Hutt Hospital – Detailed Seismic Assessments

Clock Tower Building DSA

Hutt Valley District Health Board

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Executive Summary

Scope and Basis of Assessment

Aurecon has been engaged by Te Whatu Ora – Health New Zealand, to provide a Detailed Seismic Assessment (DSA) of the Clocktower Buildings located at Hutt Hospital, Lower Hutt. The Clocktower buildings consist of three seismically separate, three-storey reinforced concrete structures from the 1940s, known as the **East Wing**, **West Wing**, and **Central Wing**.

The DSA was generally completed in accordance *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (**Red Book**), including the *updated Section C5 – Concrete Buildings – Proposed Revision to the Engineering Assessment Guidelines*, dated November 2018 (the **Yellow Chapter**). These are collectively noted as the **Guidelines**.

The Building is considered to be an **Importance Level 2 (IL2)** structure, located on a **Site Subsoil Class D** site as defined by NZS 1170.5:2004.

Results Summary

The Clocktower Buildings achieves an overall seismic capacity rating of **20-30*%NBS*(IL2)** in accordance with the **Guidelines**. This is based on the Critical Structural Weakness (CSW) of the steel roof cross bracing connections, RC Shear walls, RC diaphragm and foundations in all the buildings. Due to the limited information available, a %NBS score range is provided instead of a single score. This range reflects the uncertainty associated with the information used.

This classifies the building as **Class D** to the New Zealand Society of Earthquake Engineering (NZSEE) rating system. This may classify the building as earthquake prone to the New Zealand Building Act, subject to the Territorial Authority. A Grade D building imposes a risk 10-25 times greater than a new building.

Although these Buildings have a low seismic rating in accordance with the **Guidelines**, it is worth noting that these buildings are considered regular, has many wall elements and is well-tied together with a concrete insitu diaphragm. Buildings that contain these characteristics typically perform “better” in large earthquake shaking when compared to other irregular structures.

A peer review of the assessment has been undertaken by GDC Consultants, with high level inputs by Kestrel Group, following the draft issue of this report. The assessment has been updated as a result of this process.

The Table below presents a summary of the assessment findings.

Table: Detailed Seismic Assessment Summary Table

Building	The Clocktower Buildings consist of three seismically separate buildings known as: ▪ East Wing Building ▪ West Wing Building ▪ Central Wing Building
Storeys:	3 storeys (including a basement in the Central Wing)
Year of Design (approx.)	1940
Gross Floor Area (m^2)	▪ Central Wing is 780 m^2 ▪ East Wing 760 m^2 ▪ West Wing 760 m^2
Construction Type	Concrete with plain bar reinforcement and timber framed roof structure
Assessment Type	Detailed Seismic Assessment (DSA)
Date Building Inspected	Not Applicable
Importance Level	IL2

Structural Assessment Summary	Displacement based and force-based assessment in accordance with the **Guidelines**.
Stairs	The stairs appear to be cast into the surrounding concrete walls with no movement joints provided.
Current *%NBS* estimate	**20-30*%NBS*(IL2)** based on the rating of steel roof cross bracing connections, RC Shear walls, RC diaphragm and foundations.
List specific Structural Weaknesses, Severe Structural Weaknesses, and Life Safety Hazards	Structural Weaknesses: ▪ Steel roof braces and connections ▪ RC Shear walls ▪ RC diaphragm ▪ Foundations Severe Structural Weaknesses: ▪ No SSWs for this building
Conclusions & Recommendations	We recommend the building is seismically retrofitted to a minimum rating of **67*%NBS* (IL2).** The seismic retrofit would include: ▪ Increase the vertical lateral resisting capacity by installing new RC walls or steel braces. ▪ Increase the foundations capacity by installing new RC jackets around the existing foundations/add new piles. ▪ Increase the diaphragms tension capacity by recessing steel plates into the concrete floor. ▪ Increase the roofs lateral capacity by installing additional braces and struts. Note, the above recommendations are based on the available existing information, which is limited. Investigative works may be required during future strengthening design stages to confirm the design.

Contents

1 Introduction

1.1 Background

Aurecon has been engaged by Te Whatu Ora – Health New Zealand, to provide a Detailed Seismic Assessment (DSA) of the Clocktower Buildings located at 638 High Street, Boulcott, Lower Hutt 5010, New Zealand. Refer to Figure 1-1 for a plan view of the Hutt Hospital campus showing the location of the three Clock Tower wings.

The DSA focuses on life safety issues as the primary objective. This means that the earthquake scores or rating is based primarily on life safety considerations rather than damage to the building or its contents unless this might lead to damage to adjacent property. The earthquake rating assigned is, therefore, not reflective of serviceability performance.

Figure 1-1 Plan view of the Hutt Hospital campus showing the location of the three Clock Tower wings

1.2 Terminology and Key Definitions

See below for key terminology and key definitions as defined by the **Red Book**. Refer to **Appendix A** for additional definitions.

- ***%NBS (New Building Standard):*** The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
- **Design level/ULS earthquake:** Design level earthquake or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004
- **Ductile/ductility**: Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake

- **Structural weakness (SW):** An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS*.
- **Critical structural weakness (CSW):** The lowest scoring structural weakness determined from a DSA.

1.3 Building Description

The Clocktower buildings consist of three seismically separate, three-story reinforced concrete structures from the 1940s, known as the **East Wing**, **West Wing**, and **Central Wing**. All the concrete elements contain plain round bars.

Seismic resistance for the Clocktower buildings is provided by reinforced concrete (RC) walls in both the longitudinal and transverse directions. The floors consist of an insitu RC slab, which acts as a diaphragm to distribute the seismic lateral load to the RC walls. Refer to Figure 1-2 for the building elevations and

Figure 1-3 for a plan view of the buildings showing the RC walls and seismic gaps between the buildings.

At the roof level, steel strap cross bracing is employed to distribute the lateral load from the roof to the exterior concrete walls. The timber roof trusses span in the transverse direction and are supported by the exterior concrete walls and concrete walls/lintels along the corridor.

The buildings sit on a shallow RC strip foundation.

Alterations were carried out in the East and West wings during the 1980s, involving the demolition of some concrete walls. In 2015, lightweight metal cladding replaced the existing roof tiles on all three wings, with steel strap bracing installed beneath the new cladding.

The building underwent a previous assessment in 2008, based on limited information, resulting in a seismic rating of **34*%NBS*(IL2).** In 2021, the building was assessed according to the latest guidelines (2017) and received a seismic rating between **20%-30*%NBS*(IL2*).*** No structural drawings or original construction information were available for the Clocktower at the time of assessment.

In early 2022, demolition works were conducted in the West Wing at Level 2 for an internal fit-out. These works uncovered information about the concrete wall construction and confirmed several assumptions made in the previous assessment.

