Ministry for Culture and Heritage
National War Museum - Carillon Tower STR

Taranaki Street
Mount Cook
Wellington

Design Features Report

Version 1
01 March 2024
144373.13

Holmes

Australia Netherlands New Zealand USA

Design Features Report

National War Museum - Carillon Tower STR

Prepared for

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Limitations

The calculations provided herein are for the sole use of the Ministry of Culture and Heritage and are provided for the purpose of Peer Review for the structural design outlined on the Holmes NZ drawings dates 25/08/2023 for the project at Taranaki St, Wellington. These calculations are not intended for use by other parties and may not contain sufficient information for the purposes of other parties or other uses.

Report Issue Register

DATE	REV. NO.	REASON FOR ISSUE
25 August 2023	A	Developed Design
22 September 2023	B	Peer Review
13 December 2023	C	
01 March 2024	1	Building Consent

CONTENTS

1 GENERAL

1.1 Objective

The Design Features Report (DFR) is a detailed document defining the building's design criteria and recording key decisions or outcomes. It outlines design loading, safety, structural modelling assumptions, material properties, foundation requirements and design standards. The DFR also defines the calculation procedure and checking principles to be followed, providing a clear explanation of the full building design.

The DFR also is a live document that will be updated as the design and construction proceeds. At completion it forms a part of the design record and should be archived for future reference if required.

1.2 Scope

The scope is in accordance with the Design Brief and Conditions of Engagement. The scope for the Detailed Design phase of this project includes:

- Design of viscous damper strengthening scheme to the Carillon Tower to achieve as close to 100% NBS (IL3) as practical.
- Design of replacement Upper Bell Frame steel structure
- Design of Lower Bell Frame strengthening
- Accessibility improvements to the ground floor
- Condition survey priority 2 & 3 items

Figure 1: Photograph of the National War Memorial Carillon Tower

2 THE PROJECT

2.1 Overview of the Project

2.2 Health and Safety in Design

Key Health and Safety in Design (H&SiD) considerations for the project are as follows:

- Sequencing of construction inside a currently classified earthquake prone building.
- Design to allow new steel frames to be segmented for ease of construction and safe handling
- Working at height will be required – consider what can be preassembled to minimise working at height.

These have been communicated to the client and other project parties during the course of the design, and buildability workshops held with the contractor to discuss and design out these risks where possible. Design considerations as a result include:

- Bolted splices added throughout to minimise welding at height required, and allow steel sections to be brought in in manageable lengths. This also minimises the need to apply steel coatings on site.
- Sequence of removal of previous strengthening works and cutting of structure has been discussed with the contractor, to agree an approach that mitigates risk as much as possible.

2.3 Means of Compliance

Refer to Holmes' Compliance Pathway Memo dated 30/06/23 for further information on the means of compliance for this project.

