Manatū Taonga Ministry of Culture & Heritage
National War Museum - Carillon Tower

State Highway 1, Te Aro, Wellington 6011

Concept Strengthening Report

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22 November 2022
144373.13

Holmes

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Report
National War Museum - Carillon Tower

Prepared For:
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Report Issue Register

DATE	REV. NO.	REASON FOR ISSUE
22/11/2022	1	Concept strengthening schemes

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1 EXECUTIVE SUMMARY

The Carillon Tower, located at the National War Memorial, Wellington, NZ, has been closed to the public since February 2020 following considerations of the building being Earthquake Prone in terms of the Building Act 2004. Consequently, the Manatū Taonga Ministry of Culture & Heritage seeks to strengthen the historic building to restore it to its full function and reopen it for public use.

Holmes has been engaged to workshop and recommend concept strengthening schemes to the Carillon Tower and its bell frames. Analyses and assessments were conducted to inform the seismic performance of the structure in terms of a percent New Building Standard (%NBS) rating: a seismic rating which is a measure of the expected seismic performance of a building from a life safety point of view, compared with the minimum required by the Building Code for new buildings.

Elements were identified in the lower and upper bell frames which may cause the Wellington City Council to consider the Carillon Tower to be an Earthquake-prone Building (EPB) in terms of the Building Act 2004. Furthermore, various elements across the bell frames and Carillon Tower were identified to classify the Carillon Tower as an Earthquake Risk Building (ERB) as per the New Zealand Society for Earthquake Engineering.

Concept strengthening schemes have been produced and are recommended for the Carillon Tower and the bell frames it houses.

2 INTRODUCTION

Built in the 1930's, the Carillon Tower is a 50m-high heritage-listed building located at the National War Memorial, Wellington, NZ. Its unique function of housing a carillon bell instrument sees the tower adopt a tube-like shape with increasing, carefully planned wall porosities over the increasing height of the building to optimise acoustic performance. The structural system of the building primarily consists of RC walls, with RC columns, braces, floors and beams forming the majority of the tower's remaining structure (all insitu). A 100 mm seismic gap separates the tower to the adjacent Hall of Memories, allowing the two structures to behave independently under seismic loading.

This short report is prepared for the Manatū Taonga Ministry of Culture and Heritage to outline the outcomes of Holmes' work associated with the investigation and production of Concept strengthening alternatives for the Carillon Tower.

3 SCOPE OF WORK

Holmes Consulting was engaged for the following scope of works:

- Understanding the building and project objectives
- Understanding the earthquake risk to the building and bell frames
- Understanding the earthquake performance of the building and bell frames
- Workshopping strengthening alternatives for the building and bell frames

The report summarises how this work was completed, on what basis, the key associated findings, and correspondingly proposed Concept strengthening alternatives.

4 LIMITATIONS

Findings presented as a part of this project are for the sole use of the client in its evaluation of the subject properties. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses.

The assessment has been conducted from best interpretation of the existing drawings and investigations/visits conducted on site. The extent of the assessment has been restricted to structural aspects of the Carillon Tower only. Non-structural elements including cladding, mechanical equipment, and service connections have not been inspected or reviewed.

Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

5 WORK COMPLETED

The work was conducted in four principal stages, described within this section.

5.1 Understanding the Building

The building was studied through a combination of existing documentation and new investigations (see Section 3). The original construction drawings of the structure and bell frames formed a basis for which site investigations provided confirmation. Significantly, the understanding of the tower's rocking behaviour and reinforcing layout was refined and the condition and details of the bell frames noted. Through the investigation processes, a detailed and complete picture of the tower was formed which allowed detailed analysis models to be constructed.

5.2 Construction of Analysis Models

Detailed analysis models were constructed for the tower, bell frames, and reinforced concrete floor. The models were updated and refined as the understanding of the tower developed.

5.2.1 Tower

The analysis model of the tower is built using ANSR, Holmes' in-house non-linear modelling software. The software comprises a non-linear analysis engine further developed with additional functionality and input and output processing. Functionality of the program includes the performance and processing of linear modal analyses, non-linear gravity analyses, non-linear static analyses (NSP or pushover) and non-linear dynamic analyses (NLTHA or NDP). These allow the seismic performance of the global structure and its individual components to be studied thoroughly. 3D renders of the analysis model are shown in Figure 1.

Figure 1: Analysis Model 3D Renderings (Carillon Tower)

The analysis model incorporated the primary structural elements of the tower, captured its rocking system, and specified appropriate upper and lower bound soil properties following advice from the geotechnical engineer (ENGEO). Dummy frames were included to model the effect of the bell frames on the structure. Separate detailed models were produced to study the bell frames and reinforced concrete floors explicitly.

5.2.2 Bell Frames

The analysis models of the bell frames were built using Microstran, a general-purpose structural analysis package. Two separate models were produced for the upper and lower bell frames. This allowed static analyses to be performed on the frames under derived seismic loads (see Section 2.4). 3D renders of the upper and lower bell frames are shown in Figure 2 and Figure 3 respectively.

