

STRUCTURAL DESIGN FEATURES REPORT

FOR



PROPOSED SEISMIC RETROFIT DETAILED DESIGN STAGE 2



ROTORUA museum

Te Whare Taonga o Te Arawa ART | CULTURE | HERITAGE

OUR REF: J000949

MARCH 2019

Structural Design Feature Report

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Detail Design Stage 2-Foundation Design Rotorua Museum, Oruawhata Drive, Rotorua

Status	For consent
Revision	0

Quality Assurance Statement		
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Date: March 2020

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Specialising in Civil, Structural, Environmental & Training

PROJECT DETAILS

CLIENT:	ROTORUA LAKES COUNCIL
SITE LOCATION:	GOVERNMENT GARDENS ORUWHATA DRIVE ROTORUA 3046
DESCRIPTION:	SEISMIC STRENGTHENING OF ROTORUA MUSEUM BUILDING DETAILED DESIGN - STAGE 2 – FOUNDATION DESIGN
PROJECT NUMBER:	J000949

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SUMMARY REPORT OBJECTIVES

The Structural Design Feature Report (DFR) is a detailed document defining a set of design criteria by recording key assumptions, methodology and decisions. It summarizes the specific site geological parameters, design loadings, structural modelling assumptions, foundation requirements and material properties to be used in the structural design. The DFR also outlines the design standards followed in the analysis and design procedures including the adopted principles and methodologies behind the developed structural design.

At the detailed design stage, the DFR includes structural element descriptions and methodology to construct the design structural members and assemblies and install within the existing structure. In accordance with Construction Industry Council guidelines, detailed design drawing set is provided to describe utilized structural systems and their location within the building.

1. PROJECT INFORMATION

a. SCOPE

The scope of the construction project is in accordance with the Architectural Detailed Design drawing set - Stage 2, prepared by DPA Architects Ltd.

In general, the scope of work is:

- Detailed structural Design and Documentation for seismic strengthening of Museum Building in accordance with New Zealand Building Code and Heritage New Zealand Recommendations – Stage 2 – Foundation Design.

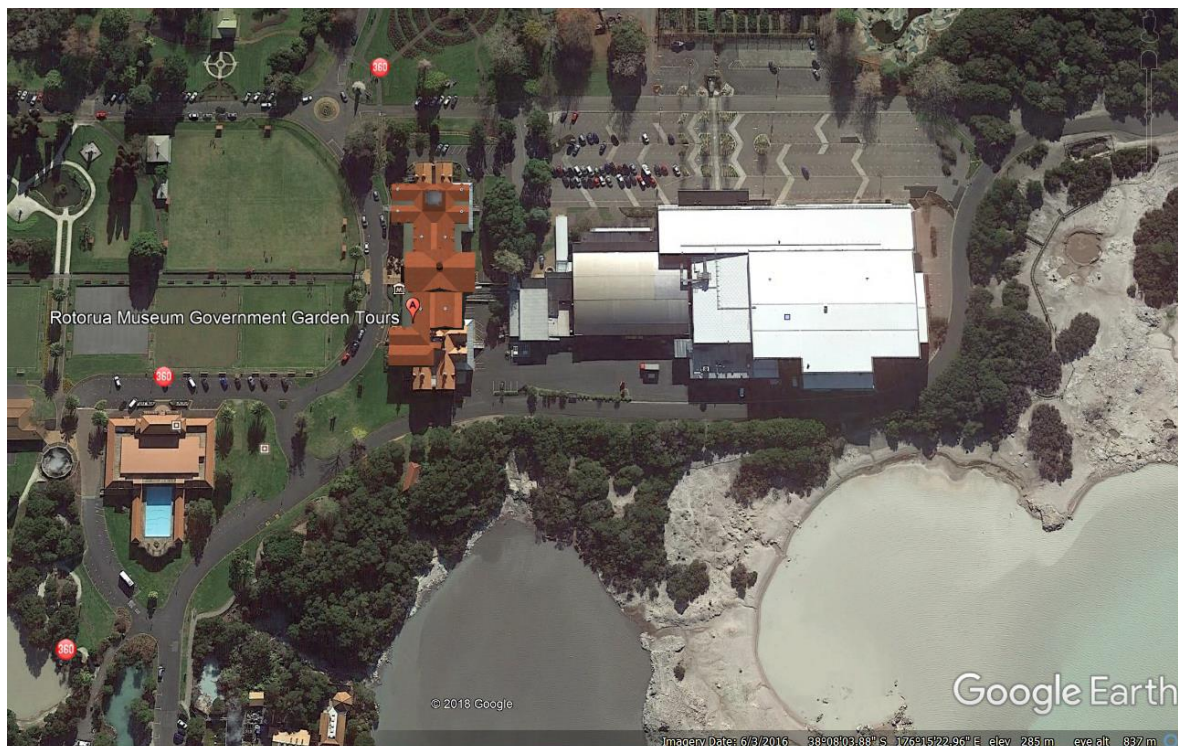
The following parts of the building and the construction method are included in Stage 2:

- Construction method for underpinning of the existing pumice concrete partition walls
- New strip footings / ground beams under proposed steel brace assemblies
- Reinforced concrete ground tie beams
- Raft / mat foundation over Atrium area

b. PROJECT LOCATION

The project site location is Rotorua Museum. The current legal description is Lot 2 SECT 2Blk I Tarawera SD Latitude 38 degrees 8'8"S, Longitude 176 degrees 15'32"E.

Figure 1 - Aerial Photo of Site Location



c. **DESIGN LIFE**

The standard building structure is designed for a minimum design life of 50 years as recommended by the New Zealand Building Code.

d. **IMPORTANCE LEVEL**

The proposed structure has been designed as an Importance level 3 (IL3) structure in accordance with NZS1170.0:2002 A2. IL3 refers to "Structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds".

e. **DESCRIPTION OF STRUCTURE**

The intended use of the building as a museum with large numbers of public visiting the exhibition, educational facility and function facility.

A specific description of the building sections is presented below:

Table 1-1 – Building Description	
Intermediate South Wing (Tarawera Gallery) – Zone 2	
Building	Description
Number of Storeys	Two (Basement + ground floor level)
Approximate Gross Floor Area(m ²)	Ground Floor – 280 m ²
Construction Period	Original construction circa 1906.
Current Use	Museum (Tarawera Gallery)
Number of uses per year	140,000
Importance Level (IL)	IL3
Heritage Status	Historic Place Category 1 (No 141)
Structural Alterations *note, many isolated activities and alterations have taken place, this is a brief overview of significant changes	From our site investigations, available as built drawings and research we established the following significant changes: - In 2010 there were major alterations to this area. The existing timber scissor roof trusses and external walls were left in place and all internal structure above the ground level floor was removed. Two steel trusses were constructed in the middle of the building to support the roof trusses. Between these truss bottom chords; a horizontal steel truss was created.

	Steel frames with Reid bar bracing were installed at the external walls and junctions between the Atrium and new South Wing. These steel frames go through the floor down to ground level and are supported on new reinforced concrete foundation pads (note, at the location they penetrate through the floor an oversized penetration was made, approximately 100mm each side of steel column, this gap is filled with polystyrene. However, the additional 100mm thick concrete topping placed on top of the ground floor, has been cast around the steel columns, effectively providing floor strut action between the columns. - Two new internal walls (approximately 6m high, constructed of 140x45 studs) run parallel to the new trusses. These are held down with hold down anchors at approximately 1m crs.
Structural Alterations (Cont.) *note, many isolated activities and alterations have taken place, this is a brief overview of significant changes	- They are lined with custom wood and glue and panel pin fixed (15 gauge). They are approximately 10m long each side. The connection at ceiling level has not been verified. - New timber framed walls were also installed next to the exterior and Atrium/South Wing steel work. This was done to hide the steel framing. Construction type appears to be 140xx45 framing at 600crs lined with Gib board lining. - The existing trusses had 140x45 extensions nailed to them (on bottom chord) and then 45x20 ceiling battens attached, then 17mm plywood and a 13mm fibrous plasterboard overlay. The ceiling was not constructed as a diaphragm. - A new 100mm overlay with 665mesh was installed throughout the Tarawera Gallery in 1967.
Basement	Unreinforced pumice concrete columns and foundation walls supporting 1906 construction Reinforced concrete pad foundations supporting new steel structure (2010)
Gravity Load Resisting System	Gravity Load Resisting System Comprises: - Heavy ceramic tile roofing supported on timber sarking which spans to timber scissor trusses - The trusses span to the external original walls and internally to two new steel trusses - Original 1906 external timber frame walls to unreinforced concrete nib walls, through to external unreinforced concrete foundation walls. The external timber frame walls consist of studs at 650-850mm.crs with mortise and tenon joints with a pumice infill concrete of approximately 50mm.