3 THE STRUCTURE

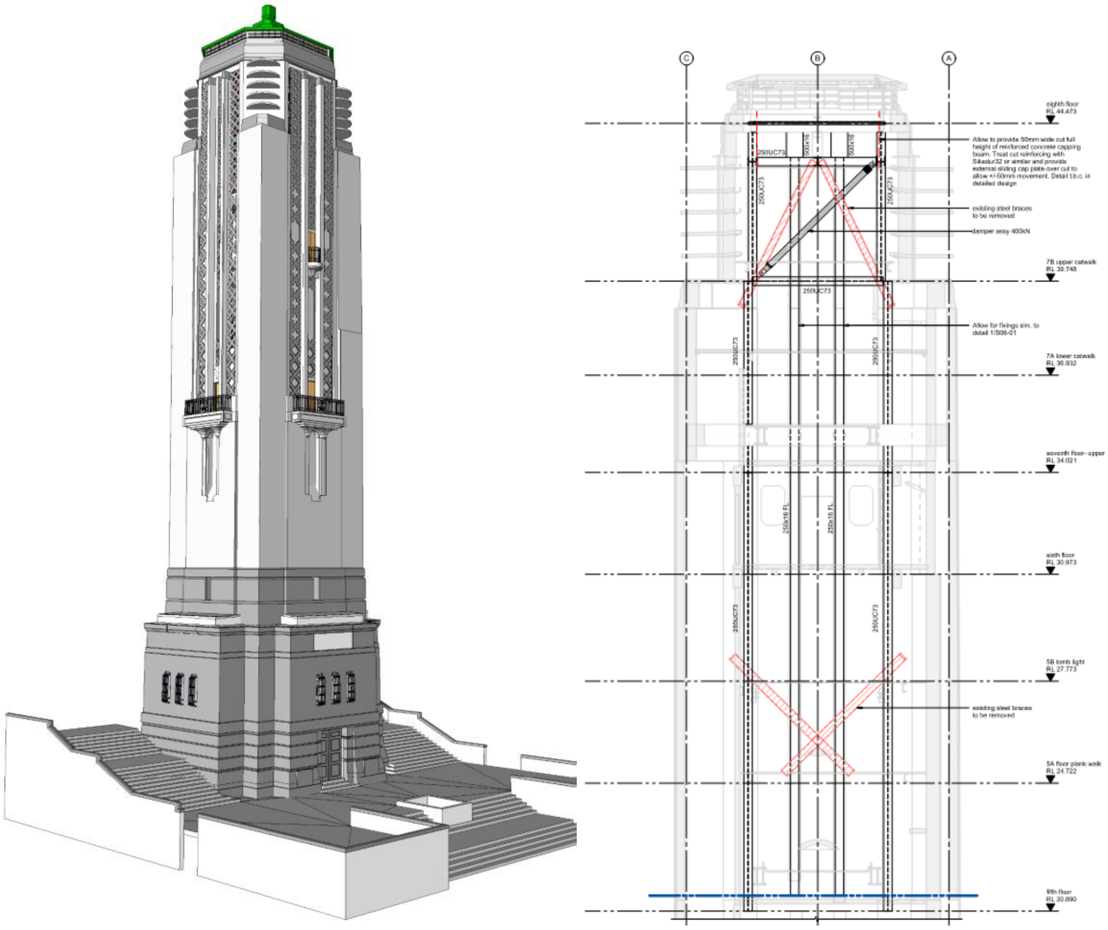

Figure 2: Perspective view of the existing tower and elevation of the upper half of the tower showing strengthening

3.1 Gravity Structure

The Carillon Tower is a simple structure for gravity, with a lightweight timber framed roof over reinforced concrete walls. Generally, the seismic strengthening does not modify the gravity load paths. The reinforced concrete structure is minimal near the top, with slender concrete piers at the four corners of the building. Below Level 7a these widen into hollow triangular concrete piers at each corner, with a precast concrete lattice structure in between allowing for the acoustic performance of the musical instrument. At Level 5 these piers join into solid reinforced concrete walls down to ground. The foundation system is a concrete shallow foundation.

The typical floors are reinforced concrete slabs over concrete beams, with large square holes in the centre to allow for raising and lowering the bells. Some timber infill floors are also used.

The bells are supported on two steel frames, at Level 5b and Level 7. These frames rise from steel girders that are embedded in the reinforced concrete walls.

3.2 Lateral Load Resisting Structure

The primary lateral load resisting system in the existing building, built in the 1930s, are the reinforced concrete walls. Concrete in the tower uses round reinforcing bar throughout. Typically the walls appear to be fairly well detailed, with stirrups at close spacings. However, the use of round bar limits the seismic capacity of the walls due to degradative debonding of the bars in a seismic event. At the top of the tower, above Level 7b, slender concrete piers provide the existing lateral structure. These widen into larger concrete piers at each corner down to level 5a. Below Level 5a, the shear walls are almost solid, with minor openings in them for doors, floors balconies, and windows.