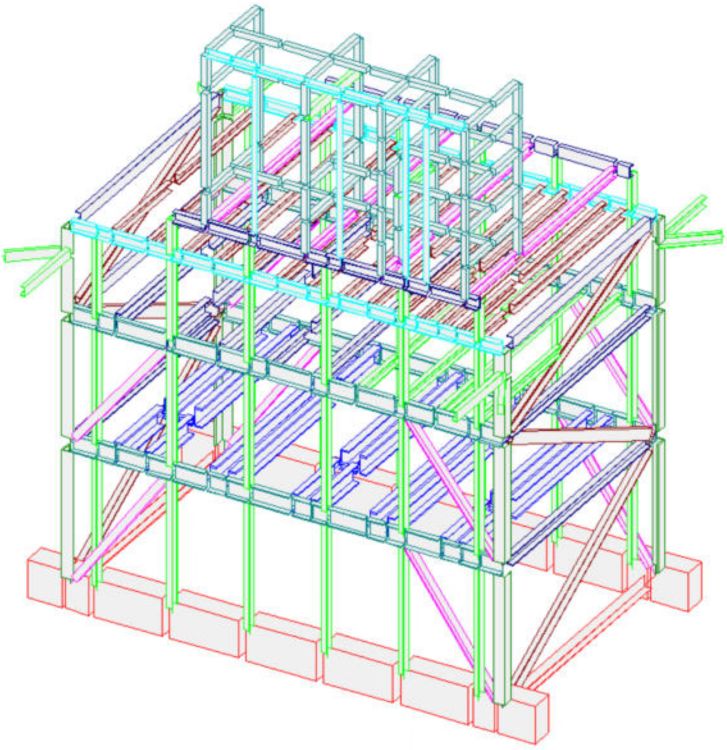


Figure 2: Analysis Model 3D Rendering - Upper Bell Frame

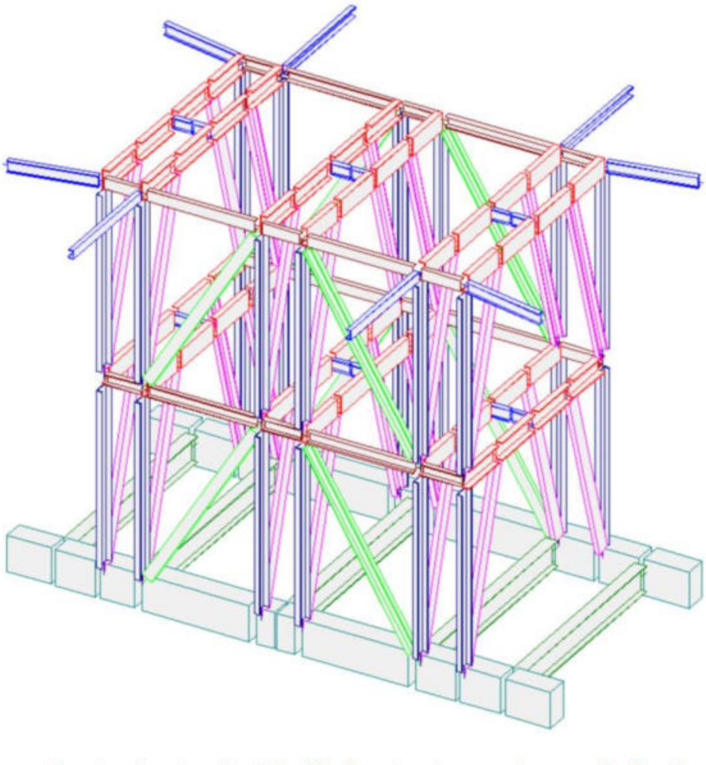


Figure 3: Analysis Model 3D Rendering - Lower Bell Frame

5.2.3 Reinforced Concrete Floors

The analysis models of the reinforced concrete floor diaphragms were also built using Microstran. Detailed grillage models were produced to study the likely in-plane performance of the floor under seismic loading. This was done because of the complex nature of the floor diaphragms due to the large central openings. A separate model was produced for each of the reinforced concrete floors (5 total). An example 3D render and load output of the L07 (UB) grillage model is shown in Figure 4 and Figure 5 respectively.

