

Precast Concrete Construction in a High Seismic Zone: Te Puna Hapori Whanganui Courthouse

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ABSTRACT

The Te Puna Hapori Whanganui Courthouse is a three-storey reinforced concrete shear wall building with a raft foundation which is currently in construction with a completion date of March 2027. It has been designed for the updated National Seismic Hazard Model, with an uplift in design loads of about 60% compared to NZS 1170.5:2004. The site is geotechnically challenging with soft, liquefiable soils that were preloaded for three months prior to construction. Embodied carbon targets were included in the project specification and concrete mix designs were reviewed for compliance.

WSP and the ECI partner Naylor Love Construction collaborated to test lateral load-resisting concepts and proceeded with a precast concrete shear wall solution. Other options tested were in-situ shear walls and steel EBFs. The precast construction has saved time on site and provided enhanced accuracy/ease of inspection in comparison to in-situ construction.

Engineers from WSP and Naylor Love worked together to detail the precast concrete shear walls which had some challenging interfaces with the raft foundation and superstructure. The panels were designed to be ductile which ruled out common precast connections such as grout tubes and grouted couplers for the base connections.

The Contractor's 3D modelling of wall and foundation reinforcement allowed review and refinement of details in the design phase and was used as a construction observation tool. Robust temporary works design and staging ensured the stability of the precast elements in construction.

1 INTRODUCTION

This paper describes the seismic design of the new Whanganui Courthouse, a three-storey building currently in construction with a completion date of March 2027. Focus is given to geotechnical challenges and to the precast concrete shear walls which are the sole lateral load-resisting system in each principal direction.

With the new courthouse, the Ministry of Justice seeks to create a more community-centred approach to justice and social outcomes. The new Whanganui Courthouse is to be part of a wider community wellbeing hub, Te Puna Hapori, which is a partnership between Whanganui Iwi and Hapū, the Ministry of Justice, New Zealand Police, and Whanganui District Council. For further information and the overall site masterplan, interested readers are directed to the New Zealand Law Society Lawtalk Issue 963 which is linked in the Reference list.

As shown in the ground floor plan (Figure 1), the courthouse consists of two seismically separate buildings – a three-storey building which is the focus of this paper and a single storey building, both wrapping around an internal courtyard.

Conceived prior to COVID-19, the building concept was developed during a time of escalating construction costs and required multiple value engineering rounds and extensive input from ECI partner Naylor Love Construction to establish a realistic construction budget and to realise the most efficient construction methodology.

The project architects are Jasmax and Baukultur, whilst WSP has provided all engineering disciplines and specialist services.

