Project Number: 2-37905.01

Chateau Tongariro

Detailed Seismic Assessment

24 February 2023

CONFIDENTIAL

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Document Details:
Date: 24 February 2023
Reference: 2-37905.01
Status: Final

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Approved for release by
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Document History and Status

Revision	Date	Author	Reviewed by	Approved by	Status
A	20/12/2022	s9(2)(a)	[redacted]	[redacted]	Draft
1	24/02/2022	s9(2)(a)	[redacted]	[redacted]	Final

Revision Details

Revision	Details
A	Draft Report Issue to Client with preliminary results
1	Final report to client with conceptual strengthening strategies.

Disclaimers and Limitations

This report ('**Report**') has been prepared by WSP exclusively for Kah New Zealand Limited ('**Client**') in relation to Detailed Seismic Assessment for Chateau Tongariro ('**Purpose**') and in accordance with the signed agreement on 17 May 2022. WSP accepts no liability whatsoever for any reliance on or use of this Report, in whole or in part, for any use or purpose other than the Purpose or any use or reliance on the Report by any third party.

In preparing the Report, WSP has relied upon data, surveys, analyses, designs, plans and other information ('**Client Data'**) provided by or on behalf of the Client. Except as otherwise stated in the Report, WSP has not verified the accuracy or completeness of the Client Data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in this Report are based in whole or part on the Client Data, those conclusions are contingent upon the accuracy and completeness of the Client Data. WSP will not be liable in relation to incorrect conclusions or findings in the Report should any Client Data be incorrect or have been concealed, withheld, misrepresented, or otherwise not fully disclosed to WSP.

In preparing the Report, WSP has made assumptions based on engineering judgement of the most likely scenario. The results of this report are subject to the validity of these assumptions and should be further investigated as work progresses in the building.

Contents

Executive Summary

The results of the DSA indicate the building's *Earthquake Rating* to be **15%NBS (IL2)** assessed in accordance with the Guidelines. Therefore, this is a **grade E building** following the New Zealand Society for Earthquake Engineering (NZSEE) grading scheme. Grade E buildings represent a risk to occupants 25 times greater than expected for a new building, indicating a **Very High-Risk safety risk exposure**.

A building with an *Earthquake Rating* less than 34%NBS when assessed in accordance with the Version 1 Guidelines (the 'Red book') 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 by the NZSEE. **The Chateau Tongariro therefore falls within the criteria that could categorise it as an EPB.**

The assessment 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

Although the building rating is dominated by failure of the unreinforced masonry infill and cladding elements, the governing mechanism is a soft storey failure at the ground floor due to a change in architectural plan from the storeys above.

Seismic Retrofit Options

The options for strengthening include:

1. Reduce seismic mass. Restraint unreinforced masonry walls in out-of-plane. Remove external staircases and replace them with modern construction.
2. Reduce demand by means of base isolation
3. Increase strength and stiffness of the structure at every floor. This will require new braces, diaphragm strengthening and strengthening of secondary structures.

The proposed works can be considered a major alteration and therefore accessibility and fire protection systems will need to be upgraded to meet current code standards as part of any Building Consent process. Remediation will also be required.

The building's status as a Category I Historic Place must be considered. It is possible that a Conservation Management Plan for the building exists; however, it has not been made available to the authors of this report at the time of writing. If there is an existing Conservation Management Plan, this should be used to guide decisions relating to the works to minimise any negative impact on the building's heritage values. If no Conservation Management Plan exists, then one should be prepared prior to the next stage of design work commencing.

As the building is scheduled as a heritage building in the Ruapehu District Plan, the District Plan rules will need to be considered, and Resource Consent for the works may be required

Technical Summary

1. Building Information	
Building Name/ Description	Chateau Tongariro Hotel, AKA Grand Chateau, Built circa 1929
Street Address	State Highway 48, National Park. Lot 6, DP436518
Territorial Authority	Ruapehu District Council
No. of Storeys	5
Area of Typical Floor (approx.)	1150-1300 m^2
Year of Design (approx.)	c.1929
NZ Standards designed to	Unknown
Structural System including Foundations	The building structure is comprised of in-situ reinforced concrete frames that supports reinforced concrete slabs. Internal and external partitions are made of brick masonry walls. Lateral structure and stability is provided with concrete portal frames and in-plane action of the infills. Top storey is comprised of a lightweight timber structure.
Does the building comprise a shared structural form or shares structural elements with any other adjacent titles?	No. Although there are other structural units in the title. This assessment focuses on the 1929 portion of the complex.
Key features of ground profile and identified geo-hazards	The building foundations consist of concrete shallow foundations.
Previous strengthening and/ or significant alteration	The building has had several alterations during its life. No strengthening registered. A new wing isolated from the original building was added in 2004.
Heritage Issues/ Status	Historic Place Category 1, List Number 7318 (Heritage New Zealand Pouhere Taonga) Group A Heritage Building, Reference No. 12 (Ruapehu District Council)
Other Relevant Information	

2. Assessment Information	
Consulting Practice	WSP NZ Limited
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings[1]	• 9(2)(a) [redacted] • 1033106 • 10 Years of experience in New Zealand performing seismic assessments and retrofits and 100+ hours of formal training on assessment and retrofit of existing buildings at the University of Canterbury and MBIE training courses on the guidelines. Specialization in Earthquake Engineering and total of 15+ years of experience in structural engineering. Practice area is the assessment and design of low to mid-rise buildings.
Documentation reviewed, including: • date/ version of drawings/ calculations[2] • previous seismic assessments	• Some of the original drawings prepared by Herbert Hall Architect, including column schedule Previous IEP report from 2012 made by Stephen Mitchell Engineers ltd.
Geotechnical Report(s)	Chateau Tongariro – Detailed Seismic Assessment Geotechnical Report – Dated 9 December 2022 by WSP
Date(s) Building Inspected and extent of inspection	Inspected 13th and 14th of October 2022, visual with invasive inspection of exterior and interior. Sampling of concrete, masonry and reinforcement for testing. Additional site visits by engineers and lab in October 2022.
Description of any structural testing undertaken and results summary	• Compressive strength of hollow clay masonry blocks • Compressive strength of partial solid brick • Compressive strength from face of three cell hollow brick • Density testing of masonry bricks and blocks • Compressive strength test for mortar • Compressive strength test for concrete core specimens Tensile and hardness testing of reinforcement bars
Previous Assessment Reports	- Initial Seismic Assessment (ISA) – By Stephen Mitchell Engineers Ltd. dated October 2012 - Initial Seismic Assessment (ISA) – By WSP, dated 16 April 2021
Other Relevant Information	Most of the timeline for alterations was carried out observing available historical pictures on the internet from the National Library and from The Fletcher Construction Company website.

[1] This should include reference to the engineer's Practice Field being in Structural Engineering, and commentary on experience in seismic assessment and recent relevant training
[2] Or justification of assumptions if no drawings were able to be obtained

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	Basement: C Conference centres, E Warehousing, equipment and plant. Ground floor: C – Dining area, C1 - Commercial kitchen, B – Offices for general use, F – light traffic area First to Third floors: A2 – Hotel Rooms, C3 – Corridors, C3 - Stairs Importance level:2
Site Subsoil Class	Site subsoil class C and D assessed as there is no complete information about the shear wave velocity profile up to a 30m depth.
Summary of how Part C was applied, including: • the analysis methodology(s) used from C2 • other sections of Part C applied	Force-based and displacement-based approach. Sections A, C1, C2, C3, C4, C5, C7, C8, C9 and C10 were used during this assessment.
Other Relevant Information	

4. Assessment Outcomes		
Assessment Status (Draft or Final)	Final	
Assessed *%NBS* Rating	15%NBS (IL2)	
Seismic Grade and Relative Risk (from Table A3.1)	E Very High risk	
Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed	URM Chimney, URM walls, Water tanks, URM parapets, Heavy Ceiling and services, internal stairs, external stairs. Note, non-structural items not within scope, but comments provided.	
Describe the Governing Critical Structural Weakness	Lack of out of plane strength of infill masonry walls and/or shear failure of concrete frames when interacting with infills.	
If the results of this DSA are being used for earthquake prone decision purposes, and elements rating <34%NBS have been identified (including Parts)[3]:	Reinforced concrete frame failure, Infills failure in plane and out of plane, URM chimneys, unrestraint plant at roof level, external stairs, internal stairs, floor diaphragms, Ceiling and suspended services, elevator support structure	-
Recommendations	WSP recommends strengthening to achieve 67%NBS (IL2) – Conceptual strengthening strategies are presented for cost estimation purposes.	

[3] If a building comprises a shared structural form or shares structural elements with other adjacent titles, information about the extent to which the low scoring elements affect, or do not affect the structure.

1 Introduction

WSP have been engaged by Kah New Zealand Limited to undertake Detailed Seismic Assessment (DSA) of the existing Chateau Tongariro Hotel, located in the Central North Island on the north-western side of Mt. Ruapehu, on SH48. (Lot 6, DP 436518). The site is comprised of several structures attached or linked to the original Grand Chateau built circa 1929. The Chateau Tongariro Hotel is listed as a Category 1 Historic Place by Heritage New Zealand Pouhere Taonga (List No. 7318), and it is scheduled as a Group A Heritage Building in the Ruapehu District Plan (Ref No. 12). A typical elevation of Chateau Tongariro is provided in Figure 1 below.

The objective of this Detailed Seismic Assessment (DSA) report is to determine the seismic rating of the Chateau Tongariro original building in accordance with the 2017 Ministry of Business, Innovation and Employment (MBIE) guidelines[4] (The Guidelines) and report on the seismic rating for the building. Scope is limited to the original 1929 portion as depicted in the site layout on Figure 10.

Figure 1 – North Elevation of the Chateau Tongariro Hotel

2 Scope of Work

A Detailed Seismic Assessment (DSA) of the Chateau Tongariro buildings have been carried out by WSP and the outcomes are provided in this report.

The scope of work for this Seismic Assessment also includes the following:

- Sourcing and review of existing of structural drawings and construction documentation
- Quantitative structural assessment comprising modelling and calculations sufficient to determine the percentage of New Building Standards (%NBS) of the building based on Importance Level 2, and in accordance with The Guidelines. This includes identification of any critical structural weaknesses or material defects.
- Preparation of a Detailed Seismic Assessment Report, incorporating our comments and findings, and provision of conceptual options for strengthening the building, in the event the building is found to be less than 67%NBS(IL2).
- The assessment focuses on the original 1929 portion of the complex only. For the rest of the structural units, please refer to the ISA report. Please refer to Figure 10 for the extent of the 1929 portion.

[4] The Seismic Assessment of Existing Buildings: Technical Guidelines for Engineering Assessments, July 2017, Version 1

Exclusions:

- Detailed condition assessment of the existing structure
- Detailed assessments of all the other structural units
- Fire and accessibility reports.
- Detailed assessment of seismic restraint of services and non-structural items. Non-structural items include restraint of service runs, plant, ceilings, and partitions.

3 Sources of Building Data

3.1 Documents

The following drawing, documents and building information have been made available from Client or were sourced on site and were referred to for this assessment:

- Some of the original drawings prepared by Herbert Hall Architect
- Previous IEP report from 2012 prepared by Stephen Mitchell Engineers Ltd.
- Site notes during the building inspection
- 3D point cloud scanning during site visits
- Material testing performed by WSP Laboratories and attached to this document.
- Additional photographs and plans from the Fletcher Trust Archives (https://www.fletcherarchives.co.nz/)

Additional publicly available information utilised also includes:

- Previous ISA carried out by WSP dated 15 April 2021.
- Historical pictures from www.retrolens.co.nz
- Historical information from the New Zealand Heritage List Online

3.2 Geotechnical Information

A site-specific geotechnical report was completed by WSP for this assessment. The report was issued on 9 December 2022. This report is used as the main source of geotechnical information for this assessment. Site specific investigations in the form of two boreholes and standpipe piezometer testing were carried out.

3.3 Site Testing

During October 2022, WSP laboratories carried out intrusive site investigations to confirm layout and material properties of masonry walls, concrete, and reinforcement. The full reports can be seen in the appendices. Probable strengths have been used in this assessment based on the statistical combination of the results, in accordance with The Guidelines.

4 Building Description

This section will focus in detailing the site, structural systems and building characteristics that are important for seismic assessment purposes.

4.1 Site Geotechnical Conditions

A site-specific geotechnical report[5] was completed by WSP. This report is used as the main source of geotechnical information for the seismic assessment.

Site specific investigations in the form of two boreholes and standpipe piezometer testing were carried out. The investigations confirmed the site is underlain by boulders in a sand and gravel matrix and that trace silts and clays may be present up to a depth of at least 15m.

The report recommended that the site be assessed for both Site Class C and Site Class D. Site Class C are therefore used for PGA dominated demands, and Site Class D will be used for spectral displacements and accelerations.

The liquefaction potential and other geohazards are discussed in detail in the geotechnical report. It was noted that the Holocene fan / Murimoto formation deposits are potentially susceptible to liquefaction. Liquefaction surface manifestation is not expected at the 500yr but can manifest for greater return periods. Other potential geohazards that were identified include land sliding, fault rupture and lahar.

The geotechnical report states that an ultimate bearing capacity of between 590kPa and 1590kPa can be taken for vertical concentric loads under the recommendations from chapter C4.

4.1.1 Step Change

The results indicate that the slope can be considered stable under both seismic and liquefaction cases and therefore a step-change in building performance or partial loss of foundation support would not be expected at 500yr return period.

4.1.2 Soil structure interaction

Soil-structure interaction has not been considered since it is deemed not to affect the results.

Based on the above, this assessment is considered to be structurally dominated.

[5] "Chateau Tongariro, Detailed Seismic Assessment Geotechnical Report, WSP, 9th December 2022.

4.2 Building Description

The Chateau Tongariro Hotel was originally built in 1929 by the Tongariro Park Tourist Company as a hotel and destination for the Tongariro National Park. The original architect was Herbert Hall and the building was built by the Fletcher Construction Company.

The original structure is comprised of moment resisting reinforced concrete frames from the basement up to the second floor, inclusive. These frames support an in-situ concrete slab, although some areas of the ground floor at the Ruapehu lounge have timber flooring supported on timber posts. The frames are part of the gravity and lateral load resisting structure of the original building and show an adequate distribution over the building with several irregularities. Columns cantilever above the second floor to support steel beams supporting timber joists for the third floor. Thus, the top acommodation floor is comprised of timber framing with an upper roof timber structure as well and a roof space above the ceiling level that accommodates services and several 1m^3 litre water tanks as a partial 4th storey. The reinforced concrete moment frames are founded on shallow pads that, according to the original drawings, sit on natural ground and are connected with tie beams in some cases although the majority of pads are isolated.

Original architectural plans can be seen in following images:

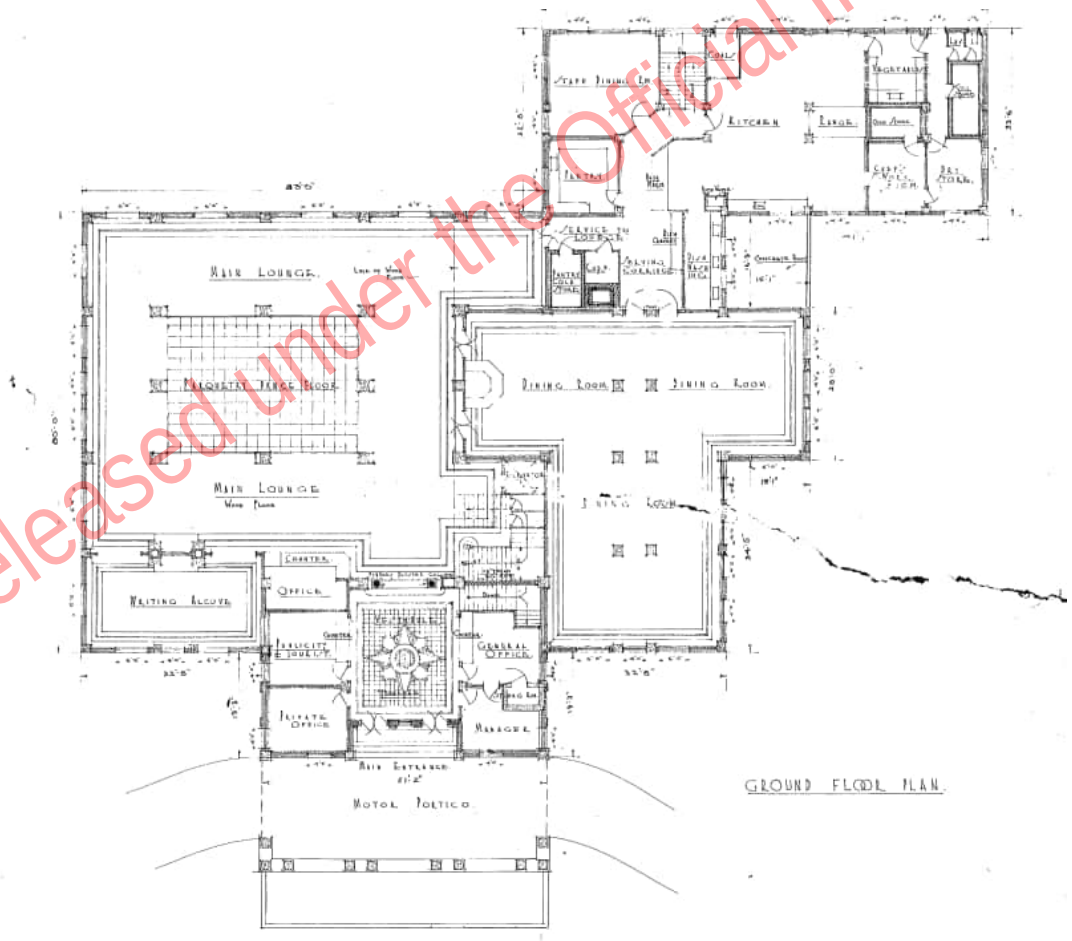

Figure 2 - Ground Floor plan of 1929 design.

External cavity masonry walls complete the external envelope of the building from the basement floor upwards on all exposed sides. The external wythe is supported at basement level and at the first floor slab and the internal wythe frames within the concrete portal frames. Standard clay bricks have been used in all the walls above the first floor, with the veneers being laid in standard running bond. However, below the first floor it is believed that a combination of solid and hollow masonry bricks have been used and plastered and lined over as per the original drawings. Historical photos around the foundation stone show a different array/size of mortar joints, which

might indicate that the current joint pattern was created over the plaster. WSP has gathered brick samples throughout the building, identifying four brick types from standard solid to hollow construction.

The brick infills provide an unintended load path as they are constructed hard up between the frame elements, turning the frame structure into an infilled structure partially with one wythe. In addition, the presente of openings weakens these elements for loading out-of-plane. WSP laboratories also conducted an exploration of the cavity wall around the first floor balcony. Wire ties have been identified around 400x400 crs tying the inner and outer wythes together. Pitting corrosion has been observed in the images taken.

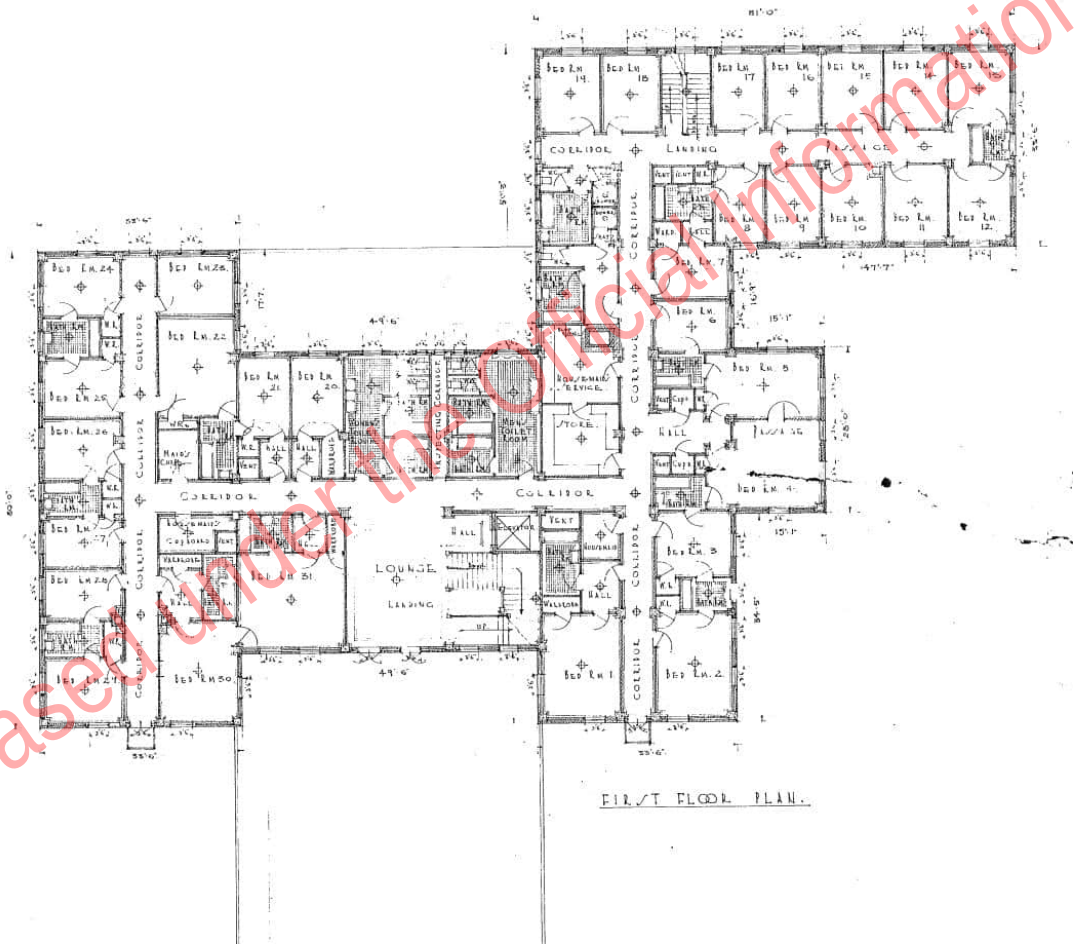

Figure 3 - First floor plan of 1929 Design.

A departure from the original drawings is the Ngauruhoe window extension to the Main Lounge, which can be seen built with concrete moment frames in the plate below. From the construction pictures the change in storey height and infill masonry between the upper storeys and the ground floor can be seen, which might indicate the presence of a vertical irregularity and potentially a soft-storey at the ground floor.

Figure 4 - Construction of Chateau Tongariro – Unknown – Circa 1929

Some basement walls against the natural grade have been completed in reinforced concrete as shown on the original drawings as means of retaining. The exposed sides of the building are completed in masonry infills. Also, not shown on the original drawings are the small reinforced concrete wall-columns as shown on the photo above for the sun piazza below the porte-cochere. The infill masonry panels can be seen in the following photograph taken during the construction of the Chateau Tongariro. Small URM parapets remain above the sun piazza and the Ngaurohoe window.

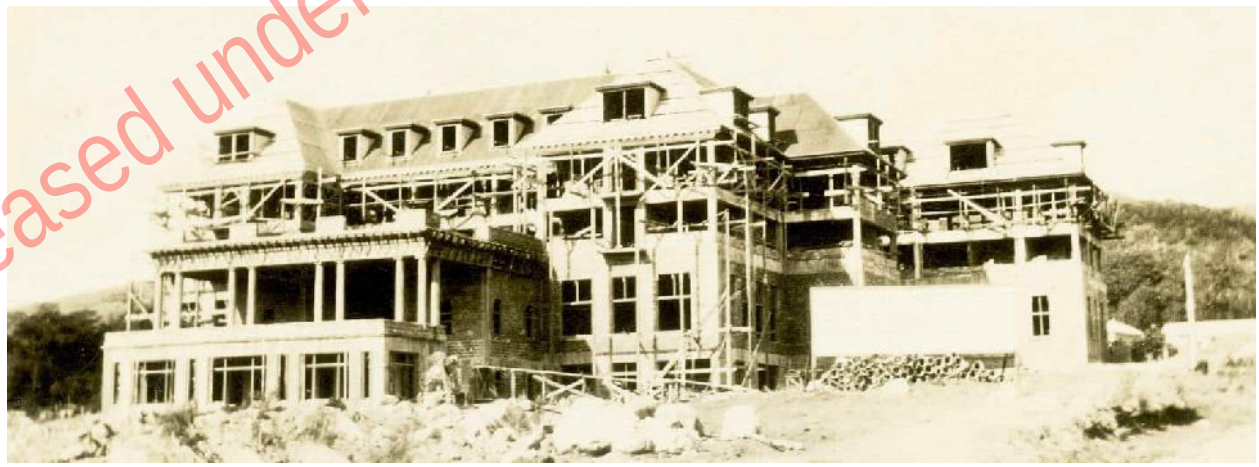

Figure 5 - Construction of the Chateau Tongariro – Unknown – Circa 1929

Internal partitions at the ground, first and second floors are depicted as masonry on the original drawings. However, the original layout has changed in different room refurbishements and alterations. During the site investigations, it can be seen that some of these walls still remain either exposed or captive in-between timber partitions in the rooms. Original toilet walls seem to be partial height to allow for a deeper ceiling for services. In addition, it was observed that random openings, demolitions and discontinuities were created during the alterations in the masonry partitions. These random alterations are invisible to the eye as most of the original partitions are now lined with timber framing. Thus, the complexities arising from different arrangements than the originals in the infills are of unknown nature at this point and can pose a risk in the interaction between infills and frames.

Several unreinforced masonry chimneys and ventilations shafts are still present in the structure and they cantilever several meters above the roof structure (see picture below). One of these

chimneys (North-west) has been replaced by a steel chimney connected to the plant ductwork perhaps as an exhaust or intake for the air conditioning and heating. Original masonry chimneys remain in place at the lounge fireplace and at the rear of the building.

Figure 6 - URM chimney/ventilation

In terms of some observed structural weaknesses identified in the walkovers, in the original garage area at the basement level, there is potential for short column effects due to the presence of part-height masonry partitions as well the possibility of shear failure in columns due to interaction with masonry frames with openings.

In addition, another structural weakness identified during the site visits is the presence of suspended timber floors without adequate bracing to the ground floor main lounge as seen below. This also creates a diaphragm discontinuity at the ground level, imposing additional demands on the concrete floors to transfer loads between the different array of infill walls between the ground floor and the basement.

Figure 7 - Timber sub-floor bracing below lounge areas.

The vehicle access ramp was built as a separate contract from the building and its construction is unknown. We understand that the building itself has concrete walls in line with the carriageway to support the soil supporting the ramp. However, details about the retaining wall construction to the sides are unknown. It will be assumed that reinforced concrete columns with reinforced masonry hollow concrete blocks were provided to support the sides of the ramp as jointing is visible on the exposed faces.

4.3 Site Visits and Lidar Scanning

The WSP team visited the building 4 times during June-December 2022. A 3D point cloud of the exposed architecture was carried out to understand the current as-built condition of the building. With this 3D scan, a Revit model of the structure was built and completed with on-site measurements of structural elements and then exported into ETABS for analysis as required. The point cloud data and the Revit model can be seen in the following plates:

Figure 8: 3D BIM model of the original Chateau building.

Figure 9: Lidar Point Cloud model with cross section through the building

4.4 Building History

The original hotel has undergone several alterations and additions over the years. Below is a chronological sequence as seen from drawings and historical and aerial pictures (www.retrolens.com, Whites Aviation, www.heritage.org.nz, etc.):

- Pre-1947: Several alterations occurred before the first aerial image that includes the chateau. These are:
  - Addition of back of house building at rear of chateau.
- Between 1947 and 1951:
  - Two staircases to the south of the original kitchen and to the south of the Ruapehu Lounge.
  - Removal of Tank tower and infill of southwest wing internal stair windows.
  - Demolition of original back of house building and replacement with new back of house with covered vehicular access.
  - Removal of central chimney
- Between 1951 and 1959:
  - Addition of north-western single storey workshop at basement level (c. 1950).
  - Extension of existing laundry into "a la carte" restaurant.
- Circa 1969-1970:
  - Addition of Tongariro Conference room on north elevation in front of Porte-cochere.
- Circa 1976:
  - Upgrade of rooms
  - Kitchen upgrade
- Circa 1980:
  - Construction of Spa pool in basement
- Circa 1995:
  - Demolition of 1950's back of house building and erection of two storey addition comprising the T-Bar, back of house and offices upstairs.
  - New roofed loading bay at the rear of the property.
  - Addition of pedestrian entrance at basement level on SH48 elevation to current games room.
  - Refurbishment of rooms
- Circa 2000:
  - Upgrade of kitchen and conversion of the above offices into accommodation.
- Circa 2002-2003:
  - it is understood that there has been a refurbishment of the original rooms including the third floor.
  - An alteration of this size would expect to have considered upgrading hold-downs and bracing capacity for the top storey accommodation.
- Circa 2005:
  - New wing 5-storey extension to the West with ground floor extension of behind the Ruapehu Lounge.
  - Addition of Atrium area at the back of the Ruapehu Lounge.
- Unknown date: several modifications to the basement walls and excavations of ground/fill down to the foundation pad level.

A summary of the current layout of structures can be seen in the photo below:

Figure 10 - Current Aerial View and Layout of Property, extent of DSA highlighted in green – image from https://maps.ruapehudc.govt.nz/intramaps90/?project=Ruapehu

Between 1947 and 1951, two staircases were added to the rear of the property. Nowadays, only one staircase remains exposed to the southwest, while the other one is concealed within the connection to the new 2005 wing. They are comprised of concrete frames with masonry infill walls (assumed) with the top of these staircases finished in light timber framing. The southwestern staircase shows some minor signs of movement (new mortar has been placed in the past) and it is disconnected to the original 1929 concrete structure. Overturning of this slender structure can be considered a potential structural weakness and pounding due to torsional effects is likely. We understand that there are no available drawings of this structure and we have not been able to investigate reinforcement detailing during our scanning exercise.

The original 1929 portion therefore has the following structural units in proximity:

- 1950s Workshop to the West Elevation: Roof is supported on the original Chateau structure, and it is flexible with no seismic gap provided.

- 1970's Tongariro Conference Room: Independent structure in front of the existing sun piazza. Flexible connection at roof level exists with minimal seismic gap between structural elements.
- 1995 Two Storey Extension to the south: this extension floor and roof is supported on the original Chateau structure. Seismic gap is minimal.
- 1950s external staircases: These structures are independent of the original Chateau structure with minimal seismic gap.
- 2004 Atrium extension: The atrium structure sits in front of the original chateau support structure portion of the roof is supported on the original concrete beams. Seismic gap is minimal.
- 2004 5 Storey Extension: This new wing is independent from the original structure with a 300mm wide seismic gap at the roof level.

4.5 Gravity Load Resisting System

The gravity load resisting system is comprised of in-situ reinforced concrete frames for the ground, first and second floors. These frames are supported on shallow pad foundations. The detailing of the reinforcement in these frames correspond to the pre-1931 practices with plain bars with hooks and stirrups sparsely spaced, curtailment of longitudinal reinforcement and infill masonry (Figure 12). External infills are comprised of a cavity wall with one wythe framing against the concrete frame while the external wythe is external and is not framed against the concrete. Internal infills are also present throughout the building with single wythe thicknesses. Four types of bricks have been identified during our site visits, please refer to the laboratory report for geometry and properties.

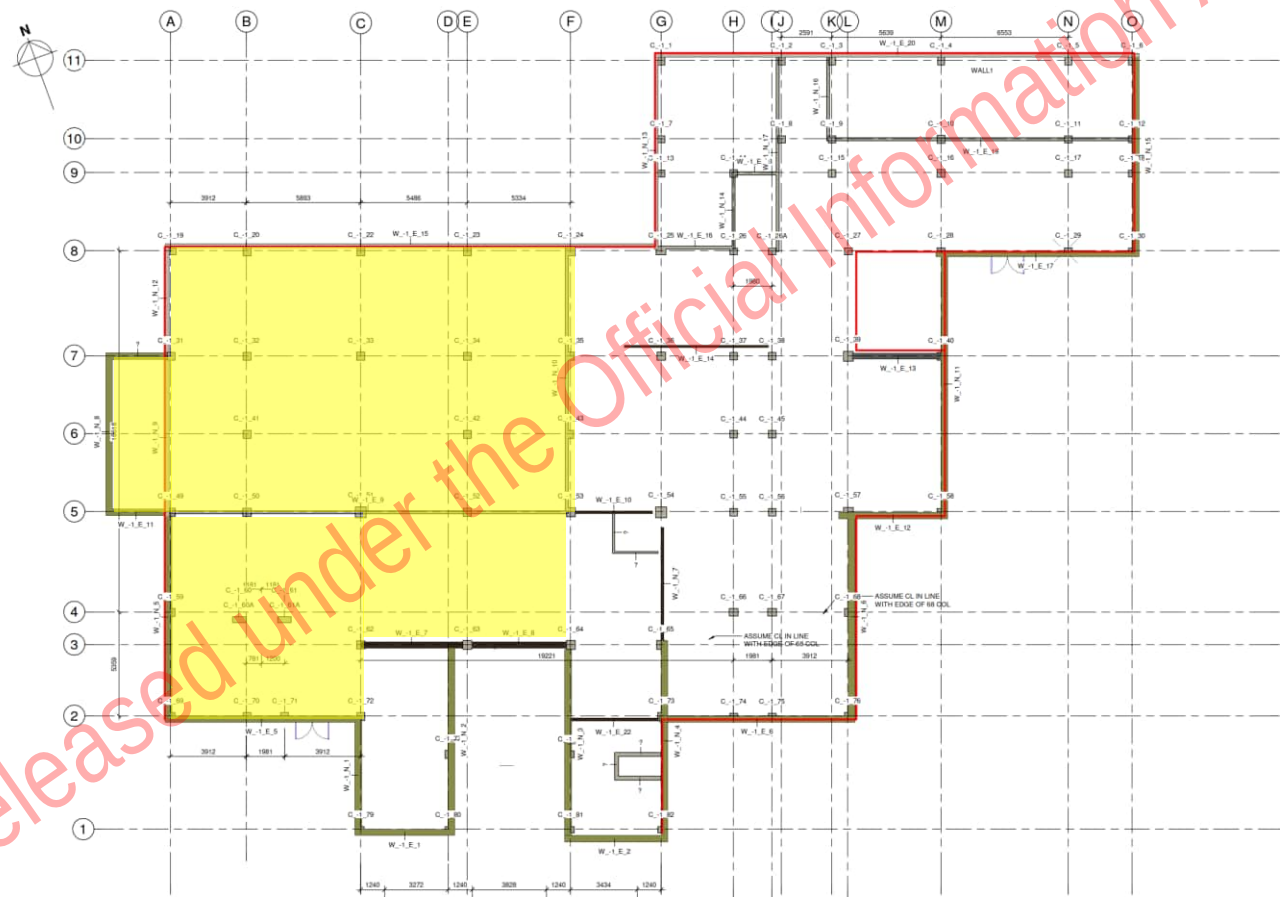

Figure 11: Plan showing structure ground floor level indicating masonry and masonry infill walls and with timber floor in yellow

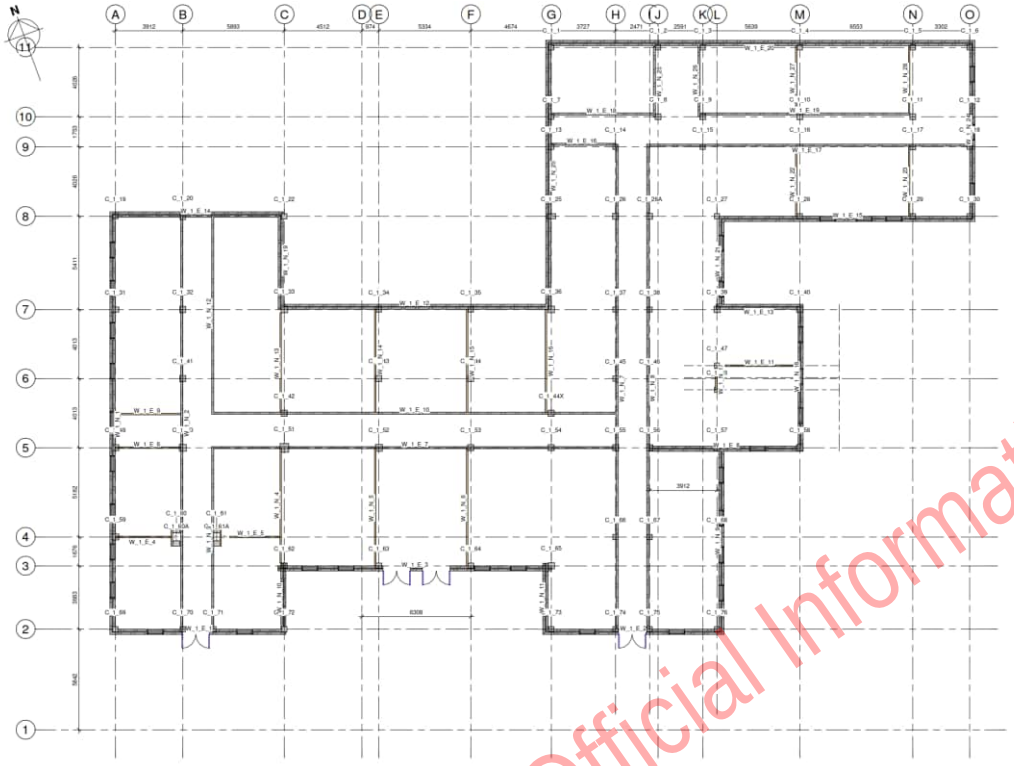

Figure 12: First floor structural plan showing infill walls and frame columns (2nd floor similar)

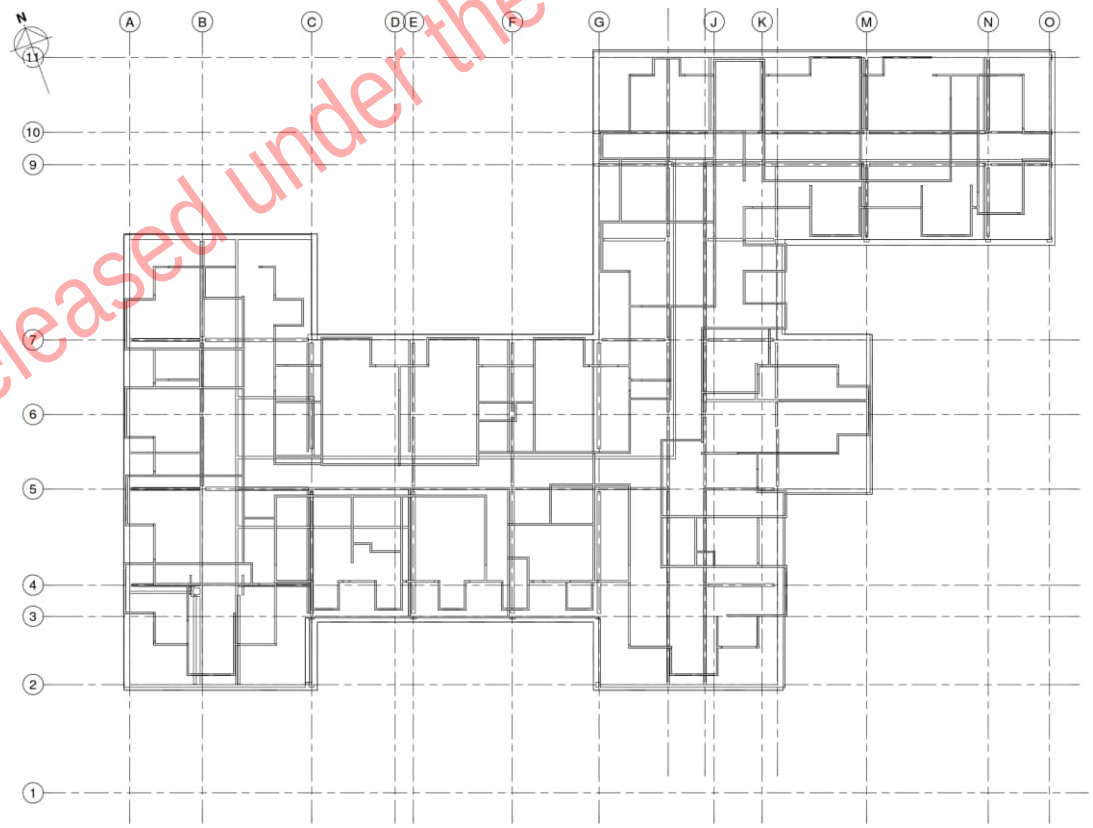

Figure 13: Third floor plan showing layout of light timber partitions

The building comprises four suspended floors with inter-storey heights of 3.2, 5.2, 2.9 and 3 metres for the ground, first, second and third respectively. Above the second storey, the main columns cantilever to provide support to steel beams at the third floor that support the timber floor structure of the third level and roof (Figure 13). Around the perimeter, a tie beam is present where the roof structure is supported as well. Internal light timber framing partitions complete

the support for the roof space (Figure 14) Connections between the steel beams and the columns are unknown.

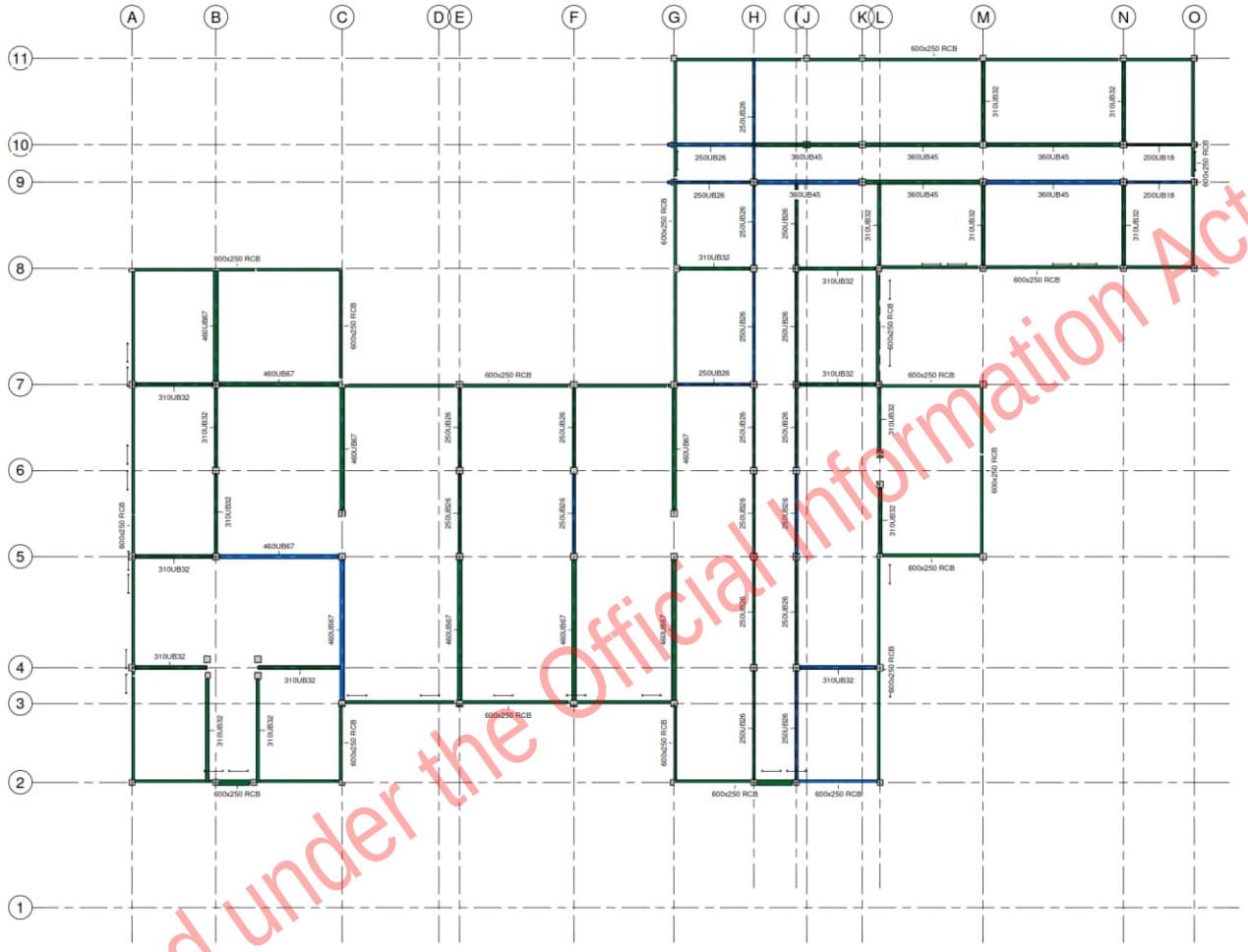

Figure 14: Third floor plan showing RC columns and steel beam arrangement.

The entrance port-cochere gravity structure is comprised by reinforced concrete columns and masonry walls that retain the main access ramp and reinforced concrete frames at the main entry. These frames show Tuscan/Doric columns in a double arrangement and lightly reinforced beams. Some of these columns also only show as steel pipes as the only reinforcement according to the drawings.

The basement is mostly comprised of a reinforced concrete slab on grade. For the ground floor, an in-situ reinforced concrete slab provides the gravity support for floor loads. However, the main lounge area has a timber suspended floor that consists of flooring and timber joists (140x45 at 400crs), spanning onto 2x190x45 bearers supported by timber posts at 2m centres that sit on small unreinforced concrete pads (600mm square) in a storage area at the basement level (highlighted yellow area).

The first-floor level and second floor have an in-situ concrete slab with reinforcing detailing for one-way spanning in the short direction. It is assumed that secondary reinforcement spans the longitudinal direction. Although the original drawings indicate the floor to be 5 inches in thickness, we have measured on site 170mm and 200mm for the first and second floors respectively. We have used the second values as they will bring more conservative results in terms of seismic demand, but additional measurements are recommended in the future, as this could be a screed which does not extend over all areas. Above the third floor, there are steel beams spanning between cantilevering concrete columns. These steel beams receive the timber bearers and joists for the third floor. Lightweight timber framing provide support for the roof structure and all the plant concealed in it.

The roof structure is comprised of timber rafters approximately 150x45mm at 750mm centres that form an A frame with a bottom chord. The ceiling for the rooms below is supported on the bottom chord of the A frames. The A frames react against the perimeter concrete beams at the third floor, and they are also supported by light timber framed walls above the third storey. There are also large plant elements supported on these frames like water tanks as well as walkways and walls. The trusses support sarking and a corrugated iron roofing that is depicted as asbestos originally, but we understand that this has been replaced . An outline of the roof structure can be seen in the image below:

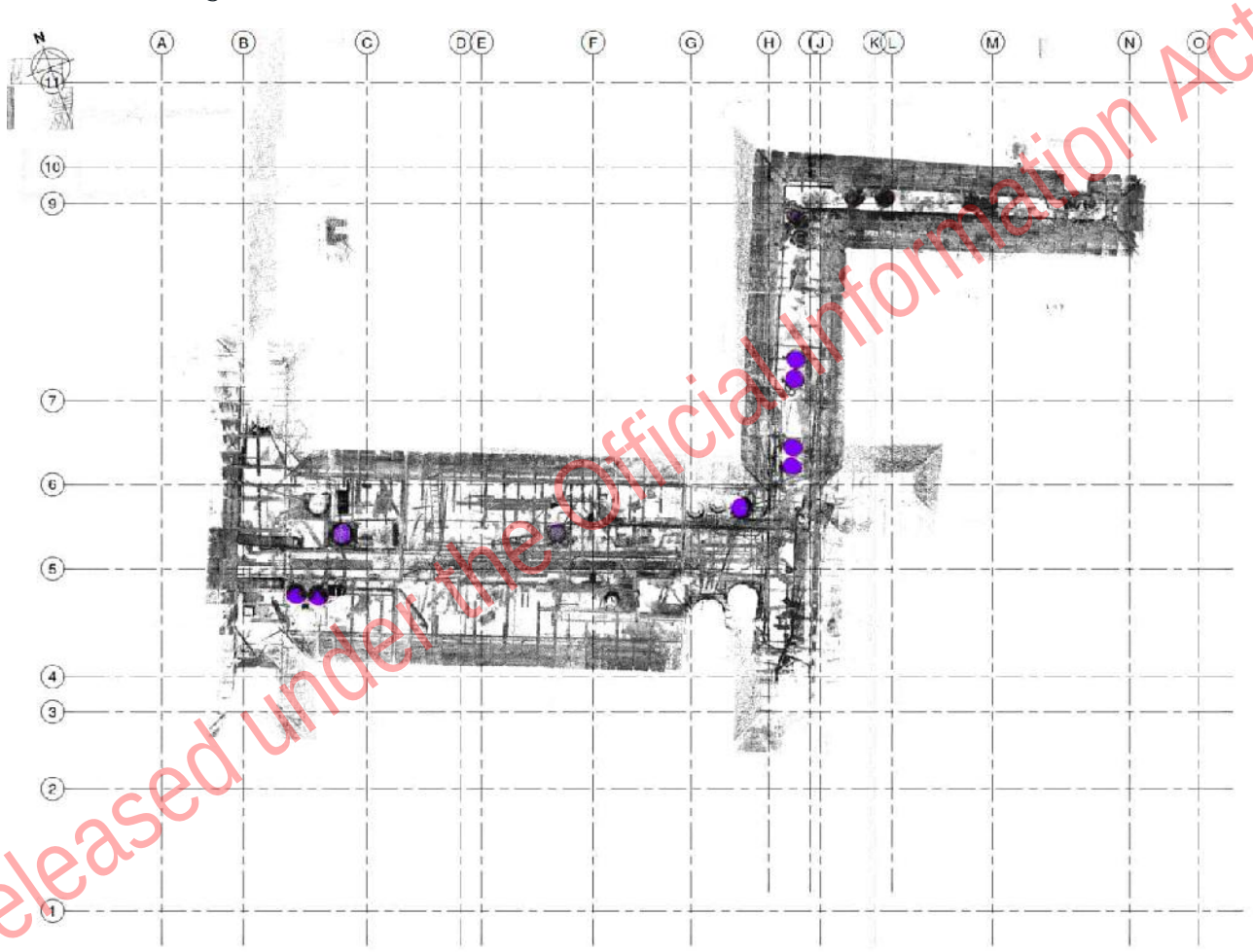

Figure 15: Roof structure plan with some of the existing water tanks highlighted in purple.

4.6 Lateral Load Resisting System

At the fourth floor & roof level, the lateral load path consists of light timber framed (LTF) shear walls in both directions that restraint the roof at the ceiling level. The upper roof structure is transversely restrained by the timber truss members and diagonals present throughout the roof space. Longitudinally, the upper roof structure is restrained by longitudinal diagonals present through the roof space and diaphragm action of the sarking in the roof plane. At the third-floor level, the perimeter of the building presents a reinforced concrete beam. This beam receives the roof rafters into a bottom plate that is fixed with cast in-situ M12 bolts to the beam at around 600mm centres. A cross section of the structure through the central body of the building can be seen in Figure 16. Diaphragm action will be minimal at the third floor ceiling level.

The lateral structure above the second floor comprises of cantilever reinforced concrete columns in both directions, with steel beams framing into them. Given the connections between the steel beams and the concrete columns are unknown, no framing action with these has been assumed. In addition to this system, the presence of masonry infills will contribute to the lateral strength in both directions, although framing of masonry walls into the frame columns and steel beams is

not known. At the third-floor level, flexible diaphragm action is provided by the timber floor and the arrangement of steel beams.

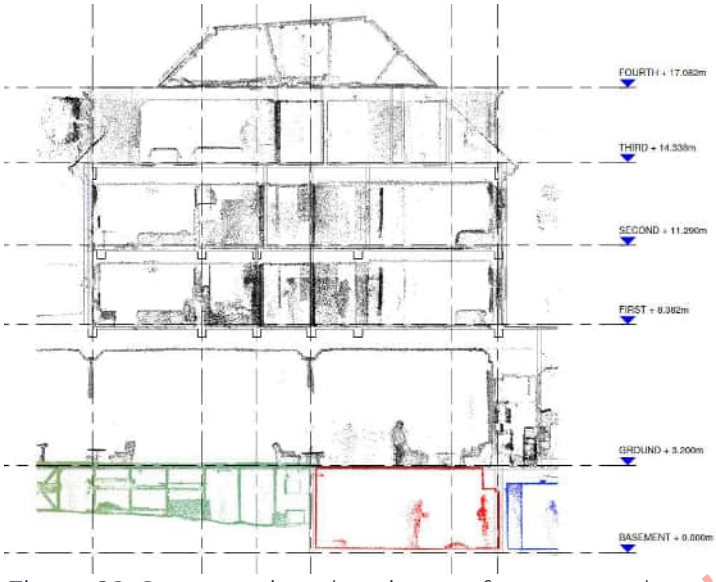

Figure 16: Cross section showing roof structure above third floor and vertical irregularity at ground floor and offset of walls at corridor

For the ground floor, first and second levels, the lateral system is comprised of in-situ reinforced concrete moment frames with masonry infills in both directions. While the arrangement of infills is limited mostly to the exterior at the ground floor, there is a regular layout of infill partitions above the first and second floors. In general, all the reinforced concrete frames were design for gravity loads with typical detailing of the era, with strong beam weak column arrangement and no beam-column joint reinforcement. An elevation through a typical frame can be seen in the following image:

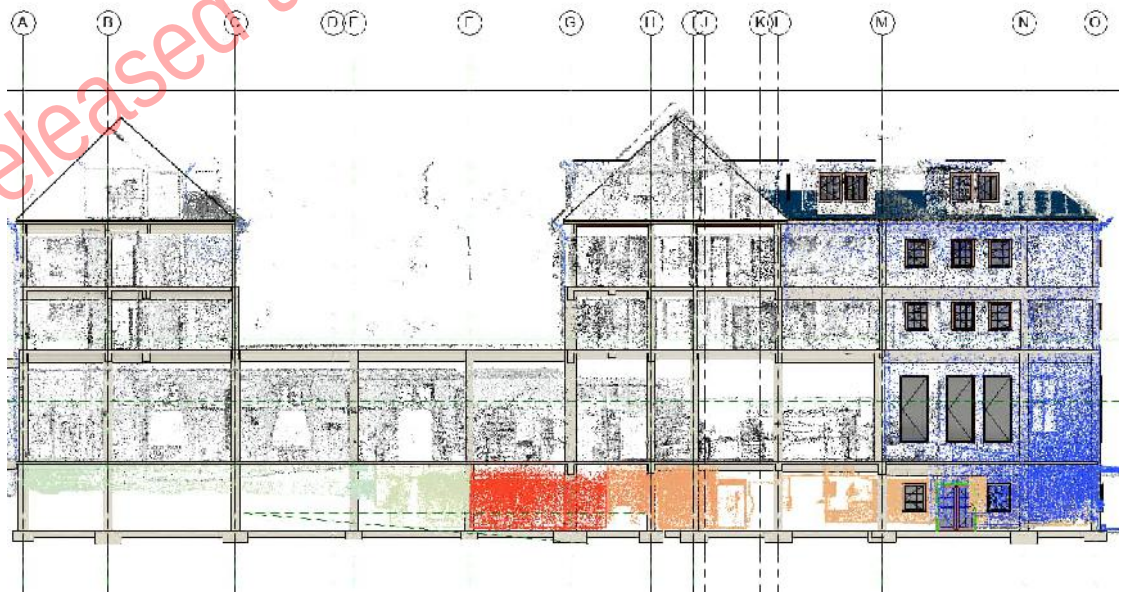

Figure 17: Longitudinal section through grid 11 showing Moment frames and infill walls in point cloud.