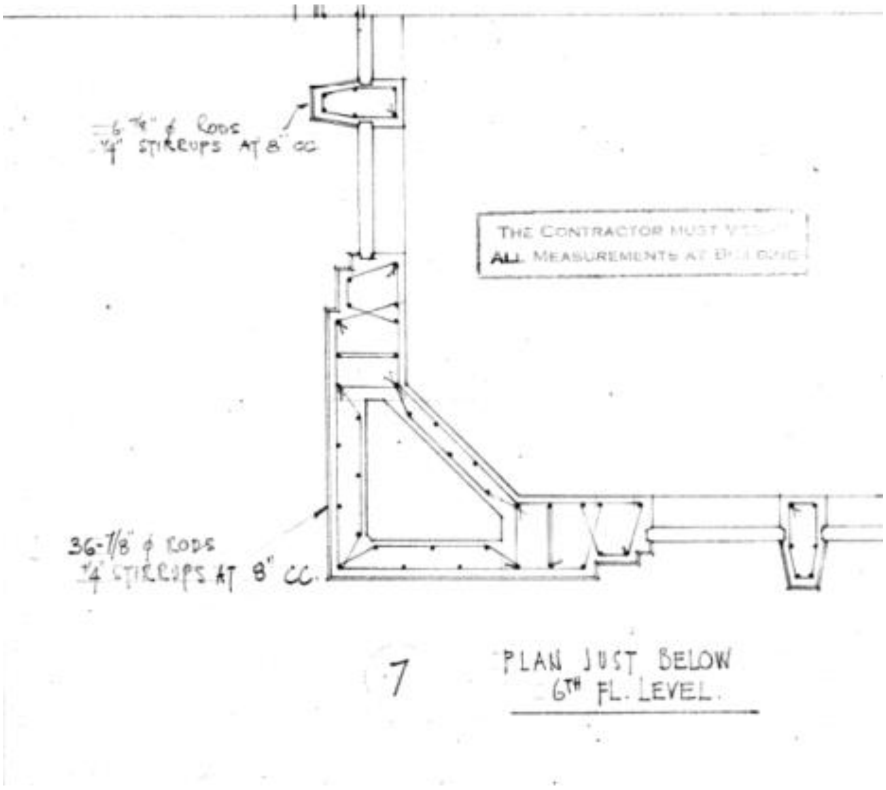

Figure 3: Typical existing reinforced concrete pier

The entire building appears to have been designed to rock on its foundations. Investigations have been undertaken on site to determine where the rocking interface might be. The onsite investigations have determined that the reinforcing bars from the tower pass through the wall to foundation interface. Therefore, the rocking interface is assumed to be at the base of the foundations between the foundation and the soils underneath.

The building has been strengthened previously, with braces added at the top of the tower and around Level 5b. A Detailed Seismic Assessment of the bell frames was carried out by Dunning Thornton in 2020 which resulted in the building being rated at below 34% NBS (IL3). The building is considered to be an Earthquake Prone Building by Wellington City Council. The critical structural weaknesses found included the upper and lower bell frames, the reinforced concrete piers between levels 5 and 6, the lower tower, and the reinforced concrete beam at the roof.

At the Concept stage, Holmes created a Non-Linear Time History model of the existing structure, in order to assess its lateral capacity. This assessment found that the steel braces that had been added in a previous strengthening phase were of limited benefit in the structural strengthening due to its deformation incompatibility with the surrounding concrete structure. Between Level 5a – Level 5b, the relative movement between corner concrete piers were found to drive large axial forces into the existing concrete and steel braces, causing structural failure at low intensities of seismic shaking. Between Level 7b – Level 8 of the structure, the steel braces were found to force unfavourable deformations into surrounding concrete elements, again causing structural failure at low intensities of seismic shaking. Refer to the Concept Strengthening Report, dated 22/11/2022 for further information.

