Item 1

Matter No. 043072007

PEER REVIEW REPORT FOR WSP DETAILED SEISMIC ASSESSMENT.

Chateau Tongariro Hotel - State Highway 48, Whakapapa Village, Mount Ruapehu

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The author(s) of this assessment where they use the singular phrase "I" or the plural "we", or similar phraseology, are referring to their role acting on behalf of Prendos New Zealand Limited ("Prendos"), not as individuals.

The findings contained in this assessment are based on a non-invasive visual investigation of the building only along with our considered interpretation of any other information made available to us at the time the report was prepared.

Prendos has carried out this peer review in their professional capacity as Chartered Professional Engineers (CPEng) and in line with the standards required by the Institution of Professional Engineers New Zealand (IPENZ). No additional warranties, express or implied are made in relation to the advice or information contained in this report.

EXECUTIVE SUMMARY

Prendos has been commissioned by Department of Conservation - WLG to undertake a peer review of the detailed seismic assessment (Dsa) For Chateau Tongariro Hotel - State Highway 48, Whakapapa Village, Mount Ruapehu.

The DSA prepared by WSP has been focused on the original 1929 portion of the complex only. Over the years, the original hotel has undergone several alterations and additions of buildings in proximity. These buildings have been constructed either with no seismic gap or with minimal gap in between the original hotel. WSP has not taken into consideration of the interaction and pounding between these buildings.

The lack of as-built drawings significantly increases the challenges associated with this peer review. Without accurate and detailed documentation, there are many unknowns and assumptions that have been made in the detailed assessment report. The beam sizes and reinforcing details of a large proportion of floor beams of this building are based on assumptions.

The seismic behaviour of the building is governed by the detailing and failure mechanisms not being considered during the design stage. The absolute strength capacities of the structural members are not necessarily critical. It's important that this building has sufficient deformation and ductility capacity to allow load redistribution within the building's lateral load resisting system.

The reinforcing bars used in the construction were mainly plain bars. The development of plain bars in tension shall rely on hooks and the development length shall be twice the development length of that of the deformed bars. The effectiveness of this reinforcement relies on the proper development of bond strength between the steel bars and the surrounding concrete. If the development length is inadequate, it can lead to premature failure or reduced load-carrying capacity. WSP has capped the deformation capacity at 1% drifts for the ground floor.

Overall, Prendos concurs with the seismic scores and structural weaknesses in the building assessed by WSP. The results of the DSA indicate the building has 15% NBS assessed in accordance with the Guidelines, and it is classified Earthquake Prone Building (EPB).

The strengthening recommendations in the DSA report are very general and lack the level of detail required to produce concept plans for seismic strengthening.

Table 1 below identified the following structural weaknesses in the building.

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
Reinforced concrete moment frames – Both directions	15-30%	Shear failure of columns due to infill damage, aggravated by torsional effects. Lack of strength and stiffness.
Masonry infill walls – Out of Plane	15%	
Foundations	30-40%	Lack of strength and insufficient bearing capacity of footings, lack of strength of basement walls.
Floor Concrete Diaphragms	30%	Lack of chord strength
Chimneys and small URM parapets	30%	Toppling Failure
Lightweight Timber Structure above third floor	40%	Lack of bracing capacity.
Internal Stairs (main and service)	30%	Lack of displacement compatibility capacity. Risk of column shear failure.
External Stairs (1950s)	25-35%	Lack of seismic gap and stability
Timber floor at lobby	40%	Lack of bracing and unknown connections to main diaphragm
Beams Supporting discontinuous columns at first floor	50%	Lack of shear capacity
Elevator shaft and motor	30%-40%	Lack of lateral capacity above third floor. Lack of displacement compatibility capacity.
Port-cochere retaining walls and ramp structure	40%-50%	Lack of strength

Table 1: Summary of Structural Weaknesses in the Building (From WSP - Chateau Tongariro DSA)

1.0 Introduction

In terms of earthquake prone buildings when assessed and measured against current standards for new buildings, it is considered that an earthquake prone building will not sustain more than 33% of the minimum design actions for the ultimate limit state.