The first-floor beam arrangement as shown in the plate below (Figure 17). As a rule, green beams sizes have been assumed, while orange beams are available on the drawings and blue beams have been measured on site. Rebar scanning for beams was only possible at the basement level due to access issues at the rest of the floors.

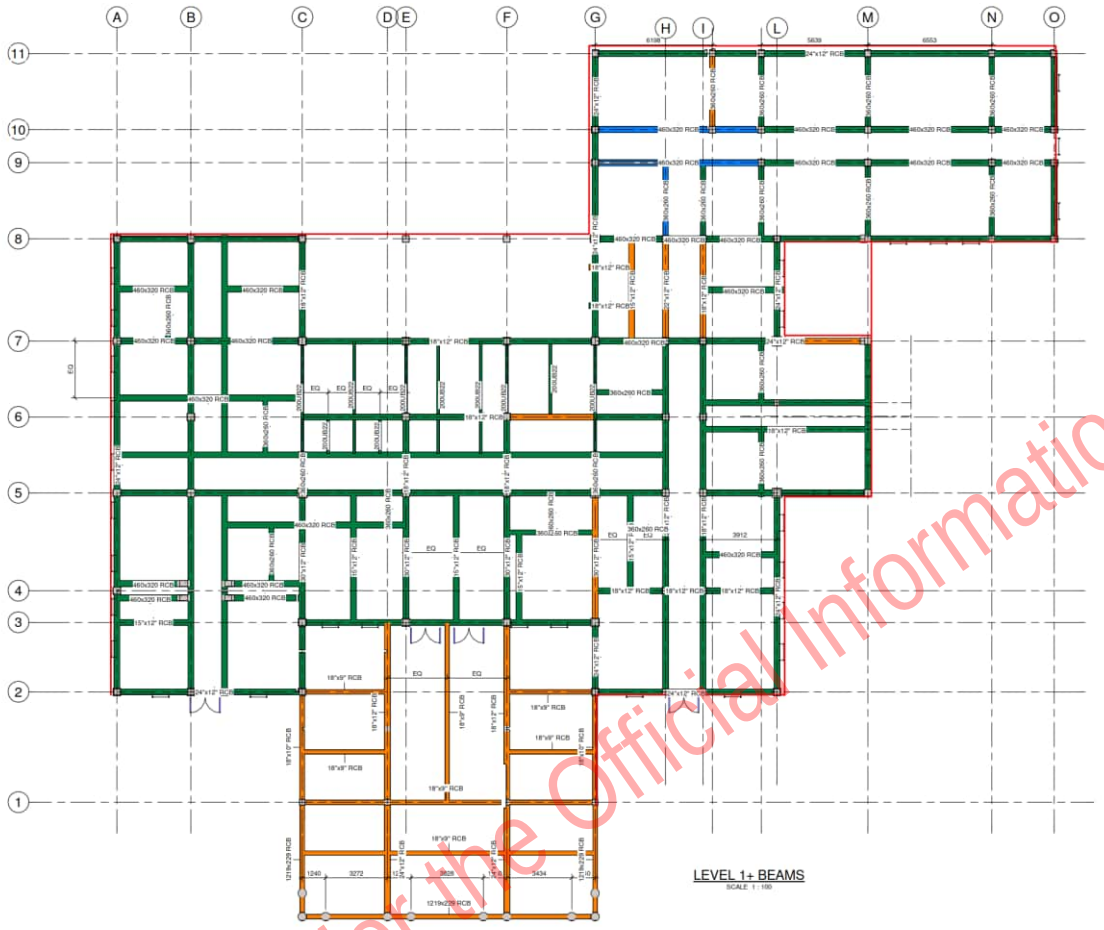

Figure 18: Level 1 reinforced concrete frames arrangement

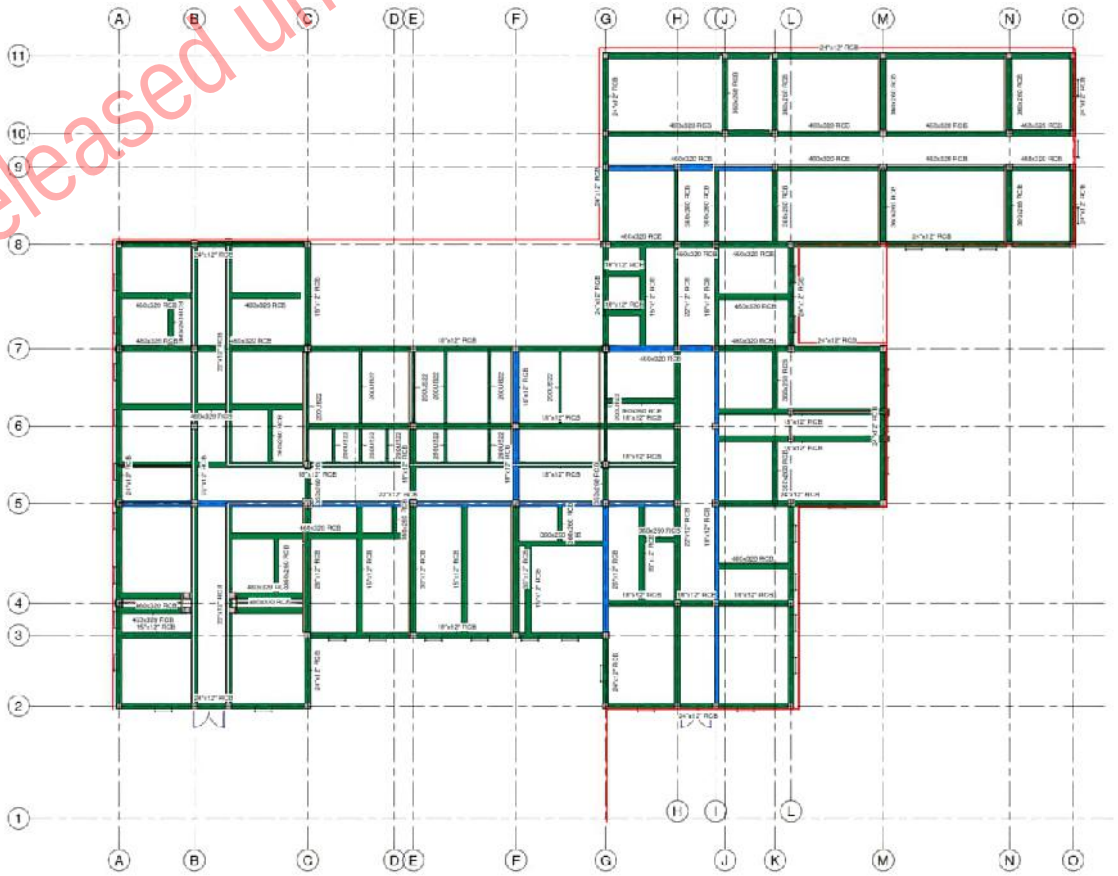

Figure 19: Level 2 reinforced concrete frames

Diaphragms for the ground, first and second floor levels are comprised of a reinforced in-situ concrete slab with plain bars. The slabs are depicted as unidirectional in the original drawings with a main and secondary reinforcement (principal and secondary directions).

As discussed in the previous section, in the main lounge area the diaphragm will be timber supported on all sides laterally by concrete beams. Limited longitudinal bracing is provided by subfloor diagonal bracing.

Our most accurate interpretation of the as-built building can be found in the attached drawings in the appendices issued from our Revit Model. We have also noted which properties have been assumed based on available information, so the drawings also portrait as a list of assumptions of the as-built layout.

4.7 Secondary Structural and Non-Structural Elements

During our site visits the following secondary structural and non-structural elements that can pose a safety risk were identified.

4.7.1 Internal Stairs

There are two set of original internal stairs in the building: main stairs by the elevator shaft and secondary back of house stairs at the southern wing.

The main internal stairs are cast-in situ reinforced concrete and span between storeys with an intermediate landing supported in the neighbouring beams. From the mid-second to the third floor a timber flight is provided. An elevation is provided in the photograph below:

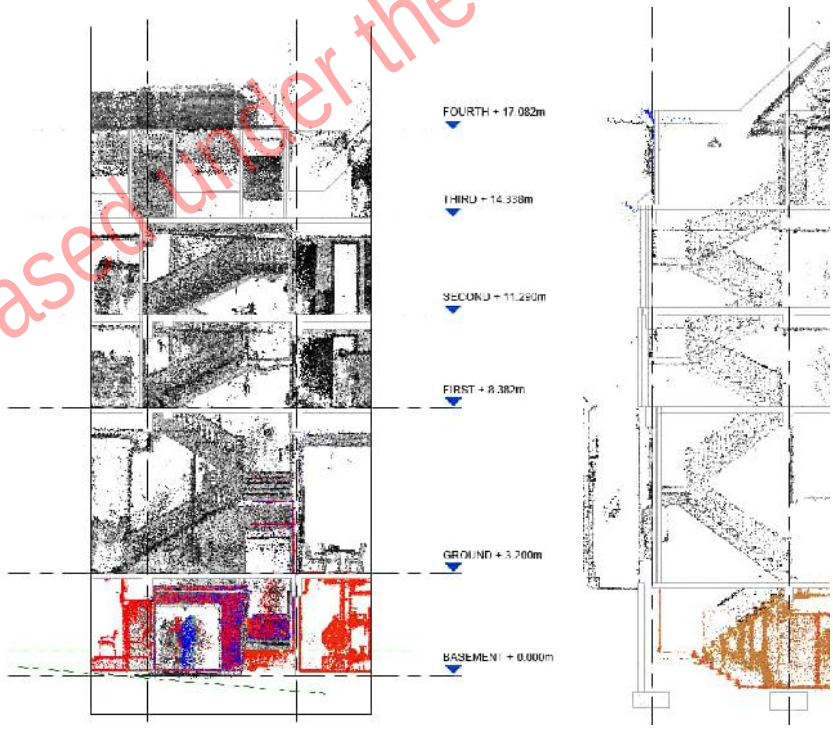

Figure 20: Internal Main Stairs Elevation (left) and Internal Service Stair Elevation (Right)

At the southern wing, the original service stair remains (Figure 20). This stair is also reinforced concrete cast in-situ with an intermediate landing. There are no sliding details available to accommodate any inter-storey drifts.

4.7.2 External Stairs

In the 1950's, two external stairs have been added to the building to provide means of evacuation. Their main gravity and lateral structure is comprised of in-situ reinforced concrete frames up to the second floor. Masonry infills provide additional lateral support to

the side and elevations. The structure above the third floor is timber framed and supported on the main structure. These stairs are not attached to the main building and visible movement is observed on the west unit. The eastern unit remains concealed from the exterior in between the 2004 eastern wing addition.

Figure 21: External west staircase on southern wing.

4.7.3 Vertical transport

The building has one lift. The lift shaft extends from the basement to the roof structure. From the basement to the second floor, the shaft is comprised of masonry walls that are supported on the floor slabs at each level. Above the second floor, it is understood that a steel structure is erected from the third level to provide gravity and lateral restraint via a steel portal frame and masonry infills. A photo from the top of this structure where the lift machinery is can be observed in the following photograph taken:

Figure 22: Elevator structure at roof space

4.7.4 Heavy plant at roof level

Several water tanks (historical and current) are present at the roof space level. These are supported on non-engineered supports and lack any lateral restraint. Newer HDPE tanks of about 1000L capacity are present to service the toilets of the floors below.

Figure 23: Central roof space with heavy unrestrained water tanks.

4.7.5 Chimneys

There are three main chimneys that can be considered a life safety risk in the building. The main elevation of the building portraits two 1.5mx1m chimneys that rise approximately 3m above the roof level. The Eastern of these chimneys is aligned with the main fireplaces in the main lounge and original reading room. The structure for this chimney consists of two bounding concrete columns around two square masonry flues supported at each storey. These flues then converge in a single flue that elevates above the third floor and roof structure. A photo of the masonry structure at the roof level can be seen in the photograph below:

Figure 24: Main chimney structure above roof level from main lounge fireplace – Photo (Left), Point cloud elevation (right)

The western chimney at the main elevation is comprised of a lightweight metal replica and does not continue below the roof ceiling level.

At the rear of the southern wing, a smaller square column cantilevers as well from the main roof. This chimney is approximately 0.8mx1.2m and cantilevers 4m above the roof level (Figure 25):

Figure 25: Masonry Chimney at southern wing

4.7.6 *Main Lounge Ceiling*

The main lounge ceiling is a heavy, decorative plaster ceiling. supported by timber framing fixed to the concrete floor above. During the site inspections, we identified that this space was also storing spare pipes and other elements and any diagonal bracing was not observed.

4.8 Basement and Podium

The basement has partial soil retaining to the south, but mostly is level with the street level to the West, North and East. There are no podium structures, and the retaining height is limited to the basement.

Locations of retaining walls are depicted in the extract from the original drawings below. Retaining walls along the southern elevation of the building are comprised of cast in-situ reinforced concrete with nominal reinforcement (Figure 26).

Around the southern perimeter of the building there are 6in thick reinforced concrete cast in-situ walls (red below). These walls are supported by surrounding concrete frames and span two way. Reinforcement is about 3/8in bars at 8in centres both ways.

In addition, internal perimeter walls exist (green below)) where the ground has not been fully excavated (orange below). Construction of these walls is similar to the perimeter walls and the retaining wall span two ways between bounding frames.

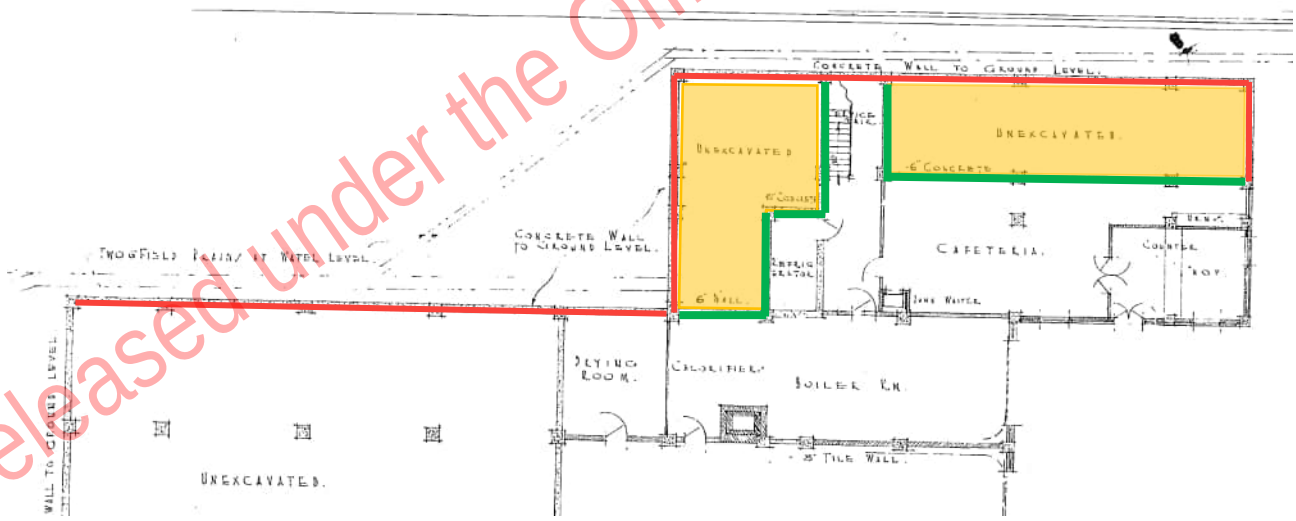

Figure 26: Retaining walls at basement level

Retaining of the main access ramp also occurs under the main entry for the main building, but these loads will be self-equilibrated for gravity with a small variation during earthquake. Additional restraint from the ramp structure acting in plane might occur at this elevation as well.

4.9 Building Irregularities

The following irregularities have been found in the building:

(a) Vertical Irregularities

(i) Vertical discontinuity of timber shear walls for the roof level. Vertical discontinuity of infill walls at ground floor.
(ii) Ground floor height is almost 5m and without as many infill walls, thus creating a vertical stiffness irregularity.
(iii) At the ground floor there is a weak storey, since it is double height and the columns in the lounge area do not frame at the ground floor.
(iv) There is vertical irregularity above the second floor.

(b) Horizontal irregularities:

(i) There are two column offsets at the first floor.
(ii) There is offset in lateral resisting walls between the first floor and the ground floor.
(iii) The building is torsional sensitive.

It is noted that the structural lateral system expected to exhibit torsional behaviour in both directions. Premature collapse of infills out-of-plane can create additional torsional discontinuities and irregularities. These inherent irregularities and behaviour can bring forward the point of structural failure.

4.10 Building Condition

The original Chateau Tongariro is almost 100 years old. During the site investigations, we identified areas where rebar rust was evident, with concrete spalling and staining. Based on the visual and intrusive investigations we can confirm 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.

Given the above, we suggest that prior to seismic strengthening, a full condition assessment be considered.

5 Assessment Criteria

5.1 Material Properties

The following material properties were used during the process of this assessment. These depart from the in-situ testing and historical properties as a mixed of statistical combination and engineering judgment of the authors in this point in time. The assessment makes provision for possible variability of materials in a qualitative manner on how these will affect the numerical results obtained.

Table 1 – Material Properties Used for The Assessment

Material	Nominal Strength	Probable Strength	Justification
Concrete		20MPa	Refer to Laboratory Testing in Appendix A and Calculations in Appendix B. Some areas at the basement have lower capacity, but for the superstructure we expect it to be better.
Reinforcement		f_y= 280MPa f_u= 475MPa ε_{su}=0.1 ϕ_0=1.25	Table C5.4 of C5 Guidelines. 310MPa obtained in tests, 410MPa Ultimate..
Structural steel	210MPa		Table C6B.5 of C6 Guideline
Masonry – exterior walls below 1st floor		f'_b= 7.3MPa f'_j= 26.4MPa f'_m= 8.9 MPa E_m = 1783 MPa Weight = 4.2 kN/m^2	Refer to Laboratory Testing in Appendix A and Calculations in Appendix B. Upper bound properties can double capacities of struts.
Masonry – exterior walls above 1st floor		f'_b= 26.7MPa f'_j= 26.4MPa f'_m= 23.5MPa E_m = 4708 MPa Weight = 4.4 kN/m^2	Refer to Laboratory Testing in Appendix A and Calculations in Appendix B. Upper bound properties can double capacities of struts.
Masonry – internal walls (infill and not infill)		f'_b= 47.6MPa f'_j= 26.5Pa f'_m= 36.3MPa E_m = 7253 MPa Weight = 1.8 kN/m^2	Refer to Laboratory Testing in Appendix A and Calculations in Appendix B. Upper bound properties can double capacities of struts.

5.2 Seismic loading

5.2.1 Design life

The facility is assessed for seismic actions based on the assumed future design life of 50 years.

5.2.2 Importance level

The seismic loading determined by NZS 1170.5:2004 is a function of importance level (IL) of the facility in accordance with AS/NZS 1170.0:2004. In accordance with the New Zealand Building Code, this building is classified as an IL2 or Normal Importance Level building.

5.2.3 Seismic loading spectrum

The structure has been assessed following the seismic actions prescribed on earthquake loading standard NZS 1170.5:2004. The parameters used for the assessment is as per Table 2.

Table 2 – Seismic Loads Parameters

Parameter	Value	Comments
Site subsoil class	C or D	Local Geotechnical Investigation – requirement to check for both subsoil classes C and D.
Z	0.27	Seismic hazard factor for Ohakune
Return period factor, R_u	1.0	Importance Level 2 for 50 years design life
Near fault factor, N(T,D)	1.0	> 20 km from nearest major fault.

The Sp factor has been considered as 1 for vertical actions in lieu of more detailed behaviour of the structure. For horizontal actions, 0.925 has been taken. Elements affected by vertical excitation include beams supporting columns on the first floor.

The balconies are small and is unlikely to generate an issue due to vertical earthquake excitation.

5.3 Seismic Weight

The seismic weight was calculated as per NZS1170.5. We have included the following live loads depending on the use of the area as shown in Table 3. In addition to this, the structures self-weight was applied to the walls and the roof and area factors have been applied.

Table 3 – Imposed Seismic Mass

Floor Level	Imposed Weight, Q_i	Combination Factor, ψ_E
Roof	0.25kPa	0.0
Corridors	4kPa	0.3
Bedrooms	2kPa	0.3
Offices	3kPa	0.3
Commercial Kitchen	5kPa	0.3
Dining area	2kPa	0.3
Stairs	4kPa	0.3
Light Traffic area	2.5kPa	0.3

Super dead loads such as screed, ceiling, services, partitions, etc. have been included in the seismic mass. Total seismic mass of the building is approximately 51MN/g.

5.4 Assessment Methodology

In general, we have followed the displacement-based methodology of the guidelines as set out in section C2.4.2 with the guidance of the C5 "Red Book". NZS1170.5 spectra has been used for the main building, while parts and portions coefficients have been used to assess secondary and non-structural elements and the lightweight top floor.

For the main building, the analysis approach taken was to carry out a Simple Lateral Mechanism Analysis or SLaMA of the concrete frame and infill structure in both directions. This allows the non-linear performance and post-elastic behaviour to be assessed. A separate analysis of the bare concrete frames and infills has been carried out. The interaction between infills and frames has been assessed independently by the analytical means in chapter C7. This analysis has been focused on the ground floor where a soft storey is likely to form, assuming a column sway mechanism will form as it is the most complete set of information. This result, therefore, will be an upper bound limit.

A separate analysis of the building bare frame including floor diaphragms was carried out using ETABs according to the Non-Linear Static procedure (NSLP). This coupled with the SLaMA outputs provided an understanding of effects of torsion on the building capacity. For the ETABS model, modal analysis has been carried out to understand the structural behaviour of the structure.

The timber roof structure above the third floor was assessed based on a force-based assessment under chapter C9, with the demands from parts and components from NZS1170.5.

Infills in the out of plane direction, chimneys and other elements have been assessed based on the demands from parts and components from NZS1170.5.

6 Detailed Seismic Assessment results

6.1 Summary of Results

The following table summarises the results obtained in our analysis.

The following results have been obtained for the Chateau Tongariro during the exercise of this DSA:

Table 4 – Assessment Results for Individual Building Components and/or Systems

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. Early onset of joint failure.
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
Lift 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

6.2 Structural Weaknesses (SWs), Severe Structural Weakness (SSW) and Critical Structural Weakness (CSW) Identified

A structural weakness (SWs) is any part or portion of the structure that does rate less than 100%NBS. Thus, all the items on table in the previous section represent a structural weakness. No Severe Structural Weaknesses (SSW) have been identified since external columns subject to joint shear failure do not exceed the axial load ratio. The Critical Structural Weakness (CSW), this is, the lowest scoring structural weaknesses are considered to be the exterior URM walls that could

collapse into evacuation routes. Main Concrete gravity frames can also be considered as a CSW since their rating is also proximate.

6.3 Analysis Summary

6.3.1 Reinforced Concrete Frames

Our analysis show that the reinforced concrete frames lack strength and stiffness. Infill contribution enhances the response and stiffness at short periods, but they have limited deformation capacity. Bare frame response has the same rating as the short period response with an assessed capacity of 30%NBS(IL2).

The reinforced concrete frames are governed by shear failure of beams at locations were infills are interrupted with openings. These unfavourable effects will occur at very low building drifts (0.5%) and therefore the rating has been established to be between 15%NBS(IL2) and 30%NBS(IL2).

A mixed beam sway mechanism is possible with joint hinging for the bare frame. Therefore, deformation capacity is capped at 1% drifts for the ground floor.

The results also include the entrance port-cochere, with columns that fall outside the assessment guidelines. We have assumed that stability of the port cochere is maintained via diaphragm action of the terrace area.

6.3.2 Masonry Infills – Out-of-plane

Our analysis show that the external masonry infills are rated less than 15%NBS(IL2). The main reason for this is that both the internal and external wythes are heavily perforated thus minimising any beneficial arching effects from the bounding frames.

6.3.3 Foundations

A simple check from the output of the ETABS model show that foundations will be between 30-40%NBS(IL2) due to insufficient size and insufficient strength.

6.3.4 Concrete Floor Diaphragms

Floor diaphragms for the first and second floors are irregular and the connection between the south wing and the main wing of the building will likely experience some damage due to torsional demands and lack of beam continuity. A simplified analysis was undertaken to check chord capacity under building overstrength actions from the ETABS model.

6.3.5 Chimneys and URM Parapets

The main lobby chimney and the chimney at the southern wing are unreinforced masonry and will exhibit toppling failure with probable collapse into the building or outside the building into egress routes.

6.3.6 Third Floor and Roof Light Timber Framed Structure

Our analysis shows that this portion is 40%NBS(IL2), based on a force-based procedure and analysing each wing separately in both directions. Noted that unrestrained heavy water tanks are present at this level.

6.3.7 Internal Stairs

The internal stairs at the main lounge and at the southern wing are expected to rate the same as the building at about 30%NBS(IL2). Column shear failure can be an issue at the lower levels where the landings are framing against columns. The locked-in stairs are expected to act as unintended braces, and will be damaged as building drift increases.

6.3.8 External Stairs

The external stairs have been rated to be 25-35%NBS(IL2) based on overturning equilibrium of the foundations assuming shallow foundations are present.

6.3.9 Beams supporting gravity columns at first floor

A simple check of the shear demands at the beam 36-54 show that the additional shear demands from the columns above will sit around 40%NBS(IL2).

6.3.10 Main Lounge ceiling

WSP has not undertaken a specific assessment and survey of ceilings. However, the ceiling at the main lobby shows signs of movement and a visual inspection of the plenum shows that it is used to store spare materials and has minimal bracing. We would expect this element to rate low and be a safety hazard in the open areas along the evacuation routes where furniture is not present.

6.3.11 Heavy plant and equipment at roof level

WSP has not undertaken a specific assessment and survey non-structural plant. We would expect toppling, sliding and potential collapse to the rooms below.

6.3.12 Lift shaft and motor

The support structure of the lift cantilevers above the third floor. We recommend further assessment is done in this structure but given the construction and connections we expect that this structure could rate between 30% and 40%NBS(IL2) and may compromise the operation of the elevator.

6.3.13 Porte-Cochere ramp structure

Although information available is insufficient to quantify the strength of the ramp to the building, we estimate that this element will rate between 40 and 50%NBS(IL2). The structure can be considered a life safety hazard as its collapse can injury people congregating outside the building during earthquake shaking and represents one of the main escape routes for the building.

7 Conclusions

The results of the DSA will indicate the building's *Earthquake Rating* for the **Chateau Tongariro** to be **15%NBS (IL2)** in accordance with The Guidelines. Therefore, this is a **grade E building** with an associated **very high life safety risk** in accordance with the NZSEE grading scheme (**Error! Reference source not found.**).

A building with an *Earthquake Rating* less than 34%NBS when assessed in accordance with the Version 1 Guidelines (the 'Red book') 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 by the NZSEE. **The Chateau Tongariro therefore falls within the criteria that could categorise it as an EPB.**

The critical structural weakness for (CSW) the building is the unreinforced masonry infills that can fail out of plane internally or externally into occupied areas of the building alongside possible shear failure of concrete frames due to the interaction with infills.

Recommended structural strengthening for the building is discussed in the "Next Steps" Section below

8 Next Steps

8.1 Condition Assessment

It is recommended that a condition assessment be carried out to inform any strengthening and repair. Any further intrusive investigations required to inform strengthening could be carried out at the same time.

8.2 Heritage Considerations

The Chateau Tongariro is listed on the New Zealand Heritage List as a Category 1 Historic Place. Therefore, any strengthening works should look to minimise impact on significant heritage fabric to avoid degradation of the building's heritage values. All works should be planned, designed and executed in accordance with the *ICOMOS New Zealand Charter for the Conservation of Places of Cultural Heritage Value 2010* (Appendix E)

Both the *ICOMOS NZ Charter* and guidance produced by Heritage New Zealand Pouhere Taonga strongly recommend that Conservation Management Plans be prepared for heritage buildings. It is possible that a Conservation Management Plan for the Chateau Tongariro exists. However, at the time of preparing this report, no Conservation Management Plan has been made available to the authors. If no Conservation Management Plan exists or can be found, then one should be obtained prior to the next stage of engineering design work commencing. This must prepared by an appropriately qualified and experienced conservation architect or heritage consultant, and must include a detailed assessment of significance of the building and its fabric, and present policies that will guide the works in accordance with the *ICOMOS NZ Charter*.

The Chateau Tongariro is scheduled as a Group A Heritage Building in the Ruapehu District Plan. This means that the Heritage Rules in Chapter HE3 of the District Plan apply. Under Rule 3.2.1, changes to the interior of a Group A Heritage Building are permitted if the interior is not specifically listed. However, any works affecting the exterior of a Group A Heritage Building that do not constitute Repairs and Maintenance will require a Resource Consent under Rule H3.3.2.

In preparing the strengthening options below we have focused our efforts on avoiding or minimising the work to areas where, based on our understanding of the building and the chronology of change presented in section 4.4, we are confident that the building fabric has heritage significance; and/or where the fabric is protected under the rules of the Ruapehu District Plan. These areas include: the exterior of the building generally; and the internal ornamental finishes at the ground floor in the entrance lobby, tea and dining areas, and the entry floor mosaic.

It is critical that a conservation architect or heritage consultant be included in the design team to advise on further development of strengthening options and associated design work.

8.3 Strengthening Recommendations to 67%NBS(IL2)

Given the level of intervention that is necessary to bring the building above the 33%NBS(IL2) minimum required level, we recommend that strengthening to a level of 67%NBS(IL2) is carried out.

To achieve this, we propose the following approach:

1. Strengthening works regardless of strengthening option. This approach focuses on reducing seismic mass and strengthen unreinforced masonry (URM) for out-of-plane actions.
    a. Remove internal masonry partitions wherever possible and replace with timber frame partitions (acoustically insulated and fire resistant as required).

b. Remove water tanks and unnecessary plant elements at the at the roof structure. Consider replacing water system with pump fed system from the ground.

c. Demolish emergency stairs added in 1950 and provide a new lightweight construction using steel, and timber framing with brick veneer.

d. Review and address all existing stair flights to avoid them taking load. This is expected to require disconnecting at the lower end of each flight.

e. External walls from ground to third floor, leave only external wythe and replace internal with timber framing and Python fixings.

2. Strengthening Option 1: Augment the frame stiffness throughout, to improve the frame drift capacity. This could be achieved using carbon fibre wrap at the beam-column joints, and columns. This option would be invasive, and the inherent high drift would mean the building is still vulnerable to damage in a seismic event. For these reasons, it is not considered further.

3. Strengthening Option 2: Provide a new reinforced concrete shear wall lateral load resisting system. This approach provides a stiff, strong solution that protects the existing vulnerable frames. This will require collectors and diaphragm strengthening, basement walls strengthening and additional work on stairs. Additional restraints and load path to the elevator support structure will be required. Strengthen entrance portico columns and first floor terrace diaphragm and connection to rear wing.

4. Strengthening Option 3: Reduce seismic acceleration and movement demand by means of base isolation at the basement level. Base isolation would reduce seismic demands by a factor of at least 4, by shifting the period of the building. The benefit if isolation, is it provides a higher performing, resilient building. This will require the following works:

   a. Strengthen the main lateral frames, though to a reduced extent (either by improvement or by adding new reinforced concrete shear walls) This would be less invasive than options 1 and 2 above.

   b. Isolating the building from ramps and other structures (or including these structures in the isolation). This will involve the creation of a moat around the building.

   c. Provision for movement connections for any services crossing the isolation zone.

   d. At the basement level, new isolators to be placed at the base of existing columns below basement level. For this, each column shall be propped temporarily, cut and jacketed above and below to allow for the insertion of a new lead rubber isolator. New foundations will be required, providing a stiff diaphragm slab around the existing pads. Above the isolation plane, some strengthening of the floor may also be required to give a competent diaphragm.

Any strengthening works will require review of accessibility and fire protection requirements as part of any Building Consent process. The building's heritage status will also need to be considered and addressed as discussed in 8.2 above.

8.4 New Seismic Hazard Model

It is noted that in 2022, the new New Zealand Seismic Hazard Model (NZSHM) was released. The outcomes of this update is not yet incorporated in legislation, however based on available information, the NSHM update may result in an increase of seismic demand by up to 40%. Therefore, we recommend that any strengthening scheme consider this pending change. A site-specific hazard study would provide more certainty in this regard.

Appendix A
Geotechnical Report

Project Number: 2-37905.01

Chateau Tongariro

Detailed Seismic Assessment
Geotechnical Report

9 December 2022

CONFIDENTIAL

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Document Details:
Date: 09/12/2022
Reference: 2-37905.01
Status: For Release

9(2)(a)

Prepared by
9(2)(a)

9(2)(a)

Reviewed by
9(2)(a)

9(2)(a)

Approved for release by
9(2)(a)

Document History and Status

Revision	Date	Author	Reviewed by	Approved by	Status
1	9/12/2022	9(2)(a) [redacted]	9(2)(a [redacted]	9(2)(a [redacted]	For Release

Revision Details

Revision	Details

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1 Introduction

1.1 Background

WSP has been engaged by Bayview International Hotels and Resorts Ltd (Kah New Zealand Limited) to undertake a Detailed Seismic Assessment (DSA) for the Chateau Tongariro at State Highway 48, Ruapehu. WSP has previously completed an Initial Seismic Assessment (ISA) published in April 2021[1]. The ISA identified uncertainty in the geotechnical ground conditions at the site due to lack of site investigation information. The purpose of this report is to present the geotechnical assessment component of the DSA carried out in general accordance with Chapter C4[2]. Other aspects of the DSA are presented under separate cover.

1.2 Scope of work

We have completed the following as part of our geotechnical DSA in accordance with Chapter C4:

- Desk study of the site including:
  - Published geological maps;
  - Review of internal and external databases for relevant information;
  - Published hazard mapping;
  - Historical aerial photos;
  - Plans and drawings available to us.
- Intrusive site investigation consisting of:
  - Two machine drilled boreholes; and,
  - Measuring of groundwater levels in standpipe piezometers.
- Geotechnical assessment in accordance with Chapter C4 including:
  - Seismic subsoil class;
  - Seismicity;
  - Bearing capacity for shallow foundations;
  - Stiffness parameters for structural modelling; and,
  - Liquefaction assessment.

2 Site description

2.1 Site location

The site is located in Tongariro National Park on the east side of SH48 in Whakapapa Village, approximately 6km southeast of the intersection of SH47 and SH48. The legal description of the land is Lots 4 6 DP 436518. The site location is shown on Figure 2-1.

[1] WSP. *Initial Seismic Assessment (ISA) for the Chateau Tongariro Hotel, Grand Chateau, State Highway 48, Ruapehu.* April 2021.
[2] Ministry of Business, Innovation, & Employment. *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments. Section C4.* July 2017.

Figure 2-1: Site location

2.2 Topography

Topography information including contours was accessed from Ruapehu District Councils map viewer[3]. The Chateau is located at an elevation of approximately 1120m. The land around the Chateau generally slopes down along SH48 towards SH47 to the northwest, and up towards Iwikau Village and Ruapehue to the south. The slope towards SH47 is about 6%, while the slope up the ridge to the south is steeper at 15% to 20%.

[3] https://maps.ruapehudc.govt.nz/intramaps90/?project=Ruapehu (Accessed October 2022)

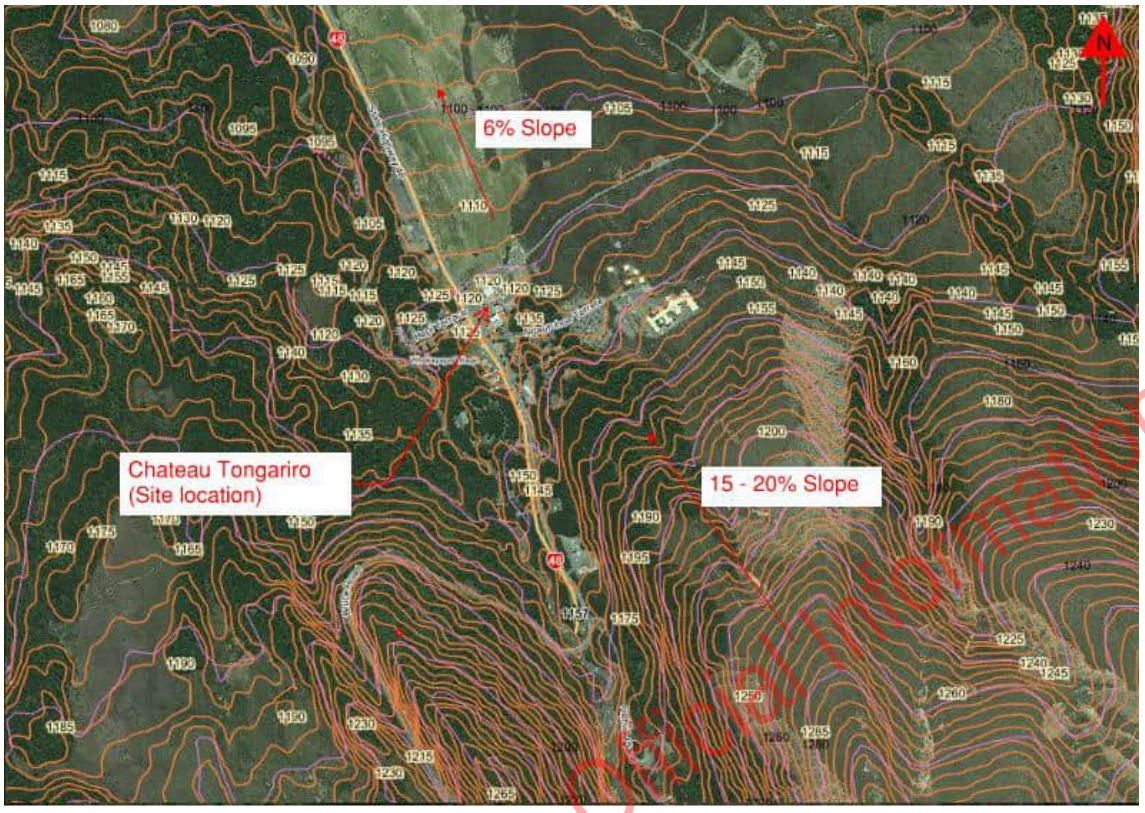


Figure 2-2: Site topography (Ruapehu DC Map Viewer[3] Accessed October 2022)

3 Desk study

3.1 Information considered

In accordance with C4 Geotechnical Considerations, this section of the report presents a list of the information reviewed as part of the desk study:

- Published geological maps from GNS;
- GNS Active fault database;
- Published geohazard mapping;
- Aerial photographs from Ruapehu District Council and Retrolens;
- Existing information on the building from the property file;
- Investigation data published in the New Zealand Geotechnical Database (NZGD).

3.2 Published geological maps

The geology of the area is described by Lee et al.[4] (2011) on the 1:250,000 geological map of Hawkes Bay and by Townsend et al.[5] (2017) in the 1:60,000 geological map of Tongariro National Park.

The 1:250,000 map indicates that the site is adjacent to or transected by the boundary between Q6 / Q7 / Q8 (Te Herenga Formation (Ruapehu Group) andesite of Tongariro Volcanic Centre and Q1S1 (Holocene fan deposits). The Te Herenga formation is described as pale-grey, crystal rich andesite flows, dikes, plugs and tuffs, with subsidiary rocks consisting of dacite breccia

[4] Heron DW (custodian). 2011. Geology of the Hawks Bay Area. Lower Hutt (NZ): GNS Science. *Institute of Geological & Nuclear Sciences 1:250,000 geological map 8* 93p. + 1 folded map.
[5] Townsend, D.B.; Leonard, G.S.; Conwar, C.E.; Eaves.; Wilson, C.J.N. (compilers) 2017 Geology of the Tongariro National Park Area. Lower Hutt, N.Z.: GNS Science. GNS Science geological map 4. 1 sheet + 109 p., scale 1:60,000.

pyroclastics 'debris flow deposits'. The site location in the context of the 1:250,000 geological map is shown on Figure 3-1.

The 1:60,000 map indicates that the site is adjacent to the boundary between holocene-aged Murimotu formation (Ir) and Pleistocene-aged till (t4). The Murimotu formation is described as andesite and dacite blocks, boulders, and gravel in a sandy-clay matrix. Sourced from the north-western summit plateau. The Murimotu formation is further described as a cohesive clay-rich diamicton, up to 12m thick, with well preserved surface topography forming conspicuous mounds up to 9m high. Blocks of andesite and dacite up to 35m long are internally shattered and set within a muddy-sandy intra block matrix. A significant portion of the material is reworked from the original collapse deposit. The till is described as comprising bouldery diamictons of volcanic clasts in a compact, sandy, muddy matrix. The eary last glacial till (t4) is further described as typically comprising landscape-forming moraines. In the vicinity of the subject site the till is described as overlying ridges of Te Herenga formation.

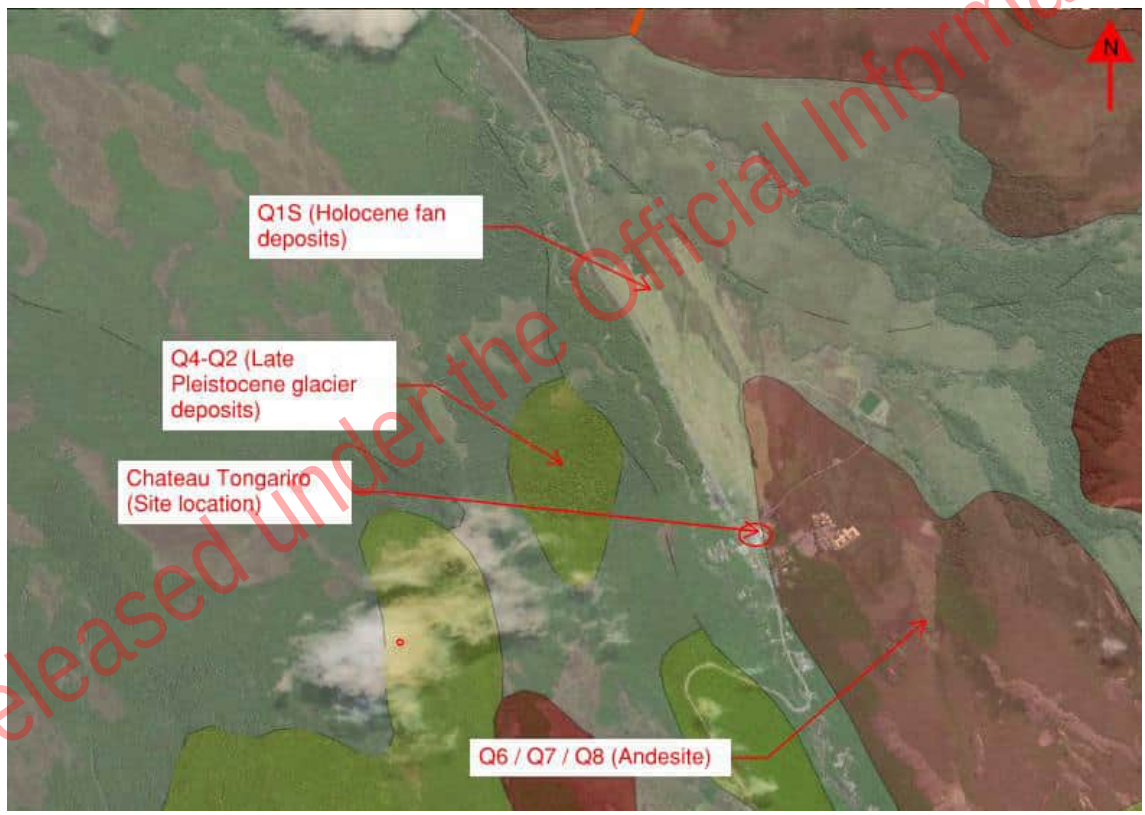


Figure 3-1: Site location in the context of the 1:250,000 Geological Map

3.3 Active faults

We have reviewed the GNS Active Faults Database for faults within 5km of the subject site. Numerous faults are mapped on the database near the site. None of the faults have reported slip rates or single event displacements. Only the Waihi Fault has a reported recurrence interval (less than 2,000 years) or a fault sense (normal). The site location in the context of published fault mapping is presented on

Figure 3-2.

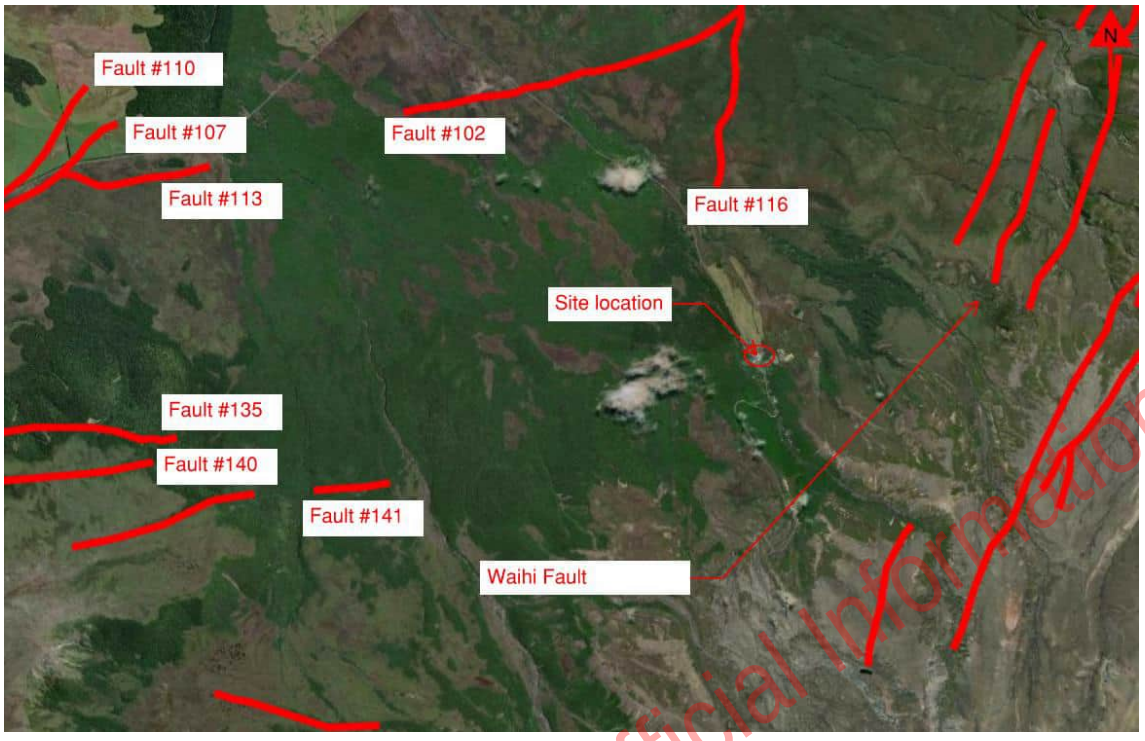


Figure 3-2: Site location in the context of the NZ fault database (NZ Active Faults Database, accessed November 2022[6])

3.4 Aerial photographs

We have reviewed published aerial photographs from Retrolens (1958 – 2002) and Ruapehu District Council (2011). The photograph review is summarised in Table 3-1.

Table 3-1: Aerial photographs

Photograph / Year	Comments
 1958	Chateau structure as it existed in 1958

[6] GNS Science - Active Faults Database (Accessed November 2022)

Photograph / Year	Comments
 1967	Further development to the south of the Chateau site; Chateau itself unchanged
 1980	Some addition development west of the Chateau site. Extension of the conference room on the front of the building.
 2000	Undeveloped; no change

Photograph / Year	Comments
 2011	New wing complete

Refer to our previous ISA report for a more complete history of the alterations around the Chateau Tongariro building.

3.5 Historical investigations

We have reviewed both external (NZGD) and internal report databases for historical investigation information or reporting near to the subject site. There is no available information within 10km of the subject site.

3.6 Published hazard data

Some published natural hazard data for the site is available in the source book for the 1:60,000 Geological map[7]. Note that the C4 geotechnical guidance recommends reviewing and reporting natural hazards as part of a holistic assessment although they do not effect the earthquake rating. Hazard information from the GNS book is summarised below. Note that other than the Lahar hazard the hazards are not mapped.

Table 3-2: Published hazard data

Hazard	Summary of published hazard data
Lahar	The site appears to straddle the boundary of 'moderate to high' and 'low to minor' lahar hazard; however high and highest lahar hazard zones are located nearby on the west side of the road
Shallow landslides	Not mapped specifically but the report notes that severe weather and earthquakes often trigger small shallow-seated landslides, and that landslide hazard may be increasing in some areas of the park due to retreat of glacial ice
Rockfall	Not mapped but notes that episodic rockfall occurs in the park area.
Crater lake break-outs	Not mapped but notes that crater lake breakouts have occurred in the park, including in 1953 and 2007, with the 1953 event causing 151 fatalities.

[7] Townsend DB, Leonard GS, Conway CE, Eaves SR, Wilson CJN, compilers. 2017. Geology of the Tongariro National Park area [map]. Lower Hutt (NZ): GNS Science. 1 sheet + 109p., scale 1:60 000. (GNS Science geological map; 4).

Hazard	Summary of published hazard data
Sector collapse	The report notes that sector collapses of the volcanic edifices has occurred in the past. The subject site is mapped as being underlain by one of the collapse deposits.
Ashfall	Not mapped specifically but the report notes hazards associated with ashfall of 1mm to greater than 300mm thickness, and also related hazards like rainfall-induced mobilization of recent ashfall slope failures and secondary lahars
Earthquakes	The site is subject to earthquakes from four sources: the Taupo rift, the North Island fault system, the subducting slab interface, and volcanic activity
Ejecta	Ejecta comprising blocks several meters in diameter are reported as traveling up to 3km from the vent.
Large explosive eruptions	Large explosive eruptions are reported to occur roughly once per century
Small crater lake eruptions	Reported to occur multiple times a decade

3.7 Existing information on buildings

Please refer to the main DSA report for a full list of available information.

4 Geotechnical investigation

Given the limited information available on the geotechnical conditions at the site, WSP carried out site investigations to understand foundation soils and the possibility of a step change for assessment purposes.

4.1 Field work

Fieldwork was carried out from September 20th, 2022, to September 22nd, 2022. Service clearance and a 0.9m depth hydrovac was carried out by Geotech Drilling prior to commencing intrusive investigations. Machine borehole drilling was carried out by Geotech Drilling using a track-mounted machine borehole rig with HQ size drilling. During drilling, a trailer mounted hydrovac was used to clean up any fluid leaking from the drilling area. Temporary measures including bentonite sealing / berms were used as needed in the drilling area to limit the amount of water loss.

4.2 Machine boreholes

Two machine boreholes were drilled to depths of 15.5m and 13.5m respectively, and slotted standpipe piezometers with toby flush mounts were installed at each location to allow for measurements of the groundwater level. One borehole was drilled near the NE corner of the original building, and another was drilled near the NW corner of the original building. Borehole locations were selected considering the locations of existing services, accessibility around the building for equipment, and ability to easily contain and manage water in the drilling area.

The ground profile encountered at both borehole locations essentially comprised 200mm of asphalt and base course gravels underlain by boulders in a sandy, gravelly matrix. Trace silt and trace clay was occasionally encountered within the sand matrix. Note that in between the SPT test locations we infer that the sand and silt content has been partially washed out by the drilling process and therefore photographs of the core in between SPT locations appear to have a significantly lower percentage of finer particles than they do in-situ.

Detailed borehole logs are attached in Appendix B and a borehole location plan is shown on Figure 1 in Appendix A.

4.3 Groundwater

Groundwater levels were measured upon completion of drilling and 6 weeks after completion of drilling. The results are summarised in Table 4-1.

Table 4-1: Groundwater levels

	Upon completion	November 7, 2022
BH1	5.1	-
BH2	5.2	5.05

5 Geotechnical Assessment

5.1 Geotechnical model

Based on the available site investigation data and our review of published geological information we propose the following geotechnical model for the site as shown in Table 5-1. Geotechnical parameters are presented in Table 5-2.

Table 5-1: Geotechnical model

Geological unit	Unit No.	Description	Starting depth	Thickness
Q1S1 (Holocene Fan) / IR (Murimoto Formation)	1	Boulders in sand and gravel matrix; trace silts and clays may be present	Ground surface	At least 15m, total depth unknown
Q6 / Q7 / Q8 Te Herenga Formation	2	Andesite	Unknown	Unknown

Table 5-2: Geotechnical parameters for assessment purposes

Geological unit	Unit No.	Friction Angle (°)	Cohesion c' (kPa)	Unit Weight γ (kN/m^3)	Elastic modulus E (MPa)
Q1S1 (Holocene Fan) / IR (Murimoto Formation)	1	35	0	20	50 – 150
Q6 / Q7 / Q8 Te Herenga Formation	2	-	-	-	-

5.2 Site Subsoil Class

New Zealand Standard NZS 1170.5:2004 provides guidance for determination of site subsoil class[8]. We have undertaken an assessment of the site subsoil class based on the site investigation data and available geological information. Based on the available information the site should be assessed for both Site Class C and Site Class D.

5.3 Geohazards

The site is in an area potentially subject to a number of geohazards. The C4 Geotechnical Considerations document recommends screening sites for geotechnical hazards. Only those seismic-triggered geohazards that originate within the building footpring influence the %NBS rating, however, the guidance recommends noting the potential for other geohazards to affect the site as part of a holistic assessment. More information is provided in Section 3.6.

Table 5-3: Geohazard summary

Geohazard	Site potentally subject to this hazard	Seismic-Related / Triggered	Originates on site
Rockfall	Unlikely	Yes	No
Landsliding (overslip above sites)	Yes, but unlikely.	Yes	No
Landsliding (underslip below site)	Unlikely unless strength loss (liquefaction) occurs	Yes	Yes
Fault Rupture[1]	Yes	Yes	No
Liquefaction	Yes	Yes	Yes
Lahar	Yes	No	No
Tsunami	No	Yes	No

[8] Standards New Zealand (2004) *NZS 1170.5:2004, Structural Design Actions – Earthquake Actions*

Geohazard	Site potentially subject to this hazard	Seismic-Related / Triggered	Originates on site
Lateral Spreading	Yes	Yes	Yes
Ash fall	Yes	No	No
Volcanic bombs	Yes	No	No
Pyroclastic Flow	Yes	No	No
Lava flow	Yes	No	No

1: No mapped faults pass through the site (refer Section 3.3) but this does not preclude unknown faults being present

5.4 Seismicity

Seismicity for the site has been assessed based on the assumption of an Importance Level 2 (IL2) structure. The MBIE has presented geotechnical design PGA's for New Zealand[9]. The Chateau is not located in an area with published PGAs. As such we have used the nearest sites (Ohakune and Waiouru) for the purpose of this report.

- 25 year earthquake: 0.09g
- 50 year earthquake: 0.13g
- 100 year earthquake: 0.18g
- 250 year earthquake: 0.27g
- 500 year earthquake: 0.36g
- 1000 year earthquake: 0.47g
- 2500 year earthquake: 0.64g

The site is located in a sedimentary bowl between ridges / dykes composed of andesite, and therefore may be subject to basin effects, which could lead to amplification of seismic waves and increased PGA at the ground surface.

5.5 Liquefaction

The purpose of this section is to present background information related to liquefaction, and our assessment of liquefaction potential at the site

5.5.1 Background

Liquefaction is a result of the tendency of soils to contract under cyclic shear loading. If the soil is saturated with water and the contraction of soil during earthquake shaking is prevented or slowed due to drainage time, excess pore water pressures are generated reducing the effective stress and leading to a loss in strength and stiffness. Liquefaction is the term used to refer to this loss of strength and stiffness.[10] Liquefaction can result in settlement of the ground surface, bearing capacity failure, and horizontal movements. Surface manifestations of liquefaction such as sand boils can also occur.