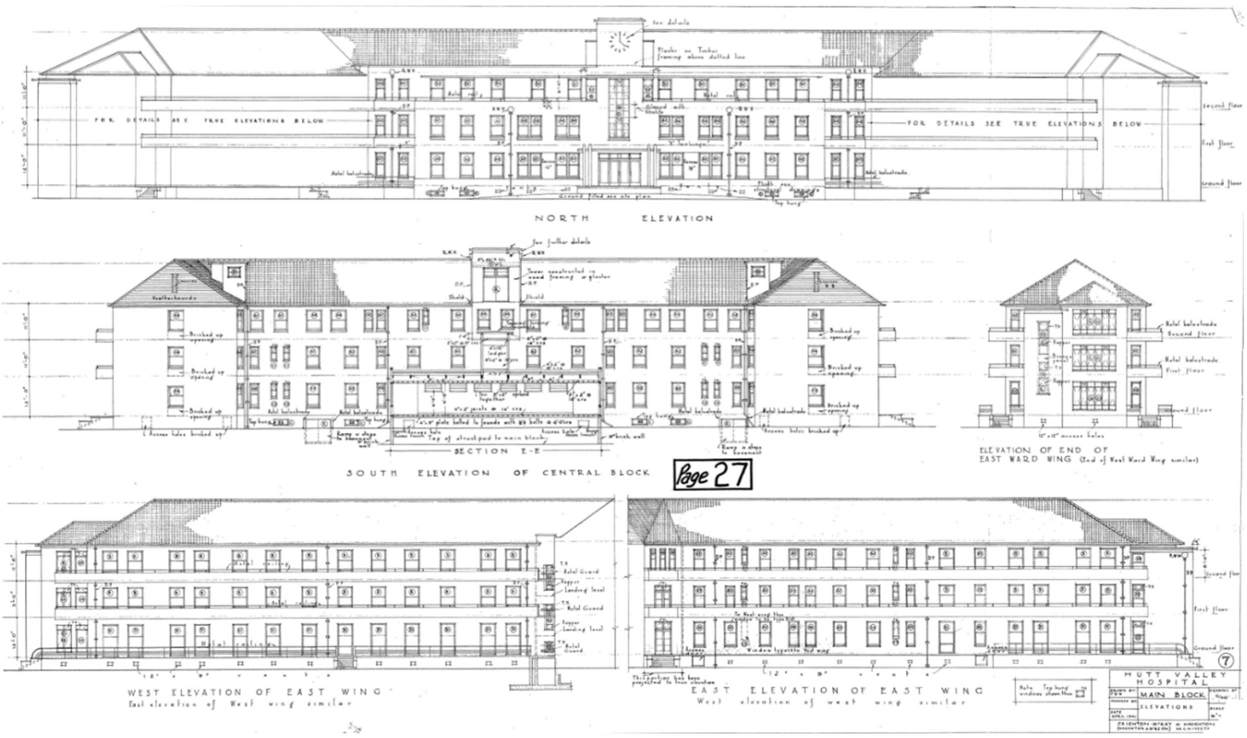

Figure 1-2 Building elevations

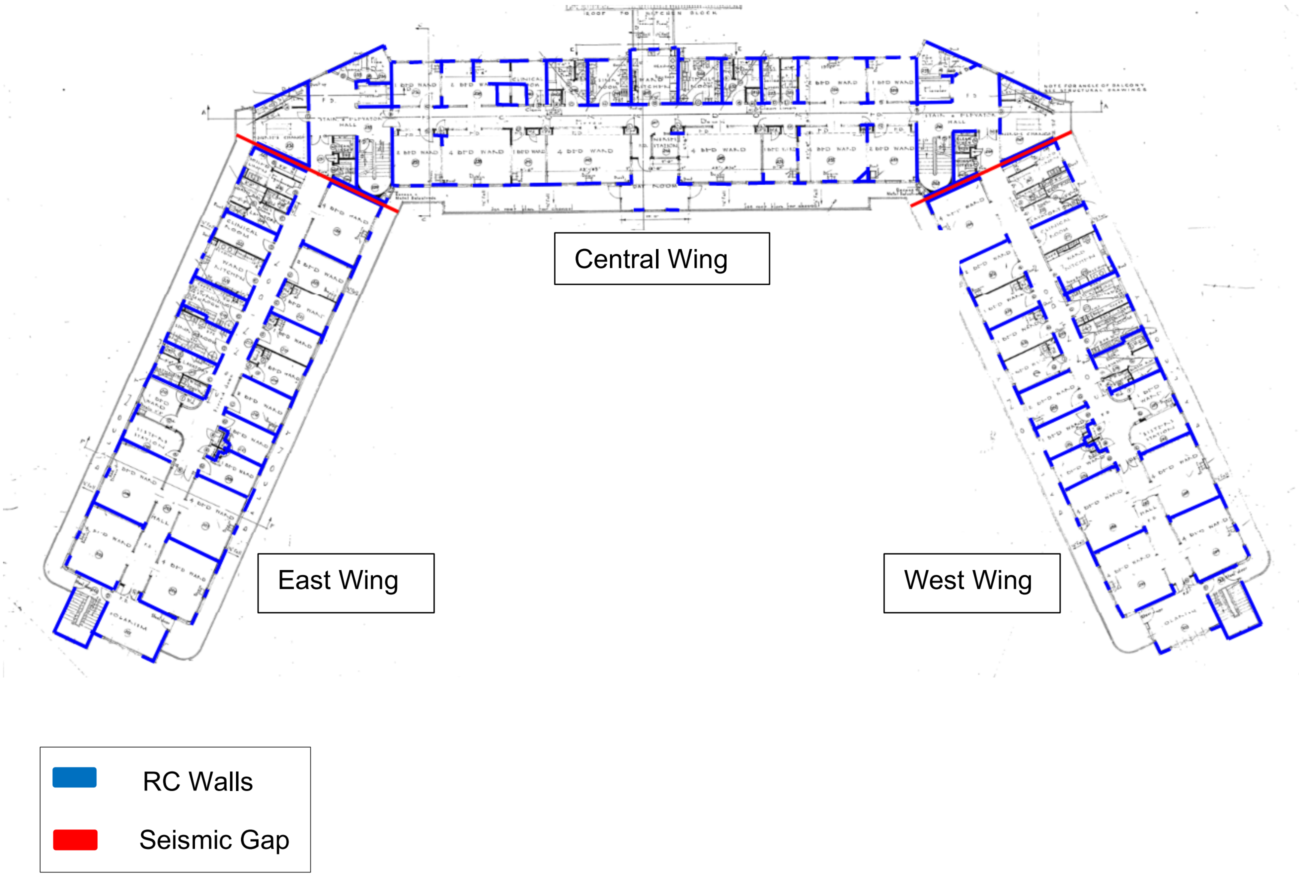

Figure 1-3 Plan View: Typical floor plan showing the RC walls and seismic gaps

1.4 On-site Demolition Works

An internal fit-out in the West Wing at Level 2 began in early 2022. Part of the works involved demolishing a selection of concrete walls at Level 2. This exposed the reinforcement in the existing concrete walls and provided further information to confirm assumptions considered in the previous assessment. The internal fit-out in the West Wing will be referred to the demolition works herein. Refer to Figure 1-4 that shows a demolished wall example showing plain round bars with no hooks at the ends of the wall.

The assumption for the assessment is stated in Appendix C. Information that was not determined from the demolitions includes but not limited to, the following:

- Reinforcing in the concrete floors, floor thickness and floor openings
- Connection between the concrete floor and walls
- Reinforcing in the concrete stairs
- Concrete compressive strengths and steel reinforcing yield strengths

Figure 1-4 Demolished wall example showing plain round bars with no hooks at the ends of the wall

1.5 Previous Assessments

In 2008, SKM issued a report titled "*Hutt Hospital Campus Three Buildings: Detailed Evaluation of Earthquake Resistant Performance*." The report indicated that the building achieved a seismic rating of **34%*NBS*(IL2)** in accordance the then current guideline 2006 *NZSEE Assessment Guidelines.*

Due to the date of the assessment, the assessment was not completed in accordance with *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (commonly known as the "**Red Book**").

Today the Red Book provides mandatory technical guidelines for engineers to use when carrying out seismic assessments of potential earthquake-prone buildings when required by the Territorial Authority. They should also be used by engineers for all seismic assessments.

In 2018, a proposed technical revision to *Section C5 of the Engineering Assessment Guidelines* (referred to as the "**Yellow Chapter**") was released by the engineering sector to provide the latest engineering knowledge on aspects involved in the assessment of concrete buildings, and to reflect what engineers learned from the investigation into the partial collapse of Statistics House following the Kaikōura earthquake.

1.6 Basis of Assessment

1.6.1 General

The DSA was generally completed in accordance *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (**Red Book**), including the *updated Section C5 – Concrete Buildings – Proposed Revision to the Engineering Assessment Guidelines*, dated November 2018 (the **Yellow Chapter**). These are collectively noted as the **Guidelines**.

1.6.2 Importance Level

The structure has been assessed at an **Importance Level 2 (IL2)** and a design life of 50 years, in accordance with the New Zealand Building Code and as agreed with HVDHB.

The West Wing building currently serves functions for ear, nose, and throat (ENT) outpatients as well as audiology on the ground floor, plastics, maxillofacial and burns on the first floor and the proposed minor skin, plastics and ophthalmology procedures on the second floor. The current and proposed use of the Clock Tower does not include any emergency medical facilities, general anaesthesia facilities, resident patients, or post disaster functions.

The Hutt Hospital has designated IL3 and IL4 buildings as shown in Figure 1-5.

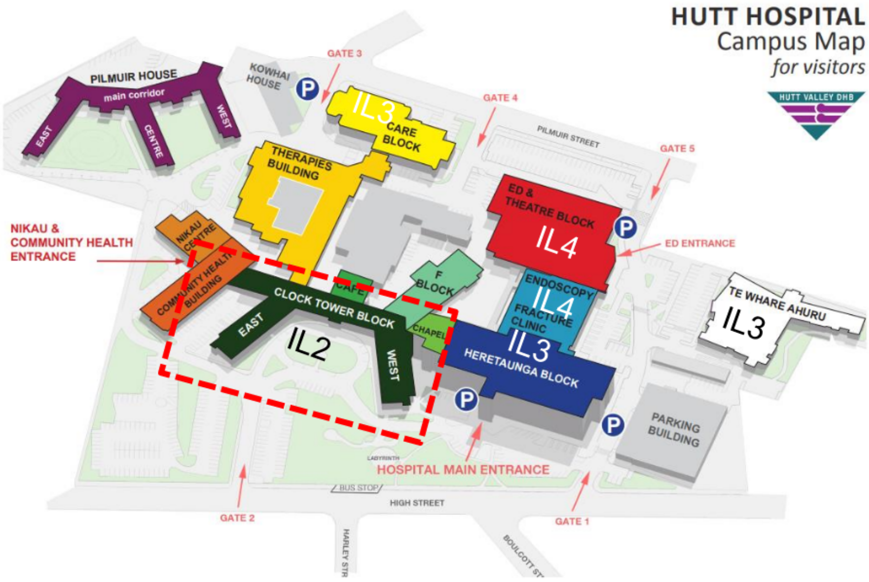

Figure 1-5 Hutt Hospital campus map with importance levels

The overarching definition in NZS1170.0 defines an IL4 building as a *'structure with special post-disaster functions*,' with a relevant example being *'medical emergency or surgical facilities*.' In 2015, the Ministry of Business and Innovation and Employment (MBIE) determined that the Main Building at Grey Base Hospital qualified as IL3 because, despite having surgical functions, it lacked any post-disaster functions. This determination was based on the precedence given to the overarching definition. The same rationale applies to the Clock Tower at Hutt Hospital, leading us to conclude that the Clock Tower falls within either IL3 or IL2.

However, it is important to note that the Grey Base Main Building differs significantly from the Clock Tower. The Grey Base Building has emergency medical functions, which the Clock Tower lacks.

The overarching definition in NZS1170.0 of an IL3 building states *'structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds*.' In the case of the Clock Tower, it does not meet any of the example definitions for people in crowds, and we do not see this as a point of contention.

The phrase *'contents of high value to the community*' is not well-defined in the standard and introduces ambiguity in assessing the importance levels. Due to this ambiguity in the overarching definition, we are compelled to consider the specific examples provided within NZS1170.0. The relevant examples for a hospital building are *'emergency medical and other emergency facilities not designated as post-disaster*' and *'health care facilities with a capacity of 50 or more resident patients but lacking surgery or emergency treatment facilities*.' In the case of the Clock Tower, all medical procedures are elective (non-acute), and there are no facilities for emergency patients. Furthermore, the Clock Tower does not have any resident patients.

According to the definition of an IL2 building as *'Normal structures and structures not in other importance levels*,' and considering that the Clock Tower does not meet the defined criteria for an IL3 or IL4 structure, we are of the opinion that it falls within the IL2 importance level.

1.6.3 Site and subsoil class

Based on our review of the published geology and historic ground investigations, we are using the NZS 1170.5:2004 **site subsoil classification of D** for this site.

Geotechnical hazards such as liquefaction, landslide and lateral spreading are outside the scope of this assessment.

1.6.4 Hazard Zone Factor

The hazard zone factor Z determines the "seismic risk" area in accordance with NZS1170.5. There are different hazard zones factors depending on the buildings located in New Zealand. From NZS1170.5, we have used a hazard factor of **Z=0.40** for Wellington.

1.6.5 Scope

The key structural elements in this assessment included the following:

- Concrete shear walls
- Floor diaphragms
- Light-weight roof

The assessment included undertaking the following:

- Review of original as-built structural drawings
- Build a 3D ETABS model of the superstructure in accordance with the structural drawings
- Calculate the code design level earthquake demand based on the factors including ductility and damping
- Specific diaphragm modelling utilising specialist software and critical review of aspects such as connections to main elements
- Calculation of the main superstructure component capacities
- Determine the total and inter-storey drifts
- Detailed assessment of the concrete elements as per the "Technical Proposal" requirements
- Calculation of the %NBS scores for the superstructure components to determine the Critical Structural Weakness (CSW)
- Identification of any potential Severe Structural Weaknesses (SSWs)
- Formal in-house verification by CPEng engineer
- Descriptive methodology for any seismic strengthening if required
- Reporting – formal DSA report
- Liaison and meetings as requested

Elements that are excluded in this DSA include, but are not limited to:

- Non-structural building elements (façade glass, ceilings, internal walls, overhead services)

2 Assessed Seismic Risk

The results of the DSA indicate the building's earthquake rating to be **20-30*%NBS* (IL2)** in accordance with the **Guidelines.** This is based on the Critical Structural Weakness (CSW) of the steel roof cross bracing connections, RC Shear walls, RC diaphragm and foundations in all buildings.

Therefore, this is a Grade D building following the NZSEE grading scheme. Grade D buildings represent a risk to occupants more than 10-25 times greater than expected for a new building, indicating a high-risk exposure. Refer to Table 2-1 that shows the relative seismic risk compared to a new building.

Table 2-1 Relative seismic risk

Seismic Grade	***%NBS*(IL2)**	**Approx. risk relative to a similar new building**	**Relative life-safety risk description**
A+	>100	<1	low risk
A	80 to 100	1 to 2 times	low risk
B	67 to 80	2 to 5 times	low to medium risk
C	33 to 67	5 to 10 times	medium risk
D	20 to 33	10 to 25 times	high risk
E	<20	more than 25 times	very high risk

A building with an earthquake rating less than **34*%NBS*** fulfils one of the requirements for the Territorial Authority 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). The Clock Tower Buildings is therefore categorised as an Earthquake-Risk Building and meets one of the criteria that could categorise it as an Earthquake Prone Building by Hutt City Council as the Territorial Authority. We note that our assessment used the **Yellow Chapter**. An assessment using the **Red Book** would likely result in similar scores to the **Yellow Chapter**.