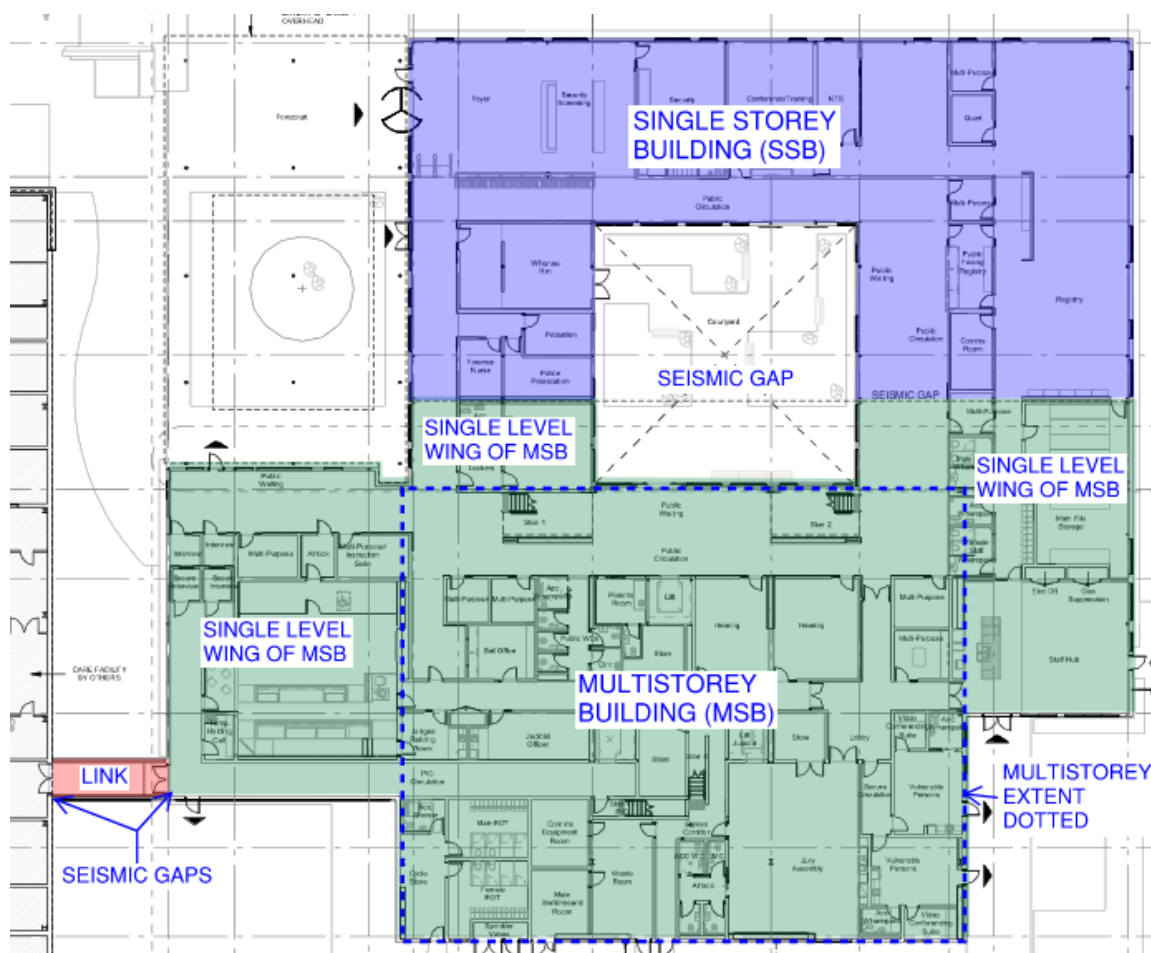


Figure 1: Courthouse ground floor plan

2 DESIGN PRINCIPLES

To improve the performance of the Courthouse against earthquake and other environmental effects, a low damage design approach has been adopted. Low damage design (LDD) involves additional and more stringent SLS performance objectives (compared to Building Code minima) for the structure and building services.

The LDD criteria were discussed and framed specifically for the project with the Ministry's technical review team including John Hare (structural reviewer) and Rick Wentz (geotechnical reviewer). A particular focus was on striking a balance between robust design to reduce the damage from a significant natural hazard event and the initial capital cost and the likely repair and reinstatement cost. This was a significant factor in the selection of the foundation, as discussed in Section 3.

We note that the technical review was a high-level review at the early design stages. An additional structural peer review for Code compliance purposes was undertaken prior to Building Consent submission.

2.1 Seismic hazard

At the time of design, the Technical Specification TS 1170.5 Structural Design Actions had not been released (in draft or in final). Therefore a site-specific hazard analysis was undertaken by WSP using the 2022 National Seismic Hazard Model. This resulted in an uplift in design loads of about 60% compared to NZS 1170.5:2004, for this particular building (IL2, $T=0.4$ seconds, $V_{s30} < 150\text{m/s}$).

3 SITE INVESTIGATIONS AND FOUNDATION DESIGN

The site is geotechnically challenging with soft, liquefiable soils – onset of liquefaction damage is expected between a 25-year and 50-year return period earthquake, with up to 200mm free-field settlement predicted at the 500-year earthquake.

The geology is described in the geotechnical interpretive report (Tonkin & Taylor, 2021) as middle to late pleistocene alluvial gravel, sand, silt and clay deposits. Surface geology was described as dune sands underlying most of the southern end of the site with some pumiceous alluvial sands and swamp deposits along the northern extent of the site near Campbell Street.

WSP undertook detailed site investigations in 2023-2024 including two boreholes to a depth of 46.75m and 40.3m BGL respectively with SPT tests every 1.5m depth. Vertical Shear Wave Profile (VSP) testing was undertaken within the boreholes, as well as two arrays of passive Multichannel Analysis of Surface Waves (MASW).