[9] Ministry of Business, Innovation, & Employment. Earthquake geotechnical engineering practice. Module 1. Overview of the guidelines. November 2021. Appendix A.
[10] Ministry of Business, Innovation & Employment. *Earthquake Geotechnical Engineering practice*. Module 3. Identification, assessment, and mitigation of liquefaction hazards. November 2021.

For liquefaction to occur, three key conditions are required[11]:

1. The soil must be susceptible to liquefaction
2. The soil must be saturated
3. An earthquake with sufficient duration and intensity to trigger liquefaction must occur

5.5.2 *Assessment process*

The assessment process for liquefaction analysis essentially comprises the following steps:

1. Assess liquefaction susceptibility
2. If potentially susceptible to liquefaction, assess liquefaction triggering
3. If liquefaction triggering occurs, assess horizontal and vertical ground deformations
4. Assess the effects of the deformations on structures

Simplified liquefaction susceptibility screening criteria are available based on geologic factors. This approach is presented in Section 5.5.4.

Quantitatively, liquefaction triggering can be assessed using the simplified method based on SPT (Standard Penetration Test)-based, CPT (Cone Penetration Test)-based, or Vs (Shear Wave Velocity)-based procedures. Liquefaction triggering in the simplified methods is assessed by using the SPT, CPT, or Vs data to calculate a Cyclic Resistance Ratio (CRR) which represents the capacity of the soil to resist liquefaction and comparing the CRR to the Cyclic Stress Ratio (CSR) which represents the seismic demand caused by the design earthquake. Vertical and lateral movements can then be predicted based on the thickness and Factor of Safety of the liquefied layers.

We have assessed liquefaction triggering using the SPT-based procedure proposed by Idriss & Boulanger (2014)[12] as presented in Section 5.5.5.

5.5.3 *Groundwater scenarios*

Measured groundwater levels in September and November 2022 are reported in Section 4.3. The levels range from 5.05m to 5.2m. We infer that these groundwater conditions are representative of the seasonal worst case. Although the published guidance recommends assuming average conditions in the absence of other data on groundwater levels, we have taken the groundwater level for the liquefaction assessment to be 5m.

5.5.4 *Geologic susceptibility screening*

ASCE[13] proposes a simplified geologic screening for estimating liquefaction susceptibility. Sites categorized as 'very low' susceptibility under the scheme can be assumed to be non-liquefiable. An assessment of the geological units at the site in the context of the screening guide is presented in Table 5-4. In summary, the holocene-age Murimoto Formation / Q1S1 units are potentially susceptible to liquefaction and require further assessment.

Table 5-4: Assessment of Chateau Tongariro Site - Geologic Susceptibility Screening

[11] Ministry of Business, Innovation & Employment. *Planning and Engineering Guidance for Potentially Liquefaction Prone Land.* (2017). 8.
[12] R.W. Boulanger, I.M. Idriss. *CPT and SPT-Based Liquefaction Triggering Procedures.* Report No. UCD/GCM-14/01. April 2014.
[13] American Society of Civil Engineers. *Seismic Evaluation and Retrofit of Existing Buildings.* (2017). ASCE 41-17. Chapter 8 – Foundations and Geologic Site Hazards.

Geological unit	Type of deposit	Age	Estimated Susceptibility to Liquefaction per Table 5-2
Te Herenga Formation (Q6 / Q7 / Q8)	Rock; but subsidiary unit includes debris flow deposits	Middle Pleistocene	Very low
Holocene Fan Deposits (Q1S1)	Alluvial fan / Colluvial fan	Holocene	Low
Murimoto Formation (IR)	Talus / Collovium / Alluvial Fan	Holocene	Low
Till (Pleistocene Aged)	Till	Pleistocene	Very low

5.5.5 *SPT-Based Liquefaction Triggering Assessment*

We have used the SPT-based method of Idriss & Boulanger (2014) for assessment of liquefaction triggering at the site using SPTs. The subject site is challenging to assess as the ground conditions have limited similarity to the ground conditions at the sites in the empirical database used to develop the triggering method and are thus an extrapolation. The presence of boulders also makes the assessment more difficult as the SPT values are influenced by the presence of large gravel sizes.

Briefly, the soil conditions at the site are described in Section 4.2, and essentially consist of boulders in sand and gravel matrix. It is possible that if the matrix is saturated and loose enough and / or subject to enough earthquake shaking, liquefaction of the matrix could occur.

To assess liquefaction triggering we have conducted two assessments using the available data to account for the limitations described above. These are:

1. Using the SPT-data as-is;
2. Selecting the SPTs that we assess are most representative of the matrix and applying those to the entire profile to assess triggering.

The results are presented in the following sections.

5.5.5.1 Case #1 Results

For Case #1 (using the SPT-data as is) no liquefaction triggering is predicted up to and including 0.36g (1 in 500 year). Detailed results are presented in Appendix D.

5.5.5.2 Case #2 Results

The SPT data for both boreholes are presented in Table 5-5. As shown, most SPTs are 50+ or would result in a 50+ value when extrapolated to the full SPT test length of 300mm. For Case #2, we have carried out an assessment of the SPT data and concluded that an SPT value of 30 could be assessed as appropriate for the sand / gravel matrix. For this case, liquefaction triggering is predicted at 13.5m depth. Detailed results are presented in Appendix D. We have summarised an assessment of PGA vs liquefaction triggering in Table 5-6. Intermediate PGAs between the 1 in 250 and 1 in 500 year PGA were assessed to determine the initial PGA that triggering occurs at.

Table 5-5: SPT Assessment

Depth (m)	BH1 – SPT	BH1 – Log Notes	BH2 – SPT	BH2 – Log Notes
1.5	50	73mm movement	21	No notes, full SPT
3.0	50	Bouncing, no movement	50	No notes, full SPT
4.5	50	35mm movement	50	205mm movement

Depth (m)	BH1 – SPT	BH1 – Log Notes	BH2 – SPT	BH2 – Log Notes
6.0	50	Bouncing, no movement	35	83mm movement
7.5	50	50mm movement	25	45mm movement
9.0	50	65mm movement	25	75mm movement
10.5	35	No notes, full SPT	25	Bouncing (no movement)
12.0	30	10mm movement	30	Bouncing (no movement)
13.5	50	Bouncing, no movement	30	Bouncing (no movement)
15.0	50	Bouncing, no movement	N/A	N/A

Table 5-6: Liquefaction triggering

PGA (g)	Liquefaction triggered
0.09 (1 in 25)	No
0.13 (1 in 50)	No
0.18 (1 in 100)	No
0.27 (1 in 250)	No
0.34 (Intermediate case to assess initial trigger)	Triggering @ 15m
0.36 (1 in 500)	Triggering @ 13.5m
0.47 (1 in 1000)	Triggering @ 9.0m
0.64 (1 in 2500)	Triggering @ 6.0m

5.5.6 Vertical deformations

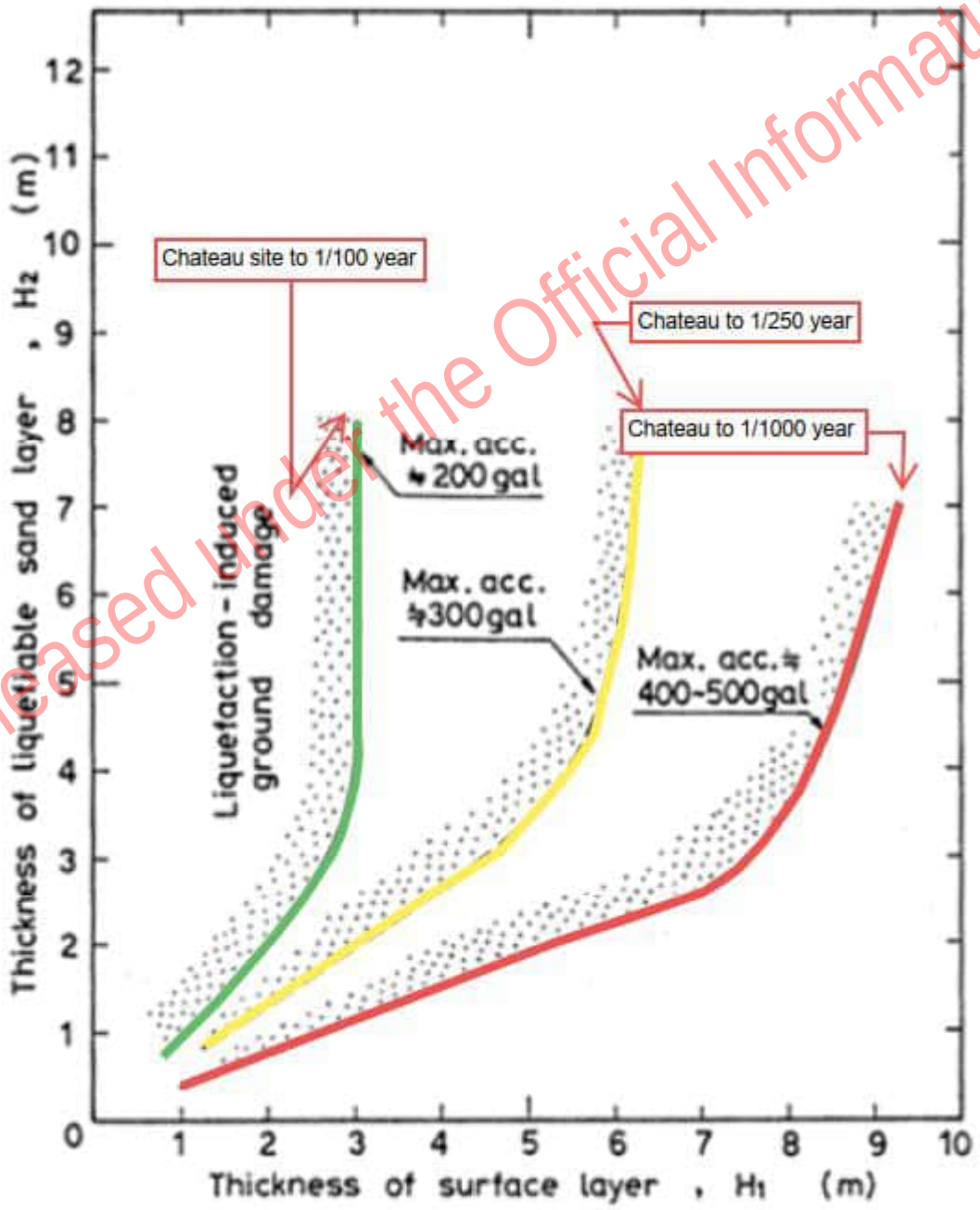
We have assessed the manifestation of vertical deformations at the ground surface using the LPI and LPI_{Ish}[14] method. LPI_{Ish} of less than 5 and LPI of less than 5 are both considered to be low risk of manifestation of liquefaction effects at the ground surface. The LSN method cannot be applied here because it requires vertical deformations calculated using Zhang which requires CPT data. However, we note that LPI typically over-predicts liquefaction effects so high LPI values can be misleading. Detailed calculations are presented in Appendix D.

Table 5-7: LPI and LPIISH

PGA (g)	LPI_{Ish}	LPI
0.09	0	0
0.13	0	0
0.18	0	0
0.27	0	0
0.36	0	1
0.47	0	4
0.64	3	10

[14] Maurer, B W. Green, A. R., Taylor, S. D. 2015. Moving towards an improved index for assessing liquefaction hazard: Lessons from historical data. Department of Civil & Environmental Engineering, Virginia Tech, Blacksburg, VA 24060, USA. US Army Corps of Engineers, Engineering Research & Development Center, Vicksburg, MS 39180 USA.

In addition to LPI and LPI_{Ish} a simplified chart to assess the influence of a non-liquefiable capping layer on suppressing surface manifestations of liquefaction was presented by Ishihara[15]. The chart is presented on Figure 5-1. For the Chateau site because the investigations do not constrain the depth of the liquefiable layer it is necessary to either assume the thickness of the layer or extrapolate the lines on the chart. For the 1 in 500-year earthquake with an H1 of 13.5m, an assessment based on the Ishihara Chart indicates no ground damage. For a 1 in 1000-year earthquake, with an H1 of 9m, the chart indicates no ground damage up to a liquefied layer thickness of 5m. The PGA for the 1 in 2500-year earthquake is not included on the chart, however, it can be inferred based on the chart that with a 6m crust and shaking greater than 0.5g, ground damage would be expected.



[15] Ishihara, K. 1985. Stability of natural soil deposits during earthquakes. Pp. 321-376 in *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering,* San Francisco, California.

Figure 5-1: Ishihara Ground Damage Chart

The Bray and Macedo method for assessing shear-induced building settlements requires CPT data which is not available; however, we consider that relative to the footing sizes the crust thickness for the 1 in 500 year and 1 in 1000 year earthquake are large enough that significant shear deformations or punching shear type failures would be unlikely. Shear induced deformations may be possible for larger earthquakes such as the 1 in 2500 year earthquake.

Based on the assessment of LPI, $LPI_{ISH,}$ and Ishihara presented in this section, significant vertical deformations or surface manifestation of liquefaction-induced damage are unlikely to manifest at the ground surface up to the 1 in 1000 year earthquake but may be possible for the 1 in 2500 year earthquake, due to the depth of the liquefiable layer. In addition, we consider that while the sand and gravel matrix is liquefiable the boulders would constrain volumetric deformations within liquefied layers.

5.5.7 Lateral spreading

Significant lateral spreading is unlikely due to the distance to a water body / free face, the depth of the groundwater table, and the depth of the potentially liquefiable materials. In addition, we consider that the substantial number of boulders would constrain deformations.

5.5.8 Slope stability

We have adopted a simplified assessment using a moderately conservative approach that reflects the uncertainty in key inputs such as the thickness of the liquefied layer. Stability calculations were performed only for an underslip originating on site as this is the case that the C4 Geotechnical Guidance requires assessment of. Note that the depth of the liquefied layer in the model has been extended in a manner that is unrealistic but that reflects the limited information available to constrain the base of the model and allows us to provide a level of confidence that slope failure would not occur.

1. Assess pre-seismic static slope stability
2. Assess pseudo-static seismic slope stability
3. Assess post-liquefaction slope stability under static driving shear stresses per Robertson[16]. A residual strength after liquefaction of 35 kPa (@ 13.5m) + 1 kPa / m was used in the assessment based on Olson and Stark[17].

The results indicate that the slope can be considered stable under both seismic and liquefaction cases and therefore a step-change in building performance or partial loss of foundation support would not be expected. Detailed calculations are presented in Appendix D.

Table 5-8: Slope stability analysis results

Analysis Case	Calculated Factor of Safety	Target Factor of Safety
Static slope stability	5.5	1.5
ULS seismic slope stability	2.7	1.1
Post-liquefaction slope stability	1.18	1.1

[17] Olsen, S.M. and Stark, T.D., 2002. Liquefied Strength ratio from liquefaction flow failure case histories. Canadian Geotechnical Journal, 39: 629-647

5.6 Foundations

Bearing capacity for rectangular spread footings has been assessed for static vertical concentric loading using the soil parameters described in Section 5.1 and the bearing capacity calculation method described in B1/VM4 as implemented in SESOC Soils. Embedment for all footings was assumed to equal the footing thickness.

Due to the depth to groundwater and the depth to the liquefied layer, we have not assessed a reduced bearing capacity due to liquefaction effects as the failure surface would not pass near to potentially liquefied material and punching shear failure is not a credible failure mode with a 13.5m crust.

The calculated ultimate geotechnical bearing capacity, q_u for shallow spread foundations ranges from 590 kPa to 1590 kPa. A strength reduction factor of 1 was used (i.e. no reduction). We expect that deformations of 5% to 10% of the footing width would normally be expected under these values for bearing capacity failure.[18]Tabulated values for different footing sizes are presented in Appendix D, along with an example calculation.

The structural engineer has requested G_o parameters for use in dynamic spring formulae. We have assessed a range for G_o values based on published values and correlations[19] as presented in more detail in Appendix D. Based on this parameterization and the limited information available a range of G_o values of 65,000 kPa/m to 400,000 kPa/m is considered applicable in the absence of shear wave velocity testing. Note that G_o is a small strain modulus value at negligible deformation and the secant or tangent moduli values at larger strains would be reduced.

[18] Fellenius, B.H., 2019. Basics of foundation design. Pile Buck International, Inc., Vero Beach, FL, Electronic Edition, www.Fellenius.net, 484p. *p 6-13 – p6-14.*
[19] Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A., Zettler, T.E. 2002. Geotechnical Engineering Circular No. 5 – Evaluation of Soil and Rock Properties. 385p.

6 Conclusions

Based on our assessment of the available information, our site investigations and our analysis we present the following conclusions:

- Liquefaction:
  - The Holocene fan / Murimoto formation deposits are potentially susceptible to liquefaction;
  - Liquefaction triggering would occur beginning at a PGA of 0.34g at a depth of 15 m; this is associated with a return period between 1/250 and 1/500 years.
  - At the 1/500 ULS PGA of 0.36g, liquefaction triggering would occur at 13.5 m;
  - At the 1/2500 ULS PGA of 0.64g, liquefaction triggering begins at 6m depth
  - Due to the depth to the liquefied layer, depth to groundwater, thickness of crust, and presents of boulders, vertical deformations would be constrained, and liquefaction effects would not manifest at the ground surface, at least up to the 1/1000 year ULS PGA. Some minor manifestation at the ground surface may occur for the 1/2500 year earthquake.;
  - Lateral spreading is considered to be unlikely due to the lack of a free face, depth to the liquefied material, and presence of boulders to suppress deformations;
  - Post liquefaction slope instability is assessed to be unlikely using residual liquefied strengths;
  - Based on the above, step-change of the building performance due to liquefaction is unlikely.
- Slope stability:
  - Underslips originating on site are unlikely and the factor of safety for this type of failure even under seismic accelerations is greater than 1.5;
  - Slope stability under static driving shear stresses using liquefied residual strengths is unlikely as the factor of safety is greater than 1.1.
- Bearing capacity:
  - Assessed bearing capacities for static vertical concentric loads varies from 590 kPa to 1590 kPa;
  - A step change reduction in bearing capacity due to liquefaction is not predicted to occur.
- Other geohazards:
  - The site is subject to a variety of geohazards due to it's location that originate offsite including volcanic, seismic, and extreme weather related geohazards.

We have also concluded that this assessment will be structurally dominated as per Chapter C4 of the MBIE assessment guidelines and that a step change is not predicted to occur. Some residual uncertainty remains in our assessment. In particular, the depth to the base of the boulder deposit and liquefied layer is not constrained by the current depth of investigation, nor can the seismic subsoil class be definitively assessed. In our opinion for the purpose of the DSA further geotechnical investigations are not necessary unless determination of the subsoil class or site shear wave velocity is deemed significant.

Appendix A
Site investigation plan

Legend

Machine borehole

Appendix B
Borehole logs

Borehole No. BH1

Project:	Chateau Tongariro	*Coordinates:*	Not established
Client:	Bayview International Hotels and Resorts (Kah New Zealand Limited)	*Hole Dia.:*	n/a
Project No.:	2-37905.01	*R.L.:*	Not established
Location:	NE Corner of Original Building	*Datum:*	

Depth: 15.45 m

Inclination: Vertical

GEOLOGY	MAIN DESCRIPTION / DETAIL DESCRIPTION	R.L. (m) / DEPTH (m)	TESTS: SPT 'N' VALUE	TESTS: SPT BLOW COUNTS OR SHEAR VALUE	CORE: CORE TYPE	CORE: TOTAL CORE RECOVERY (%)	DRILLING: DRILLING METHOD	DRILLING: CASING	DRILLING: BASE OF HOLE & WATER LEVEL	NOTES / OTHER TESTS
Fill	Asphalt and Base Course Gravels									
Q1S1	Boulders, with some cobbles, with some gravels, with trace of to minor sand, light grey to dark grey with occasional red and black, medium dense, dry, well graded (Boulders in a sand / gravel / cobble matrix)						HQ Size, Triple Tube, Wireline Rotary Coring			
	0.90-1.50m - ...boulder	1	50	50// 24/26 for 75mm						
	1.80-3.00m - ...Cobbles, with some medium to coarse gravels, with trace to some sand, light grey to dark grey, dry	2								
	3.37-4.50m - ...boulder	3	50	24// 11/11/18/10						
	4.50-6.00m - ...boulders, cobbles, gravelly,	4	50+	20 for initial 85mm						SPT:..20 blows for 35mm penetration, bouncing
	5.00m - ...from 5m, wet	5								
	6.00-7.50m - ...boulders, cobbly	6	50+	50 for initial 0mm						SPT:...bouncing, no penetration
	7.63-8.20m - ...Coarse gravel, fine to coarse, with trace to some sand, light to dark grey	7	50+	45 for initial 125mm						SPT:...bouncing @ end of second set
	8.20-9.00m - ...Cobbles, gravelly, trace to some sand, dense, light to dark grey	8								
	9.37-10.50m - ...Boulders, gravelly (medium to coarse),	9	50+	25// 11/21/18 for 65mm						SPT:...bouncing @ end of 5th set

INSTALLATION DETAILS: Slotted pipe with sand backfill

Notes:

Started: 20/09/2022 *Finished:* 21/09/2022

Drilling Co.: Geotech Drilling *Drilling Rig:*

Logged by: JP *Checked by:*

Logged in accordance with NZ Geotechnical Society Guidelines (2005). See attached key sheet for explanation of symbols.

Scale 1:50 @ A4

Borehole No. BH1

Project:	Chateau Tongariro	*Coordinates:*	Not established
Client:	Bayview International Hotels and Resorts (Kah New Zealand Limited)	*Hole Dia.:*	n/a
Project No.:	2-37905.01	*R.L.:*	Not established
Location:	NE Corner of Original Building	*Datum:*	

Depth: 15.45 m
Inclination: Vertical

GEOLOGY	MAIN DESCRIPTION / DETAIL DESCRIPTION	R.L. (m) / DEPTH (m)	GRAPHIC LOG	TESTS: SPT 'N' VALUE	TESTS: SPT BLOW COUNTS OR SHEAR VALUE	CORE: CORE TYPE	CORE: TOTAL CORE RECOVERY (%)	DRILLING: DRILLING METHOD	DRILLING: CASING	DRILLING: BASE OF HOLE & WATER LEVEL	NOTES / OTHER TESTS	INSTALLATION DETAILS
Q1S1	Boulders, with some cobbles, with some gravels, with trace of to minor sand, light grey to dark grey with occasional red and black, medium dense, dry, well graded (Boulders in a sand / gravel / cobble matrix)*(continued)*	11		35	18// 5/6/10/14 for 75mm			HQ Size, Triple Tube, Wireline Rotary Coring				
	10.95-12.00m - ...Gravel (medium to coarse) with minor cobbles, trace sand	12		50+	30 for initial 85mm						SPT:...bouncing @ end of second set	
	12.00-15.00m - ...Boulders, cobbly	13										
		14		50+	50						SPT:..bouncing first set	
		15		50+	50						SPT:...bouncing first set	
	END OF BOREHOLE AT 15.45m - Target Depth Reached	16										
		17										
		18										
		19										

Notes:

Started: 20/09/2022 *Finished:* 21/09/2022
Drilling Co.: Geotech Drilling *Drilling Rig:*
Logged by: JP *Checked by:*

Logged in accordance with NZ Geotechnical Society Guidelines (2005). See attached key sheet for explanation of symbols.
Scale 1:50 @ A4

Borehole No. BH2

Project:	Chateau Tongariro	*Coordinates:*	Not established
Client:	Bayview International Hotels and Resorts (Kah New Zealand Limited)	*Hole Dia.:*	n/a
Project No.:	2-37905.01	*R.L.:*	Not established
Location:	NW Corner of Original Builing	*Datum:*	

Depth: 13.575 m

Inclination: Vertical

GEOLOGY	MAIN DESCRIPTION / DETAIL DESCRIPTION	R.L. (m) / DEPTH (m)	GRAPHIC LOG	TESTS: SPT 'N' VALUE	TESTS: SPT BLOW COUNTS OR SHEAR VALUE	CORE: CORE TYPE	CORE: TOTAL CORE RECOVERY (%)	DRILLING: DRILLING METHOD	DRILLING: CASING	DRILLING: BASE OF HOLE & WATER LEVEL	NOTES / OTHER TESTS	INSTALLATION DETAILS
	Asphalt and Base Course Gravels											
	Boulders, with some cobbles, with some gravels, with trace of to minor sand, light grey to dark grey with occasional red and black, medium dense, dry, well graded (Boulders in a sand / gravel / cobble matrix)	1 2 3 4 5 6 7 8 9						HQ Size, Triple Tube, Wireline Rotary Coring			3.00-3.45m - Last set 16 for 55mm 4.50-4.55m - ...10 blows for 15mm, then hammer bouncing 6.00-6.10m - ...10 blows for 8mm, then hammer bouncing 7.50-7.60m - ...last set 25 blows for 45mm 9.00-9.10m - ...hammer bouncing	

Notes:

Started: 21/09/2022 *Finished:* 22/09/2022

Drilling Co.: Geotech Drilling *Drilling Rig:*

Logged by: JP *Checked by:*

Logged in accordance with NZ Geotechnical Society Guidelines (2005). See attached key sheet for explanation of symbols.

Scale 1:50 @ A4

Borehole No. BH2

Project:	Chateau Tongariro	*Coordinates:*	Not established
Client:	Bayview International Hotels and Resorts (Kah New Zealand Limited)	*Hole Dia.:*	n/a
Project No.:	2-37905.01	*R.L.:*	Not established
Location:	NW Corner of Original Builing	*Datum:*	

Depth: 13.575 m
Inclination: Vertical

GEOLOGY	MAIN DESCRIPTION / DETAIL DESCRIPTION	R.L. (m) / DEPTH (m)	GRAPHIC LOG	TESTS: SPT 'N' VALUE	TESTS: SPT BLOW COUNTS OR SHEAR VALUE	CORE: CORE TYPE	CORE: TOTAL CORE RECOVERY (%)	DRILLING: DRILLING METHOD	DRILLING: CASING	DRILLING: BASE OF HOLE & WATER LEVEL	NOTES / OTHER TESTS	INSTALLATION DETAILS
	Boulders, with some cobbles, with some gravels, with trace of to minor sand, light grey to dark grey with occasional red and black, medium dense, dry, well graded (Boulders in a sand / gravel / cobble matrix)*(continued)*	11						HQ Size, Triple Tube, Wireline Rotary Coring			10.50-10.55m - ...hammer bouncing	
		12									12.00-12.55m - ...hammer bouncing	
		13									13.50-13.58m - ....hammer bouncing	
	END OF BOREHOLE AT 13.575m - Unable to Advance Auger - Too Hard	14										
		15										
		16										
		17										
		18										
		19										

Notes:

Started: 21/09/2022 *Finished:* 22/09/2022
Drilling Co.: Geotech Drilling *Drilling Rig:*
Logged by: JP *Checked by:*

Logged in accordance with NZ Geotechnical Society Guidelines (2005). See attached key sheet for explanation of symbols.

Scale 1:50 @ A4

Appendix C
Supporting documents

Column Number	Foundation			Basement to Ground Floor			Ground to First Floor			First Floor to Second			Second Floor to Third		
	Size	Reinforcement		Size	Reinforcement		Size	Reinforcement		Size	Reinforcement		Size	Reinforcement	
1	5'0"x 5'0"x 1'0"	1/2" ϕ @ 6" crs	both ways		NIL		16"x 16"	4 - 7/8" ϕ	3/8" @ 9"	12"x 12"	4 - 7/8" ϕ	3/8" @ 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
2	5'6"x 5'6"x 1'0"	1/2" ϕ at 6" crs	both ways		NIL		16"x 14"	4 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	11"x 11"	4 - 7/8" ϕ	3/8" at 9"
3	do.				NIL		do			do.			do		
4	5'6"x 5'6"x 1'6"	1/2" ϕ at 6" crs	both ways		NIL		16"x 14"	6 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
5	5'5"x 5'5"x 1'6"	1/2" ϕ at 6" crs	both ways		NIL		16"x 14"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
6	5'0"x 5'0"x 1'6"	1/2" ϕ at 6" crs	both ways		NIL		16"x 16"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
7	5'6"x 5'6"x 1'9"	1/2" ϕ at 6" crs	both ways	16"x 14"	6 - 1" ϕ	3/8" at 9"	16"x 14"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	11"x 11"	4 - 3 1/2" ϕ	3/8" at 9"
8	5'9"x 5'9"x 1'9"	1/2" ϕ at 6" crs	both ways	15"x 15"	6 - 1" ϕ	3/8" at 9"	15"x 15"	6 - 1" ϕ	3/8" at 9"	13"x 13"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
9.	do.			do.			do.			do.			do.		
10	5'9"x 5'9"x 1'9"	1/2" ϕ at 6" crs	both ways	16"x 16"	6 - 1" ϕ	3/8" at 9"	15"x 15"	6 - 1" ϕ	5/8" at 4"	13"x 13"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
11	5'6"x 5'6"x 1'9"	1/2" ϕ at 6" crs	both ways	15"x 15"	6 - 1" ϕ	3/8" at 9"	15"x 15"	4 - 1" ϕ	3/8" at 9"	13"x 13"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
12	5'5"x 5'5"x 1'6"	1/2" ϕ at 6" crs	both ways	16"x 14"	4 - 1" ϕ	3/8" at 9"	16"x 14"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
13	5'0"x 5'0"x 1'6"	1/2" ϕ at 6" crs	both ways		NIL		16"x 14"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
14	5'9"x 5'9"x 1'9"	1/2" ϕ at 6" crs	both ways	15"x 15"	6 - 1"	3/8" at 4"	15"x 15"	4 - 1" ϕ	5/8" at 4"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
15	5'9"x 5'9"x 1'9"	1/2" ϕ at 6" crs	both ways	15"x 15"	6 - 1"	3/8" at 4"	15"x 15"	4 - 1" ϕ	3/8" at 9"	do.			12"x 12"	4 - 3 1/2" ϕ	3/8" at 9"
16	5'5"x 5'5"x 2'3"	5/8" ϕ at 6" crs	both ways	16"x 16"	6 - 1 1/8" ϕ	3/8" at 3"	15"x 15"	6 - 1" ϕ	3/8" at 4"	13"x 13"	4 - 1" ϕ	3/8" at 9"	do.		
17	4'0"x 4'0"x 2'3"	5/8" ϕ at 6" crs	both ways	15"x 15"	6 - 1" ϕ	3/8" at 3"	18"x 15"	4 - 1" ϕ	3/8" at 9"	15"x 15"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
18	5'6"x 5'6"x 1'9"	1/2" ϕ at 6" crs	both ways	16"x 14"	4 - 1" ϕ	3/8" at 9"	16"x 14"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
19	5'5"x 5'5"x 1'6"	1/2" ϕ at 6" crs	both ways		NIL		16"x 16"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
20	4'0"x 4'0"x 2'0"	5/8" ϕ at 6" crs	both ways		NIL		16"x 16"	6 - 1" ϕ	3/8" at 3" Spiral	16"x 12"	4 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
								OUT							
22	5'9"x 5'9"x 1'9"	1/2" ϕ at 6" crs	both ways		NIL		16"x 16"	4 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
23	2'6"x 2'6"x 1'0"	1/2" ϕ at 6" crs	both ways		NIL		12"x 12"	4 - 3/4" ϕ	3/8" at 9"						
24	do.				NIL		do								
25	5'0"x 4'0"x 1'9"	5/8" ϕ at 6" crs	both ways	20"x 16"	6 - 1 1/8" ϕ	3/8" at 9"	20"x 16"	6 - 7/8" ϕ	3/8" at 9"	20"x 12"	6 - 3/4" ϕ	3/8" at 9"	20"x 12"	4 - 3/4" ϕ	3/8" at 9"
26	3'9"x 3'9"x 1'9"	1/2" ϕ at 6" crs	both ways	15"x 15"	6 - 1" ϕ	3/8" at 3"	15"x 15"	6 - 1" ϕ	3/8" at 9"	13"x 13"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
26A	4'3"x 4'0"x 2'0"	5/8" ϕ at 6" crs	both ways	15"x 15"	6 - 1 1/8" ϕ	3/8" at 4"	15"x 15"	6 - 1" ϕ	3/8" at 9"	do.			do.		
27	do.			16"x 16"	6 - 1 1/8" ϕ	3/8" at 3"	16"x 14"	6 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
28	4'5"x 4'5"x 2'0"	5/8" ϕ at 6" crs	both ways	16"x 16"	6 - 1 1/8" ϕ	3/8" at 3"	do.			do.			do.		
29	4'0"x 4'0"x 2'0"	5/8" ϕ at 6" crs	both ways	16"x 16"	6 - 1 1/8" ϕ	3/8" at 9"	16"x 14"	6 - 1" ϕ	3/8" at 9"	do.			do.		
30	5'5"x 5'5"x 1'6"	1/2" ϕ at 6" crs	both ways	16"x 14"	4 - 1" ϕ	5/8" at 9"	16"x 14"	4 - 7/8" ϕ	5/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
31.	4'0"x 4'0"x 1'7"	5/8" ϕ at 6" crs	both ways		NIL		16"x 16"	4 - 1 1/8" ϕ	3/8" at 9"	16"x 12"	4 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
32	4'9"x 4'9"x 2'3"	5/8" ϕ at 6" crs	both ways		NIL		16"x 16"	6 - 1 1/8" ϕ	3/8" at 3" Spiral	14"x 14"	6 - 1" ϕ	5/8" at 3" Spiral	12"x 12"	4 - 7/8" ϕ	3/8" at 9"
33	5'6"x 5'6"x 2'6"	5/8" ϕ at 6" crs	both ways		NIL		19"x 19"	6 - 1 1/8" ϕ	3/8" at 3" Spiral	16"x 16"	6 - 1" ϕ	3/8" at 3" Spiral	12"x 12"	4 - 1" ϕ	3/8" at 9"
34	5'9"x 5'9"x 1'9"	1/2" ϕ at 6" crs	both ways		NIL		16"x 16"	4 - 1 1/8" ϕ	3/8" at 3" Spiral	12"x 12"	4 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
35	5'3"x 5'3"x 2'3"	5/8" ϕ at 6" crs	both ways	17"x 17"	6 - 1 1/8" ϕ	3/8" at 3" Spiral	16"x 16"	6 - 1 1/8" ϕ	3/8" at 3"	12"x 12"	6 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
36	5'9"x 5'9"x 2'6"	5/8" ϕ at 6" crs	both ways	19"x 19"	6 - 1 1/2" ϕ	3/8" at 3"	19"x 19"	6 - 1" ϕ	3/8" at 9"	15"x 15"	6 - 3/4" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
37	4'0"x 4'0"x 2'0"	5/8" ϕ at 6" crs	both ways	15"x 15"	6 - 1 1/8" ϕ	3/8" at 3"	15"x 15"	6 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
38	do.			do.			do.								
39	5'0"x 5'0"x 2'3"	5/8" ϕ at 6" crs	both ways	20"x 20"	6 - 1 1/8" ϕ	3/8" at 3"	20"x 20"	6 - 1" ϕ	3/8" at 9"	14"x 14"	6 - 3/4" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
40	4'3"x 4'3"x 1'9"	5/8" ϕ at 6" crs	both ways	16"x 16"	6 - 1" ϕ	5/8" at 3"	14"x 14"	4 - 7/8" ϕ	5/8" at 9"	14"x 14"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
41	4'3"x 4'3"x 2'3"	5/8" ϕ at 6" crs	both ways				16"x 16"	5 - 1 1/8" ϕ	3/4" at 3" Spiral	14"x 14"	5 - 1" ϕ	3/8" at 4"	12"x 12"	4 - 1" ϕ	3/8" at 9"
42.	NONE. CARRIED ON BEAM.									14"x 14"	6 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
43	4'3"x 4'3"x 1'9"	5/8" ϕ at 6" crs	both ways				16"x 16"	5 - 1" ϕ	5/8" at 3" Spiral	14"x 14"	5 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
44	4'9"x 4'9"x 2'0"	5/8" ϕ at 6" crs	both ways	16"x 16"	6 - 1 1/8" ϕ	3/8" at 3"	16"x 16"	6 - 7/8" ϕ	3/8" at 3"	14"x 14"	6 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
45	5'6"x 5'6"x 1'9"	1/2" ϕ at 6" crs	both ways	15"x 15"	4 - 1 1/8" ϕ	5/8" at 9"	15"x 15"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
46	4'0"x 4'0"x 2'0"	5/8" ϕ at 6" crs	both ways	15"x 15"	6 - 1 1/8" ϕ	3/8" at 3"	15"x 15"	6 - 1" ϕ	5/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
47 & 48	NIL			NIL			NIL								
49	5'0"x 5'0"x 2'3"	5/8" ϕ at 6" crs	both ways	17"x 17"	6 - 1 1/8" ϕ	3/8" at 3" Spiral	16"x 16"	6 - 1 1/8" ϕ	3/8" at 3" Spiral	14"x 14"	6 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
50	5'3"x 5'3"x 2'3"	5/8" ϕ at 6" crs	both ways	17"x 17"	6 - 1 1/8" ϕ	3/8" at 3"	16"x 16"	6 - 1 1/8" ϕ	3/8" at 3"	14"x 14"	6 - 1" ϕ	3/8" at 5"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
51	6'6"x 6'6"x 2'9"	5/8" ϕ at 6" crs	both ways	22"x 22"	8 - 1 1/8" ϕ	3/8" at 3" Spiral	20"x 20"	8 - 1 1/4" ϕ	3/8" at 3" Spiral	14"x 14"	4 - 1" ϕ	3/8" at 4"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
52	5'6"x 5'6"x 2'6"	5/8" ϕ at 6" crs	both ways	17"x 17"	6 - 1 1/8" ϕ	3/8" at 3" Spiral	16"x 16"	6 - 1 1/4" ϕ	3/8" at 3" Spiral	14"x 14"	6 - 1" ϕ	3/8" at 6"			
53	5'6"x 5'6"x 2'3"	5/8" ϕ at 6" crs	both ways	18"x 18"	6 - 1 1/4" ϕ	3/8" at 3" Spiral	16"x 16"	6 - 1 1/4" ϕ	3/8" at 3" Spiral	14"x 14"	6 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
54	6'6"x 6'6"x 2'9"	5/8" ϕ at 6" crs	both ways	22"x 22"	7 - 1 1/4" ϕ	3/8" at 3"	20"x 20"	7 - 1 1/4" ϕ	3/8" at 3"	14"x 14"	6 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
55	3'9"x 3'9"x 1'9"	1/2" ϕ at 6" crs	both ways	15"x 15"	6 - 1" ϕ	3/8" at 3"	15"x 15"	6 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
56	4'0"x 4'0"x 2'0"	5/8" ϕ at 6" crs	both ways	15"x 15"	6 - 1 1/8" ϕ	3/8" at 3"	17"x 15"	4 - 1 1/8" ϕ	3/8" at 9"	do.			do.		
57	5'0"x 5'0"x 2'3"	5/8" ϕ at 6" crs	both ways	20"x 20"	6 - 1 1/8" ϕ	3/8" at 3"	20"x 20"	6 - 1" ϕ	3/8" at 9"	14"x 14"	6 - 3/4" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
58	4'3"x 4'3"x 1'9"	5/8" ϕ at 6" crs	both ways	16"x 16"	6 - 1" ϕ	3/8" at 3"	16"x 14"	4 - 1" ϕ	3/8" at 9"	14"x 14"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
59	4'3"x 4'3"x 2'3"	5/8" ϕ at 6" crs	both ways	16"x 16"	6 - 1 1/8" ϕ	3/8" at 3"	16"x 16"	6 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"
60 & 60A	COMBINED	1/2" ϕ at 6" crs	both ways	27"x 12"	4 - 7/8" ϕ	5/8" at 9"	27"x 12"	4 - 7/8" ϕ	5/8" at 9"	18"x 12"	4 - 3/4"	5/8" at 9"	12"x 12"	4 - 3/4" ϕ	5/8" at 9"
61 & 61A	12'0"x 6'0"x 1'6"									do.			do.		
62	5'6"x 5'6"x 2'6"	5/8" ϕ at 6" crs	both ways	17"x 17"	6 - 1 1/4" ϕ	3/8" at 3" Spiral	17"x 17"	6 - 1" ϕ	3/4" at 9"	13"x 13"	6 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 7/8"	3/8" at 9"
63 & 64.	5'6"x 5'6"x 2'6"	5/8" ϕ at 6" crs	both ways	18"x 18"	7 - 1 1/4" ϕ	3/8" at 3" Spiral	17"x 17"	6 - 1" ϕ	3/4" at 9"	15"x 15"	6 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 7/8"	3/8" at 9"
65	5'6"x 5'6"x 2'3"	5/8" ϕ at 6" crs	both ways	18"x 18"	6 - 1 1/8" ϕ	3/8" at 3"	18"x 15"	6 - 1 1/4" ϕ	3/8" at 9"	14"x 14"	6 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4"	3/8" at 9"
66	4'3"x 4'3"x 2'0"	5/8" ϕ at 6" crs	both ways	15"x 15"	6 - 1 1/8" ϕ	3/8" at 9"	15"x 15"	6 - 1 1/8" ϕ	3/8" at 9"	12"x 11"	4 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4"	3/8" at 9"
67	NIL			do.			do.			do.			do.		
68	4'0"x 4'0"x 2'0"	5/8" ϕ at 6" crs	both ways	16"x 16"	6 - 1" ϕ	3/8" at 3"	16"x 16"	4 - 1" ϕ	5/8" at 9"	12"x 12"	4 - 7/8" ϕ	5/8" at 9"	12"x 12"	4 - 3/4" ϕ	5/8" at 9"
69	5'3"x 5'3"x 1'9"	1/2" ϕ at 6" crs	both ways	16"x 16"	4 - 1 1/8"	3/8" at 9"	16"x 16"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3/4" ϕ	3/8" at 9"
70 & 71	5'9"x 5'9"x 1'6"	5/8" ϕ at 6" crs	both ways	16"x 16"	4 - 1" ϕ	3/8" at 9"	16"x 16"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3 1/2" ϕ	3/8" at 9"
72	4'0"x 4'0"x 1'9"	5/8" ϕ at 9" crs	both ways	16"x 16"	6 - 1" ϕ	3/8" at 9"	16"x 16"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3 1/2" ϕ	3/8" at 9"
73	4'3"x 4'3"x 1'9"	5/8" ϕ at 6" crs	both ways	16"x 16"	6 - 1 1/8" ϕ	3/8" at 9"	16"x 16"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3 1/2" ϕ	3/8" at 9"
74 & 75	5'6"x 5'6"x 1'9"	1/2" ϕ at 6" crs	both ways	16"x 14"	6 - 1" ϕ	3/8" at 9"	16"x 14"	6 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3 1/2" ϕ	3/8" at 9"
76	5'0"x 5'0"x 1'6"	1/2" ϕ at 6" crs	both ways	16"x 16"	4 - 1" ϕ	3/8" at 9"	16"x 16"	4 - 7/8" ϕ	3/8" at 9"	12"x 12"	4 - 7/8" ϕ	3/8" at 9"	11"x 11"	4 - 3 1/2" ϕ	3/8" at 9"
77	4'0"x 4'0"x 1'9"	3/4" ϕ at 6" crs	both ways	14"x 14"	6 - 7/8" ϕ	5/8" at 3" Spiral	10"x 10"	4 - 7/8" ϕ	5/8" at 9"						
78.	4'0"x 4'0"x 1'9"	3/4" ϕ at 6" crs	both ways	14"x 14"	5 - 1 1/8" ϕ	5/8" at 3" Spiral	12"x 12"	4 - 3/4" ϕ	5/8" at 9"						
78A	2'0"x 2'0"x 1'3"	1/2" ϕ at 6" crs	both ways	12"x 12"	4 - 5/8" ϕ	5/8" at 9"									
79	3'0"x 3'0"x 1'5"	5/8" ϕ at 6" crs	both ways	14"x 14"	4 - 1 1/8" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"						
80	4'0"x 4'0"x 1'9"	3/4" ϕ at 6" crs	both ways	14"x 14"	5 - 1" ϕ	3/8" at 9"	11"x 11"	4 - 1" ϕ	3/8" at 9"						
81	4'0"x 4'0"x 1'9"	3/4" ϕ at 6" crs	both ways	14"x 14"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"						
82	3'0"x 3'0"x 1'5"	3/4" ϕ at 6" crs	both ways	14"x 14"	4 - 1" ϕ	3/8" at 9"	12"x 12"	4 - 5/8" ϕ	5/8" at 9"						
83 & 84	2'6"x 2'6"x 1'0"	5/8" ϕ at 6" crs	both ways	18"x 18"	6" PIPE IN CENTRE		18" Diam.	6" PIPE IN CENTRE							
85, 87, 90, 92.	2'6"x 2'6"x 1'0"	5/8" ϕ at 6" crs	both ways	18"x 18"	4 - 3/4" ϕ	3/8" at 9"	18" DIAM.	4 - 3/4" ϕ	3/8" at 9"						
86, 88, 89, 91.	2'6"x 2'6"x 1'0"	5/8" at 6" crs	both ways	18"x 18"	6" PIPE IN CENTRE		18" DIAM.	6" PIPE IN CENTRE							
44 X.										14"x 14"	4 - 3/4" ϕ	3/8" at 9"	12"x 12"	4 - 3/4" ϕ	3/8" at 9"

Appendix D
Calculations

Liquefaction Triggering

Post liquefaction residual strength

Slope stability

Bearing capacity

Liquefaction Triggering Calculation
Project No: 2-37905.01
Client: Bayview / Kah NZ
Project Name: Chateau Tongariro

Office: Taupō
Date: 21/11/2022
Author: 9(2)(a)

Calculation Method: Idriss & Boulanger 2014 (CPT and SPT Based Liquefaction Triggering Procedures Report No. UCD/CGM-14/01)

Software: CLiq v.3.0.2.4 (Geologismiki)

Inputs

Seismicity: PGA's from MBIE Module 1 Appendix A are adopted. Note that the subject site (Chateau Tongariro) is not located in a town or city with published PGAs. The PGAs for the nearest towns with published PGAs are used instead (Ohakune, 28km, Waiouru, 32km)

Design PGAs for 1/25 to 1/500 year return periods are listed below:

- 25 year earthquake: 0.09g
- 50 year earthquake: 0.13g
- 100 year earthquake: 0.18g
- 250 year earthquake: 0.27g
- 500 year earthquake: 0.36g
- 1000 year earthquake: 0.47g
- 2500 year earthquake: 0.64g

Earthquake Magnitude: 6.3

Groundwater: Measurements of two piezometers on site resulted in groundwater levels of 5.1 – 5.2m bgl in September 2022 and 5.05m bgl in November 2022.

We have adopted a groundwater level of 5m bgl.

Ground conditions: Soil conditions essentially consist of boulders in sand and gravel matrix up to 15m which is the maximum depth investigated. It is possible that if the matrix is saturated and loose enough and / or subject to enough earthquake shaking, liquefaction of the matrix could occur.

Standard Penetration Tests: SPTs from two boreholes are the basis of the assessment. We have assessed two cases. In Case 1, the SPTs from each borehole are input directly into the calculation. In Case 2, the SPTs are adjusted using the test results judged to be most representative of the sand and gravel matrix.

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Depth (m)	BH1 – SPT	BH1 – Log Notes	BH2 – SPT	BH2 – Log Notes
1.5	50	73mm movement	21	No notes, full SPT
3.0	50	Bouncing, no movement	50	No notes, full SPT
4.5	50	35mm movement	50	205mm movement
6.0	50	Bouncing, no movement	35	83mm movement
7.5	50	50mm movement	25	45mm movement
9.0	50	65mm movement	25	75mm movement
10.5	35	No notes, full SPT	25	Bouncing (no movement)
12.0	30	10mm movement	30	Bouncing (no movement)
13.5	50	Bouncing, no movement	30	Bouncing (no movement)
15.0	50	Bouncing, no movement	N/A	N/A

Results:

Results of the calculations are summarised in the attached PDF outputs from CLIQ. In summary, no liquefaction triggering is predicted for 'Case 1' where the SPTs are input as-is into the software. In 'Case 2' liquefaction triggering is possible at depth, beginning at 13.5m with a 0.36g PGA and 15m with a 0.34g PGA. Triggering occurs @ 9.0m at 0.47g (1 in 1000 year) and at 6.0m at 0.64g (1 in 2500).