3 Structural System Description

3.1 Vertical Lateral Resisting Elements

Building Design

The Clocktower buildings were constructed in the 1940s, a time when there were no specific seismic requirements in place. These buildings contain singly reinforced walls with plain round bars. Singly reinforced walls generally performed poorly during the Canterbury Earthquakes due to a lack of confinement and insufficient restraint against local bar buckling. These walls also tend not to develop desirable distributed flexural cracks at the base during significant seismic activity. Instead, they may exhibit a rocking response and form a single crack. This mechanism results in inferior energy dissipation capacity compared to modern walls.

However, it is worth noting that despite these limitations, the Clocktower buildings are considered regular structures with multiple vertical lateral resisting elements. In general, structures that are regular and have numerous vertical lateral resisting elements tend to perform better during intense seismic shaking when compared to irregular structures.

Transverse and Longitudinal direction

The primary lateral resisting system for the **East Wing**, **West Wing**, and **Central Wing** buildings in both directions consists of:

- RC shear walls that extend the full height of the buildings
- The concrete wall thickness and wall reinforcement was measured in several locations during the demolition works. The walls were measured as approximately 200mm thick. The wall reinforcement was observed as one layer of 9.5mm diameter plain round bars at approximately 150mm centres in each direction.
- Refer to Figure 3-1, Figure 3-2, Figure 3-3 and Figure 3-4 for the location of the RC shear walls.

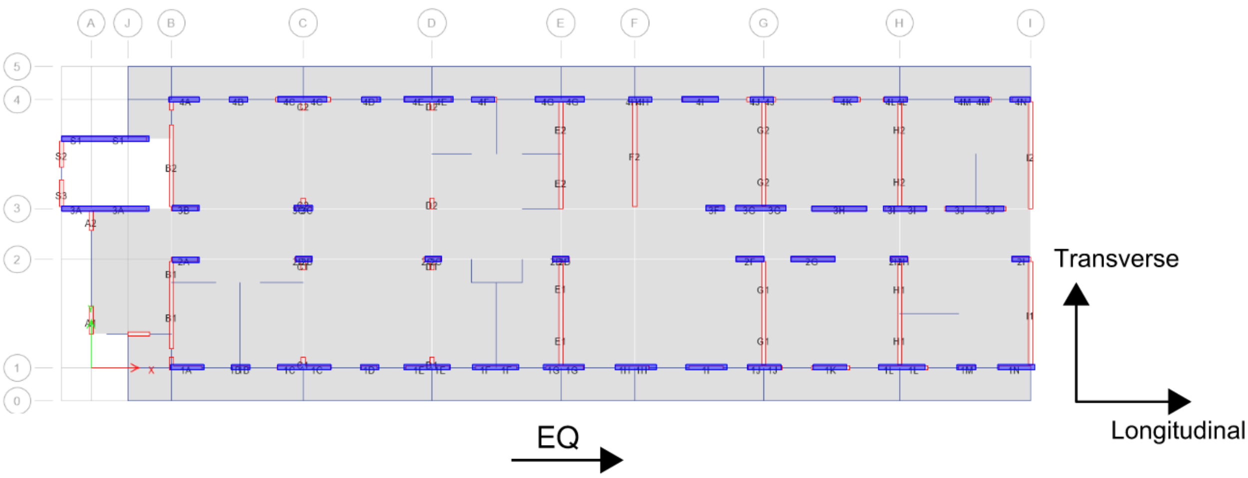

Figure 3-1 East Wing and West Wing lateral resisting elements in the longitudinal direction

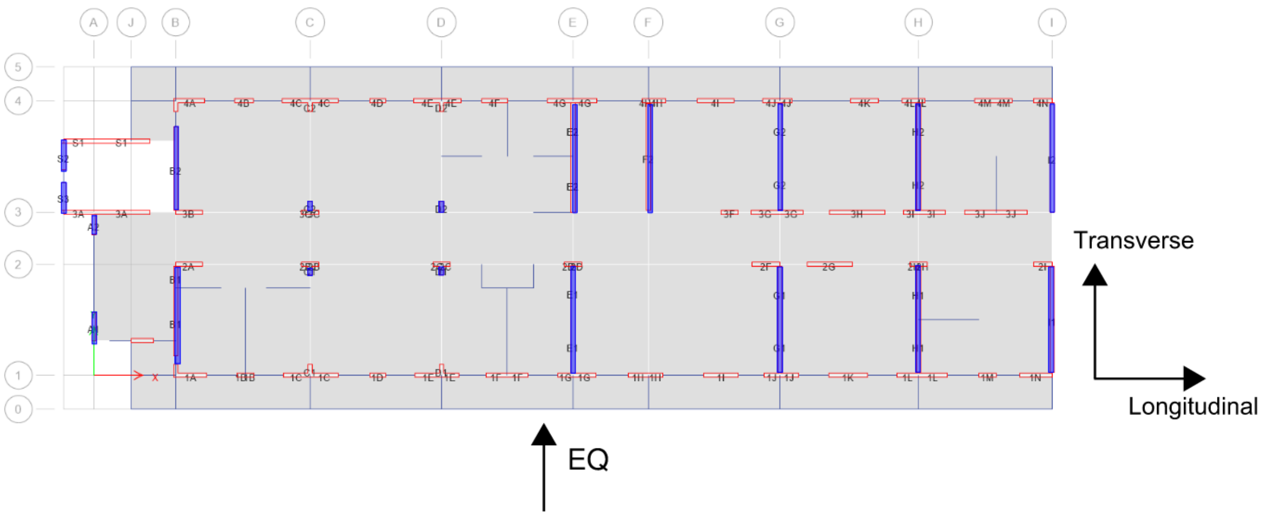

Figure 3-2 East Wing and West Wing lateral resisting elements in the transverse direction

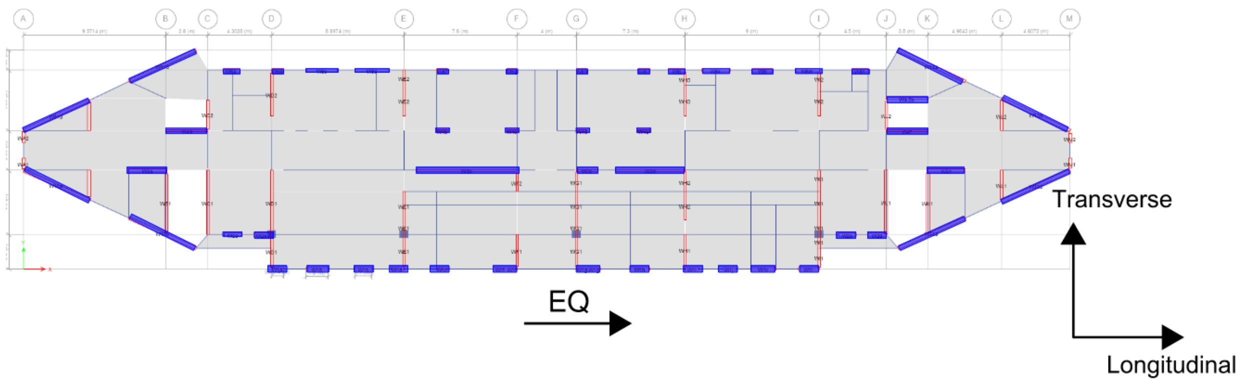

Figure 3-3 Central Wing lateral resisting elements in the longitudinal direction

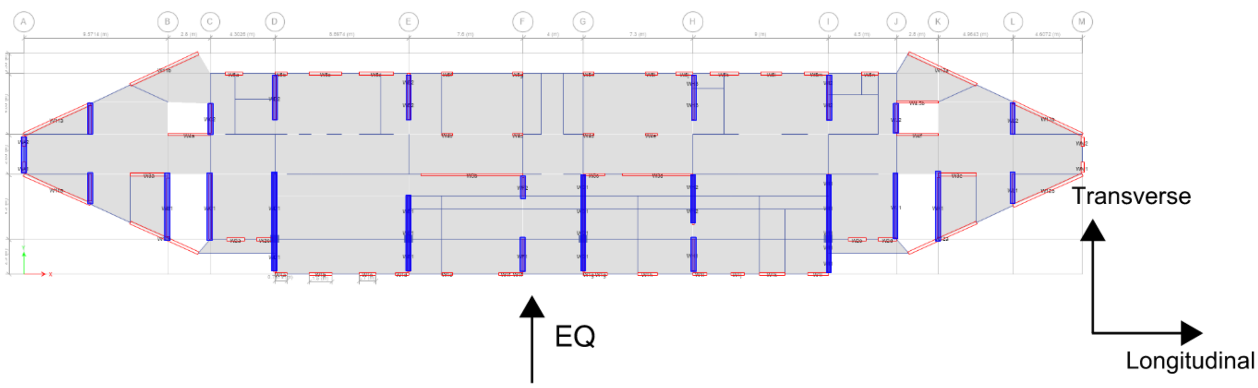

Figure 3-4 Central Wing lateral resisting elements in the transverse direction

3.2 Horizontal Lateral Load Resisting System

3.2.1 Floor slab

Based on visual observation, the floor slabs are constructed using cast-in-situ concrete. The thickness of the concrete slab was measured on-site by coring through the Level 2 floor in the West Wing. The measurement revealed a thickness of 160mm, with an additional 40mm non-structural topping. This thickness was assumed for all concrete floors during the assessment.

3.2.2 Roof

For the roof diaphragm, Lumberlok Multibrace steel braces are employed to distribute the seismic forces from the timber roof structure to the concrete walls. Figure 3-5 illustrates the plan location of these braces at the roof level. The steel braces are connected to timber elements through nailed connections.

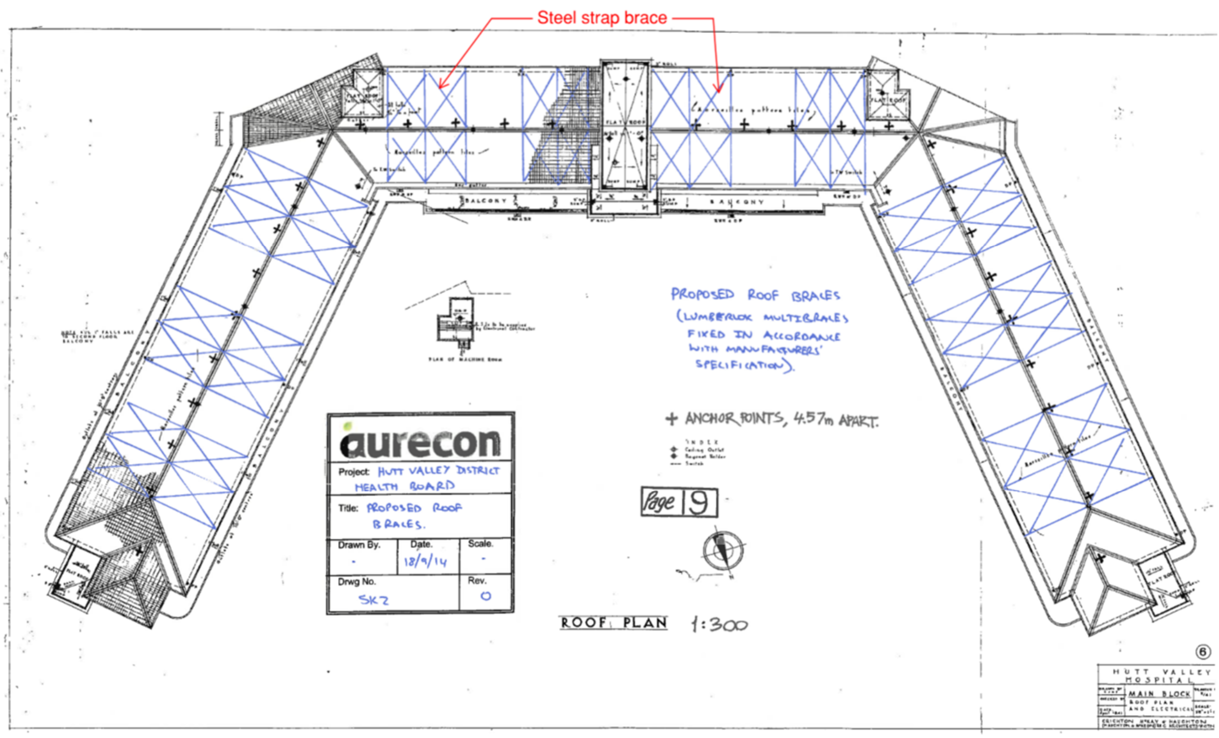

Figure 3-5 Clocktower Roof Plan View

3.3 Foundations

Based on the existing architectural drawings, it was evident that the building is supported by concrete shallow foundations. Unfortunately, the size of the foundations and the specifics of the reinforcement were not available. Please refer to Figure 3-6 and Figure 3-7 for sections through the Clocktower Wings, as depicted in the existing architectural drawings. Detailed structural information regarding the foundations was not accessible. The dimensions of the footings were measured whenever possible from the architectural drawings, and additional insights were derived from the 2008 SKM report.