	- The timber trusses are supported by steel columns (part of bracing frame structure) which project to ground level and supported by reinforced concrete pads
Gravity Load Resisting System	- Internal unreinforced concrete arched/vaulted floor spans to unreinforced concrete columns then unreinforced pad footings.
Lateral Load Resisting System	In both principal directions, the seismic loads are primarily resisted by the steel framed structure which has internal Reid bar braced bays. Portal action will be developed on the central truss frames in the north-south direction.
Lateral Load Resisting System (Cont.)	In the north-south direction a secondary system of the 6m high partition walls (2, parallel to steel trusses) lined with MDF will also provide limited bracing capacity, even though this was not the design intent. The original 1906 external walls also provide bracing capacity as do the timber framed walls adjacent to the steel frames. Due to the differing stiffness's this creates a complex model for resisting earthquake forces. In the east-west direction there is a secondary system of timber framed internal walls adjacent to the steel frames. In the horizontal direction, the loads in the east-west direction are transferred by the horizontal steel truss, however this not at the same level as the ceiling so its effect is limited. Limited diaphragm action will be achieved by the ceilings lined in plywood due to the pitched nature of the scissor trusses. In the north-west direction, as mentioned above the ceiling will provide limited diaphragm action and at the roof plane level the sarking will provide diaphragm action to transfer lateral loads to the bracing lines. The suspended ground floor section is laterally resisted by the external and internal unreinforced concrete walls.
Roof System	The roof comprises: - Heavy ceramic tile roofing supported on timber sarking which spans to timber scissor trusses
Ground Floor System	The ground floor system is an unreinforced concrete slab arching in both directions which is supported on unreinforced concrete columns and walls with a 100mm overlay with 665 mesh installed in 1967.
Foundation System	Unreinforced concrete pads support columns. Perimeter internal and external concrete walls are supported by unreinforced strip footings.
Geotechnical Considerations	Soil Class D as per Cheal report

Adjacent Buildings	To the north of the building is the Atrium, this is integrally connected through the external walls and end truss of the two gable ends. At the south of the building is the new South Wing extension. There has been a 50mm seismic gap created internally throughout but there appears to be a connection still at the eastern interface (this was due to the original external walls being left in place in the construction of the new South Wing extension).
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Central Atrium Section – Zone 3	
Building	Description
Number of Storeys	Three (Basement, ground & mezzanine)
Approximate Gross Floor Area(m2)	Ground Floor – 345 m2 Mezzanine – 275 m2 (Excluding void area)
Construction Period	Original construction circa 1906.
Current Use	Museum
Number of uses per year	140,000
Importance Level (IL)	IL3
Heritage Status	Historic Place Category 1 (No 141)
Structural Alterations *note, many isolated activities and alterations have taken place, this is a brief overview of significant changes	From our site investigations, available drawings and research we established the following changes: - There is anecdotal evidence that the main turret was removed after the 1931 Napier Earthquake. It however can be seen that the turret has been strengthened at some time with the installation of 8 steel rods which are tensioned at each end and fixed into the supporting structure. - Installation of new 30mm reinforced concrete floor across Atrium (original floor was cut down and overlay installed). - Infilling of central bath to create level floor area.
Structural Alterations (Cont.) *note, many isolated activities and alterations have taken place, this is a brief overview of significant changes	- Installation of mezzanine dance floor circa 1964, then removal and reinstatement to match the original in 1998. - Addition of timber partition walls on the mezzanine floor at the east end, to form office space.

			- Application of spray on Renderroc concrete from basement level to increase overall thickness of slab in multiple slab locations.
Basement			Unreinforced pumice concrete columns and foundation walls supporting 1906 construction.
Gravity System	Load	Resisting	Gravity Load Resisting System Comprises: - Heavy ceramic tile roof supported on timber sarking supported on king post/hammerbeam timber trusses which span the width of the atrium (north-south direction). Trusses are supported on the north and south internal walls. Truss timber is Kauri. - North and south unreinforced concrete walls to truss corbel height (i.e. to the apex height of the adjacent north and south gallery gables). The remaining height of the wall up to the ceiling is constructed of timber framing with concrete infill panels. At basement level, these walls are supported by unreinforced concrete foundation walls. External timber frame walls on the east and west side to unreinforced concrete nib walls, through to external unreinforced concrete foundation walls. The external timber frame walls consist of 200x150mm Totara timber studs at 650mm to 850mm crs. with mortise and tenon joints, and a pumice infill concrete of approximately 50mm. - Timber truss frames at the east end of the roof are supported on the outer timber framed walls and their mid-supports are supported by internal walls below. - Steel beam and column structure support timber joist mezzanine floor. Floor joists seated and embedded into the north and south concrete side walls. - The central turret is constructed of 8 timber posts projecting from its truss/beam base support. Each timber post has a tensioned steel rod extending full height to tie the turret down to its supporting beams. - Internal unreinforced concrete arched/vaulted ground floor spans to unreinforced concrete columns then unreinforced pad footings.
Lateral Load Resisting System			Lateral Load Resisting System Comprises: - In the north-south direction the seismic loads are resisted by the in-plane action of the timber mortise and tenon jointed frames of the external walls which have diagonal cut in braces (compression only). The load is then transferred to the concrete nib walls and through into the

	foundation walls. Also in the north-south direction, seismic loads will be
Lateral Load Resisting System (Cont.)	- resisted by the out of plane action of the concrete side walls. - The building is integrally connected to the Intermediate North Wing and Intermediate South Wing. Therefore, some load transfer will occur at roof and ceiling level between these sections. - In the east-west direction the primary lateral load resisting elements are the unreinforced concrete walls. These walls transfer the loads down to ground level. In conjunction with this, there are sections of mortise and tenon jointed frames. These frames are overlaid internally with lath and plaster. - The main turret is braced by a series of tensioned rods - In the horizontal direction, there is a tongue and groove timber diaphragm to distribute lateral loads fixed to the exposed timber trusses at ceiling level. - Above the eastern side mezzanine floor where there are no trusses there are a series of diagonal timber beams to transfer lateral loads. At roof plane level, the sarking will provide diaphragm action to transfer lateral loads to the bracing lines. - The suspended ground floor section is laterally resisted by the external and internal unreinforced concrete walls. Connections include the following: - The timber trusses have pinned bolt and plate connections at member joints. The ends of the trusses are bolted into the supporting side walls. - The mezzanine floor joists are socketed into the north and south concrete walls, at an embedment depth and height of approx. 300mm. At the other end, the floor joists are connected to the supporting steel beams by bolted plates. - Sarking and roof trusses are connected using nail joints. - Rafters connected to truss intermediate beams with nails. - Timber wall framing connected by tenon and mortise joints. - Timber wall baseplate connected to concrete nib wall with cast in anchors at 900/1200mm spacing. - Corner wall timber stud connected to concrete wall by bolts at 1000mm spacing.

Roof System	The roof comprises: - Heavy ceramic tile roofing and 25mm thick Rimu sarking supported on timber rafters and trusses.
Floor System	The ground floor system is an unreinforced concrete slab arching in both directions which is supported on unreinforced concrete columns and walls. The mezzanine floor is timber joists supported by a beam and column arrangement.
Foundation System	Unreinforced concrete pads support columns. Perimeter internal and external concrete walls are supported by unreinforced strip footings.
Geotechnical Considerations	Soil Class D as per referenced Cheal report.
Adjacent Buildings	The building is integrally connected to the Intermediate North Wing and Intermediate South Wing (Tarawera Gallery). Therefore, load transfer will occur at roof and ceiling level between these sections during an earthquake event.

Intermediate North Wing – Zone 4			
Building	Description		
Number of Storeys	Partly Two (Basement & ground floor level) Partly Three (Basement, ground & 1 st floor)		
Approximate Gross Floor Area(m²)	Ground Floor – 282 m ² 1 st floor Classroom – 124 m ²		
Construction Period	Original construction circa 1906.		
Current Use	Museum/Cafeteria		
Number of uses per year	140,000		
Importance Level (IL)	IL3		
Heritage Status	Historic Place Category 1 (No 141)		

Structural Alterations *note, many isolated activities and alterations have taken place, this is a brief overview of significant changes	From our site investigations, available drawings, and research undertaken, we established the following significant changes: - Application of spray on Renderroc concrete from basement level to increase overall thickness of slab in multiple slab locations. - Installation of gravity timber frames in basement to support the unreinforced concrete floor. - Original public Rachel bath has been covered over with timber frame and ply/boards. This is situated at the south end of the café floor. - The west wall along the café has had plasterboard installed over the original wall's internal lining. Creation of café, 1st floor kitchen in the mid 1960's, then a third gallery in the 1980s, then demolition of the third gallery circa 1998 to create the current café and active theatre area. Steel cantilever posts (propped on floor) where installed to create the open café area, fixed down into new reinforced concrete foundations. - The creation of the "active theatre" on the eastern side. This consisted of constructing new concrete foundations, columns and floor to support the existing floor and structure above this room.
Basement	Unreinforced pumice concrete columns and foundation walls supporting 1906 construction
Gravity Load Resisting System	Gravity Load Resisting System Comprises: - Heavy ceramic tile roofing supported on timber sarking which spans to timber trusses. The trusses span approximately half the east-width direction to create a twin gable. - On the external edges of the building the trusses are supported on external timber frame walls consisting of 200x150mm Totara timber studs at 650mm to 850mm crs. - with mortise and tenon joints, and a pumice infill concrete of approximately 50mm. Internal wall lining is lath and plaster. These walls are supported by unreinforced concrete nib walls which are in turn supported by the unreinforced concrete foundation walls.