LPI and LPIISH:

LPI is calculated using the following equations:

$$LPI = \int_0^{20m} Fw(z)dz$$

$$F = 1 - FS \quad \text{for } FS \leq 1, \text{ and}$$

$$F = 0 \quad \text{for } FS > 1, \text{ and}$$

$$w(z) = 10 - 0.5z$$

LPIISH is calculated using the following equations:

$$LPI_{ISH} = \int_0^{20\,m} F(FS)\frac{25.56}{z}\,dz$$

$$F(FS) = \begin{cases} 1 - FS \; if \; FS \leq 1 \cap H_1 \cdot m(FS) \leq 3 \\ 0 \; otherwise \end{cases}$$

$$m(FS) = exp\left(\frac{5}{25.56(1 - FS)}\right) - 1$$

The results are below:

PGA (g)	LPI_{lsh}	LPI
0.09	0	0
0.13	0	0
0.18	0	0
0.27	0	0
0.36	0	1
0.47	0	4
0.64	3	10

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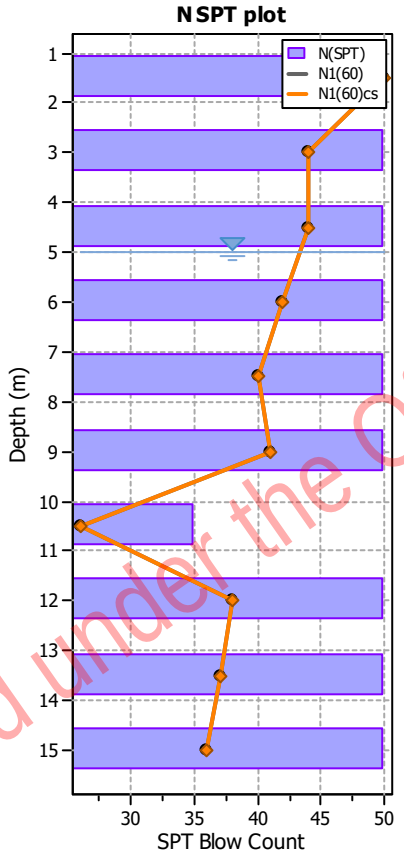
SPT BASED LIQUEFACTION ANALYSIS REPORT

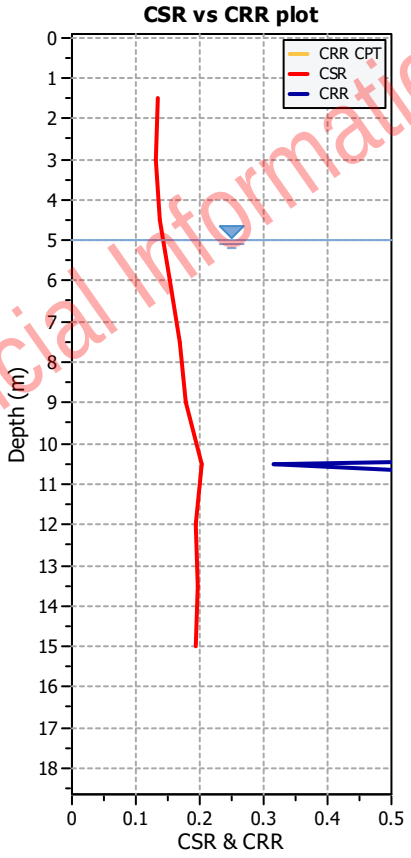
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

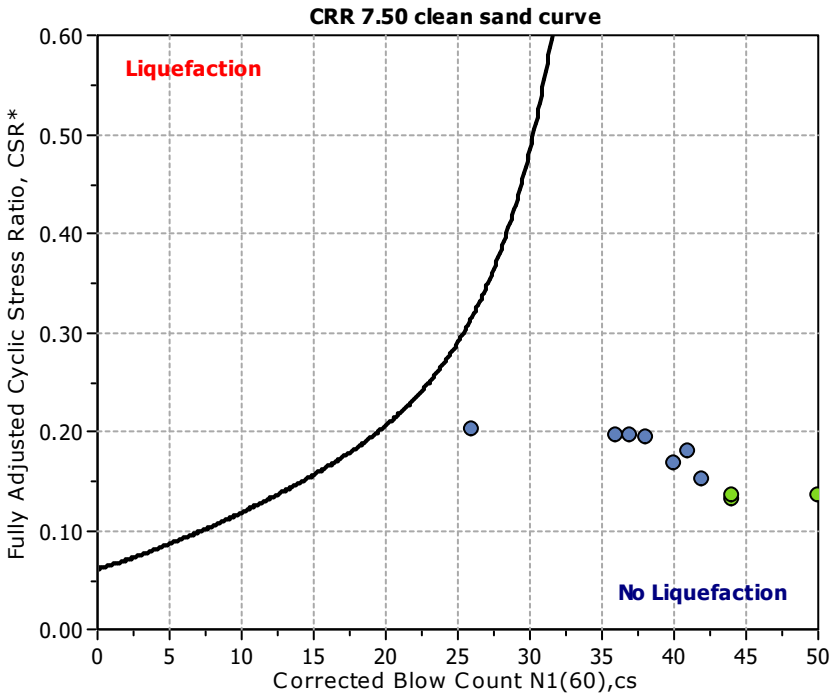
Borehole Name : BH1

:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.36		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				







:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_o (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.231	1.10	2.20	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.226	1.10	2.20	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.219	1.03	2.20	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.231	0.97	2.20	1.37	0.152
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.245	0.93	2.20	1.37	0.168
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.252	0.90	2.20	1.37	0.179
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.254	0.93	1.77	1.37	0.202
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.253	0.84	2.20	1.37	0.193
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.250	0.82	2.20	1.37	0.196
10	15.00	20.00	98.10	300.00	0.00	201.90	0.71	0.245	0.81	2.20	1.37	0.195

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_o (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	50	1.38	0.80	1.00	1.00	0.90	50	0.00	50	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	50	1.15	0.85	1.00	1.00	0.90	44	0.00	44	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	50	1.03	0.95	1.00	1.00	0.90	44	0.00	44	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	50	0.98	0.95	1.00	1.00	0.90	42	0.00	42	8.408	2.00
5	7.50	0.00	24.53	150.00	125.47	50	0.94	0.95	1.00	1.00	0.90	40	0.00	40	4.133	2.00
6	9.00	0.00	39.24	180.00	140.76	50	0.91	1.00	1.00	1.00	0.90	41	0.00	41	5.808	2.00
7	10.50	0.00	53.96	210.00	156.04	35	0.84	1.00	1.00	1.00	0.90	26	0.00	26	0.316	1.56
8	12.00	0.00	68.67	240.00	171.33	50	0.85	1.00	1.00	1.00	0.90	38	0.00	38	2.273	2.00
9	13.50	0.00	83.39	270.00	186.61	50	0.82	1.00	1.00	1.00	0.90	37	0.00	37	1.750	2.00
10	15.00	0.00	98.10	300.00	201.90	50	0.80	1.00	1.00	1.00	0.90	36	0.00	36	1.378	2.00

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

WSP

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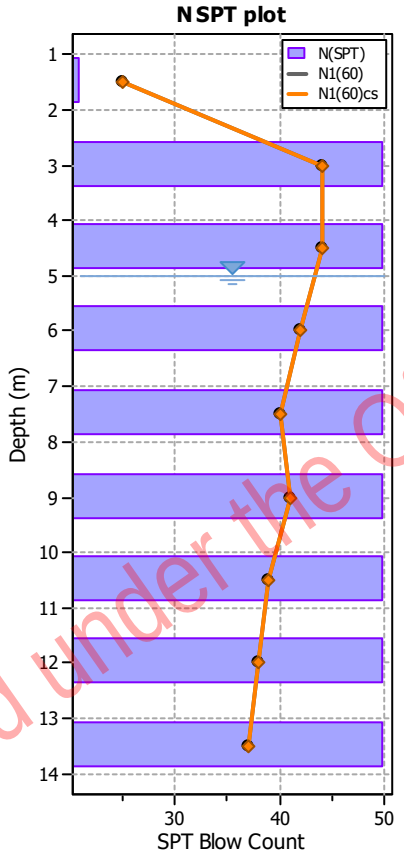
SPT BASED LIQUEFACTION ANALYSIS REPORT

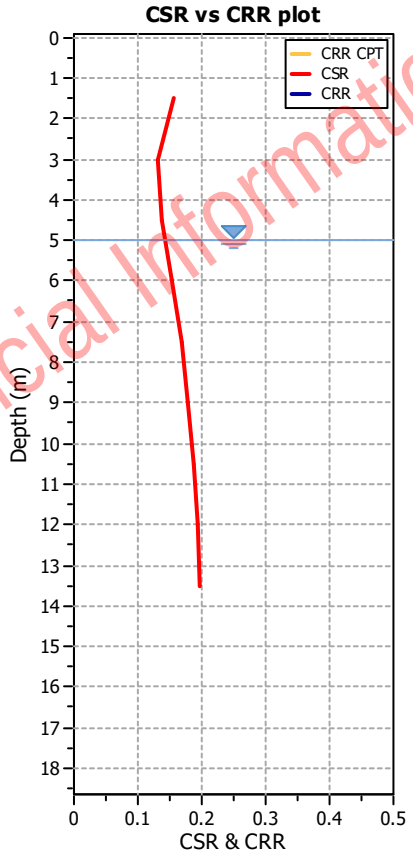
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

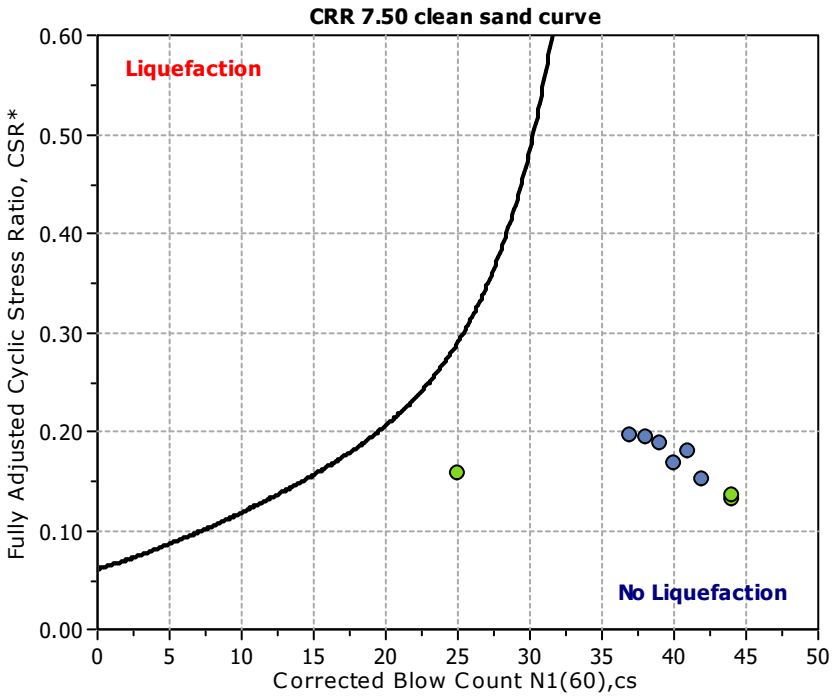
Borehole Name : BH1

:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.36		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				







:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_o (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.231	1.10	1.72	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.226	1.10	2.20	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.219	1.03	2.20	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.231	0.97	2.20	1.37	0.152
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.245	0.93	2.20	1.37	0.168
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.252	0.90	2.20	1.37	0.179
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.254	0.87	2.20	1.37	0.188
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.253	0.84	2.20	1.37	0.193
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.250	0.82	2.20	1.37	0.196

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_o (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	21	1.64	0.80	1.00	1.00	0.90	25	0.00	25	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	50	1.15	0.85	1.00	1.00	0.90	44	0.00	44	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	50	1.03	0.95	1.00	1.00	0.90	44	0.00	44	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	50	0.98	0.95	1.00	1.00	0.90	42	0.00	42	8.408	2.00
5	7.50	0.00	24.53	150.00	125.47	50	0.94	0.95	1.00	1.00	0.90	40	0.00	40	4.133	2.00
6	9.00	0.00	39.24	180.00	140.76	50	0.91	1.00	1.00	1.00	0.90	41	0.00	41	5.808	2.00
7	10.50	0.00	53.96	210.00	156.04	50	0.88	1.00	1.00	1.00	0.90	39	0.00	39	3.025	2.00
8	12.00	0.00	68.67	240.00	171.33	50	0.85	1.00	1.00	1.00	0.90	38	0.00	38	2.273	2.00
9	13.50	0.00	83.39	270.00	186.61	50	0.82	1.00	1.00	1.00	0.90	37	0.00	37	1.750	2.00

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

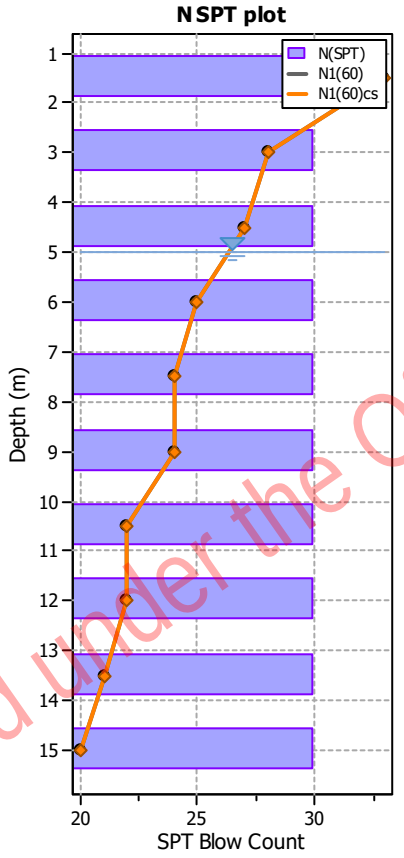
SPT BASED LIQUEFACTION ANALYSIS REPORT

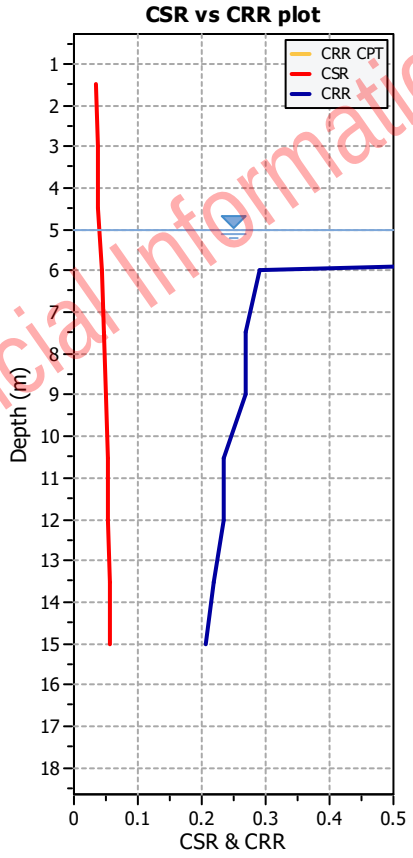
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

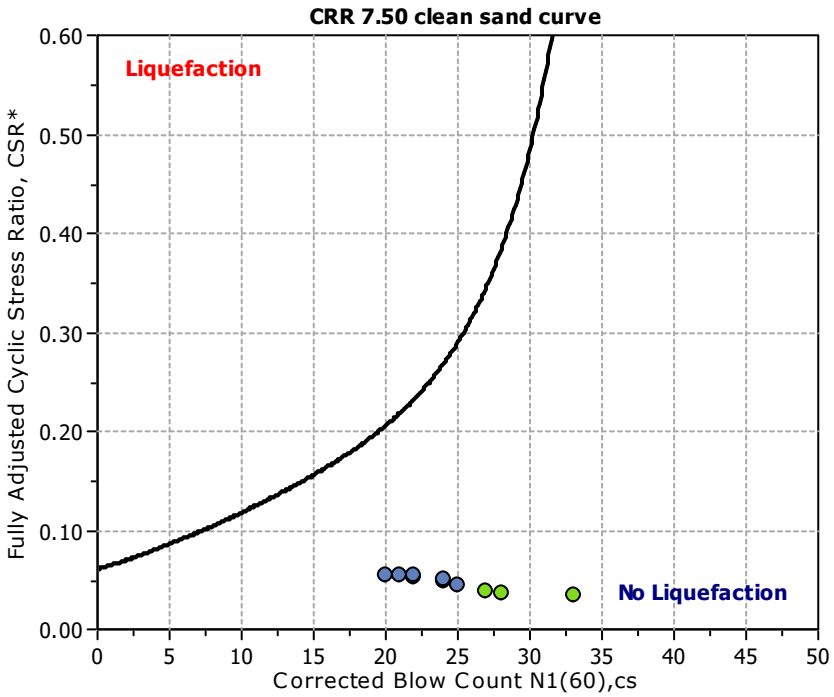
Borehole Name : BH1_BH2

:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.09		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				







:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_0 (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.058	1.10	2.19	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.056	1.10	1.88	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.055	1.02	1.82	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.058	0.99	1.72	1.37	0.044
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.061	0.97	1.67	1.37	0.048
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.063	0.95	1.67	1.37	0.051
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.063	0.94	1.58	1.37	0.053
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.063	0.92	1.58	1.37	0.054
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.062	0.91	1.53	1.37	0.055
10	15.00	20.00	98.10	300.00	0.00	201.90	0.71	0.061	0.91	1.49	1.37	0.055

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_0 (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	30	1.52	0.80	1.00	1.00	0.90	33	0.00	33	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	30	1.22	0.85	1.00	1.00	0.90	28	0.00	28	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	30	1.05	0.95	1.00	1.00	0.90	27	0.00	27	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	30	0.97	0.95	1.00	1.00	0.90	25	0.00	25	0.290	2.00
5	7.50	0.00	24.53	150.00	125.47	30	0.92	0.95	1.00	1.00	0.90	24	0.00	24	0.268	2.00
6	9.00	0.00	39.24	180.00	140.76	30	0.87	1.00	1.00	1.00	0.90	24	0.00	24	0.268	2.00
7	10.50	0.00	53.96	210.00	156.04	30	0.83	1.00	1.00	1.00	0.90	22	0.00	22	0.233	2.00
8	12.00	0.00	68.67	240.00	171.33	30	0.80	1.00	1.00	1.00	0.90	22	0.00	22	0.233	2.00
9	13.50	0.00	83.39	270.00	186.61	30	0.77	1.00	1.00	1.00	0.90	21	0.00	21	0.219	2.00
10	15.00	0.00	98.10	300.00	201.90	30	0.74	1.00	1.00	1.00	0.90	20	0.00	20	0.206	2.00

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

SPT BASED LIQUEFACTION ANALYSIS REPORT

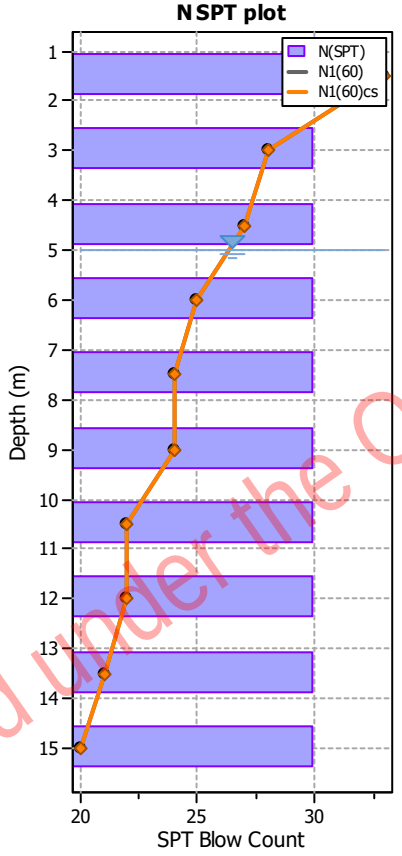
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

Borehole Name : BH1_BH2

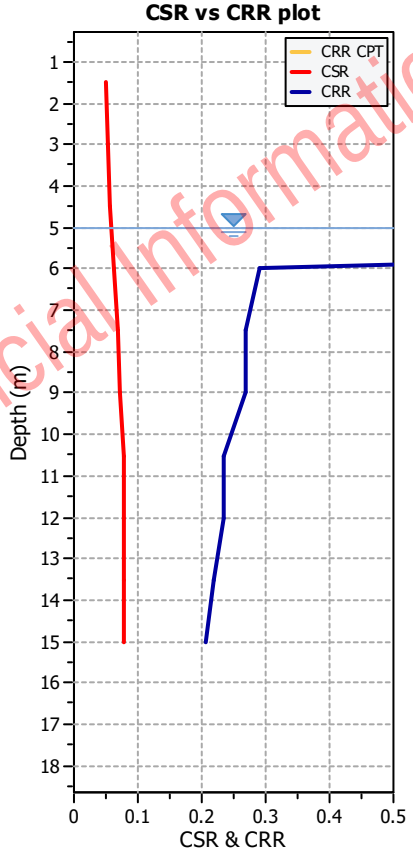
:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.13		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				

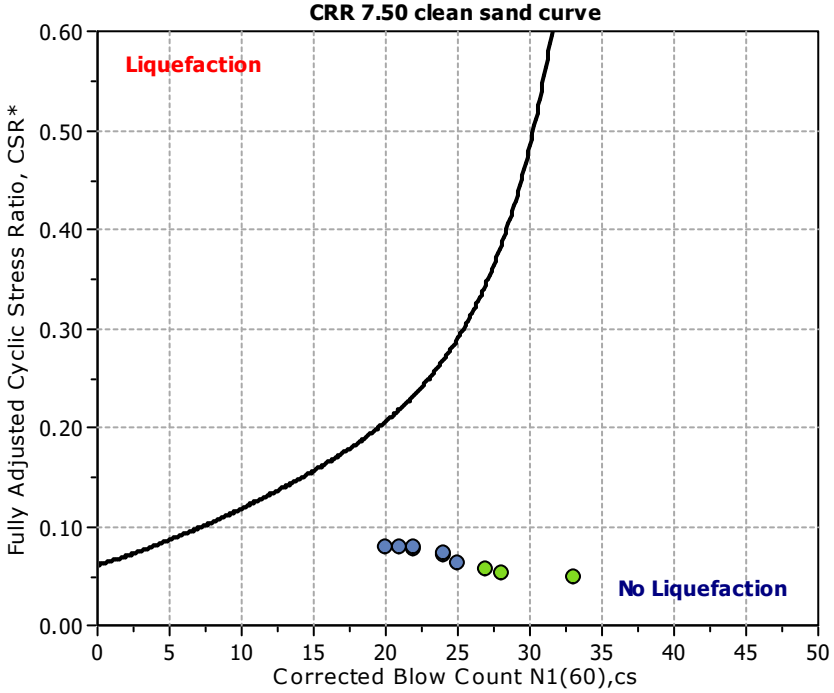
N SPT plot



CSR vs CRR plot



CRR 7.50 clean sand curve



:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_o (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.083	1.10	2.19	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.081	1.10	1.88	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.079	1.02	1.82	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.083	0.99	1.72	1.37	0.063
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.088	0.97	1.67	1.37	0.070
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.091	0.95	1.67	1.37	0.073
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.092	0.94	1.58	1.37	0.077
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.091	0.92	1.58	1.37	0.078
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.090	0.91	1.53	1.37	0.079
10	15.00	20.00	98.10	300.00	0.00	201.90	0.71	0.089	0.91	1.49	1.37	0.079

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_o (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	30	1.52	0.80	1.00	1.00	0.90	33	0.00	33	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	30	1.22	0.85	1.00	1.00	0.90	28	0.00	28	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	30	1.05	0.95	1.00	1.00	0.90	27	0.00	27	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	30	0.97	0.95	1.00	1.00	0.90	25	0.00	25	0.290	2.00
5	7.50	0.00	24.53	150.00	125.47	30	0.92	0.95	1.00	1.00	0.90	24	0.00	24	0.268	2.00
6	9.00	0.00	39.24	180.00	140.76	30	0.87	1.00	1.00	1.00	0.90	24	0.00	24	0.268	2.00
7	10.50	0.00	53.96	210.00	156.04	30	0.83	1.00	1.00	1.00	0.90	22	0.00	22	0.233	2.00
8	12.00	0.00	68.67	240.00	171.33	30	0.80	1.00	1.00	1.00	0.90	22	0.00	22	0.233	2.00
9	13.50	0.00	83.39	270.00	186.61	30	0.77	1.00	1.00	1.00	0.90	21	0.00	21	0.219	2.00
10	15.00	0.00	98.10	300.00	201.90	30	0.74	1.00	1.00	1.00	0.90	20	0.00	20	0.206	2.00

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

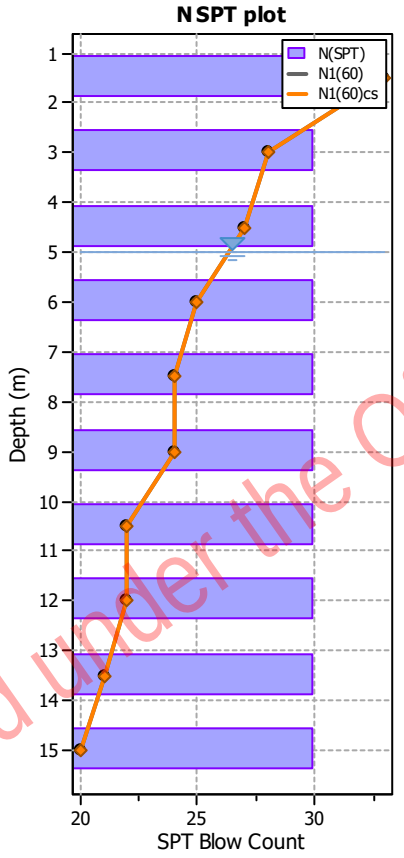
SPT BASED LIQUEFACTION ANALYSIS REPORT

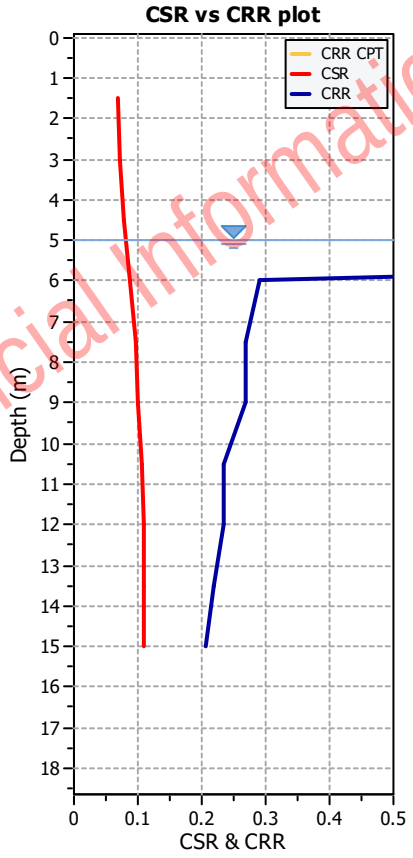
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

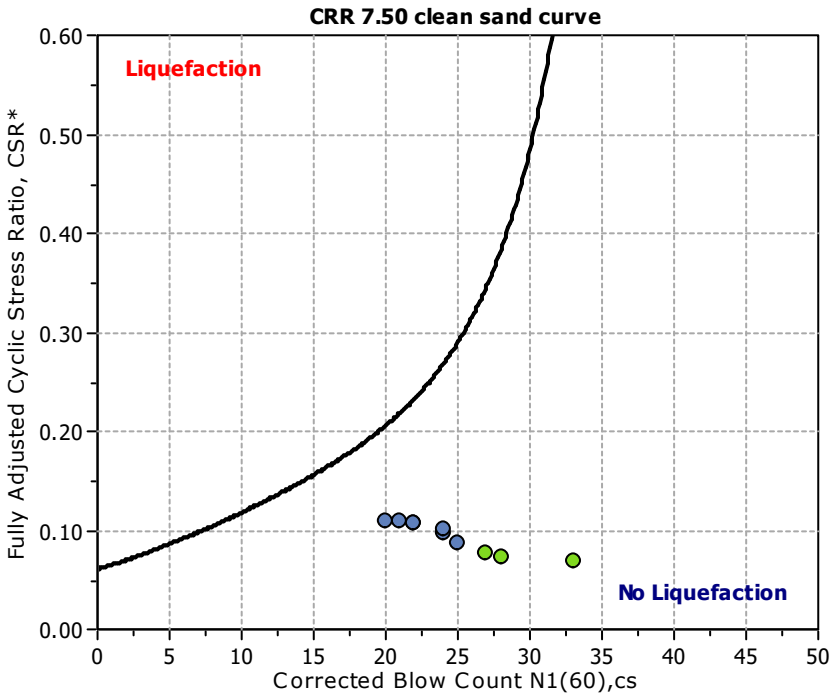
Borehole Name : BH1_BH2

:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.18		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				







:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_o (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.116	1.10	2.19	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.113	1.10	1.88	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.110	1.02	1.82	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.115	0.99	1.72	1.37	0.088
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.122	0.97	1.67	1.37	0.097
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.126	0.95	1.67	1.37	0.101
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.127	0.94	1.58	1.37	0.107
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.127	0.92	1.58	1.37	0.108
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.125	0.91	1.53	1.37	0.109
10	15.00	20.00	98.10	300.00	0.00	201.90	0.71	0.123	0.91	1.49	1.37	0.110

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_o (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	30	1.52	0.80	1.00	1.00	0.90	33	0.00	33	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	30	1.22	0.85	1.00	1.00	0.90	28	0.00	28	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	30	1.05	0.95	1.00	1.00	0.90	27	0.00	27	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	30	0.97	0.95	1.00	1.00	0.90	25	0.00	25	0.290	2.00
5	7.50	0.00	24.53	150.00	125.47	30	0.92	0.95	1.00	1.00	0.90	24	0.00	24	0.268	2.00
6	9.00	0.00	39.24	180.00	140.76	30	0.87	1.00	1.00	1.00	0.90	24	0.00	24	0.268	2.00
7	10.50	0.00	53.96	210.00	156.04	30	0.83	1.00	1.00	1.00	0.90	22	0.00	22	0.233	2.00
8	12.00	0.00	68.67	240.00	171.33	30	0.80	1.00	1.00	1.00	0.90	22	0.00	22	0.233	2.00
9	13.50	0.00	83.39	270.00	186.61	30	0.77	1.00	1.00	1.00	0.90	21	0.00	21	0.219	2.00
10	15.00	0.00	98.10	300.00	201.90	30	0.74	1.00	1.00	1.00	0.90	20	0.00	20	0.206	1.87

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

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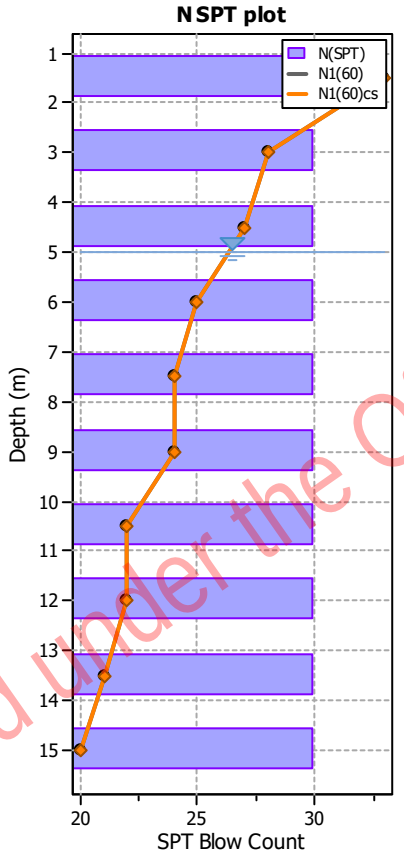
SPT BASED LIQUEFACTION ANALYSIS REPORT

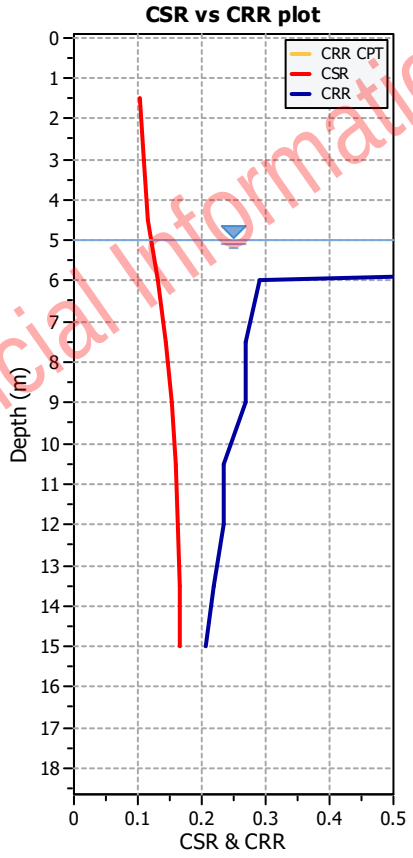
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

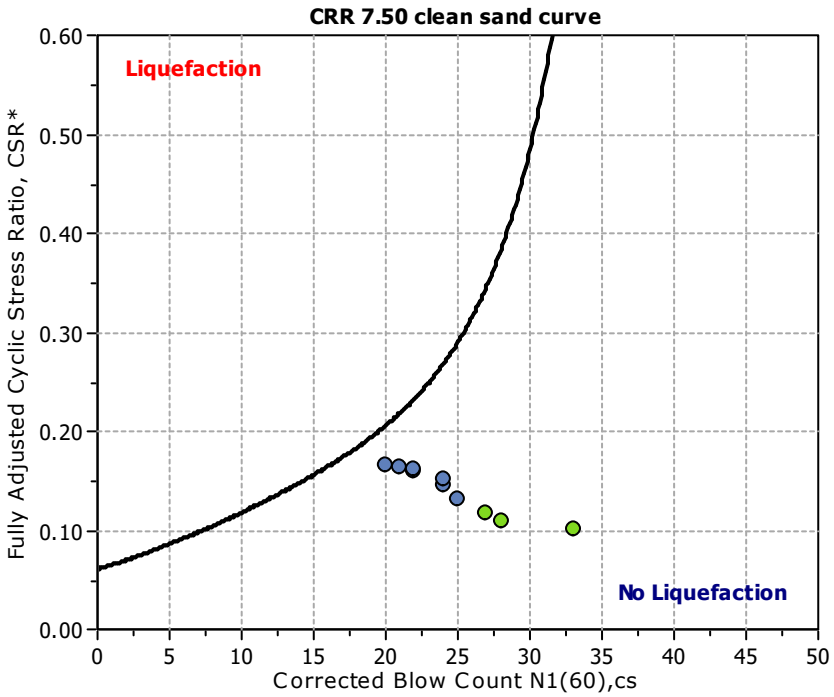
Borehole Name : BH1_BH2

:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.27		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				







:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_o (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.173	1.10	2.19	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.169	1.10	1.88	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.164	1.02	1.82	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.173	0.99	1.72	1.37	0.132
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.183	0.97	1.67	1.37	0.145
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.189	0.95	1.67	1.37	0.152
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.190	0.94	1.58	1.37	0.160
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.190	0.92	1.58	1.37	0.162
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.187	0.91	1.53	1.37	0.164
10	15.00	20.00	98.10	300.00	0.00	201.90	0.71	0.184	0.91	1.49	1.37	0.165

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_o (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	30	1.52	0.80	1.00	1.00	0.90	33	0.00	33	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	30	1.22	0.85	1.00	1.00	0.90	28	0.00	28	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	30	1.05	0.95	1.00	1.00	0.90	27	0.00	27	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	30	0.97	0.95	1.00	1.00	0.90	25	0.00	25	0.290	2.00
5	7.50	0.00	24.53	150.00	125.47	30	0.92	0.95	1.00	1.00	0.90	24	0.00	24	0.268	1.85
6	9.00	0.00	39.24	180.00	140.76	30	0.87	1.00	1.00	1.00	0.90	24	0.00	24	0.268	1.77
7	10.50	0.00	53.96	210.00	156.04	30	0.83	1.00	1.00	1.00	0.90	22	0.00	22	0.233	1.45
8	12.00	0.00	68.67	240.00	171.33	30	0.80	1.00	1.00	1.00	0.90	22	0.00	22	0.233	1.44
9	13.50	0.00	83.39	270.00	186.61	30	0.77	1.00	1.00	1.00	0.90	21	0.00	21	0.219	1.33
10	15.00	0.00	98.10	300.00	201.90	30	0.74	1.00	1.00	1.00	0.90	20	0.00	20	0.206	1.25

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

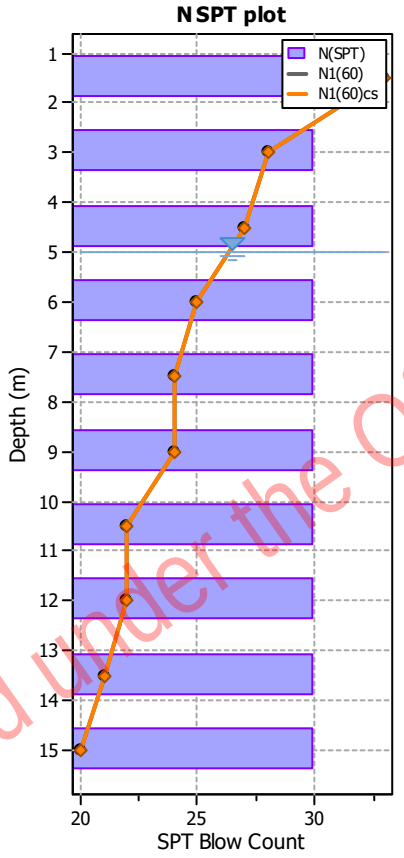
SPT BASED LIQUEFACTION ANALYSIS REPORT

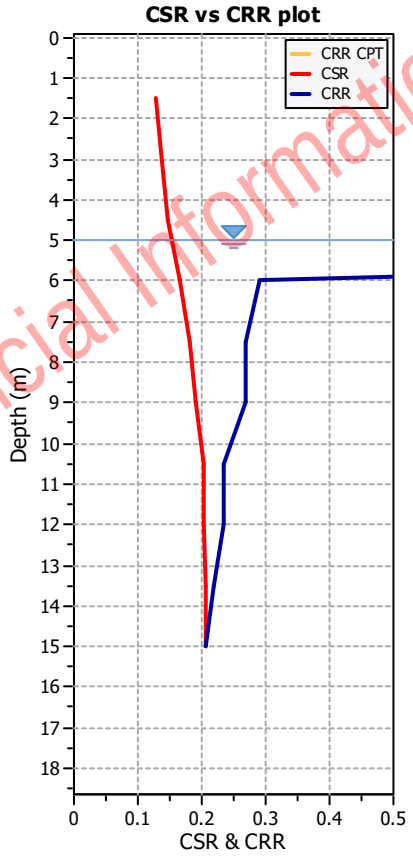
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

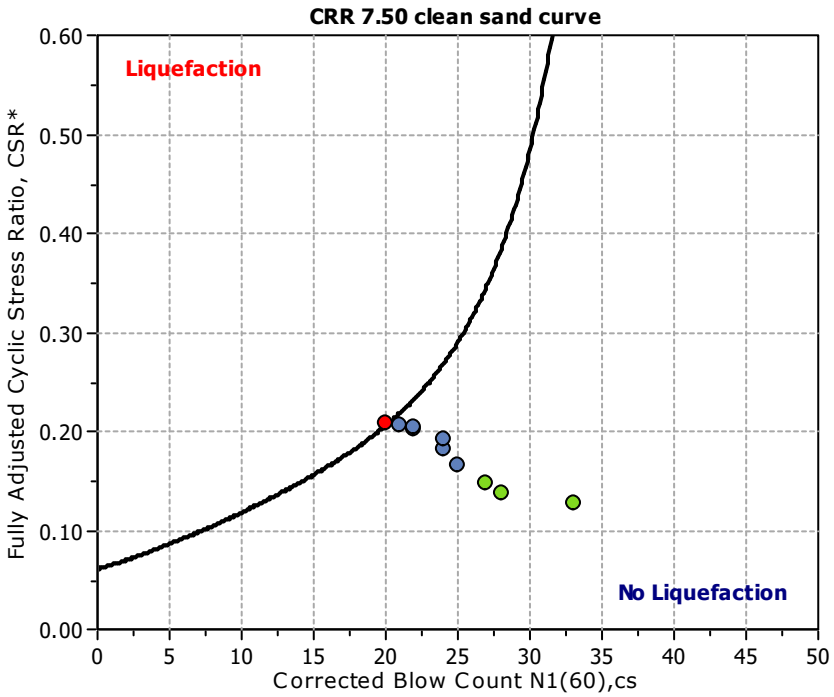
Borehole Name : BH1_BH2_Case2

:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.34		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				







:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_o (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.218	1.10	2.19	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.213	1.10	1.88	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.207	1.02	1.82	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.218	0.99	1.72	1.37	0.166
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.231	0.97	1.67	1.37	0.182
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.238	0.95	1.67	1.37	0.191
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.240	0.94	1.58	1.37	0.202
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.239	0.92	1.58	1.37	0.204
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.236	0.91	1.53	1.37	0.207
10	15.00	20.00	98.10	300.00	0.00	201.90	0.71	0.232	0.91	1.49	1.37	0.208

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_o (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	30	1.52	0.80	1.00	1.00	0.90	33	0.00	33	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	30	1.22	0.85	1.00	1.00	0.90	28	0.00	28	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	30	1.05	0.95	1.00	1.00	0.90	27	0.00	27	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	30	0.97	0.95	1.00	1.00	0.90	25	0.00	25	0.290	1.75
5	7.50	0.00	24.53	150.00	125.47	30	0.92	0.95	1.00	1.00	0.90	24	0.00	24	0.268	1.47
6	9.00	0.00	39.24	180.00	140.76	30	0.87	1.00	1.00	1.00	0.90	24	0.00	24	0.268	1.40
7	10.50	0.00	53.96	210.00	156.04	30	0.83	1.00	1.00	1.00	0.90	22	0.00	22	0.233	1.15
8	12.00	0.00	68.67	240.00	171.33	30	0.80	1.00	1.00	1.00	0.90	22	0.00	22	0.233	1.14
9	13.50	0.00	83.39	270.00	186.61	30	0.77	1.00	1.00	1.00	0.90	21	0.00	21	0.219	1.06
10	15.00	0.00	98.10	300.00	201.90	30	0.74	1.00	1.00	1.00	0.90	20	0.00	20	0.206	0.99

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

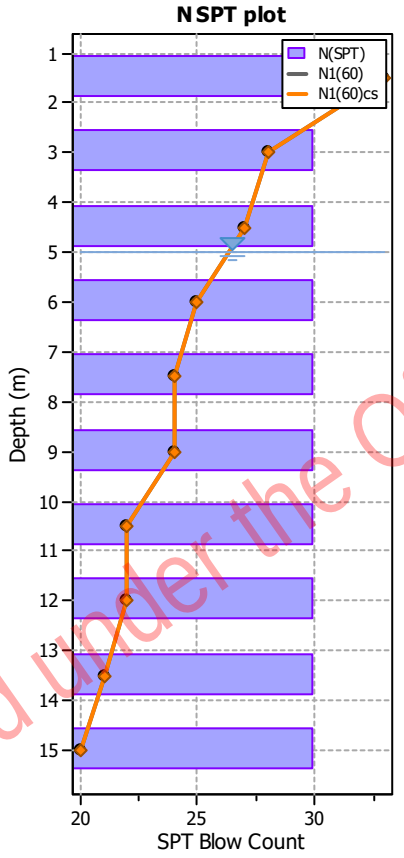
SPT BASED LIQUEFACTION ANALYSIS REPORT

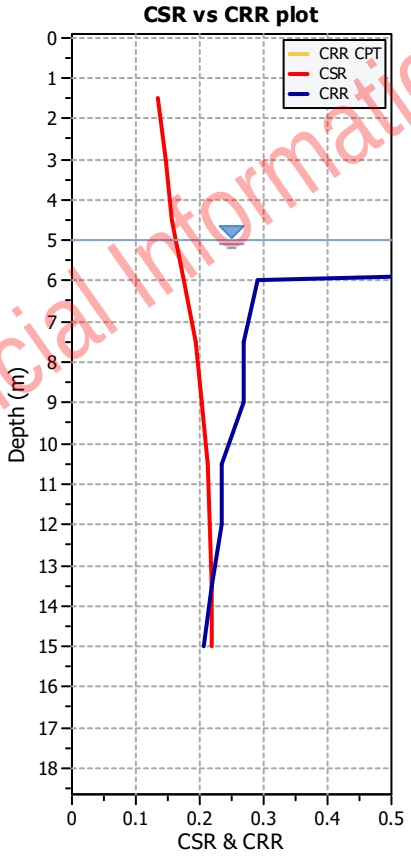
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

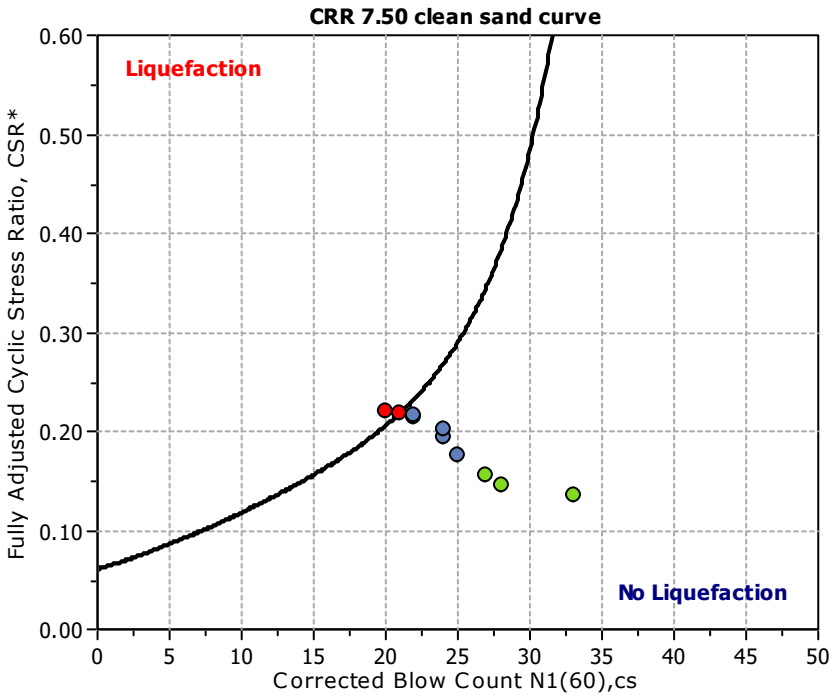
Borehole Name : BH1_BH2_Case2

:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.36		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				







:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_o (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.231	1.10	2.19	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.226	1.10	1.88	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.219	1.02	1.82	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.231	0.99	1.72	1.37	0.176
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.245	0.97	1.67	1.37	0.193
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.252	0.95	1.67	1.37	0.202
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.254	0.94	1.58	1.37	0.214
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.253	0.92	1.58	1.37	0.216
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.250	0.91	1.53	1.37	0.219
10	15.00	20.00	98.10	300.00	0.00	201.90	0.71	0.245	0.91	1.49	1.37	0.220

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_o (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	30	1.52	0.80	1.00	1.00	0.90	33	0.00	33	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	30	1.22	0.85	1.00	1.00	0.90	28	0.00	28	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	30	1.05	0.95	1.00	1.00	0.90	27	0.00	27	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	30	0.97	0.95	1.00	1.00	0.90	25	0.00	25	0.290	1.65
5	7.50	0.00	24.53	150.00	125.47	30	0.92	0.95	1.00	1.00	0.90	24	0.00	24	0.268	1.39
6	9.00	0.00	39.24	180.00	140.76	30	0.87	1.00	1.00	1.00	0.90	24	0.00	24	0.268	1.33
7	10.50	0.00	53.96	210.00	156.04	30	0.83	1.00	1.00	1.00	0.90	22	0.00	22	0.233	1.09
8	12.00	0.00	68.67	240.00	171.33	30	0.80	1.00	1.00	1.00	0.90	22	0.00	22	0.233	1.08
9	13.50	0.00	83.39	270.00	186.61	30	0.77	1.00	1.00	1.00	0.90	21	0.00	21	0.219	1.00
10	15.00	0.00	98.10	300.00	201.90	30	0.74	1.00	1.00	1.00	0.90	20	0.00	20	0.206	0.94

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

SPT BASED LIQUEFACTION ANALYSIS REPORT

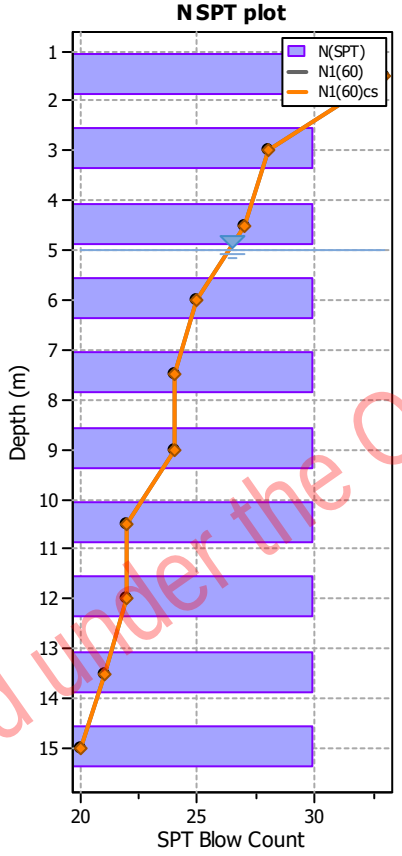
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

Borehole Name : BH1_BH2_Case2

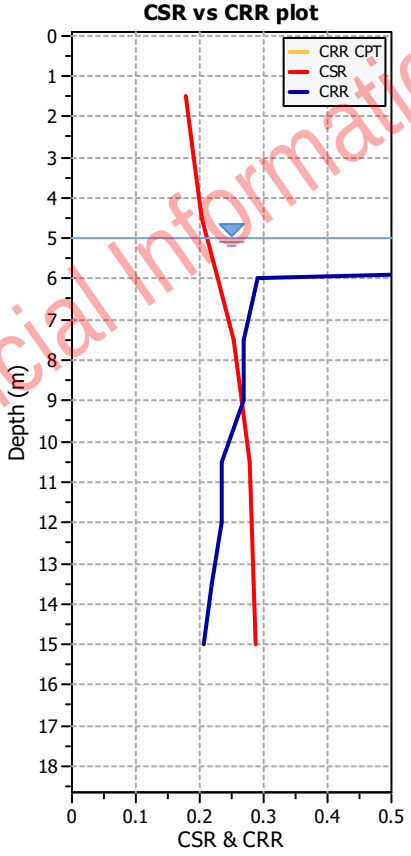
:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.47		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				

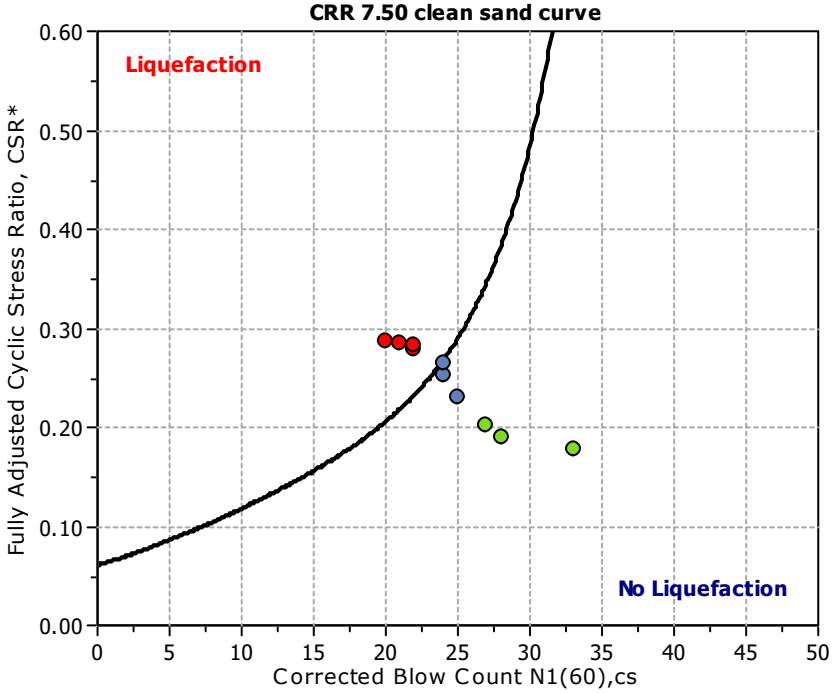
N SPT plot



CSR vs CRR plot



CRR 7.50 clean sand curve



:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_o (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.302	1.10	2.19	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.294	1.10	1.88	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.286	1.02	1.82	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.302	0.99	1.72	1.37	0.229
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.319	0.97	1.67	1.37	0.252
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.328	0.95	1.67	1.37	0.264
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.331	0.94	1.58	1.37	0.279
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.330	0.92	1.58	1.37	0.282
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.326	0.91	1.53	1.37	0.286
10	15.00	20.00	98.10	300.00	0.00	201.90	0.71	0.320	0.91	1.49	1.37	0.287

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_o (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	30	1.52	0.80	1.00	1.00	0.90	33	0.00	33	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	30	1.22	0.85	1.00	1.00	0.90	28	0.00	28	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	30	1.05	0.95	1.00	1.00	0.90	27	0.00	27	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	30	0.97	0.95	1.00	1.00	0.90	25	0.00	25	0.290	1.26
5	7.50	0.00	24.53	150.00	125.47	30	0.92	0.95	1.00	1.00	0.90	24	0.00	24	0.268	1.06
6	9.00	0.00	39.24	180.00	140.76	30	0.87	1.00	1.00	1.00	0.90	24	0.00	24	0.268	1.01
7	10.50	0.00	53.96	210.00	156.04	30	0.83	1.00	1.00	1.00	0.90	22	0.00	22	0.233	0.84
8	12.00	0.00	68.67	240.00	171.33	30	0.80	1.00	1.00	1.00	0.90	22	0.00	22	0.233	0.83
9	13.50	0.00	83.39	270.00	186.61	30	0.77	1.00	1.00	1.00	0.90	21	0.00	21	0.219	0.77
10	15.00	0.00	98.10	300.00	201.90	30	0.74	1.00	1.00	1.00	0.90	20	0.00	20	0.206	0.72

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

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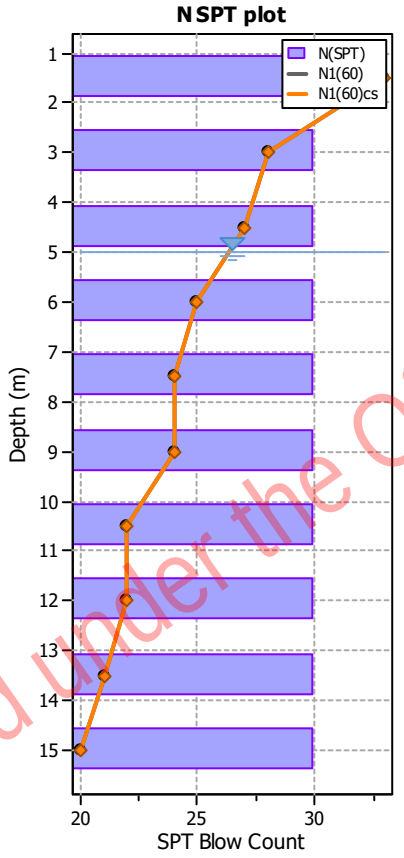
SPT BASED LIQUEFACTION ANALYSIS REPORT

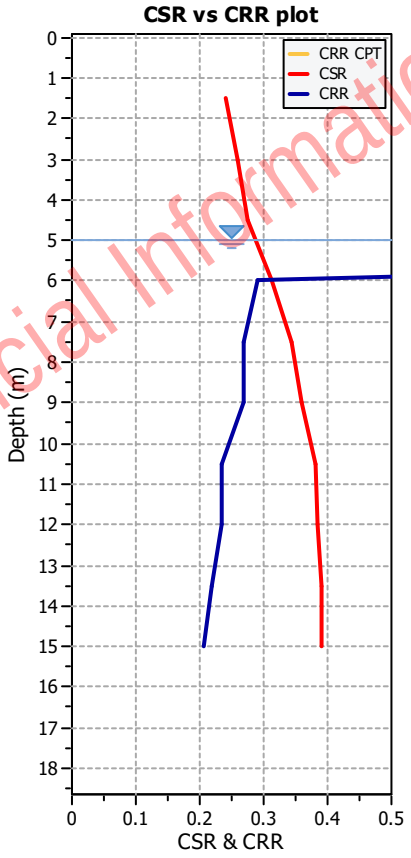
Project title : Chateau Tongariro DSA **Location : Chateau Tongariro**

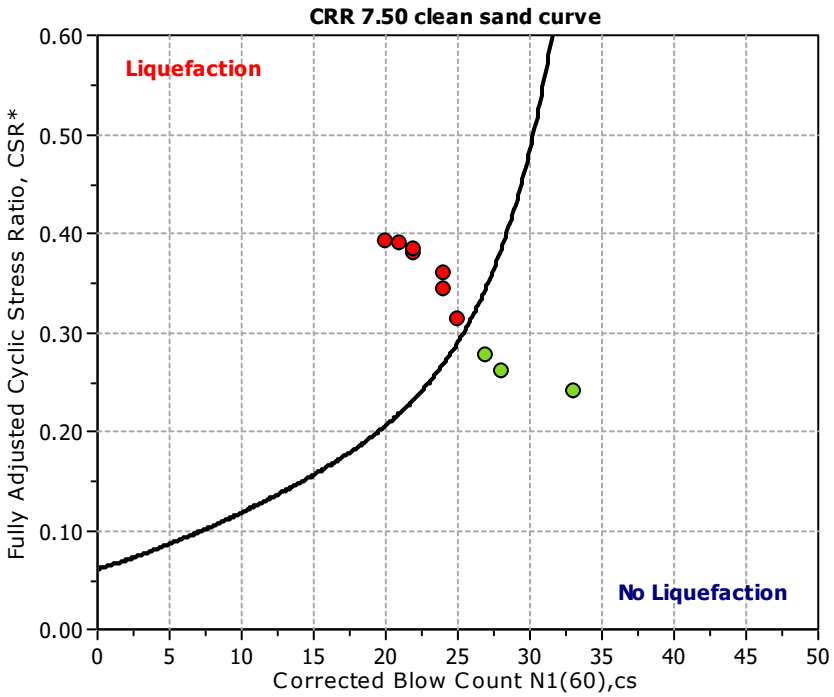
Borehole Name : BH1_BH2_Case2

:: Input parameters and analysis properties ::

Analysis method:	Idriss & Boulanger 2014	G.W.T. (in-situ):	5.00	EQ site conditions:	Same as initial
Fines correction method:	Idriss & Boulanger 2014	G.W.T. (earthq.):	5.00		
Sampling method:	Standard Sample	Earthquake magnitude M_w:	6.30		
Borehole diameter:	65 mm to 11 5mm	Peak ground acceleration:	0.64		
Rod length:	1.50	SPT results rounding mode:	Nearest		
Hammer energy ratio:	0.90				







:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (m)	Weight (kN/m³)	u_o (kPa)	σ_v (kPa)	Ext. Load (kPa)	σ'_v (kPa)	r_d	CSR	K_σ	MSF_{max}	MSF	CSR*
1	1.50	20.00	0.00	30.00	0.00	30.00	0.99	0.411	1.10	2.19	1.37	2.000
2	3.00	20.00	0.00	60.00	0.00	60.00	0.96	0.401	1.10	1.88	1.37	2.000
3	4.50	20.00	0.00	90.00	0.00	90.00	0.94	0.390	1.02	1.82	1.37	2.000
4	6.00	20.00	9.81	120.00	0.00	110.19	0.91	0.411	0.99	1.72	1.37	0.312
5	7.50	20.00	24.53	150.00	0.00	125.47	0.87	0.435	0.97	1.67	1.37	0.343
6	9.00	20.00	39.24	180.00	0.00	140.76	0.84	0.447	0.95	1.67	1.37	0.360
7	10.50	20.00	53.96	210.00	0.00	156.04	0.81	0.451	0.94	1.58	1.37	0.380
8	12.00	20.00	68.67	240.00	0.00	171.33	0.77	0.450	0.92	1.58	1.37	0.384
9	13.50	20.00	83.39	270.00	0.00	186.61	0.74	0.444	0.91	1.53	1.37	0.389
10	15.00	20.00	98.10	300.00	0.00	201.90	0.71	0.436	0.91	1.49	1.37	0.391

Abbreviations

Depth: Depth from free surface where SPT was performed (m) during eq.
u_0: Water pressure at test point (kPa) during eq.
σ_v: Total overburden pressure at test point (kPa) during eq.
σ_v': Effective overburden pressure based on GWT during earthquake (kPa) during eq.
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (m)	Fines %	u_o (kPa)	σ_v (kPa)	σ'_v (kPa)	N_{SPT}	C_N	C_R	C_B	C_S	C_E	$N_{1(60)}$	$\Delta(N_1)_{60}$	$N_{1(60),cs}$	$CRR_{7.5}$	F.S.
1	1.50	0.00	0.00	30.00	30.00	30	1.52	0.80	1.00	1.00	0.90	33	0.00	33	4.000	2.00
2	3.00	0.00	0.00	60.00	60.00	30	1.22	0.85	1.00	1.00	0.90	28	0.00	28	4.000	2.00
3	4.50	0.00	0.00	90.00	90.00	30	1.05	0.95	1.00	1.00	0.90	27	0.00	27	4.000	2.00
4	6.00	0.00	9.81	120.00	110.19	30	0.97	0.95	1.00	1.00	0.90	25	0.00	25	0.290	0.93
5	7.50	0.00	24.53	150.00	125.47	30	0.92	0.95	1.00	1.00	0.90	24	0.00	24	0.268	0.78
6	9.00	0.00	39.24	180.00	140.76	30	0.87	1.00	1.00	1.00	0.90	24	0.00	24	0.268	0.75
7	10.50	0.00	53.96	210.00	156.04	30	0.83	1.00	1.00	1.00	0.90	22	0.00	22	0.233	0.61
8	12.00	0.00	68.67	240.00	171.33	30	0.80	1.00	1.00	1.00	0.90	22	0.00	22	0.233	0.61
9	13.50	0.00	83.39	270.00	186.61	30	0.77	1.00	1.00	1.00	0.90	21	0.00	21	0.219	0.56
10	15.00	0.00	98.10	300.00	201.90	30	0.74	1.00	1.00	1.00	0.90	20	0.00	20	0.206	0.53

Abbreviations

Depth: Depth from free surface where SPT was performed (m)
Weight: Soil unit weight from previous test point to current (kN/m³)
u_0: Water pressure at test point (kPa)
σ_v: Total overburden pressure at test point (kPa)
σ_v': Effective overburden pressure based on in situ GWT (kPa)
N_{SPT}: Number of blows count in the field (blows/30 cm)
C_N: Overburden pressure factor
C_E: Energy ratio factor
C_B: Borehole diameter factor
C_R: Rod length factor
C_S: Sampling method factor
$N_{1(60)}$: Number of blows corrected for 60% energy
$\Delta N_{1(60),cs}$ Fines correction
$N_{1(60),cs}$: Number of blows corrected for 60% energy and fines
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction

Post Liquefaction Residual Strength
Project No: 2-37905.01
Client: Bayview / Kah NZ
Project Name: Chateau Tongariro

Office: Taupō
Date: 21/11/2022
Author: 9(2)(a)

Calculation Method: Olson and Stark (2002)

Software: Excel

Inputs

Unit Weight: A unit weight of 20 kn/m3 is assumed for the calculated

Groundwater: Measurements of two piezometers on site resulted in groundwater levels of 5.1 – 5.2m bgl in September 2022 and 5.05m bgl in November 2022.