A building is considered an 'earthquake risk building' when assessed against current standards for new buildings. It will only sustain between 33% and 66% of the minimum design actions for the ultimate limit state (BRANZ, Earthquake Terminology, 2013).

This report references The Building (Earthquake-prone Buildings) Amendment Act 2016, which came into effect on 1 July 2017 alongside EPB (Earthquake-prone building) methodology to be brought out by Ministry of Business, Innovation and Employment. The EPB methodology will set out the criteria for Territorial Authorities to accept Engineering Assessments (ISA or DSA) carried out in accordance with 2016 Technical Guidelines.

1.1. Scope of Work

Prendos has been engaged to provide structural engineering services as per Prendos' Scope of Services.

- Review of the documents provided by the client, including the WSP DSA report, calculations, and Etabs Model.
- Confirm the seismic rating (%NBS) of the original 1929 portion of the Chateau complex.
- Provide a report providing our opinion on the information peer reviewed and %NBS.
- Options and recommendation for next course of action.

1.2. Description of The Building

Chateau Tongariro, constructed around 1929. The building's primary load-bearing system consists of in-situ reinforced concrete frames, which provides support to the reinforced concrete slabs. The building's internal and external partitions are composed of brick masonry walls.

For lateral load resisting and overall stability, the building relies on frame actions provided by reinforced concrete frames and in-plane action of the infills.

The top storey of Chateau Tongariro is constructed using a lightweight timber structure. This design choice reduces the mass of the upper levels, which is beneficial in minimizing seismic forces due to the reduction in inertia.

During the site investigations, WSP identified areas where rebar rust was evident, with concrete spalling and staining. Based on the visual and intrusive investigations WSP confirms the following:

- Basement level: the concrete has poor condition, with worst areas towards the southern end, where retaining of the surrounding soil occurs and where there is historical water ingress.
- Third floor and roof structure: timber condition is fair with some signs of water ingress and decay. Steel condition is unknown although it is expected to be fair, and a recoating is due.
- Bricks and mortar were in overall good condition.

- Prior to seismic strengthening, a full structural condition assessment be considered.

2.0 Review of Provided Documents

2.1. DSA Report

WSP has carried out the DSA in accordance with AS/NZS 1170 (Parts 0, 1 and 5) and the MBIE-NZSEE Guidelines Part A and Part C (C1, C2, C3, C5, C7, C8 and C9).

WSP has focused on the original 1929 portion of the complex only. The extent of DSA highlighted in green in Figure 1. Over the years, the original hotel has undergone several alterations and additions of buildings in proximity. These buildings have been constructed either with no seismic gap or with minimal gap between the original hotel. WSP has not taken into consideration the interaction and pounding between these buildings. The potential pounding could cause serious damage to these buildings.