Gravity Load Resisting System (Cont.)	- Internally the trusses are supported by timber framed walls which are lined in lath and plaster which land on the unreinforced concrete floor with either columns and walls below to ground level. - Ceilings are supported on timber ceiling joists which span to internal & external walls. The ceilings have lath and plaster overlay. - As per the structural alterations section, the classroom/reception room above the active theatre is supported by a series of steel beams and columns. Beneath the active theatre there is a new reinforced concrete floor, columns and foundation pads. In the cafe area, there is a series of cantilever steel columns and steel beams which support the roof to ground level.
Lateral Load Resisting System	In the north-south direction the seismic loads are resisted by the timber mortise and tenon jointed frames of the external walls which have diagonal cut in braces (compression only). The load is then transferred to the concrete nib walls and through into the foundation walls. In addition to this there are the internal timber lath and plaster walls which transfer the load to the unreinforced concrete midfloor and then into the unreinforced foundation walls. At the café section, there are two parallel rows of four (4) cantilever steel columns which are propped at ground floor level by the unreinforced concrete slab. The building is integrally connected to the atrium and north wing so load transfer will occur at roof and ceiling level between these sections. In the east-west direction the primary lateral load resisting element is the unreinforced concrete walls (2) at the intersection of the atrium and north wing. In addition to this there are two (2) internal timber framed, lath and plaster walls (in the corridor between the active theatre and the toilets) which provide some lateral resistance. These loads are transferred to ground level through unreinforced foundation walls. In the horizontal direction at ceiling level there are lath and plaster ceilings which provide some diaphragm action. Under earthquake loading parallel to the roof gables the roof sarking provides a load path for lateral loads, however this has been severely degraded in some areas, due to timber moisture decay and advanced corrosion of nail connections.
Lateral Load Resisting System (Cont.)	The suspended ground floor section is laterally resisted by the external and internal unreinforced concrete walls. Connections include the following: - Roof end rafters are connected to the concrete gable walls with 12mm diameter bolts at 1000mm spacing. - Sarking and roof trusses are connected using nail joints. - Roof trusses are nailed to the outer walls top plate. - Timber framing connected by tenon and mortise joints.

	- Timber wall baseplate connected to concrete nib wall with cast in anchors at 1200mm spacing. - Corner wall timber stud connected to concrete wall by bolts at 1000mm spacing.
Roof System	The roof comprises: - Heavy ceramic tile roofing supported on 25mm thick Rimu sarking spanning between timber trusses. Sarking and rafters have experienced advanced moisture decay and corrosion of nail fixings (see Prendos report). - Timber ceiling joists support a lath and plaster ceiling.
Floor System	The ground floor system is an unreinforced concrete slab arching in both directions which is supported on unreinforced concrete columns and walls. Sprayed RenderRoc concrete overlay has been applied from the ground level in multiple areas to increase the overall thickness of the floor.
Foundation System	Concrete pads support intermediate columns and are not connected. Perimeter internal and external concrete walls are supported by strip footings.
Geotechnical Considerations	Soil Class D as per referenced Cheal report.
Adjacent Buildings	The building is integrally connected with the Atrium and North Wing.

North Wing – Zone 5			
Building	Description		
Number of Storeys	2.5 (Basement, ground, viewing platform)		
Approximate Gross Floor Area(m ²)	Ground Floor – 500 m ² Viewing Platform – 43 m ²		
Construction Period	Original construction circa 1906.		
Current Use	Museum		
Number of uses per year	140,000		
Importance Level (IL)	IL3		
Heritage Status	Historic Place Category 1 (No 141)		
Structural Alterations	From our site visits, available drawings and research we established the following significant changes - The western wing of the existing structure was made open plan for gallery space. Original internal load bearing walls were removed and replaced by the installation of two (2) parallel beam structures (assuming steel). This supports the roof trusses above. A horizontal steel truss was installed between the two beams. - Application of spray Renderroc concrete from basement level to increase overall thickness of slab in multiple slab locations		
Structural Alterations (Cont.)	- Installation of gravity timber frames in basement to support the unreinforced concrete floor. - Removal of original internal bath partition walls to create the Bathhouse Expedition Room. - Creation of viewing platform at top of the roof gable height. This is constructed in structural steel and has a midlevel floor above ceiling level.		
Basement	Unreinforced pumice concrete columns and foundation walls supporting 1906 construction		
Gravity System	Load	Resisting	Gravity Load Resisting System Comprises: - Heavy ceramic tile roofing with supporting sarking spanning to timber rafters and trusses. - Trusses in east and west wings span onto outer timber framed walls and onto internal timber walls (with the exception of the NW Gallery, where beams support the trusses). - Central section of the North Wing roof is comprised of queen post trusses supporting the roof framing. The end supports of the trusses are embedded into the concrete

	walls on the north and south ends. The internal truss supports are the internal timber walls below, which in turn are supported by rows of concrete basement columns. - Viewing platform supported on steel structure which is fixed on steel beams which sit on existing timber frame structure. - Timber frame walls to concrete nib wall - Concrete basement walls and concrete columns
Lateral Load Resisting System	In the north-south direction the seismic loads are resisted by the timber mortise and tenon jointed frames of the external walls which have diagonal cut in braces (compression only). The load is then transferred to the concrete nib walls and through into the foundation walls. In addition to this there are the internal timber lath and plaster walls which transfer the load to the unreinforced concrete midfloor and then into the unreinforced foundation walls/rows of basement columns. The building is integrally connected to the intermediate north wing so load transfer will occur at roof and ceiling level between these sections. In the east-west direction the primary lateral load resisting element is the unreinforced concrete walls (2) at the intersection of the North Wing and new North Wing Extension. In addition to these there are internal lath and plaster walls which provide lateral resistance. These loads are transferred to ground level through unreinforced foundation walls/columns. In the horizontal direction at ceiling level there are lath and plaster ceilings which provide some diaphragm action. Under earthquake loading parallel to the roof gables the roof sarking provides a load path for lateral loads, however this has been severely degraded in
Lateral Load Resisting System (Cont.)	some areas (see Prendos report). The suspended ground floor section is laterally resisted by the external and internal unreinforced concrete walls.
Roof System	The roof comprises: - Heavy ceramic tile roofing supported on 25mm thick Rimu sarking spanning between timber trusses and rafters. Sarking and rafters have experienced partial moisture decay and corrosion of nail fixings (see Prendos report). - Timber ceiling joists support a lath and plaster ceiling
Floor System	The ground floor system is an unreinforced concrete slab arching in both directions which is supported on unreinforced concrete columns and walls.

	Sprayed Renderoc concrete overlay has been applied from the ground level in multiple areas to increase the overall thickness of the floor.
Foundation System	Concrete pads support intermediate columns and are not connected. Perimeter internal and external concrete walls are supported by strip footings.
Geotechnical Considerations	Soil class D as per Cheal report
Adjacent Buildings	To the south of the building is the intermediate north wing, this is integrally connected. To the north is the new North Wing Extension. This has a 50mm seismic gap between.

2. GEOTECHNICAL CONSIDERATIONS

a. DESCRIPTION OF SOIL SITE CONDITIONS

The site is located within the Government Gardens area to the east of Rotorua central business district and is located on the east side of Oruawhata Drive. To the east of the building approximately 20m away is the Rotorua Energy Events centre. The surrounding area is characterised by geothermal activity: Sulphur Point in Lake Rotorua is located approximately 60m south from the southern extent of the building; the Polynesian Spa and the Blue Baths are located nearby.

The site can be considered generally flat with the elevation varied from the western to eastern side by approximately 3m. This has been formed by the construction of a basement foundation wall and backfill of earth to the western side of the building.

Cheal Consultants were engaged to provide geotechnical and hazard reporting for the museum (Geotechnical Factual Report – May 2017 and Geotechnical Interpretive Report-May 2017). They have advised that the site is, in general, underlain by Holocene river deposits comprising alluvial gravel, sand, silt, mud and clay with local peat, includes modern riverbeds.

The site can be considered to have two distinct periods in regard to ground bearing capacity. Cheal referenced a July 2009 Terrane Consultants Ltd (Terrane) report that stated:

- *“The foundations of the original part of the Museum buildings is underlain by highly variable, soft to extreme soft, soils to a depth of approximately 2.0 metres. The material contains cavities, some of which extend directly underneath the foundations. The soil would be classified as having a collapsible structure;*
- *The risk of local failure of the foundations is accentuated by the absence of a surface hard bar which would otherwise help to raft over the very soft soils;*
- *The building has previously been classified as being earthquake prone. The severity of the ground conditions further increases the susceptibility to earthquake shaking”*

The following information became available after a detailed review from the geotechnical engineer in February 2020. Please refer to the Appendix 1 for current inputs from geotechnical engineer- Cheal Consultants Ltd.