To alleviate the concrete structure from these deformation driven forces, the existing steel braces are to be removed between Level 7b – Level 8 and Level 5a and 5b. Viscous dampers are instead introduced between Level 7b- Level 8, where concrete cantilever columns are left to form the lateral load resisting system. The dampers act to reduce the acceleration between the levels by means of absorbing kinetic energy, hence reducing the load felt by the concrete columns. Equally, they allow the columns to deform freely without added displacement-driven forces as imposed by the rigid steel braces, forming a more efficient and effective structural system. The dampers are tuned such that the effective lateral loading felt by the concrete columns is sufficiently reduced so that they can satisfy seismic performance criteria at the corresponding target intensities of seismic shaking.

In addition, the concrete ring beam running around the top of the tower is cut vertically at strategic locations. This is intended to minimise the interaction in deformation between the ring beam and pier, preventing this from driving large loads into the slender concrete columns central to each face of the tower. Steel straps are added the face of these columns, acting in tension to resist uplift loads in the columns resulting from the deformation of the tower.

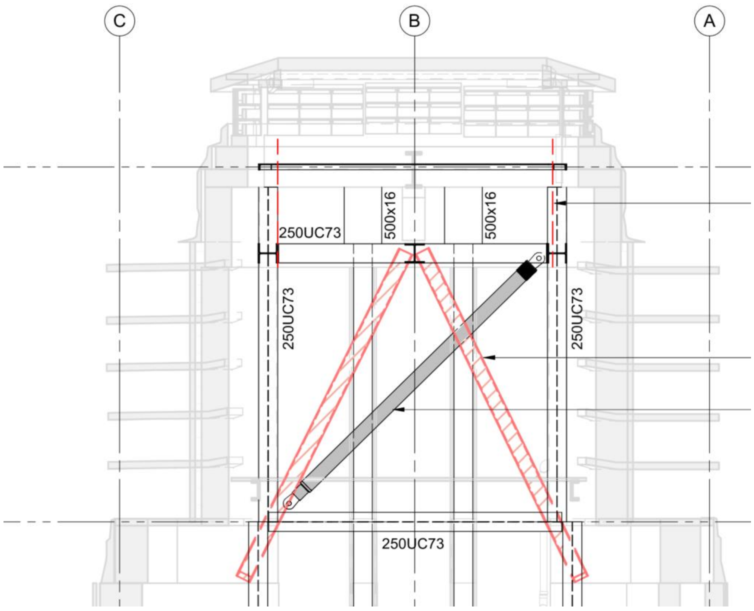

Figure 4: Typical viscous damper frame at the top of the tower

At Level 5a the solid concrete wall between the corner piers drops off, disconnecting the piers. This forms a sudden change of stiffness in the lateral system of the tower, and cause large bending moments to be driven into the piers at this level.

To increase the flexural and axial capacity of these piers, the steel columns of the viscous damper frame are extended down below Level 5a. This solution has been developed from preliminary design to remove 3 levels of viscous dampers, as the Non-Linear Time History Analysis showed that the additional flexural and axial capacity provided by the steel columns is adequate to achieve 100% NBS (IL3).

At the base of the tower, the surrounding ground level ramps up across the width of the tower, with external stairs running up on the East and West side of the building. During rocking, the tower displaces laterally against this surrounding soil. This is expected to induce significant reaction loads against the wall of the tower, causing local wall failures. To remedy this response, a layer of soil between the stair and the tower is to be replaced with a compressible material to alleviate local pressures on the tower during rocking.

The upper and lower bell frames are also being strengthened as part of this project. The steel structure supporting the bells is lacking in bracing capacity, and has been significantly corroded in some locations. Corrosion is to be addressed by a specialist corrosion expert, so is not in scope for this report. Generally corroded members will be either remediated or replaced like for like. Additional bracing members are added to each frame in the horizontal and vertical planes, to take the bell loads out to the concrete walls.

3.3 Significant Design Features

The National War Memorial Carillon is a Category 1 heritage listed building. It has been deemed to be an Importance Level 3 building due to its cultural and heritage significance. Therefore, the project is aimed at achieved as close as possible to 100% NBS (IL3) while interfering as little as possible with the heritage fabric and value.