Deep foundation systems were ruled out due to a lack of a competent bearing layer(s) at a reasonable depth. Our focus turned to shallow foundation systems with ground improvement to mitigate against liquefaction. Multiple ground improvement options were investigated, including soil reinforcement (DSM or CSM lattice walls), soil densification, and mass stabilisation (cement/soil mixing). These were ultimately ruled out due to prohibitive capital costs and/or other technical reasons. To keep this paper brief, these options are not described in any detail here.

The final foundation solution is a reinforced concrete raft slab which provides a large bearing surface to mitigate against ground liquefaction and differential settlements (it extends 4.5m in plan past the building façade generally). The foundation was designed for overstrength seismic loads from the reinforced concrete shear walls. It can be re-levelled post-liquefaction using grout injection methods (confirmed with a specialist contractor). The pre-existing dune sands have been utilised as a natural crust/bearing layer.

The multistorey building footprint was preloaded for three months to take out the majority of the consolidation in the soft soil layers, before commencement of construction.

3.1 Concrete raft mix design

The raft mix design employed 16% fly ash to reduce embodied carbon. Other SCM's were permitted but fly ash was the supplier's preference. A 40 MPa design compressive strength was required to resist the significant punching loads at the shear wall toes. At highly congested areas around shear walls, a high slump semi-flowable self-compacting concrete (SFSCC) mix design was designed and employed in consultation with the Firth plant engineer.

4 SUPERSTRUCTURE DESIGN DEVELOPMENT

Early designs involved a singular two-storey building wrapping around the internal courtyard. In a bid to reduce gross floor area, the courtrooms were consolidated into a three-storey block with an adjacent single storey building serving as the public entrance and registry etc. The three-storey block has repetition in the floor plate for the court layouts and a circulation core which lends itself to the use of shear wall construction.

The shear walls were positioned both in the core and relatively evenly across the floor plate to distribute the seismic loads more uniformly across the raft foundation, avoiding concentrated loads and thereby reducing the required foundation depth. This was a critical driver in the design, as constructing the foundation above the water table significantly reduced construction complexity, lowered cost, and decreased project risk. An alternate structural steel EBF option was considered; however, the EBF geometries did not align well with the architectural layouts and detailing and were less space efficient in terms of GFA than the concrete walls.

4.1 Precast concrete shear walls

The shear walls are 350mm thick from ground to second floor and reduce to 250mm thick above second floor, as separate panels joined on site. There are eight 3.5m long walls in the north-south direction and five 5m long walls in the east-west direction. Refer to Figure 2 for a perspective view and Figure 7 for a progress photo. The first two stories are 5.0m tall in order to accommodate the high courtroom ceilings. This makes the 350mm thick panels 11.5m tall (including foundation depth and transition detail at second floor).

All walls incorporate ductile detailing to NZS 3101 Section 11.4, including confined end regions and transverse links throughout (refer to Figure 5).