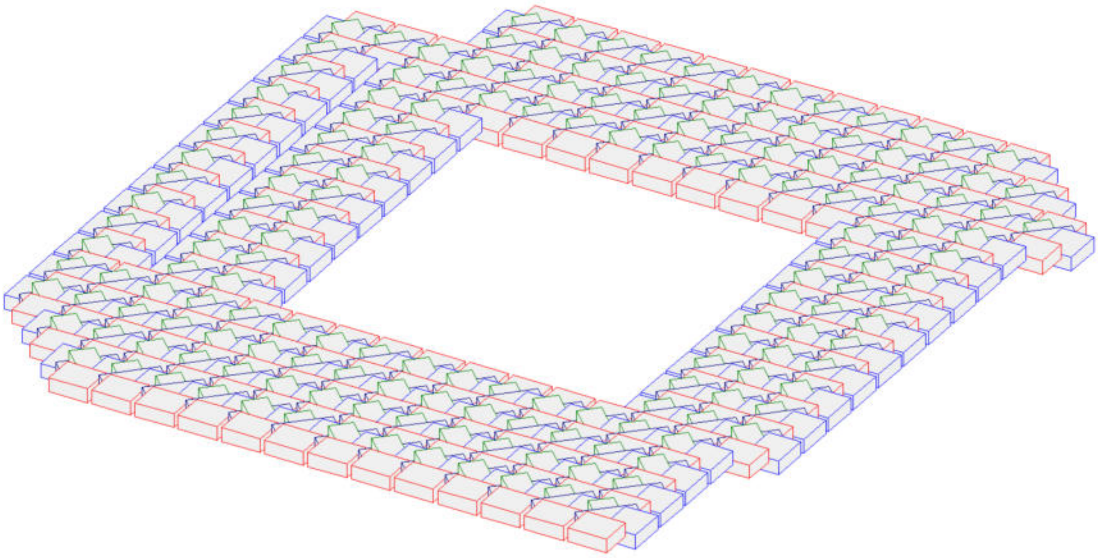


Figure 4: Analysis Model 3D Rendering - L07 (UB) Reinforced Concrete Floor

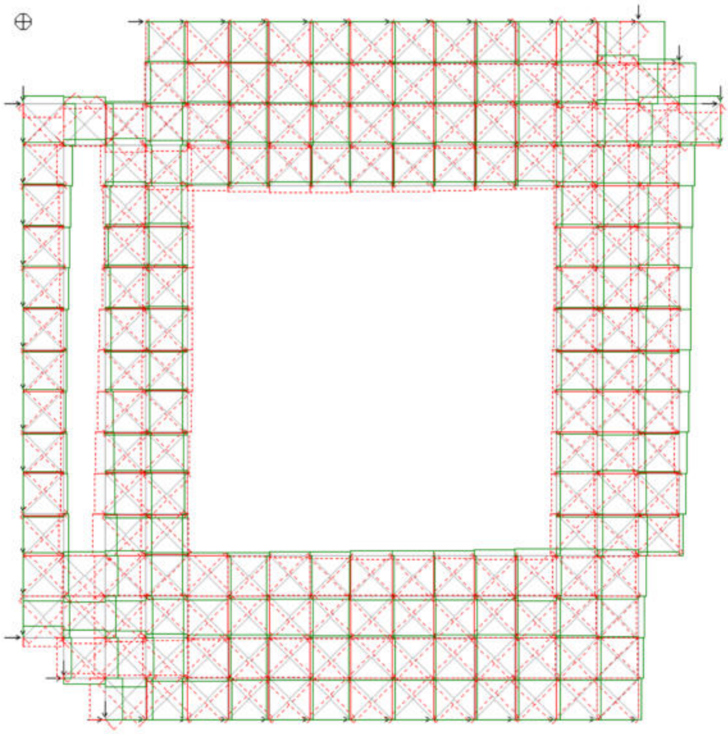


Figure 5: Analysis Model Loading - L07 (UB) Reinforced Concrete Floor

5.3 Seismic Hazard Determination

The seismic hazard determination was completed to obtain suitable seismic demands for the Carillon Tower, which was assumed to be an Importance Level 3 (IL3) building in accordance with AS/NZS 1170.0:2011. This was completed in accordance with the Seismic Assessment Guidelines (2017). Eleven earthquake ground motion records were selected and amplitude scaled in accordance with the importance level, dynamic properties and specific geographic and geological site conditions of the tower. This informed the loading imposed on the structure under its seismic analysis and assessment.

The parameters in Table 1 were used to generate the target spectrum for the tower. A Structural Performance factor of Sp = 1.0 was used in accordance with the Seismic Isolation Guidelines (2019) due to the rocking ability of the structure.

Table 1: Seismic parameters for target spectrum

Loading Standard	NZS 1170.5:2004
Hazard Factor, Z	0.4
Return Period Factor, R	1.3
Site Subsoil Classification	C
Source Distance, D	> 20km
S_p	1.0
ULS Target spectrum adjustment for S_p	1.0

Ground motion selection was based on a 1000 year return period. Records were scaled to the ULS target spectrum to define 100% ULS shaking.

5.4 Seismic Analysis and Assessment of the Building

The object of the seismic analysis and assessment is to quantify the seismic performance of the tower in terms of a Percent New Building Standard (%NBS) rating. The %NBS rating provides a seismic rating which is a measure of the expected seismic performance of a building from a life safety point of view, compared with the minimum required by the Building Code for new buildings.

In compliance with Section C1.6.2 the Seismic Assessment Guidelines (2017), the tower assessment included intensities of earthquake loading corresponding to the two following building performance limit states:

1. Ultimate limit state (ULS): elements which failure would constitute a significant human life safety hazard

2. Collapse avoidance limit state (CALS): elements which failure would likely cause building collapse (assessed at 1.8 x ULS intensity of shaking)

In accordance with the seismic hazard, 110 total earthquake simulations were produced against which the tower was analysed and assessed through non-linear time history analysis (NLTHA) (CALS and ULS intensities, 11 ground motion records, 5 mass applications). This was done in an effort to capture the likely building performance under a full range of earthquake durations and intensities the tower might face during its design life.

The tower was subjected to various iterations of the NLTHAs at various levels of scaling of the earthquake intensities. Throughout, the performance and behaviour of the global building as well as individual structural components of interest were studied carefully and evaluated against the acceptance criteria detailed in the Seismic Assessment Guidelines (2017). This enabled local and global failure mechanisms and hierarchies to be understood. Appropriate corresponding %NBS ratings were formed and suitable concept strengthening schemes produced targeting the unique behaviour of the structure.

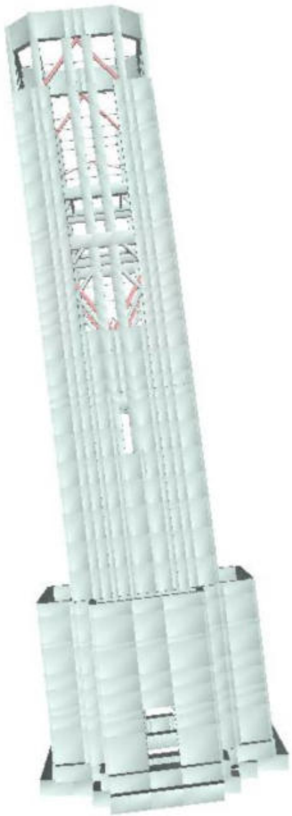


Figure 6: 3D Rendering of Analysis Model Rocking under NLTHA Loading (Carillon Tower)

Design loads on the RC floors and bell frames were obtained from the global building response as a refined load demand. These were applied to the corresponding models as outlined in Section 2.2.

6 BASIS OF ASSESSMENT

The assessment was based on a mixture of gathered documentation and site visits.

