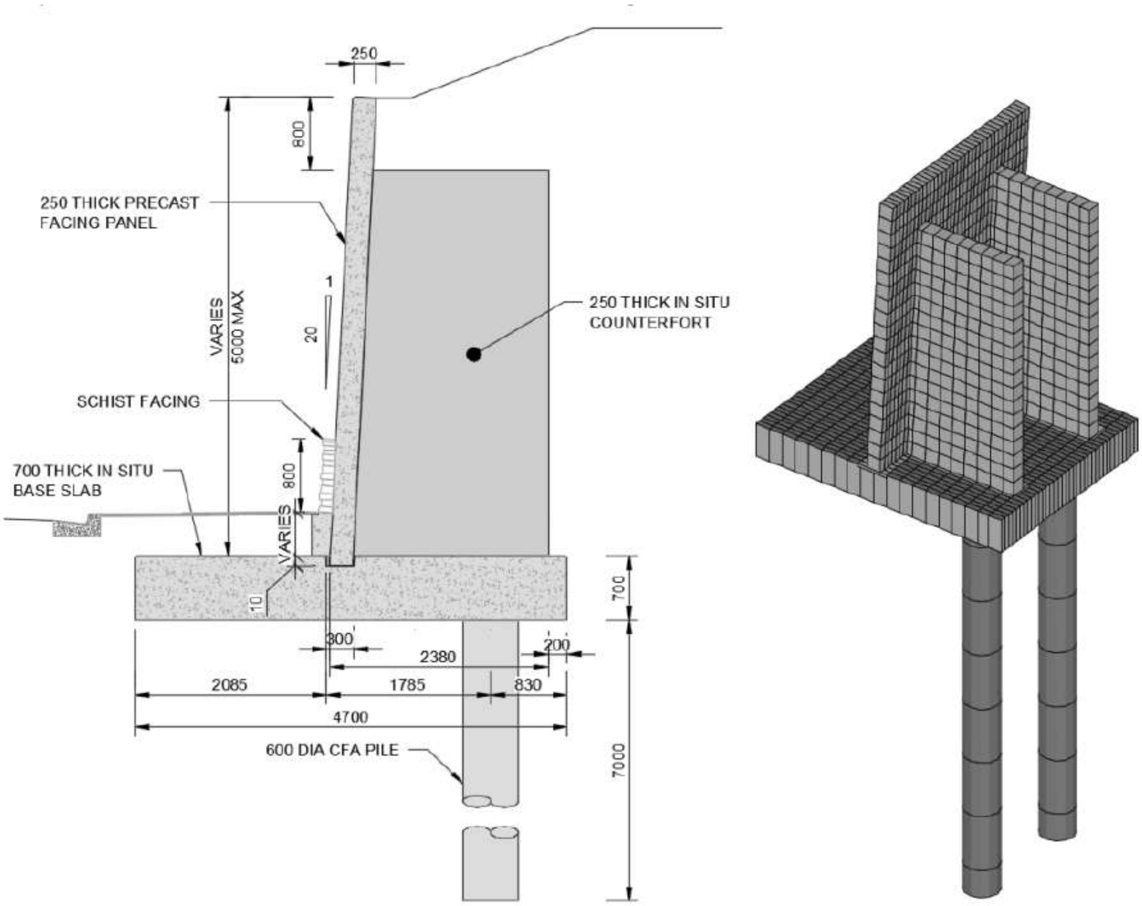


Figure 3. Typical cross section and 3D view of the counterfort wall (Type B).

To obtain structural demands and check for overall stability, the counterfort walls were modelled in Plaxis. The design seismic earth pressure was calculated assuming a “stiff” wall as per the RRU Bulletin 84 Volume 2.

- The pile was modelled with a full moment connection with the foundation slab. An ultimate skin friction of 60kPa and an assumed ultimate end bearing capacity of 725kN was assumed for the pile.
- The double T counterfort wall unit is modelled as a plate with a full moment connection with the foundation slab. The assumed EI was 3.548 x 106kN/m$^2$ per metre and the assumed EA was 11.80 x 106kN/m.
- The wall base slab was modelled as a plate.

The overall wall height is greater than 5m and the face area is greater than 100m$^2$. Furthermore, collapse of the wall would affect the Arterial Stage 1 route. Therefore, the wall was designed as an Importance Level 3 structure. The design life for the structure was considered to be 100 years.

The wall design called for 36 reinforced concrete piles, assumed to be constructed using continuous flight auger due to the ground conditions. The base slab and counterforts were also designed as reinforced concrete to be cast in-situ. The facing panel was designed as precast concrete for constructability reasons (to minimise site works and formwork required) and to

allow a higher surface finish to be achieved. The precast panels were fabricated in Christchurch and transported to Queenstown.

The facing panels feature a colour-stained sandblasted artwork designed by Aukaha’s Mana Ahurea team, manu whenua and local artists. The artwork pattern references the story of the local woman Hākitekura, who was the first person to swim across Lake Whakatipu, and features representations of prominent peaks around Queenstown: Walter Peak, Cecil Peak, Queenstown Hill and The Southern Alps. The artwork also includes statements describing local Māori occupation in the Queenstown area and features images of local flora and fauna. Set within the corner panel is a selection of pounamu (greenstone) tiles. A schist stone facing up to 0.8m height is provided along the bottom of the wall facing, which provides continuity with other schist stonework around Queenstown.

Figure 4. St Josephs Wall counterforts prior to backfill.

**WALL CONSTRUCTION**

Construction of the wall commenced in February 2023, beginning with vegetation removal, demolition of the existing walls and excavation down to the base slab level along Melbourne Street. During construction, some changes to the design were made and challenges overcome due to the conditions on site.

<u>Piling Methodology</u>

During design, it was thought that the in-situ ground may not support open bore piles and the piles were assumed to be constructed using a continuous flight auger methodology, with reinforcing cages plunged after concreting. However, it was discovered elsewhere on the Arterial Stage 1 site that the ground is generally stable with open bore piles (with the exception of an area at a lower elevation further along the alignment where a layer of grey, silty material

and groundwater resulted in unstable bores) and the Construction Team elected to use an open bore methodology. Aside from occasional overbreak when boulders were encountered by the auger, the pile bores were stable and all piles were constructed with an open bore, down to depths of 9.6m below ground level.

Construction Sequencing and Counterfort Alignment

Typical construction sequencing involves constructing elements with the smallest allowance for tolerance first, and elements with a higher allowance constructed afterwards to suit. However, the sequencing of the St Josephs Wall required the opposite of this, with the first elements to be constructed being the piles, followed by the reinforced concrete footing, then finally by constructing the precast panels to suit. Some variability in pile positions resulted in uneven counterfort spacings, however this wasn't incorporated into the precast panel construction. Therefore, misalignment of the starter reinforcing bars from the footing with the starters from the facing panels for the counterforts occurred, with up to 110mm misalignment for some panels. A specific detail was developed to accommodate the misaligned starters and connect everything structurally.

Beetham Street Tie-in

The existing retaining wall along Beetham Street continues approximately 45m beyond the Arterial Road. However, wall replacement could not continue indefinitely and the decision was made to undertake wall replacement for the extent of the existing stacked stone wall. The existing wall beyond this point is concrete, however the construction details are unknown. Furthermore, the wall appeared to be in poor condition with some large cracks and visible overturning of the wall. Construction of the counterfort wall as per the typical details up to the existing concrete wall would be difficult without destabilising the existing wall, due to the excavation depths required.

The typical design was modified in this area to slope the base slab up to meet the top of the wall at the tie-in, to minimise excavation depths. The excavated slope was stabilised with reinforced shotcrete shortly after excavation, so that the main in-situ base slab could be constructed (refer Figure 5). This allowed the counterfort wall to be constructed with minimal disturbance to the existing wall.

Figure 5. Shotcrete stabilisation of the Beetham Street tie-in.

Due to the sloped footing, the standard precast facing panel details were not able to be used. The decision was made to pour the concrete for this portion of the wall facing in-situ. Several additional requirements were introduced to ensure consistency with the adjacent precast facing panels.

The precast facing panels for the rest of the wall were constructed in Christchurch, including the sandblasted artwork pattern, and shipped to Queenstown. However, concrete for the in-situ facing would need to come from a local concrete plant. Several trials pours were undertaken with the local concrete mix to ensure a close colour match would be achieved.

Casting the precast facing panels on a smooth casting bed allowed the F5 surface finish requirement to be consistently achieved. However, replication of this with in-situ construction would be more difficult, especially with an inclined wall face at a slope of 1:20. The Construction Team spent a lot of time to ensure the formwork for the front face was smooth and would provide an F5 finish.

The 3.5m-tall in-situ facing for the wall tapers from 600mm thick at the base to 250mm thick at the top. To eliminate the risk of colour variations between concrete batches, and to minimise construction joints, the Construction Team elected to pour the entire facing in a single pour. Patched holes from formwork tie-bars were also not desirable for the finish, so these were avoided for the pour and tie-bars substituted for significant propping to each face (refer Figure 6). To aid concrete placement for the tapered wall, the wall was poured using a letterbox methodology with openings on the back face used for concrete placement, which were closed-up as the concrete level increased. Once the concrete had reached an acceptable strength to remove the formwork, the artwork pattern was continued with in-situ sandblasting.

Figure 6. Propping for Beetham St tie-in in-situ facing pour.

St Josephs Church Proximity

Due to the lowering of the road level with the new alignment, construction of the counterfort wall was required in front of the heritage listed St Josephs Church (refer Figure 7). To enable the excavations required to construct the wall base slab, a temporary driven H-pile wall was installed in front of the church prior to excavation. Excavation could then proceed in front of the temporary wall with minimal disturbance to the church. The church was constantly monitored during construction for movement and vibration and no damage to the church has been detected. After the counterfort wall was constructed and backfilled, the temporary wall was removed.

Figure 7. Piling rig and temporary retaining wall in front of the St Josephs Church, illustrating the proximity of the works to the heritage listed structure.