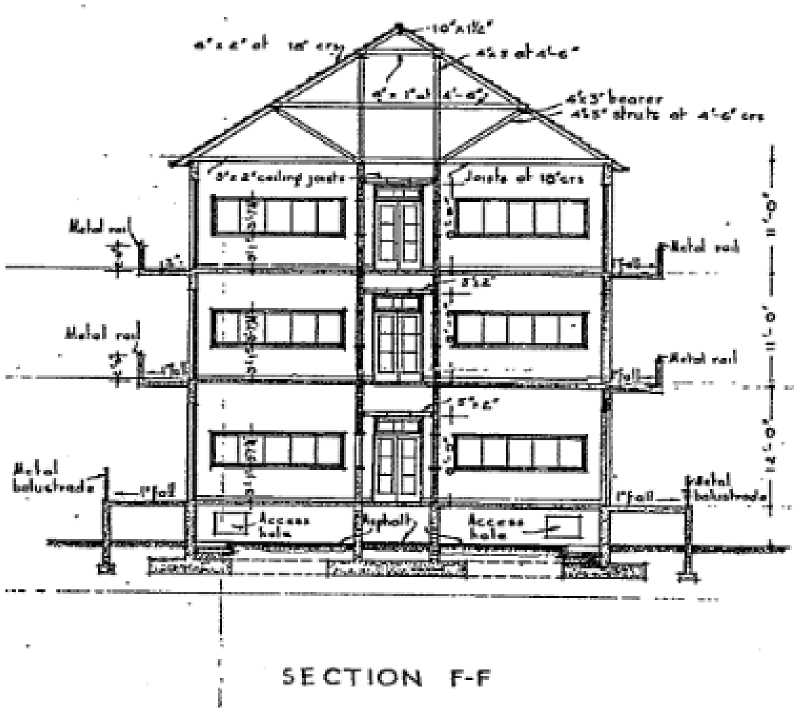

Figure 3-6 Architectural section through the East and West Wings

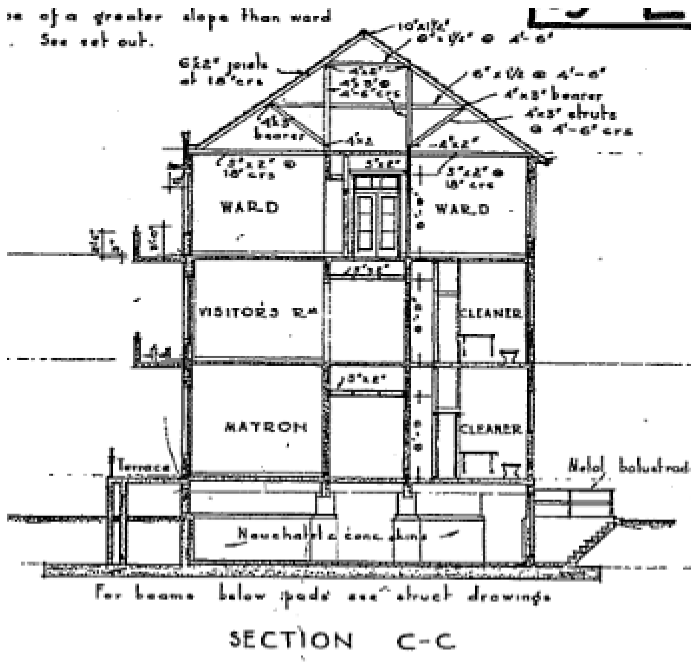

Figure 3-7 Architectural section through the Central Wing

3.4 Site Geology and Subsoil Classification

A geotechnical desktop study was conducted to assess the ground conditions beneath the Clocktower. The study relied on limited existing geotechnical information in the vicinity of the site, and no specific investigations were carried out at the site itself.

The study identified that the soils at depths between 3 to 6 meters below ground level may exhibit liquefaction potential under the Ultimate Limit State seismic condition. However, it should be noted that the thickness of these soil layers can vary across the site, as well as the depth to groundwater, which will impact the risk of liquefaction.

The ultimate seismic bearing capacity of the strip footings was determined to be 90kPa, considering the possibility of liquefaction. This capacity was based on a footing width of 2.3 meters and a foundation depth of 0.9 meters below ground level.

To confirm the effects of liquefaction, further geotechnical investigation in close proximity to the Clocktower site would be required.

3.5 Non-Structural Building Elements

From our recent experience in evaluating similar buildings in Christchurch and Wellington, non-structural building elements (façade glass, ceilings, internal walls, overhead services) constitute a significant portion of the repair/reinstatement cost following an earthquake. In a moderate seismic event, non-structural element damage may contribute heavily to downtime and repair costs, and therefore the performance of these non-structural elements following a moderate seismic event could affect business continuity.

Assessment of these non-structural elements' performance is not part of this DSA. However, a desktop study of the available documentation did not identify any large plant, ceilings, and partitions that would raise concern.

4 Assessment Methodology

4.1 Assessment Description

The DSA was completed in accordance with the Guidelines. The Guidelines provide solutions and methods for the assessment of existing buildings and give guidance for strengthening methodologies that are considered acceptable. Refer to **Appendix B** for the Assessment Inputs.

We have undertaken a stepped analysis approach to assess this building. We started with simpler elastic analysis methods and progressed with more complex analysis (displacement-based analysis) to determine the seismic performance of the building.

Refer to **Appendix C** for the Assessment Assumptions and Limitations

4.2 Computer Modelling

4.2.1 Primary lateral resisting system

A computer model of the structure was developed using the ETABS computer program. Refer to Figure 4-1 and Figure 4-2 for the 3D View of the ETABS Models. The global structures behaviour was captured using a Simple Lateral Mechanism Analysis (SLaMA) procedure.

The SLaMA analysis was conducted to gain understanding of both the overall seismic behaviour and the post-elastic capacity of RC walls in the building. This analysis involved combining the individual strength to deformation relationships of each of the RC walls to establish the strength to deformation relationship for the entire building. To determine the %NBS that the lateral system achieved, the acceleration-displacement response spectrum (ADRS) method was utilised. This was done for each direction at the effective height of the structure.

To determine the post-elastic rotation capacity of RC walls, as outlined in the **Guidelines** for walls reinforced with plain round bars, we have selected the smaller value from the following:

1. Equivalent post-yield rocking capacity
2. Deformed bar probable rotation capacity.
3. The onset of OOP wall lateral instability

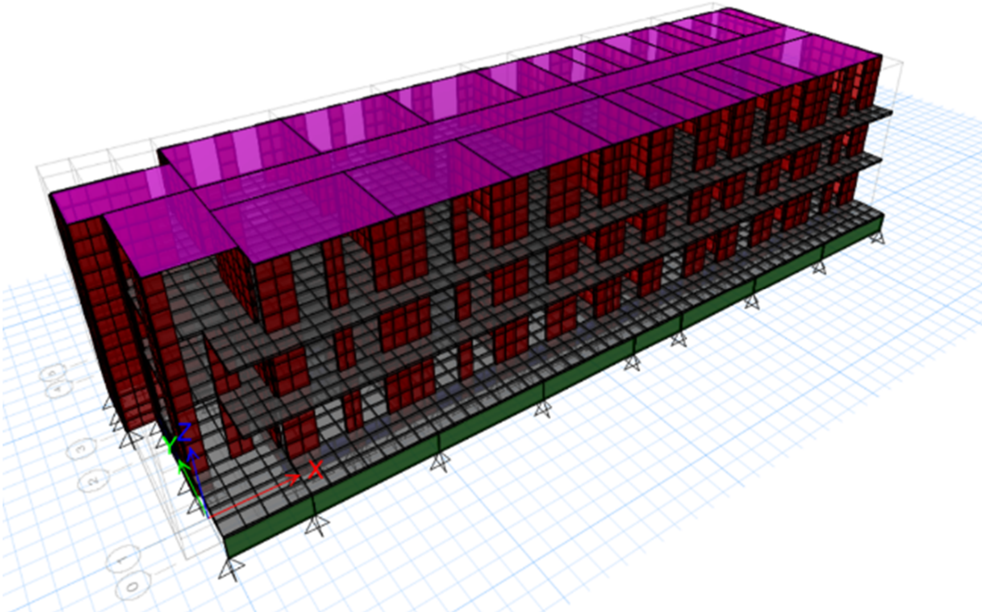

Figure 4-1 3D ETABS model for the East and West Wing

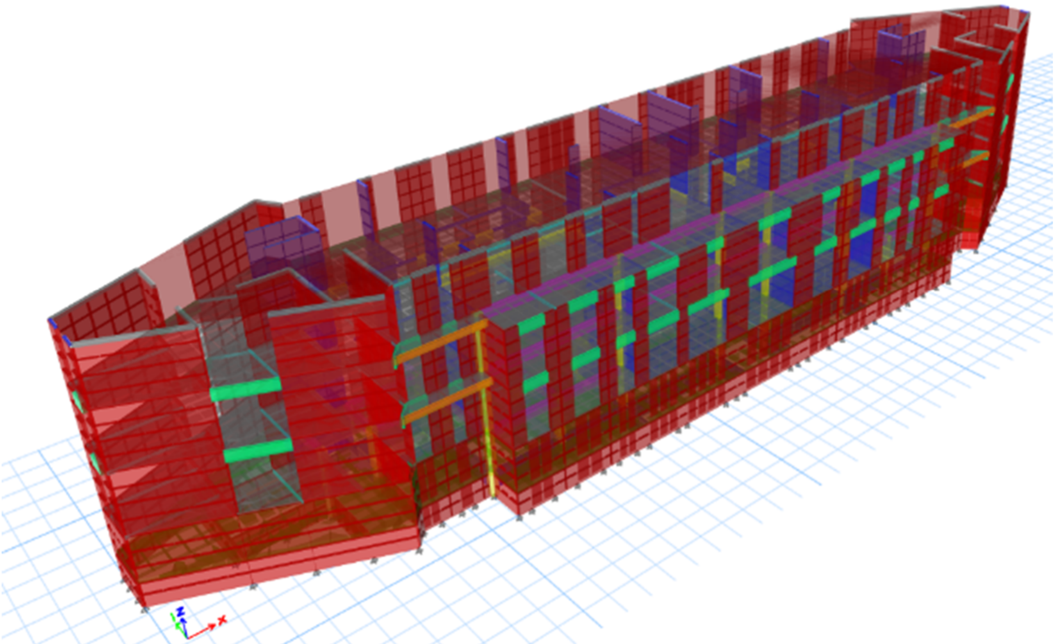

Figure 4-2 3D ETABS model for the Central Wing

4.2.2 Diaphragms

The diaphragm acceleration demands were determined by the pESA method as recommended in NZS1170.5 C5.7.2.