- The site has varying subsoil conditions.

On the eastern site firm pumice sand layer of up to 3m depth is present.

On the western site medium to soft silt layer of up to 3m is present. This is overlaid by firm crust.

The conditions improve from south to north. The worst case at present is the Atrium Area – Zone 3.

The architects have recovered a construction photo clearly indicating that the site was artificially levelled with fill placed on the eastern site. The elevated ground in front of the building at present is artificial and created from the spoils from bulk excavation.

It is also noted that the recent construction of the New South Wing – Stage 1a involved replacement of about 2500mm of residual soils with hardfill.

Main Geotechnical Design parameters remaining are:

- Liquefaction – Low Risk.
- Ground Water Level – Varies – 2-4m below basement level.
- The variety of the soil bearing capacity is reflected by using soil spring constants as determined by the Geotechnical Engineer, which are summarised in Table 2-1.

b. SOIL DESIGN VALUES FOR DEEP/PILED FOUNDATIONS

No deep/piled foundations are proposed.

c. SOIL DESIGN VALUES FOR SHALLOW FOUNDATIONS

The soil parameters are advised by the geotechnical engineer after a further study. These values are summarised in the Table 2-1.

Table 2-1 Summary of Soil Parameters Provided by Cheal Consultants Ltd

Sub-Zones	Soil Description	Undrained Shear Strength, Su	Allowable B.C (F.O.S=3)	Dependable B.C (long-term)	Dependable BC (short-term)	Spring Value K, KN/m3
3a-5a	Soft soil (Clay/Silt) at Founding Depth	30 kPa	60 kPa	90 kPa	144 kPa	11,000
3b-5b	Stiffer soils at Founding Level with Soft soils encountered at depth	50 kPa	70 kPa	105 kPa	168 kPa	17,000
3c-5c	Stiffer soil (Clay/Silt) at Founding Level	45 kPa	85 kPa	127.5 kPa	204 kPa	30,000
3d-5d	Pumice sand and Gravels below Silt/Clay	60 kPa	100 kPa	150 kPa	240 kPa	30,000

Note: Please refer to Appendix-1 for the Cheal Consultants Ltd's Geotechnical Plan: Sub-Zones and Soil Parameters

d. SITE SEISMIC RESPONSE

For seismic purposes, the soil classification of type D (deep soil) was assumed. This design parameter greatly influences the seismic design of the building and has been specifically addressed by the Geotechnical Engineer. The classification reflects the geothermal nature of the soil formation.

e. LIQUIFACTION

Potential soil liquefaction was assessed by Geotechnical Engineer and Low risk for Liquefaction was determined.

3. COMPLIANCE AND DESIGN STANDARDS

a. MEANS OF COMPLIANCE

Whilst the existing structure of the museum building will remain largely unchanged the proposed elements of seismic strengthening have to be considered as new built structure. Therefore, the design of the strengthening structure is in compliance with the New Zealand Building Code, section B1 “Structure”. Foundation design will be in accordance with the verification method B1/VM4. All other components are designed in accordance with standards listed in verification method B1/VM1 and B1/AS1. Where appropriate guidance is not available in cited standards, alternative solutions are utilized in accordance with New Zealand or international best practice. Given the significant historical value of the building the recommendations of Heritage New Zealand have been also implemented in the design.

b. DESIGN STANDARDS

The following design standards are deemed relevant for this project:

- AS/NZS1170.0: 2002 Structural Design Actions Part 0: General Principals
- AS/NZS1170.1: 2002 Structural Design Actions Part 1: Permanent, Imposed and Other Actions
- AS/NZS1170.2: 2011 Structural Design Actions Part 2: Wind Actions
- AS/NZS1170.5: 2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand
- NZS 3603: 1993 Timber Structures Standard
- NZS 3101: 2006 Concrete Structures Standard
- NZS 3404: 1997 Steel Structures Standard
- NZS 3404: Part 1: 2009 Steel Structures Standard (Part 1: Materials, Fabrication and Construction)

Standards are to be utilized as modified by the Building Code (B1 amendment 12 effective 14 February 2014).

The above list is not exclusive and other New Zealand Standards and technical references/documents shall be referred to as appropriate and the design and construction of structural elements shall comply with these also.

4. DESIGN PARAMETERS

a. GRAVITY (VERTICAL) LOADS

The following vertical loads have been used on this project:

Table 4-1 Vertical loads including Superimposed Dead and Live Loads

Level/Area	Use	Superimposed Dead Load	Live Load	
		UDL	UDL	Point Load
Roof	R2: Maintenance only	0.90 kPa	0.25kPa	1.1 kN
Ceiling	R2: Maintenance only	0.50 kPa	0.00kPa	1.1 kN
Plant Room	E8: Equipment	0.70 kPa	4.00kPa	4.5 kN/m*
Mezzanine Floor/Offices	C3: Communal area	1.00 kPa	4.00kPa	4.5 kN
Ground Floor	C3: Communal Area	0.50 kPa	4.00 kPa	4.5 kN**
Concrete Walls		15 kN/qu.m		
Partition Walls		0.5 kPa		
Masonry Walls		4.0 kPa		

* To be confirmed with the plant documentation

**To be confirmed with existing museum lifting equipment.

b. BARRIER LOADS

Barrier loads for this project have been determined as occupancy type:

C3: Areas without obstacles for moving people not susceptible to over-crowding.

These apply specifically to stairwells, external balconies and roof edges as recommended in AS/NZS1170.1:2002.

Table 4-2 Barrier and Handrail Loads Occupancy Type C3

Top Edge			Infill	
Horizontal	Vertical	Any Direction	Horizontal	Any Direction
0.75 kN/m	0.75 kN/m	0.6 kN	1.0 kPa	0.5kN

c. **WIND LOADS**

Wind actions acting on the structure are designed according to AS/NZS1170.2:2011. Site wind design parameters are as follows:

Table 4-3 Site Wind Parameters (AS/NZS1170.2:2011)

Importance Level	IL3
Design Life	50 years
Annual Probability of Exceedance	1/1000 (ULS) 1/25 (SLS1)
Region/Location	A6 (Rotorua – outside of lee zone)
Height of structure, z	15m
Wind zone - High	V _{sit,b} = 45m/sec
Aerodynamic Coefficient	C _{fig} = 1.1

d. **SNOW LOADS**

The building elevation is located in Rotorua and there is no significant snow below 1200m in the Rotorua region. Snow and ice are not considered during our design process.

e. **SEISMIC LOADS**

Seismic actions are determined in accordance with AS/NZS1170.5:2004, using equivalent static or modal response spectrum or time history method, whichever is applicable to the structure or specific structural elements.

Seismic design parameters are as follows:

Table 4-5 Site Seismic Parameters (AS/NZS1170.2:2004)

Method of Analysis	Equivalent Static Method, Model Response Spectrum Analysis
Site Subsoil Class	D (deep soil)
Design Spectra	AS/NZS 1170.5:2004
Importance Level	IL3
Design Life	50 years
Annual Probability of Exceedance	1/1000 (ULS) 1/25 (SLS1)
Region/Location	Rotorua
Hazard Factor, Z	0.24 - Rotorua
Period, T ₁	1.40 sec
Ductility Factor, μ	1.25 (ULS – superstructure) 2.0 (ULS – foundation)

***Seismic parameters are derived from the design brief.**

For the purpose of the analysis, the project x and y directions are chosen to be the project north and west directions respectively.

f. **CONSTRUCTION LOADS**

Light mechanical equipment is envisaged in the proposed Construction Methodology accounting for the limitation of existing structural members load capacity.

g. **FIRE RESISTANCE RATINGS**

Fire design and analysis is conducted by a Fire Engineer. All fire resistance requirements are in accordance with project fire engineering specification, structural compliance aspects to be achieved for the following materials through:

Concrete	Covers in accordance with NZS3101:1995.
Steel	Painted via intumescent coating-exposed steel (to be advised).
Overall Structure	Sprinkler System – Specifically designed.

5. FOUNDATION DESIGN

The proposed foundation general layout is as following:

a. Underpinning Pumice Walls.

At present the pumice walls are only able to transfer compression as there is no reinforcement used in the construction. It is acknowledged that the underpinning is designed with substantial RC ground beams. The rationale behind the design and the load path between the structural elements are as following.

The walls have central opening dividing the wall in two sections interconnected by a spandrel above GF doors. The shear capacity of the spandrel is based on the shear strength of the pumice concrete – brittle and varying with condition of the wall. This element is excluded from the load path. In the worst case the wall halves would resist the lateral forces acting independently. The generated tension component of the rocking wall area is resisted by the strong back acting in tension and flexural capacity of the underpinning ground beam. The latter is fixed to the ground by the compressed ends of the wall sections. The analysis results pattern agrees well with simplified crude strut and tie model.

The load path of the retrofitted wall includes the following elements:

Gravity Forces – Direct Compression.

Lateral Forces – Wall (acting as collector for the supported part of the building) – proposed steel Strong Backs (UC sections acting in tension only) – new RC footing – Ground.

b. Strip Footing / Ground Beam.