Gathered documentation included:

- Original construction drawings and specification (tower and bell frames)
- Geotechnical advice gathered through liaison with geotechnical engineer (ENGEO)
- Tower weight information details provided by the Ministry of Culture and Heritage (bells, cladding, other significant loads)

Site visit investigations included:

- Scanning of reinforcing within reinforced concrete walls
- Intrusive foundation investigations
- General site observations

7 RESULTS SUMMARY

General summary

A summary of the building performance is reported in terms of a percent New Building Standard (%NBS). The earthquake ratings assume the Carillon Tower to be an Importance Level 3 (IL3) building in accordance with AS/NZS 1170.0:2011.

A building with an earthquake rating less than 34%NBS fulfils one of the requirements for the Territorial Authority (Wellington City Council) to consider it to be an Earthquake-prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67%NBS is considered as an Earthquake Risk Building (ERB) by the New Zealand Society for Earthquake Engineering.

The Carillon Tower, as is, is categorised as an Earthquake Risk Building (ERB) and meets one of the criteria that could categorise it as an Earthquake-prone Building due to the low performance of the bell frame structures.

Tower

A summary of the structural weaknesses identified in the tower is provided in Table 2 in order of expected failure hierarchy. A summary of the principal components verified to perform at >100%NBS is provided in Table 3.

Notably, the building appears to be subjected to deformation incompatibilities: a phenomenon which occurs when tied together structural elements want to deform differently due to the different stiffness properties of the structural system. This drives large local forces through the structure. Locations were this is primarily noted is highlighted in Figure 7. Various described structural weaknesses are found in these highlighted areas.

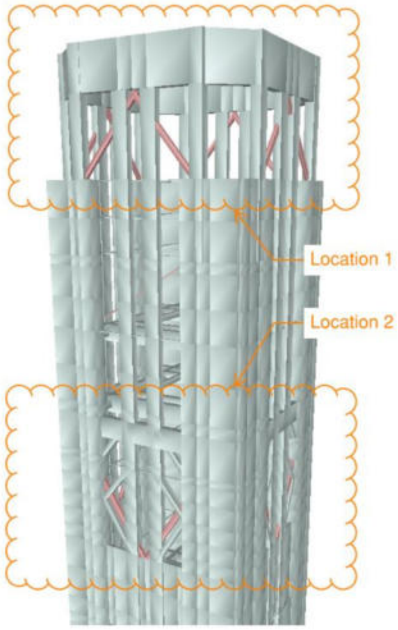
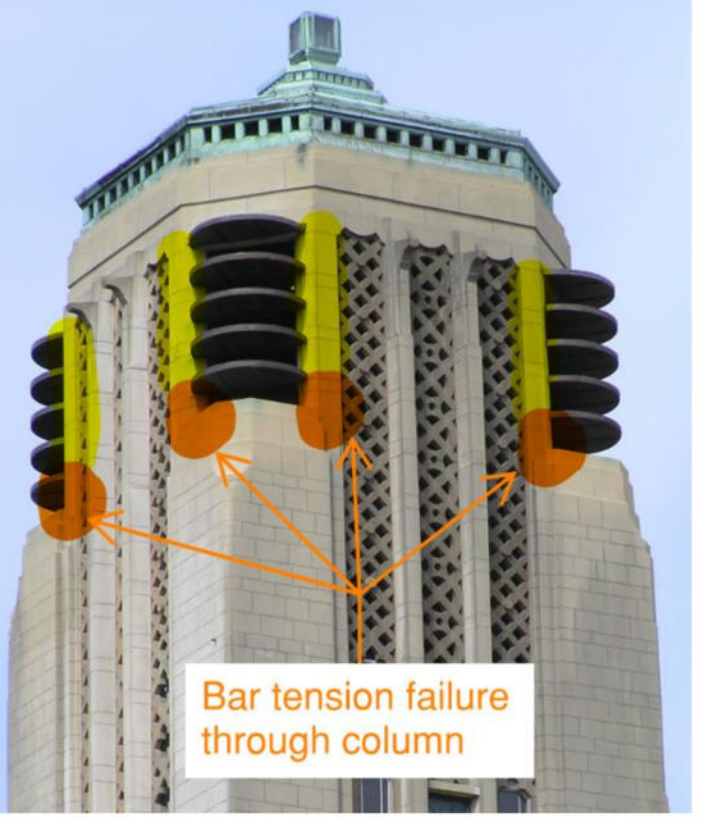
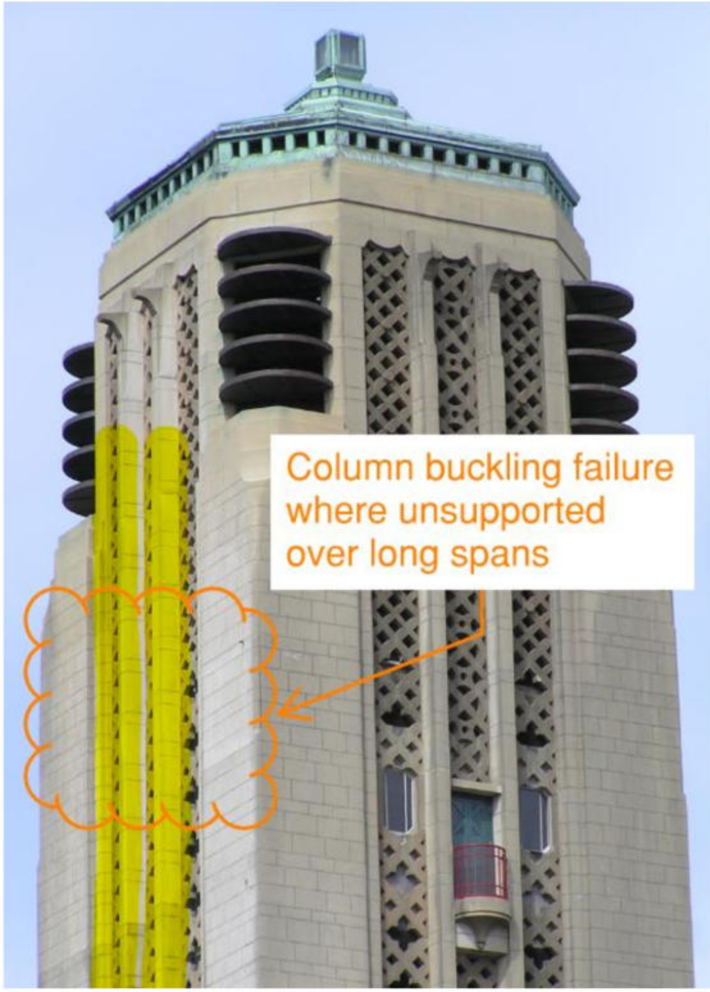
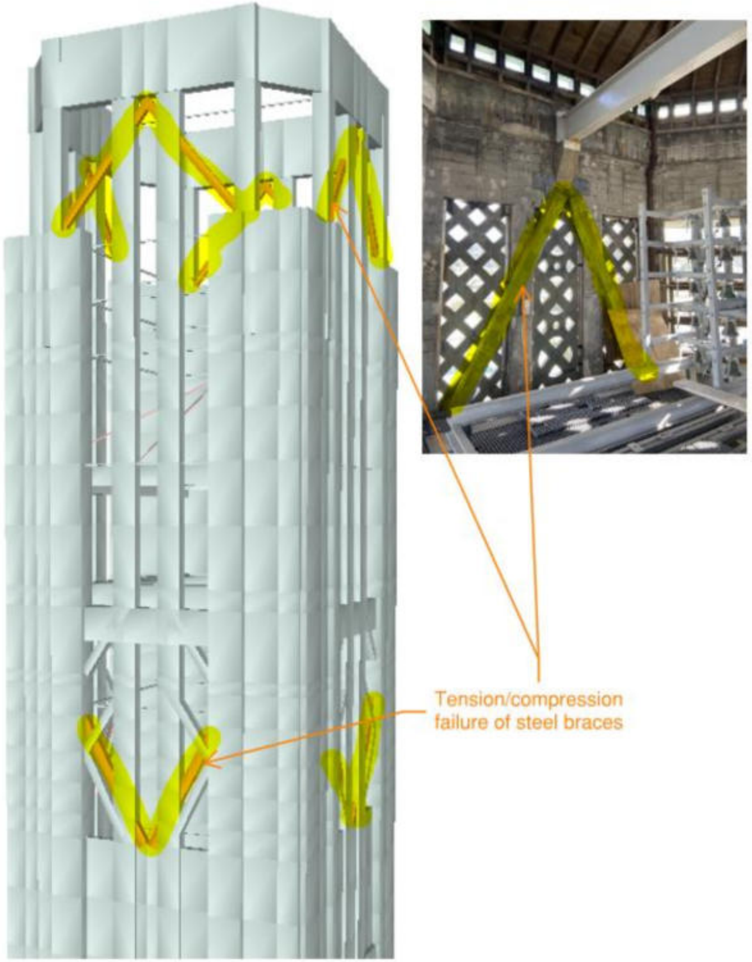