These design accelerations/forces were then applied to the centre of mass of each diaphragm of the 3D ETABS model. For each diaphragm and for each direction of loading, the shear entering/exiting each vertical lateral resisting element (difference in shear above and below the level being considered) was extracted.

Due to the complexity of the diaphragms the diaphragm demands were assessed using the Grillage Method as recommended in the **Guidelines**. It is essentially an automated strut and tie analysis method to obtain demands. Capacities were determined using Appendix A of NZS 3101:2006.

5 Peer Review Process

After the Clocktower DSA draft report was issued, a peer review of the assessment was conducted by GDC Consultants, with inputs from Kestrel Group on behalf of NZ Health. This process involved GDC Consultants reviewing the calculations prepared as part of the building assessment and providing comments and queries to which Aurecon had to respond. Several meetings were held between Aurecon, GDC Consultants, and Kestrel Group to discuss these items. Once an agreement was reached between Aurecon and GDC Consultants, the scores for elements were updated accordingly.

GDC Consultants issued a letter titled "*Review of the Hutt Hospital Clock Tower Building 2022 Detailed Seismic Assessment*." The letter concluded that the Clock Tower buildings score less than **34%*NBS*(IL2)**, meeting one of the criteria for being classified as earthquake-prone by the Territorial Authority. Aurecon generally agrees with GDC Consultants' letter that the buildings score less than **34%*NBS*(IL2)** in accordance with the **Guidelines**.

As a result of the review, there have been minor changes in the reported seismic scores for various aspects of the building, which are summarised in Table 5-1 and Table 5-2 Table 6-2below. Further details of the element scores can be found in Section 6 of this report.

Table 5-1 East and West Wing pre-peer review verse post-peer review %*NBS* summary

Element	Pre Peer-Review %*NBS*(IL2)	Post Peer Review %*NBS*(IL2)	Commentary
Concrete Shear Walls – both directions	20-30%	30%	▪ Aurecon and GDC Consultants agreed that the shear walls should be scored based on a displacement-based analysis method.
RC diaphragms – both directions	20-30%	20-30%	▪ The peer reviewer did not provide any comments regarding the %*NBS* score of the RC diaphragms. Therefore, no change in %*NBS*.
Steel Cross Bracing Roof – both directions	20-30%	20-30%	▪ The peer reviewer did not provide any comments regarding the %*NBS* score of the steel cross bracing roof. Therefore, no change in %*NBS*.

Table 5-2 Central Wing pre-peer review verse post-peer review %*NBS* summary

Element	Pre Peer-Review %*NBS*(IL2)	Post Peer Review %*NBS*(IL2)	Commentary
Concrete Shear Walls – both directions	20-30%	25%	▪ Aurecon and GDC Consultants agreed that the shear walls should be scored based on a displacement-based analysis method.
RC diaphragms – both directions	20-30%	20-30%	▪ The peer reviewer did not provide any comments regarding the %*NBS* score of the RC diaphragms. Therefore, no change in %*NBS*.
Steel Cross Bracing Roof – both directions	20-30%	20-30%	▪ The peer reviewer did not provide any comments regarding the %*NBS* score of the steel cross bracing roof. Therefore, no change in %*NBS*.

6 Assessment Results

6.1 Assessment Results Summary

The results of the DSA indicate that the Building's earthquake rating to be **20-30%*NBS*(IL2**) in accordance with the **Guidelines**. The earthquake rating is based on the lowest scoring element shown in Table 6-1 and Table 6-2.

Table 6-1 East and West Wing Summary of Building Elements %NBS scores

Element	*%NBS*(IL2)	Commentary
Concrete Shear Walls – both directions	30%	▪ The shear walls have insufficient shear and flexural capacity to resist 100% ULS loading. ▪ The lap lengths of the plain round bars are insufficient to yield the bars.
RC diaphragms – both directions	20-30%	▪ The diaphragm and diaphragm connection to the RC walls have insufficient tension capacity to transfer 100%ULS inertia forces to the shear walls
Steel Cross Bracing Roof – both directions	20-30%	▪ The Steel Cross Bracing have insufficient tension capacity to transfer 100%ULS parts loading to the shear walls

Table 6-2 Central Wing Summary of Building Elements %NBS scores

Element	*%NBS*(IL2)	Commentary
Concrete Shear Walls – both directions	25%	▪ The shear walls have insufficient shear and flexural capacity to resist 100% ULS loading. ▪ The lap lengths of the plain round bars are insufficient to yield the bars.
RC diaphragms – both directions	20-30%	▪ The diaphragms have insufficient tension capacity to transfer 100%ULS inertia forces to the shear walls
Steel Cross Bracing Roof – both directions	20-30%	▪ The Steel Cross Bracing have insufficient tension capacity to transfer 100%ULS parts loading to the shear walls

6.2 Displacements and Inter-storey Drifts

The maximum inter-storey drift under 100%ULS shaking, for the **East Wing**, **West Wing**, and **Central Wing building** allowing for the kdm modification factor, is shown in Table 6-3. In both directions the drifts are less than the design code limit of 2.5%.

Table 6-3 Estimated Maximum Inter-Storey Drift for 100% ULS shaking for all buildings

Direction	Maximum Inter-storey Drift
Longitudinal	0.7%
Transverse	0.9%

6.3 Structural Weaknesses

A structural weakness is an aspect of the building structure and/or the foundation that scores less than 100%*NBS*(IL2). The Critical Structural Weakness (CSW) is the lowest scoring structural weakness determined in the assessment. Based on the results of the DSA, the CSW for this building is:

- Steel roof braces and connections
- RC shear walls
- RC diaphragm
- RC foundations

A Severe Structural Weakness (SSW) is a defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge.

- There are no SSWs identified for this building.

6.4 Primary Lateral Resisting Systems – RC walls

The RC walls have insufficient flexural and/or shear capacities to withstand a global system ductility ranging from 1.25 to 2 under 100% ULS shaking (representing the earthquake rating of 100%NBS). In both directions, the lap lengths of the plain round bars at the base of the walls are inadequate to yield the bars. The required development length was taken as twice that required for an equivalent deformed bar as determined by NZS 3101:2006. Consequently, during an ULS earthquake, the bars are expected to slip, leading to the formation of a single diagonal crack. Once this crack appears, the walls may experience a rocking response and potentially lead to sliding shear failure. Some walls are also susceptible to out-of-plane lateral instability and shear failure during a design-level earthquake. Refer to Figure 6-1 and Figure 6-2 that shows the governing RC post-elastic rotation mechanism for each of the buildings in each direction.

When rocking, shear failure and/or out-of-plane instability occur, significant spalling of concrete cover on the RC walls may transpire, contributing to increased building displacements. As the displacements increase, non-structural elements such as doors, windows, and building services are anticipated to sustain significant damage. Additionally, the gap between adjacent buildings may result in collision and cause local crushing of the slab and wall edges. Once substantial shear sliding occurs in the walls, their capacity to carry gravity loads may be compromised.

Although there is potential for damage in a major event, the RC walls are expected to perform at a level above the assigned score. This is because the building is well-connected with an in-situ diaphragm and features multiple RC shear walls. Once the capacity of one RC wall is exceeded, seismic load can redistribute to the other RC walls.

Furthermore, the buildings are considered structurally regular. Observations from the Christchurch earthquake in 2011 revealed that regular buildings exhibited better behaviour compared to irregular buildings.

It is important to note that our assessment does not encompass the post-failure behaviour of the structure. Predicting how building elements will behave post failure is extremely challenging due to numerous uncertainties involved.

Finally, it should be noted that the spandrel beams plain round bars are expected to slip under moderate earthquake shaking. Once the bars slip, there is no restoring force acting on the spandrels as they are not subjected to compression load. Consequently, the spandrels do not significantly contribute to the lateral load resistance during ULS shaking.