We have adopted a groundwater level of 5m bgl.

Standard Penetration Tests: SPTs from two boreholes are the basis of the assessment. We have assessed the SPT data and selected an SPT N of 30 as representative of the matrix.

Depth (m)	BH1 – SPT	BH1 – Log Notes	BH2 – SPT	BH2 – Log Notes
1.5	50	73mm movement	21	No notes, full SPT
3.0	50	Bouncing, no movement	50	No notes, full SPT
4.5	50	35mm movement	50	205mm movement
6.0	50	Bouncing, no movement	35	83mm movement
7.5	50	50mm movement	25	45mm movement
9.0	50	65mm movement	25	75mm movement
10.5	35	No notes, full SPT	25	Bouncing (no movement)
12.0	30	10mm movement	30	Bouncing (no movement)
13.5	50	Bouncing, no movement	30	Bouncing (no movement)
15.0	50	Bouncing, no movement	N/A	N/A

$N1_{60}$ values are copied from CLIQ outputs.

Formulae

Olson and Stark (2002) proposed the following relation for Su(liq) which we have adopted

$$\frac{S_{u(liq)}}{\sigma'_v} = 0.03 + 0.0075 * N1_{60} \pm 0.03$$

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Calculation

The calculation of Su(Liq) is presented below. Note that Su(liq) has been calculated for all depths however only the 13.5m and 15.0m depths are assessed to liquefy. Calculated values are reported to the nearest whole number. The calculation is extended to 31.5m depth to allow an estimation of increase in Su(Liq) vs depth for later use in the slope stability model.

Depth (m)	σ'v	N160	Su(Liq) (kPa)
1.5	30	33	8
3	60	28	14
4.5	90	27	21
6	110	25	24
7.5	125	24	26
9	141	24	30
10.5	156	22	30
12	171	22	33
13.5	187	21	35
15	202	20	36
16.5	217	19	37
18	232	18	38
19.5	248	18	41
21	263	17	41
22.5	278	17	44
24	294	16	44
25.5	309	16	46
27	324	15	46
28.5	339	15	48
30	355	14	48
31.5	370	14	50

Increase in Su(Liq) with depth 13.5m – 31.5m: (50 kPa – 35 kPa) / (31.5m – 13.5m) = 0.83 kPa/m ; take as approximately 1 kPa/m

Post Liquefaction Slope Stability
Project No: 2-37905.01
Client: Bayview / Kah NZ
Project Name: Chateau Tongariro

Office: Taupō
Date: 21/11/2022
Author: 9(2)(a)

Calculation Method: Morgenstern-Price LEM

Software: GeoStudio Slope/W 2021 R2

Purpose

The purpose of this calculation is to assess slope stability for static and seismic cases and also post-liquefaction using residual liquefied shear strengths.

Inputs

Unit Weight: A unit weight of 20 kn/m3 is assumed

Strength parameters (non liquefied): Phi=35°, c'=0 kPa

Strength parameters (liquefied): Su(Liq) = 35 kPa + 1 kPa / m depth

Groundwater: Measurements of two piezometers on site resulted in groundwater levels of 5.1 – 5.2m bgl in September 2022 and 5.05m bgl in November 2022.

We have adopted a groundwater level of 5m bgl.

Building Load: The Chateau is modelled as a UDL of 50 kPa with a 60m width on a flat bench

Site geometry: A simplified section is assumed based on the topographical map, with a 60m wide flat bench for the Chateau, a 6% slope to the north, and a 17.5% slope to the south.

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Ground model: The ground profile is assumed to consist of a 5m non-liquefied layer underlain by a liquefied layer. In the absence of data to constrain the model base and to be moderately conservative the liquefied layer is assumed to be of arbitrary depth

Material Models:

Non-liquefied layer (Boulders in a sand and gravel matrix): The non-liquefied layer is modelled a Mohr-Coulomb material

Liquefied layer: The liquefied layer is modified with an undrained model that is a function of depth, with the Su(Liq) increasing linearly with depth as suggested in the post-liquefaction residual strength calculation

Slip surface geometry:

The slip surface search is ran using an entry and exit procedure with the entry points along the flat bench where the Chateau is and the exit points downslope. Note that the geotechnical C4 guidance only requires considering slope failure that begin on site so overslips above the site are not assessed. Two cases were ran, one with the entry-and-exit nodes constrained to within 100m of the Chateau, and one with the entry-and-exit nodes extending 1km downslope.

Results

Note that for the type of material model adopted it is expected that unrealistically large failure surfaces will develop extending to either the base of the model or the edge of the exit nodes for the slip surfaces. The extension of the base and the end of the exit nodes in this case is done given the lack of information to constrain the base of the model / base of the liquefied layer.

The results are:

Static:

Case 1 (Entry and exit nodes constrained to within 100m of the Chateau): FOS = 5.4

Case 2: (Entry and exit nodes extending 1km from the Chateau): FOS = 5.5

Post-Liquefaction:

Case 1 (Entry and exit nodes constrained to within 100m of the Chateau): 2.5

Case 2: (Entry and exit nodes extending 1km from the Chateau): 1.1

Seismic:

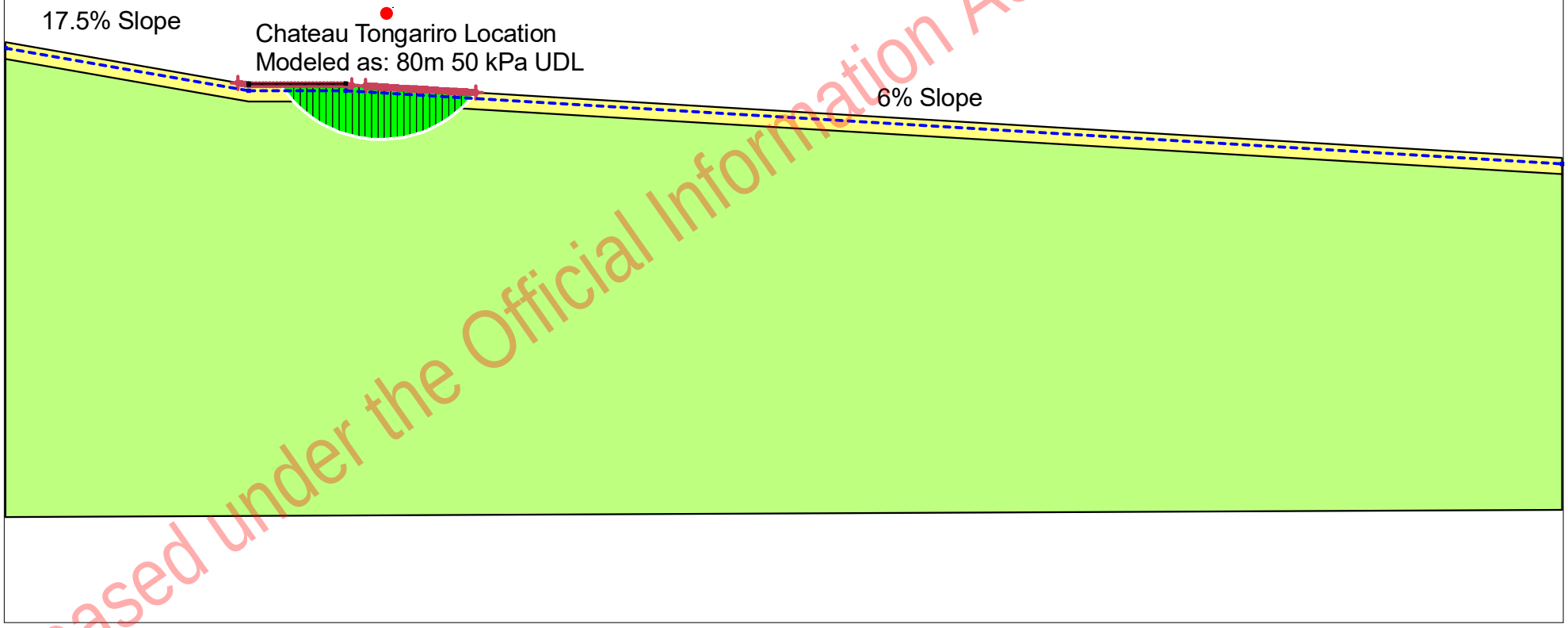
Case 1 (Entry and exit nodes constrained to within 100m of the Chateau): 2.3

Case 2: (Entry and exit nodes extending 1km from the Chateau): 1.7

Conclusions

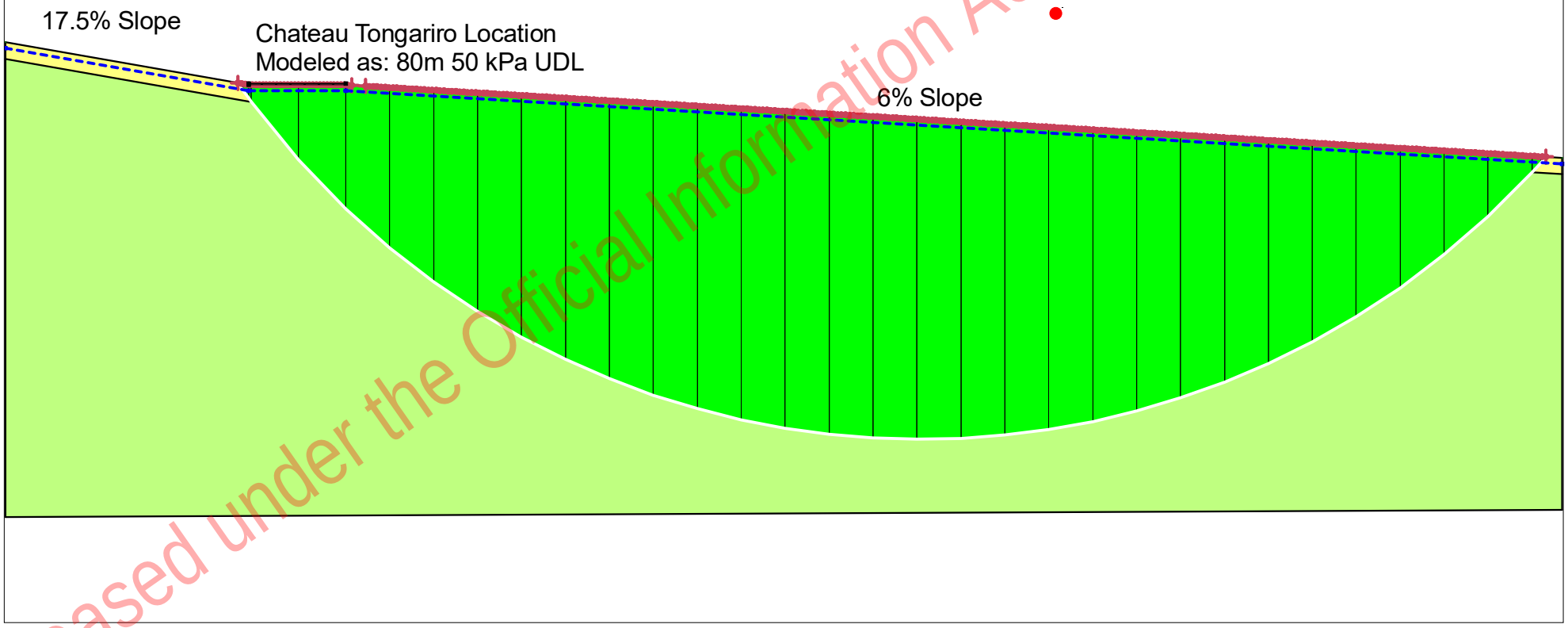
Given that FoS > 1 even with liquefied shear strengths and an extremely conservative assumption about the depth of liquefied material we conclude that post-liquefaction slope failure is unlikely.

Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Boulders in a Sand and Gravel Matrix	Mohr-Coulomb	20				0	35
	Boulders in a Sand and Gravel Matrix (Liquefied)	S=f(depth)	20	35	1	180		



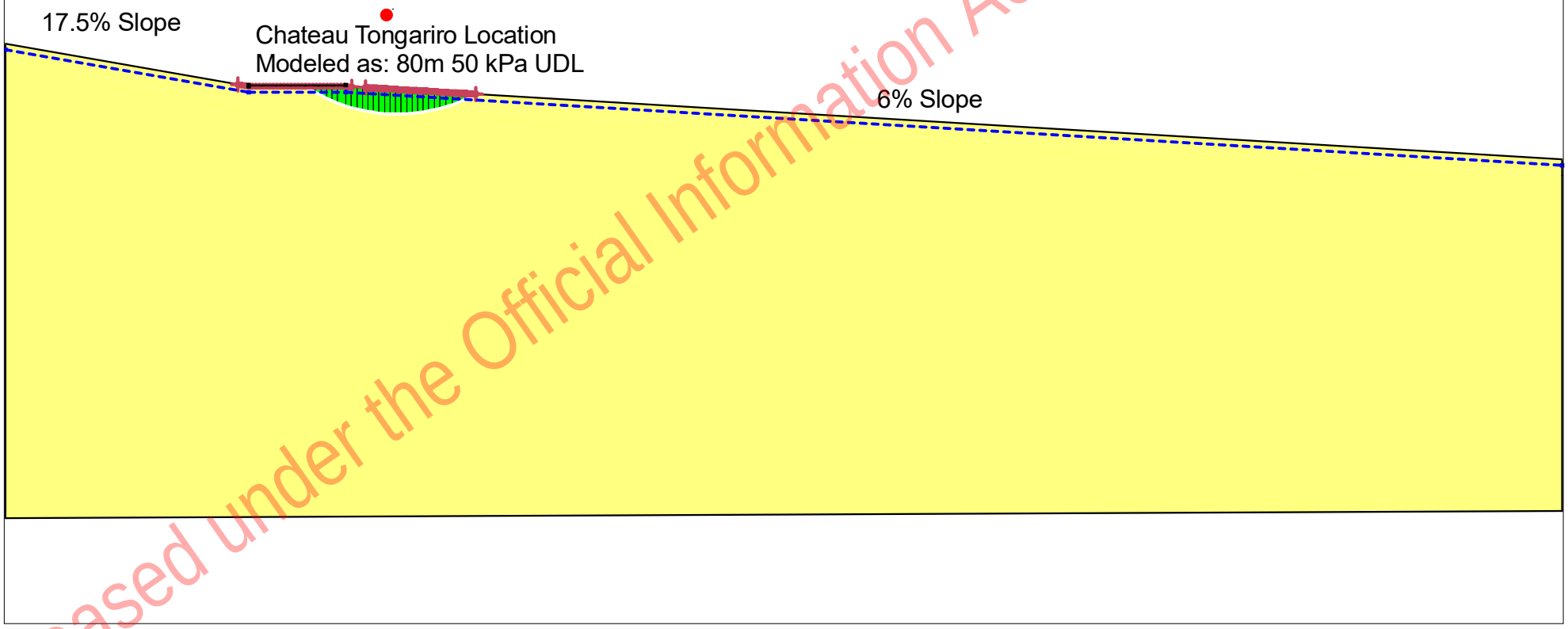
Project:	Chateau Tongariro			Method	Morgenstern-Price
Analysis:	Post Liquefaction Stability - Constrained Entry and Exit			Scale	1:4,927
				PGA	g
Modelled By:	9(2)(a)	Checked By: Name	Analysis Type: SLOPE/W	Date: 22/11/2022	FOS 2.489

Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Boulders in a Sand and Gravel Matrix	Mohr-Coulomb	20				0	35
	Boulders in a Sand and Gravel Matrix (Liquefied)	S=f(depth)	20	35	1	180		



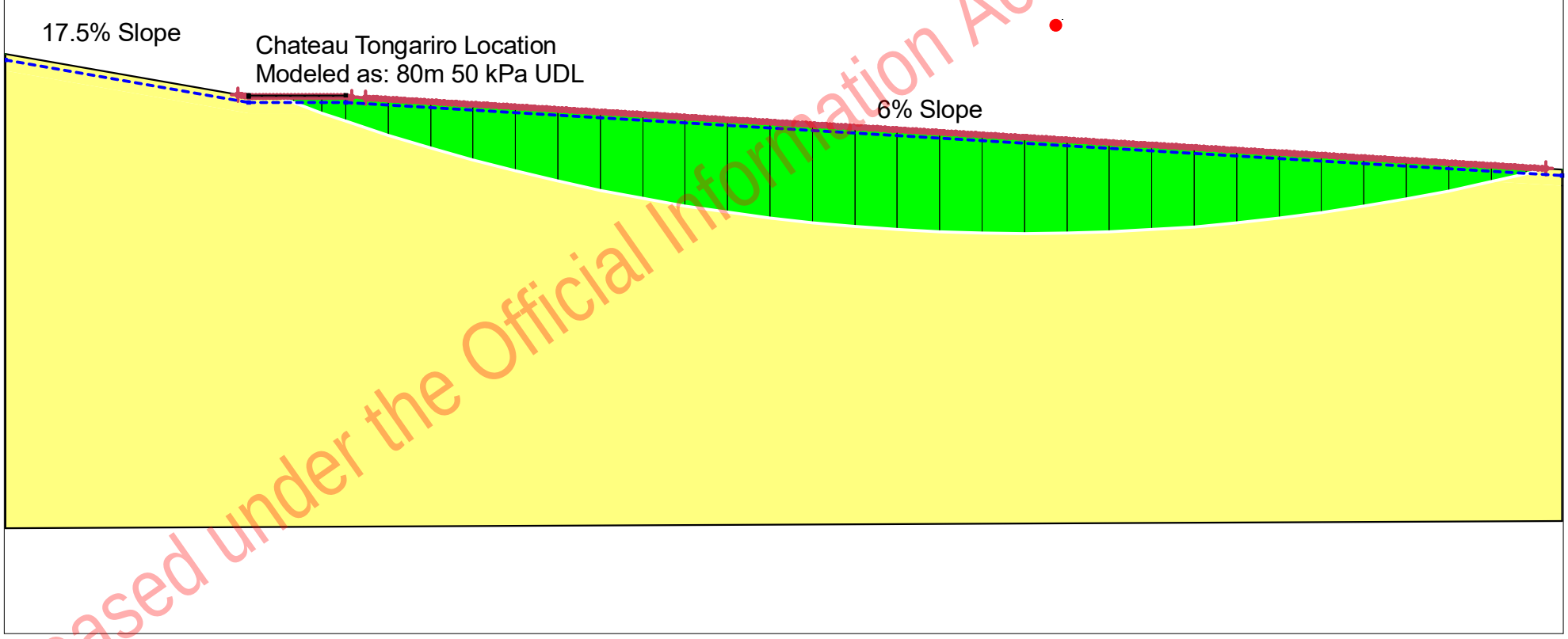
Project:	Chateau Tongariro			Method	Morgenstern-Price
Analysis:	Post Liquefaction Stability - Longer Entry and Exit			Scale	1:4,927
				PGA	g
Modelled By:	9(2)(a)	Checked By: Name	Analysis Type: SLOPE/W	Date: 22/11/2022	FOS 1.057

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Boulders in a Sand and Gravel Matrix	Mohr-Coulomb	20	0	35



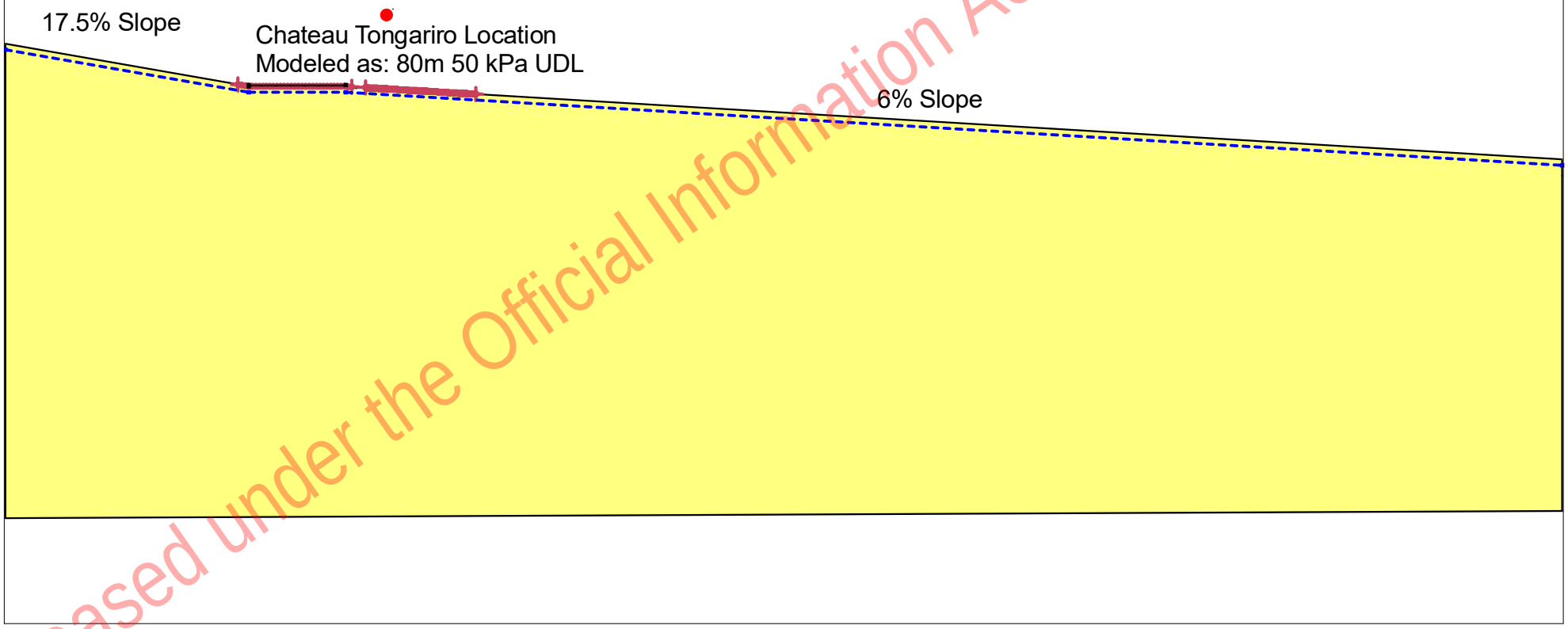
Project:	Chateau Tongariro			Method	Morgenstern-Price
Analysis:	Pre-Liquefaction Seismic - Constrained Entry and Exit			Scale	1:4,927
				PGA	0.18g
Modelled By:	9(2)(a)	Checked By: Name	Analysis Type: SLOPE/W	FOS	2.254
			Date: 22/11/2022		

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Boulders in a Sand and Gravel Matrix	Mohr-Coulomb	20	0	35



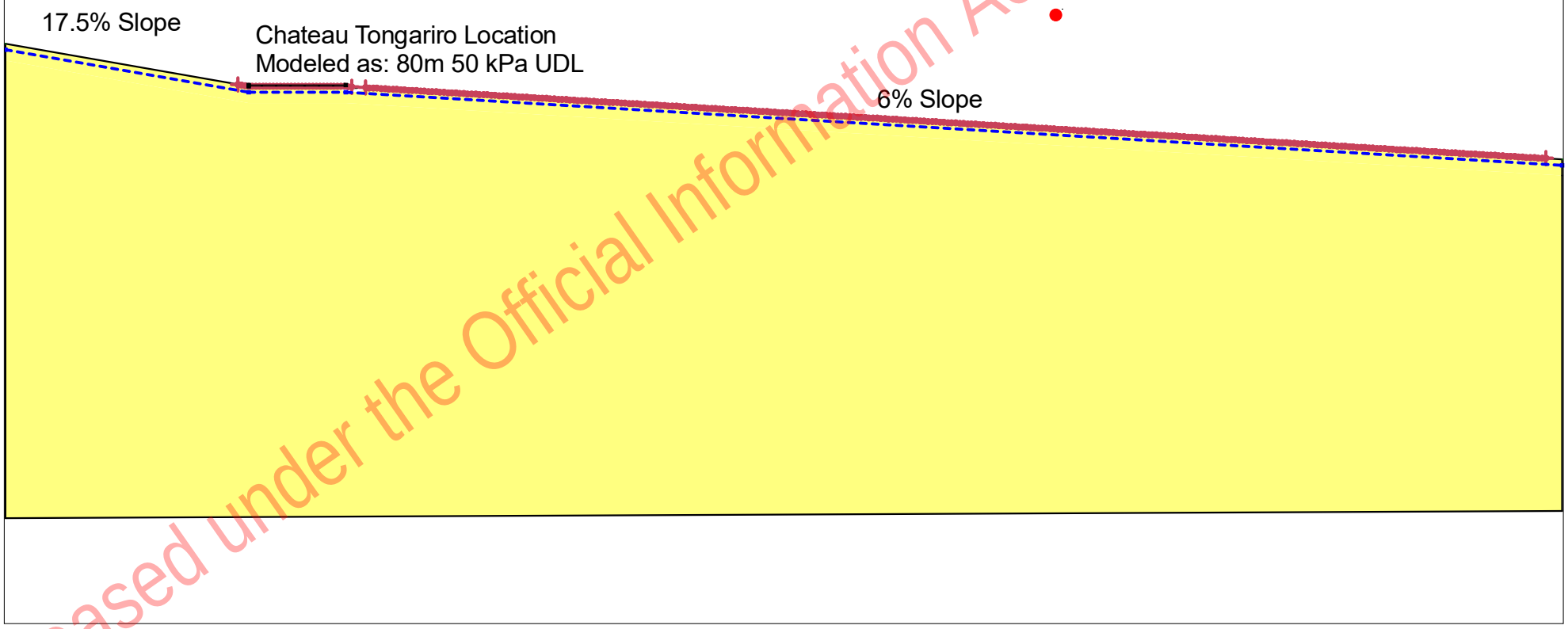
Project:	Chateau Tongariro			Method	Morgenstern-Price
Analysis:	Pre-Liquefaction Seismic - Longer Entry and Exit			Scale	1:4,927
				PGA	0.18g
Modelled By:	9(2)(a)	Checked By: Name	Analysis Type: SLOPE/W	FOS	1.657
			Date: 22/11/2022		

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Boulders in a Sand and Gravel Matrix	Mohr-Coulomb	20	0	35



Project:	Chateau Tongariro			Method	Morgenstern-Price
Analysis:	Static - Constrained Entry and Exit			Scale	1:4,927
				PGA	0g
Modelled By:	9(2)(a)	Checked By: Name	Analysis Type: SLOPE/W	Date: 22/11/2022	FOS 5.543

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Boulders in a Sand and Gravel Matrix	Mohr-Coulomb	20	0	35



Project:	Chateau Tongariro			Method	Morgenstern-Price
Analysis:	Static - Longer Entry and Exit			Scale	1:4,927
				PGA	0g
Modelled By:	9(2)(a)	Checked By: Name	Analysis Type: SLOPE/W	Date: 22/11/2022	FOS 5.543

SESOC | SOILS

Version: 3.3.1.2152, Published: 9 Jun 2022
Soils design to N.Z. Building Code document B1/VM4

DESIGNER: 9(2)(a)

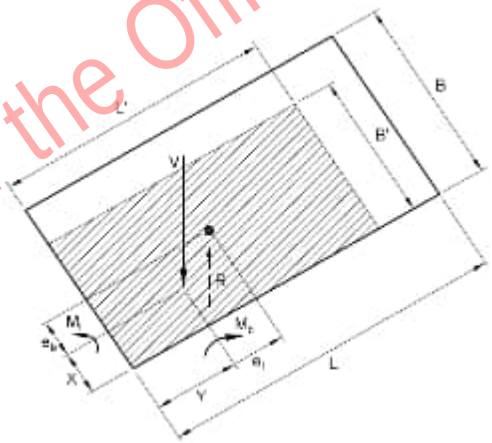
PROJECT: Chateau Tongariro

DESCRIPTION: Bearing Capacity Calculations

ANALYSIS ID:

DATE: 22/11/2022 TIME: 1:34 pm

PAD FOOTING - RECTANGULAR



THE LOADING CONDITION CHOSEN IS LONG TERM

Size in metres:-	Pad Length: 0.91 Pad Width: 0.99
Load position:-	X = 0.50 Y = 0.45 m.
Vertical Load :-	V = 1.0 kN.
Horizontal Force:	H = 0.0 kN. which is angled to long side at 0.0 deg.
Moment || to Length :-	Mb = 0.0 kNm.
Moment || to Width :-	Ml = 0.0 kNm.
Eccentricity || to Length :-	el = 0.00 m.
Eccentricity || to Width :-	eb = 0.00 m.
Equivalent Area :-	A = 0.897 m^2.
Equivalent Length :-	L' = 0.906 m.
Equivalent Width :-	B' = 0.990 m.
Design Bearing Pressure :-	qd = 1.11 kPa.

Sliding Resistance :- S = 0.3 kN.
Angle of friction with soil:- δ = 17.50 deg.
Sliding Resistance Capacity Ratio :- CRs = 0.000
The footing is cast in place and poured on the ground, friction factor = 1.000
The Applied [redacted]ed loads of the Loading Standard.

SITE DETAILS

Site Type 0.
Minimum Depth of footing:- Df = 0.46 m.
Depth from surface to water table:- Dw = 5.00 m.

SOIL AND CAPACITY ANALYSIS

THE SOIL TYPE IS 0
Cohesion:- c = 0.00 kPa.
Eff.Angle of Shearing Resistance:- φ' = 35.000 deg.
Actual Adjacent pressure:- q = 9.144 kPa.
Soil Density:- γ = 20.00 kN/m^3
Under water Soil Density:- γ' = 10.19 kN/m^3
Bearing Strength Reduction Factor Φbc = 0.45
Passive Strength Reduction Factor Φpp = 0.45
Sliding Strength Reduction Factor Φsl = 0.80
Earthquake Str. Reduction Factor Φeq = 0.60
Effective Adjacent pressure:- q' = 9.1 kPa.
Pore water pressure at depth Df:- uf = 0.00

ANALYSIS RESULTS:

Bearing Strength Factor:- Nc = 46.12
Bearing Strength Factor:- Nq = 33.30
Bearing Strength Factor:- Nγ = 45.23

Explanation of Soil Capacity Modification Factors:

λ Values are dimensionless factors relating to Phi and the following:
c relates to: Cohesion.
d relates to: Footing depth.
g relates to: Site slope.
i relates to: Applied Load inclination i.e. H/V.
q relates to: Surcharge effects.

s relates to: The shape of the footing i.e. L/B

λcs:-	= 1.789
λcd:-	= 1.123
λci:-	= 1.000
λcg:-	= 1.000
λqs:-	= 1.765
λqd:-	= 1.118
λqi:-	= 1.000
λqg:-	= 1.000
λgs:-	= 0.563
λgd:-	= 1.000
λgi:-	= 1.000
λgg:-	= 1.000

ULTIMATE BEARING CAPACITY Long Term = 852.7 kPa.
AVAILABLE BEARING CAP: = 383.7 kPa.
AVAILABLE LOAD CAPACITY: = 344.2 kN.

BEARING CAPACITY RATIO: = 0.00
SLIDING CAPACITY RATIO: = 0.00

68	4'0"	4'0"	2'0"	48	48	24	1.2192	1.2192	0.6096	1109
69	3'9"	3'9"	1'9"	45	45	21	1.143	1.143	0.5334	986
70 & 71	3'9"	3'9"	1'9"	45	45	21	1.143	1.143	0.5334	986
72	3'9"	3'9"	1'6"'	45	45	18	1.143	1.143	0.4572	880
73	4'3"	4'3"	1'9"	51	51	21	1.2954	1.2954	0.5334	1019
74 & 75	3'6"	3'6"	1'9"	42	42	21	1.0668	1.0668	0.5334	970
76	3'0"	3'0"	1'6"	36	36	18	0.9144	0.9144	0.4572	832
78	4'0"	4'0"	1'9"	48	48	21	1.2192	1.2192	0.5334	1002
78A	2'0"	2'0"	1'3"	24	24	15	0.6096	0.6096	0.381	665
79	3'0"	3'0"	1'5"	36	36	17	0.9144	0.9144	0.4318	796
80	4'0"	4'0"	1'9"	48	48	21	1.2192	1.2192	0.5334	1002
81	4'0"	4'0"	1'9"	48	48	21	1.2192	1.2192	0.5334	1002
82	3'0"	3'0"	1'5"	36	36	17	0.9144	0.9144	0.4318	796
83 & 84	2'6"	2'6"	12"	30	30	12	0.762	0.762	0.3048	587
85, 87, 90, 92	2'6"	2'6"	12"	30	30	12	0.762	0.762	0.3048	587
86, 88, 89, 91	2'6"	2'6"	12"	30	30	12	0.762	0.762	0.3048	587

Dynamic Spring Parameterization
Project No: 2-37905.01
Client: Bayview / Kah NZ
Project Name: Chateau Tongariro

Office: Taupō
Date: 8/12/2022
Author: 9(2)(a)

Calculation Method: Published correlations and tables (Evaluation of Soil & Rock Properties FHWA & ASCE 41-17

Software: Excel

Purpose

The purpose of this calculation package is to assess G_o values using various methods to obtain a range of values for use by the structural engineer.

Inputs

SPT Values: A range of SPT values is inferred based on the site investigation data ranging from 30 to 50+. While the maximum reported value is 50+ it should be understood that this value is for low or no movement. Therefore, a value of 100 has been taken as an upper-bound for use in formulae.

Method 1

Method 1 is to calculate G_o using the equation below (Equation 55 in Evaluation of Soil & Rock Properties by FHWA)

$$G_o = 15{,}560 (N_{60})^{0.68}$$

Lower Bound: N = 30 -> 157200 kPa/m

Upper Bound: N = 100 -> 356459 kPa/m

Method 2

Method 2 is to calculate G_o using the equation below.

$$G_0 \cong 435 (N_1)_{60}^{1/3} \sqrt{p_a \sigma'_{mp}}$$

σ'_{mp} = mean effective stress $(\sigma'_1 + \sigma'_2 + \sigma'_3)$ averaged over the relevant region below the footing. The σ'_{mp} may be obtained as the larger value from Eqs. (8-8) and (8-9).

$$\sigma'_{mp} = \frac{1}{6}\left(0.52 - 0.04\frac{L_f}{B_f}\right)\frac{Q_{Gf}}{A_f} \qquad (8\text{-}8)$$

$$\sigma'_{mp} \geq \sigma'_{vo} = (\gamma_t)(D_f + B_f/2) - u \qquad (8\text{-}9)$$

Lf – Taken as a representative average based on foundation details of 1.2m

Bf - Taken as a representative average based on foundation details of 1.2m

Df – Taken as a representative average based on foundation details of 0.5m

Qgf – Taken as 150 kN based on an assumed applied load of 100 kPa over 1.2x1.2m

Lower Bound: N = 30 -> 63397 kPa/m

Upper Bound: N = 100 -> 94704 kPa/m

Method 3

Method 3 is to use the shear-wave velocity based relation shown below. Note that we do not have site-specific Vs measurements here but a published range is available of 180 m/s for 480 m/s for soils.

$$G_o = \rho V_s^2$$

Lower Bound (180 m/s): 64800 kPa/m

Upper Bound (480 m/s): 460800 kPa / m

Method 4: Published values

Typical values are presented in the table below

Soil Type	Small-strain shear modulus, G_o (kPa)
Soft clays	2,750 to 13,750
Firm clays	6,900 to 34,500
Silty sands	27,600 138,000
Dense sands and gravels	69,000 to 345,000

Conclusions

Based on the information presented we conclude a range of Go values of 65,000 to 400,000 kPa/m is reasonable.

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Appendix B
Laboratory Report

Memorandum

To	9(2)(a)
Copy	9(2)(a)
From	9(2)(a)
Office	WSP Research, Petone
Date	28 October 2022
File	2-37905.01 0SITE
Subject	Chateau Tongariro – Results of Materials Strength Testing

1. Scope

This memo presents the results of materials strength testing completed on samples of brick masonry and concrete collected from the Chateau Tongariro for the purpose of informing preparation of a Detailed Seismic Assessment. This document augments a previous memo, dated 18 October[1] that was primarily concerned with on-site intrusive investigations to determine reinforcement configuration within concrete frame elements. However, the earlier document also contains some additional information on sample recovery and non-invasive evaluation of the tensile strengths of larger diameter reinforcing bars and should be read in conjunction with these findings.

2. Methodology

The array of different samples received or extracted for testing is illustrated by Figure 1 and the particular methodology used to determine the material strength of each specimen type is described further below.

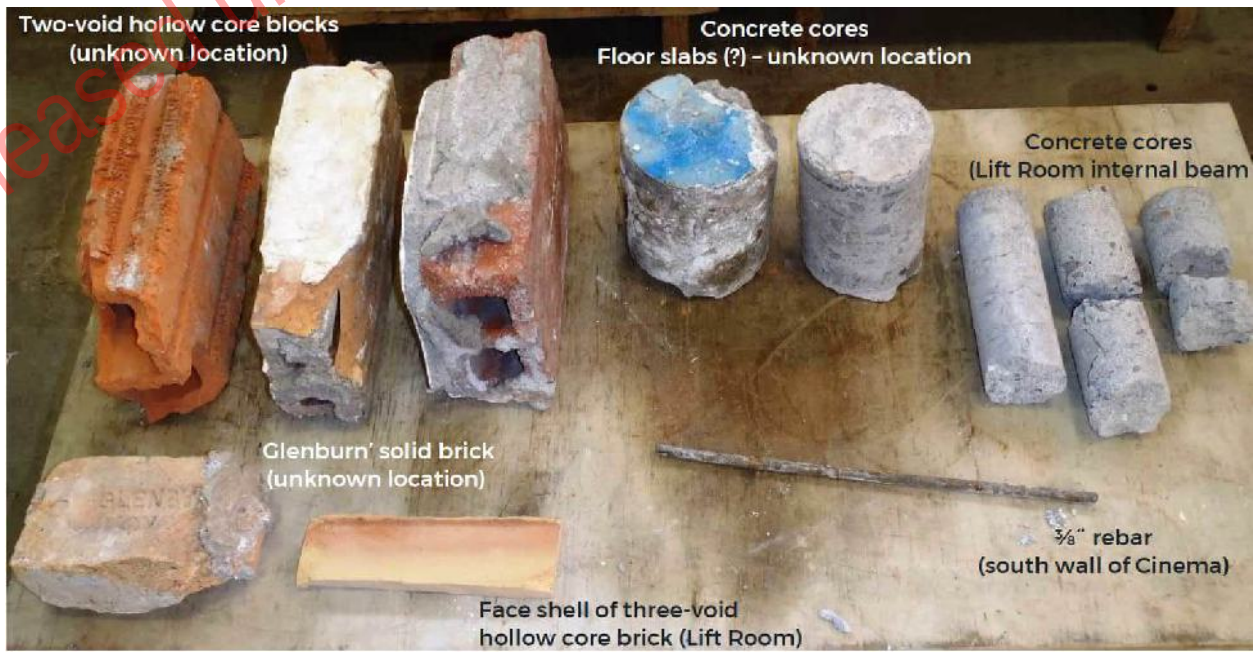

Figure 1. Photograph of samples in as-received condition.

2.1 Fired Clay Masonry

Where possible, the unconfined compressive strength of the two-void hollow core masonry was determined on full depth regular prismatic sections of the blocks, trimmed to eliminate any damage sustained and tested in the orientation in which they were laid in accordance with

[1] Lee, N.P. 'Chateau Tongariro Intrusive Investigation'. WSP memo dated 18 October 2022. Project no. 2-37905.01 0SITE

ASTM C67 – 14 'Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile'. This is the reference method for this property specified by Section C8 'Unreinforced Masonry Buildings' of the MBIE 'Engineering Assessment Guidelines' for seismic evaluation of existing structures. The testing was conducting using a 1,000 kN capacity 'Shimadzu UDH 100' universal testing with a traceable Class 1 (±1%) calibration certificate.

For the single available sample of solid brick, pre-existing damage to one of the bedding faces prevented entire intact halves of the brick from being tested. In this instance, three smaller sub-specimens were cut as an alternative, preserving the aspect ratio (height to lateral dimensions) of the normal sample dimensions. Similarly, cubic specimens from the extracted face shell of an in-situ three-void hollow core block were tested to determine the density and compressive strength of the constituent fired clay material and permit the strength of a whole unit to be calculated via finite element modelling if its geometry is known.

The ambient density of the block and brick specimens was measured in accordance with AS/NZS 4456.8:2008 'Masonry Units, Segmental Pavers & Flags – Determining Moisture Content, Dry & Ambient Density'. Where the original nominal work size was known, or could be inferred, the density was used to calculate the unit mass of the individual bricks.

The brick bedding mortar was tested by an ad-hoc procedure outlined in C8 whereby irregular dags (fragments) recovered primarily from within the cavity voids are loaded to failure after preparation by cutting into approximately cubic shapes. The failure loads obtained are normalized for equivalence with a standard 50 mm mortar cube using the geometry corrections provided in the Assessment Guidelines. Due to the intrinsic variability inherent in testing small mortar fragments, multiple replicate specimens were sub-sampled from individual pieces of the recovered material.

All of the specimens were capped with high strength gypsum cement to ensure two opposing plane and parallel surfaces suitable for application of a compressive load without appreciable eccentricity. Examples of the prepared test specimens are illustrated in Figure 2.

2.2 Concrete

The concrete cores were prepared and testing in accordance with Section 9 'Determination of strength in compression of drilled concrete cores' of NZS 3112: Part 2: 1986 'Methods of test for concrete – Tests relating to the determination of strength of concrete'. The concrete was tested in the 'dry condition', i.e. following a minimum of 7 days' drying in the ambient laboratory environment after completion of all preparation except capping. This particular conditioning state is considered to give results most representative of the in-situ strength of the concrete.

Where the cores supplied could not be trimmed and capped to yield specimens with the recommended aspect ratio of 2.0, interpolated corrections factors from ASTM C39/C39M - 12a 'Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens' & ASTM C42/C42M - 13 'Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete' were applied to permit comparison of the results with the conventional norms for expressing concrete strength.

2.1 Steel Reinforcement

Properties of the reinforcing bars were measured in accordance with AS 1391-2007 'Metallic materials - Tensile testing at ambient temperature', which is the reference method cited by AS/NZS 4671-2001 'Steel reinforcing material'. The testing was carried out with a computerised 250 kN capacity servo-mechanical 'Instron' universal testing machine with a Class 1 calibration. 'V-shaped' jaws appropriate to the bar diameter were attached to its hydraulic grips of the machine to hold the sample. An extensometer coupled to the machine's actuator allowed for automatic generation of stress – strain curves via the attached 'MTS series 793' controller and data-logger.

Concrete core

2-Void Hollow Core Block

Solid Bric

Face shell of 3-Void Hollow Core Block

Bedding Mortar

Figure 2. Examples of prepared test specimens prior to testing.

Results

The unconfined compressive strength measured for each of the three largely intact hollow core blocks collected is presented in Table 1. All three blocks were imprinted with the 'Mastertile' product name, with one unit manufactured at the Glenburn brickworks and the other two from the Silverstream kiln. Despite identical markings, the pair of Silverstream blocks differ in several significant aspects including: the pattern of ribs on their sides; the minimum thickness of the extruded walls; the extent of firing crack defects present; and the quantity of aggregate visible within the clay. The Glenburn block has a notably smaller cross-section than the Silverstream sourced products (ca. 150 x 82 mm vs 159 x 90 mm) and possesses the yellow colour characteristic of West Auckland clay deposits. Figure 3 illustrates these differences.

Table 1. Compressive strength of prepared sections of 2-void hollow clay masonry block.

Hollow Core Block	Dimensions (mm)						Area	Geometry Factors		Failure Load	Compressive Strength	
	length		width		height		(mm^2)	h/t	K_a#	(kN)	f'_b (MPa)	Normalised
Glenburn	118.6	118.6	81.3	81.6	150.1	150.1	9659	1.84	0.981	84.5	8.6	8.7
Silverstream 1	112.2	112.2	89.9	89.7	161.7	161.6	10074	1.80	0.967	40.7	3.9	4.0
Silverstream 2a	117.6	117.3	91.3	91.6	158.6	158.6	10743	1.73	0.945	94.5	8.3	8.8
Silverstream 2b	114.9	115.0	91.6	90.9	159.3	159.3	10489	1.75	0.949	94.1	8.5	9.0

Correction factor for normalisation of the aspect ratio of masonry test specimens in compliance with the norms of AS/NZS 4456 part 4.[2] Note that the uncorrected compressive strength (i.e. f'_b) is the reportable property requested by the C8 Guidelines.

Figure 3. Cross-section of the three hollow core block specimens, illustrating their slightly different geometry.

[2] AS/NZS 4456.4:2003. Masonry Units, Segmental Pavers & Flags – Methods of Test. Method 4: Determining Compressive Strength of Masonry Units.

The compressive strength of the specimens prepared from the sample of solid brick, which was stamped 'Glenburn Avondale' and included a single large frog on the upper surface only, is presented in Table 2.

Table 2. Compressive strength of cut cubes prepared from recovered partial 'Glenburn' solid brick.

Solid Brick	Dimensions (mm)						Area	Geometry Factors		Failure Load	Compressive Strength	
	length		width		height		(mm^2)	h/t	K_a	(kN)	f'_b (MPa)	Normalised
i	48.50	48.48	47.29	47.31	55.70	55.68	2294	1.18	0.759	85.2	28.2	37.1
ii	50.10	50.12	51.79	51.82	55.93	55.92	2596	1.12	0.739	48.5	13.8	18.7
iii	59.26	29.27	51.43	51.38	53.15	53.10	2275	1.20	0.767	55.5	18.7	24.4

Table 3 gives the results of testing the strength of the constituent fired clay sampled from the face shell of a three-void hollow core block of unknown provenance used to construct an internal partition wall in the basement Lift Room. Note that this value is not equivalent to the unconfined compressive strength of the whole unit.

Table 3. Compressive strength of cut cubes prepared from face shell of 3 cell hollow core brick from internal partition wall of Lift Room.

Face Shell	Dimensions (mm)			Area	Geometry Factors		Failure Load	Compressive Strength	
	length	width	height	(mm^2)	h/t	K_a	(kN)	f'_b (MPa)	Normalised
i	21.02	21.01	22.67	442	1.08	0.726	22.1509	36.4	50.2
ii	22.13	22.08	22.50	489	1.02	0.706	19.6977	28.5	40.3
iii	20.36	21.22	21.91	432	1.08	0.725	19.0471	32.0	44.1
iv	21.97	21.03	22.71	462	1.08	0.727	25.4491	40.0	55.1
v	21.70	21.78	21.79	473	1.00	0.701	22.7130	33.7	48.1

The ambient density and consequent estimated unit masses of the masonry bricks and blocks is presented in Table 4.

Table 4. Density & unit weight of collected brick specimens.

Specimen Type	Ambient Mass	Water-Saturated Mass		Ambient Density	S.S.D. Density	Unit Weight
	(g)	In air (g)	In water (g)	(kg/m^3)	(kg/m^3)	(kg)
Glenburn hollow core block	1535.3	1740.8	848.3	1720	1950	3.9 #
Silverstream 1 hollow core block	1451.7	1702.2	808.6	1620	1900	3.9 #
Silverstream 2 hollow core block	1800.1	1958.7	1004.3	1890	2050	4.7 #
Face shell of 3 cavity hollow core block	465.2	520.3	251.3	1730	1930	Insufficient information to determine
Glenburn solid brick	703.4	793.4	389.8	1740	1970	3.2 &

Calculated from the measured ambient density of regular prismatic test pieces and assuming the length of an intact block is 305 mm.

& Calculated assuming intact brick is 225 mm long, i.e. the historical NZ standard dimensions for solid brick. The measured width and height of the partial brick (108 mm & 75 mm respectively) support this contention.

The compressive strength of the bedding mortar, predominantly recovered from dags within the hollow core block voids is stated in Table 5, whilst Table 6 shows the strengths measured for the concrete floors. Qualitative assessment of the observed concrete quality suggests the low compressive strength measured is likely typical of the beams and columns throughout the basement area, but the floor slabs were apparently cast from a noticeably better mix.

Table 5. Compressive strength of recovered mortar fragments.

Mortar Specimens	Dimensions (mm)			Area	Geometry Corrections[&]			Failure Load	Compressive Strength (MPa)[@]	
	height	length	width	(mm^2)	h/t ratio[#]	α_{ht}	α_{tl}	(kN)	f'_{ji}(raw)	f'_j(normalised)
i	20.23	18.21	17.52	319.0	1.15	1.09	0.98	8.4517	26.5	28.4
ii	19.73	16.75	16.95	283.9	1.18	1.09	1.00	8.0393	28.3	30.8
iii	19.73	19.78	17.22	340.6	1.15	1.09	0.95	9.8087	28.8	29.6
iv	23.99	17.10	16.88	288.6	1.42	0.96	0.99	6.7744	23.5	22.3
v	23.52	15.50	16.43	254.7	1.52	0.85	0.98	5.7317	22.5	18.7
vi	19.64	16.52	15.49	255.9	1.27	1.07	0.97	6.4845	25.3	26.5
vii	19.45	15.13	15.91	240.7	1.29	1.07	0.98	7.1049	29.5	30.8
viii	21.59	15.74	15.30	240.8	1.41	0.97	0.99	6.0814	25.3	24.1

[&] Geometry correction factors are as defined by clause C8A.3.3 of 'Seismic Assessment of Existing Buildings – Section C8: Unreinforced Masonry Buildings'.

[#] Ratio of the prepared specimen height to its least lateral dimension.

[@] f'_{ji} is the measured compressive strength of the irregular mortar fragment specimens; f'_j is normalised to the compressive strength of a 50 mm mortar cube as per C8A.3.3.

Table 6. Compressive strength of concrete core specimens.

Core Specimen & Source		N – S Beam in Basement Lift Room			Floor Slab
		i	ii	iii	(unknown location)
Length after capping	(mm)	145.9	83.5	76.4	137.6
Diameter, mean	(mm)	68.96	68.81	68.85	116.73
Aspect ratio (L/D)		2.12	1.21	1.11	1.18
Requires correction for L/D?		no	yes	yes	yes
Reinforcement or defects present?		no	no	no	yes[&]
Fracture mode	(ASTM C39)	Normal cones	Normal cones	Normal cones	Columnar splitting
Load at Failure	(kN)	28.8	54.6	38.7	362.9
Correction factor		1.014	0.923	0.901	0.916
Compressive strength	(MPa)	8.0	13.5	9.5	31.0

[&] 19 mm diameter thick-walled steel conduit present in specimen (perpendicular to longitudinal axis of core) at approximately mid-height.

Tensile properties of the tested reinforcing steel are summarised by Table 7. Results for the ⅜″ diameter bar were derived from mechanical testing of an extracted specimen and the full IANZ accredited test report including the stress – strain curve is included as an addendum to this memo. Average figures for the larger bars derive from indirect Leeb Hardness testing and this process is described in greater detail in the preceding memo.

Table 7. Tensile properties determined for reinforcing bar specimen.

Bar Description	Yield Stress R_{el} (MPa)	Ultimate Tensile Strength R_m (MPa)	Elongation at Max. Force A_{gt} (%)	Source of Data
⅜″ round bar	309	401	18.4	Appended test report
1″ round bar	232	451	n/a	Leeb Hardness calculation (refer previous memo)
1½″ round bar	270	387	n/a	Leeb Hardness calculation (refer previous memo

I trust this information is helpful to completion of your DSA. Please contact me if anything in this memo requires further clarification.

Kind Regards

9(2)(a)

Concrete Technologist

REINFORCING BAR TENSILE PROPERTIES TEST REPORT

Report Number	**:**	**Chateau Tongariro – WSP Hamilton**
Sample Number	:	4-22/987
Client	:	WSP Hamilton
Project	:	Chateau Tongariro
Description	:	3/8 inch round reinforcing bar
Steel grade	:	Unknown
Steel producer	:	Unknown
Cast number	:	Unknown
Received On	:	14/10/2022
Tested On	:	25/10/2022
Straightened	:	No
Aged prior testing	:	No
Testing apparatus	:	5-003 Instron – 250 kN range
Extensometer	:	5-145 – 80 mm gauge length

TEST SAMPLES

WSP Research Sample #	Bar Type	Test date
4-22/987	3/8” round bar	25/10/2022

TEST RESULTS

WSP Sample Number	Yield Stress Rel [MPa]	UTS* [MPa]	Uniform Elongation A_{gt} **[%]	UTS*/Yield ratio	Comments
4-22/987	309	401	18.4	1.30	Typical ductile failure mode edge of gauge length.

* Ultimate Tensile Strength

** Uniform Elongation taken when force drops 0.18% below UTS

WSP New Zealand Limited
WSP Research
Quality Management Systems Certified to ISO 9001

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New Zealand

Telephone +64 4 587 0600
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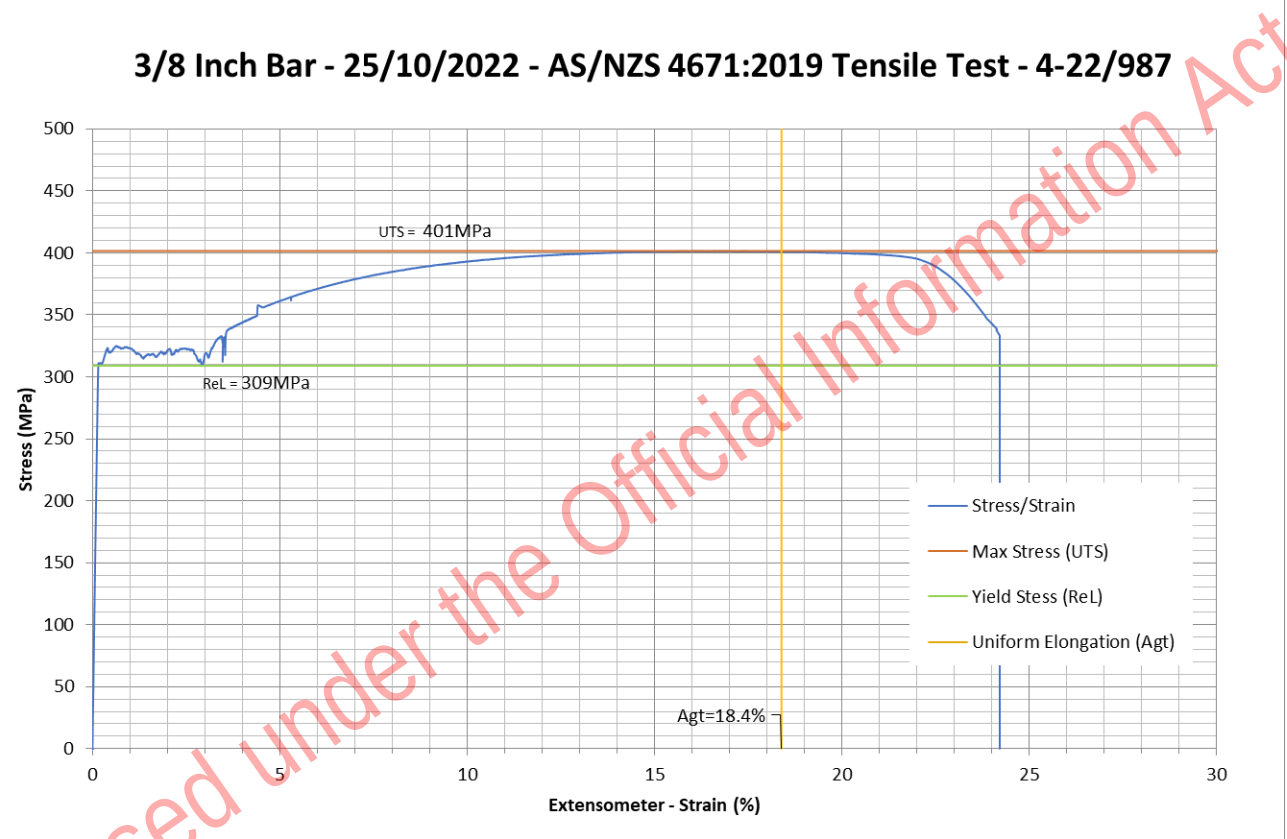
Website www.wsp.com

Test Methods

Tensile Testing of steel reinforcing materials to ISO 6892-1:2016 *Metallic materials – Tensile testing – Method of test at room temperature*, ISO 15630-1 2019 *Steel for the reinforcing and prestressing of concrete – Test Methods* and AS 1391:2020 *Metallic materials – Tensile testing at ambient temperature*, in accordance with AS/NZS 4671:2019 *Steel reinforcing materials.*



Comments : This report may only be reproduced in full.

