
Seismic Strengthening of the Carillon Tower – a National Historic Landmark in Wellington, New Zealand

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ABSTRACT

This paper presents the seismic strengthening of Wellington's National Historic Landmark and Category 1 heritage listed Carillon Tower to 100% New Building Standard. The 52 metre, 1930s reinforced concrete tower forms part of the National War Memorial site and houses the world's third-largest carillon with 74 bronze bells. Its signature structural characteristic - a transition from a stiff, solid shear wall base to a more flexible cantilever wall system - is acoustically beneficial yet leads to high seismic demand and drift toward the apex.

A displacement-led retrofit, using non-linear time history analysis (NLTHA), was developed to control global response while preserving heritage fabric and acoustic performance. Viscous dampers were installed within discrete steel braced frames at the tower summit, with device properties and frame stiffness tuned to the modelled deflection profile. The solution provides supplemental damping and targeted stiffness, effectively reducing seismic forces and relieving actions on original concrete elements and connections. The intervention minimises added mass and avoids intrusive modification of primary walls, maintaining exterior appearance, heritage fabric and sound propagation pathways.

Sustainability was considered through a low-embodied-carbon steelwork strengthening solution, rationalised detailing, and comprehensive corrosion protection tailored to the maritime environment. There was significant (70%) embodied carbon savings and cost savings (50%) of this solution when compared to a more traditional, heavy construction strengthening approach proposed by a previous seismic strengthening scheme. The use of NLTHA and a carefully tuned damping approach led to significant time, cost and embodied carbon savings for this National Historic Landmark, allowing it to continue to serve its commemorative and musical purposes for generations to come.

1 CARILLON HISTORY

The Wellington Carillon was constructed in 1931 and opened on ANZAC Day 1932 as the first structure on the National War Memorial site at Mt Cook, Wellington. It was conceived as a national landmark commemorating New Zealand's military service in the First World War. The tower was designed by Gummer and Ford, who won a design competition for the Carillon and were also responsible for designing the Auckland Railway Station, the Dunedin Cenotaph and the Bridge of Remembrance in Christchurch.



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

The Carillon is a fixed-bell musical instrument. The Wellington Carillon houses 74 bronze bells with masses ranging from approximately 12 tonnes to 4 kg. The original 1932 installation had 49 bells, later extended to 74, giving a total bell mass exceeding 70 tonnes and making it one of the largest carillons in the world. The bells are supported in upper and lower bell frames, with a clavier chamber between (Figure 3). Each bell is rigidly fixed to a headstock, with its clapper connected by a wire to a manual key and pedal keyboard in the clavier room, where the carillonist plays the keys to pull the wire to strike the bells.

2 CARILLON STRUCTURE

The Carillon Tower is a 52 m tall reinforced concrete structure. At lower levels it comprises solid reinforced concrete walls about 380 mm thick, forming a tubular core. At higher levels these walls transition to cantilever corner piers and discrete concrete columns (Figure 2 and Figure 3), providing the openness required for sound transmission across central Wellington. The plan footprint at the base is approximately 10.3 m by 12.7 m (about 131 m²).

The tower is supported on shallow raft foundations founded on weathered Greywacke. The original design calculations explicitly considered uplift and rocking of the foundation system, indicating that controlled rocking was intended as part of the seismic response. A nominal 100 mm seismic gap separates the tower from the adjacent Hall of Memories. Under NZS 1170:2002, the Carillon is classified as an Importance Level 3 (IL3) structure, reflecting both its heritage significance and the high value of the instrument to the community.