The viscous dampers, which form an alternative solution under the NZ Building Code, are designed via a capacity spectrum based design approach and verified using Non-Linear Time History Analysis. They reduce the building accelerations at the top of the structure by absorbing kinetic energy, therefore reducing seismic load demands felt by the surrounding structure. The dampers have been adopted as an effective means to minimize the deformation incompatibility and seismic loads felt by the current structure, forming a more efficient structural system while minimising the impact on the heritage structure of the building. Seismic design requirements outlined in ASCE-17 Chapter 18 have been followed for the damper design, excluding Clauses 18.2.1.1 and Clause 18.2.4.6. as they do not apply to the seismic strengthening of existing buildings.

The tower is deemed to be a rocking type structure. It is supported by shallow raft foundations sitting on top of soil. During sufficiently large intensities of seismic shaking, the tower is expected to rock between the soil and the foundations, significantly reducing the seismic accelerations transmitted up the height of the structure.

The Carillon Tower is a functioning musical instrument housing 72 bells. The frames supporting the bells were found to be below 34% NBS, so will be strengthened as part of this project. The strengthening solution has been developed with consultation from the carillonist in order to maintain the musical qualities.

4 SOIL CONDITIONS

Geotechnical investigation, reporting and advice for the project has been provided by Engeo. Refer to the Engeo detailed design report dated 13/10/2023.

4.1 Description of Site Soil Conditions

The existing foundations likely sit on weathered greywacke. The ultimate bearing capacity of this has been assessed to be 850 kPa (without strength reduction factors applied).

For foundations 1.3 m to 4.5 m deep below ground surface, into the completely weathered greywacke, a lower bound vertical subgrade spring stiffness of 23 MPa/m and an upper bound vertical subgrade stiffness of 225 MPa/m were adopted on Engeo's advice.

The site subsoil Class C has been assumed as per NZS 1170.5:2004.

4.2 Seismic Soil Classification

The geotechnical advice confirms that Site Subsoil Class C – Shallow Soil is appropriate for determining seismic loads for the buildings in accordance with NZS1170.5:2004.

DESIGN LOADS

5.1 General

For the purposes of consideration of loading, this building is Importance Level 3 in accordance with AS/NZS 1170.0:2011. This was advised by the Ministry of Culture and Heritage, as the building contains items of high value to the community. The building has a design working life of 50 years. Table 5-1 summarises annual probability of exceedance for wind, snow and earthquake loads as specified in AS/NZS 1170.0:2011.

Table 5-1 Annual probability of exceedance for wind, snow and earthquake loads

Limit State	Wind	Snow	Earthquake
SLS1	1/25	1/25	1/25
ULS	1/1000	1/250	1/1000

5.2 Imposed Loads

5.2.1 Vertical Loads

As the building is not undergoing a change of use, gravity performance of the floors was assessed as part of the condition survey work previously completed by Holmes. No gravity strengthening was required. The below imposed gravity loads have been assumed where required:

Table 5-2 Imposed gravity loads

Design Use	Nature of Load	Uniformly Distributed Loads (kPa)	Concentrated Point Loads (kN)
All floors	LL	0.5	
	SDL	0.5	

5.3 Wind Loads

Wind loads are assumed to not be significant in the seismic strengthening project.

5.4 Snow and Ice Loads

Design snow loads are derived from AS/NZS 1170.3:2003 (including the modification to basic snow load given in Amendment 9 to the New Zealand Building Code). The building is in Region N1. The site elevation is less than 400 m above sea level. Snow and ice loads are not significant for this building.

5.5 Seismic Loads

Design seismic loads are derived from NZS1170.5: 2004. Table 5-3 details the seismic load site parameters applicable to the site.

Table 5-3 Seismic load site parameters

Design Parameter	Value
Site subsoil class	C
Hazard factor, Z	0.40
Return period factors, R	
SLS1	0.25
ULS	1.30
CALS*	2.34

Design Parameter	Value
Near-fault factor, N(T,D)	1.0

*In compliance with the Seismic Assessment Guidelines (2017), elements whose failure would likely cause building collapse are further assessed under the collapse avoidance limit state (CALS) (1.8 x ULS intensity of seismic shaking).