The retrofit scope includes new strip footing supporting the steel brace assemblies across the main walls. These are also to balance the wall uplift at the tensioned end.

c. Ground Tie Beams.

The retrofit scope includes new ground beam ties to connect brace footings with the building perimeter walls and main walls across.

d. Raft Slab Foundation under the atrium.

The proposed 500mm thick raft floor serves to:

- Integrally connect all the elements in the atrium area. Further it is acknowledged the atrium foundations will resist significantly more than the tributary gravity loads as the adjacent areas share seismic loads with Atrium structure.
- Assist in spreading vertical load and uplift resistance.
- The raft slab is designed to bridge (and/or cantilever) the potential “soft spots” up to 5m as recommended by Cheal.

The proposed upgrade enhances the bearing capacities of the main walls and perimeter walls. The gravity load path remains principally the same.

The improvement is related to the larger footing bases and lower stress transferred to the supporting soil. Soil bearing pressures are well below the soil bearing capacities.

A ductility factor of 2 is applied for the analysis of wall in-plane action since the building primary lateral resisting system will be the steel structures after strengthening. Ductility factor 2 is considered justifiable given the building maximum ULS deformation is under 50mm which should be easily accommodated for the building after strengthened. A model of non-linear supports for the footings is assumed. The compression only spring values for this non-linear model were developed by the geotechnical engineer according to the interpolated soil properties, which are summarised in Table 2-1.

The maximum expected settlement is to be 25mm for all the foundations.

Whilst shallow foundations are used in the retrofit (from practical reasons and heritage restrictions) the overall horizontal displacement at the top of the building is well below allowed limits.

6. CONSTRUCTION METHODOLOGY

In order to carry out the foundation works safely without undermining the existing load bearing walls/façade being retained, the following rules have been set to accompany the construction methods. In general, underpinning works for the existing load bearing structure shall be carry out in a hit and miss sequence in small sections, typically less than 3m long (wall end sections shall be less than 2m).

Prior to excavation work, existing pumice walls need to be sandwiched with continuous steel PFCs (380PFC) throughout at the bottom. Hydro excavation should be avoided to prevent the changes of moisture content in basement. No more than two underpinning segments are excavated at one time.

7 days curing period is required for the poured concrete prior to excavating the adjacent underpinning section.

When the new foundations encounter the existing footings of basement columns/walls, it is contractor's responsibility to provide appropriate temporary propping to assure the temporary stability of the structures during underpinning works. The temporary propping has to be remained in place until 28 days after the pouring of new foundation concrete.

7. SERVICEABILITY CONSIDERATIONS

a. DESIGN FOR DURABILITY

The location of the building and presence of geothermal environment immediately within and around the building requires specific approach to durability aspects of the design.

Target design life for the new components is:

Foundation: 100 years
Superstructure: 50 years

Durability provisions are achieved for the following materials through:

Concrete	Reinforced concrete members with chemical exposure XA3 – Table 3.3 NZS3101: Part 1: 2006. To secure the design life we propose: ▪ Minimum concrete cover: 60mm with DPM at side and bottom surfaces ▪ Minimum concrete strength 40 MPa ▪ Improve concrete density with concrete additives – silica fume ▪ Improve concrete neutral chemical composition by replacing Portland cement with fly ash
Steel	Surface treatment in accordance with NZS/AS 2312:2014. The replacement parts are galvanised.
Timber	Timber treatment in accordance with NZS 3602:2003. It is noted that no new timber structural elements included in the strengthening structure except new platform.

b. FLOOR VIBRATION

In stage 2 no floor strengthening is included.

8. STRENGTHENING STRUCTURE DETAILED DESCRIPTION

Refer to the Foundation Element Description which is a part of the documents submitted for the building consent application.

9. SUSTAINABLE DESIGN CONSIDERATIONS

Some sustainable design principles that are implemented in the in the structural design for Detailed Design - Stage 2 are:

- Reduce resource consumption. Re-use the temporary propping steel structure (380PFC for pumice wall) for the connection of strong-back assemblies.
- Increased durability with a specific concrete design mix.

Due to significant heritage value this project has not been targeted to specifically address sustainability. However, our design considers the principles above in the following ways:

- By achieving an economic design using the minimum amount of material required.
- Selecting structural system that allows major building refurbishment without modification to the primary structure.
- Locating all steel structure internally and separating from aggressive exposure increases usable building life over and above the intended 50-year design life.
- Consideration of the building's substructure to last 100 years is a minimal cost item that increases usable building life over and above the intended 50-year design life.

10. SAFETY IN DESIGN

Safety aspects in design which are considered for Detailed Design - Stage 2 are outlined below:

a. CONSTRUCTION

Table 8-1 Safety in Design – Construction

Risk/Activity	Comments
Persons falling	Risk of failure and subsequently persons falling
Falling objects	Tools etc. falling from height
Chemical Exposure	Hazardous chemicals and contaminants on-site
Work within Confined Space	Risk of injuries associated with limited operational space

b. IN SERVICE

Table 8-2 Safety in Design - In Service

Risk/Activity	Comments
Glass windows/ Balustrades	Risk of failure and subsequently persons falling
Viewing Platform in Ceiling and Roof	Platform access required for safety of the visitors.

c. DURING MAINTENANCE (SCHEDULED SERVICING AND MINOR REPAIR)

Table 8-3 Safety in Design - During Maintenance

Risk/Activity	Comments
Cleaning the building	Safety concerns, access rails and/or fall arrest anchor points will be required.
Lift maintenance	Access to the lift-pit for maintenance, confined space concern.
Access to sprinkler system	Ceiling access required for safe maintenance activities.
Access to Plant Platforms	Platform access required for safe maintenance activities
Access to basement safety systems	Basement access required for safe maintenance

d. **MAJOR REFURBISHMENT**

Table 8-4 - Safety in Design - Major Refurbishment

Risk/Activity	Comments
Penetrations in concrete ground floor	Necessary precautions during installation of new vertical bracing elements, kit penetrations etc..
Modifications to buildings	Access between adjacent building zones to accommodate differential residual horizontal displacement after seismic event.
Electrical strike from buried or enclosed cabling	Precautions are in place for excavation, cutting in locations where wires/cables are in operation.

e. **DEMOLITION**

Table 8-5 Safety in Design – Demolition

Risk/Activity	Comments
Chemicals exposure	Hazardous chemicals and contaminants on-site
Structural stability	Non-complicated load paths for predictable demolition

11. RISKS AND UNKNOWNNS

The main items of risk or unknowns relating to the structure identified during the design stage are described below. These will be reviewed and updated during the ongoing stages of detailed design, coordination and contractor design development with the goal of minimizing or removing them. The following elements are relevant to Detailed Design – Stage 2.

a. IDENTIFIED RISKS SPECIFIC TO THIS PROJECT – STAGE 2.

- Temporary stability of the load bearing structure elements during underpinning work.

b. GENERAL PROJECT RISKS

A suitable provision should be made for these items in both the cost plan and construction programme:

- Coordination of services with the structure;
- Allowance for heavy items of plant in confined space;
- Contractor shall consider the temporary propping for load bearing structural elements prior to demolition/ excavation the existing footings;

APPENDIX 1

CHEAL CONSULTANTS LTD'S INPUT FOR SOIL PARAMETERS

CPT03

cheal

Designed
IWJ

Checked

Approved

Date
26/02/20

Notes
Zone 3
Geotechnical Plan: Sub-Zones & Soil Parameters

Drawing Title
Rotorua Museum
Seismic Strengthening

Drawing Number
16592 - SK002

Taupo T: +64 7 378 6405 | Level 1, 4 Horematalangi Street | PO Box 165, Taupo 3351

Sub-Zone 3a
Soft soils (Clay/Silt) at Founding Depth (RL282.7m approx.).

Representative Soil Strength Parameters:
- $S_u = 30\text{kPa}$ (max.)
- All. $BC^* = 60\text{kPa}$
- $k = 11,000\text{kN/m}^3$

Sub-Zone 3b
Stiffer soils at Founding Level with Soft soils encountered at depth.

Representative Soil Strength Parameters:
- $S_u = 50\text{kPa}$ (30kPa at depth)
- All. $BC^* = 70\text{kPa}$
- $k = 17,000\text{kN/m}^3$

Sub-Zone 3c
Stiffer Soils (Clay/Silt) at Founding Level.

Representative Soil Strength Parameters:
- $S_u = 45\text{kPa}$ (min.)
- All. $BC^* = 85\text{kPa}$
- $k = 30,000\text{kN/m}^3$

Sub-Zone 3d
Pumice Sands and Gravels below Silt/Clay

Representative Soil Strength Parameters:
- $S_u = 60\text{kPa}$ (min.) for Silts/Clay
- All. $BC^* = 100\text{kPa}$
- $k =$ (assume same as Sub-Zone 3c)

- Cheal (2017)
- Scala
- CPT
- Fully Cored BH
- Terrane (2007-09)
- Fully Cored BH
- Hand Auger
- Test Pit (with Shear Vanes)
- Kerslake (1985)
- Plate Bearing Test.