Date: 27 October 2022

ACCREDITED IANZ TESTING LABORATORY

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

Tested and Reported by:
9(2)(a)
Graduate Engineer

9(2)(a)

Approved Signatory:
Reviewed by:
9(2)(a)
Mechatronics Engineer

9(2)(a)

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Memorandum

To	9(2)(a)
Copy	9(2)(a)
From	9(2)(a
Office	WSP Research, Petone
Date	18 October 2022
File	2-37905.01 0SITE
Subject	Chateau Tongariro Intrusive Investigation

Scope

This memo briefly summaries the findings of intrusive investigations undertaken at the Chateau Tongariro on the 13th & 14th of October 2022 in order to elucidate details of the building's construction and better inform preparation of a Detailed Seismic Assessment. Note that some results – notably measurements of beam stirrup spacings – which were verbally reported on site have been omitted.

Physical site works to support the testing were completed by 9(2)(a) from the specialist Concrete Materials Group at WSP Research (Petone), guided by Senior Structural Engineer 9(2)(a), from the WSP Hamilton office.

Methodology

The presence and layout of reinforcement in the nominated elements of interest was determined non-invasively using a GSSI '*StructureScan Mini HR*' hand-held ground penetrating radar (GPR) unit equipped with a 2,600 MHz antenna. The GPR antenna functions by emitting radio wave pulses into the underlying concrete substrate and measuring the two-way travel time for the returning echo. Reflections are created whenever the pulse encounter an interface with a material with contrasting electromagnetic properties – dielectric constant and conductivity – to the medium through which it is propagating, i.e. a change in structure. Depending on the relative dielectric strength of the two materials, the polarity of the radio wave can also be reversed in the returned reflection.

The GPR unit was augmented with use of a conventional electromagnetic cover-meter to identify miscellaneous disguised ferrous components, notably brick ties in masonry veneer walls.

In the absence of a reliable non-destructive technique for accurate determination of reinforcement diameter, exploratory breakout of representative bars was conducted to physically expose bars, permitting their size to be measured with a pair of callipers.

Where the reinforcement size permitted, a Proceq '*Equotip*' instrument was employed to measure the dynamic rebound hardness of the exposed bars in accordance with ASTM A956 *Standard Test Method for Leeb Hardness Testing of Steel Products* The resulting Leeb hardness numbers generated were then processed to give an estimate of the tensile strength of the steel using the following procedure:

- The Leeb hardness values, corrected for the orientation of the probe during testing, were converted to Brinell indentation hardness through an empirical correlation developed by the instrument supplier.
- The derived Brinell values are in-turn converted to an estimation of ultimate tensile strength (UTS) for the steel, making use of the well-recognised relationship between

hardness and tensile flow for materials such as carbon steel that possess a relatively low capacity for strain hardening. These relations are documented by Table A.1 of Annex A of ISO 18265: 2013 (E) *Metallic materials – Conversion of hardness values*. The ISO standard states that "*hardness testing is a form of materials testing that provides information on the mechanical properties of a material with limited destruction of the specimen and within a relatively short period of time*" and "*hardness values and tensile strength values are positively correlated and so it is possible to draw up empirical relationships for limited applications*."

- Finally, the UTS values were converted to yield strength, which is typically the desired output for a structural analysis, by assuming a yield/ultimate tensile strength ratio of 0.60. This figure derives from a database of measured historical values for steel of a similar era to the age of the bridge, collated by The Ministry for Works and Development.

Concrete core samples were recovered by water-cooled drilling and a number of opportunistic masonry brick and mortar specimens collected for subsequent laboratory testing, subject to confirmation of need.

Reinforcing Bar Size and Layout

Results from determinations of reinforcing bar size and configuration in selected locations on the concrete frame of the Chateau building in the basement level of the building are presented in Table 1. For clarity the particular elements examined are indicated on the plan reproduced as Figure 2. All bars uncovered were of plain round form; an example of a typical breakout to measure the reinforcement diameter is shown in Figure 1.

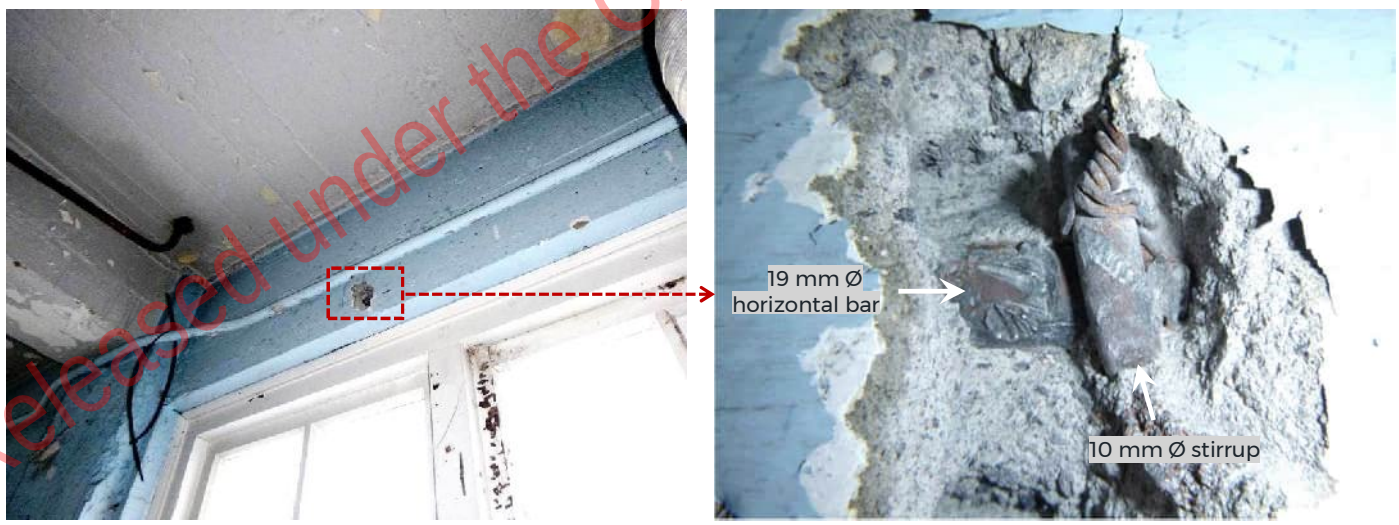


Figure 1. Exploratory breakout to expose reinforcement in the east – west beam on the exterior wall of the Boiler Room.

Table 1. Summary of reinforcement in investigated elements (refer to Figure 2 for locations).

Location	Element	Dimensions	Reinforcement Layout
pace directly south of Movie Theatre	Column A (In-wall)	560 mm wide face (column face stands 210 mm proud of wall)	37 mm (1 ½″) Ø vertical bars – 3 across face of column 10 mm (⅜″) Ø stirrups – 215 mm mean spacing
	Column B (Free-standing)	400 mm square	25 mm (1″) Ø vertical bars – 2 across face (1 per corner) 10 mm (⅜″) Ø stirrups at 240 mm mean centres. Note 1
	Column C (Free-standing)	400 mm square	2 vertical bars across face of column Stirrups at 280 mm mean spacings (Bar diameters not determined: Assumed to match Column B)
	Column D (Free-standing)	470 mm square	28 mm (1⅛″) Ø vertical bars – 3 across face 10 mm (⅜″) Ø stirrups – 280 mm mean spacings
	Column E (Free-standing)	400 mm square	2 vertical bars across face Stirrups at 265 mm mean spacings (Bar diameters not determined: Assumed to match Column B)
	Internal partition wall (F)	ca. 170 mm thick (measured at physical penetration for vent)	Single layer of 10 mm (⅜″) Ø horizontal & vertical bars at nominal 300 mm spacing.
	Retaining wall (G)	ca, 210 mm thick (measured at physical penetration for vent)	Single layer of 10 mm (⅜″) Ø horizontal & vertical bars at nominal 300 mm spacing.
Lift Room	Beam H	1.96 m long	32 mm (1¼″) horizontal bars – 2 only at base of beam 10 mm (⅜″) Ø stirrups at irregular 90 to 670 mm centres Note 2
Boiler Room	Beam I on external E – W wall		19 mm (¾″) Ø horizontal bars – 2 only at base of beam 10 mm (⅜″) Ø stirrups
	Beam J on external N – S wall	460 mm deep	32 mm (1¼″) Ø horizontal bars – 2 only at base of beam 10 mm (⅜″) Ø stirrups at 165 mm centres
	Beam K on internal partition wall (Deeper section at north end)		25 mm (1″) Ø horizontal bars – 2 only at base of beam 10 mm (⅜″) Ø stirrups
	Beam L on internal partition wall (typical section depth)	470 mm deep	28 mm (1⅛″) Ø horizontal bars – 2 only at base of beam 10 mm (⅜″) Ø stirrups
	Internal N – S roof beam (M)	310 mm x 310 mm	22 mm (⅞″) Ø horizontal bars – 2 only at base of beam 10 mm (⅜″) Ø stirrups

Notes to table:
[1] No stirrups are present in the in the first 830 mm of the column above its plinth;
[2] Spacing of stirrups along the beam from south to north were: 90 mm, 250 mm, 160 mm, 130 mm, 80 mm, 90 mm and 190 mm, followed by a 670 mm unconfined length into the column.

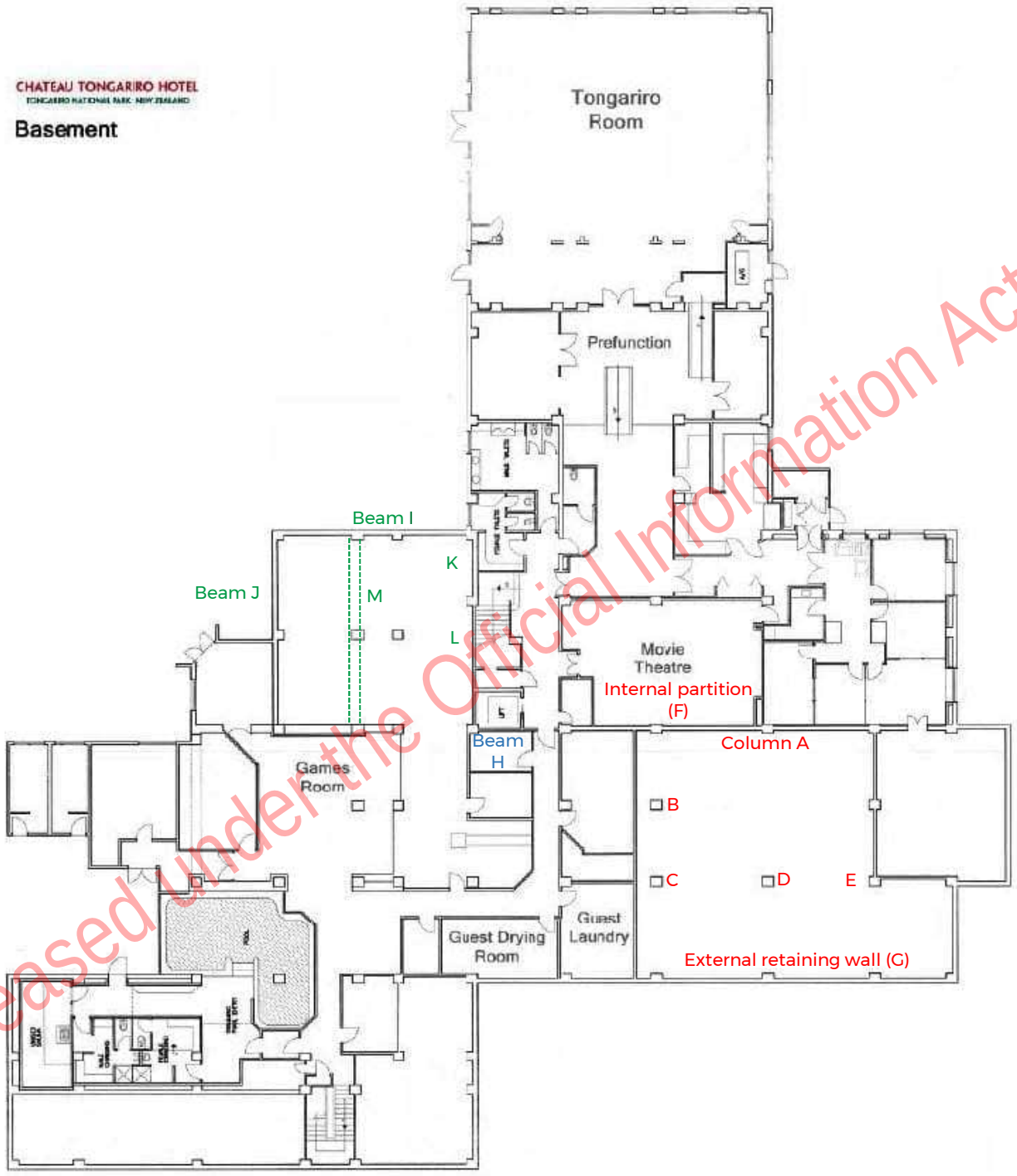


Figure 2. Key to investigative locations for determination of reinforcement size and layout in basement.

Tensile Properties of Reinforcement

The results of in-situ Leeb Hardness testing completed on pairs of 25 mm and 37 mm diameter bars exposed in basement columns (Figure 3) are presented in Table 2. While each bar diameter possesses an apparently distinct steel strength grade, the (conservative) yield strength estimated for each size is noticeably greater than the 210 MPa nominal historic value commonly assumed for structures constructed pre-1932.

A sample of 10 mm diameter reinforcement was also recovered from an internal partition wall within the basement. Bars of diameter < 19 mm are too small to be reliably tested by Leeb Hardness techniques, but the tensile properties of this specimen can be determined directly in the laboratory if desired.

Table 2. Tensile properties derived from Leeb Hardness testing completed on column vertical bars.

Reinforcing Bar Tested[1]		Leeb Hardness[1]		Brinell Hardness	Ultimate Tensile Strength	Yield Strength[2]
		Individual Readings	Mean Value	[kgf/mm²]	[MPa]	[MPa]
In-Wall Column A 37 mm (1½″) ø bars	Bar 1	390, 389, 390, 389, 392	390	133	450	270
	Bar 2	392, 388, 393, 389, 393	391	134	452	271
Freestanding Column B 25 mm (1″) ø bars	Bar 1	362, 369, 362, 362, 363	364	116	390	234
	Bar 2	358, 363, 358, 362, 361	360	114	385	231

Notes to table:

[1] Quoted Leeb hardness values have been corrected for the probe's orientation during testing.

[2] A yield to ultimate tensile strength ratio of 0.60 has been assumed, based on the original 1929 construction date of the Chateau and historical records of typical reinforcing steel properties compiled by the Ministry of Works.

Column A Column B

Figure 3. Examples of prepared Lee Hardness test sites on the exposed surfaces of vertical column bars. Note the generally poor quality of the concrete, with consolidation defects evident on Column A and a poorly proportioned mix with a large paste fraction used to cast Column B.

Brick Veneer Ties

Investigation of the brick veneer wall claddings from the Portico roof indicate that galvanised wire brick ties are present in every 4th masonry course at centres of approximately 700 mm (Figure 4 & Figure 5).

The veneer wythe is 95 mm wide and the cavity width ca. 100 mm

Figure 4. Ties generally appear to be present in the masonry veneer walls in every 4th course, laid in a staggered pattern at centres of approximately 700 mm. The yellow circles depict locations where cover-meter response suggested the presence of a tie; the nature of selected examples was confirmed via a borescope inspection of the wall cavity; representative images captured are reproduced as Figure 5.

Figure 5. Borescope imagery captured within the cavity of the masonry veneer wall cladding above the portico. The veneer ties appear to be constructed from simple loops of No. 8 SWG galvanised wire. The ties are physically intact, although some level of active red rust present on the surface indicates the protective galvanic coating has been largely consumed and corrosion pitting may have reduced the effective section of the wire in the worst cases.

Miscellaneous Sampling

Attempts were made to recover concrete core samples for compressive strength testing from the element nominated in the testing brief. (Beam H within the Lift Room, as depicted in Figure 6). This operation was only modestly successful due to the very low strength of the concrete and consolidation defects present in the beam, with only one core recovered fully intact. In general the concrete in the beams and columns in the basement appears to be of poor quality and is qualitatively estimated to likely fall in the compressive strength range of 5 – 15 MPa, although the ground floor soffit concrete seems noticeably better.

Figure 6. Coring sites on Beam H in the Lift Room, depicted after repair by dry-packing.

A sample of the face shell of the fired clay brick masonry was also extracted for testing from the Lift Room wall, in lieu of removal of a full unit. It was noted that the bricks used to construct this internal partition are three-cell hollow blocks, whereas the typical GPR response from the masonry infill panels scanned on external walls suggests a two-cavity block, consistent with the examples of '*Silverstream Mastertile*' brick collected from site.

Figure 7. Brick face shell sample removed from the Lift Room. Note the three-cavity hollow block design.

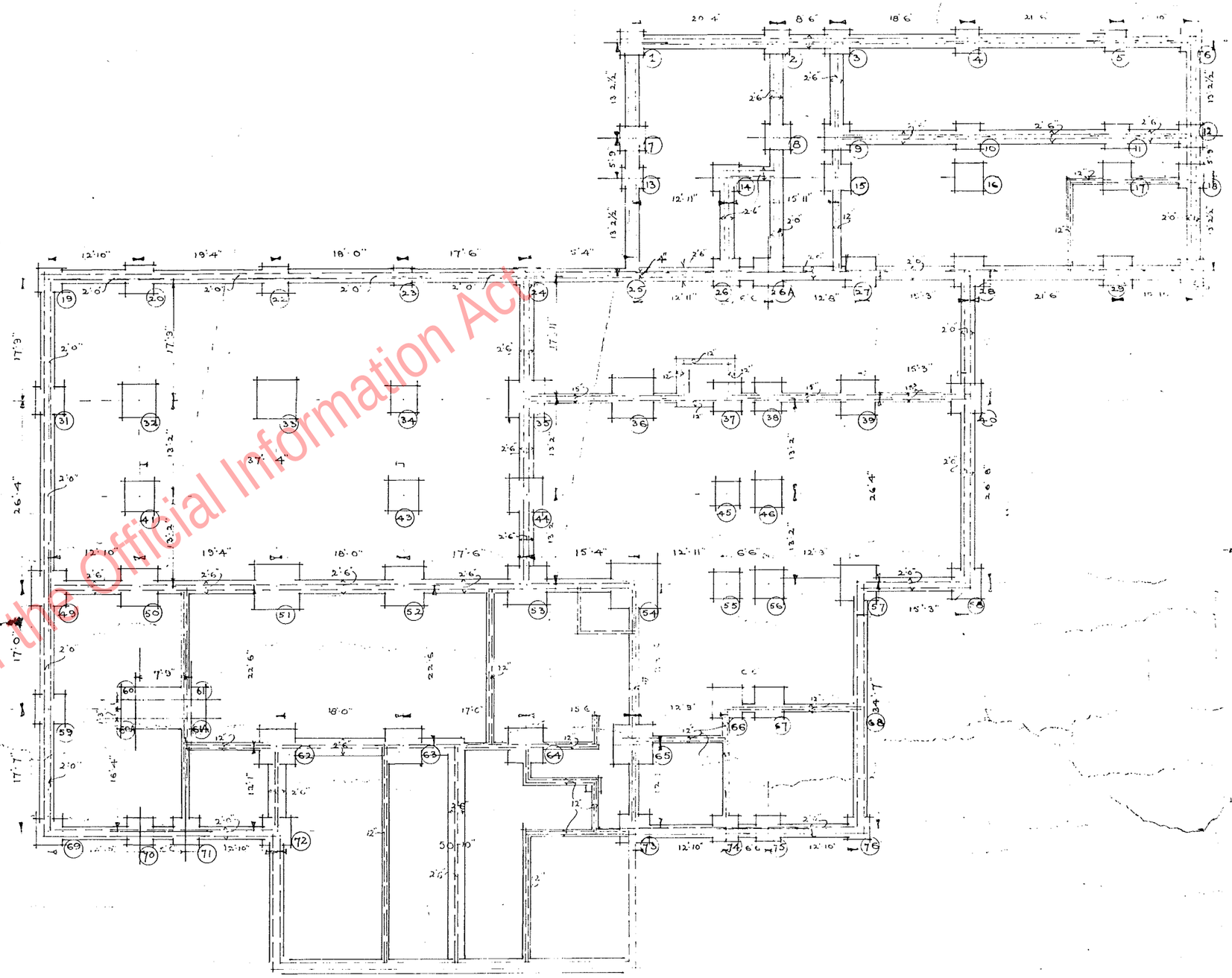
I trust this information is helpful to completion of your DSA. Please contact me if anything in this memo requires further clarification.

Kind Regards

9(2)(a)

Concrete Technologist

Appendix C
Building Information



FOUNDATION PLAN

SCALE 1 INCH TO EIGHT FEET
DRAWN & TRACED BY
DATE

· CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND ·

HERBERT HALL
ARCHITECT
WELLINGTON

CONCRETE WALL TO GROUND LEVEL.

UNEXCAVATED

ICE AIR

UNEXCAVATED.

6" CONCRETE

6" CONCRETE

REFRIG ERATOR

CAFETERIA.

VENT.

COUNTER

N.O.P.

TWO FIELD PLAINS AT WATER LEVEL.

TO CREEK

CONCRETE WALL TO GROUND LEVEL.

6" WALL.

DUMB WAITER

DRYING ROOM.

CALORIFIER.

BOILER RM.

8" TILE WALL.

CONCRETE WALL TO GROUND LEVEL.

UNEXCAVATED.

TUBE & NETTING

LIFT. MACHINERY.

GARAGE.

6" CONCRETE WALL.

ELEVATOR

GYMNASIUM.

CINEMA.

HALL

HALL

CORRIDOR.

BOOT.

CHANGE ROOM

BARBER'S SHOP

BASIN.

BAGGAGE ROOM.

RACK

RACK

LAVATORY.

WC

6" WALL.

RETAINING WALLS AND ROADWAY NOT IN CONTRACT.

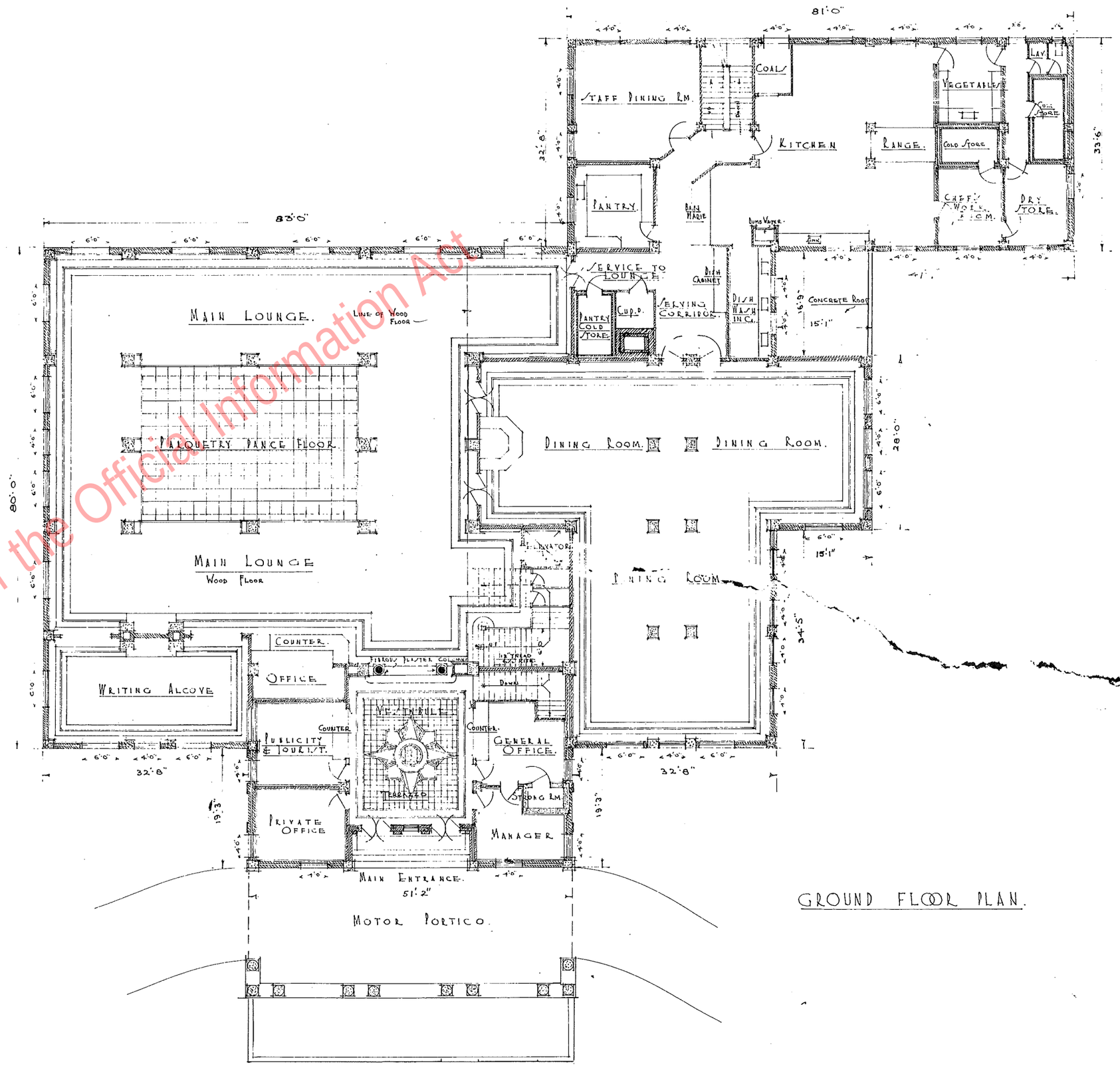
SUN PIAZZA

BASEMENT FLOOR PLAN.

SCALE-1INCH TO EIGHT FEET.
DRAWN & TRACED BY. EDS & HAL
PASSED BY.

CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND.

HERBERT HALL. ARCHITECT.

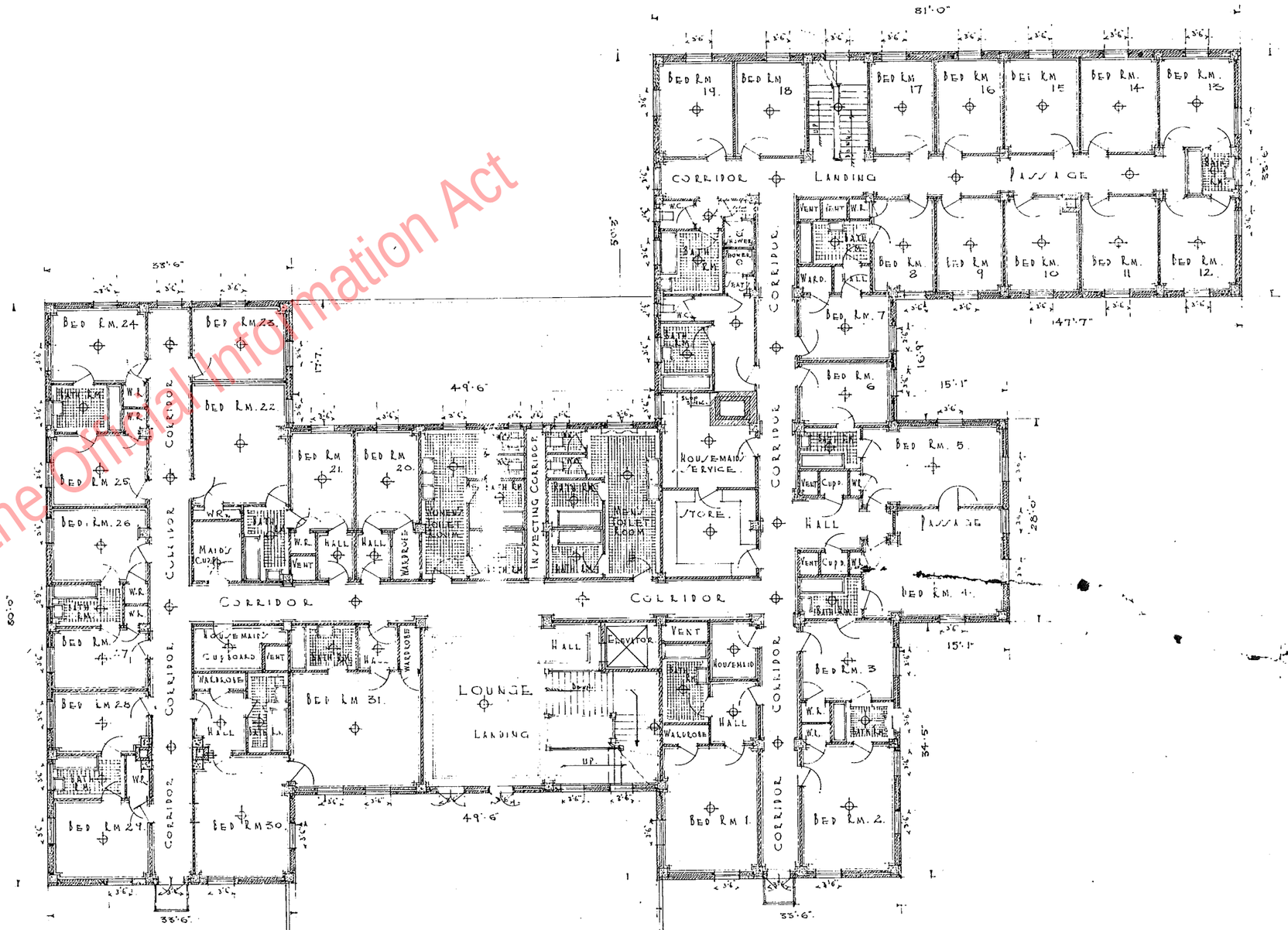


GROUND FLOOR PLAN.

SCALE · 1INCH TO EIGHT FEET.
DRAWN & TRACED BY · F.R.S. & W.L.
PASSED BY.

· CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND.

HERBERT HALL
ARCHITECT
TIMARU . N.Z.

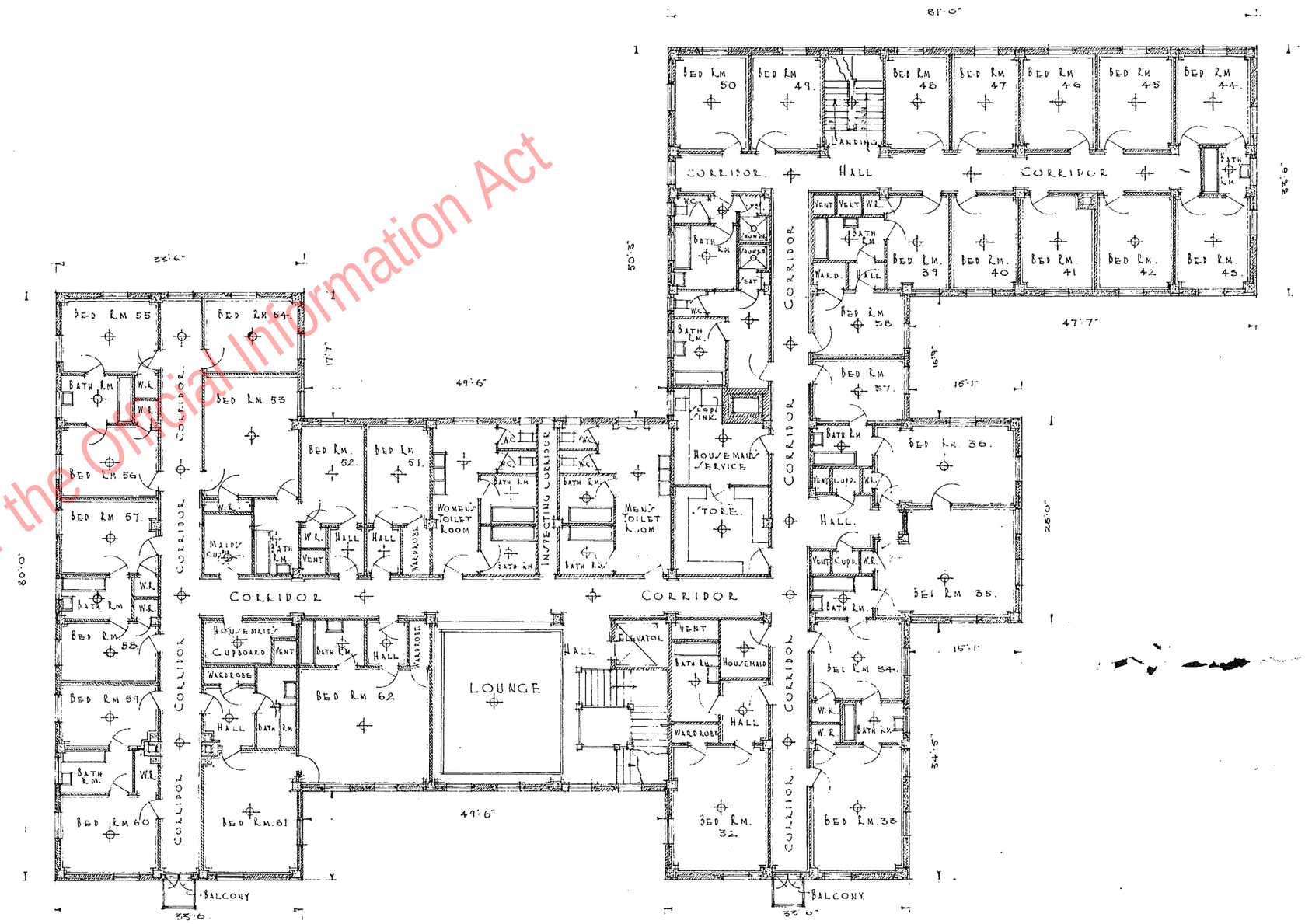


FIRST FLOOR PLAN.

SCALE: 1 INCH TO EIGHT FEET
DRAWN & TRACED BY - F.D.S. & H.H.
PASSED BY -

CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND.

HERBERT HALL.
ARCHITECT.
TIMARU. N.Z.

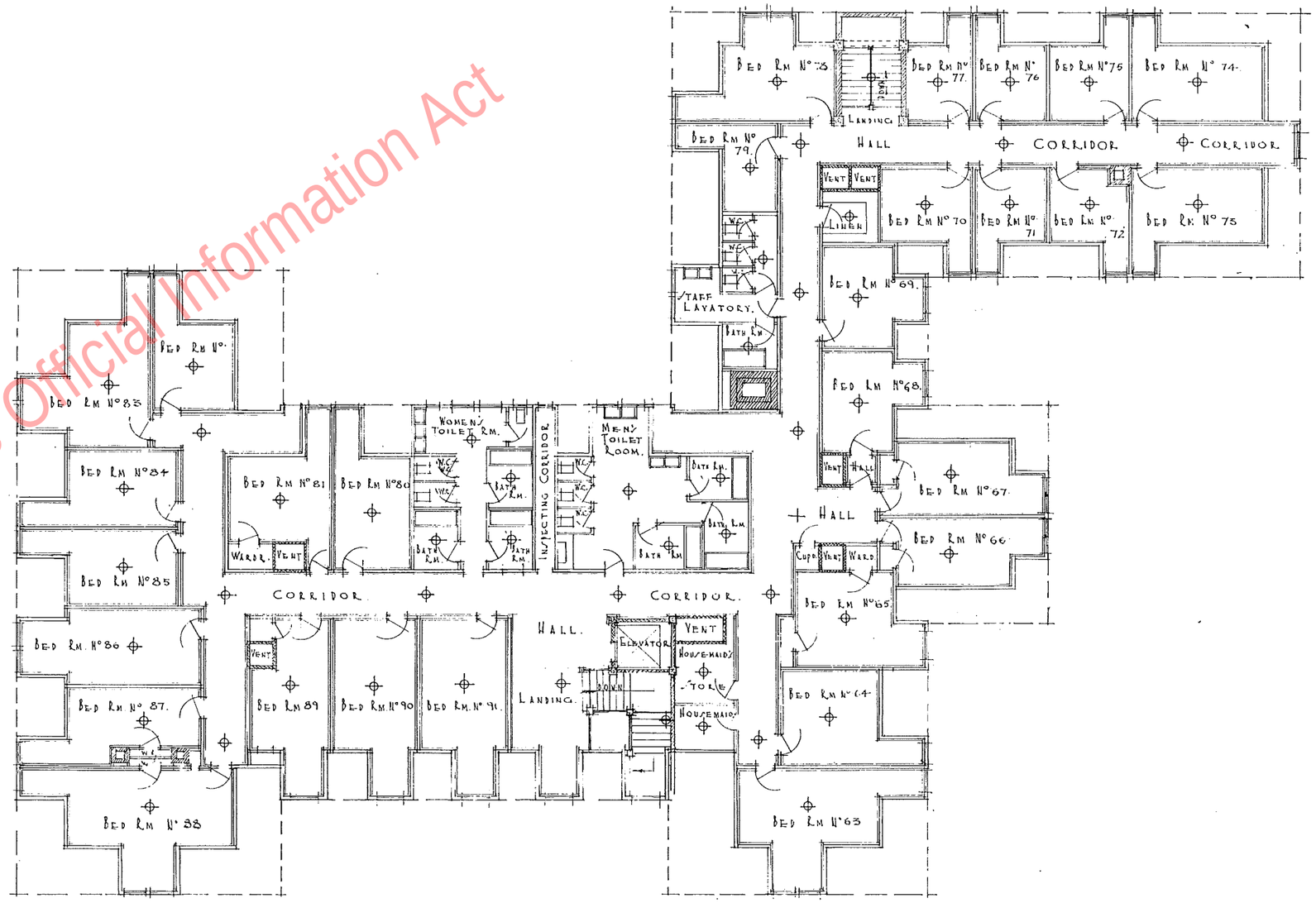


SECOND FLOOR PLAN.

SCALE: 1 INCH TO EIGHT FEET.
DRAWN & TRACED BY - EDS HALL
PASSED BY -

· CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND.

HERBERT HALL.
ARCHITECT.
TIMARU. N.Z.

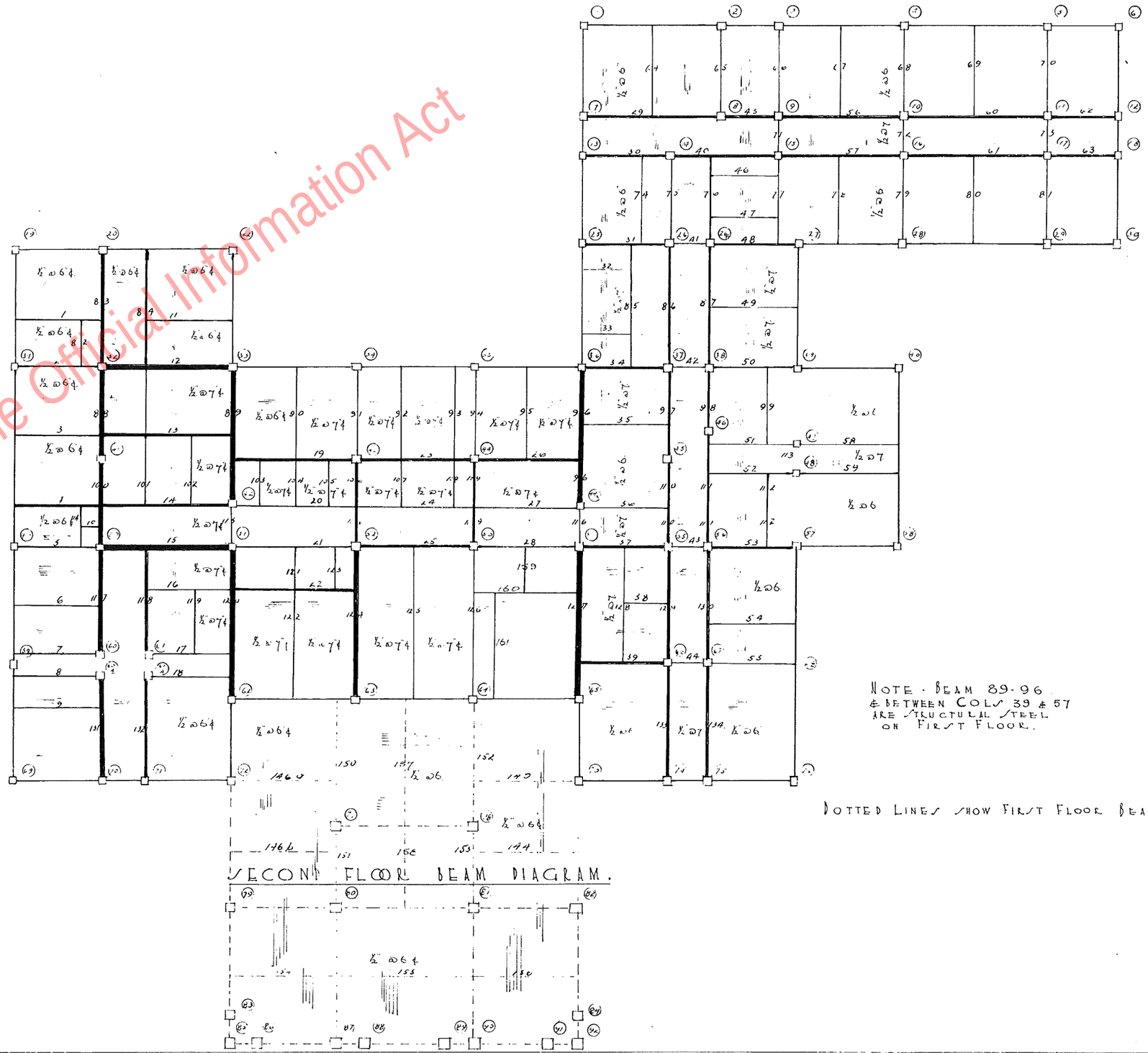


THIRD FLOOR PLAN.

SCALE: 1 INCH TO EIGHT FEET.
DRAWN & TRACED BY: F.D.S. & W.H.
ISSUED BY:—

CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND.

HERBERT HALL.
ARCHITECT.
TIMARU. N.Z.



NOTE · BEAM 89-96 & BETWEEN COLS 39 & 57 ARE STRUCTURAL STEEL ON FIRST FLOOR.

DOTTED LINES SHOW FIRST FLOOR BEAMS

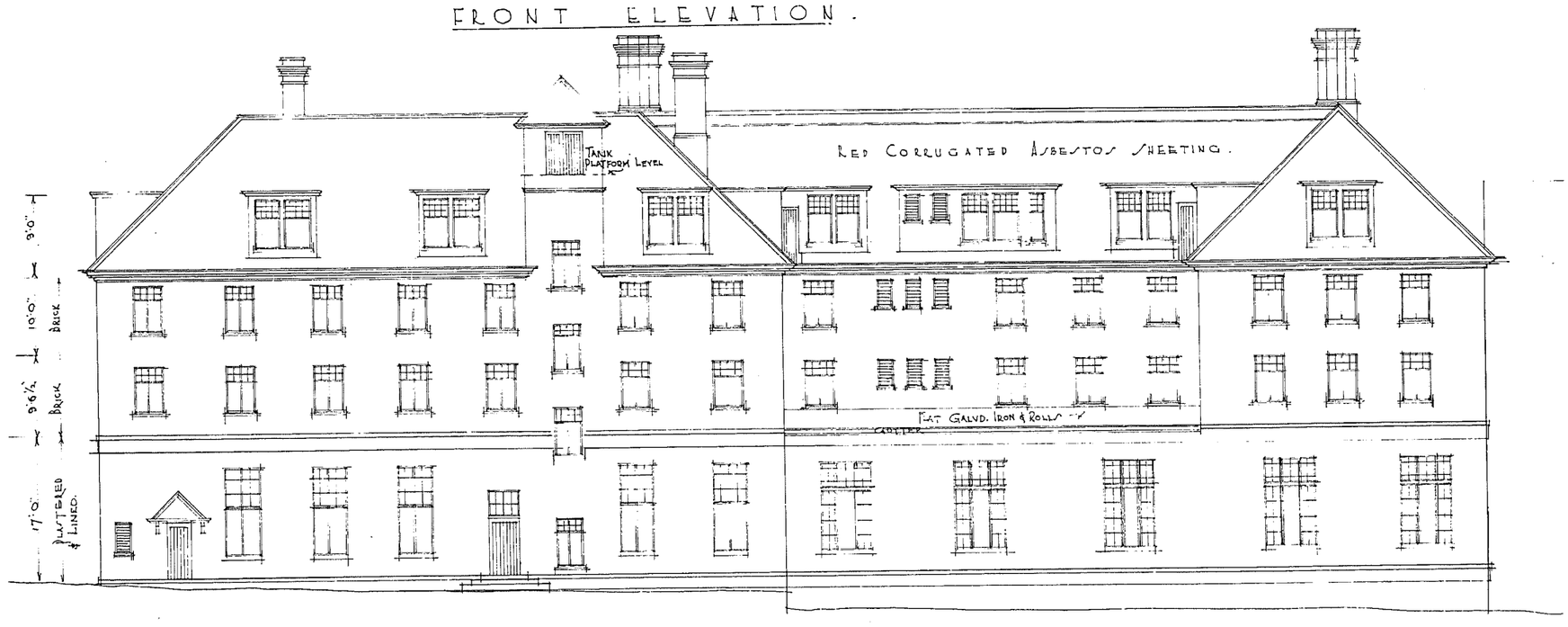
SECOND FLOOR BEAM DIAGRAM.

SCALE 1 INCH TO EIGHT FEET
DRAWN & TRACED BY
PASSED BY

CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND.

HERBERT HALL
ARCHITECT
TIMARU. N.Z.

FRONT ELEVATION.



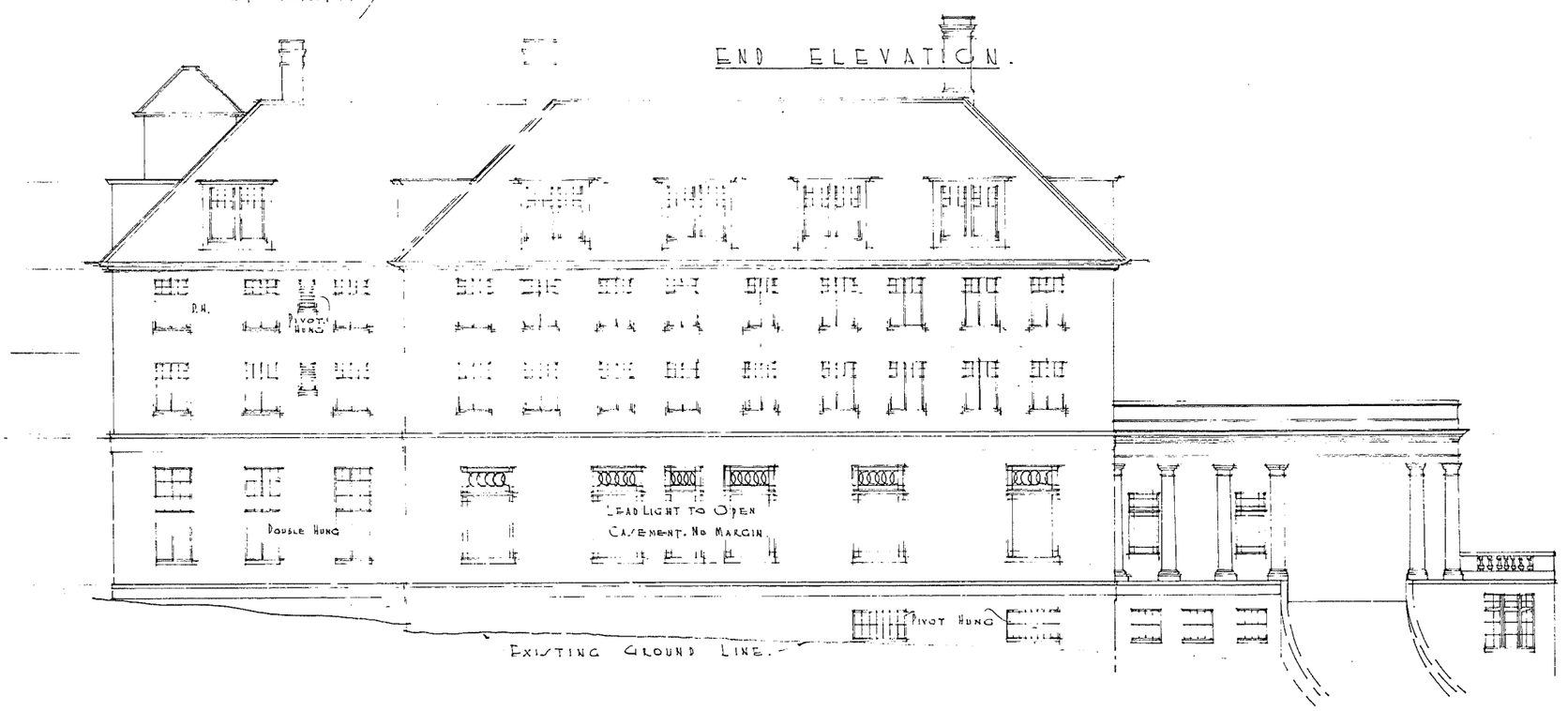
REAR ELEVATION.

SCALE - ⅛" = 1'0"
DRAWN & TRACED: N. STEWART
PASSED BY:

CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND.

HERBERT HALL
ARCHITECT
TIMARU. N.Z.

END ELEVATION.



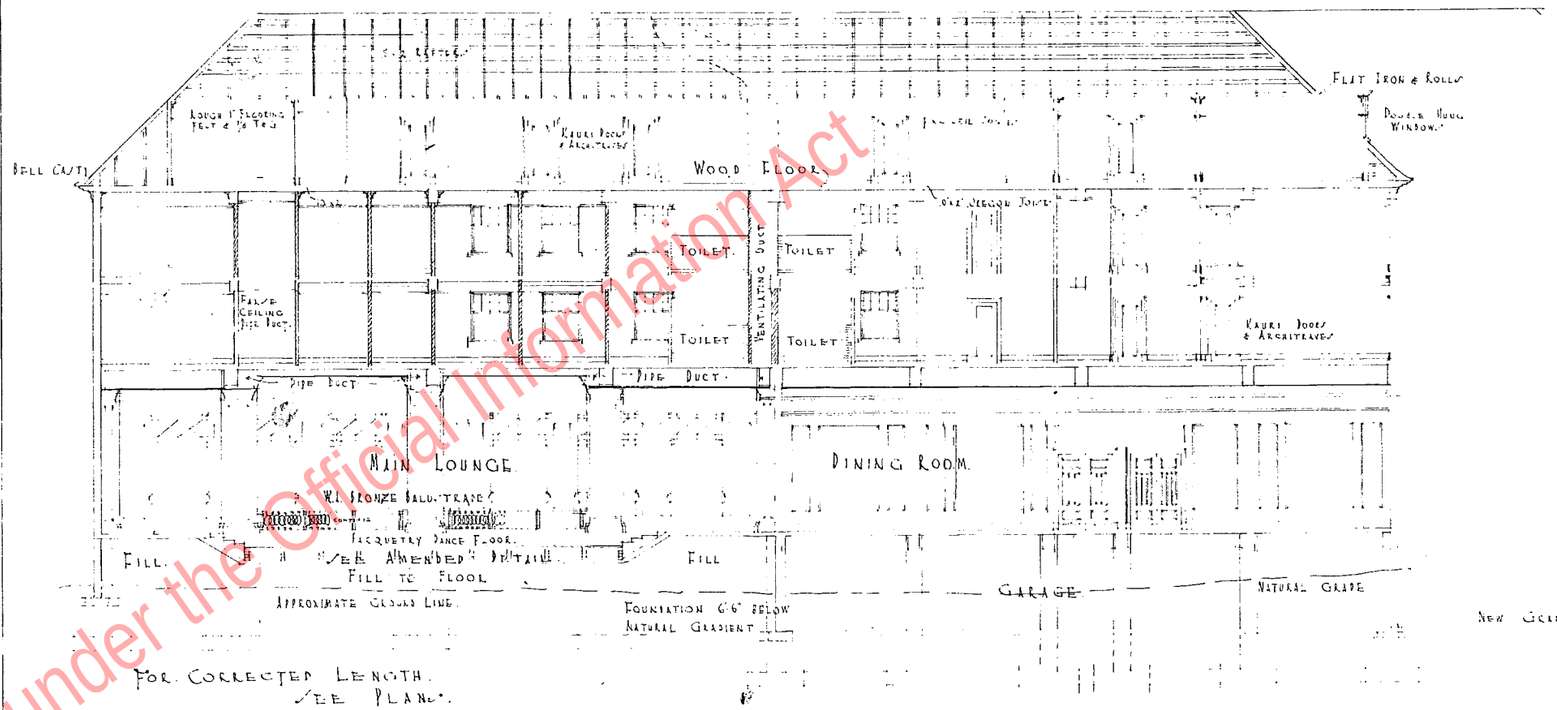
END ELEVATION.

SCALE - ⅛" = 1'0"
DRAWN & TRACED F.D. Stewart
PASSED BY.

CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND.