5.5.1 Seismic Design Strategy

The seismic design strategy adopted for the project is outlined in the Compliance Pathway memo dated 30/06/2023.

For the purposes of the seismic design, the project X and Z directions are considered to be the project EW and NS directions respectively.

5.5.2 Seismic Loads - Primary Structure

The time history analysis method set out in NZS1170.5 is applicable and is used to calculate the seismic design loads on the primary structure. Table 5-4 details the seismic design load parameters and for the building. The seismic weight of the building is 21,800 kN.

Table 5-4 Seismic design actions for primary lateral load resisting system

Design Parameter	ULS		CALS	
	X Dir	Z Dir	X Dir	Z Dir
Structural ductility factor, μ	1.0	1.0	1.0	1.0
Structural performance factor, S_p	1.0	1.0	1.0	1.0
Fundamental period (s)	0.66	0.68	0.66	0.68
Horizontal base shear coefficient, $C(T_1)$	0.28	0.23	0.36	0.33
Horizontal base shear (kN)	6070	5020	7800	7220

5.5.3 Seismic Loads - Parts and Components

Seismic design actions for building parts, and non-structural components, are computed using NZS 1170.5 Section 8. Table 5-5 details the site hazard coefficient, C(0), for the site.

Table 5-5 Site hazard coefficient C(0)

Limit State		
SLS1	SLS2	ULS
0.17	NA	1.23

The design response coefficient for a part is given by:

$$C_p(T_p) = C(0)\, C_{hi}\, C_i(T_p)$$

Where for h_i>12m and T_p<0.75s:

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Floor Height Coefficient : $C_{Hi} = 3.0$

Part Spectral Shape Factor : $C_i(T_p) = 2.0$

And the horizontal design actions on a part, F_{ph} are given by:

$$F_{ph} = C_p(T_p)\, C_{ph}\, R_p\, W_p < 3.6W_p$$

Where as shown in Table 5-6 the Part Response Factor, C_{ph}, is dependent on the ductility of the part, μ_p, and the Part Risk Factor, R_p, is dependent on the classification of the part (refer Table 5-7). Computed horizontal design actions for parts and components are summarised in Table 5-8 for short period parts ($T_p \leq 0.75s$) with a part risk factor, R_p, = 1.0.

Table 5-6 Part Response Factor

Ductility of Part μ_p	C_{ph} or C_{pv}
1.0	1.0
1.25	0.85
2.0	0.55
3.0 or greater	0.45

Table 5-7 Classification of Parts (from NZS 1170.5 Table 8.1)

Category	Criteria	Part risk factor R_p	Structure limit state[1]
P.1	Represents a hazard to human life outside the structure.[2,3]	1.0	ULS
P.2 and P.3	Represents a hazard to human life within the structure.[2,3]	1.0	ULS
P.4	Required for the continuing function of the evacuation (after earthquake) and human life support systems within the structure.	1.0	ULS
P.5	IL4 buildings: Required to maintain operational continuity[4,6] and/or All buildings: Required.to be operational/functional for the building to be occupied.[5,6]	1.0	SLS2
P.6	Where the consequential damage caused by its failure is disproportionately great.	$2.0\ R_u$[4]	SLS1
P.7	All other parts	1.0	SLS1

Table 5-8 Horizontal design actions for short period parts ($T_p \leq 0.75s$) and $R_p = 1.0$.

Level	SLS1	ULS
	$\mu_p = 1.0$	$\mu_p = 1.0$
Roof	0.80	3.60
Above Level 5	0.80	3.60

5.6 Construction Loads

To be reviewed as work proceeds – typically will be as a result of construction requirements.

6 SERVICEABILITY CRITERIA

6.1 Gravity Deflections

Any new structure is designed to the recommended serviceability deflection limits of AS/NZS 1170.0:2002, Table C1. Otherwise no worse than existing performance is acceptable.

6.2 Seismic Deflections

The seismic deflection profile of the tower at ULS and CALS are shown in Figure 5.