FOR COORDINATION

0	Revision 0	RZ & DD	MO	17/02/2020
Rev	Details	By	Chkd	Date

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Client
ROTORUA LAKES COUNCIL

Project
ROTORUA MUSEUM DEVELOPMENT

Drawing Title
FOUNDATION - ZONE 2A, 3 - OVERLAID WITH EXISTING COLUMNS & WALLS

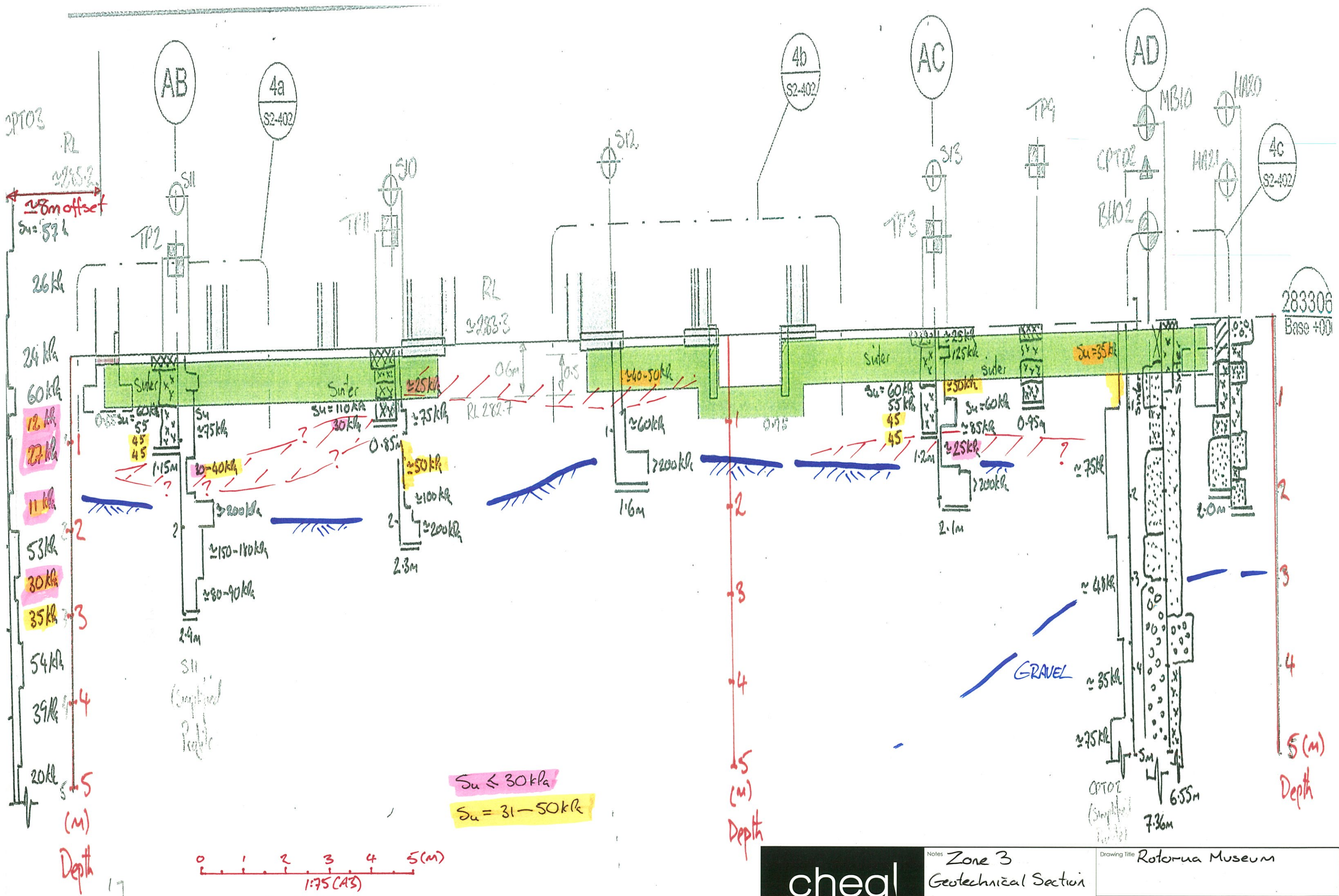
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Date FEB 2020		Original Size A1
Job No J000949	Drawing No S2-212	Issue 0

Original Scale
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All. $BC^* =$ Allowable Bearing Capacity including FOS = 3,

1 Foundation Plan - Zone 2A, 3 - Overlaid With Existing Columns & Walls
SCALE: 1:75

CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE



$S_u \leq 30 \text{ kl}$
 $S_u = 31 - 50 \text{ kl}$

0 1 2 3 4 5 (m)
 1:75 (AS)
 Vert = 2 x Hor.

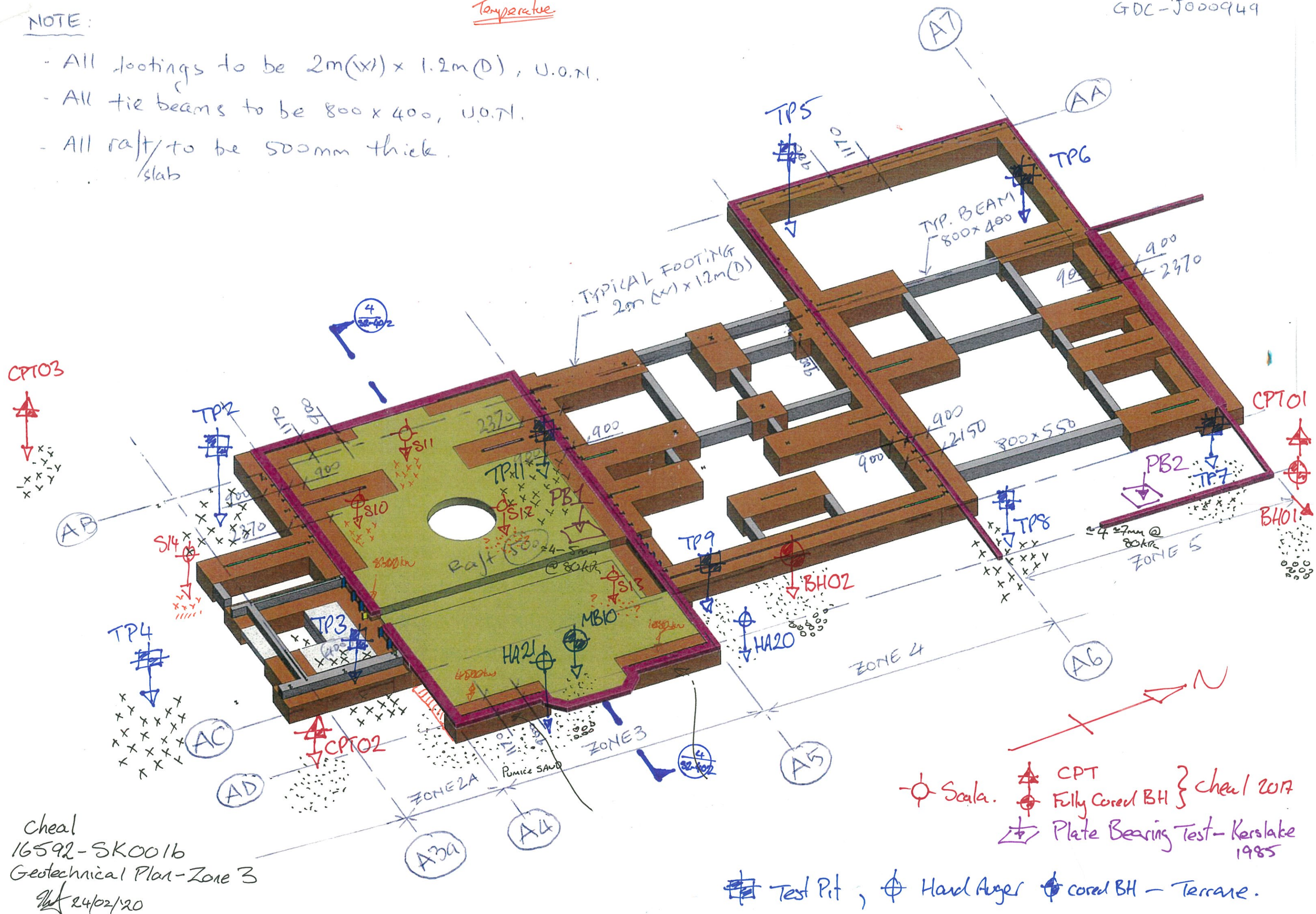
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Designed	Checked	Approved	Date				
			24/02/20				