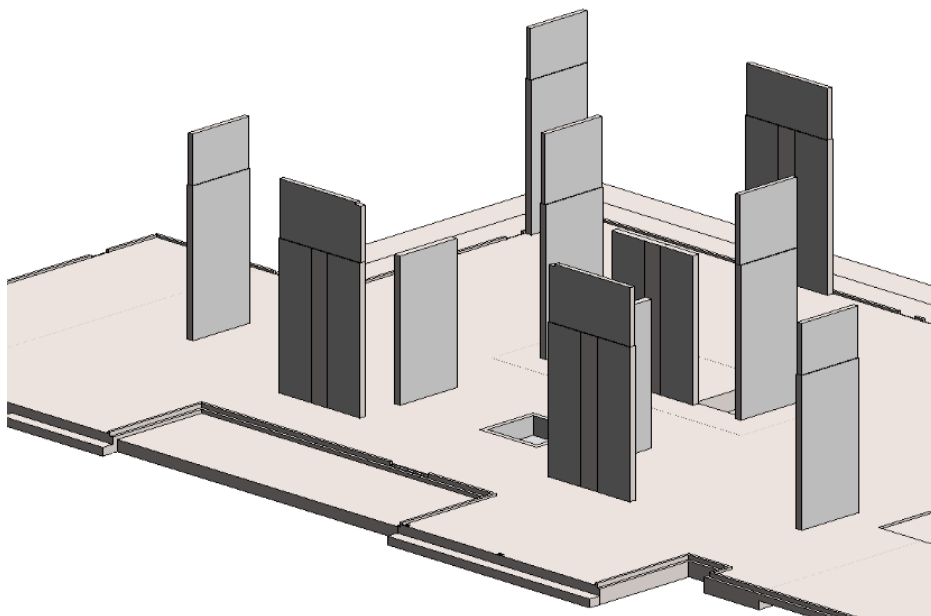


Figure 2: View of precast shear walls and raft foundation

5 NOTABLE PRECAST CONCRETE DETAILS

Various details of interest relating to the precast concrete walls are described below.

5.1 Base connections to raft slab

Figure 3 shows the typical panel base connection. The panels were designed to be ductile which ruled out common precast wall base details such as vertical grout tubes or vertical grouted couplers. Instead, the panels extend to the base of the raft slab and directly connected to the raft reinforcement via the Reid RB32 couplers positioned centrally in the precast unit. Another option for such a connection is horizontal ducts with the raft reinforcement passed through the ducts. However, this would have required the ducts to be individually filled with a thixotropic grout. Without this additional step, there is possibility of a void forming in the duct which can be detrimental to the strength and stiffness of the connection.

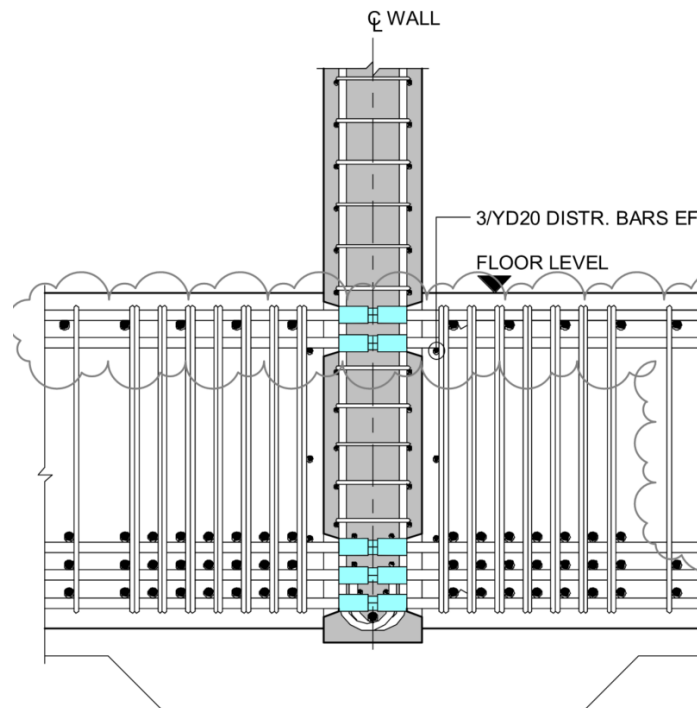


Figure 3: Wall base connection

5.2 Top connections to roof steelwork

Similar to the floor connections, it was deemed easiest to site weld the roof steelwork to the shear walls. Figure 4 shows the main connections, developed collaboratively between WSP and Naylor Love during whiteboard sessions as part of the ECI workshops.

Steelwork was run over the top of the precast panels to simplify the bracing connections, allowing the complex connections to be made steel-to-steel. This enabled a straightforward shear connection using an inverted U-shaped folded plate to drag loads into the top of the shear wall.

The U-shaped plate provided built-in vertical and horizontal tolerance between the precast panel and the roof steelwork. The plate was site-welded to both the precast panel and the roof steelwork. Because this connection allowed for significant tolerance, it eliminated the need for site measuring between precast elements followed by shop drawing and custom fabrication of steelwork. The flexibility of this detail resulted in a programme saving by removing a critical path construction / design / coordination step, and resulted in completion of this work set ahead of schedule.



Figure 4: Roof steelwork/precast connections – to UB rafter (top) and to SHS collector element (bottom)

5.3 In situ stitch joints to 5m shear walls

Due to craneage limitations, the 5m long shear walls were constructed as two 2m long shear walls with a 1m in situ stitch joint the full height of the panel (two stories). Figure 5 shows the joint reinforcing in plan and an Figure 6 an example stitch joint prior to forming and pouring. With precise placement of the horizontal starter bars in the precast yard, the joints were constructed with minimal issues on site.