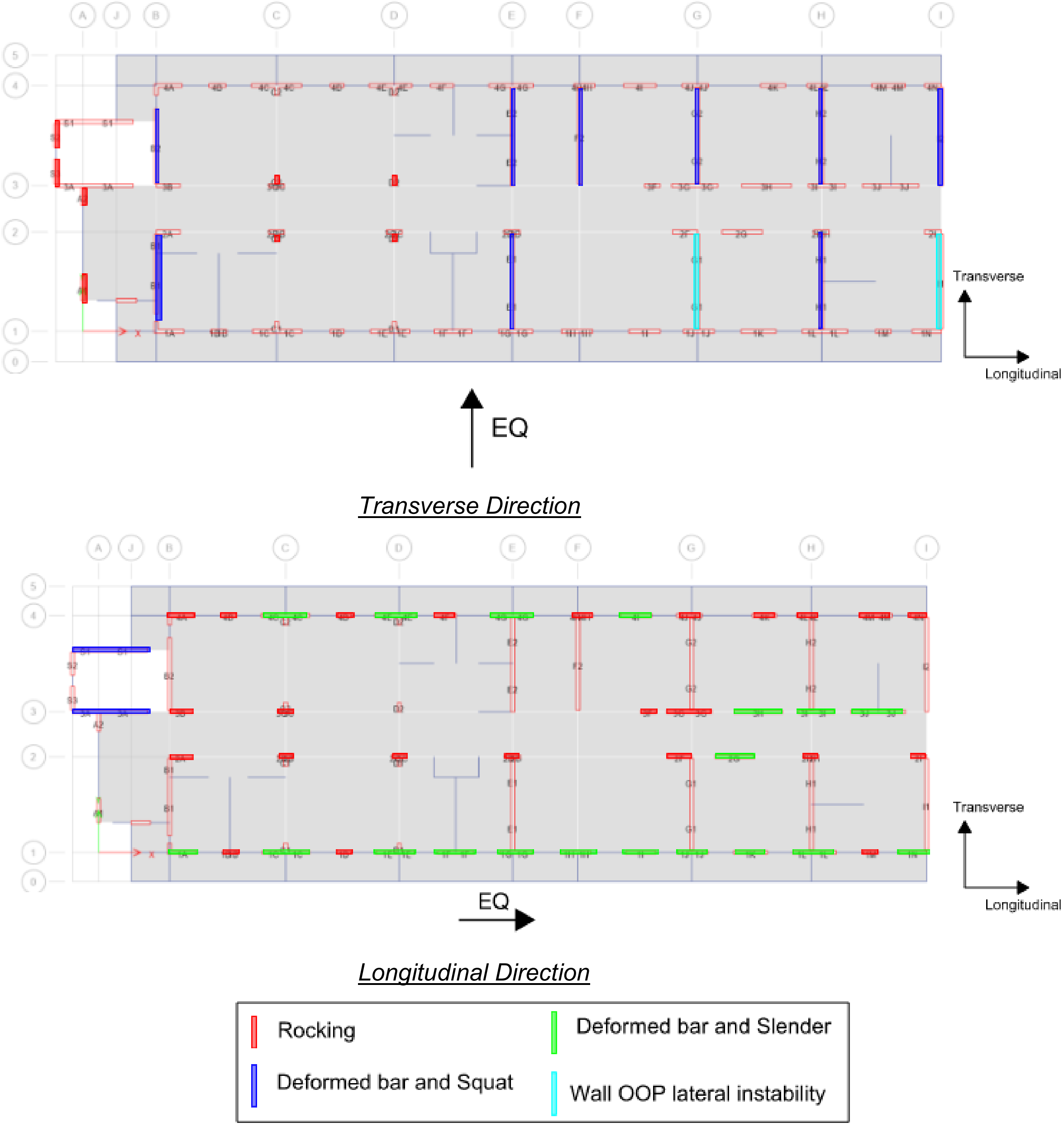

Figure 6-1 East and West Wing RC walls governing post-elastic rotation mechanism

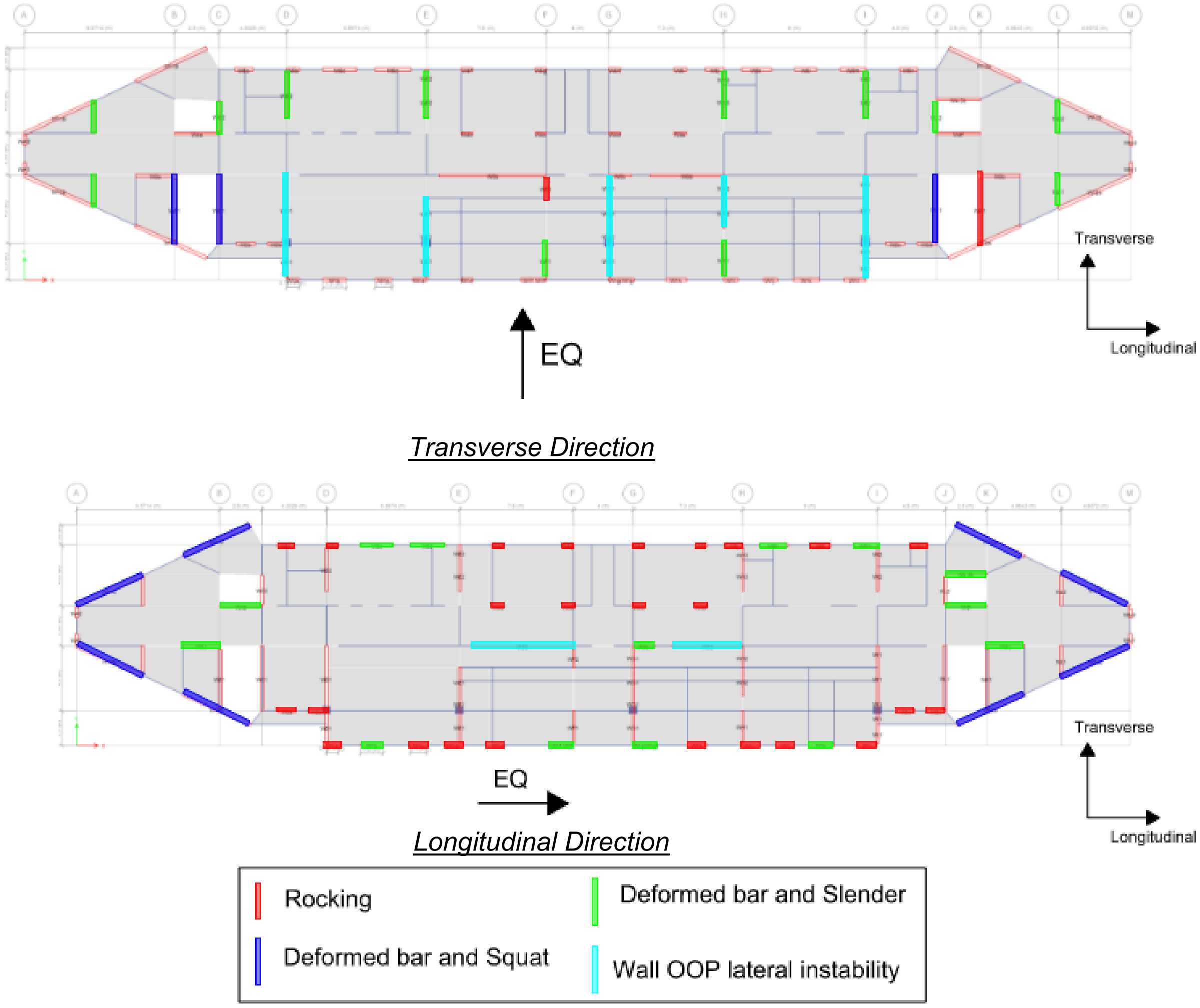

Figure 6-2 Central Wing RC walls governing post-elastic rotation mechanism

6.5 RC diaphragm

The diaphragm and diaphragm connection to the RC walls have insufficient tension capacity to transfer 100%ULS inertia forces to the shear walls.

Diaphragm load must be transferred into the shear walls either at the ends of the wall (through compression bearing or a tension tie) or on the side walls (through shear-friction). Refer **to** Figure 6-3 for the load transfer mechanism.

Once the diaphragm connection is exceeded, the diaphragm may detach from the RC walls. Seismic load may then redistribute to the other in-plane shear walls and overload them. RC walls out-of-plane may also provide an alternative load path.

The diaphragms score is based upon specific requirements in the current assessment guidelines, which include factors of safety in the applied loadings and limit the capacity of the diaphragms based on the first "failure". The diaphragms are likely to perform at a level above the given score even if there is potential for damage in a major event.

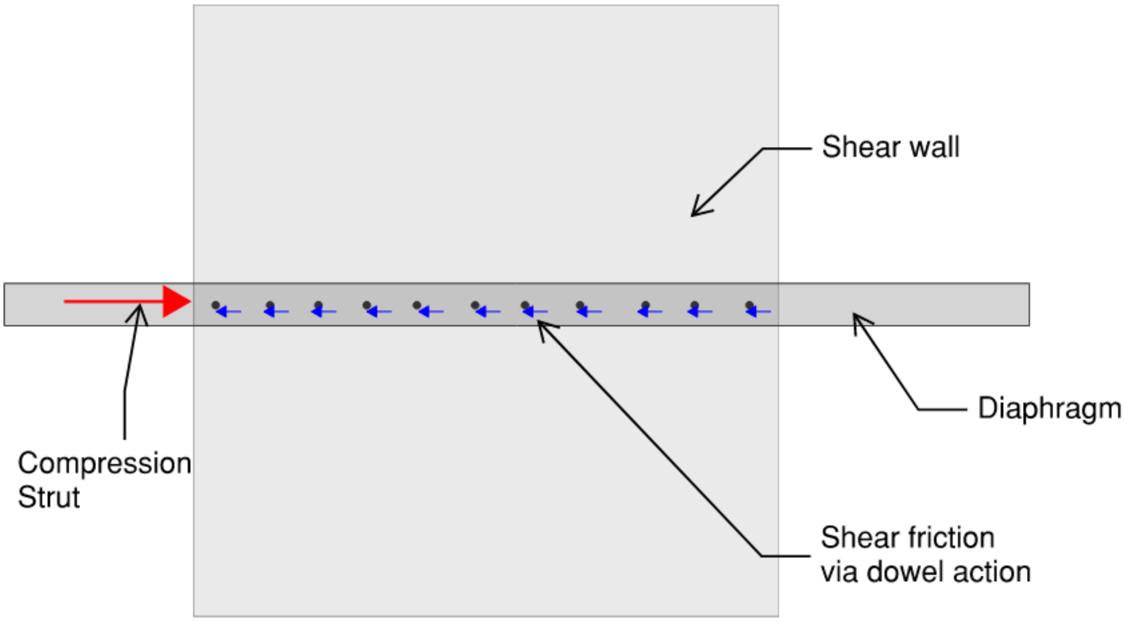

Figure 6-3 Shear wall elevation showing the load transfer mechanism

6.6 RC foundations

The shallow RC foundations have insufficient structural and bearing capacity to resist the building overturning moment in an ULS earthquake. Once the bearing capacity is exceeded, the building is expected to undergo significant vertical and horizontal differential settlements. The building may settle and possibly tilt. This will likely cause significant damage to the RC walls, diaphragms, and non-structural items.

6.7 Roof braces and connections

The steel cross bracing and connections have insufficient tension capacity to transfer 100%ULS seismic parts loading to the shear walls. Once the brace capacity is exceeded in an ULS earthquake, the roof may become flexible and local portions of the roof may lose gravity carrying capacity.

However, as the roof is light weight, losing a local portion of the roof is not considered a significant life safety hazard.

6.8 Stairs

Due to the absence of detailed information in the available documentation, it was not possible to assign a %*NBS* score to the stairs. However, upon conducting onsite investigations, it was observed that the connections between the stairs and landings appeared to be fixed, lacking provisions for sliding or accommodating seismic movement. As a result, in the event of a design-level earthquake, the stairs could inadvertently act as unintended struts, potentially transmitting forces in undesired ways. However, as the stairs are located next to a RC shear wall, it is expected that the walls “protect” the stairs from attracting significant in-plane seismic loading.

7 Potential Strengthening Options

7.1 Scope of Strengthening

We recommend the building is seismically retrofitted to a minimum rating of **67*%NBS* (IL2)**. The 67%*NBS*(IL2) level is regarded as the industry standard requirement for the strengthening of existing buildings. The strengthening options recommended are only of a schematic level detail, and a detailed design will be required for construction documents. It is noted that the schematic design presented is one structural solution and there may be other solutions for the building.

Listed below are some high-level retrofit solutions that could be implemented in order to improve the overall earthquake rating of the building to 67*%NBS* (IL2). We envisage that the strengthening work would be completed in stages (i.e., floor by floor or groups of floors) to minimise occupant disruption. We note that the noise due to drilling and other construction activities will have impact on the building occupants. The seismic retrofit would include:

- Increase the buildings vertical lateral resisting capacity by installing new RC walls or steel braces
- Increase the foundations capacity by installing new RC jackets around the existing foundations/add new piles
- Increase the diaphragms tension capacity by recessing steel plates into the concrete floor
- Increase the roofs lateral capacity by installing additional braces and struts

We also recommend that part of any seismic upgrade or future fitout that the non-structural building elements (ceilings, internal walls, overhead services and plant and equipment etc) is seismically restrained to meet the current standards. It should be noted that no large plant was identified in the building that would need seismic support. No ceilings, partitions and façade were identified while studying the existing documentation that would raise concern.

We further recommend that in designing any seismic retrofit that the building owner should also consider the proposed increase in seismic hazard levels in Wellington. This would insulate the building against further future reductions in the seismic rating.

8 Future Code Changes

8.1 Hazard Zone Factor

The results of the updated National Seismic Hazard Model (NSHM) were released in October 2022. The previous update to the NSHM was in 2010. Since then, the science behind estimating earthquake rates and understanding and complexity of ground motion modelling have significantly advanced.

The NZSM provides the basis for setting the seismic demands in the design code NZS1170.5. Although the results are not a design standard or design loadings standard, they provide an indication of how the code may reflect the updated seismic hazard in future revisions. A possible outcome of this review will be an increase in the hazard zone factor, Z, for the Wellington region. This factor is used to determine the seismic risk for the area and hence the design standard for new buildings.