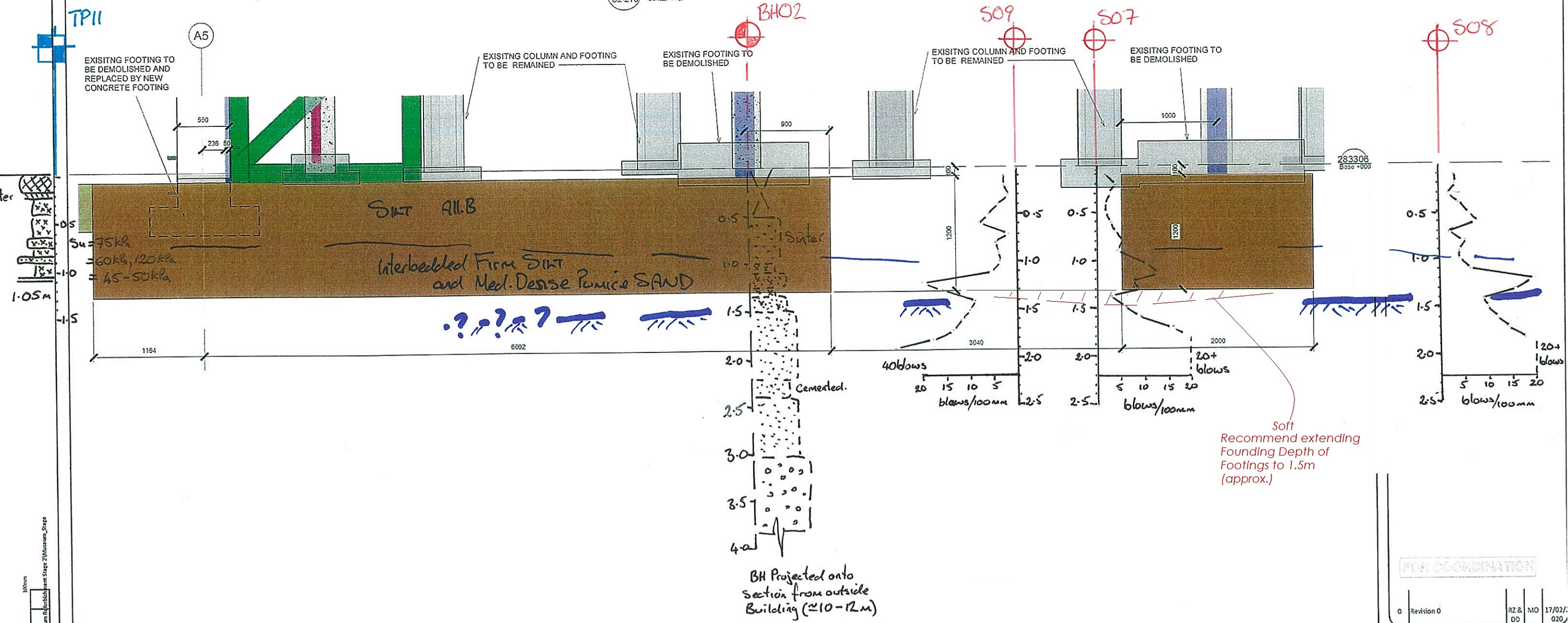
NOTE:

- All footings to be 2m(W) x 1.2m(D), U.O.N.
- All tie beams to be 800 x 400, U.O.N.
- All raft to be 500mm thick slab

Temperature

GDC-J000949





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Rev	Details	By	Chkd	Date

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Project
ROTORUA MUSEUM DEVELOPMENT

Drawing Title
FOUNDATION SECTIONS - SHEET 4

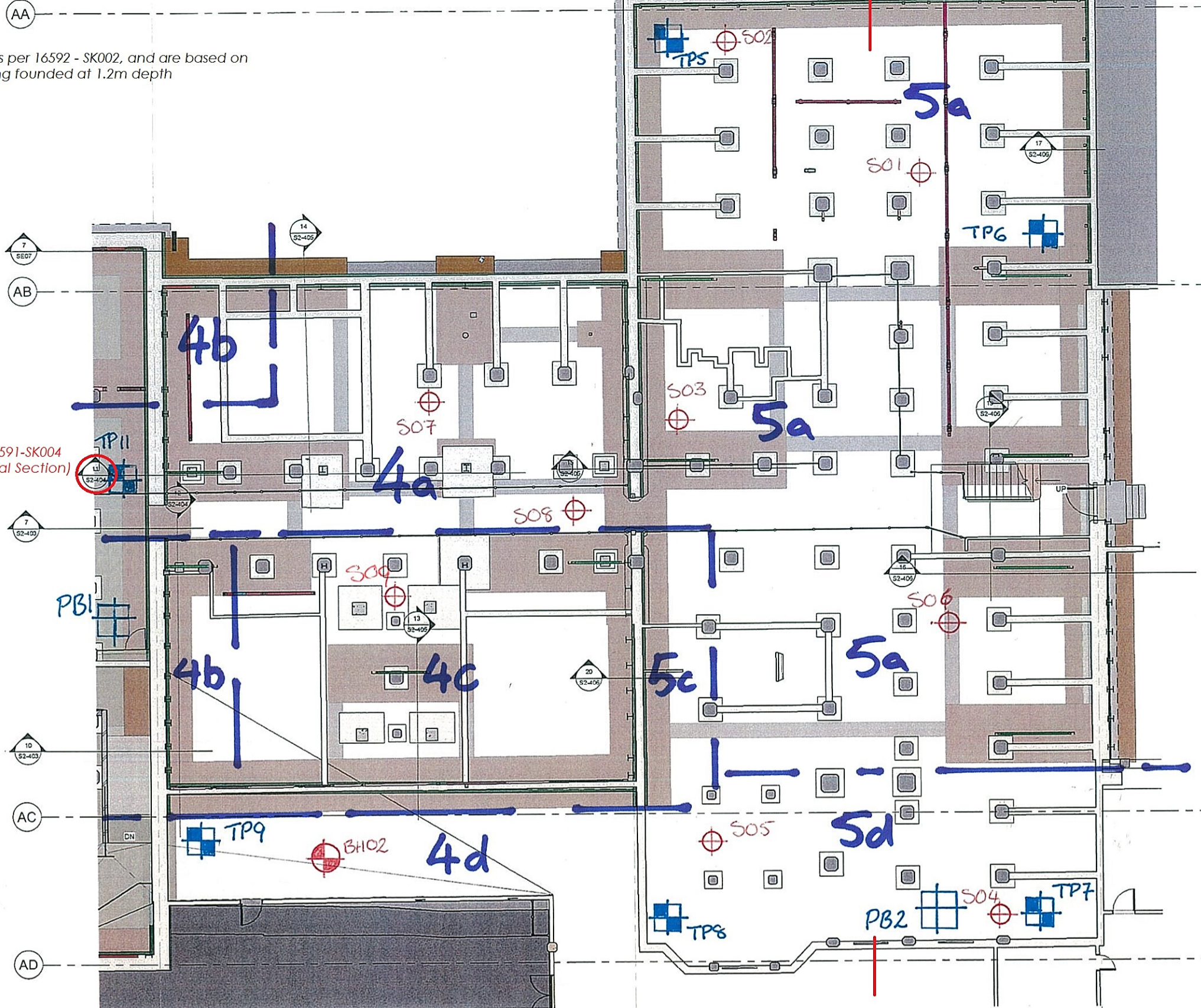
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Date FEB 2020	Original Size A1	
Job No J000949	Drawing No S2-404	Issue 0

				Notes Zone 4 Geotechnical Section	Drawing Title Rotorua Museum Seismic Strengthening
				Drawing Number 16592 - SK004	
Designed IWJ	Checked	Approved	Date 27/02/20		

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Sub-Zones are as per 16592 - SK002, and are based on
the footing being founded at 1.2m depth

See Dwg 16592 - SK002
for Key to Investigation
Symbols.



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0	Revision 0	RZ & DD	MO	17/02/2020
Rev	Details	By	Chkd	Date

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Client

ROTORUA LAKES
COUNCIL

ORUA MUSEUM
DEVELOPMENT

Drawing Title
FOUNDATION - ZONE 4, 5 - OVERLAID
WITH EXISTING COLUMNS & WALLS

Designed Designer	Scale 1:75	Drawn Author
Date FEB 2020	Original Size A1	
Job No J000949	Drawing No S2-214	Issue 0