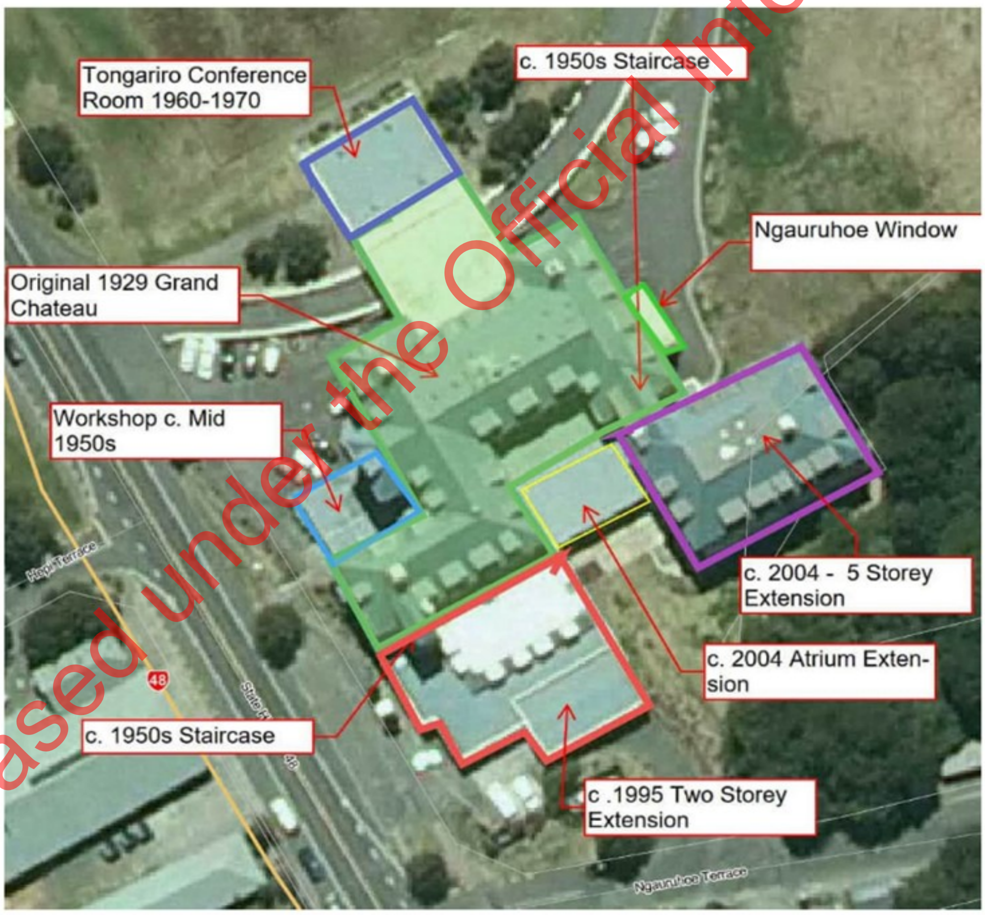

Figure 1 – Current layout of the Chateau Hotel Complex

The building is in an area subject to multiple geological hazards, including lahars, ash falls, lava flows and others. The effects of these geohazards on the performance of the building have not been addressed, posing additional risks to the occupants.

Strengthening Options

We agree with WSP reducing the seismic mass of the building is a crucial strategy for improving the building's seismic performance. The seismic mass includes all the weight that contributes to the seismic forces acting on the structure during an earthquake. To achieve a reduction in seismic mass, WSP has identified items to be removed or replaced in section 8.3 Item 1(a) to (e).

It is essential to highlight that the DSA of the building indicates that its lateral load-resisting system and overall stability are primarily dependent on the frame actions provided by the reinforced concrete frames and the in-plane action of the infills. Upon finalizing the strengthening options, it is important to recheck the strength of the bare frames and evaluate the overall building deformation. This re-evaluation ensures that the proposed strengthening measures effectively enhance the building's structural performance and compliance with the required standards.

In 2022, the New Zealand Seismic Hazard Model (NZSHM) was updated, potentially increasing seismic demand by up to 40%. While not yet incorporated into legislation, this change suggests that buildings may design for stronger earthquake forces in the future. It is recommended to consider these potential increases in any current strengthening schemes to ensure future compliance and safety. Conducting a site-specific hazard study can provide more accurate information tailored to the specific conditions of a site, enhancing the effectiveness of strengthening measures.

The DSA report has included the following seismic strengthening options in section 8. The strengthening recommendations in the report are very general and lack the level of detail required to produce concept plans for seismic strengthening.

Option1 – Augmentation of frame stiffness to improve drift capacity.

To address the issue of excessive frame drift under seismic loading, one potential strategy involves augmenting the stiffness of the structural frame. This enhancement can be achieved through the application of carbon fibre reinforced polymer (CFRP) wraps at critical locations, specifically at beam-column joints and along the columns themselves.

However, the invasive installation process and potential aesthetic impact, combined with residual vulnerability to high drift levels in severe seismic scenarios, warrant careful consideration. WSP considered this option is not suitable.

We also consider the original frame design's flexibility cannot be entirely mitigated, leaving the building susceptible to damage, hence we also consider this option is not suitable. Taking these considerations into account, we also recommend focusing on other more comprehensive solutions that provide a better balance between performance improvement and practical feasibility.