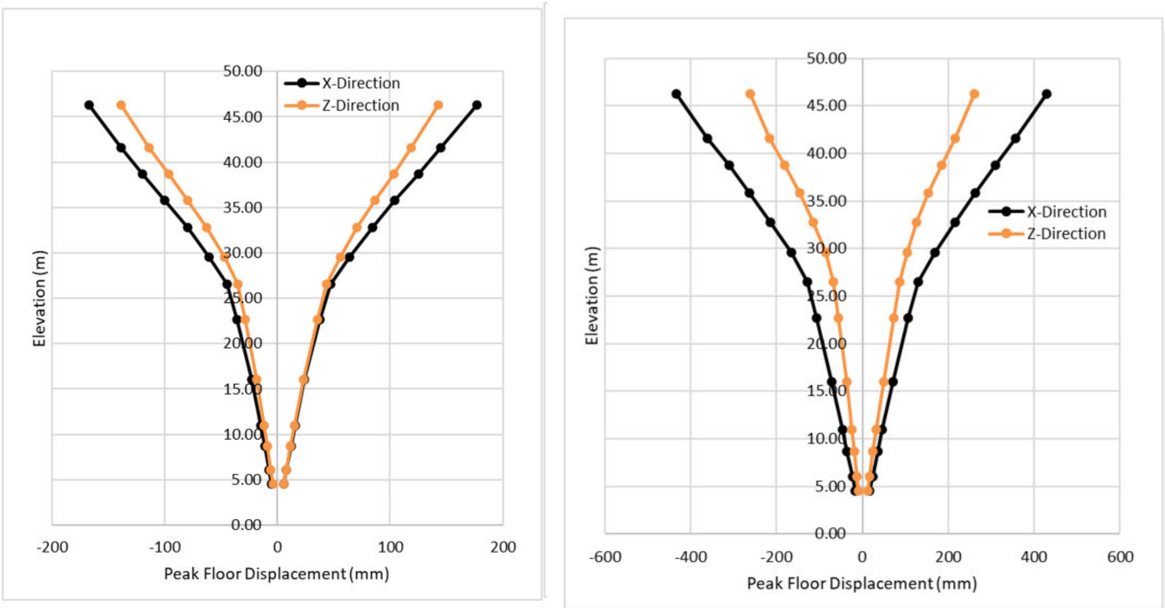

Figure 5: Seismic Deflection Profile - Tower (ULS -left; CALS - right)

6.3 Seismic Gaps

The requirement for a seismic gap between the base of the tower and the external stairs either side has been removed following input from the geotechnical engineer. Instead, a layer of soil between the stair and the tower is to be replaced with a compressible material to alleviate local pressures on the tower during rocking.

The existing drawings outline a seismic gap of 4 in (102 mm) between the Carillon Tower and the Hall of Memories. Dunning Thornton's Detailed Seismic Assessment Report (2020) states that the expected displacement of the Hall of Memories at 100%NBS (IL3) is less then 10 mm. Analysis of the Carillon Tower shows a deflection of <50 mm at the height of the Hall of Memories at 100%NBS (IL3). Hence, no pounding is expected to occur between the two structures.

Holmes understands that the Hall of Memories has undergone seismic strengthening. It is assumed that the failure threshold of the Hall of Memories is similar to that of the tower, and that the Hall of Memories does not impact the Tower's performance.

7 DURABILITY

This section sets out the basis of compliance in accordance with Clause B2 (Durability) of the Building Code for structural elements designed by Holmes NZ LP.

7.1 Design Life for Durability

The design lives for durability for the project are summarised in Table 7-1.

Table 7-1 Project durability design lives

Building Component	Design Life (years)
Foundations	50
Superstructure	50
Cladding	15

7.2 Site Location

Refer to Figure 7-1 for site location.