Figure 2: Structural 3D model of the Carillon Tower

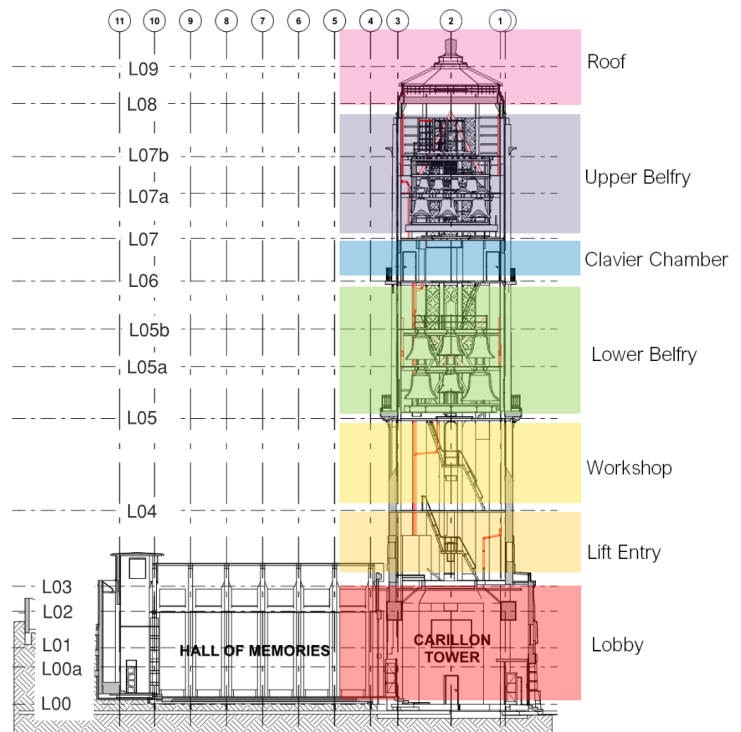


Figure 3: Section showing layout of the Carillon (Studio Pacific Architecture 2023).

The tower has undergone several previous seismic interventions. In the 1980s concentric steel braced frames were installed near the levels of the upper and lower bell frames to increase lateral stiffness and strength. In 2012 further minor strengthening and access upgrades were implemented. A Detailed Seismic Assessment (DSA) in 2020 identified several primary elements with capacities less than 34% NBS. Holmes was initially engaged to peer review this assessment and was subsequently commissioned to undertake a detailed non-linear assessment and to design the current strengthening works.

3 ASSESSMENT METHOD

Seismic assessment and retrofit design were carried out using ANSR, Holmes' in-house non-linear structural analysis software. ANSR incorporates the ANSR-II non-linear analysis engine (Mondkar and Powell, 1979) with additional element models and pre/post-processing. The tower was modelled using a concentrated (lumped) plasticity approach, with non-linear behaviour captured by rotational hinges or concentrated plastic regions at critical locations in walls, beams and columns.

Non-linear time history analysis (NLTHA) was adopted as the main assessment method, enabling modelling of the inelastic response, redistribution and failure progression. This provided a detailed understanding of deformation patterns and damage mechanisms under severe shaking, and allowed the retrofit to be focused on the critical elements and modes that controlled capacity. Material properties for concrete and reinforcing steel were based on the original specifications, calibrated using the NZSEE Assessment Guidelines (NZSEE 2017, 2018). Modal analysis was performed to obtain fundamental periods and mode shapes; the dominant translational and torsional modes of the strengthened building are summarised in Figure 4.

Ground motion selection and scaling generally followed ASCE 41-17 procedures for response history analysis, using records scaled to be hazard-consistent with the IL3 performance objectives and including near-fault effects appropriate to Wellington.

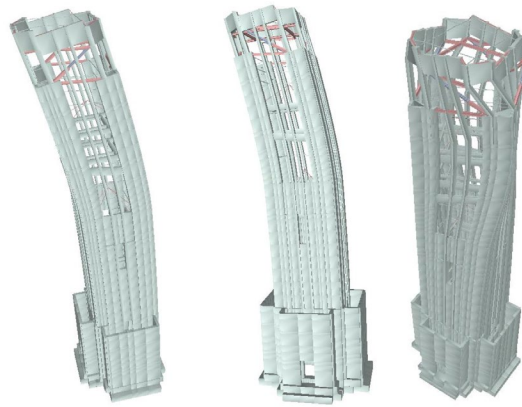


Figure 4: Fundamental modes of 0.85s X translation, 0.81s Z translation and 0.13s torsional translation (strengthened building).