A future increase in the Hazard Factor will lead to an increase in the design level for new buildings in Wellington and potentially increase the standard required for existing buildings to achieve 100%NBS when assessed against that new standard.

8.2 Basin Edge Effects

The 2016 Kaikōura earthquake exposed the concept of the “basin edge effects.” The basin edge efforts cause amplification of ground shaking due to the presence of soft soils in the sedimentary basin and cause larger peak ground accelerations than expected. The edge efforts are currently not incorporated in the Earthquake actions design code NZS 1170.5.

The basin edge effects have the potential to significantly increase the design standard for new buildings in particular locations in Wellington and potentially may increase the standard required for existing buildings to achieve 100%NBS (IL2) when assessed against that new standard. The “basin edge effects” is currently being discussed and reviewed by industry experts with no fixed timeframe when it will be introduced into the design standards.

The Clock Tower Building is expected to be significantly affected by these effects.

8.3 Seismic Guidelines

Section C5 – Concrete Buildings – Proposed Revision to the Engineering Assessment Guidelines, dated November 2018, provides the latest engineering knowledge on aspects involved in the assessment of concrete buildings, and to reflect what engineers learned from the Kaikōura earthquake.

However, its impact to the industry to still being assessed before it can be incorporated into regulation. Therefore, some aspects of the Guidelines may potentially change and hence affect the standard required for existing buildings to achieve 100%*NBS* (IL2).

9 Yellow book vs Red Book

The Amendment Section C5, also known as the "**Yellow Chapter**," provides engineers with the latest information regarding the seismic performance of existing concrete buildings. It offers a more accurate assessment of a building's expected seismic behaviour compared to the *original Section C5 of The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (referred to as the "**Red Book**").

According to the Ministry of Business Innovation and Employment (MBIE), engineers should use the revised Yellow Chapter version for assessments, with one exception. The Red Book version should only be used to determine whether a building is potentially earthquake-prone under the Building Act 2004, as this legislation recognizes the July 2017 version as the formal standard.

It is worth noting that diaphragm assessments conducted in accordance with the Red Book or Yellow Chapter versions of the Guidelines may yield different scores due to the different analysis methodologies and assumptions employed. Under the Yellow Chapter, diaphragms with non-ductile mesh require a sophisticated grillage model that considers local stress concentrations. In contrast, the simplified strut and tie model used in the Red Book does not capture these local stress concentrations. Furthermore, the Yellow Chapter provides more guidance and general commentary on the strain compatibility and fracture susceptibility of brittle mesh.

On the other hand, assessments of remaining elements such as RC walls, stairs, and lateral deflection conducted in accordance with the Red Book or Yellow Chapter are likely to yield similar scores due to the similarity in analysis methodologies and assumptions.

Based on the information above, a Detailed Seismic Assessment conducted using the Red Book is likely to yield a seismic rating similar to the one obtained from this Detailed Seismic Assessment that was undertaken in accordance with the Yellow Book.

10 Conclusions and Recommendations

10.1 Conclusion

The Clocktower Buildings achieves an overall seismic capacity rating of **20-30*%NBS*(IL2)** in accordance with the **Guidelines**. This is based on the Critical Structural Weakness (CSW) of the steel roof cross bracing connections, RC Shear walls, RC diaphragm and foundations in all the buildings.

Although these Buildings have a low seismic rating in accordance with the **Guidelines**, it is worth noting that these buildings are considered regular, has many wall elements and is well-tied together with a concrete diaphragm. Buildings that contain these characteristics typically perform “better” in large earthquake shaking when compared to other irregular structures.

10.2 Recommendations

We recommend the building is seismically retrofitted to a minimum rating of 67*%NBS* (IL2). The seismic retrofit would include:

- Increase the buildings vertical lateral resisting capacity by installing new RC walls or steel braces
- Increase the foundations capacity by installing new RC jackets around the existing foundations/add new piles
- Increase the diaphragms tension capacity by recessing steel plates into the concrete floor
- Increase the roofs lateral capacity by installing additional braces and struts

11 Explanatory Notes

- The information contained in this report has been prepared by Aurecon at the request of the Hutt Valley District Health Board. and is exclusively for the Hutt Valley District Health Board's use and reliance. It is not possible to make a proper assessment of this review without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to and the assumptions made by Aurecon. The report will not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. Aurecon accepts no responsibility or liability to any third party for any loss or damage whatsoever arising out of the use of or reliance on this report by that party or any party other than our Client.
- This report contains the professional opinion of Aurecon as to the matters set out herein, in the light of the information available to it during preparation, using its professional judgment and acting in accordance with the standard of care and skill usually exercised by professional engineers providing similar services in similar circumstances. Aurecon is not able to give any warranty or guarantee that all possible damage, defects, conditions or qualities have been identified.
- The report is based on information that has been provided to Aurecon from other sources or by other parties. The report has been prepared strictly on the basis that the information that has been provided is accurate, complete and adequate, except where otherwise identified during site investigation inspections. To the extent that any information is inaccurate, incomplete or inadequate, Aurecon takes no responsibility and disclaims all liability whatsoever for any loss or damage that results from any conclusions based on information that has been provided to Aurecon.
- The inspections of the building discussed in this report have been undertaken to inspect the structure and confirm the adequacy of the existing drawings. This report does not address building defects. Where site inspections were undertaken, they were restricted to visual inspections with intent to determine existing building main structural elements only.
- We have not undertaken a review of secondary elements such as ceilings, building services, plant and partitions.

Appendix A - Definitions and Acronyms

ADRS	Acceleration-displacement response spectrum
Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable yield capacity is exceeded. A brittle structure has little tendency to deform before it fractures.
Critical Structural Weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential CSWs.
Damping	The value of equivalent viscous damping corresponding to the energy dissipated by the structure, or its systems and elements, during the earthquake. It is generally used in nonlinear assessment procedures. For elastic procedures, a constant 5% damping as per NZS 1170.5:2004 is used.
Design Level or ULS earthquake	Design level earthquake or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004 (refer to Section C3)
Detailed Seismic Assessment (DSA)	A seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor or ceiling or a braced roof structure) that is strongly connected to the vertical elements around it and that distributes earthquake lateral forces to vertical elements, such as walls, of the primary lateral system. Diaphragms can be classified as flexible or rigid.
Ductile or Ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Elastic Analysis	Structural analysis technique that relies on linear-elastic assumptions and maintains the use of linear stress-strain and force-displacement relationships. Implicit material nonlinearity (e.g. cracked section) and geometric nonlinearity may be included. Includes equivalent static analysis and modal response spectrum dynamic analysis.
Flexible diaphragm	A diaphragm which for practical purposes is considered so flexible that it is unable to transfer the earthquake loads to shear walls even if the floors/roof are well connected to the walls. Floors and roofs constructed of timber, and/or steel bracing in a URM building, or precast concrete without reinforced concrete topping fall in this category. A diaphragm with a maximum horizontal deformation along its length that is greater than or equal to twice the average inter-storey drift. In a URM building a diaphragm constructed of timber and/or steel bracing.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Nonlinear analysis	Structural analysis technique that incorporates the material nonlinearity (strength, stiffness and hysteretic behaviour) as part of the analysis. Includes nonlinear static (pushover) analysis and nonlinear time history dynamic analysis.

Non-structural item	An item within the building that is not considered to be part of either the primary or secondary structure. Non-structural items such as individual window glazing, ceilings, general building services and building contents are not typically included in the assessment of the building's earthquake rating.
OTM	Overturning moment.
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also incorporate the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Rigid diaphragm	A diaphragm that is not a flexible diaphragm
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but, nevertheless, is required to transfer inertial and vertical loads for which assessment/design by a structural engineer would be expected. Includes precast panels, curtain wall framing systems, stairs and supports to significant building services items
Serviceability limit state (SLS)	Limit state as defined in AS/NZS 1170.0:2002 (or NZS 4203:1992) being the point at which the structure can no longer be used as originally intended without repair
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge
Simple Lateral Mechanism Analysis (SlaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (pushover) relationship for the building as a whole
Single-degree-of- freedom (SDOF)	A simple inverted pendulum system with a single mass
Structural element	Combinations of structural members that can be considered to work together; e.g. the piers and spandrels in a penetrated wall, or beams and columns in a moment resisting frame
Structural member	Individual items of a building structure, e.g. beams, columns, beam-column joints, walls, spandrels, piers
Structural sub-system	Combination of structural elements that form a recognisable means of lateral or gravity load support for a portion of the building: e.g. moment resisting frame, frame/wall. The combination of all of the sub-systems creates the structural system.
Structural system	Combinations of structural elements that form a recognisable means of lateral or gravity load support; e.g. moment resisting frame, frame/wall. Also used to describe the way in which support/restraint is provided by the foundation soils.

Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100%NBS. Note that an aspect of the building structure scoring less than 100%NBS but greater than or equal to 67%NBS is still considered to be a SW even though it is considered to represent an acceptable risk.
Ultimate Limit State (seismic)	A term defined in regulations that describes the limiting capacity of a building for it to be determined to be an earthquake-prone building. This is typically taken as the probable capacity but with the additional requirement that exceeding the probable capacity must be associated with the loss of gravity support (i.e. creates a significant life safety hazard).
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings.
XXX%NBS	The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
XXX%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

Appendix B – Assessment Inputs

Structural Layout

The building layout, member sizes & detailing, and material grades have been taken from available design drawings and calculations. A site inspection of the interior and exterior was carried out to confirm that the drawings and documentation was generally in accordance with the as-built configuration. The following drawing documentation was available at the time of the assessment:

- Architectural floor plans of the original Clock Tower building, dated 1941
- Structural engineering report by SKM titled "*Hutt Hospital Campus, Three Buildings: Detailed Evaluation of Earthquake Resistant Performance*" dated 2008
- Architectural floor plans of the East and West Wings showing proposed alterations to the existing building, dated 1979-1982
- Structural engineering report titled "*Hutt Hospital – Clock Tower Block Structural Report on Re Roofing and Existing Building Structure*" dated 2015.

Dead, Superimposed Dead Loads and Live Loads.

See Table below for the Dead, Superimposed dead loads and Live Loads used in the assessment. The self-weight of the walls, frame members and slabs are calculated by the structural analysis program based on the input section size and unit weight. The design live loads were adopted as indicated as per structural drawings and in accordance with NZS1170.1 loading.

Table: Dead, Superimposed dead loads and Live Loads used in the assessment

Load Type	Load
Dead Load	Calculated by the structural analysis program based on the input section size and unit weight
Super Imposed Dead Load	0.5 kPa
Live Load	Hospital ward 2.0 kPa Corridor 4.0 kPa Office 3.0 kPa Storage 5.0 kPa Stair 4.0 kPa Balcony 4.0 kPa

Seismic Weight

The seismic mass was calculated based on the NZS 1170.5:2004 loading combination W = G + ΨEQu, where ΨE = 0.0 for roof. Where applicable, an area reduction factor was also applied to the live load in accordance with clause 3.4.2 of AS/NZS 1170.1:2002.

Wind Loads

Consideration of wind loads is outside the scope of this assessment.

Seismic loading

The seismic loads were determined in accordance with NZS1170.5 with the following parameters.