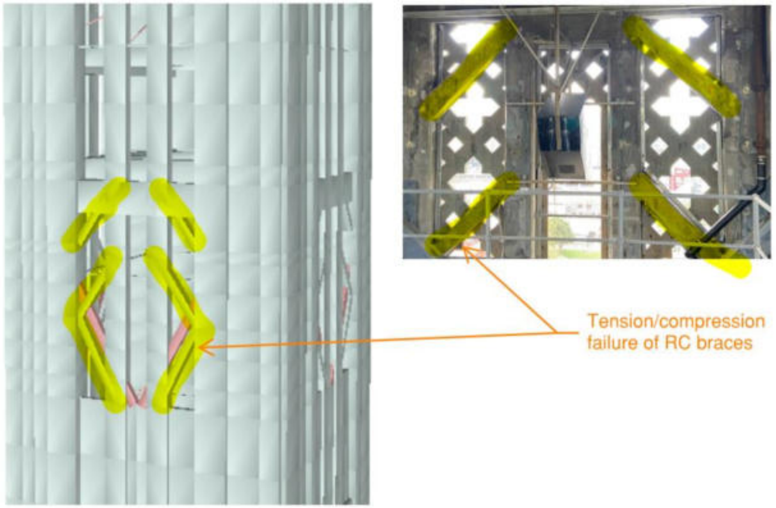
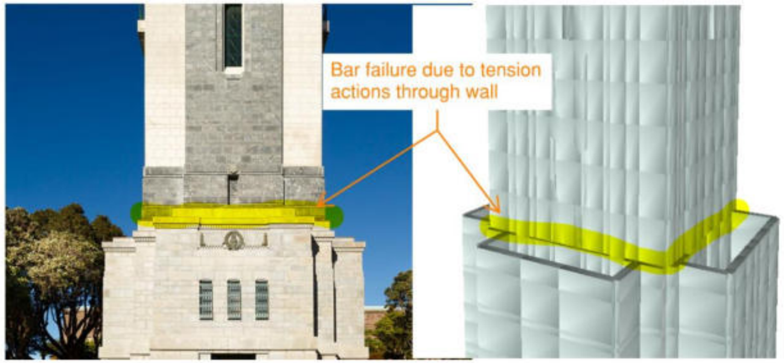
Figure 7: Locations subjected to deformation incompatibility issues (Carillon Tower)

Table 2: Structural Weaknesses of Tower

Element	%NBS	Description of Weakness
Level 7b- L8 reinforced concrete corner columns	NBS rating: NBS = 34% (IL3)	Bar failure under tension action on column. 