**CONCLUSION**

At the time of writing, construction of the new wall is largely complete, with only small portions at either end yet to be constructed. The wall is due for completion in July and the Arterial Stage 1 is expected to be open to the public by Christmas 2024.

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# 3D MODELLING REINFORCING STEEL – A NEW RESOURCE FOR SAFETY & QUALITY

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## SUMMARY

Detailing reinforcing steel transforms design visions into practical construction plans, ensuring structural integrity and safety. Traditional manual detailing, though effective, often encounters errors, inefficiencies, and on-site hazards. 3D modelling technology enhances precision, reduces on-site adjustments, and improves communication among stakeholders, fostering better project outcomes. Steel & Tube's adoption of 3D modelling tools like Tekla Structures demonstrates significant benefits, including increased safety, quality, and collaboration.

## INTRODUCTION

### Background

Detailing reinforcing steel is the art of taking the architect's and engineer's vision from design drawings and transforming it into a practical and precise plan for construction. This crucial step bridges the gap between concept and reality by interpreting intricate design specifications and translating them into detailed bar bending schedules and layout plans.

A reinforcing steel detailer acts as a master translator, deciphering complex engineering drawings to determine the exact placement, size, and shape of steel reinforcement bars needed to give strength and stability to concrete structures. They meticulously calculate the spacing between bars, specify bending dimensions, and map out the configuration of the reinforcement grid with utmost accuracy.

This process is vital because it ensures that the structural integrity of the building is maintained, adhering to safety standards and design requirements. By creating these detailed plans, reinforcing steel detailers play a pivotal role in laying the groundwork for successful construction projects, providing the blueprint that construction teams rely on to bring the designer's vision to life.

The reinforcing steel industry in New Zealand, much like elsewhere, has traditionally relied on manual detailing methods. These methods, while effective, have been fraught with challenges such as human error, wastage, archaic information sharing, and general inefficiencies. With advancements in technology, 3D modelling has emerged as an innovative tool that offers significant improvements over traditional practices.

### Challenges with Traditional Manual Detailing

Manual detailing involves creating detailed drawings and instructions for the bending and placement of reinforcing steel bars. This process is labour intensive and requires high

precision to avoid costly errors. Human error, the time involved, and unperceived clashes with other services or elements are common issues associated with manual detailing.

Overview of 3D Modelling Technology

3D modelling technology involves creating detailed digital representations of reinforcing steel arrangements. These models provide a precise, visual depiction of the steel framework within a construction project, enhancing accuracy, efficiency, and collaboration among stakeholders.

Aim of the Study

This paper aims to explore the health and safety benefits of integrating 3D modelling technology in reinforcing steel practices. It will also highlight the advantages of sharing these models with stakeholders in construction projects, focusing on the improvements in coordination, quality, and overall project outcomes.

## CURRENT PRACTICES IN REINFORCING STEEL DETAILING

Manual Detailing Methods

Detailing of reinforcing steel involves creating manual detail sheets and marking up structural steel drawings to indicate placement positions, physical prototyping, and frequent on-site adjustments. These processes are time-consuming and prone to inaccuracies. For instance, at Steel & Tube, manual detailing has often led to delays and increased costs due to the need for on-site corrections.

Common Issues and Limitations

Key issues with manual detailing include:

- Human Error: Mistakes in drawings can lead to misalignments, short-supplied items, wastage, and non-conformance of the design intent.
- Inefficiencies: The time required for manually detailing the requirements of complex projects can be cumbersome and adds cost to the process.
- On-site Hazards: On-site adjustments and not realizing the opportunity for off-site prefabrication pose additional safety risks to workers.

Steel & Tube have experienced several challenges with manual detailing. For example, in several projects, misaligned steel bars due to drawing errors resulted in significant rework and delays, highlighting the need for a more accurate and efficient detailing method.

## ADVANTAGES OF 3D MODELLING TECHNOLOGY

Enhanced Precision and Accuracy

3D modelling technology offers higher accuracy than manual drawings, reducing the likelihood of errors. Digital models provide a precise visual representation, ensuring that all components fit perfectly together, thereby minimizing rework.

Safety Improvements

The use of 3D models reduces the need for on-site adjustments, thereby minimizing worker exposure to hazardous conditions. Additionally, these models can facilitate safer construction practices by providing clear and accurate instructions and allowing opportunities for off-site prefabrication to be easily visualized.

Quality Control and Assurance

With 3D modelling, quality control is significantly enhanced. The precise nature of digital models ensures that the final construction meets all specifications and standards, leading to higher quality outcomes.

Real-time Updates and Adjustments

3D models can be easily updated and adjusted in real-time, allowing for quick responses to any changes or issues that arise during the construction process. This flexibility ensures that the project remains on track and meets all requirements.

**IMPLEMENTATION OF 3D MODELLING IN REINFORCING STEEL**

Steps Taken by Steel & Tube

Steel & Tube conducted extensive market research into various software packages capable of 3D modelling reinforcing steel and its ability to synergize with our key existing software packages. 5 Software options were considered.

| **Software** | **Supplier** | **NZ Reseller** | **Reason for inclusion** |
|---|---|---|---|
| Revit | Autodesk | AD2K | Ubiquitous software used by Clients and other parts of S&T |
| ProConcrete | Bentley / aSa | aSa | Strong interface with aSa (Manufacturing Software) |
| Tekla | Trimble | Building Point | Strong reputation for steel & concrete modelling |
| Adda | Adda | Adda | Start-up but with impressive project resume |
| AllPlan | Nemetschek | Rossi Concepts | European brand with strong Reo DNA being rolled out to Australasia |

Selection Criteria were chosen:

| Selection Criteria | | | |
|---|---|---|---|
| Features | Productivity | Export to aSa | 2D modelling |
| Support | User base | BIM Modelling | Import / export for customization |
| Model exchange | Ease of use | Stability in use | Configurable for S&T |
| Third party add-on | High level customization | Annual cost per seat amortised over 7 years | Future prospects |

<u>Software and Tools Used</u>

The primary software decided upon by Steel & Tube is industry-leading 3D modelling tool Tekla Structures. This tool offers comprehensive features for creating detailed and accurate models of reinforcing steel arrangements. The company adopted 3D modelling technology by first investing in the necessary software and tools. The company then trained its staff to effectively use the technology, ensuring a smooth transition from manual to digital detailing.

<u>Training and Technical Expertise Required</u>

Implementing 3D modelling technology required significant investment in training. Steel & Tube provided extensive training sessions for its detailers, ensuring they were proficient in using the new software and understanding its capabilities.

<u>Overcoming Initial Challenges</u>

The transition to 3D modelling was not without challenges. Initial issues included resistance to change from staff accustomed to manual detailing, and the need to integrate the new technology with existing workflows. However, with proper training and support, these challenges were successfully overcome.

**IMPACT ON STAKEHOLDER COLLABORATION**

<u>Improved Communication among Stakeholders</u>

3D modelling technology has significantly improved communication among stakeholders.

By providing a clear and accurate representation of the reinforcing steel arrangements, all parties involved can better understand the project requirements and provide input, leading to more effective collaboration.

Information sharing is cloud based and accessible with any modern device such as a computer, tablet or phone. QR codes are created and by simply scanning this QR code access to a viewable model is granted via Trimble Connect.

Case Studies Showcasing Successful Collaboration

In a recent large Hangar Project at the Auckland Airport, the use of 3D models facilitated seamless collaboration among the engineers and contractors. The shared models enabled real-time feedback and adjustments, ensuring that everyone was aligned and working towards the same goals.

Feedback from Project Partners

Feedback from project partners has been overwhelmingly positive. Stakeholders have noted the improved clarity and understanding provided by the 3D models, leading to smoother project execution and better overall results.

*“We engaged with Steel & Tube on a proof-of-concept digital model at 25–50% into the developed design. We’re finding this early involvement doesn’t change the amount of reinforcing; it simply raises any issues earlier and we can rework them digitally. We’re seeing cost efficiencies through innovation, efficient design and good building, and fewer or no variations on site.” - Jimmy Corric, Preconstruction and Innovation Manager, NZ Strong Construction*

**HEALTH AND SAFETY BENEFITS**

Reduction in On-site Accidents

The use of 3D models reduces the need for on-site adjustments, thereby minimizing worker exposure to hazardous conditions. This reduction in on-site work leads to fewer accidents and improved overall safety.

Off-site Prefabrication and Its Impact

3D modelling facilitates off-site prefabrication of steel components. By prefabricating components in a controlled environment, the risks associated with on-site construction are significantly reduced, leading to a safer working environment.

Off-site prefabrication gets the workers out of nature’s elements, as it could be done in a safer workshop or covered environment, therefore reducing time spent in the trenches on a construction site and the associated hazards.

Improved Worker Training and Safety Protocols

The visual nature of 3D models enhances training and communication among workers. Clear, visual instructions help workers understand their tasks better, leading to safer and more efficient execution.

Case Study showing improved Health & Safety outcomes

A government building project in Dunedin utilized BIM and 3D modelling technology to design and implement its reinforcing steel arrangements. The project aimed to enhance safety and efficiency through advanced technology.

- Safety Enhancements: The digital models allowed for detailed planning, reducing the need for on-site adjustments, and minimizing worker exposure to hazards.
- Quality Improvements: The precise models led to higher quality installations, with fewer errors and rework required.
- Collaboration: Real-time sharing of models facilitated better coordination among engineers and contractors, ensuring alignment and reducing conflicts.

**QUALITY AND EFFICIENCY GAINS**

Consistency and Standardization

Digital models ensure consistency and standardization in the design and implementation of reinforcing steel. This leads to higher quality construction and fewer defects.

Reduction in Errors and Rework

The precision offered by 3D models reduces the likelihood of errors, thereby minimizing the need for costly rework and ensuring that projects are completed on time and within budget.