Strengthening 2 – Reinforced concrete shear wall lateral load resisting system.

The proposed solution involves the introduction of a new reinforced concrete shear wall lateral load-resisting system, designed to provide a robust and stiff solution for enhancing the building's seismic performance. This system will significantly improve the overall structural integrity by effectively managing lateral loads and protecting the existing vulnerable frames.

To achieve this, the proposed solution will require the installation of collectors and the strengthening of floor diaphragms. This ensures that the lateral forces are effectively transferred to the new shear walls.

Additionally, the basement walls strengthening and additional work on stair and the elevator support structure will be required.

Overall, this strengthening approach will provide a comprehensive and robust solution to the building's seismic vulnerabilities, ensuring both safety and structural integrity in the event of an earthquake.

Strengthening option 3 – Reduce seismic acceleration and movement demand by means of base isolation at the basement level.

The report explores the feasibility and benefits of implementing base isolation as a seismic retrofitting solution for enhancing the building's resilience.

Base isolation offers significant advantages by reducing seismic acceleration and movement demands, potentially improving the building's seismic performance by a factor of at least four. WSP recommends works to be implemented in conjunction with base isolation include strengthening main lateral frames, isolating the building from adjacent structures, providing movement connections for services, and installing new isolators at the basement level.

However, one of the most significant challenges in implementing base isolation is the difficulty involved in jacking the building. Jacking the building to insert the isolators is a complex and delicate process, requiring precise control to prevent damage to the structure. This process involves temporarily supporting the entire building while isolators are inserted, which is technically demanding and requires specialized equipment and expertise.

Due to the complexity and technical demands of this process, it is crucial to seek advice from a specialist in base isolation systems. A specialist can provide detailed feasibility studies, assess the specific conditions of the building, and determine the best approach to implement base isolation safely and effectively. Their expertise will be essential in planning the jacking process, designing the isolation system, and ensuring that all necessary precautions are taken to protect the integrity of the building during the implementation.

2.2. Material Properties

The material properties are critical to the accuracy and reliability of the DSA analyses. WSP has obtained the material properties from the following sources:

Laboratory testing: Samples of concrete and masonry were tested to determine their actual properties. This method provides the most accurate reflection of the material's current condition.

Historical Properties: Reinforcement steel and structural steel were based on properties documented from the time the structure was originally built. They may be derived from old construction records or standards that were in place during the construction period.

Prendos concurs with the material properties listed in section 5.1 of the DSA report.

2.3. Seismic Loading

The seismic loading has been determined in accordance with 1170.5:2004, with the building’s important level IL2. The Chateau Tongariro is listed on the New Zealand Heritage List as a Category 1 Historic Place of special or outstanding historical or cultural significance. According to AS/NZS 1170.0 Table 3.1, historical buildings of very great cultural significance should be assigned Importance level 3.

The seismic load parameters (site class C and class D) complied with the Geotech report’s recommendations.

2.4. Seismic Weight

The seismic weight was calculated according to NZS 1170.5, incorporating live loads based on the area's use. In addition, the self-weight of the walls and roof, and area factors were considered. Super dead loads, such as screed, ceiling, services, and partitions, were also included in the seismic mass.

2.5. Etabs Model Checks

According to the DSA, the assessment has followed the displacement-based methodology in accordance with the guidelines set out in the MBIE-NZSEE Part C section C2.4.2. For displacement-based methods, a displacement response spectrum is required. The pushover analysis features in ETABS based on the acceleration spectra that are used for force-based design.

The Chateau Tongariro is listed on the New Zealand Heritage List as a Category 1 Historic Place of special or outstanding historical or cultural significance and it should be assigned Importance level 3, the DSA considered the Chateau has Importance Level 2. Based on a building with 50 years design life, the Chateau should have been assessed for a 1000-year return period earthquake event rather than a 500-year return period event.

2.6. SLAMA Check

Simple lateral mechanism analysis (SLAMA) a method used in structural engineering to enhance the accuracy and reliability of seismic assessments. This approach involves modifying the existing structural models, such as those created in ETABS, to better simulate the real-world behaviour of buildings under seismic loads.