Reinforced concrete column at locations of long unsupported spans	NBS rating: NBS = 35% (IL3)	Buckling failure (combination of compression and bending). Column unsupported laterally over long spans. 

Top (200PFC) and bottom (250PFC) steel braces	NBS rating: NBS = 40% (bottom braces), 60% (top braces) (IL3)	Tension/compression failure of steel braces. 

Reinforced concrete braces, level 5a-6	NBS rating: NBS 45% (IL3) (governed by upper brace under tension)	Tension/compression failure of reinforced concrete braces. 
Reinforce concrete wall interface at level 3	NBS rating: 50-60% (IL3)	Bar failure due to flexural actions through wall system. At the highlighted location, the wall experiences a sharp 'kink' in flexural demand due to the localised geometry change of the tower. 

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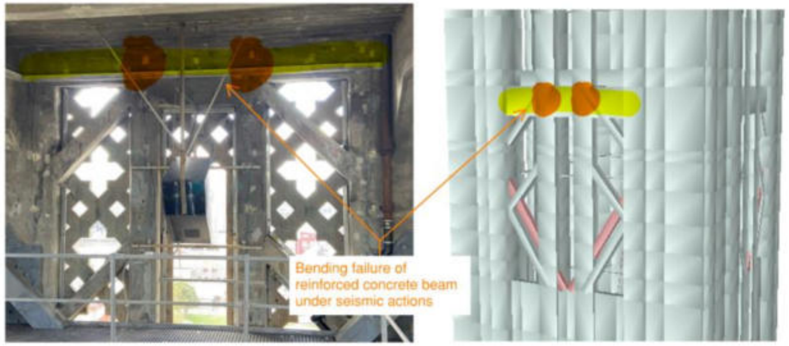
Level 7b- L8 reinforced concrete centre columns	NBS rating: NBS 70% (IL3)	Bar failure under tension action on column. 
Reinforced concrete edge beam, level 6	NBS rating: 60-70% (IL3)	Bending failure of reinforced concrete beam under seismic actions. 

Table 3: Elements of Tower Verified to be Performing at >100%NBS

Element	%NBS
Reinforced concrete floor diaphragms	>100%
Reinforced concrete floor-supporting beams	>100%
Reinforced concrete walls in structure	>100%
Foundations (soil bearing capacity)	>100%

Bell frames

An explicit assessment was conducted for both the lower and upper bell frame. This included the structural integrity of the steel frames to the point at which its seismic load is delivered into the wider tower. The assessment identified several structural weaknesses in the lower and upper bell frames. These are summarised in Figure 8 and Figure 9 respectively below, where members and connections rated below 100% NBS are highlighted.

The lower and upper bell frame were both found to contain members critical to their global structural integrity which perform at levels <34% NBS. This categorises the global performance of both frames as <34% NBS.

The seismic ratings did not consider the potential local weaking of connections or members due to corrosion as it was not expected to impact the global %NBS rating of the frames. Corrosion was found to be significant within the structure. Under future strengthening of the frames, the condition of the steel along with corresponding repair or replacement is recommended to be considered.