Streamlined Review and Approval Processes

The ability to share and review models among all stakeholders streamlines the review process. Stakeholders can provide feedback and make necessary adjustments before construction begins, ensuring that the project meets all specifications and requirements.

**FUTURE PROSPECTS AND RECOMMENDATIONS**

Continued Adoption of 3D Modelling Technology

The success of 3D modelling technology in reinforcing steel practices underscores the importance of continued adoption and investment in technology. Companies like Steel & Tube should continue to explore innovative ways to leverage technology to improve safety, quality, and efficiency in construction projects.

Potential for Further Advancements

While 3D modelling technology has already brought significant improvements to the industry, there is still room for further advancements. Future research should focus on developing new tools and techniques to enhance the capabilities of 3D modelling, with a specific emphasis on improving safety and collaboration.

Recommendations for Industry-Wide Adoption

To encourage industry-wide adoption of 3D modelling technology, stakeholders should collaborate to develop standards and best practices for implementation. Additionally, government agencies and industry associations can play a role in promoting the benefits of technology and providing support to companies looking to adopt it.

**CONCLUSIONS**

The adoption of 3D modelling technology has had a transformative impact on safety and collaboration in reinforcing steel practices. By providing precise and accurate models, this technology enhances planning, reduces on-site work, reduces the overall construction programme, and improves overall construction quality. Additionally, shared 3D models foster collaboration and coordination among stakeholders, leading to better project outcomes. Companies like Steel & Tube have demonstrated the value of embracing technology and leveraging it to drive innovation and improve safety in the construction industry.

Through continued investment in technology and collaboration among industry stakeholders, the construction industry in New Zealand can build upon the success of 3D modelling and pave the way for a safer, more efficient future.

# PARAMETRIC ANALYSIS TO DETERMINE THE CONNECTION CAPACITY OF THE HEADED STUD JOINT CONNECTION IN STEEL-PRESTRESSED CONCRETE HYBRID GIRDER

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## SUMMARY

This study analytically investigates critical parameters of the joint segment in a hybrid girder, combining a steel girder within a prestressed concrete girder cross-section. Emphasizing a mechanical model for parametric analysis, the research addresses uneven shear force distribution along shear connections, a key challenge in designing joints for steel-concrete composite elements linked to prestressed concrete in its anchorage zone. Factors such as the number of shear connectors, length of the connection, stiffness of shear connectors, and influence of connecting elements on shear force distribution and slip displacement were examined, leading to a simplified formulation for configuring the anchorage zone in practical applications.

## INTRODUCTION

Researchers and practicing civil engineers seek novel design solutions to reduce construction time and optimize structural load-bearing capacity, aiming for cost savings. Steel-concrete hybrid structures, extensively detailed in the literature, optimize cross-sectional and longitudinal profiles for enhanced strength and serviceability. Innovative hybrid girders, by Shinozaki et al.(2014) followed by Haque et al.(2019), embed steel girder of a composite girder within a prestressed concrete girder section (herein after steel-PC hybrid girder) for bridge applications, considering longitudinal composition and required member strength, as illustrated in Figure 1(a). Shinozaki et al. (2014) proposed that the headed studs installed on the upper and lower flanges along with themselves counteract shear forces from bending (M) and axial loads (N). Meanwhile, the studs on the web ($P_{wvd}$/ $P_{whd}$) resist shear forces from the applied load (S) and prestress ($F_{ps}$) with additional shear connectors on the web preventing its rotation, as depicted in Figure 1(b).

The authors analyzed the shear behaviour of the hybrid girder, specifically studying the stud shear forces, as illustrated in Figure 1(c). Analysis indicates relatively lower, uneven stud shear forces in the web and flange, attributed to similar bending resistance, particularly with lower flange studs. While the design method of Shinozaki et al. (2014) suggests greater joint capacity, its verification remains elusive. Current research aims to identify critical parameters to analytically verify the experimental joint capacity. A steel-PC hybrid girder potentially varies load capacity along its length. Integrating two cross-section types within one element necessitates solving the problem of anchorage zone. Koziol et al.(2017) and Lorenc et al.(2022) provide key reference data for developing analytical and experimental methods,

addressing transition zone load-bearing capacity based on destructive tests. Following a comprehensive literature review, a parametric analysis with a mechanical model was conducted to establish an experimental method for verifying hybrid girder connection capacity, addressing crucial uneven force distribution in joint connections during steel-concrete composite to PC element transitions.

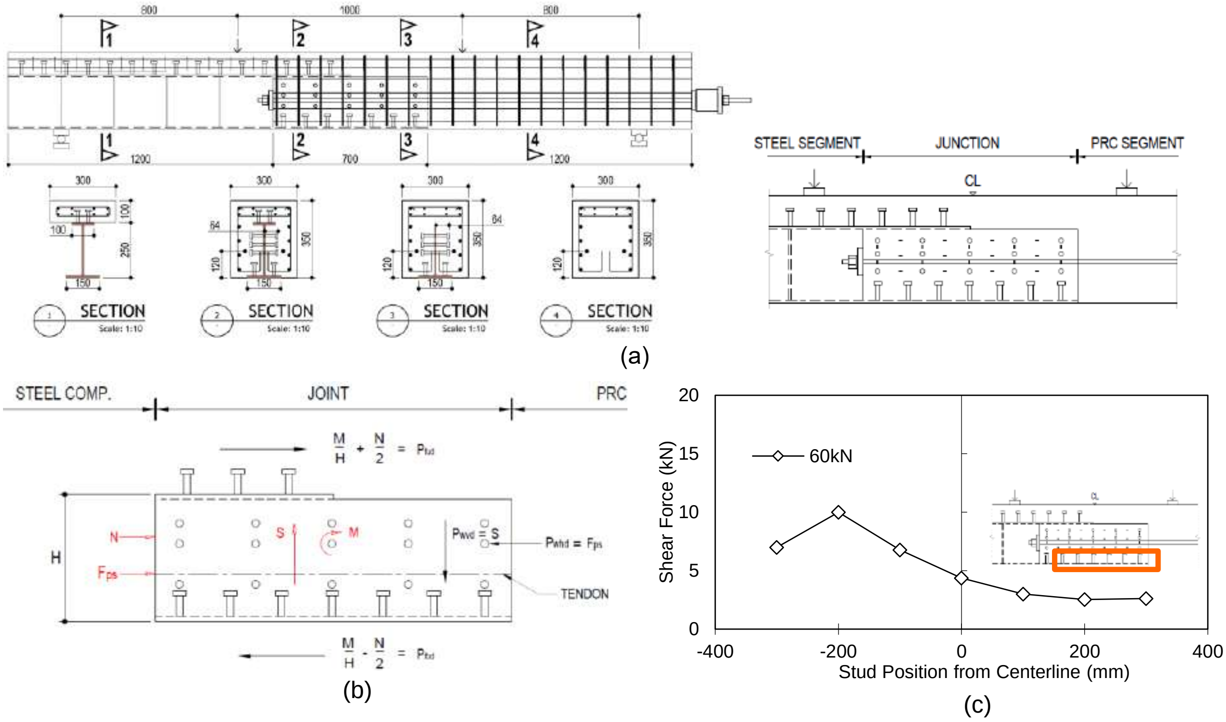


Figure 1. (a) Hybrid Girder Specimen Details Adopted by Haque et al. (b) Sectional Forces on the Joint and Conceptual Forces Acting on Studs (c) Shear Forces Generated in Stud Shear Connectors under Static Loading (60 kN Design Load).

## OVERVIEW OF THE ANALYTICAL MODEL

A parametric analysis was performed to interpret the push-out behaviour exhibited by the headed studs in the joint connection of the hybrid girder. A discrete system was adopted to assess the influence of various factors on force distribution, connecting a concrete plate and a steel plate using headed studs, as depicted in Figure 2. The steel elements were modelled with T-shaped cross-sections, their areas corresponding to the total resistance of the shear connectors. Boundary conditions allowed horizontal relative slip displacement of the steel plates, with non-slip support at the right end of the concrete plate. The mechanical model integrated a shear force-slip displacement relationship proposed by Shima et al.(2010), as detailed in equations (1), (2), (3), (4), and (5), into the headed stud. The Hognestad model describes the concrete behaviour, and the elasticity models represent the steel plates highlighted in equations (6) and (7), respectively.

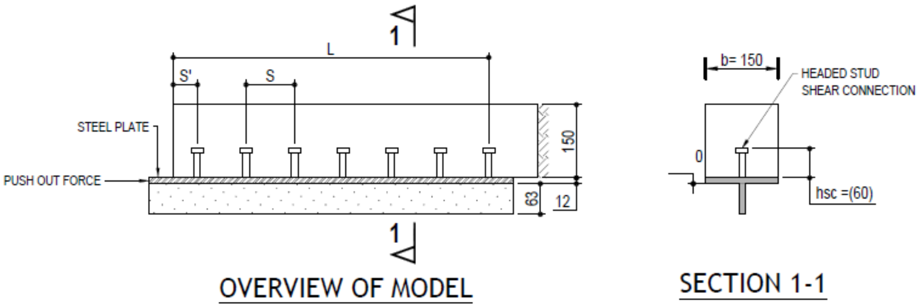


Figure 2. Mechanical Model for Shear Connectors (*S*: Stud Spacing, *Ś*: Distance to the First Stud, *L:* Length of the Connection)

$$V=V_{su}(1-e^{-\alpha\delta/\varphi})^{2/5} \tag{1}$$

$$\alpha=11.5[1.1(\gamma-1)^2+1]f_c^{'}/f_{c0}^{'} \tag{2}$$

$$\gamma=V_{su2}/V_{su1}: \tag{3}$$

$$V_{su1}=31A_{ss}\sqrt{\frac{h_{ss}}{d_{ss}}f_c^{'}}+10000 \quad (N) \tag{4}$$

$$V_{su2}=A_{ss}f_{su} \quad (N) \tag{5}$$

where, V: shear force (N), $V_{su}$: shear capacity (N)=min ($V_{su1}$,$V_{su2}$), δ: slip displacement (mm), ϕ: diameter of stud (mm), $f_c^{'}$: compressive strength of concrete (MPa), $f_{c0}^{'}=30\,(N/mm^2)$, $\gamma=V_{su2}/V_{su1}$: capacity ratio, $A_{ss}$: cross-sectional area of stud ($mm^2$), $d_{ss}$ : stud diameter, $h_{ss}$: stud height, $f_{su}$: tensile strength of stud (MPa)

$$\frac{\sigma_c}{f_c^{'}}=2\left(\frac{\epsilon}{\epsilon_0}\right)-\left(\frac{\epsilon}{\epsilon_0}\right)^2 \tag{6}$$

where $\sigma_c$ : peak stress in concrete, $\epsilon_0$ : strain at peak stress, $f_c^{'}$ : compressive strength of concrete, ϵ: strain in concrete.

$$\sigma_s=E_s\epsilon_s \tag{7}$$

where $\sigma_s$ : stress in steel, $\epsilon_s$ : strain in steel, $E_s$: modulus of elasticity of steel.