Numerical acceptance criteria were primarily taken from ASCE 41-17 for reinforced concrete walls and beams, using the recommended modelling parameters and deformation limits for non-linear procedures. For columns, acceptance criteria followed NZSEE Assessment Guidelines Section C5, reflecting New Zealand practice for existing reinforced concrete detailing.

For the existing configuration, NLTHA indicated a global capacity of around 34% NBS (IL3 demand), beyond which progressive local failures would lead to loss of global stability. The main structural weaknesses were:

- Low deformation capacity and early onset of non-ductile behaviour in the lower-level 1980s steel bracing and associated concrete columns, where the stiff steel braces attract large seismic forces and impose high demand on the adjoining concrete elements.
- Displacement incompatibility at upper levels between the steel bracing and surrounding concrete walls/columns, with the braces forcing a stiff braced-frame deformation profile on the more flexible cantilever wall/column system, leading to local overstress and detailing-incompatible demands.

4 CONCEPT STRENGTHENING SCHEMES

Using the calibrated ANSR model, four principal retrofit options were developed to concept level and evaluated with NLTHA. All of the solutions involved removing the 1980's steel braced strengthening and replacing them with more modern, complementary options:

- Fluid viscous dampers integrated into a steel moment-resisting frame (MRF) at upper levels, targeting 100% NBS.
- A compact system combining buckling restrained braces (BRBs) and post-tensioning of the corner piers, constrained by available space.
- A higher damage solution with a conventional steel MRF only, without supplemental damping.
- A more traditional fully braced internal steel frame system.

Each option was assessed for seismic performance, construction complexity, heritage impact, sustainability, damage control and cost impacts. NLTHA was used to compare drift, acceleration and force demands, and to quantify local demand redistribution and residual drifts. The preferred scheme, selected by the client, was the viscous-damped MRF solution designed to achieve 100% NBS (IL3).

Because the existing tower has limited inherent damping and is subject to high Wellington seismic demands and near-fault effects, supplemental damping with fluid viscous dampers provided an efficient means of reducing response. Order-of-magnitude costing indicated that the viscous-damped scheme was roughly half the cost of the fully braced internal frame. Quantification of material quantities showed that the viscous-damped solution used about 30% of the embodied carbon of the fully braced alternative, a reduction of approximately 70%, while still achieving the same performance objective.

5 STRENGTHENING DESIGN

The adopted strengthening system consists of new steel MRFs, with supplemental fluid viscous dampers, installed in the upper portion of the tower, integrated with the existing concrete structure. The frames provide a robust load path where seismic demands are highest and are continued down below the level where the concrete columns and pier wall system transitions to solid walls (approximately Level 5), improving vertical continuity and increasing the flexural capacity of the concrete piers. Original, slender reinforced concrete braces within the external lattice structure were selectively weakened to prevent these braces, and the adjacent slender concrete columns, from failing under large seismic displacements.

In ANSR, the viscous dampers are modelled as purely velocity-dependent elements (dashpots), with force defined as a function of relative velocity across the damper. Damper coefficients and velocity exponents were iteratively tuned so that global and local responses under NLTHA met the IL3 performance objectives for drifts, accelerations and member demands. Direct calibration of damper properties within the non-linear model was central to obtaining an efficient configuration that reduced both construction cost and embodied carbon.

Compared with a fully braced steel frame, the viscous-damped system reduces the seismic forces transmitted into the existing structure by dissipating energy rather than primarily increasing stiffness. Because the supporting frames are relatively flexible and the dampers are velocity-dependent, the natural displacement profile of the original tower is largely preserved, avoiding the severe stiffening and demand redistribution that the earlier bracing had introduced. The retrofitted configuration exhibits reduced displacements, accelerations and internal forces compared with the existing building, improving overall performance and robustness.