In this DSA, SLAMA was utilized to adjust the ETABS model, aiming to provide a more realistic representation of the building's response to earthquakes. WSP incorporated the nonlinear behaviour of building's columns to capture potential inelastic deformations during a design seismic event.

By applying SLAMA to the ETABS model, the resulting analysis provides a more accurate estimation of the building's seismic performance, leading to better-informed decisions for strengthening the structure to meet safety standards.

The lack of as-built drawings significantly increases the challenges associated with this peer review. Without accurate and detailed documentation, there are many unknowns and assumptions that have been made in the detailed assessment report. The beam sizes and reinforcing details of a large proportion of floor beams of this building are based on assumptions.

These uncertainties can lead to significant discrepancies in the analysis, and unreliable results. The critical factor in ensuring the seismic resilience of the building is not the absolute strength capacities of the structural members but rather the building's ability to undergo sufficient deformation and exhibit ductility. To correctly assess the building's actual deformation and ductility capacities, it's crucial to have a structural model with correct data of all primary beams and columns.

The DSA report also indicates that the reinforcing bars used in the construction were mainly plain bars. The development of plain bars in tension shall rely on hooks and the development length shall be twice the development length of that of the deformed bars. The effectiveness of this reinforcement relies on the proper development of bond strength between the steel bars and the surrounding concrete. If the development length is inadequate, it can lead to premature failure or reduced load-carrying capacity.

In summary the inherent limitations in the original design and the lack of detailed construction records have led to inaccuracies in the SLAMA analysis. Based on the assumptions made in the report, the rating of 15%NBS is on the upper bound limit and any further analyses would not significantly improve this rating.

We recommend a strengthening design with a precise structural model should be developed, incorporating the correct primary structural elements, and the new layout to align with structural integrity requirements such as adding a new shear wall system that provides a robust and stiff solution for enhancing the building's seismic performance.

2.7. Chimney and URM Calculations

WSP has carried out the Detailed Seismic Assessment (DSA) of the chimney and URM calculations in accordance with AS/NZS 1170 (Parts 0, 1 and 5) and the MBIE-NZSEE Guidelines Part A and Part C (C7, C8).

There are two original URM chimneys cantilevering from the building's roof. They have been assessed as URM wall. The DSA gives these chimneys a rating of 30%NBS.

The exterior long span masonry walls of the ground floor level are confined by concrete frames, and they have been assessed as moment resisting frames with infill panels. The DSA gives this masonry wall a rating of 15%NBS.

The exterior wythe of the cavity walls above ground floor is not confined by concrete frames, it has been assessed as an URM wall. Fixings are unknown and WSP assumes the walls are tied back to the structure at each floor level. This assumption will need to be checked and reconfirmed during strengthening design stage. The exterior wythe has a rating of 15%NBS.

The chimneys and URM walls are likely to experience toppling failure with probable collapse into the building or outside the building into egress routes.

2.8. Fourth Floor Bracing

WSP has carried out the Detailed Seismic Assessment (DSA) of the fourth-floor bracing in accordance with AS/NZS 1170 (Parts 0, 1 and 5) and the MBIE-NZSEE Guidelines Part A and Part C (C9).

WSP assumes all timber framed walls on the fourth floor have been constructed as bracing wall with two sides of gypsum board and fixed at 300mm centres and a probable strength value of 2KN/m.

The seismic force of the fourth floor has been determined based on design coefficients for parts in accordance with AS/NZS 1170.5. The assessment gives the fourth floor bracing a rating of 40%NB due to lack of bracing capacity in the timber framed wall.

Prendos concurs with this rating.

2.9. Concrete Diaphragm

WSP has carried out the Detailed Seismic Assessment (DSA) of the concrete diaphragms in accordance with AS/NZS 1170 (Parts 0, 1 and 5) and the MBIE-NZSEE Guidelines Part A and Part C (C1, C2, C3). However, based on the calculations provided, it appears that they have not carried out diaphragm grillage modelling and analysis methodology in accordance with MBIE-NZSEE Guidelines C5.