The mechanical model computes the shear force-slip displacement relationship and shear force distribution in shear connectors. When the loaded-end connector displaces, the model determines its load. Then, the slip displacement of the second connector is calculated from the difference between the loaded-end slip displacement and the relative slip and, subsequently, the load sustained by it. This process iterates to the free end, where the load equals the tensile force in the steel bed plate. Iteration continues for various loaded-end slip displacements until the total load at the end of the concrete plate equals the applied load and steel plate displacement becomes zero. The maximum load obtained is the capacity of shear connectors. A multi-parametric analysis subjected the model to loading in five steps, applying force equal to the shear connector count times its strength, to examine the impact of the number of shear connectors, length of the connection, stiffness of the shear connectors, and

stiffness of steel elements influence on force redistribution. Analysis cases are detailed in Table 1.

Table 1. Detailed Analysis Cases

| Analysis Case | n | $d_{ss}$ (mm) | $h_{ss}$ (mm) | S (mm) | $t_s$ (mm) | $L_c$ (mm) |
|---|---|---|---|---|---|---|
| AN-7a | 7 | 13 | 60 | 50 | 12 | 500 |
| AN-7b | 7 | 13 | 60 | 100 | 12 | 800 |
| AN-7c | 7 | 10 | 60 | 100 | 12 | 800 |
| AN-7d | 7 | 16 | 60 | 100 | 12 | 800 |
| AN-7e | 7 | 13 | 60 | 100 | 32 | 800 |
| AN-7f | 7 | 13 | 60 | 100 | 50 | 800 |
| AN-14 | 14 | 13 | 60 | 50 | 12 | 800 |

where n: number of studs, $d_{ss}$ : stud diameter, $h_{ss}$: stud height, S: stud spacing, $t_s$: steel plate thickness, $L_c$: length of the connection.

## ANALYSIS RESULTS AND DISCUSSION

Distribution of Shear Force along the Shear Connection

In the mechanical model, an equal compressive force is exerted on the steel plate upon load application, distributing this force to the headed studs along the connection. Figure 3 depicts the load distribution among headed studs at different loading steps for analysis case AN-7b. At the low loading steps, headed studs near the loaded and free ends bear a higher load, gradually diminishing towards the middle of the connection. This variation stems from differences in steel and concrete plate contraction between shear connectors, with more significant effects at the extreme parts of the connection, resulting in higher stress on edge connectors than in the middle. As the applied load increases to its maximum, the shear force transmitted by intermediate-headed studs gradually increases. At the maximum load, the shear force among all connectors becomes nearly equal. This phenomenon occurs because the stiffness assumed in the shear force-slip displacement relationship for headed studs decreases as shear force increases. Consequently, the increment of shear force transmitted at extreme connectors, where shear force is higher, decreases(Shima and Watanabe 2010). Conversely, the shear force transmitted by headed studs in the middle region increases accordingly.

Parametric Analysis of Critical Parameters on Shear Force Distribution

Number of Shear Connectors/ Length of the Connection

The load-slip relationship with shear connectors spaced at 50 mm (AN-14) compared to 100 mm (AN-7b) being kept a constant length, is shown in Figure 4. By spacing them out, reducing the number of shear connectors enhances its capacity. Strain variation along its length results from differential contractions between steel and concrete plates, more significant at the connection extremes. Edge connectors thus experience greater compression than the middle ones. 7 studs spaced 100 mm (AN-7b) and 7 studs at 50 mm intervals (AN-7a) result in varying connection lengths, with greater spacing leading to increased force discrepancies, as in Figure 4.

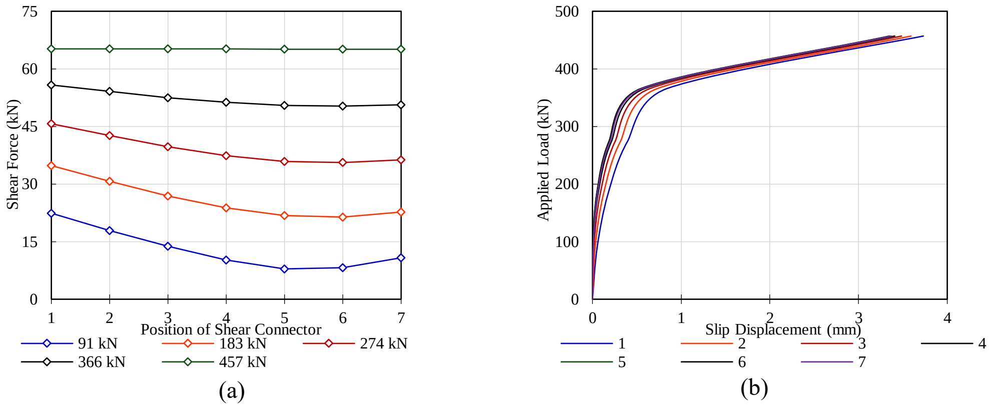


Figure 3. (a) Shear Force Distribution (b) Load-Slip Relationship

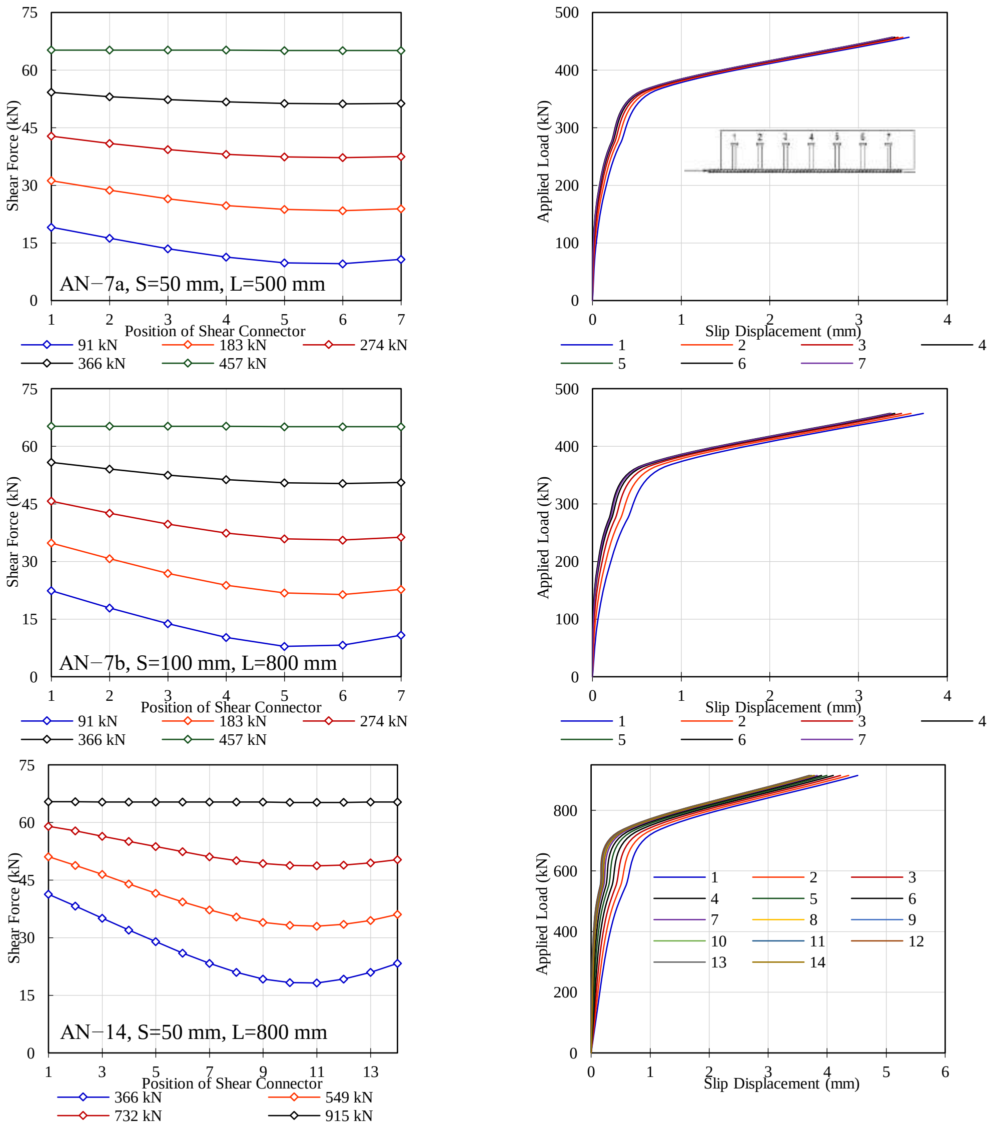


Figure 4. Shear Force Distribution and Its Load-Slip Relationships with a Different Number of Connectors/Lengths of the Connection.