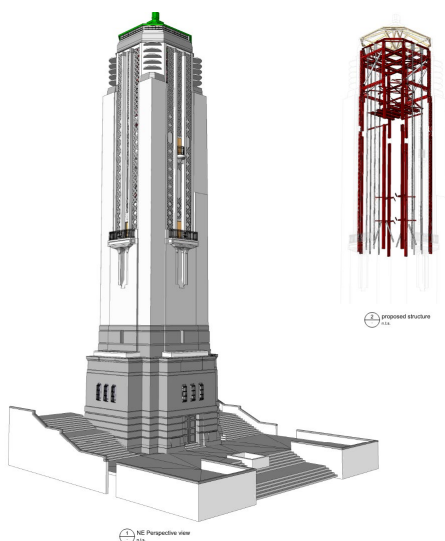


Figure 5: Structural steel added to building

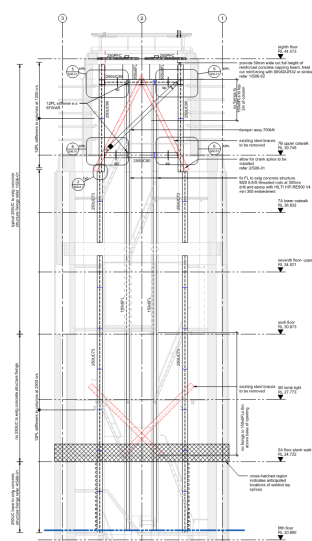


Figure 6: Top half of one face showing new VD moment frame (existing braces to be removed in red)

The new MRFs also provide additional flexural strength and stiffness to the upper concrete piers. The vertical steel elements extend from the tower summit down to Level 5, tying the flexible upper storeys to the stiffer lower walls and increasing redundancy. The connection between the steel columns and the concrete wall is capacity designed, ensuring a ductile flexural failure mode. Figure 5 shows the new steelwork isolated from the structural model, and Figure 6 provides an elevation of one face of the strengthening scheme in the upper half of the tower.

The influence of the viscous dampers on the global seismic response is illustrated in Figure 7, based on comparative non-linear analyses of models with and without the viscous dampers. The damped case shows a marked reduction in peak accelerations, displacements and interstorey drifts at the upper level, and a more even distribution of seismic demand over the height of the structure. The dynamic "whip" effect at the top, associated with the flexible upper column system, is substantially mitigated, with corresponding demand redistributed down the height of the tower.

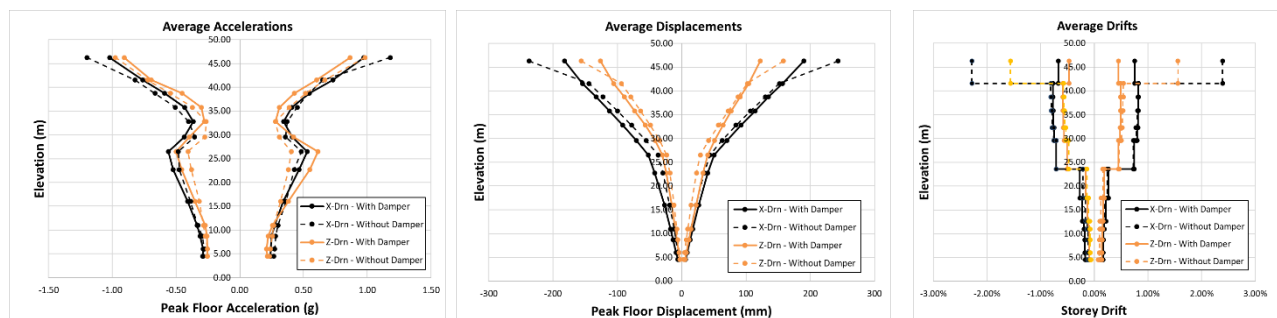


Figure 7: Peak floor acceleration, displacement and interstorey drift plots for the tower with (solid line) and without (dashed line) dampers. Orange lines are Z direction and black are X direction.