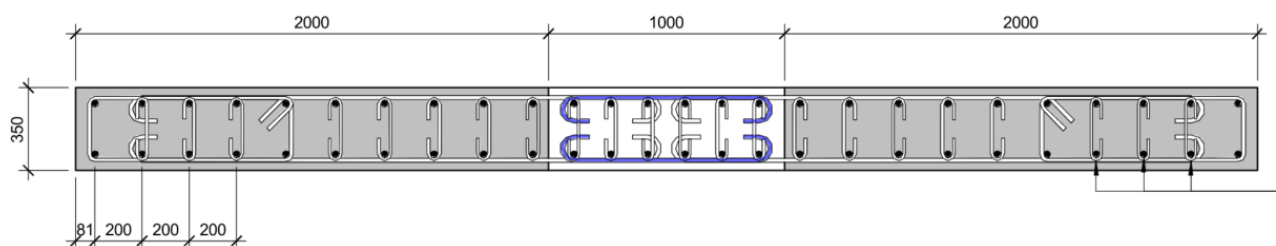


Figure 5: In situ stitch joint plan detail



Figure 6: In situ stitch joint construction example

6 CONSTRUCTION SEQUENCING AND TEMPORARY WORKS

Due to the weight of the precast panels (up to 36 tonnes each) and their layout, propping back to deadmen was not preferred. The scale of temporary works required would have been excessive, and the size and placement of deadmen would have conflicted with the raft foundation layout, causing additional cost and programme impacts. To address this, a staged construction methodology was developed with the site team during the ECI phase. Key portions of the raft were poured early to provide structure to prop the panels, effectively integrating the temporary works into the permanent works. Refer to Figure 7.

Construction joint locations and the reinforcing installation sequence at each precast position were carefully planned to avoid sequencing constraints.



Figure 7: Precast shear wall installation in progress

Localised complexities arose around the lift shafts due to a void on one side and local reinforcement congestion in this area. The construction sequencing required meant that some of the panels could not extend to the bottom of the raft, as a result they were temporarily supported on ‘stilts’ made up of steel UC sections, allowing starter bars to project unobstructed and enabling reinforcement to be placed and tied around them without interference (Figure 8).

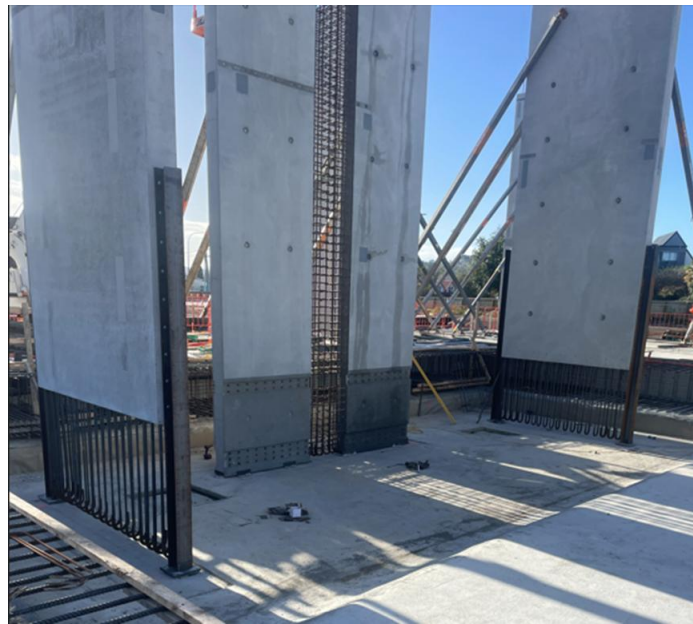


Figure 8: Precast panels beside lift shafts, supported on steel UC ‘stilts’

The ECI planning, of the reinforcing construction sequencing, was carried directly into the shop models / drawings, with all wall and foundation reinforcement fully modelled (Figure 9) and coordination between panel couplers and raft reinforcement fully checked. This transitioned smoothly into the construction phase, reinforcement was installed according to plan, with minimal need for on-site design modifications.