## Stiffness of the Shear Connectors

Authors examined various cases based on the actual stiffness of shear connectors, expressed in terms of headed stud diameter: $d_1$ = 10 mm (AN-7c), $d_2$ = 13 mm (AN-7b), $d_3$ = 16 mm (AN-7d), as shown in Figure 5(Zheng et al. 2016). Varying connector stiffness leads to different forces within connectors. Connectors with lower stiffness are preferable to equalize forces, but a potential increase in joint slip occurs. Figure 5 illustrates the influence of headed stud diameter variation on the load-slip displacement relationship, with thinner connectors exhibiting more significant displacement under identical loads than thicker, stiffer connectors up to the maximum load.

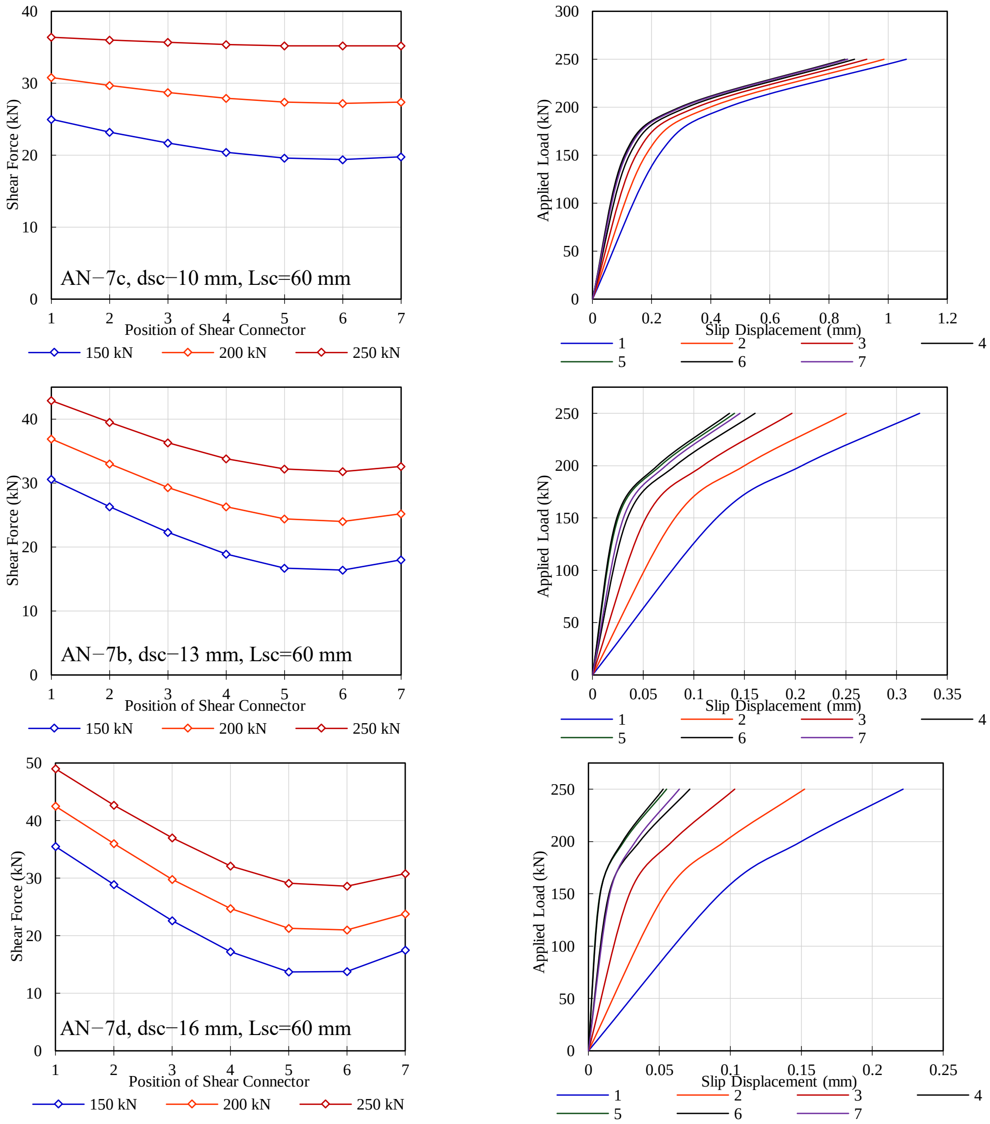


Figure 5. Effect of Shear Connector Rigidity on Shear Force Distribution

## Stiffness of the Steel Elements

Steel plate stiffness impact on force distribution was analyzed by varying steel plate thickness leading to cross-sectional areas. Low-stiffness plates show uneven distribution with significant concentration at extreme connectors even at low forces. Conversely, high-stiffness plates distribute forces evenly. Infinitely stiff plates ensure equal forces in all connectors at every load level. Increasing the steel plate thickness enhances its rigidity, affecting the interaction with the pushout force between shear connectors. Thicker plates create a composite member with concrete, improving overall load-bearing capacity(Dobashi et al. 2011). However, pushout force remains consistent regardless of plate thickness. Thicker plates ensure uniform transfer capacity among connectors. Analysis shows distinct load-slip displacement relationships for 12 mm (AN-7b) and 32 mm (AN-7e) plates, but the minimal difference between 32 mm (AN-7e) and 50 mm (AN-7f) is illustrated in Figure 6. Around 32mm thickness, plate rigidity adequately disregards its impact on transfer force.

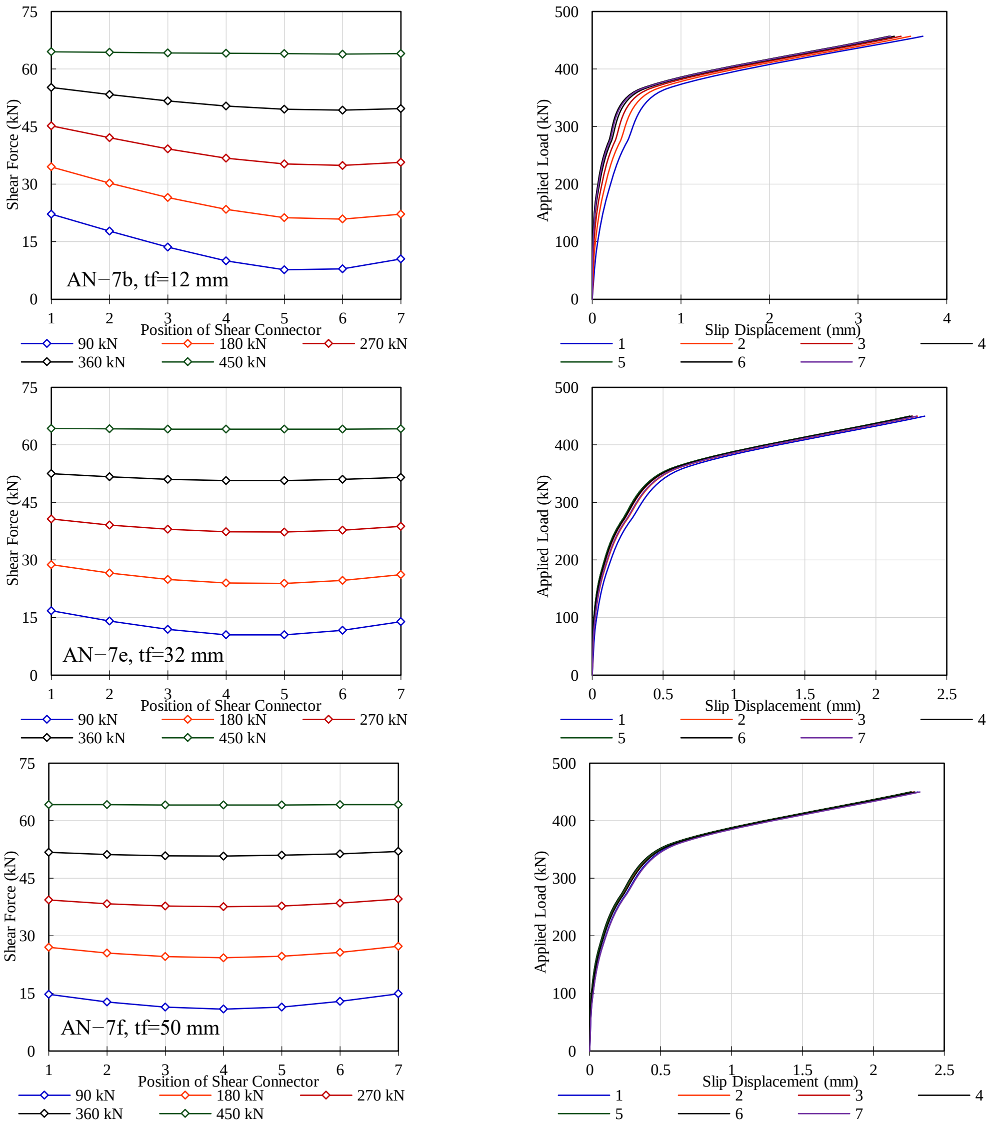


Figure 6. Comparison of Shear Forces in Load Steps; Steel Plate Rigidity in Thickness; 12 mm, 32 mm, and 50 mm.

Formulation of a Simplified Model to Configure the Anchorage Zone

To establish a simple equation for anchorage zone configuration, ensuring uniform slip displacement among shear connectors is essential for equalizing shear forces in the headed stud of the connection. In the mechanical model, zero relative slip between steel and concrete plates results in equal slip displacement for each shear connector. Contractions of steel and concrete plates under push-out loading, presented in equations (8) and (9) respectively, yield the relative slip, informing the derivation of equation (10) for equalizing transfer shear force.

$$\delta_c=\epsilon_0\left(1-\sqrt{1-\frac{\sigma_c}{f_c'}}\right)L_c \tag{8}$$

$$\delta_s=\frac{\left(Q-i\frac{Q}{n}\right)}{A_sE_s}L_s\ ;\ i=[1\sim(n-1)] \tag{9}$$

$$A_s=\left(\frac{n-i}{n}\right)\frac{1}{E_s}\frac{Q}{\epsilon_0\left(1-\sqrt{1-\frac{iQ}{nBHf_c'}}\right)} \tag{10}$$

where $\epsilon_0, \sigma_c, f_c'$ and $E_s$ are defined as those of equations (2) and (3). $\delta_c$ : concrete contraction, $\delta_s$ : steel contraction, Q: applied load, $A_s$ : steel area, n: number of shear connectors, i: stud shear position, B: width of the concrete plate, H: height of the concrete plate, $L_c$: spacing of the concrete plate, $L_s$: spacing of the steel plate.