The NZ Building Code Verification Method B1/VM1 does not explicitly cover fluid viscous dampers. The retrofit was therefore treated as an Alternative Solution under the New Zealand Building Code, supported by a performance-based design and independent peer review.

6 BELL FRAME STRENGTHENING AND CORROSION ISSUES

The tower is located approximately 1 km from the coast, and sea-salt laden winds enter the open upper levels. Internal concrete and steel surfaces are not regularly washed by rain, resulting in a highly aggressive chloride environment. Consequently, the steel elements of the upper bell frame and adjacent steel bracing have experienced widespread corrosion (Figure 8), with previous recoating and member replacement providing only short-term improvement.

Reinforced concrete elements have also been affected, with cracking and spalling observed at various elevations. A past impressed-current cathodic protection system was unsuccessful, largely due to failure of titanium anode wires under wind-induced movement. A specialist corrosion consultant was therefore engaged to develop a durable mitigation strategy. The adopted approach includes full abrasive blasting and recoating of bell frame steel in a controlled environment, using a high-performance coating system appropriate for internal marine exposure. New steelwork is detailed and coated for the same exposure. Concrete deterioration is being addressed through repair and a maintenance regime including regular washdown to remove deposited salts and reduce chloride ingress.

Seismic strengthening of the bell frames used acceleration demands at the frame levels obtained from the ANSR NLTHA model. These were compared against accelerations derived from NZS 1170.5:2004 Section 8

(Parts and Components). NLTHA-based accelerations in the upper tower were more than 30% lower than the code Parts values, which significantly reduced the strengthening required for the bell frames and their connections.



Figure 8: Upper bell frame - Locations of typical corrosion



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

7 CONSTRUCTION PROGRESS

The Ministry for Culture and Heritage awarded the main construction contract to Maycroft Construction Limited in April 2024. At the time of writing, strengthening works are nearing completion, with reopening targeted before ANZAC Day 2026.

Construction within the tower is constrained by very limited access. Internal circulation is by steep ladders, and a small non-commercial lift which only serves Levels 3 to 6 and cannot transport steel members forming part of the strengthening scheme. In practice, steel members had to be brought into the tower as relatively short pieces through existing or temporary floor openings. This strongly influenced member sizing, splicing and connection details. The tower has been fully scaffolded and wrapped externally to provide safe access and weather protection, and extensive internal temporary works and rigging were required for lifting and positioning steel within the confined interior.

The contractor undertook 3D laser scanning of the internal spaces prior to fabrication. The resulting point cloud informed the construction methodology and the steel shop drawings. This allowed clashes and tolerance issues between new and existing work to be identified and resolved digitally. As a result, steel erection has proceeded with minimal fit-up problems or rework (Figure 9).

8 CONCLUSIONS

The seismic strengthening of the Category 1 heritage-listed Wellington Carillon Tower to 100% NBS for IL3 demonstrates the value of combining performance-based earthquake engineering with sensitive heritage intervention. Detailed non-linear time history analysis provided insight into the interaction between the original reinforced concrete structure and earlier steel bracing, identified the mechanisms limiting capacity at about 34% NBS, and guided a targeted retrofit focused on those mechanisms.

The adopted solution integrates fluid viscous dampers within new steel MRFs at the upper levels, together with flexural strengthening of the upper concrete piers through steel framing extending down into the solid wall levels. The NLTHA results reduces accelerations in the upper tower by more than 30% relative to NZS 1170.5 Parts values which lowers force and deformation demands on both the existing structure and the bell frames. Compared with a conventional fully braced internal frame, the selected system achieves the 100% NBS (IL3) performance objective at about half the construction cost and with an estimated 70% reduction in embodied carbon.

The project illustrates how the effective use of non-linear analysis and supplemental damping demonstrated by an Alternative Solution compliance path can deliver efficient, low-intrusion, seismic strengthening for a complex heritage structure such as the Carillon tower. This has ensured that this nationally significant landmark, and world significant carillon instrument, is here for future generations to enjoy.

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