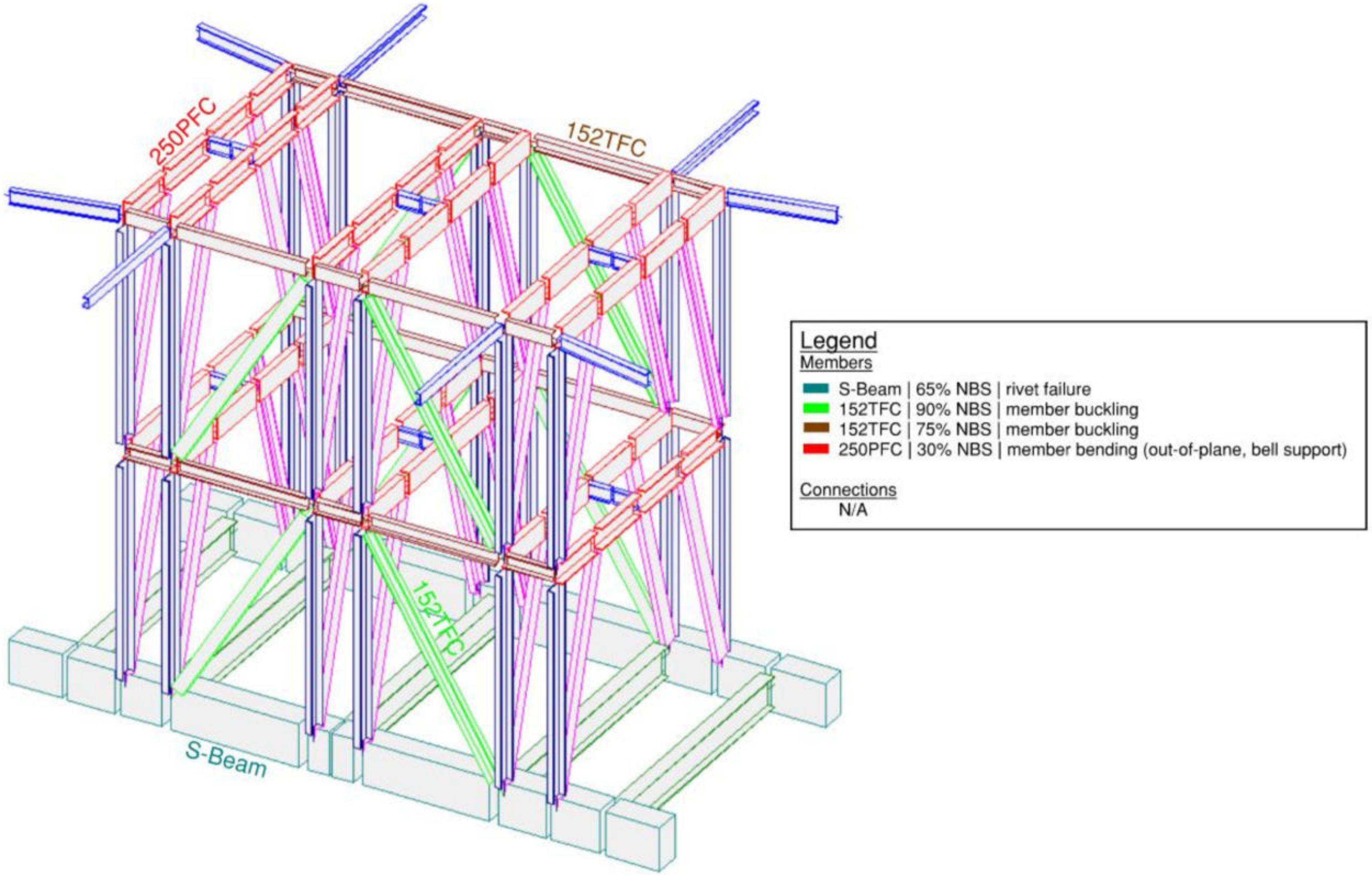


Figure 8: Lower Bell Frame - NBS Performance Summary

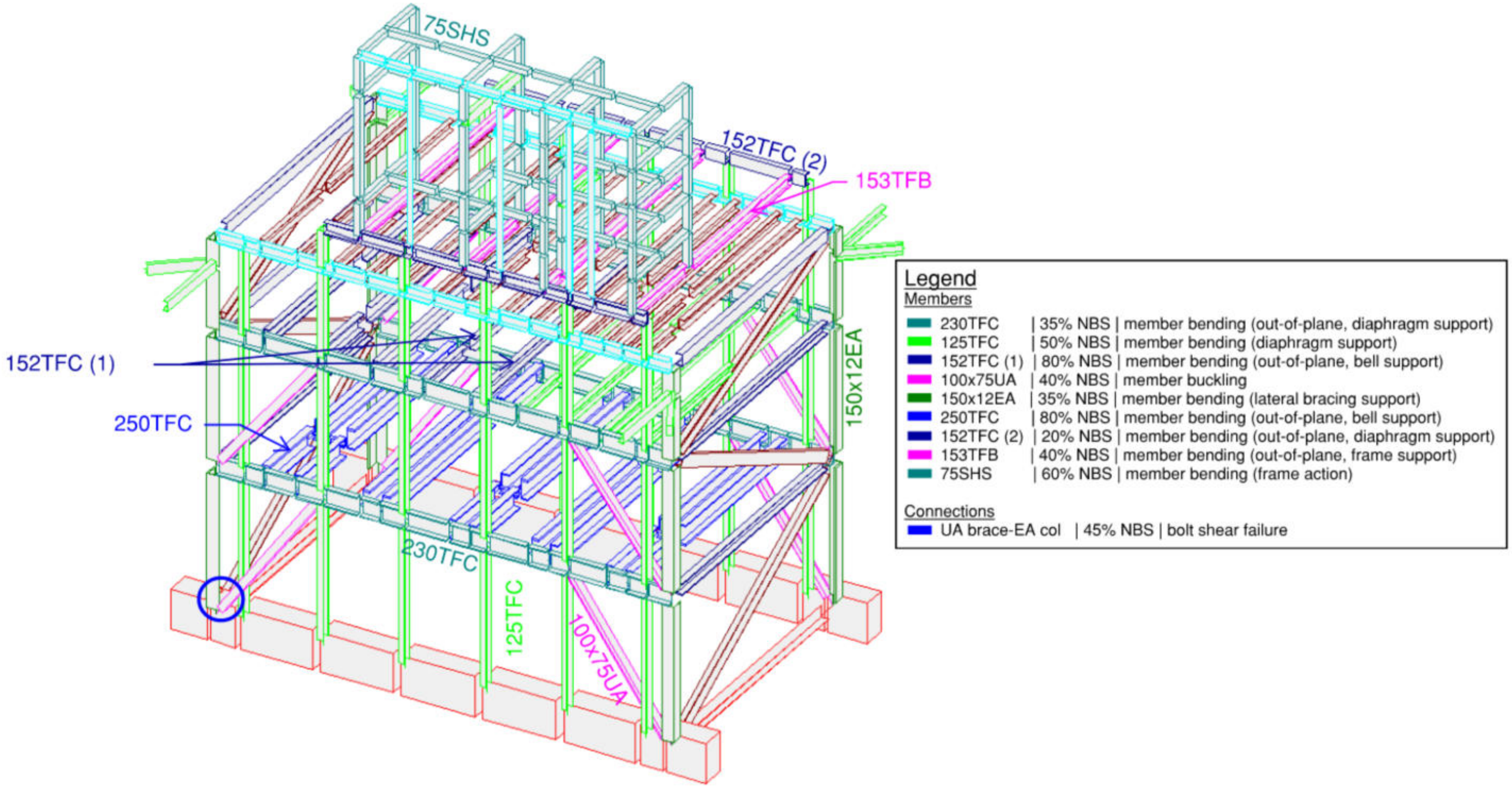


Figure 9: Upper Bell Frame - NBS Performance Summary

8 RECOMMENDED CONCEPT STRENGTHENING SCHEMES

Concept strengthening schemes have been produced and are recommended under the consideration of various factors. Through liaison with Manatu Taonga Ministry for Culture & Heritage (MCH, client) and Studio Pacific Architecture (heritage architects), common interests and targeted outcomes for the strengthening of Carillon Tower were established. Primarily, the following factors were considered:

1. To restore the Carillon Tower to its full ceremonial/commemorative function on completion of the project.

   A final seismic rating as close as possible to 100%NBS is sought on which basis the concept strengthening schemes are recommended.