Figure 7-1: Site location shown in blue

7.3 Timber Elements

Means of Compliance:	NZBC- Clause B2: B2/AS1 Sub-clause 3.2
Referred/cited Standard:	NZS 3602: Part1 as modified by paragraph 3.2.2 of B2/AS1 NZS 3604: As modified by paragraph 3.2.1(a) & (b) of B2/AS1 NZS 3640: As modified by paragraph 3.2.3 of B2/AS1
Design Life:	50 years

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Refer to Table 7-2 for specified timber and timber treatment.

Table 7-2 Specified timber and timber treatment (refer to Table 1A of B2/AS1)

Timber Components	Ref No. (Table 1A B2/AS1)	Species	Level of Treatment
Wall studs and nogs, wall plates and blocking, interior flooring	1C.1, 1C.2, 1E.7	Radiata Pine	H1.2
Internals walls, ceiling framing	1.E.5	Radiata Pine	H1.2

7.4 Existing concrete elements

A corrosion expert has been engaged to provided long term durability advice for the 100 year old reinforced concrete walls. A cathodic protection system is being considered for the existing elements.

7.5 Structural Steel Elements

Means of Compliance: NZBC - Clause B2: B2/AS1 Sub-clause 3.6

Referred/cited Standard: SNZ TS 3404: Durability requirements for steel structures and components

Design Life: 50 years

A corrosion expert has been engaged to advise on corrosion protection to steel members; this is outside of Holmes' scope.

7.5.1 Minimum Time to First Major Maintenance for Protective Coatings

Minimum time to first major maintenance for protective coatings is determined in accordance with NZBC - Clause B2: B2/AS1 Table 1 as summarised in Table 7- below.

Table 7-)) NZBC Clause B2/AS1 minimum time to first major maintenance for protective coating systems

Protective Coating System	Minimum Time to First Major Maintenance
Protective coating systems that are difficult to access or replace	40+ years
Protective coating systems that are easy to access or replace	15 years

SNZ TS 3404 Section 1.6 permits the use of protective coating systems with a shorter time to first major maintenance when used in conjunction with an inspection and maintenance programme, which together will meet the durability provisions of NZBC Clause B2.

7.5.2 Atmospheric Corrosivity Category

Determined in accordance with SNZ TS 3404 Section 2.2.1:

Macroclimate Corrosion Cat: C3 (refer SNZ TS 3404 Figure 4)

7.5.3 Surface-Specific Corrosivity Categories and Specified Minimum Life to First Maintenance

Specific steel coatings and corrosivity categories will be specified by the corrosion expert.

8 SOFTWARE

Table 8-1 summarises the computer applications used for the project.

Table 8-1 Computer applications used for the project.

Analysis Type	Software Used	Archive Files
Non Linear Time History Analysis	ANSR	\\holmes\data\HC\Projects\144373.13\Working Calculations - Phase 2\02 Analysis\ANSR\ANSR Models Carillon Tower_Lower Bell Frame_Strengthen_125PFC_152X76X6RHS_V9.msw 220601_Carillon Tower_Upper Bell Frame_V14.msw"
3D frame analysis (bell frames)	Microstran v10.1	
General spreadsheet design	HCG Design	
Steel to concrete connection design	Hilti Profis	

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9 CONSTRUCTION NOTES

9.1 Assumptions and investigations

Assumptions made during design and investigations required to confirm design items are recorded in the Assumption and Investigations Register. This will be provided to the Contractor.

9.2 Constructability

During the detailed design phase, a buildability review was conducted with the contractor. As a result of this review and subsequent discussions, the following items have been considered:

- Splice details are provided for the steel members to allow these to be brought in short segments. These splice details can be applied anywhere in the steel members, so the Contractor has flexibility to locate splices to suit. These were updated following contractor review to maximise bolting rather than welding.

- Typical details will be provided for known situations where existing structure clashes with the new strengthening. This will allow the Contractor to resolve clashes as they arise.

- Review of the impact of cutting existing RSJ beams was carried out due to the contractor raising concerns about the buildability of this detail. Up to 15 mm of the beam flange can be removed with no negative impact, and this has been communicated to the contractor.