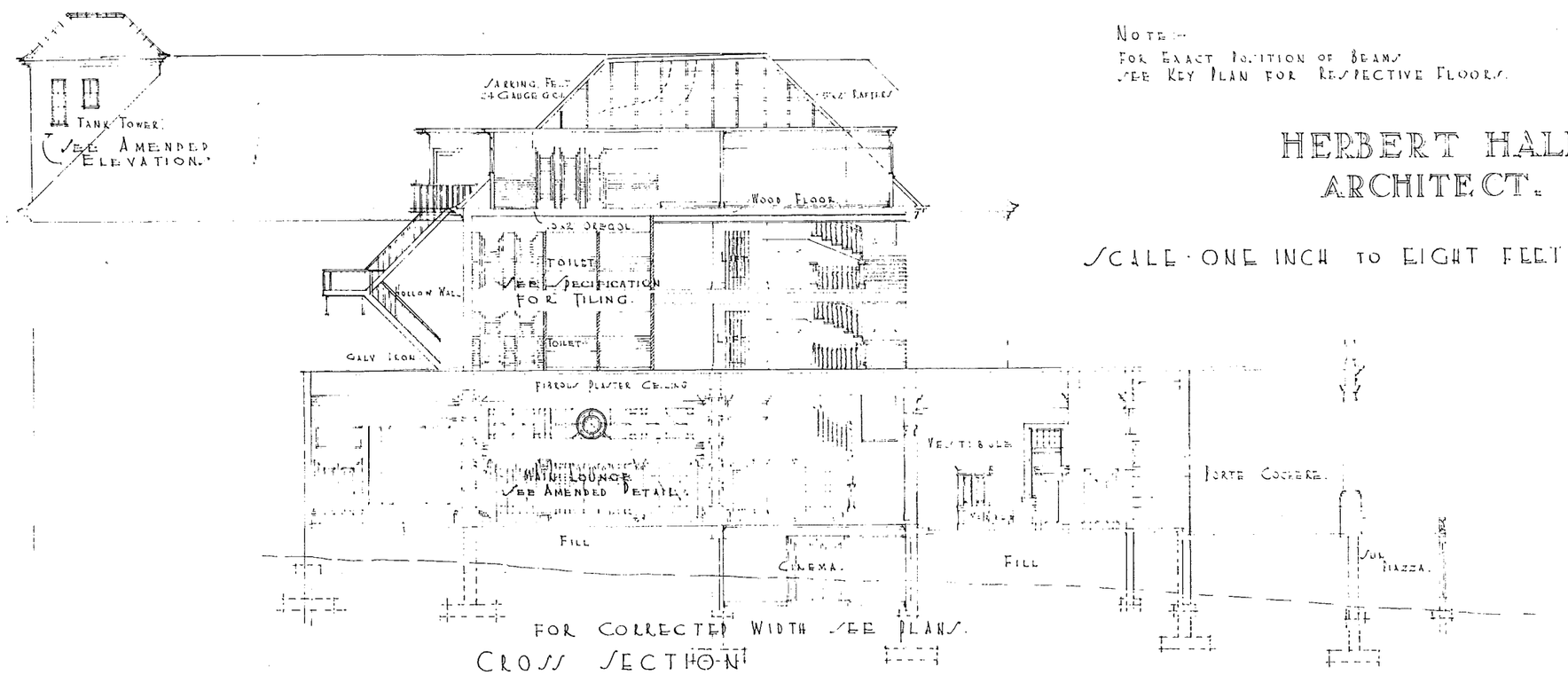
HERBERT HALL.
ARCHITECT.
TIMARU. N.Z.

CHATEAU TONGARIRO
NATIONAL PARK



For Corrected Length
See Plans.

LONGITUDINAL SECTION



For Corrected Width See Plans.

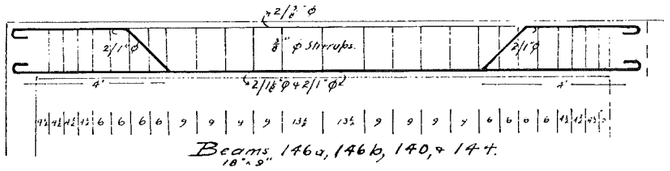
CROSS SECTION

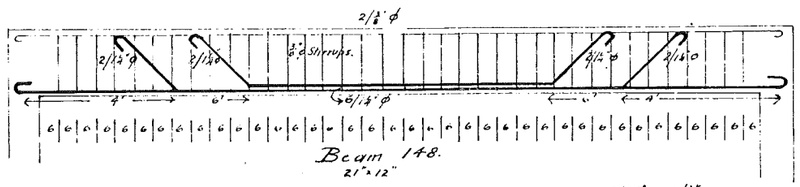
Note:-
For Exact Position of Beams
See Key Plan for Respective Floors.

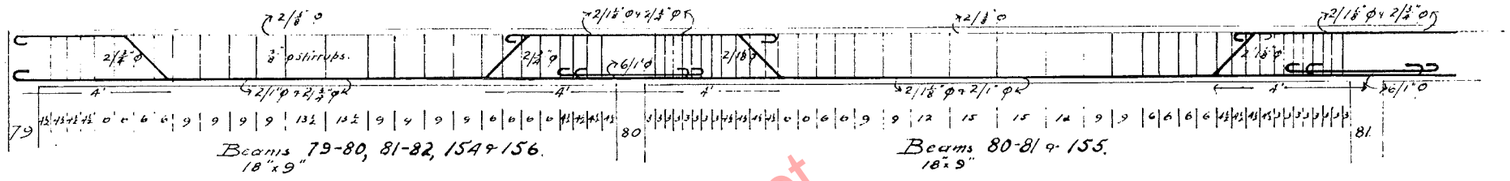
HERBERT HALL
ARCHITECT.

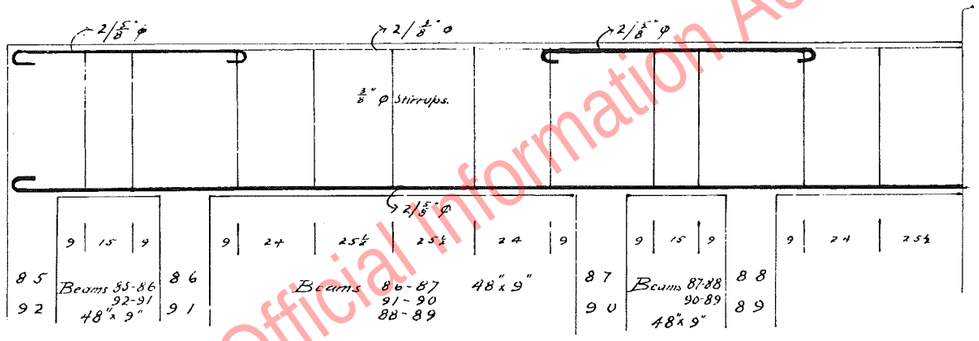
Scale - One Inch to Eight Feet.

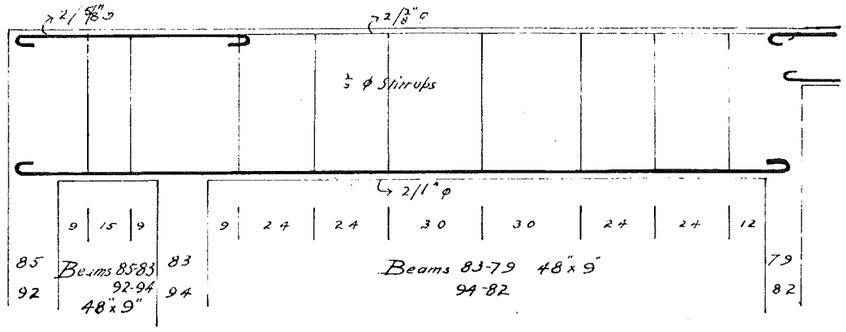
8

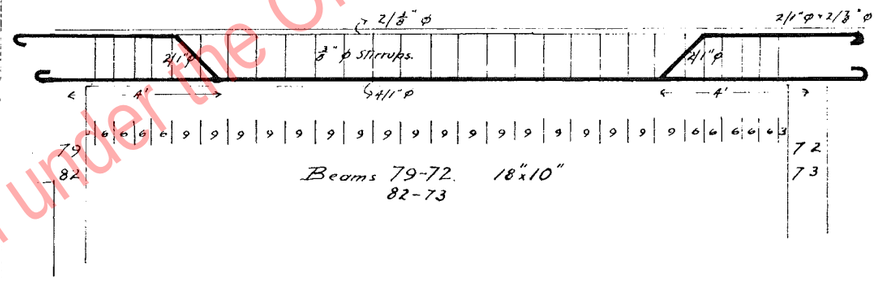


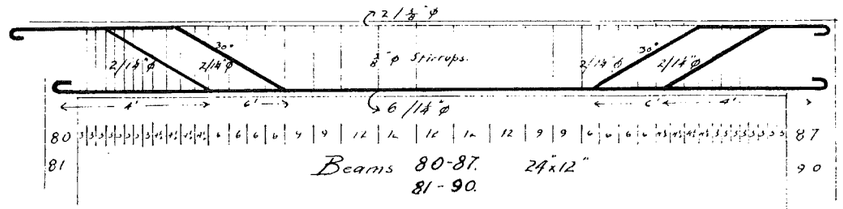


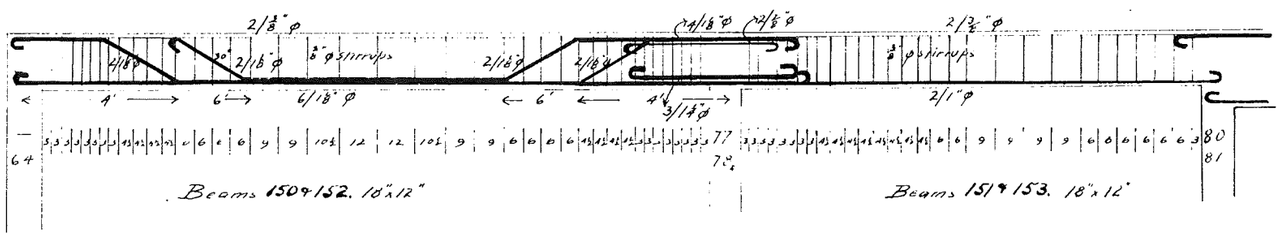


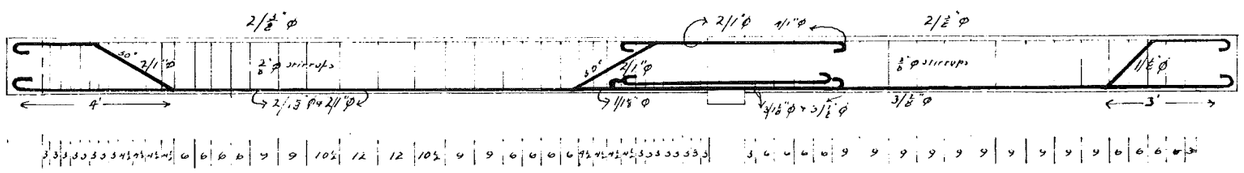


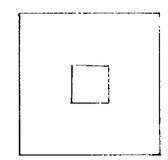










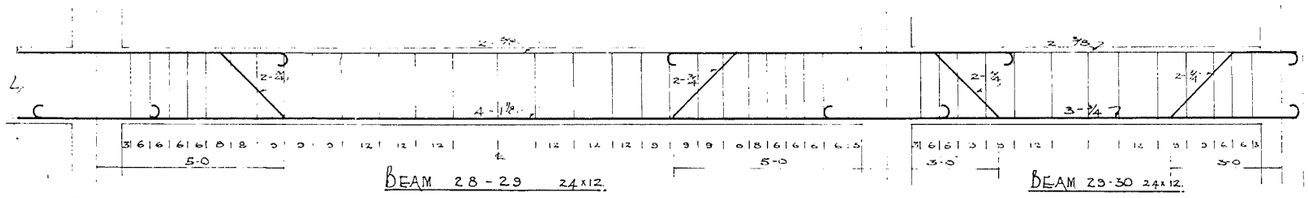


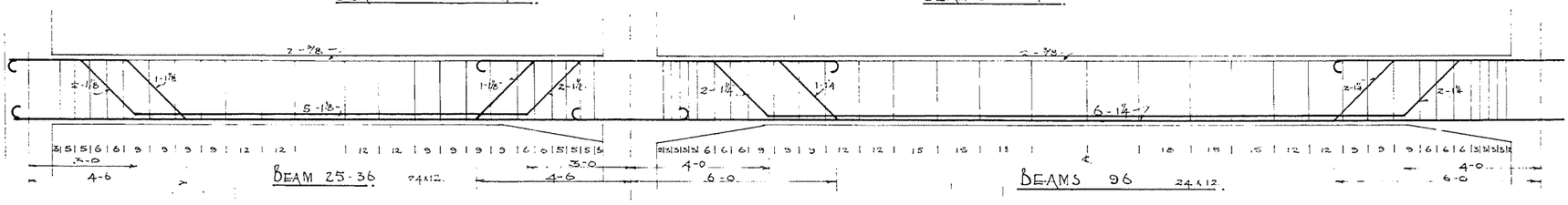
Typical Foundation.

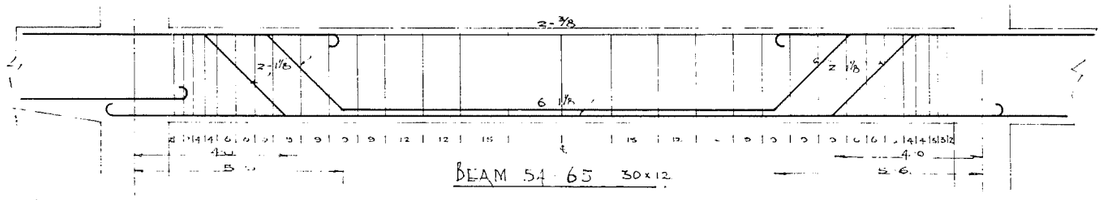
SCALE: HALF INCH TO ONE FOOT
DRAWN & TRACED BY
PASSED BY

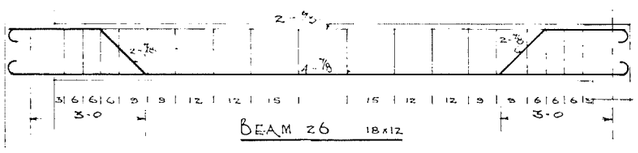
CHATEAU TONGARIRO · NATIONAL PARK · NEW ZEALAND.

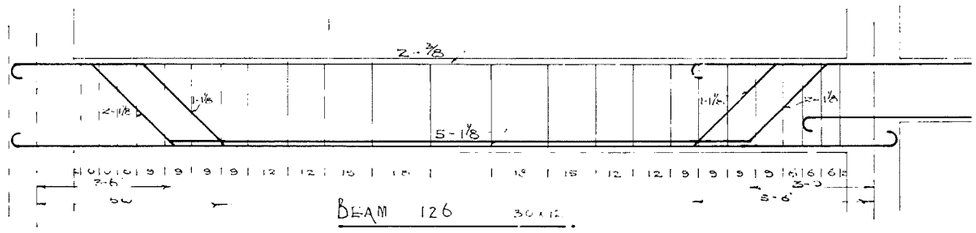
HERBERT HALL
ARCHITECT
TIMARU. N.Z.

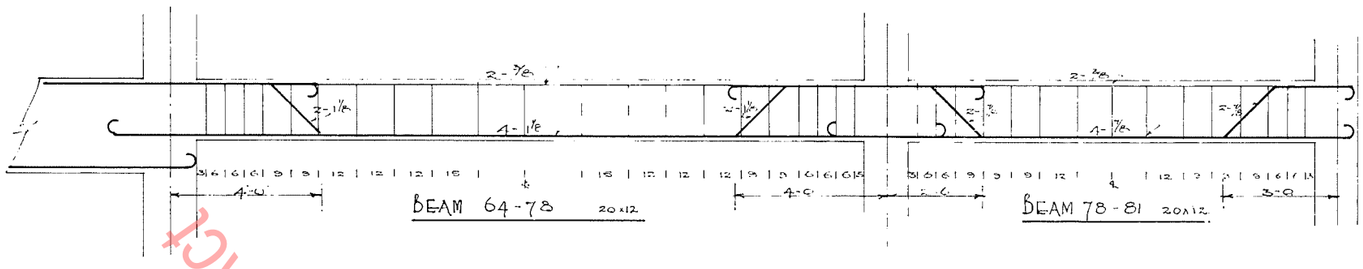


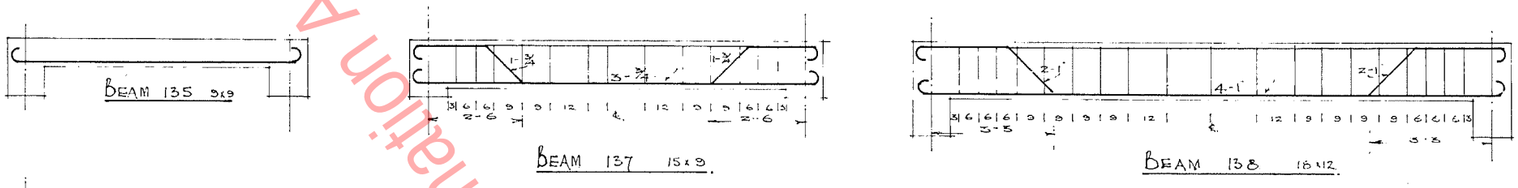


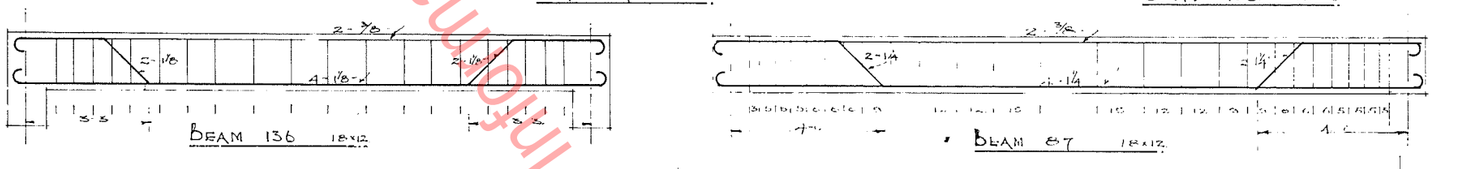


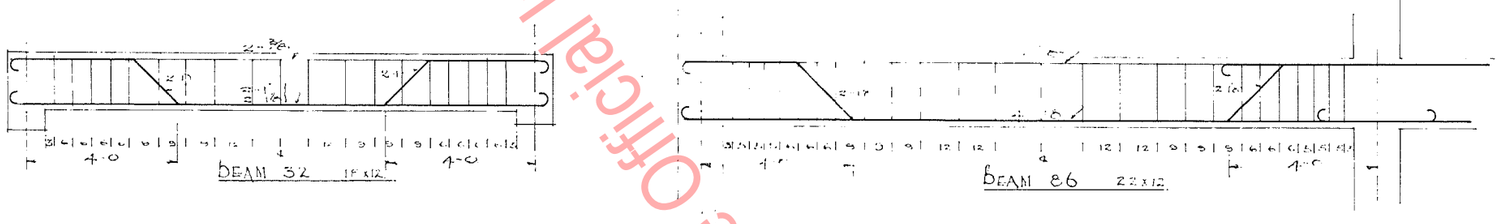


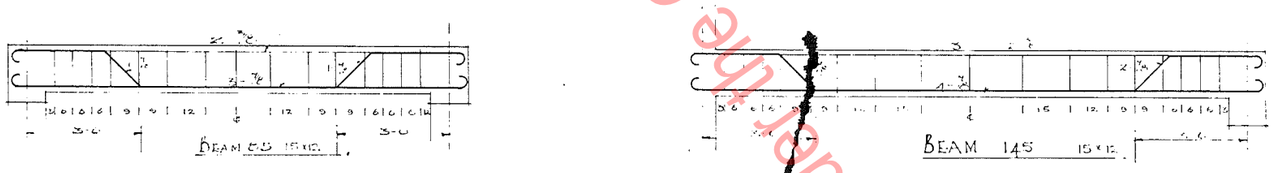












GROUND FLOOR BEAMS.

DRAWN BY NB.
TRACED BY NB.
PASSED BY

CHATEAU · TONGARIRO · NATIONAL PARK · N.Z.
HALF INCH DETAILS OF BEAMS.

HERBERT HALL
ARCHITECT
TIMARU · N.Z.

COLUMN NUMBER	FOUNDATION		BASEMENT TO GROUND FLOOR			GROUND TO FIRST FLOOR			FIRST FLOOR TO SECOND			SECOND FLOOR TO THIRD		
	SIZE	REINFORCEMENT	SIZE	REINFORCEMENT		SIZE	REINFORCEMENT		SIZE	REINFORCEMENT		SIZE	REINFORCEMENT	
1	3'-3" x 3'-3" x 1'-6"	1/2" ø @ 6" crs		NIL		16" x 16"	4 / 7/8" ø	3/8" @ 9"	12" x 12"	4 - 7/8" ø	3/8" @ 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
2	3'-6" x 3'-6" x 1'-6"	1/2" ø at 6" crs		NIL		16" x 14"	4 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 1" ø	3/8" at 9"	11" x 11"	4 - 7/8" ø	3/8" at 9"
3	do.			NIL		do.			do.			do.		
4	3'-6" x 3'-6" x 1'-6"	1/2" ø at 6" crs		NIL		16" x 14"	6 - 1" ø	3/8" at 9"	12" x 12"	4 - 1" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
5	3'-3" x 3'-3" x 1'-6"	1/2" ø at 6" crs		NIL		16" x 14"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 1" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
6	3'-0" x 3'-0" x 1'-6"	1/2" ø at 6" crs		NIL		16" x 16"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
7	3'-6" x 3'-6" x 1'-9"	1/2" ø at 6" crs	16" x 14"	6 - 1" ø	3/8" at 9"	16" x 14"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 1" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
8	3'-9" x 3'-9" x 1'-9"	1/2" ø at 6" crs	15" x 15"	6 - 1" ø	3/8" at 4"	15" x 15"	6 - 1" ø	3/8" at 9"	13" x 13"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
9	do.		do.			do.			do.			do.		
10	3'-9" x 3'-9" x 1'-9"	1/2" ø at 6" crs	16" x 16"	6 - 1" ø	3/8" at 3"	15" x 15"	6 - 1" ø	3/8" at 4"	13" x 13"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
11	5'-6" x 3'-6" x 1'-9"	1/2" ø at 6" crs	15" x 15"	6 - 1" ø	3/8" at 9"	15" x 15"	4 - 1" ø	3/8" at 9"	13" x 13"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
12	3'-3" x 3'-3" x 1'-6"	1/2" ø at 6" crs	16" x 14"	4 - 1" ø	3/8" at 9"	16" x 14"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
13	3'-0" x 3'-0" x 1'-6"	1/2" ø at 6" crs		NIL		16" x 14"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
14	3'-9" x 3'-9" x 1'-9"	1/2" ø at 6" crs	15" x 15"	6 - 1"	3/8" at 7"	15" x 15"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
15	3'-9" x 3'-9" x 1'-9"	1/2" ø at 6" crs	15" x 15"	6 - 1"	3/8" at 4"	15" x 15"	4 - 1" ø	3/8" at 9"	do.			do.		
16	4'-3" x 4'-3" x 2'-3"	5/8" ø at 3" crs	16" x 16"	6 - 1 1/8" ø	3/8" at 3"	15" x 15"	6 - 1" ø	3/8" at 4"	13" x 13"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
17	4'-0" x 4'-0" x 2'-3"	3/8" ø at 6" crs	15" x 15"	6 - 1" ø	3/8" at 3"	15" x 15"	4 - 1" ø	3/8" at 9"	13" x 13"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
18	3'-6" x 3'-6" x 1'-9"	1/2" ø at 6" crs	16" x 14"	4 - 1" ø	3/8" at 9"	16" x 14"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
19	3'-3" x 3'-3" x 1'-6"	1/2" ø at 6" crs		NIL		16" x 16"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
20	4'-0" x 4'-0" x 2'-0"	5/8" ø at 6" crs		NIL		16" x 16"	6 - 1" ø	3/8" at 5" spiral	16" x 12"	4 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
							OUT.							
22	3'-9" x 3'-9" x 1'-9"	1/2" ø at 6" crs		NIL		16" x 16"	4 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 1" ø	3/8 at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
23	2'-6" x 2'-6" x 1'-3"	1/2" ø at 6" crs		NIL		12" x 12"	4 - 3/4" ø	3/8" at 9"						
24	do.			NIL		do.								
25	5'-0" x 4'-0" x 1'-9"	5/8" ø at 6" crs	20" x 16"	6 - 1 1/8" ø	3/8" at 9"	20" x 16"	6 - 7/8" ø	3/8" at 9"	20" x 12"	6 - 3/4" ø	3/8" at 9"	20" x 12"	4 - 3/4" ø	3/8" at 9"
26	3'-9" x 3'-9" x 1'-9"	1/2" ø at 6" crs	15" x 15"	6 - 1" ø	3/8" at 3"	15" x 15"	6 - 1" ø	3/8" at 9"	13" x 13"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
26A	4'-3" x 4'-3" x 2'-0"	5/8" ø at 6" crs	15" x 15"	6 - 1 1/8" ø	3/8" at 4"	15" x 15"	6 - 1" ø	3/8" at 9"	do.			do.		
27	do.		16" x 16"	6 - 1 1/8" ø	3/8" at 3"	10" x 14"	6 - 1" ø	3/8" at 9"	12" x 12"	4 - 1" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
28	4'-3" x 4'-3" x 2'-0"	5/8" ø at 6" crs	16" x 16"	6 - 1 1/8" ø	3/8" at 3"	do.			do.			do.		
29	4'-0" x 4'-0" x 2'-0"	5/8" ø at 6" crs	16" x 16"	6 - 1 1/8" ø	3/8" at 6"	16" x 14"	6 - 1" ø	3/8" at 9"	do.			do.		
30	5'-4" x 3'-4" x 1'-6"	1/2" ø at 6" crs	16" x 14"	4 - 1" ø	3/8 at 9"	16" x 14"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
31	4'-0" x 4'-0" x 1'-9"	5/8" ø at 6" crs		NIL		16" x 16"	6 - 1 1/8" ø	3/8" at 9"	16" x 12"	4 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
32	4'-9" x 4'-9" x 2'-3"	5/8" ø at 6" crs		NIL		16" x 16"	6 - 1 1/8" ø	3/8" at 3" spiral	14" x 14"	6 - 1" ø	3/8" at 3" spiral	12" x 12"	4 - 7/8" ø	3/8" at 9"
33	5'-6" x 5'-6" x 2'-6"	5/8" ø at 6" crs		NIL		19" x 19"	6 - 1 1/8" ø	3/8" at 3" spiral	15" x 15"	6 - 1" ø	3/8" at 3" spiral	12" x 12"	4 - 1" ø	3/8" at 9"
34	3'-9" x 3'-9" x 1'-9"	1/2" ø at 6" crs		NIL		16" x 16"	4 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
35	5'-3" x 5'-3" x 2'-3"	5/8" ø at 6" crs	17" x 17"	6 - 1 1/4" ø	3/8" at 3" spiral	16" x 16"	6 - 1 1/8" ø	3/8" at 9"	12" x 12"	6 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
36	5'-9" x 5'-9" x 2'-6"	5/8" ø at 6" crs	19" x 19"	6 - 1 1/2" ø	3/8" at 3"	19" x 19"	6 - 1" ø	3/8" at 9"	15" x 15"	6 - 3/4" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
37	4'-0" x 4'-0" x 2'-0"	5/8" ø at 6" crs	15" x 15"	6 - 1 1/8" ø	3/8" at 3"	15" x 15"	6 - 1" ø	3/8" at 9"	12" x 12"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
38	do.		do.			do.			do.			do.		
39	5'-0" x 5'-0" x 2'-3"	5/8" ø at 5" crs	20" x 20"	6 - 1 1/8" ø	3/8" at 3"	20" x 20"	6 - 1" ø	3/8" at 9"	14" x 14"	6 - 3/4" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
40	4'-3" x 4'-3" x 1'-9"	5/8" ø at 6" crs	16" x 16"	6 - 1" ø	3/8" at 3"	16" x 14"	4 - 1" ø	3/8" at 9"	14" x 14"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
41	4'-3" x 4'-3" x 2'-0"	5/8" ø at 6" crs				16" x 16"	5 - 1 1/8" ø	3/8" at 3" spiral	14" x 14"	5 - 1" ø	3/8" at 4"	12" x 12"	4 - 1" ø	3/8" at 9"
42	NONE. CARRIED ON BEAM.								14" x 14"	6 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
43	4'-3" x 4'-3" x 1'-9"	5/8" ø at 6" crs				16" x 16"	5 - 1" ø	3/8 at 3" spiral	14" x 14"	5 - 1"	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
44	4'-9" x 4'-9" x 2'-0"	5/8" ø at 6" crs	16" x 16"	6 - 1 1/4" ø	3/8" at 3"	16" x 16"	6 - 7/8" ø	3/8" at 3"	14" x 14"	6 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	
45	3'-6" x 3'-6" x 1'-9"	1/2" ø at 6" crs	15" x 15"	4 - 1 1/8" ø	3/8" at 6"	15" x 15"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
46	4'-0" x 4'-0" x 2'-0"	5/8" ø at 6" crs	15" x 15"	6 - 1 1/8" ø	3/8" at 3"	15" x 15"	6 - 1" ø	3/8" at 9"	12" x 12"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
47 & 48	NIL		NIL			NIL			do.			do.		
49	5'-0" x 5'-0" x 2'-3"	5/8" ø at 6" crs	17" x 17"	6 - 1 1/4" ø	3/8" at 3" spiral	16" x 16"	6 - 1 1/8"	3/8" at 3" spiral	16" x 14"	6 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"
50	5'-3" x 5'-3" x 2'-3"	5/8" ø at 6" crs	17" x 17"	6 - 1 1/4" ø	3/8" at 3"	16" x 16"	6 - 1 1/8" ø	3/8" at 3"	14" x 14"	6 - 1" ø	3/8" at 3"	12" x 12"	4 - 3/4" ø	3/8" at 9"
51	6'-6" x 6'-6" x 2'-9"	5/8" ø at 6" crs	22" x 22"	8 - 1 3/8" ø	3/8" at 3" spiral	20 x 20"	8 - 1 1/4" ø	3/8" at 3" spiral	14" x 14"	4 - 1" ø	3/8" at 4"	12" x 12"	4 - 3/4" ø	3/8" at 9"
52	5'-6" x 5'-6" x 2'-6"	5/8" ø at 6" crs	17" x 17"	6 - 1 1/8"	3/8" at 3" spiral	16" x 16"	6 - 1 1/4" ø	3/8" at 3" spiral	14" x 14"	6 - 1" ø	3/8" at 6"			
53	5'-6" x 5'-6" x 2'-3"	5/8" ø at 6" crs	18" x 18"	6 - 1 1/4" ø	3/8" at 3" spiral	16" x 16"	6 - 1 1/4" ø	3/8" at 3" spiral	14" x 14"	6 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
54	6'-6" x 6'-6" x 2'-9"	5/8" ø at 6" crs	22" x 22"	7 - 1 1/2" ø	3/8" at 3"	20" x 20"	7 - 1 1/4" ø	3/8" at 3"	14" x 14"	6 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
55	3'-9" x 3'-9" x 1'-9"	1/2" ø at 6" crs	15" x 15"	6 - 1" ø	3/8" at 3"	15" x 15"	6 - 1" ø	3/8" at 9"	12" x 12"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
56	4'-0" x 4'-0" x 2'-0"	5/8" ø at 6" crs	15" x 15"	6 - 1 1/8" ø	3/8" at 3"	15" x 15"	4 - 1 1/8" ø	3/8" at 9"	do.			do.		
57	5'-0" x 5'-0" x 2'-3"	5/8" ø at 6" crs	20" x 20"	6 - 1 1/8" ø	3/8" at 3"	20" x 20"	6 - 1" ø	3/8" at 9"	14" x 14"	6 - 3/4" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
58	4'-3" x 4'-3" x 1'-9"	5/8" ø at 6" crs	16" x 16"	6 - 1" ø	3/8" at 3"	16" x 14"	4 - 1" ø	3/8" at 9"	14" x 14"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
59	4'-3" x 4'-3" x 2'-0"	5/8" ø at 6" crs	16" x 16"	6 - 1 1/8" ø	3/8" at 3"	16" x 16"	6 - 1 1/8" ø	3/8" at 9"	16" x 12"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
60 & 60A	COMBINED	1/2" ø at 6" crs	27" x 12"	4 - 7/8" ø	3/8" at 9"	27" x 12"	4 - 7/8" ø	3/8" at 9"	18" x 12"	4 - 3/4"	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
61 & 61A	12'-0" x 6'-0" x 1'-6"		do.			do.			do.			do.		
62	5'-6" x 5'-6" x 2'-6"	3/4" ø at 6" crs	17" x 17"	6 - 1 1/4" ø	3/8" at 3" sp	17" x 17"	6 - 1" ø	3/8" at 9"	13" x 13"	6 - 1" ø	3/8" at 9"	12" x 12"	4 - 7/8"	3/8" at 9"
63 & 64	5'-6" x 5'-6" x 2'-6"	3/4" ø at 6" crs	18" x 18"	7 - 1 1/4" ø	3/8" at 3" sp	17" x 17"	6 - 1" ø	3/8" at 9"	13" x 13"	6 - 1" ø	3/8" at 9"	12" x 12"	4 - 7/8"	3/8" at 9"
65	5'-6" x 5'-6" x 2'-3"	5/8" ø at 6" crs	18" x 18"	6 - 1 3/8" ø	3/8" at 3"	18" x 18"	6 - 1 1/4" ø	3/8" at 9"	14" x 14"	6 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 3/4"	3/8" at 9"
66	4'-3" x 4'-3" x 2'-0"	5/8" ø at 6" crs	15" x 15"	6 - 1 1/8" ø	3/8" at 9"	15" x 15"	6 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 3/4"	3/8" at 9"
67	do.		do.			do.			do.			do.		
68	4'-0" x 4'-0" x 2'-0"	5/8" ø at 6" crs	16" x 16"	6 - 1" ø	3/8" at 3"	16" x 16"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"
69	3'-9" x 3'-9" x 1'-9"	1/2" ø at 6" crs	16" x 16"	4 - 1/8	3/8" at 9"	16" x 16"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
70 & 71	3'-9" x 3'-9" x 1'-6"	5/8" ø at 6" crs	16" x 16"	4 - 1" ø	3/8" at 9"	16" x 16"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
72	4'-0" x 4'-0" x 1'-9"	5/8" ø at 9" crs	16" x 16"	6 - 1" ø	3/8" at 9"	16" x 16"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
73	4'-3" x 4'-3" x 1'-9"	5/8" ø at 6" crs	16" x 16"	6 - 1 1/8" ø	3/8" at 9"	16" x 16"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
74 & 75	3'-6" x 3'-6" x 1'-9"	1/2" ø at 6" crs	16" x 14"	6 - 1" ø	3/8" at 9"	16" x 14"	6 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
76	3'-0" x 3'-0" x 1'-6"	1/2" ø at 6" crs	16" x 16"	4 - 1" ø	3/8" at 9"	16" x 16"	4 - 7/8" ø	3/8" at 9"	12" x 12"	4 - 7/8" ø	3/8" at 9"	11" x 11"	4 - 3/4" ø	3/8" at 9"
77	4'-0" x 4'-0" x 1'-9"	3/4" ø at 6" crs	14" x 14"	6 - 7/8" ø	3/8" at 3" sp	10" x 10"	4 - 7/8" ø	3/8" at 9"						
78	4'-0" x 4'-0" x 1'-9"	3/4" ø at 6" crs	14" x 14"	5 - 1 1/8" ø	3/8" at 3" sp	12" x 12"	4 - 3/4" ø	3/8" at 9"						
78A	2'-0" x 2'-0" x 1'-3"	1/2" ø at 6" crs	12" x 12"	4 - 3/4" ø	3/8" at 9"									
79	3'-0" x 3'-0" x 1'-5"	5/8" ø at 6" crs	14" x 14"	4 - 1 1/8" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"						
80	4'-0" x 4'-0" x 1'-9"	3/4" ø at 6" crs	14" x 14"	5 - 1" ø	3/8" at 9"	11" x 11"	4 - 1" ø	3/8" at 9"						
81	4'-0" x 4'-0" x 1'-9"	3/4" ø at 6" crs	14" x 14"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"						
82	3'-0" x 3'-0" x 1'-5"	3/4" ø at 6" crs	14" x 14"	4 - 1" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"						
83 & 84	2'-6" x 2'-6" x 1'-2"	5/8" ø at 6" crs	18" x 18"	6" PIPE IN CENTRE		18" Diam.	6" PIPE IN CENTRE							
85, 87, 90, 92.	2'-6" x 2'-6" x 1'-3"	5/8" ø at 6" crs	18" x 18"	4 - 3/4" ø	3/8" at 9"	18" Diam.	4 - 3/4" ø	3/8" at 9"						
86, 88, 89, 91.	2'-6" x 2'-6" x 1'-3"	5/8" at 6" crs	18" x 18"	6" PIPE IN CENTRE		18" Diam.	6" PIPE IN CENTRE							
44 X.									14" x 14"	4 - 3/4" ø	3/8" at 9"	12" x 12"	4 - 3/4" ø	3/8" at 9"

DRAWING LIST	
DWG NO	DRAWING NAME
S-0301	PERSPECTIVE VIEWS
S-0302	PERSPECTIVE VIEWS POINT CLOUD
S-1211	FOUNDATION PLAN
S-1301	BASEMENT WALLS AND COLUMNS
S-1302	GROUND FLOOR - WALLS AND COLUMNS
S-1303	FIRST FLOOR - WALLS AND COLUMNS
S-1304	SECOND FLOOR WALLS AND COLUMNS
S-1305	THIRD FLOOR - WALLS AND COLUMNS
S-1306	ROOF - PLAN VIEW
S-1401	GROUND FLOOR BEAMS
S-1411	FIRST FLOOR BEAMS
S-1412	SECOND FLOOR BEAMS
S-1413	THIRD FLOOR BEAMS
S-1811	FIRST FLOOR FIRE RATED STRUCTURE PLAN
S-1901	GROUND FLOOR SERVICES SEISMIC RESTRAINT PLAN
S-1911	FIRST FLOOR SERVICES SEISMIC RESTRAINT PLAN
S-3001	SECTION AT GRID 1
S-3003	SECTION AT GRID 2
S-3005	SECTION AT GRID 3
S-3101	SECTION AT GRID A
S-3103	SECTION AT GRID B
S-3105	SECTION AT GRID C
S-4001	LIFT/STAIR CORE 1 PLANS
S-4011	LIFT/STAIR CORE 1 SECTIONS/ELEVATIONS
S-4021	LIFT/STAIR CORE 1 CONCRETE DETAILS
S-4031	LIFT/STAIR CORE 1 STEELWORK DETAILS
S-4101	LIFT/STAIR CORE 2 PLANS
S-4111	LIFT/STAIR CORE 2 SECTIONS/ELEVATIONS
S-4121	LIFT/STAIR CORE 2 CONCRETE DETAILS
S-4131	LIFT/STAIR CORE 2 STEELWORK DETAILS
S-5001	PRECAST CONCRETE PANEL ELEVATIONS
S-5003	PRECAST CONCRETE PANEL ELEVATIONS
S-6001	PILE DETAILS
S-7001	CONCRETE DETAILS
S-7003	CONCRETE DETAILS
S-8001	STEELWORK DETAILS
S-8003	STEELWORK DETAILS
S-9001	TIMBER DETAILS
S-9003	TIMBER DETAILS

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CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT

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Project No: 2-37905.01

NOTES:

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A	FOR INFORMATION	HKN	2023-01-26

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SCALE: 1 : 1 @ A1
ORIGINAL SIZE: A1

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APPROVED DATE: 2023-01-26

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TITLE
PERSPECTIVE VIEWS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
2-37905.01	237905-WSP-01-DR	S-0301	A

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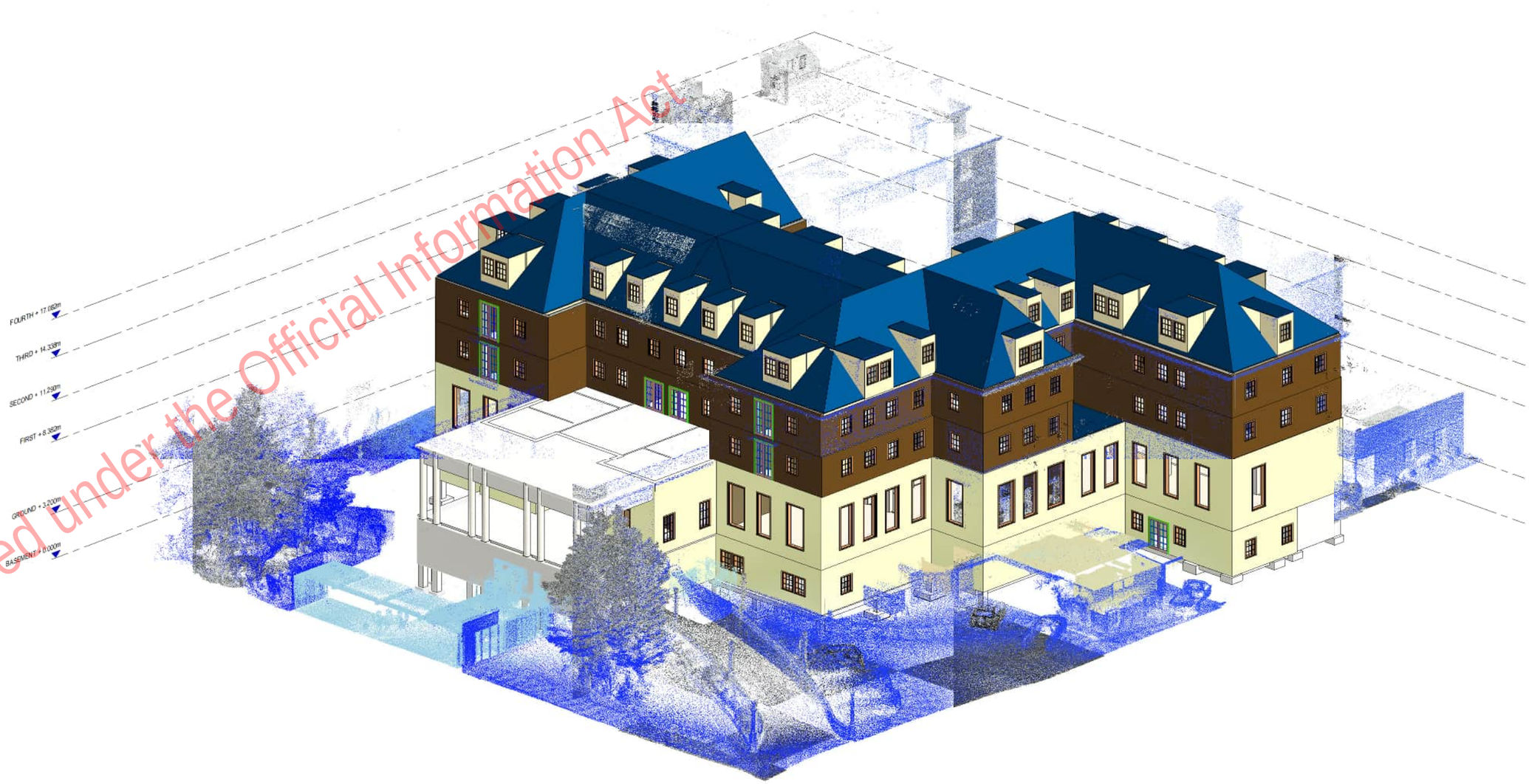

3D - MODEL WITH POINT CLOUD
SCALE

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DESIGN VERIFIED
APPROVED DATE 2023-01-26

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PROJECT
KAH NEW ZEALAND LTD
CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT
TITLE
PERSPECTIVE VIEWS POINT CLOUD

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
2-37905.01	237905-WSP-01-DR	S-0302	A

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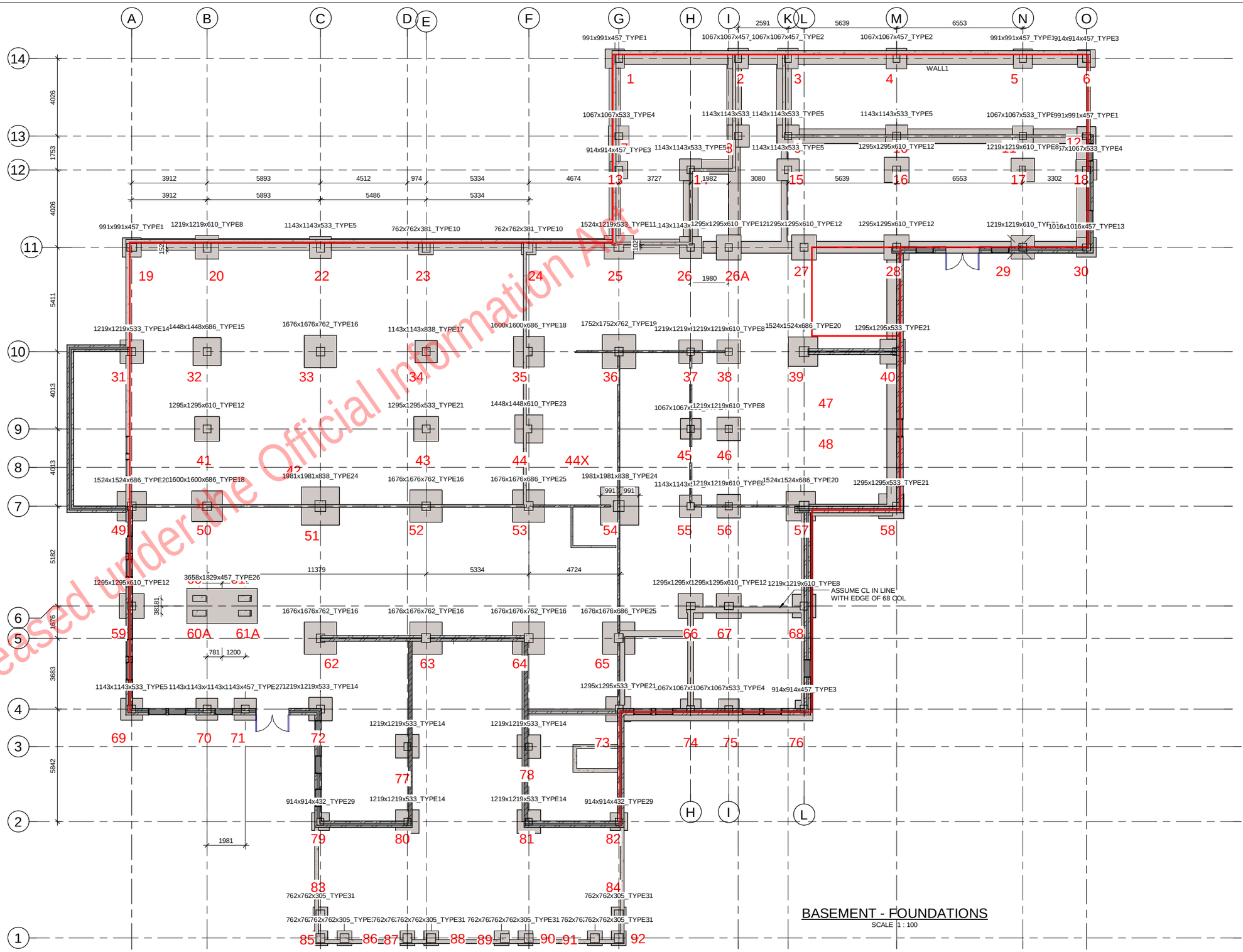

NOTES:

1. ALL DWGS TO BE READ IN CONJUNCTION WITH THE WSP STRUCTURAL SPECIFICATION.
2. EXISTING STRUCTURE IS SHOWN INDICATIVELY ONLY. VERIFY DIMENSIONS & CONFIGURATION OF EXISTING STRUCTURE ON SITE BEFORE COMMENCING ANY WORKS & NOTIFY ENGINEER IF ANY DIFFERENCES TO WHAT IS INDICATED ON THE DWGS ARE ENCOUNTERED.
3. EXISTING SERVICES NOT SHOWN. VERIFY LOCATION OF ALL EXISTING SERVICES ON SITE BEFORE COMMENCING ANY WORKS.
4. REFER TO ARCH DWGS FOR ALL SET OUT DIMENSIONS, LEVELS, FALLS, SLAB SET DOWNS, REBATES, EDGE DETAILS & CAST IN ARCH ITEMS U.N.O.
5. ANY SERVICES PENETRATIONS & FIXTURES SHOWN ARE SHOWN INDICATIVELY ONLY. REFER SERVICES DWGS FOR LOCATION & SIZE OF SERVICES, SERVICES PENETRATIONS & SERVICES FIXTURES U.N.O.
6. ALL EXTERNALLY EXPOSED STEELWORK INCLUDING FIXINGS TO BE HOT DIP GALVANIZED U.N.O.

BASEMENT - FOUNDATIONS

SCALE 1 : 100

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PROJECT
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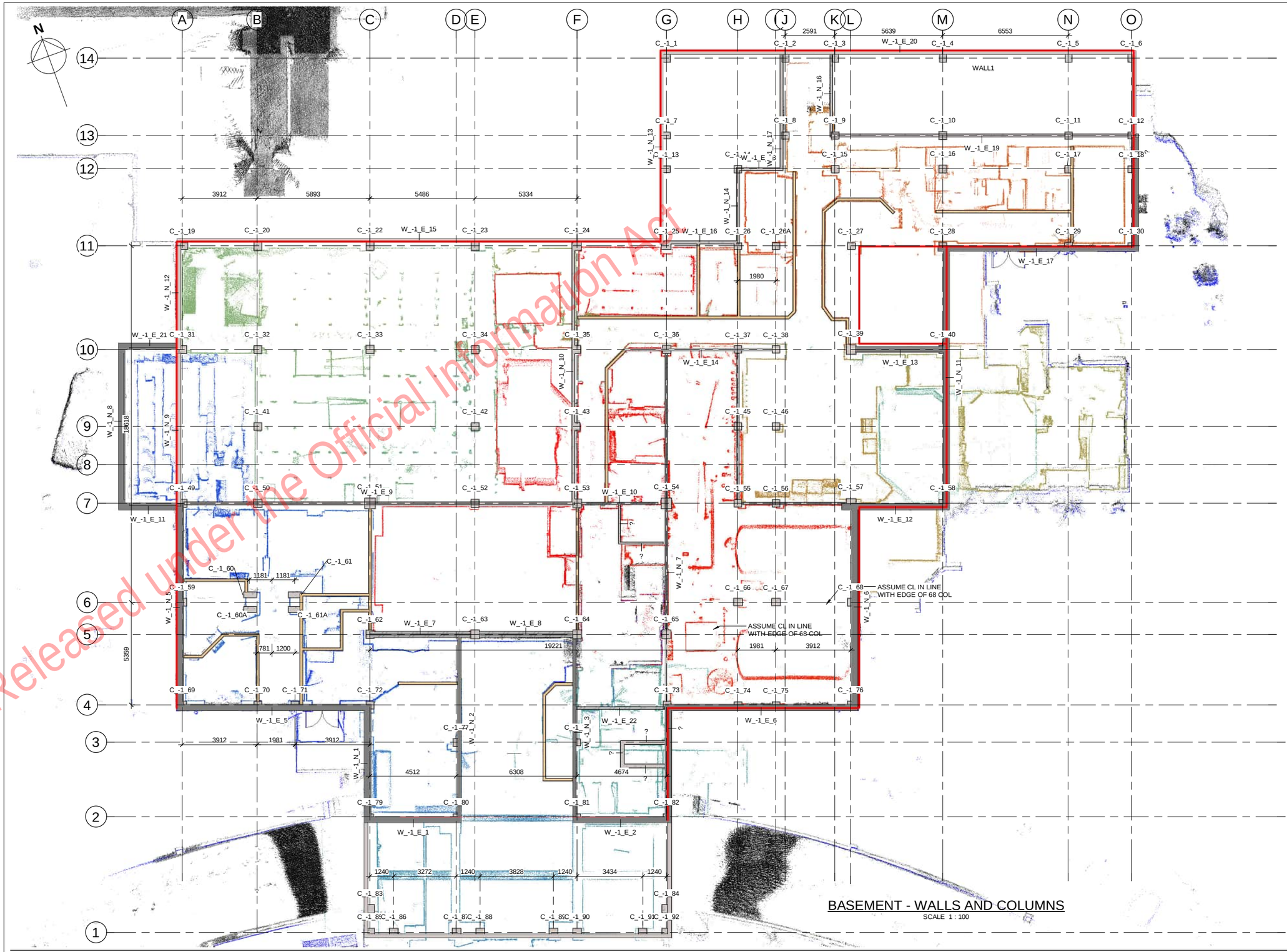
DETAILED SEISMIC ASSESSMENT

TITLE
FOUNDATION PLAN

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
2-37905.01	237905-WSP-01-DR	S-1211	A



NOTES

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BASEMENT - WALLS AND COLUMNS

SCALE 1 : 100

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TITLE
BASEMENT WALLS AND COLUMNS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
2-37905.01	237905-WSP-01-DR	S-1301	A

NOTES

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GROUND FLOOR - WALLS AND COLUMNS

SCALE 1 : 100

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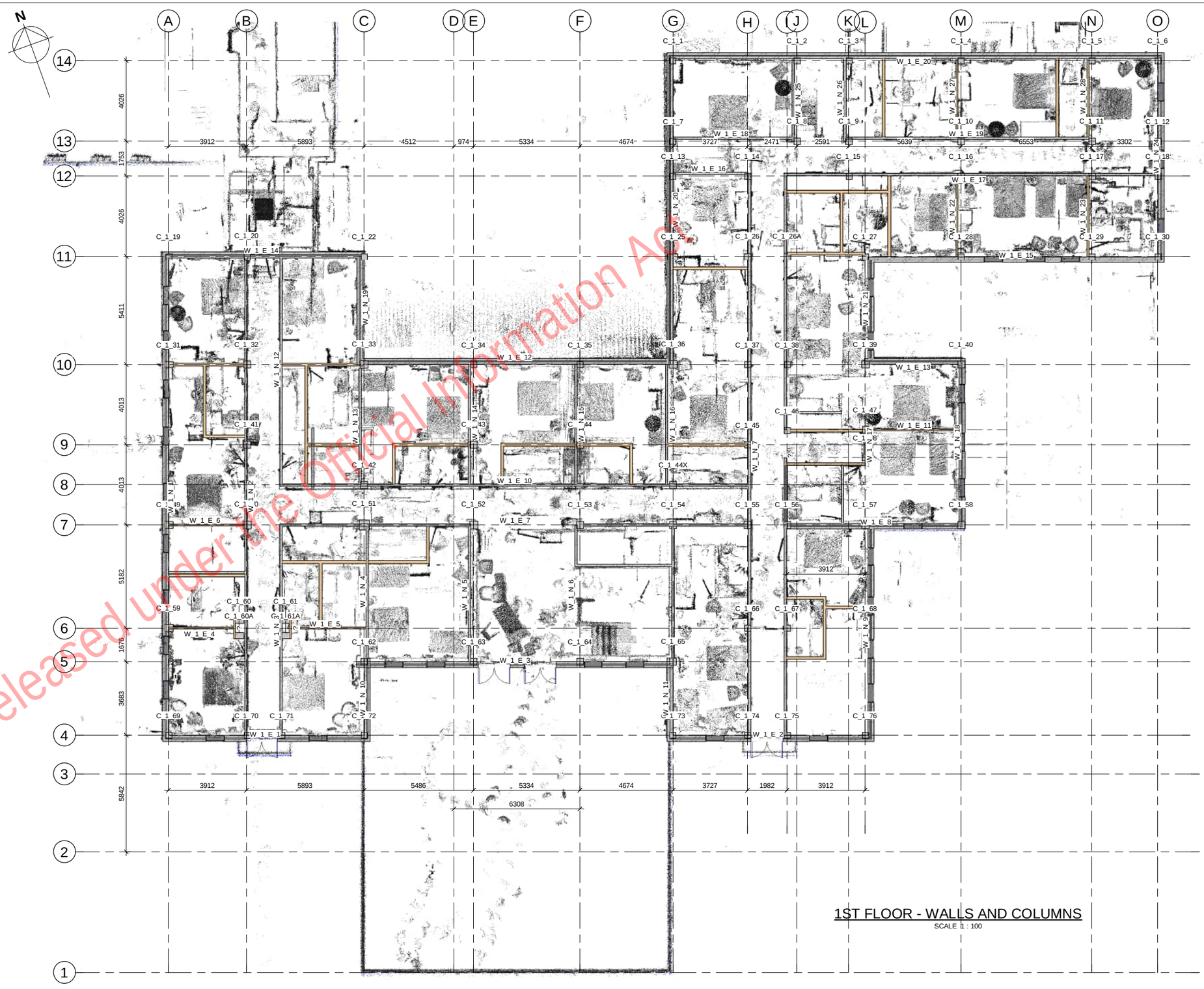
TITLE
GROUND FLOOR - WALLS AND COLUMNS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
2-37905.01	237905-WSP-01-DR	S-1302	A



NOTES

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1ST FLOOR - WALLS AND COLUMNS

SCALE 1 : 100

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APPROVED DATE: 2023-01-26

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PROJECT
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CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT

TITLE
FIRST FLOOR - WALLS AND COLUMNS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
2-37905.01	237905-WSP-01-DR	S-1303	A

NOTES

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2ND FLOOR - WALLS AND COLUMNS

SCALE 1 : 100

DRAWING IN PROGRESS

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PROJECT: KAH NEW ZEALAND LTD
CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT

TITLE: SECOND FLOOR WALLS AND COLUMNS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
2-37905.01	237905-WSP-01-DR	S-1304	A

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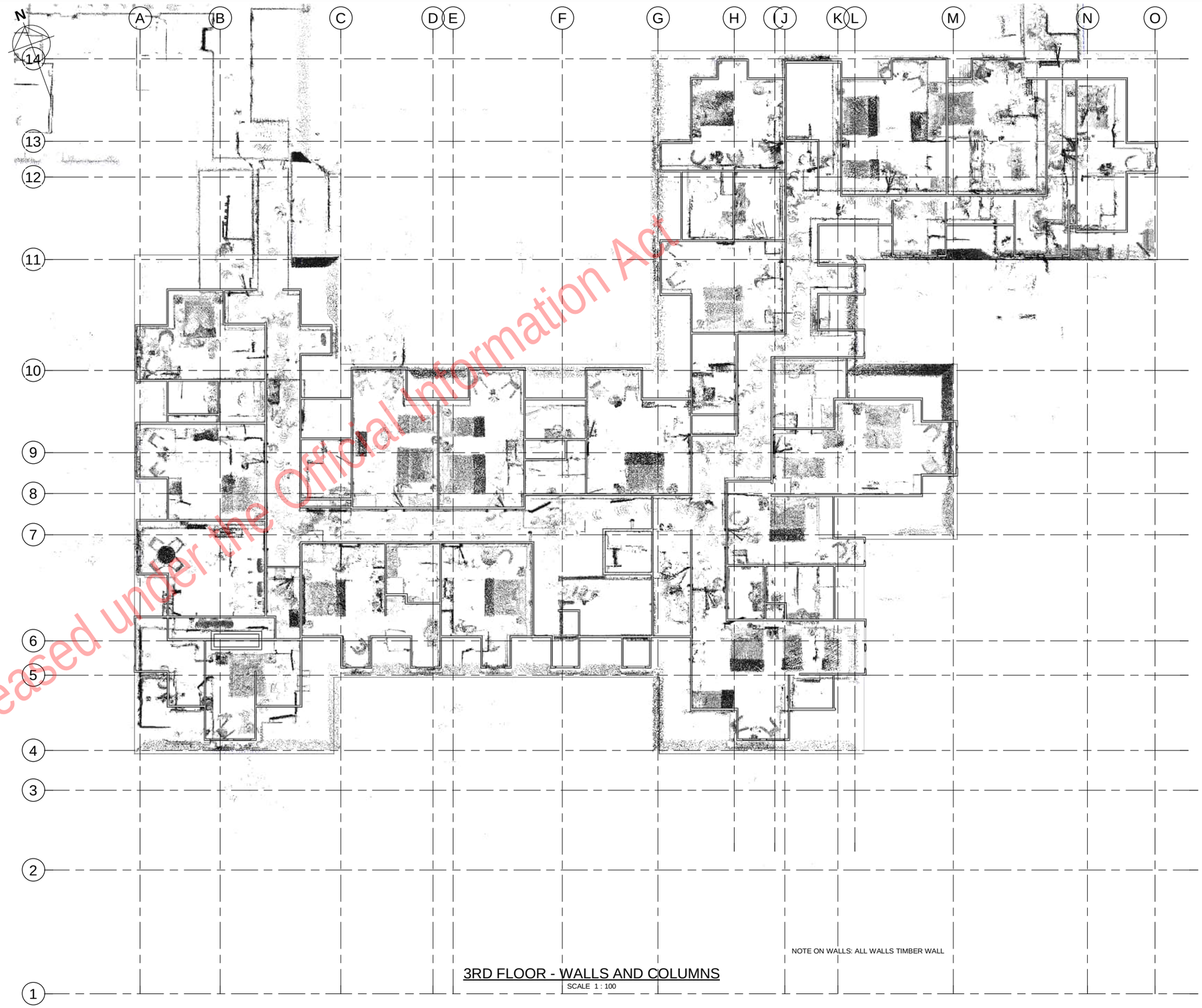

NOTES

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6. ALL EXTERNALLY EXPOSED STEELWORK INCLUDING FIXINGS TO BE HOT DIP GALVANIZED U.N.O.