2. The Carillon Bell Tower is strengthened to a resilience level where the MCH is comfortable to open the building for public use, and the heritage values are retained.

   Low-damage strengthening alternatives were considered which seek to minimise the life safety hazard, provide alternative load paths to increase structural resilience, and minimise the structural damage caused by future earthquakes.

3. The design solution does not compromise the Carillon Bell Tower's heritage values and includes the carillon instrument being playable upon completion.

   The strengthening schemes seek to provide minimal disruption to the external appearance of the tower, as well as its wall openings and internal layout in a way that may affect the functionality or playability of the instrument.

4. To deliver the project within the allocated budget.

   The strengthening schemes recognise potential budget constraints and hence provide a range of options at various levels of expected costs. Costs associated with the general maintenance and seismic repair of the alternatives were considered, specifically in the face of the durability issues of steel corrosion. The strengthening alternatives are targeted to provide support at the weakest locations of the tower and seek to unify with (as opposed to fight) the natural deformation of the structure. This is done to produce high value and efficient strengthening with minimised required structural repair costs due to future seismic events.

8.1 Bell Frames

The assessment of the bell frames identified members, critical to their global structural integrity, with a capacity <34% NBS. This may cause the Territorial Authority to consider the tower to be an Earthquake-prone Building (EPB) in terms of the Building Act 2004 due to the associated life safety risks.

Local strengthening applied to the bell frame is expected to suffice in terms of increasing the global rating of the bell frames to ≥34%NBS. Corresponding strengthening schemes which will achieve this are not outlined in this memorandum and may be requested separately.

Two alternative concept strengthening schemes are proposed for the upper and lower bell frame (appended to report): one that provides strengthening of the existing frame and another which considers the full replacement of it. Both strengthening alternatives target a 100%NBS rating (IL3).

8.2 Tower

Four alternative Concept strengthening schemes are proposed for the tower (appended to report). It should be noted that these provide conceptual solutions only. The final details are subject to further development through detailed analysis and design throughout the following stages of the project. All strengthening alternatives target, as far as practicable, a 100%NBS rating (IL3).

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Memorandum

To:	[redacted]
Company:	Manatū Taonga - Ministry for Culture and Heritage
From:	[redacted]
Date	7 December 2022 Project No: 144373.13
Subject:	National War Museum - Carillon Tower – Building Performance and Seismic Risk Summary

This memorandum provides a brief summary of the seismic performance and relative earthquake risk of the Carillon Tower and its upper and lower bell frames.

Limitations

Findings presented as a part of this project are issued pursuant to our contract with Manatū Taonga - Ministry for Culture and Heritage dated 22/03/2022 and for the sole use of Manatū Taonga - Ministry for Culture and Heritage in its evaluation of the subject property. The findings are not intended for use by other parties and Holmes NZ assumes no liability to any party other than Manatū Taonga - Ministry for Culture and Heritage.

Assessed seismic ratings

Carillon Tower

The results of the seismic assessment of the tower indicate the building's seismic rating to be 34%NBS (IL3), governed by the upper reinforced concrete columns within the tower (base of level 7b, upper catwalk). The columns undergo a ductile failure mechanism in which the reinforcing bars within the columns yield under excessive tensile action. At early onset failure, seismic load is expected to be spread between the different reinforced concrete columns and steel bracing, which adds resilience to the load path of the top structure (level 7b and above). Failure of the described combined system may cause collapse of the top structure, which could present a significant life safety hazard to people in and around the building. However, this failure is localised to the top of the structure and is not expected to cause global failure of the tower in a seismic event.

Bell frames

The results of the seismic assessment of the upper and lower bell frames indicate their seismic rating to be 20%NBS (IL3) and 30%NBS (IL3) respectively, governed by local beam member failure within the frame. The failure mechanism of these steel members is ductile and occurs through excessive out-of-plane bending. Failure of these elements is expected to cause local instabilities within the bell frame. Fixity of the bells to the frame is expected to be maintained up to seismic levels >34%NBS (IL3).

Relative earthquake risk compared with a new building

The building and bell frames are classified as a Grade D building following the NZSEE grading scheme (NZSEE, 2006), as shown in Table 1**Error! Reference source not found.**. Grade D buildings represent a risk to occupants 10-25 times that expected for a new building, indicating a high risk exposure relative to a new building if a large earthquake occurs.

Table 1: Grading system for earthquake risks relative to a new building

Percentage of New Building Standard (%NBS)	Grade	Approx. risk relative to a new building	Life-safety risk description
>100	A+	Less than or comparable to	Low risk
80-100	A	1-2 times greater	Low risk
67-79	B	2-5 times greater	Low to Medium risk
34-66	C	5-10 times greater	Medium risk
20-33	**D**	**10-25 times greater**	**High risk**
<20	E	25 times greater	Very high risk

Should the critical bell frame elements, rated <34%NBS (IL3), be strengthened to bring the building above 34%NBS, the risk would be reduced to 5-10 times that expected for a new building (Grade C, medium risk) governed by the failure of the upper columns as discussed above.

We are happy to discuss the results of this assessment or further risk items at any time.

Regards,

[redacted]

[redacted]

STRUCTURAL ENGINEER

Holmes NZ LP

1 REFERENCES

NZSEE, 2006. *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, Corrigendum N° 3.*