MBIE-NZSEE Guidelines C5 recommends buildings with significant asymmetry in the location of lateral force elements, a grillage method can be used to obtain design actions. In the grillage model, inertial forces (floor forces) will be applied as equal point loads to all the joints in the model. Bracing demands in columns and shear walls will be applied on the same way but opposite direction on joints where the brace is located. Thus, the system will be in perfect equilibrium to then analyse internal forces in the diaphragm.

WSP identified the connecting corridor beams to the rear wing as the critical section of the concrete diaphragm, and they checked the transfer demands in this critical location to determine the diaphragm seismic rating. The tie capacity of the connecting corridor was governed by 2 no. 3/8" continuous top bars in the beam.

To thoroughly review the diaphragm capacity of a concrete floor for seismic loading and to get an understanding of the strengthening scope, it's essential to consider multiple locations across the floor. Checking only one critical location does not provide an accurate representation of the diaphragm's overall performance.

This DSA report has been prepared for the tenant with the sole purpose of providing %NBS result, and Prendos concurs that the 30%NBS rating is realistic for the concrete diaphragm.

The diaphragm strengthening recommendations in the DSA report are very general and lack the level of detail required to produce concept plans for seismic strengthening.

Prendos considers that a strut & tie analysis of the floor diaphragm in accordance with MBIE-NZSEE Guidelines is required in order properly assess the diaphragm and to develop an acceptable strengthening strategy targeted to meet specific areas of deficiency.

2.10. Foundation Assessment

WSP has carried out the Detailed Seismic Assessment (DSA) of the footings in accordance with AS/NZS 1170 (Parts 0, 1 and 5) and the MBIE-NZSEE Guidelines Part A and Part C (C1, C2, C3 and C5).

They have identified three critical pad foundations in the DSA.

The bearing capacity of the shallow spread foundation of 500KPA with a strength reduction faction of 1.0, and this complies with the geotechnical report recommendations.

The bearing of the footings has been assessed for vertical concentric loading only. The assessment has not checked for uplift force in the foundations.

The retaining walls in the basement under seismic loading has not been assessed.

The assessment gives the foundation a rating of 30%NB based on lack of strength and insufficient bearing capacity of footings and lack of strength of basement walls.

Prendos does not consider sufficient evaluation of the foundations and retaining walls has been undertaken to provide %NBS for the foundations and retaining walls.

2.11. Exterior Stair Assessment

There are 2 external staircases in this building, WSP identified that these staircases are not attached to the main building. The main structure of the staircase is in-situ reinforced concrete frames supporting stair flights, and masonry infill between concrete frames providing lateral support to the side and elevations.

WSP has carried out the Detailed Seismic Assessment (DSA) of the stair by checking the overturning of the overall staircase structure. The assessment checked the self-weight of the staircase structure and foundation weight against the destabilizing force generated in a design seismic event. The assessment gives the staircase structure a rating of 25%NB based on overturning of the shallow foundation.

It is important to note that the DSA has been prepared for the tenant with sole purpose of providing %NBS result. To assess the seismic vulnerabilities of staircase structure there are several possible failure modes for masonry infill frames that need to be included in the assessment and these include:

- tension or compression failure of the frame elements.
- shear failure of the masonry infill panel.
- corner crushing compression failure of the infill panel.
- flexural or shear failure of the frame elements.
- out-of-plane failure of the infill panel, and
- tensile failure of beam to column connections due to compressive prying action from the infill panel.

Prendos do not consider sufficient evaluation of the exterior stairs has been undertaken to provide %NBS for the exterior stairs.

These exterior stairs are seismic risk, and we recommend they are to be removed and replaced with steel framing and light staircase.

3.0 Conclusion

For displacement-based methods, a displacement response spectrum is required. The pushover analysis featured in the ETABS modelling is based on the acceleration spectra that are used for force-based design.

The Chateau Tongariro is listed as a Category 1 Historic Place of special or outstanding historical or cultural significance and it should be assigned Importance level 3. The Chateau should have been assessed for a 1000-year return period earthquake event rather than a 500-year return period event.