NOTE ON WALLS: ALL WALLS TIMBER WALL

3RD FLOOR - WALLS AND COLUMNS

SCALE 1 : 100

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PROJECT: KAH NEW ZEALAND LTD
CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT

TITLE: THIRD FLOOR - WALLS AND COLUMNS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
2-37905.01	237905-WSP-01-DR	S-1305	A

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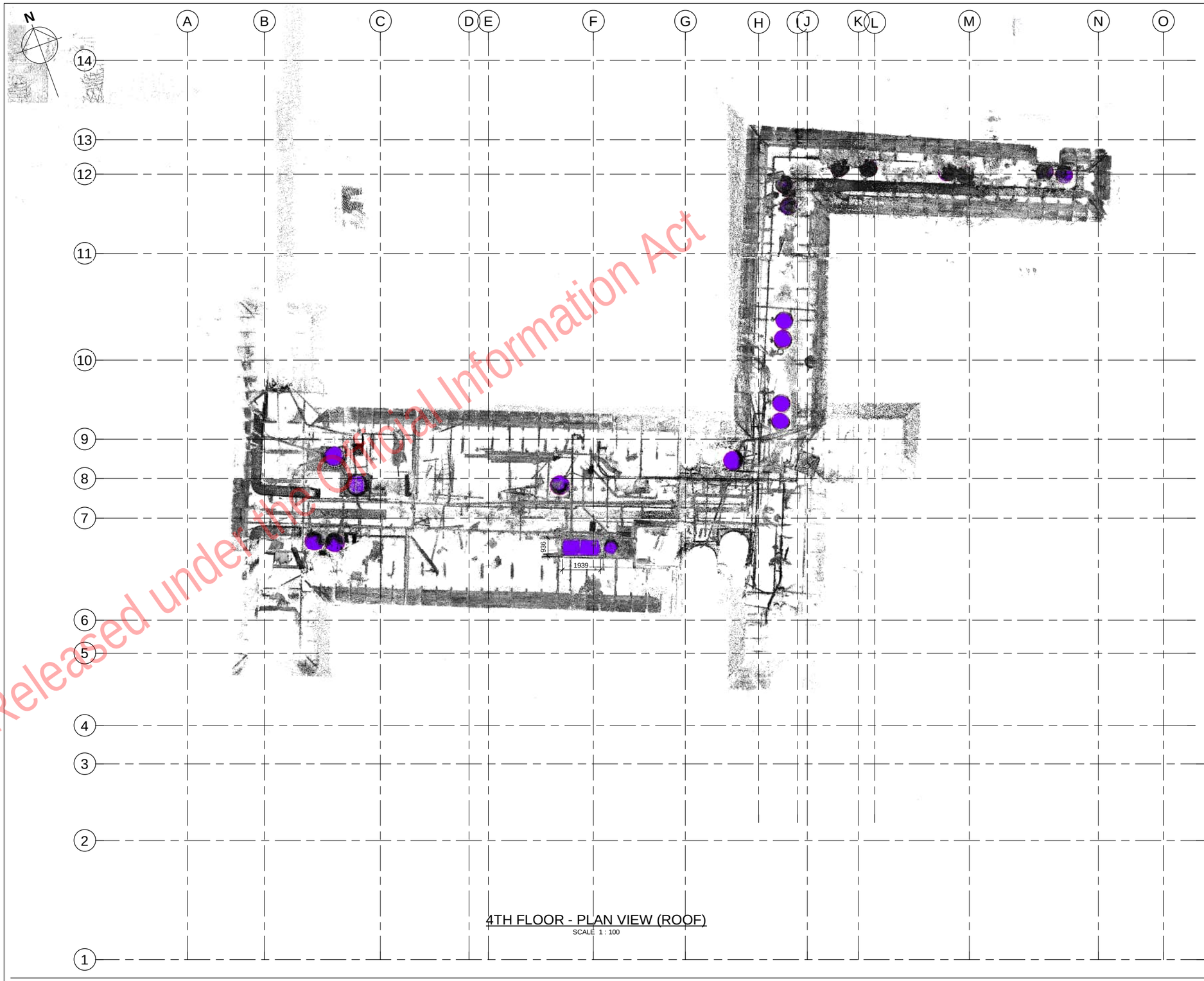

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300mm
200mm
100
50
0 10mm

4TH FLOOR - PLAN VIEW (ROOF)

SCALE 1 : 100

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CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT

TITLE
ROOF - PLAN VIEW

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
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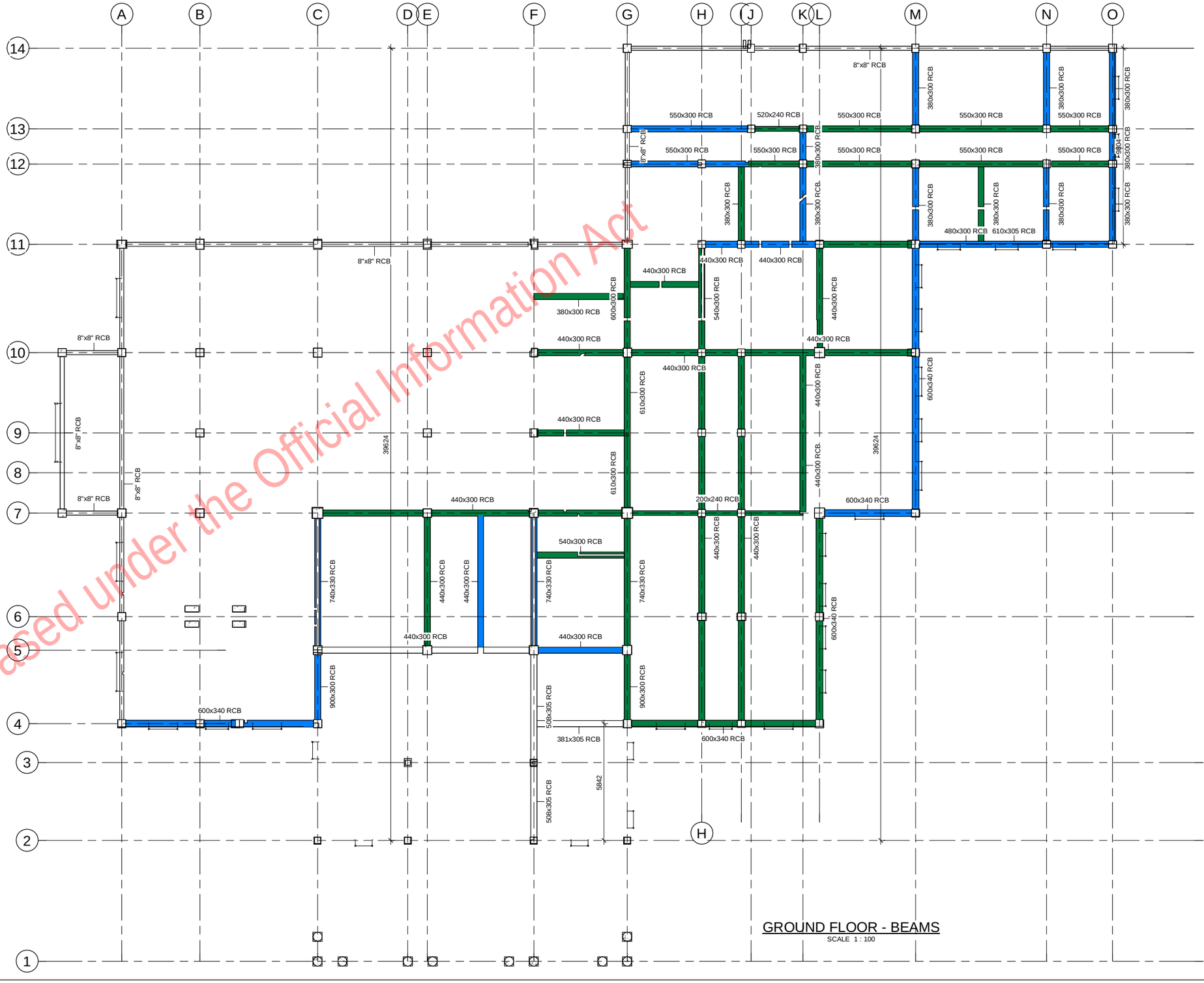

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LEGEND

- 150 — 150 THK SLAB WITH SE82 TOP ON DPC ON 25 NOM SAND BLINDING ON 150 MIN WELL COMPACTED GAP65 HARDFILL ON WELL COMPACTED SUBGRADE.
- 500 — 500 DP RAFT SLAB.
- 250 THK CONC WALL.
- STEP IN FLOOR (REFER ARCH DWGS U.N.O).
- SIZES MEASURED
- SIZES ASSUMED
- SIZES FROM DRAWINGS

GROUND FLOOR - BEAMS

SCALE 1 : 100

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PROJECT: KAH NEW ZEALAND LTD
CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT

TITLE: GROUND FLOOR BEAMS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
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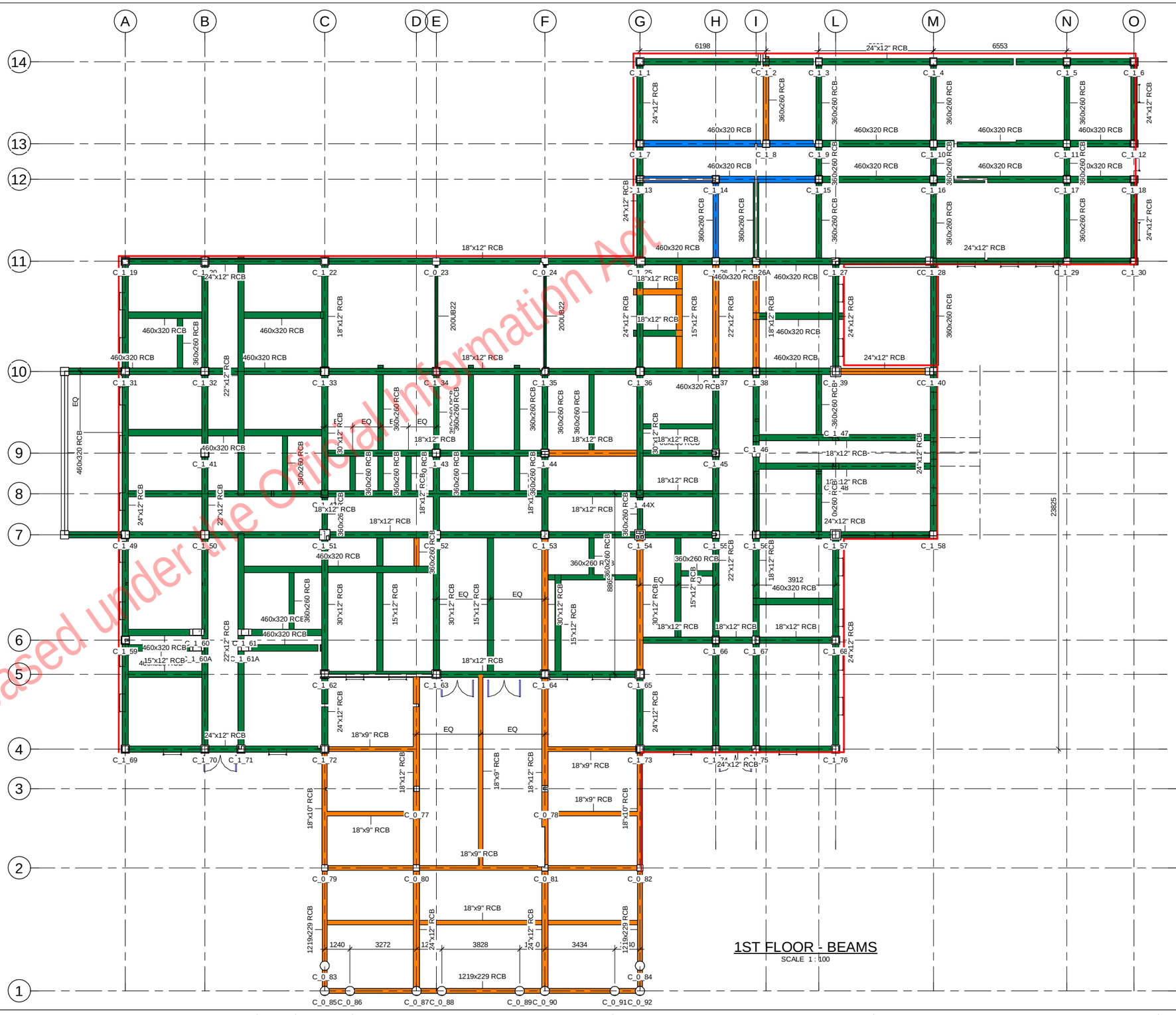

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1ST FLOOR - BEAMS

SCALE 1 : 100

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PROJECT: KAH NEW ZEALAND LTD
CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT

TITLE: FIRST FLOOR BEAMS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
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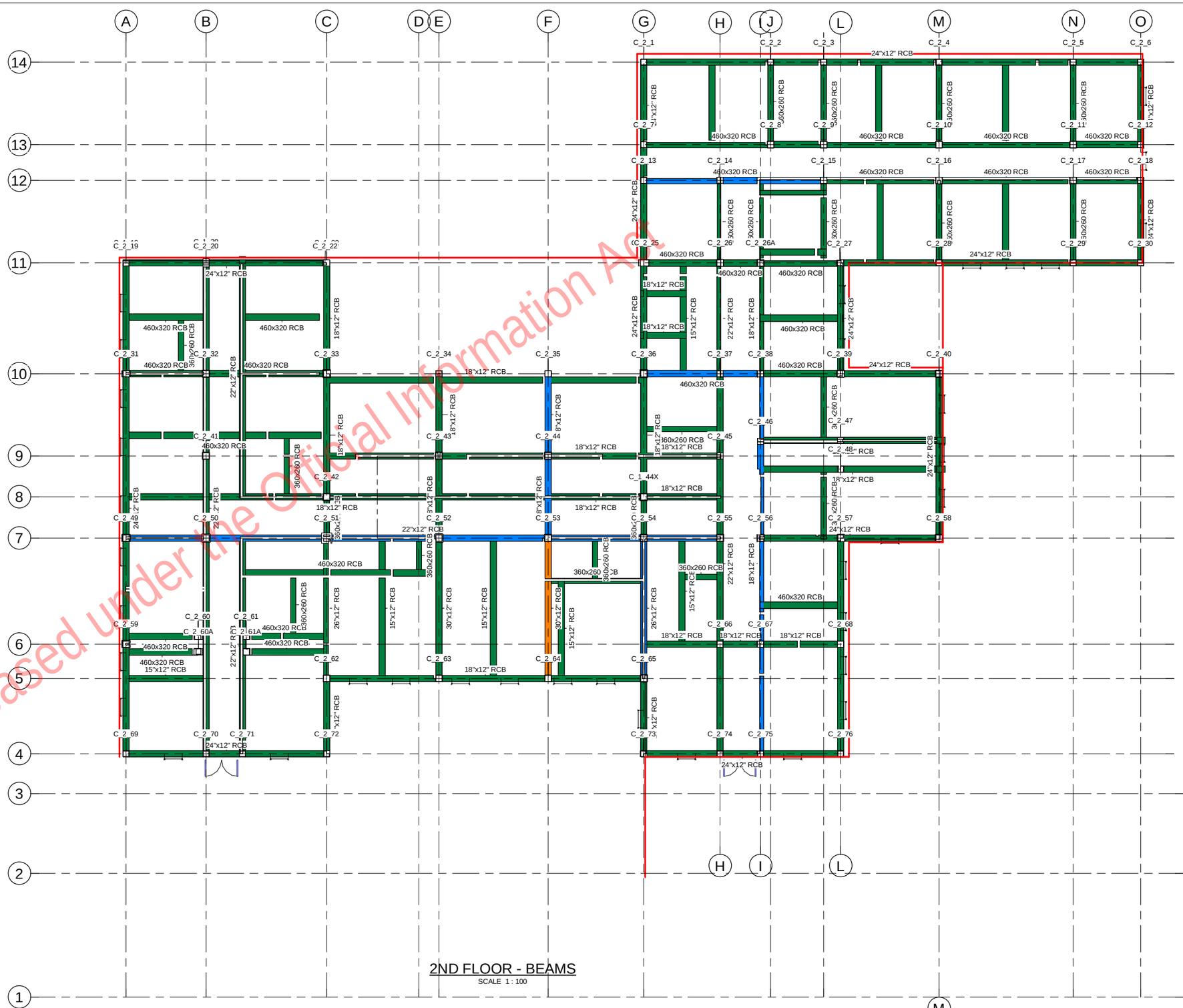

2ND FLOOR - BEAMS

SCALE 1 : 100

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CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT

TITLE: SECOND FLOOR BEAMS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
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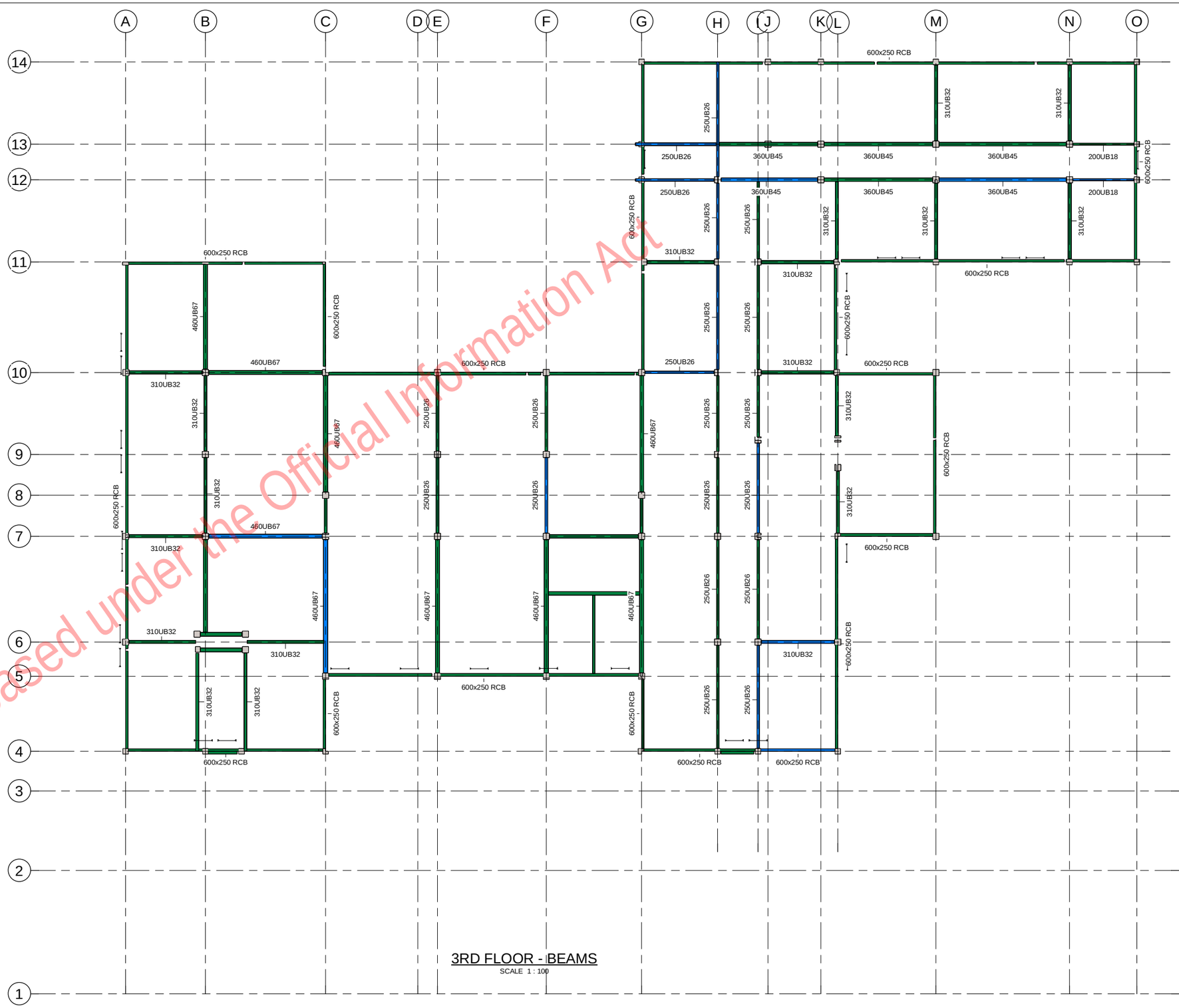

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3RD FLOOR - BEAMS

SCALE 1 : 100

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PROJECT
KAH NEW ZEALAND LTD
CHATEAU TONGARIRO
DETAILED SEISMIC ASSESSMENT

TITLE
THIRD FLOOR BEAMS

WSP PROJECT NO.	PROJ-ORIG-VOL-TYPE	SHEET NO.	REVISION
2-37905.01	237905-WSP-01-DR	S-1413	A

Appendix D
Assumptions

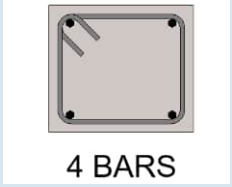
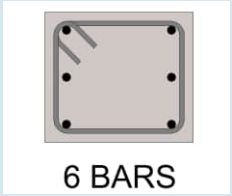
Due to the lack of as-built information of the building, it has been necessary to propose some assumptions. Assumptions are made based on information from available documentation, inspections observations and testing, and, where necessary, from typical construction practices of the era. We recommend that these assumptions are confirmed during strengthening works design once the structure becomes accessible for further investigations.

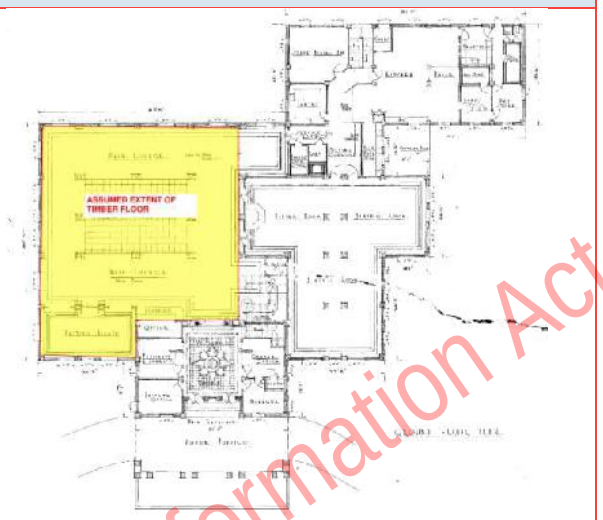
Column detailing

The following assumptions have been made for column detailing as we only have available a reinforcement schedule and some limited construction photos:

- Assumed column splice practice at that time, assuming 70 diameters occurring both above the two pours as indicated in historical photos and when there is a change of diameter in the column schedule.
- Lap splices of hooked plain bars assumed
- Development lengths of reinforcing are not known. It is assumed splices and reinforcing termination does not govern flexural capacity.
- It is assumed based on construction photos, that splices were formed above each floor and within the plastic hinge region if the diameter of the bars change from the column below to the column being assessed.
- We have assumed that the connections between the steel beams and the columns have adequate capacity to transfer design inertia forces and overturning demands without compromising gravity carrying capacity. Steel beam assessment for point loads of bracing was not carried out.
- Shear reinforcing is present for the length of the columns, but no stirrups are within the beam-column joint. The beam-column joint is therefore expected to be shear-governed.

Table 5: Column assessment assumptions

Item	Value	Reason
Cover to stirrups	25.4mm	Assumed
Spacing of stirrups and bar sizes	Varies	Based on column schedule from original construction drawings
Plain bars hooked	Yes	Assumed, based on typical practice of the era 
Number of legs per stirrup	2	Typical
Reinforcement inside and outside plastic hinge region is the same	Yes	Assumed
Bar layout 5 bars used as 4 7 bars used as 6	As shown	  Note: For square columns the main bending direction is assumed, and bars placed accordingly 
Splice in plastic region	Yes	Only above first floor and below second floor assumed or when there is a big change in bar numbers (
Ld provided for columns	70Diameters	Assumed

Item	Value	Reason
Extent of timber floor	As Shown	
Splice length crossed by two or more tie groups	Yes	Assumed
All columns at foundation assumed pinned		Small footprint for foundation

There are also some assumptions regarding the columns plan layout which is captured in the Revit drawings. These assumptions are summarised in the table below:

Table 6: Column layout assumptions

Item (col #)	Reason/Assumed information
23	None above 1st floor
24	None above 1st floor
42	Column from 1st floor up
47	Column from 1st floor up
48	Column from 1st floor up
Column data assumed based on other columns:	
Col 1 to 6 below GF	Same as columns between GF and 1st
Col 9	Same as Col 8
Col 13	Same as Col 7
Col 19, 20, 22, 23,24, 31, 32, 33, 34, 38, 41, 43 below GF	Same as column between GF and 1st

Reinforced concrete Beams

Analysis is based on detailing of two beams that are available in the original drawings to estimate rotation capacities, splice failure and beam-column joint behaviour. Limited drawing information regarding beam reinforcement is available, and only limited information has been gained through our scanning.

Steel beams

Steel beam layout about the third floor is not fully known. Assumptions have been made in terms of the layout and the span direction of bearers.

We have assumed that the connection between the steel beams and the reinforced concrete beams have sufficient capacity since they are unknown at this point in time. Sizes adopted are current UB sections closest to the measured sizes on site.

At the first floor, it has been assumed that steel beams have been provided to the original roof above the main lobby area. Thus, we have interpreted that all internal beams at the first floor are concrete.

Roof level structure

We have assumed that all the walls provide bracing capacity and therefore that they meet the minimum detailing provisions set out in the guidelines for the adopted strength. No intrusive investigations have been carried out to determine the presence of hold-down bolts, etc.

Masonry Walls and timber walls

The following assumptions have been made for the masonry walls.

Table 7: Wall Assumptions

Item	Value	Reason
Masonry Wall elastic modulus	As per Eq C8.4	Reduce effect of walls with current assessment
Timber walls above third floor	Capacity of 2kN/m	Gypsum plasterboard at both sides with fixings at 300 crs.

Internal Stairs

Reinforcing details of stairs are missing. However, we will assume that gravity only reinforcement is available for the flights and landings as per standard practices of the era.

External Stairs

Reinforcement unknown, shallow foundations assumed. assessment is only limited to global overturning stability and probable out-of-plane failure of walls that are assumed to be cavity and of similar construction as per the original 1929 building.

Assessment procedure assumptions

Analytical and assessment assumptions also include:

1. Given the gaps in the information we have assumed that a column sway will occur. This might not be the case but will set up an upper bound for the rating of the bare frames.
2. Bare concrete frames SLaMa have been carried and contribution of the infills added separately as per recommendations by Gentile, R., et.al, “Non-linear analysis of RC masonry-infilled frames using the SLaMA method: part 1—mechanical interpretation of the infill/frame interaction and formulation of the procedure”, Bulletin of Earthquake Engineering, 2019. Interaction between infill and frames has been done only by hand calculations as per chapter C7.
3. For the exterior infills, only one wythe is in contact with the frames, while the exterior is tied to the interior and the ties are strong enough. Therefore, out-of-plane actions are resisted by the internal wythe only.
4. Some infill walls have a ratio of opening vs total area greater than 20%. These have been calculated according to chapter C8.

Appendix E
ICOMOS New Zealand Charter for the Conserv
of Places of Cultural Heritage Value (2010)

ICOMOS New Zealand Charter

for the Conservation of Places of Cultural Heritage Value

Revised 2010

Preamble

New Zealand retains a unique assemblage of **places** of **cultural heritage value** relating to its indigenous and more recent peoples. These areas, **cultural landscapes** and features, buildings and **structures**, gardens, archaeological sites, traditional sites, monuments, and sacred **places** are treasures of distinctive value that have accrued meanings over time. New Zealand shares a general responsibility with the rest of humanity to safeguard its cultural heritage **places** for present and future generations. More specifically, the people of New Zealand have particular ways of perceiving, relating to, and conserving their cultural heritage **places**.

Following the spirit of the International Charter for the Conservation and Restoration of Monuments and Sites (the Venice Charter - 1964), this charter sets out principles to guide the **conservation** of **places** of **cultural heritage value** in New Zealand. It is a statement of professional principles for members of ICOMOS New Zealand.

This charter is also intended to guide all those involved in the various aspects of **conservation** work, including owners, guardians, managers, developers, planners, architects, engineers, craftspeople and those in the construction trades, heritage practitioners and advisors, and local and central government authorities. It offers guidance for communities, organisations, and individuals involved with the **conservation** and management of cultural heritage **places**.

This charter should be made an integral part of statutory or regulatory heritage management policies or plans, and should provide support for decision makers in statutory or regulatory processes.

Each article of this charter must be read in the light of all the others. Words in bold in the text are defined in the definitions section of this charter.

This revised charter was adopted by the New Zealand National Committee of the International Council on Monuments and Sites at its meeting on 4 September 2010.

Purpose of conservation

1. The purpose of conservation

The purpose of **conservation** is to care for **places** of **cultural heritage value**.

In general, such **places**:

(i) have lasting values and can be appreciated in their own right;
(ii) inform us about the past and the cultures of those who came before us;
(iii) provide tangible evidence of the continuity between past, present, and future;
(iv) underpin and reinforce community identity and relationships to ancestors and the land; and
(v) provide a measure against which the achievements of the present can be compared.

It is the purpose of **conservation** to retain and reveal such values, and to support the ongoing meanings and functions of **places** of **cultural heritage value**, in the interests of present and future generations.

Conservation principles

2. Understanding cultural heritage value

Conservation of a **place** should be based on an understanding and appreciation of all aspects of its **cultural heritage value**, both **tangible** and **intangible**. All available forms of knowledge and evidence provide the means of understanding a **place** and its **cultural heritage value** and cultural **heritage significance**. **Cultural heritage value** should be understood through consultation with **connected people**, systematic documentary and oral research, physical investigation and **recording** of the **place**, and other relevant methods.

All relevant **cultural heritage values** should be recognised, respected, and, where appropriate, revealed, including values which differ, conflict, or compete.

The policy for managing all aspects of a **place**, including its **conservation** and its **use**, and the implementation of the policy, must be based on an understanding of its **cultural heritage value**.

3. Indigenous cultural heritage

The indigenous cultural heritage of **tangata whenua** relates to **whanau**, **hapu**, and **iwi** groups. It shapes identity and enhances well-being, and it has particular cultural meanings and values for the present, and associations with those who have gone before. Indigenous cultural heritage brings with it responsibilities of guardianship and the practical application and passing on of associated knowledge, traditional skills, and practices.

The Treaty of Waitangi is the founding document of our nation. Article 2 of the Treaty recognises and guarantees the protection of **tino rangatiratanga**, and so empowers **kaitiakitanga** as customary trusteeship to be exercised by **tangata whenua**. This customary trusteeship is exercised over their **taonga**, such as sacred and traditional **places**, built heritage, traditional practices, and other cultural heritage resources. This obligation extends beyond current legal ownership wherever such cultural heritage exists.

Particular **matauranga**, or knowledge of cultural heritage meaning, value, and practice, is associated with **places**. **Matauranga** is sustained and transmitted through oral, written, and physical forms determined by **tangata whenua**. The **conservation** of such **places** is therefore conditional on decisions made in associated **tangata whenua** communities, and should proceed only in this context. In particular, protocols of access, authority, ritual, and practice are determined at a local level and should be respected.

4. Planning for conservation

Conservation should be subject to prior documented assessment and planning.

All **conservation** work should be based on a **conservation plan** which identifies the **cultural heritage value** and **cultural heritage significance** of the **place**, the **conservation** policies, and the extent of the recommended works.

The **conservation plan** should give the highest priority to the **authenticity** and **integrity** of the **place**.

Other guiding documents such as, but not limited to, management plans, cyclical **maintenance** plans, specifications for **conservation** work, interpretation plans, risk mitigation plans, or emergency plans should be guided by a **conservation plan**.

5. Respect for surviving evidence and knowledge

Conservation maintains and reveals the **authenticity** and **integrity** of a **place**, and involves the least possible loss of **fabric** or evidence of **cultural heritage value**. Respect for all forms of knowledge and existing evidence, of both **tangible** and **intangible values,** is essential to the **authenticity** and **integrity** of the **place**.

Conservation recognises the evidence of time and the contributions of all periods. The **conservation** of a **place** should identify and respect all aspects of its **cultural heritage value** without unwarranted emphasis on any one value at the expense of others.

The removal or obscuring of any physical evidence of any period or activity should be minimised, and should be explicitly justified where it does occur. The **fabric** of a particular period or activity may be obscured or removed if assessment shows that its removal would not diminish the **cultural heritage value** of the **place.**

In **conservation**, evidence of the functions and intangible meanings of **places** of **cultural heritage value** should be respected.

6. Minimum intervention

Work undertaken at a **place** of **cultural heritage value** should involve the least degree of **intervention** consistent with **conservation** and the principles of this charter.

Intervention should be the minimum necessary to ensure the retention of **tangible** and **intangible values** and the continuation of **uses** integral to those values. The removal of **fabric** or the alteration of features and spaces that have **cultural heritage value** should be avoided.

7. Physical investigation

Physical investigation of a **place** provides primary evidence that cannot be gained from any other source. Physical investigation should be carried out according to currently accepted professional standards, and should be documented through systematic **recording**.

Invasive investigation of **fabric** of any period should be carried out only where knowledge may be significantly extended, or where it is necessary to establish the existence of **fabric** of **cultural heritage value**, or where it is necessary for **conservation** work, or where such **fabric** is about to be damaged or destroyed or made inaccessible. The extent of invasive investigation should minimise the disturbance of significant **fabric**.

8. Use

The **conservation** of a **place** of **cultural heritage value** is usually facilitated by the **place** serving a useful purpose.

Where the **use** of a **place** is integral to its **cultural heritage value**, that **use** should be retained.

Where a change of **use** is proposed, the new **use** should be compatible with the **cultural heritage value** of the **place**, and should have little or no adverse effect on the **cultural heritage value**.

9. Setting

Where the **setting** of a **place** is integral to its **cultural heritage value**, that **setting** should be conserved with the **place** itself. If the **setting** no longer contributes to the **cultural heritage value** of the **place**, and if **reconstruction** of the **setting** can be justified, any **reconstruction** of the **setting** should be based on an understanding of all aspects of the **cultural heritage value** of the **place**.

10. Relocation

The on-going association of a **structure** or feature of **cultural heritage value** with its location, site, curtilage, and **setting** is essential to its **authenticity** and **integrity**. Therefore, a **structure** or feature of **cultural heritage value** should remain on its original site.

Relocation of a **structure** or feature of **cultural heritage value**, where its removal is required in order to clear its site for a different purpose or construction, or where its removal is required to enable its **use** on a different site, is not a desirable outcome and is not a **conservation** process.

In exceptional circumstances, a **structure** of **cultural heritage value** may be relocated if its current site is in imminent danger, and if all other means of retaining the **structure** in its current location have been exhausted. In this event, the new location should provide a **setting** compatible with the **cultural heritage value** of the **structure**.

11. Documentation and archiving

The **cultural heritage value** and **cultural heritage significance** of a **place**, and all aspects of its **conservation**, should be fully documented to ensure that this information is available to present and future generations.

Documentation includes information about all changes to the **place** and any decisions made during the **conservation** process.

Documentation should be carried out to archival standards to maximise the longevity of the record, and should be placed in an appropriate archival repository.

Documentation should be made available to **connected people** and other interested parties. Where reasons for confidentiality exist, such as security, privacy, or cultural appropriateness, some information may not always be publicly accessible.

12. Recording

Evidence provided by the **fabric** of a **place** should be identified and understood through systematic research, **recording**, and analysis.

Recording is an essential part of the physical investigation of a **place**. It informs and guides the **conservation** process and its planning. Systematic **recording** should occur prior to, during, and following any **intervention**. It should include the **recording** of new evidence revealed, and any **fabric** obscured or removed.

Recording of the changes to a **place** should continue throughout its life.

13. Fixtures, fittings, and contents

Fixtures, fittings, and **contents** that are integral to the **cultural heritage value** of a **place** should be retained and conserved with the **place**. Such fixtures, fittings, and **contents** may include carving, painting, weaving, stained glass, wallpaper, surface decoration, works of art, equipment and machinery, furniture, and personal belongings.

Conservation of any such material should involve specialist **conservation** expertise appropriate to the material. Where it is necessary to remove any such material, it should be recorded, retained, and protected, until such time as it can be reinstated.

Conservation processes and practice

14. Conservation plans

A **conservation plan**, based on the principles of this charter, should:

(i) be based on a comprehensive understanding of the **cultural heritage value** of the **place** and assessment of its **cultural heritage significance**;

(ii) include an assessment of the **fabric** of the **place**, and its condition;

(iii) give the highest priority to the **authenticity** and **integrity** of the **place**;

(iv) include the entirety of the **place**, including the **setting**;

(v) be prepared by objective professionals in appropriate disciplines;

(vi) consider the needs, abilities, and resources of **connected people**;

(vii) not be influenced by prior expectations of change or development;

(viii) specify **conservation** policies to guide decision making and to guide any work to be undertaken;

(ix) make recommendations for the **conservation** of the **place;** and

(x) be regularly revised and kept up to date.

15. Conservation projects

Conservation projects should include the following:

(i) consultation with interested parties and **connected people**, continuing throughout the project;

(ii) opportunities for interested parties and **connected people** to contribute to and participate in the project;

(iii) research into documentary and oral history, using all relevant sources and repositories of knowledge;

(iv) physical investigation of the **place** as appropriate;

(v) use of all appropriate methods of **recording**, such as written, drawn, and photographic;

(vi) the preparation of a **conservation plan** which meets the principles of this charter;

(vii) guidance on appropriate **use** of the **place**;

(viii) the implementation of any planned **conservation** work;

(ix) the **documentation** of the **conservation** work as it proceeds; and

(x) where appropriate, the deposit of all records in an archival repository.

A **conservation** project must not be commenced until any required statutory authorisation has been granted.

16. Professional, trade, and craft skills

All aspects of **conservation** work should be planned, directed, supervised, and undertaken by people with appropriate **conservation** training and experience directly relevant to the project.

All **conservation** disciplines, arts, crafts, trades, and traditional skills and practices that are relevant to the project should be applied and promoted.

17. Degrees of intervention for conservation purposes

Following research, **recording**, assessment, and planning, **intervention** for **conservation** purposes may include, in increasing degrees of **intervention**:

(i) **preservation**, through **stabilisation, maintenance**, or **repair**;
(ii) **restoration**, through **reassembly**, **reinstatement**, or removal;
(iii) **reconstruction**; and
(iv) **adaptation**.

In many **conservation** projects a range of processes may be utilised. Where appropriate, **conservation** processes may be applied to individual parts or components of a **place** of **cultural heritage value**.

The extent of any **intervention** for **conservation** purposes should be guided by the **cultural heritage value** of a **place** and the policies for its management as identified in a **conservation plan**. Any **intervention** which would reduce or compromise **cultural heritage value** is undesirable and should not occur.

Preference should be given to the least degree of **intervention**, consistent with this charter.

Re-creation, meaning the conjectural **reconstruction** of a **structure** or **place**; replication, meaning to make a copy of an existing or former **structure** or **place**; or the construction of generalised representations of typical features or **structures**, are not **conservation** processes and are outside the scope of this charter.

18. Preservation

Preservation of a **place** involves as little **intervention** as possible, to ensure its long-term survival and the continuation of its **cultural heritage value**.

Preservation processes should not obscure or remove the patina of age, particularly where it contributes to the **authenticity** and **integrity** of the **place,** or where it contributes to the structural stability of materials.

i. Stabilisation

Processes of decay should be slowed by providing treatment or support.

ii. Maintenance

A **place** of **cultural heritage value** should be maintained regularly. **Maintenance** should be carried out according to a plan or work programme.

iii. Repair

Repair of a **place** of **cultural heritage value** should utilise matching or similar materials. Where it is necessary to employ new materials, they should be distinguishable by experts, and should be documented.

Traditional methods and materials should be given preference in **conservation** work.

Repair of a technically higher standard than that achieved with the existing materials or construction practices may be justified only where the stability or life expectancy of the site or material is increased, where the new material is compatible with the old, and where the **cultural heritage value** is not diminished.

19. Restoration

The process of **restoration** typically involves **reassembly** and **reinstatement,** and may involve the removal of accretions that detract from the **cultural heritage value** of a **place**.

Restoration is based on respect for existing **fabric,** and on the identification and analysis of all available evidence, so that the **cultural heritage value** of a **place** is recovered or revealed. **Restoration** should be carried out only if the **cultural heritage value** of the **place** is recovered or revealed by the process.

Restoration does not involve conjecture.

i. Reassembly and reinstatement

Reassembly uses existing material and, through the process of **reinstatement**, returns it to its former position. **Reassembly** is more likely to involve work on part of a **place** rather than the whole **place**.

ii. Removal

Occasionally, existing **fabric** may need to be permanently removed from a **place**. This may be for reasons of advanced decay, or loss of structural **integrity**, or because particular **fabric** has been identified in a **conservation plan** as detracting from the **cultural heritage value** of the **place**.

The **fabric** removed should be systematically **recorded** before and during its removal. In some cases it may be appropriate to store, on a long-term basis, material of evidential value that has been removed.

20. Reconstruction

Reconstruction is distinguished from **restoration** by the introduction of new material to replace material that has been lost.

Reconstruction is appropriate if it is essential to the function, **integrity**, **intangible value**, or understanding of a **place**, if sufficient physical and documentary evidence exists to minimise conjecture, and if surviving **cultural heritage value** is preserved.

Reconstructed elements should not usually constitute the majority of a **place** or **structure**.

21. Adaptation

The **conservation** of a **place** of **cultural heritage value** is usually facilitated by the **place** serving a useful purpose. Proposals for **adaptation** of a **place** may arise from maintaining its continuing **use**, or from a proposed change of **use**.

Alterations and additions may be acceptable where they are necessary for a **compatible use** of the **place**. Any change should be the minimum necessary, should be substantially reversible, and should have little or no adverse effect on the **cultural heritage value** of the **place**.

Any alterations or additions should be compatible with the original form and **fabric** of the **place,** and should avoid inappropriate or incompatible contrasts of form, scale, mass, colour, and material. **Adaptation** should not dominate or substantially obscure the original form and **fabric**, and should not adversely affect the **setting** of a **place** of **cultural heritage value**. New work should complement the original form and **fabric**.

22. Non-intervention

In some circumstances, assessment of the **cultural heritage value** of a **place** may show that it is not desirable to undertake any **conservation intervention** at that time. This approach may be appropriate where undisturbed constancy of **intangible values,** such as the spiritual associations of a sacred **place,** may be more important than its physical attributes.

23. Interpretation

Interpretation actively enhances public understanding of all aspects of **places** of **cultural heritage value** and their **conservation**. Relevant cultural protocols are integral to that understanding, and should be identified and observed.

Where appropriate, interpretation should assist the understanding of **tangible** and **intangible values** of a **place** which may not be readily perceived, such as the sequence of construction and change, and the meanings and associations of the **place** for **connected people**.

Any interpretation should respect the **cultural heritage value** of a **place**. Interpretation methods should be appropriate to the **place**. Physical **interventions** for interpretation purposes should not detract from the experience of the **place,** and should not have an adverse effect on its **tangible** or **intangible values**.

24. Risk mitigation

Places of **cultural heritage value** may be vulnerable to natural disasters such as flood, storm, or earthquake; or to humanly induced threats and risks such as those arising from earthworks, subdivision and development, buildings works, or wilful damage or neglect. In order to safeguard **cultural heritage value**, planning for risk mitigation and emergency management is necessary.

Potential risks to any **place** of **cultural heritage value** should be assessed. Where appropriate, a risk mitigation plan, an emergency plan, and/or a protection plan should be prepared, and implemented as far as possible, with reference to a conservation plan.

Definitions

For the purposes of this charter:

Adaptation means the process(es) of modifying a **place** for a **compatible use** while retaining its **cultural heritage value. Adaptation** processes include alteration and addition.

Authenticity means the credibility or truthfulness of the surviving evidence and knowledge of the **cultural heritage value** of a **place.** Relevant evidence includes form and design, substance and **fabric**, technology and craftsmanship, location and surroundings, context and **setting**, **use** and function, traditions, spiritual essence, and sense of place, and includes **tangible** and **intangible values.** Assessment of **authenticity** is based on identification and analysis of relevant evidence and knowledge, and respect for its cultural context.

Compatible use means a **use** which is consistent with the **cultural heritage value** of a **place,** and which has little or no adverse impact on its **authenticity** and **integrity.**

Connected people means any groups, organisations, or individuals having a sense of association with or responsibility for a **place** of **cultural heritage value.**

Conservation means all the processes of understanding and caring for a **place** so as to safeguard its **cultural heritage value**. **Conservation** is based on respect for the existing **fabric**, associations, meanings, and **use** of the **place**. It requires a cautious approach of doing as much work as necessary but as little as possible, and retaining **authenticity** and **integrity**, to ensure that the **place** and its values are passed on to future generations.

Conservation plan means an objective report which documents the history, **fabric,** and **cultural heritage value** of a **place**, assesses its **cultural heritage significance,** describes the condition of the **place**, outlines **conservation** policies for managing the **place**, and makes recommendations for the **conservation** of the **place**.

Contents means moveable objects, collections, chattels, documents, works of art, and ephemera that are not fixed or fitted to a **place,** and which have been assessed as being integral to its **cultural heritage value**.

Cultural heritage significance means the **cultural heritage value** of a **place** relative to other similar or comparable **places**, recognising the particular cultural context of the **place**.

Cultural heritage value/s means possessing aesthetic, archaeological, architectural, commemorative, functional, historical, landscape, monumental, scientific, social, spiritual, symbolic, technological, traditional, or other **tangible** or **intangible values,** associated with human activity.

Cultural landscapes means an area possessing **cultural heritage value** arising from the relationships between people and the environment. **Cultural landscapes** may have been designed, such as gardens, or may have evolved from human settlement and land use over time, resulting in a diversity of distinctive landscapes in different areas. Associative **cultural landscapes**, such as sacred mountains, may lack **tangible** cultural elements but may have strong **intangible** cultural or spiritual associations.

Documentation means collecting, **recording**, keeping, and managing information about a **place** and its **cultural heritage value**, including information about its history, **fabric,** and meaning; information about decisions taken; and information about physical changes and **interventions** made to the **place**.

Fabric means all the physical material of a **place**, including subsurface material, **structures**, and interior and exterior surfaces including the patina of age; and including fixtures and fittings, and gardens and plantings.

Hapu means a section of a large tribe of the **tangata whenua**.

Intangible value means the abstract **cultural heritage value** of the meanings or associations of a **place**, including commemorative, historical, social, spiritual, symbolic, or traditional values.

Integrity means the wholeness or intactness of a **place**, including its meaning and sense of **place**, and all the **tangible** and **intangible** attributes and elements necessary to express its **cultural heritage value**.

Intervention means any activity that causes disturbance of or alteration to a **place** or its **fabric**. **Intervention** includes archaeological excavation, invasive investigation of built **structures,** and any **intervention** for **conservation** purposes.

Iwi means a tribe of the **tangata whenua**.

Kaitiakitanga means the duty of customary trusteeship, stewardship, guardianship, and protection of land, resources, or **taonga.**

Maintenance means regular and on-going protective care of a **place** to prevent deterioration and to retain its **cultural heritage value**.

Matauranga means traditional or cultural knowledge of the **tangata whenua**.

Non-intervention means to choose not to undertake any activity that causes disturbance of or alteration to a **place** or its **fabric**.

Place means any land having **cultural heritage value** in New Zealand, including areas; **cultural landscapes**; buildings, **structures,** and monuments; groups of buildings, **structures,** or monuments; gardens and plantings; archaeological sites and features; traditional sites; sacred **places**; townscapes and streetscapes; and settlements. **Place** may also include land covered by water, and any body of water. **Place** includes the **setting** of any such **place**.

Preservation means to maintain a **place** with as little change as possible.

Reassembly means to put existing but disarticulated parts of a **structure** back together.

Reconstruction means to build again as closely as possible to a documented earlier form, using new materials.

Recording means the process of capturing information and creating an archival record of the **fabric** and **setting** of a **place**, including its configuration, condition, **use**, and change over time.

Reinstatement means to put material components of a **place**, including the products of **reassembly**, back in position.

Repair means to make good decayed or damaged **fabric** using identical, closely similar, or otherwise appropriate material.

Restoration means to return a **place** to a known earlier form, by **reassembly** and **reinstatement**, and/or by removal of elements that detract from its **cultural heritage value**.

Setting means the area around and/or adjacent to a **place** of **cultural heritage value** that is integral to its function, meaning, and relationships. **Setting** includes the **structures**, outbuildings, features, gardens, curtilage, airspace, and accessways forming the spatial context of the **place** or used

in association with the **place**. **Setting** also includes **cultural landscapes**, townscapes, and streetscapes; perspectives, views, and viewshafts to and from a **place**; and relationships with other **places** which contribute to the **cultural heritage value** of the **place**. **Setting** may extend beyond the area defined by legal title, and may include a buffer zone necessary for the long-term protection of the **cultural heritage value** of the **place**.

Stabilisation means the arrest or slowing of the processes of decay.

Structure means any building, standing remains, equipment, device, or other facility made by people and which is fixed to the land.

Tangata whenua means generally the original indigenous inhabitants of the land; and means specifically the people exercising **kaitiakitanga** over particular land, resources, or **taonga**.

Tangible value means the physically observable **cultural heritage value** of a **place**, including archaeological, architectural, landscape, monumental, scientific, or technological values.

Taonga means anything highly prized for its cultural, economic, historical, spiritual, or traditional value, including land and natural and cultural resources.

Tino rangatiratanga means the exercise of full chieftainship, authority, and responsibility.

Use means the functions of a **place**, and the activities and practices that may occur at the **place**. The functions, activities, and practices may in themselves be of **cultural heritage value**.

Whanau means an extended family which is part of a **hapu** or **iwi**.

ISBN 978-0-473-17116-2 (PDF)

English language text first published 1993
Bilingual text first published 1995

This revised text replaces the 1993 and 1995 versions and should be referenced as the *ICOMOS New Zealand Charter for the Conservation of Places of Cultural Heritage Value* (ICOMOS New Zealand Charter 2010).

This revision incorporates changes in conservation philosophy and best practice since 1993 and is the only version of the ICOMOS New Zealand Charter approved by ICOMOS New Zealand (Inc.) for use.

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