This formula computes the steel plate area needed for even shear force distribution across each stud, irrespective of stud spacing. More connectors or applied force increases the required area. Using the simplified formula, Figure 7 illustrates how the shear connector position correlates with the required steel area. In practical application, this method estimates the web height and flange width of the steel H-section for anchorage zone configuration. The steel section dimensions may ideally follow a parabolic change, as per option (c) in Figure 8, however, practical constraints allow for linear or bi-linear adjustments ensuring nearly uniform shear transfer as emphasized in option (a) and option (b) in Figure 8. In practical application, Shinozaki's design approach enables the designer to determine the required number of headed studs based on both the maximum applied load and the shear capacity of the studs(Shinozaki et al. 2014). Stud geometry and shear strength are then chosen accordingly. Subsequently, spacing and connection length are to be estimated. However, the simplified formula confirms uniform shear force distribution, regardless of stud spacing. Longer connections intensify force discrepancies, favouring shorter lengths. Yet, shorter connections, while ensuring uniform shear stress, endanger local failure due to inadequate lever arm, inducing sudden web plate rotation(Shinozaki et al. 2014). Considering variable stiffness along the connection length, the derived correlation overcomes the limitations of stud spacing, enabling moderate connection length, and spacing alignment following Shinozaki's approach(Shinozaki et al. 2014).

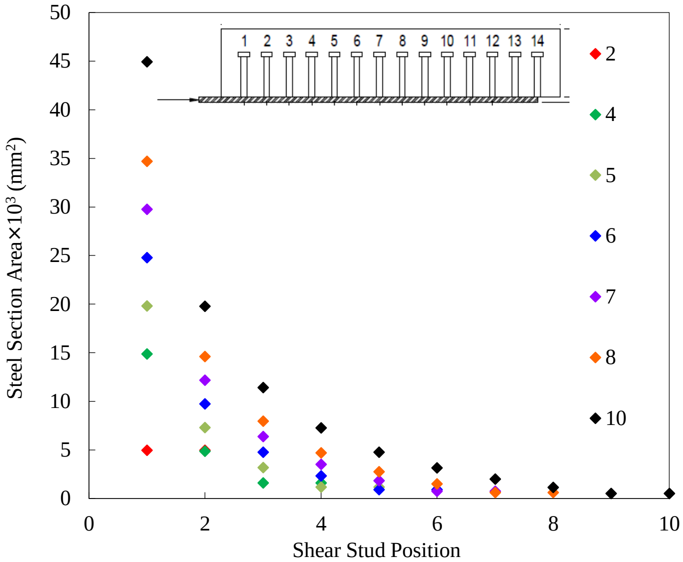


Figure 7. Steel Section Area per Stud Position

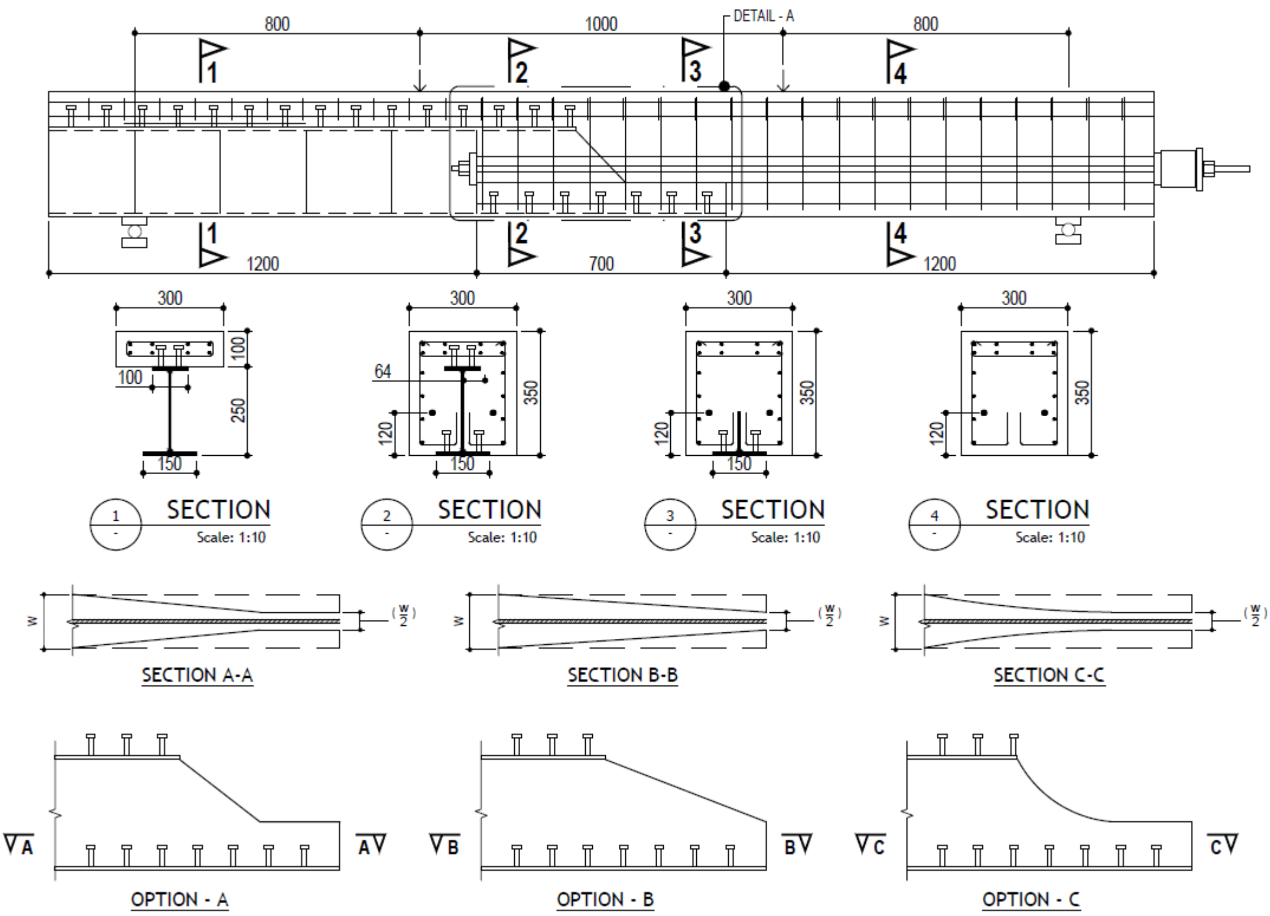


Figure 8. Proposed Configurations for the Transition Zone

## CONCLUSIONS

Unequal force distribution in the joint affects the structural steel to PC concrete connection in the anchorage zone. Analysis reveals factors influencing force redistribution, showing equalization through connector plasticization governed by its stiffness, and steel element stiffness. The developed formulation ensures uniform shear force distribution independent of stud spacing. The shear connector position correlates with the required steel area, configuring variable stiffness along the connection length. Ideally, steel section dimensions may vary parabolically, but linear or bi-linear adjustments suffice for uniform shear transfer within practical limits.

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BUILDING RESILIENCE
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# BUILDING RESILIENCE WITH CONCRETE

*(HOW LOCAL AUTHORITIES CAN USE CONCRETE TO BUILD STRONGER AND SAFER COMMUNITIES)*

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[1]Concrete New Zealand Inc. (CNZ)

## SUMMARY

Comprehensive planning and rigorous construction standards are the foundations of resilient communities. By ensuring that buildings and infrastructure are designed for an extended service life, community wellbeing can be chieved through durable, high-performance structures.

At the heart of this approach is concrete, providing unmatched durability, strength, fire resistance, sound insulation, passive heating and cooling, security, and overall resilience.

Concrete is essential for creating communities that can withstand the challenges of climate change. Adopting stricter requirements that incorporate low carbon concrete and cement in planning and development strategies is vital for our future.

Along with its member companies, Concrete NZ is eager to collaborate with local authorities to identify the best solutions for protecting New Zealand communities. Let's build a resilient future together!

## INTRODUCTION

The 'Building Resilience with Concrete' Project is an endeavour by the Concrete Ready-Mix Sector Group. Local governing bodies have the capability to foster robust and secure communities using concrete. The goal of this project is to enlighten these bodies about the possibilities that concrete offers to help them attain optimal results.

Local governing bodies need to comprehend that concrete is not only sturdy and long-lasting but also a material with low carbon footprint, with its emissions continually decreasing. By the year 2050, the concrete industry in New Zealand is set to produce concrete with net-zero carbon emissions. But as climate change progresses, we anticipate more severe weather conditions, including floods, hurricanes, and earthquakes, and we must also adapt.

Councils have a choice of materials but are often not fully aware of the potentials and in-service properties of concrete. Concrete is a material which helps to adapt and which can ensure the safety of our communities.

In New Zealand, there are 78 local authorities that can utilize concrete to construct more resilient, safer communities. They can also use concrete to erect energy-efficient, sustainable buildings that contribute to our well-being.
The 78 local authorities consist of:

- 11 Regional Councils
- 12 City Councils
- 54 District Councils
- 1 Auckland Council (which was formed by merging 8 councils in 2010)

These authorities are responsible for various areas such as sustainable well-being, river management, transportation, and construction of infrastructure and of housing. Over the past year Concrete NZ had the chance to engage with most of these authorities through a series of webinars and a paper given at the annual conference of the Building Officials Institute of New Zealand (BOINZ).