The inherent limitations in the original design and the lack of detailed construction records have led to inaccuracies in the SLAMA analysis.

The assessment of the diaphragm capacity of the concrete floor needs to consider multiple locations across the floor. Checking only one critical location does not provide an accurate representation of the diaphragm's overall performance.

The strengthening recommendations in the DSA report are very general and lack the level of detail required to produce concept plans for seismic strengthening.

Overall, Prendos concurs with the seismic scores and structural weaknesses in the building assessed by WSP. The results of the DSA indicate the building has a seismic rating of 15%NBS assessed in accordance with the Guidelines, and it is classified Earthquake Prone Building (EPB). Based on the assumptions made in the report, this rating is on the upper bound limit and any further analyses would not significantly improve this rating.

It important to note, Prendos concurs with the 15 %NBS overall but fundamentally the DSA does not meet the guideline requirements completely. Relying on this DSA report that lacks sufficient detail can pose the following risks:

1. Incomplete Risk Assessment: The DSA may miss critical structural weaknesses or vulnerabilities, leading to an incomplete understanding of the building's seismic performance. This can result in an underestimation of the actual risk during an earthquake.

2. Inadequate Mitigation Strategies: If the DSA does not fully identify all potential weaknesses, the recommended mitigation strategies might be insufficient. This could leave a strengthened building more susceptible to damage or failure during an earthquake.

3. Financial Consequences: Relying on an incomplete DSA might lead to unforeseen strengthening costs if additional issues are discovered later. Early and thorough assessment helps in budgeting accurately for necessary upgrades.

4. Safety Concerns: The primary risk is the safety of occupants. If the building's vulnerabilities are not fully addressed, there is an increased risk to life and property during seismic events.

4.0 Recommendations

Firstly, carry out a detailed evaluation of all primary structural elements including confirmation of the sizes of beams and columns and using concrete scanning techniques to confirm the sizes and reinforcing details. This ensures the accuracy of that structural data, which is crucial for subsequent analyses. Floor slabs will also undergo similar scanning to verify the reinforcing steel present.

The building layout to be strategically re-planned to reduce seismic mass. This involves identifying and removing non-structural elements that contribute unnecessary weight, such as heavy cladding, outdated fixtures, and redundant mechanical equipment.

- Where feasible, replace heavy materials with lighter alternatives—for example replacing concrete roofing tiles with lightweight metal or composite materials, and heavy unreinforced brick walls with timber framed walls.
- Optimize the building's contents by minimizing the weight of stored items, A precise structural particularly on higher floors, thereby reducing the impact of seismic forces.

Strengthening elements should be designed to maintain the current load path. This results in the best design efficiency.

- model should then be developed, incorporating the correct primary structural elements, and the new layout to align with structural integrity requirements that provides a robust and stiff solution for enhancing the building's seismic performance, for example adding a new concrete shear wall lateral-resisting system. It is importance to re-evaluate the strength of the bare frames and assess overall building deformation. This ensures that any proposed strengthening measures will effectively improve the building's structural performance, bringing it into compliance with current safety standards.

To investigate the feasibility of applying base isolation systems in this building, you will require a specialist in base isolation systems to carry out detailed feasibility studies, assess the specific conditions of the building, and determine the best approach to implement base isolation safely and effectively.

Base isolation companies will focus on designing, manufacturing, and installing the isolation system. They can collaborate with structural engineers, either work together to integrate isolation system into the building, ensuring compatibility with architectural and structural plans, or engineers provide site-specific data and requirements, and companies develop tailored isolation systems.

Finally, a detailed strengthening design should be carried out incorporating the new layout to align with both structural integrity requirements and heritage considerations. This holistic approach aims to ensure the building is not only safe and compliant but also preserves its historical significance.

To address the potential future changes in legislation due to The New Zealand Seismic Hazard Model (NZSHM) update in 2022 and ensure buildings are adequately prepared, it is highly recommended to conduct a site-specific hazard study. This study will provide precise data tailored to the unique conditions of the site, improving the accuracy and effectiveness of strengthening measures.