Table: Seismic parameters for building assessments

Parameter	Value
Design Working Life	50
Importance level	2
Return Period Factor (R)	1.0
Site Subsoil Classification	D
Period (seconds)	Longitudinal Direction – 0.4 seconds Transverse Direction – 0.4 seconds
Hazard Factor (Z)	0.4
Near Fault Factor (N)	$N_{max} = 1$

Material Properties

The following material properties and corresponding characteristic and probable strengths were used as per the Assessment Guideline Tables C5.3, C5.4 and Section C6. No material specification regarding the concrete and steel used at the time was found in the structural drawings. No physical materials testing has been undertaken to validate the assumed material properties.

Table: Material properties

Item	Characteristic Design Strength (MPa)	Assessment (Probable) Strength (MPa)
Reinforcing Steel – Beams	275 MPa	324 MPa
Concrete	20 MPa	30 MPa

Geotechnical Parameters

A geotechnical desktop study was conducted to assess the ground conditions beneath the Clocktower. The study relied on limited existing geotechnical information in the vicinity of the site, and no specific investigations were carried out at the site itself.

The study identified that the soils at depths between 3 to 6 meters below ground level may exhibit liquefaction potential under the Ultimate Limit State seismic condition. However, it should be noted that the thickness of these soil layers can vary across the site, as well as the depth to groundwater, which will impact the risk of liquefaction.

The ultimate seismic bearing capacity of the strip footings was determined to be 90kPa, considering the possibility of liquefaction. This capacity was based on a footing width of 2.3 meters and a foundation depth of 0.9 meters below ground level.

To confirm the effects of liquefaction, further geotechnical investigation in close proximity to the Clocktower site would be required.

Appendix C - Assessment Assumptions and Limitations

The demolition works occurred at Level 2 in the West Wing only. It was assumed that the three separate wings were constructed at the same time due to their similar appearance and design, as well as appearing on the same architectural drawings. The observations from the demolition works have therefore been assumed for the structure of all the wings.

The findings from the demolition works were assumed are as follows:

Wall - vertical reinforcing:

- Typical: 9.5mm dia bars, single layer, plain bars. The spacing of the reinforcement was approximately 150mm cs on average.
- Ends of the wall: one 15mm dia, plain bar.
- Laps: occur at the base of the wall and were about 450-500mm in length.

Wall - horizontal reinforcing:

- Typical: 9.5mm dia bars, single layer, plain bars. The spacing of the reinforcement was approximately 150mm cs on average.
- Bars hooked at end of the wall. hook length was approximately 70mm.
- Top of the half height walls: one 15mm dia plain bar.
- Bars hooked at end of the wall. Hook length was approximately 100-120mm
- Most of the concrete walls measured were 200mm thick. The assessment for all the walls will assume 200mm thick walls
- The half-height walls were not connected to the adjacent wall piers; the horizontal reinforcement stopped short of the wall pier.

The assumptions and limitations of the assessment are as follows:

- No structural drawings are available.
- The wall locations and lengths have been measured from architectural drawings.
- The East Wing, Central Wing and West Wing are separate structures and were considered to not interact.
- Pounding was not considered. The buildings are relatively stiff, and the seismic displacements are low. The buildings are the same height, and the floors align, and the mass of the floors are similar. Therefore, the impact of pounding will be small.

Due to the lack of information, the following considerations were made:

- The global ductility capacity was determined using the SLAMA analysis method.
- The concrete shear walls have been considered as straight walls to resist in-plane loads; flange contribution was not considered. Effective anchorage of the web horizontal reinforcement in the flange is required to mobilise the flanges, however this could not be confirmed from existing information or on-site during the demolitions.
- The thickness of the concrete slab was measured on-site through a core in the Level 2 floor in the West Wing. The slab was measured as 160mm thick with a 40mm non-structural topping. This thickness was assumed for all concrete floors for the assessment.
- The floor diaphragm reinforcement across the entire floor plate was not known. Redistribution was therefore not considered. During the demolition works, bottom reinforcing in the slab at approximately 120mm centres was observed. The slab reinforcement was assumed as 3/8” bars @ 120mm centres in each direction for all the diaphragms.

- There was no existing information about the connection between the diaphragm and concrete shear walls. During the demolition works, top reinforcing at the perimeter of the slab (where the slab meets the concrete walls) was observed, confirming that the diaphragm is connected to the concrete walls. However, the extent of this connection and the detailing was still unknown. For the assessment, it was assumed that the connections can sufficiently transfer the lateral loads to the walls.
- The size and reinforcement in the foundations was not known. 12mm diameter bars at 200mm centres were assumed.
- The concrete compressive strength and reinforcing yield strength were taken from the Assessment Guidelines.
- Sarking partially covers the roof. It was assumed the sarking covers two-thirds of the roof area and that it provides some capacity to the roof diaphragm (in combination with the steel cross bracing). The probable strength of the sarking was taken from the Assessment Guidelines Table C9.3.
- The steel brace connections were assumed as 5/75 x 3.15mm nails to the top and side of the timber member. This assumption was based on limited photos and site reports from the West Wing re-roofing information. The new blocking could not be seen in the photos, so its construction was assumed as in the sketches available.

Appendix D – Importance Level Description

Importance Levels for Building Types – New Zealand Structures

Importance Level:	Comment:	Example:
1	Structures presenting a low degree of hazard to life and other property	Structures with a total floor area of <30 m2 Farm buildings, isolated structures, towers in rural situations Fences, masts, walls, in-ground swimming pools
2	Normal structures and structures not in other importance levels	Buildings not included in Importance Levels 1, 3 or 4 Single family dwellings and Car parking buildings
3	Structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds	Buildings and facilities as follows: a) Where more than 300 people can congregate in one area b) Day care facilities with a capacity greater than150 c) Primary school or secondary school facilities with a capacity greater than 250 d) Colleges or adult education facilities with a capacity greater than 500 e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities f) Airport terminals, principal railway stations with a capacity greater than 250 g) Correctional institutions h) Multi-occupancy residential, commercial (including shops), industrial office and retailing buildings designed to accommodate more than 5000 people and with a gross area greater than 10 000m2 i) Public assembly buildings, theatres and cinemas of greater than 1000m2 Emergency medical and other emergency facilities not designated as post-disaster Power-generating facilities, water treatment and wastewater treatment facilities and other public utilities not designated as post-disaster Buildings and facilities not designated as post-disaster containing hazardous materials capable of causing hazardous conditions that do not extend beyond the property boundaries

4	Structures with special post-disaster functions	Buildings and facilities designated as essential facilities Buildings and facilities with special post-disaster function Medical emergency or surgical facilities Emergency service facilities such as fire, police stations and emergency vehicle garages Utilities or emergency supplies or installations required as backup for buildings and facilities of Importance Level 4 Designated emergency shelters, designated emergency centres and ancillary facilities Buildings and facilities containing hazardous materials capable of causing hazardous conditions that extend beyond the property boundaries
5	Special structures (outside the scope of this Standard-acceptable probability of failure to be determined by special study)	Structures that have special functions or whose failure poses catastrophic risk to a large area (e.g. 100 km2) or a large number of people (e.g., 100 000) Major dams, extreme hazard facilities

Annual Probability of Exceedance

Design Working Life:	Importance Level:	Annual probability of exceedance for ultimate limit states			Annual probability of exceedance for serviceability limit states	
		Wind	Snow	Earthquake	SLS1	SLS2 Importance level 4 only
Construction equipment	2	1/100	1/50	1/100	1/25	-
Less than 6 months	1 2 3 4	1/25 1/100 1/250 1/1000	1/25 1/50 1/100 1/250	1/25 1/100 1/250 1/1000	- 1/25 1/25 1/25	-
5 years	1 2 3 4	1/25 1/250 1/500 1/1000	1/25 1/50 1/100 1/250	1/25 1/250 1/500 1/1000	- 1/25 1/25 1/25	-
25 years	1 2 3 4	1/50 1/250 1/500 1/1000	1/25 1/50 1/100 1/250	1/50 1/250 1/500 1/1000	- 1/25 1/25 1/25	- - - 1/250
50 years	1 2 3 4	1/100 1/500 1/1000 1/2500	1/50 1/150 1/250 1/500	1/100 1/500 1/1000 1/2500	- 1/25 1/25 1/25	- - - 1/500
100 years or more	1 2 3 4	1/250 1/1000 1/2500 *	1/150 1/250 1/500 *	1/250 1/1000 1/2500 *	- 1/25 1/25 1/25	- - - *

Appendix E – Assessment Summary

1. Building Information	
Building Name/ Description:	The Clock Tower
Street Address	638 High Street, Boulcott, Lower Hutt 5010, New Zealand.
Territorial Authority	Hutt City Council
No. of Storeys	3
Area of Typical Floor (approx.)	6000m^2
Year of Design (approx.)	~1940
NZ Standards designed to	N/A
Structural System including Foundations	Lateral system consists of RC shear walls, spandrels, and piers. Foundation system is RC strip footings
Does the building comprise a shared structural form or shares structural elements with any other adjacent titles?	No
Key features of ground profile and identified geohazards	The site subsoil classification, in terms of NZS1170.5:2004 Clause 3.1.3, is Class D.
Previous strengthening and/ or significant alteration	An internal fit-out in the West Wing at Level 2 began in early 2022. Part of the works involved demolishing a selection of concrete walls at Level 2
Heritage Issues/ Status	N/A
Other Relevant Information	N/A

2. Assessment Information	
Consulting Practice	Aurecon NZ Ltd
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings	▪ Sam Jones ▪ 229819 ▪ 21 years' experience as a structural engineer with significant experience in the seismic assessment of existing buildings
Documentation reviewed, including: • date/ version of drawings/ calculations • previous seismic assessments	▪ Architectural floor plans of the original Clock Tower building, dated 1941
Geotechnical Report(s)	NA
Date(s) Building Inspected and extent of inspection	Early 2022
Description of any structural testing undertaken and results summary	N/A
Previous Assessment Reports	2008 SKM Detailed Evaluation of Earthquake Resistant performance.
Other Relevant Information	N/A

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	2
Site Subsoil Class	D

For a DSA: Summary of how Part C was applied, including: • the analysis methodology(s) used from C2 • other sections of Part C applied	SLaMA and force-based analysis
Other Relevant Information	N/A

<table>
<tr><th colspan="3">4. Assessment Outcomes</th></tr>
<tr><td>Assessment Status</td><td colspan="2">Final</td></tr>
<tr><td>Assessed %NBS Rating</td><td colspan="2">20-30%</td></tr>
<tr><td>For a DSA:</td><td colspan="2"></td></tr>
<tr><td>Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed</td><td colspan="2">Non-structural elements have not been assessed at this stage.</td></tr>
<tr><td>Describe the Governing Critical Structural Weakness</td><td colspan="2">Seismic Performance of the following elements:
▪ Steel roof cross bracing connections
▪ RC Shear walls
▪ RC diaphragm
▪ Foundations</td></tr>
<tr><td>If the results of this DSA are being used for earthquake prone decision purposes, and elements rating <34%NBS have been identified (including Parts):</td><td>Engineering Statement of Structural Weaknesses and Location:
▪ Steel roof cross bracing connections
▪ RC Shear walls
▪ RC diaphragm
▪ Foundations</td><td>Mode of Failure and Physical Consequence Statement(s):
Life-safety</td></tr>
<tr><td>Recommendations
(Optional for EPB purposes)</td><td colspan="2">Strengthening should be undertaken to increase the structure’s rating to a minimum of 67%NBS(IL2) if feasible.</td></tr>
</table>

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