This paper introduces the relevant communication material used at meetings and webinars with authorities and throws a spotlight on some of the applications where concrete is undoubtably the most suitable material.

## RESILIENCE - THE ANSWER TO MANY CHALLENGES

Resilience is the ability to withstand, recover and adapt to changing conditions and challenges. In terms of the built environment, this includes climate change, natural disasters, social unrest and economic upheaval. Resilience is essential for society, as it ensures continuity and quality of life for people and communities in the face of uncertainty and risk. Resilience is not only a matter of survival, but also of opportunity and innovation. One of the key factors that contributes to resilient communities is the choice of materials for infrastructure and buildings. The materials used to construct the built environment have a significant impact on its performance. Amongst these materials, concrete stands out as a preferred option for resilience. Manufactured from cement and water, as well as fine and coarse aggregate, concrete embodies resilience, offering strength, durability, fire resistance, sound attenuation and thermal mass. Did you know that concrete is also a natural carbon sink!

The following pages outline concrete's resilient properties and offer local authorities recommendations around how concrete can be used effectively and efficiently to achieve resilience. Examples of concrete structures that have demonstrated resilience in different scenarios are provided to further demonstrate that the world's most widely used construction material will continue to provide for stronger and safer communities.

## CONCRETE'S ROLE IN RESILIENT DEVELOPMENT

The built environment must be resilient to withstand the impact of climate change. Concrete can help protect society through its properties that include:

**STRENGTH:** Concrete is a strong material that supports high loads and stresses without collapsing or failing. Concrete structures can endure floods, landslides, hurricanes, earthquakes and other natural disasters, without significant damage or loss of function.

**DURABILITY:** Concrete's durability and strength mean it ensures the resiliency and adaptive capacity of communities to climate change. Concrete structures can cope with harsh environmental conditions, such as high or low temperatures, humidity, salinity, acidity, chemical attack and pollution, without requiring frequent maintenance or repair.

**FIRE RESISTANCE:** Concrete is inherently non-combustible, which gives the occupants of concrete structures peace of mind that they, along with their family and possessions, are protected during fire events.

**SOUND ATTENUATION:** Noise from vehicles, building work and the urban environment in general can have a negative impact on quality of life. As a high mass material concrete cancels out disruptive and unpleasant exterior noise.

**PASSIVE COOLING:** Thermal mass is a material property that can be used to mitigate impacts of heatwaves, which are predicted to become more common and more extreme. Concrete is a high-mass material that can absorb heat during the day to moderate the internal temperature, and then slowly release that heat as the external temperature drops in the evening. This way thermal mass can be utilised by designers to increase comfort, and in extreme cases, save lives. Concrete's thermal mass can also reduce demand on space conditioning and in turn energy consumption.

**SECURITY:** Concrete is a secure material that protects people and property from external threats, such as fire, vandalism, theft, or explosions. Concrete has a high fire resistance rating, meaning it can prevent or slow down the spread of fire, reducing the risk of injury or death. Concrete can also withstand high impact forces, such as blasts or projectiles, preventing or minimizing penetration of, or damage to, the structure.

**NATURAL CARBON SINK:** Concrete is at the heart of efforts to improve the sustainability and resilience of the urban environment. For example, exposed concrete surfaces can absorb CO2 from the atmosphere to reduce the urban heat island effect.

**DISASTER RESILIENCE:** Concrete protects against natural disasters such as flooding. Pervious concrete pavements and permeable concrete pavers are being used to address peak rain events and surface flooding. They enable surface water to infiltrate directly into the ground or into attenuated drainage systems. Using low damage seismic technologies, concrete structures can be designed to withstand earthquakes, as shown by Wellington's Te Papa and Christchurch Women's Hospital. Across the country sea walls protect communities from rising sea levels and extreme swell events. For example, the small West Coast community of Granity now sleeps easy following the installation of a new interlocking concrete block seawall.

There's no doubt that the built environment must be resilient to endure the effects of climate change. Concrete can safeguard communities through a set of inherent properties that elevate it above other construction materials.

## RECOMMENDATIONS FOR LOCAL AUTHORITIES ON HOW TO USE CONCRETE FOR RESILIENCE

It is recommended that local authorities conduct thorough scoping of new developments to understand how resilience and adaptation to climate change can be achieved to ensure residents and assets are protected.
Suitable solutions to upgrade, refurbish or replace community infrastructure are informed by performance requirements with regard to severe weather events, such as storms, flooding, heat waves and earthquakes.
On a 2050 path to decarbonise, concrete is a proven material that can resist the impact of climate change through a range of properties based on unrivalled durability. These properties make concrete the material of choice material for building resilient infrastructure and protecting communities.

# AVOIDING PROBLEMS IN CONCRETE CONSTRUCTION

ALEX GRAY

Impact Project Management, Wellington

## SUMMARY

Many current construction staff do not have a wide knowledge of potential problems and defects in concrete construction. This paper will cover potential problems and defects and how to avoid them.

## INTRODUCTION

There is an old phrase in concrete construction- “you get one chance to place concrete correctly”. Here are a few steps which need to be considered for most concrete pours.

## MIX DESIGN

Probably the most important items here are the 28-day concrete strength, the aggregate size and whether you are pumping the mix. Bear in mind that if you are pouring in winter, you may need a higher early strength. There are numerous additives now available to add to concrete mixes. These range from accelerators, retarders, superplasticisers, and fly ash. All of these additives are useful at the appropriate time. If you are uncertain this subject should be discussed with your concrete supplier. Bear in mind that superplasticisers, retarders and accelerators have limited life in the mix, and you need to allow for transport time and any traffic delays in your planning. If you are placing self-compacting concrete, ensure that all the mix is on site before starting. If one truck is delayed in traffic the mix may solidify!

## COVER TO REINFORCING STEEL

Ensuring the minimum specified cover is critical for durability of concrete structures. Cover becomes even more critical in aggressive environments e.g. marine exposure zones. Checking for cover is often overlooked and can have serious consequences. Also, when placing concrete slabs, the cover is often compromised by the weight of the placing crew walking on the mesh or reinforcing steel. Make sure you have plenty of form spacers to avoid this problem.
When using an enclosed form make sure you have pockets at the base to check cover and also to clean out any debris.

Column with less than 30mm minimum cover in marine environment


Beam Column Joint with almost zero cover when 30mm minimum specified.

**PLACING TECHNIQUES**

Checking the weather forecast before pouring is obvious but commonly overlooked!
Before accepting the concrete it's important to check that the concrete slump is as ordered or within the NZS tolerance of +- 25mm. Most projects no longer carry out slump tests on site, yet the consequences can be serious. A mix that is too wet can result in lower strength and a greater risk of cracking.
Any formwork needs to be adequate to hold the weight of wet concrete. There have been many failures of formwork some of which have resulted in injuries to workers. Also, some formwork limits the rate at which concrete can be placed e.g. in a wall form. Ensure you do know this rate and do not exceed it! How the concrete is placed will be determined by the placers. Unless self-compacting concrete is used vibration is used to ensure the concrete fills the formwork. The surface finish will be specified by the client or designer. The placer needs to plan how to achieve this finish. Often, slab concrete pours start early in the morning so that the concrete has developed an initial set. This allows the finishing machinery to then smooth the concrete to the required finish.

**FORM RELEASE AGENTS**

These can make a major difference to the final finished surface. There are a large number of different release agents so seek advice and plan before pouring. Be very careful if you are pouring an architectural finish with ribs as its difficult to remove forms without damaging the concrete. On one large wall pour the Contractor used flattened fire hoses and water pressure to gently and slowly release the form.

**CURING OF CONCRETE**

Adequate curing is often overlooked and results in early and unnecessary cracking of the concrete. There are different methods of curing including water spray (with or without damp hessian), chemical spray or insulation. Insulation is often used on mass concrete pours to prevent water evaporation and also keeps the concrete at a constant temperature.
The curing method should be planned and agreed well before the concrete pour. Curing is particularly important for thin topping slabs which can crack within hours during warm or hot weather.
If curing is not carried out correctly the end result in cracked concrete. It may be possible to epoxy inject cracks but surely the better long-term solution is proper curing.

**FILLING POST-TENSIONING DUCTS WITH GROUT**

There have been several bridges overseas where the post-tensioned cables corroded due to the ducts not having been filled properly. The worst cases are those countries that use de-icing salts in winter as salt is particularly corrosive on high carbon steels. Probably the best example is the Westway viaduct in London UK. This was designed with heating in the deck to prevent ice formation. However, the power bill was excessive, so de-icing salts was used instead.

The 4km long viaduct had to be urgently closed and external post-tensioning installed at a huge cost. The solution is to calculate how much grout is needed to fill post-tensioning ducts and ensure this volume is used.

**AVOIDING COLLAPSE DURING CONSTRUCTION**

Many structures have unfortunately collapsed during construction. As my experience has been with bridge construction I will focus on that aspect.
The 300 metre Injaka launch bridge in South Africa collapsed in 1998 during launching with the loss of 14 lives. The primary causes were lack of competent personnel and inadequate supervision. The design was not peer reviewed and the designer had only 3 years post graduate experience. She died in the collapse. The first cracks in the bridge deck occurred 1 month before the collapse. The site engineer advised the design office (which was 4 hours drive away) and a senior engineer did some calculations and without a site visit instructed the launch to proceed. The key fault was that the launching bearings were positioned under the bottom slab rather than the stronger web of the bridge. They punched through the slab doubling the span and the structure collapsed.

Injaka Bridge South Africa after collapsing during launching.

I compared this collapse with my experience on the Ngauranga launched bridges from 1980-1984. These were designed by a senior engineer at the Ministry of Works and Development. As these were the first incrementally launched bridges in Australasia the design was checked by Leonhardt and Andra in the then West Germany. They were the developers of the push bridge construction method. On site we had a large team supervising the construction which included checking the casting bed which had a tolerance of 0.5mm on the corners. Every delivery of concrete was slump tested and cover was also checked before pouring.