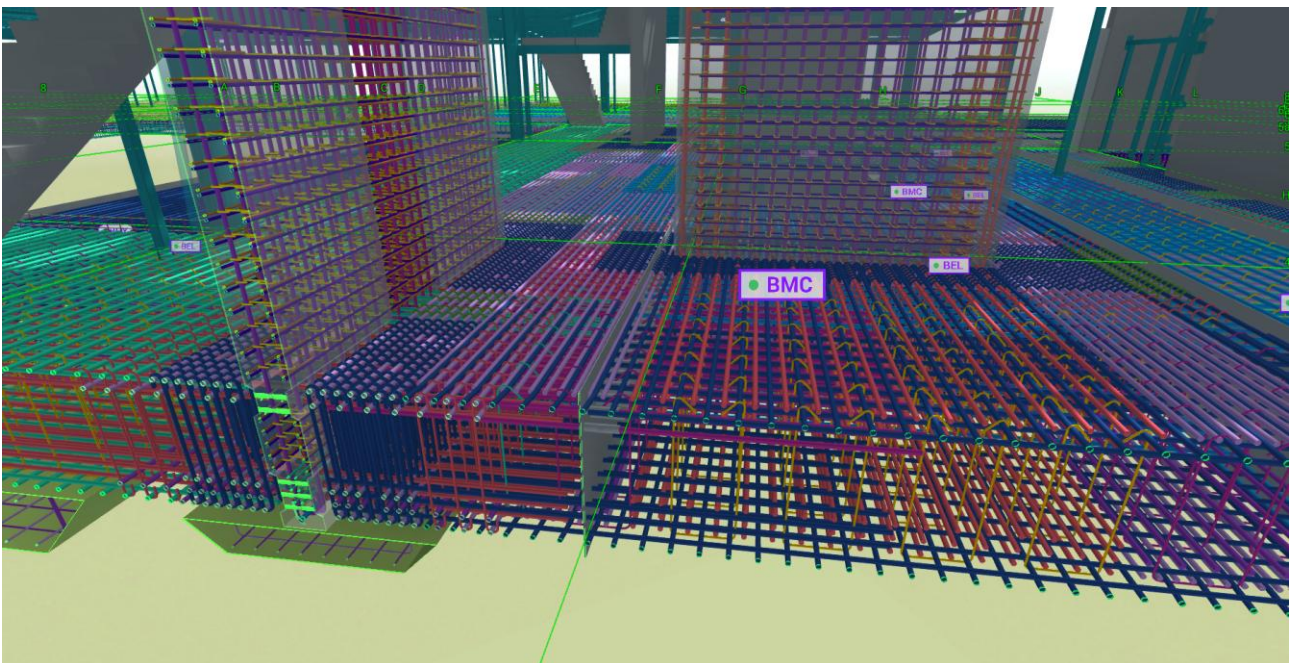


Figure 9: Excerpt from reinforcing model

7 CONCLUSIONS

This paper has described the seismic design of the new Whanganui Courthouse, a three-storey building currently in construction with a completion date of March 2027. Focus has been given to geotechnical challenges and to the seismic design/detailing of the precast concrete shear walls which are the sole lateral load-resisting system in each principal direction.

The paper has also explored the benefits of Early Contractor Involvement in the design stages to challenge the structural design and solve complex construction staging and detailing. The most cost-effective design solution on paper is not necessarily the solution that brings the most construction benefits and cost/risk reduction. For example, making the raft slab thicker would reduce the reinforcement quantity/complexity but increase construction complexity due to the high site water table. In a similar vein, welding on site is generally avoided as a design office rule of thumb. However, it was easily achievable on this project and allowed the steelwork to connect seamlessly to the precast shear walls. 3D reinforcement modelling was an invaluable tool in testing the raft construction sequencing and the congestion of complex reinforcement.

REFERENCES

New Zealand Law Society. *Lawtalk Issue 963, Te Puna Hapori*.

<https://www.lawsociety.org.nz/news/publications/lawtalk/lawtalk-issue-963/te-puna-hapori/> (Accessed 25 January 2026)

Tonkin & Taylor (2021). “*Te Puna Hapori o Whanganui Redevelopment Geotechnical Interpretative Assessment Report*”. Tonkin & Taylor, Wellington.