Despite all the checks one pier on the northbound bridge cracked during launching. We stopped launching and the designer visited the site and subsequently designed substantial strengthening. The project was delayed 3 months while the northbound piers were strengthened. At that stage the southbound piers had not been built so the hammerhead was redesigned.

This project had a high level of supervision as the southbound bridge was launched across the SH2 Motorway which remained open during launching.

Southbound flyover bridge being pushed over the Hutt Road.
PHOTO SUPPLIED BY MAINZEAL CONSTRUCTION LTD

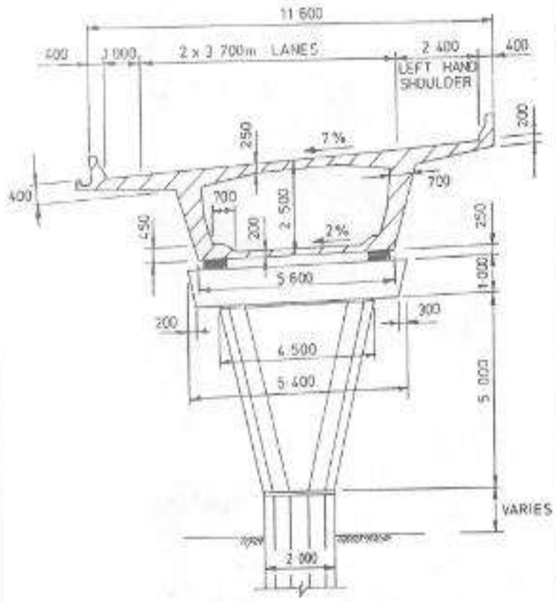



FIGURE 3 CROSS SECTION & PIER

NOTE BOX GIRDER SECTION IS CONSTANT OVER FULL LENGTH EXCEPT FOR LEADING 38m OF NORTHBOUND BRIDGE

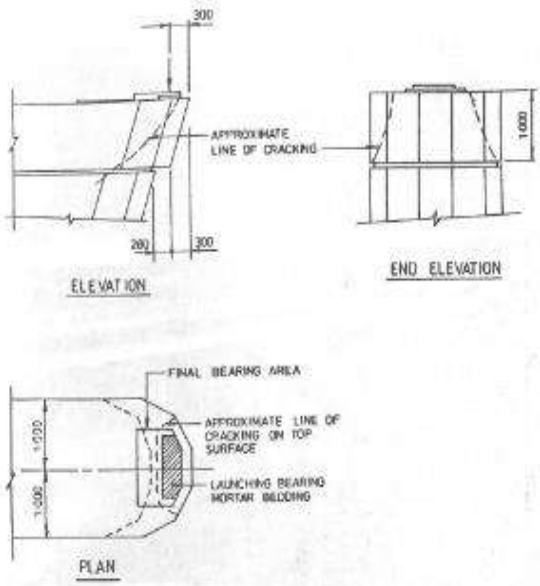



FIGURE 4 LOCATION OF CRACKING IN PIER CAP OF NORTHBOUND BRIDGE

**STRENGTHENING PIER CAPS — NGAURANGA OVERBRIDGE**

Small cracks appeared in the cap of pier 3 during a launch of the bridge. The external prestressing shown in the photos encircles the pier cap and bears against machined plates epoxy glued with Epirez 133 to the existing concrete. The prestressing wires were tensioned by two 600 ml flat jacks positioned at each end of the pier and inflated with Epirez 8859 to a pressure of 11.5 MPa.

*Photos by courtesy N.Z. Engineering*

Pier Strengthening at Ngauranga Flyovers following Pier Cracking during Launching

By comparison the Miami Bridge Collapse in 2018 is a lesson in how not to build a safe bridge. The Florida International University Pedestrian Bridge collapse occurred 5 days after erection with 8 lives lost and 10 injuries. The bridge was 53 metres long and weighed 950 tonnes. Unusually it was a post tensioned concrete truss bridge built under a design-build contract. The bridge spanned 8 lanes of traffic and was built off site and erected during an overnight lane closure. During erection the north end cracked after some temporary prestressing was released. Two days before the collapse the designers proposed remedial work but did not consider it necessary to close the road or prop the bridge.

At the time of the collapse the designers Figg Bridge Engineers were meeting the Florida Department of Transport and assured them the structural integrity of the bridge was not compromised and there were no safety concerns.

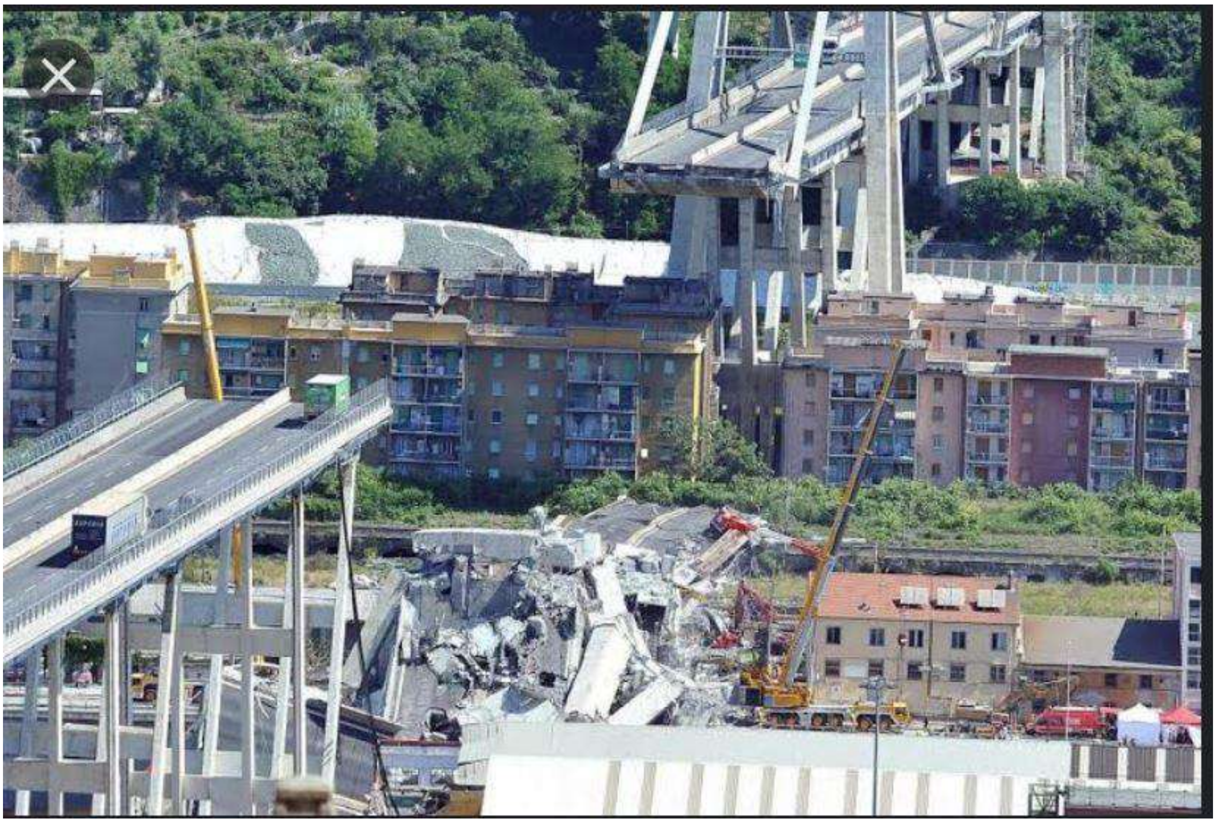
The subsequent inquiry the bridge had design deficiencies and the peer review was inadequate. Also, the severe cracking was wrongly ignored by the Engineer of Record.

The Contractor went through a Chapter 11 bankruptcy and subsequently reached a settlement with the victims and their families which totalled US$42 Million.

The replacement bridge is a cable stayed steel box girder bridge designed by the Florida Department of Transport. The new bridge budget is US$14.6 Million.

**AVOIDING COLLAPSE DUE TO INADEQUATE MAINTENANCE.**

The Morandi Bridge Collapse during a storm in Genoa Italy in 2018 resulted in the loss of 43 lives.
The primary causes were the lack of maintenance and the lack of redundancy in the structure.



The structure was considered innovative when it opened in 1967. It had a 210-metre main span and was 40 metres up in the air. At the time of the collapse, it was carrying 70,000 vehicles per day—far more than it was originally designed for.
The bridge was situated close to the sea and there were also factories close by emitting fumes which were corrosive to the prestressing wires encased in concrete.
The spans were supported by steel cables which were protected by prestressed concrete shells poured in-situ. However, the concrete was only prestressed to 10MPa making it susceptible to cracks, water intrusion and corrosion of the internal steel.
The premature corrosion of the stays on this bridge were known as far as the 1990's. Some repairs had been completed but not on the span which collapsed. Deck cracks were noticed on this span 2 weeks before the collapse.
The bridge was structurally redundant which is uncommon in modern bridges. All the tendons were contained in one large single composite tendon. Nowadays, Cable-stayed bridges have multiple single cables which can be inspected and replaced if necessary, without compromising the structure as a whole.
Finally in an unusual twist the firm collecting the tolls for this bridge and motorway were also responsible for the maintenance of the bridge. This firm was linked to the Benetton company in Italy who are well known as clothing manufacturers.

Following the collapse the rest of the structure was demolished and a new steel box girder bridge was built about 1 year later. The Italian government is now responsible for maintenance of the new bridge—lesson learnt!

**CONCLUSION**

If you are uncertain on how to place concrete in a particular situation my advice is to seek advice of others who have more experience.
There are numerous problems that can be overcome with planning and forethought.
The more complex the pour the more people and prior planning should take place before the pour.
Large pours are usually carefully planned—it is often the smaller less significant pours that have on occasions had significant problems.