1
S1-301 Foundation Plan - Zone 4, 5 - Overlaid With Existing Columns & Walls
SCALE: 1:75

CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

Desi
IWJ

Checked

Approved

Date
27/02/'20

Scale

16592 - SK005

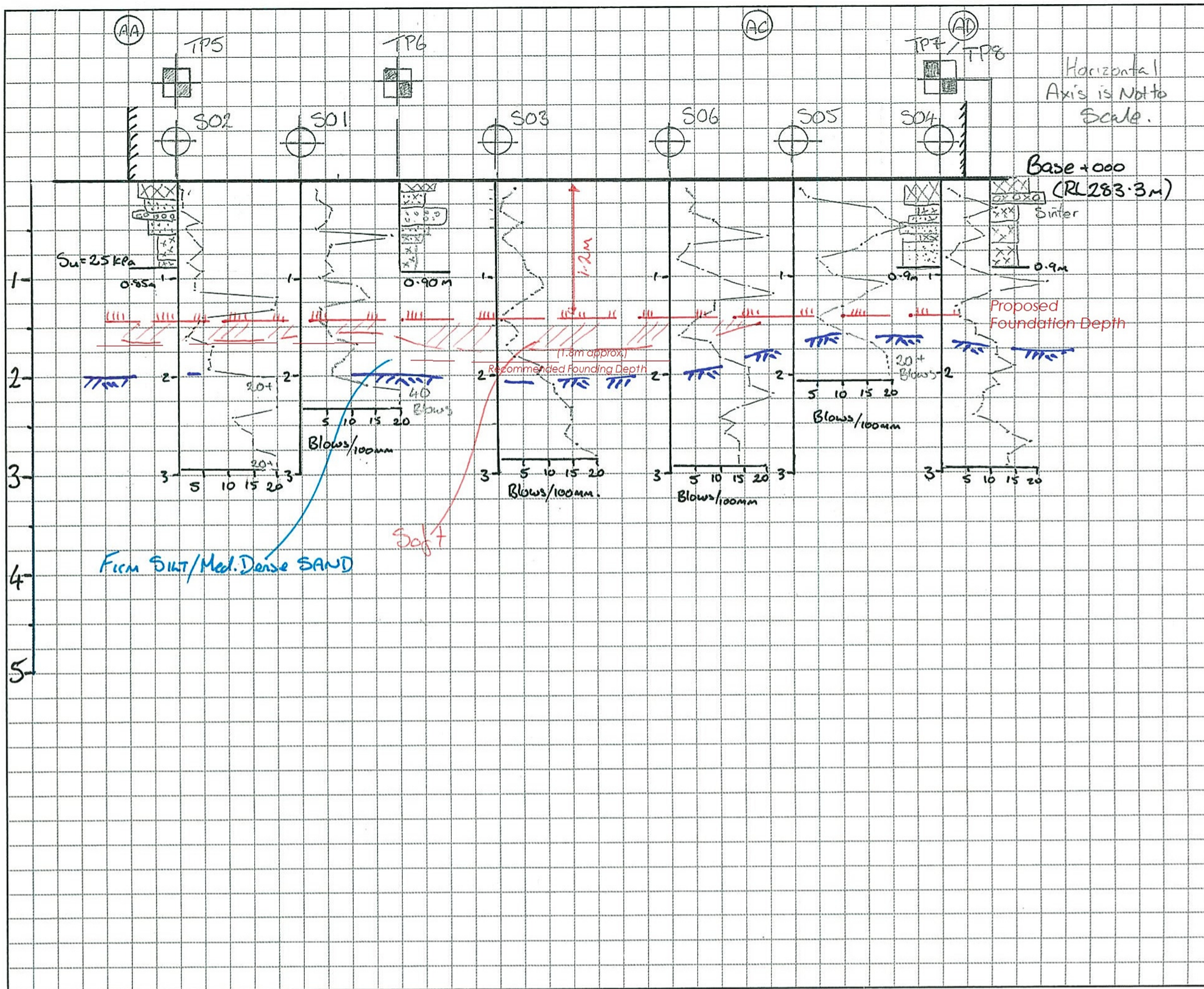
Drawing Number

Notes

Zone 5

Geotechnical Section

Drawing Title

Rotorua Museum
Seismic Strengthening

CPT03

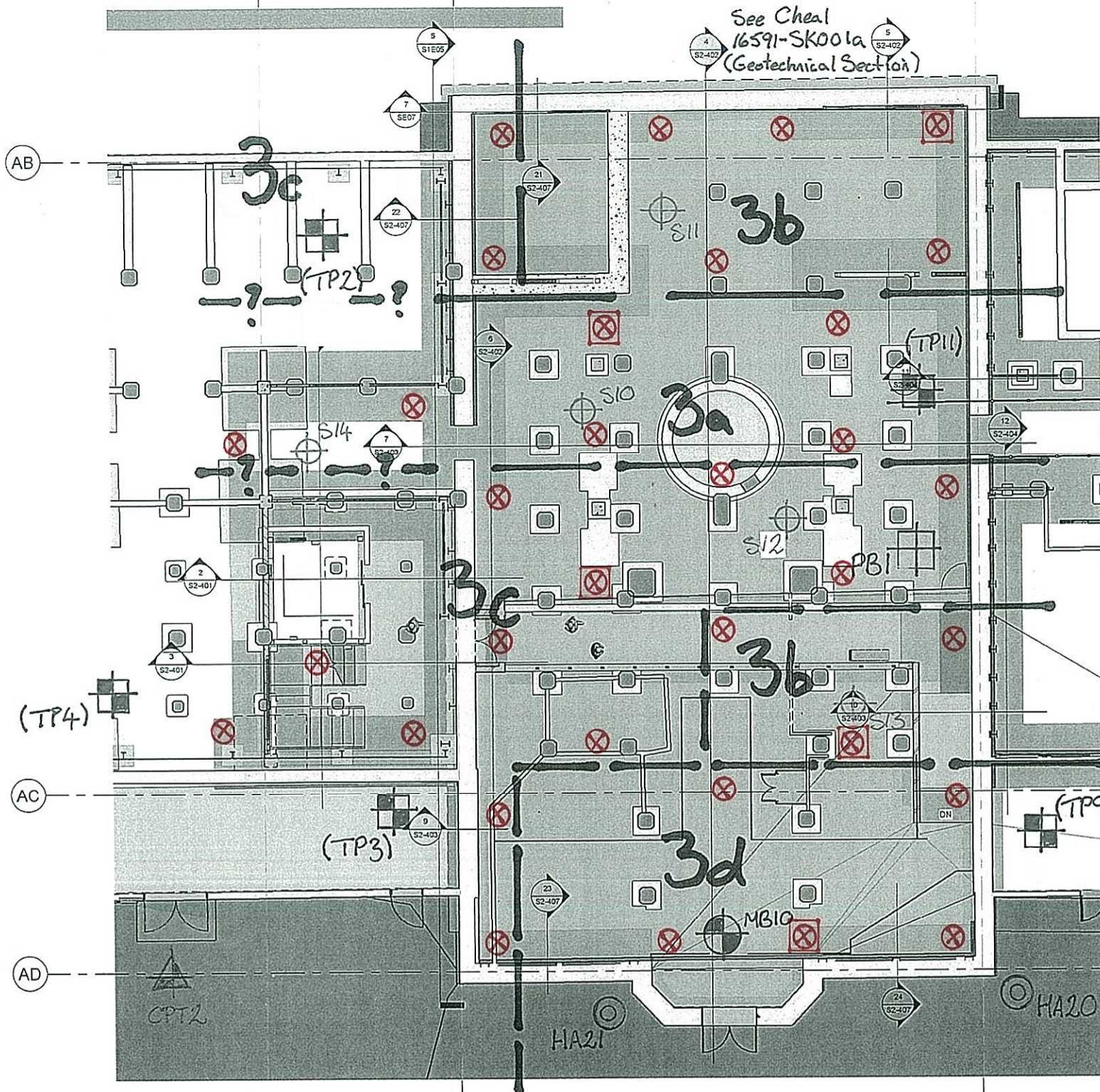
cheal

Notes Zone 2A & 3
Verification Testing for
Construction

Drawing Title Rotorua Museum
Seismic
Strengthening

Drawing Number
16591 - SK006a

Taupo T: +64 7 376 6405 | Level 1, 4 Haramatangi Street | PO Box 165, Taupo 3351



Test Position
Depth Profile*

Existing Ground Level (RL 283.3m) Base +0.00	
Shear Vane	Plate Bearing
0.6m	0.5m
0.8m	1.0m
1.0m	1.5m
1.25m	1.75m
1.50m	2.00m
1.75m	
2.00m	

⊗ Shear Vane
Testing
(Multiple Depths)*

□ Plate Bearing
Test
(at Founding Depth)

Plate Bearing Tests shall be carried out using a 300 x 300mm plate.

Where practical a second Plate Bearing test shall be carried out approx. 0.5m below Founding Depth.

Where there is a significant difference in Shear Vane test reading between adjacent test positions an additional set out test shall be carried out approx. 1/2 way between the original tests.

The Geotechnical Engineer shall supervise the testing and review the results, and direct additional undercut were required.

Rev	Details	By	Chkd	Date
0	Revision 0	RZ	MO	17/02/2020

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Client
**ROTORUA LAKES
COUNCIL**

Project
**ROTORUA MUSEUM
DEVELOPMENT**

Drawing Title
**FOUNDATION - ZONE 2A, 3 - OVERLAID
WITH EXISTING COLUMNS & WALLS**

Designed	Scale	Drawn	Author
Designer	1:75		
Date	FEB 2020	Original Size	A1
Job No	Drawing No	Issue	
J000949	S2-212	0	

CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

1 Foundation Plan - Zone 2A, 3 - Overlaid With Existing Columns & Walls
S1-301 SCALE: 1:75

Zone 2A

Zone 3

Zone 4
(See 16591 - SK006b)

Original Scale
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2_C37\IMD_2020_02_24_Site\G37.rvt

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APPENDIX 2 (in separate file)

STAGE 2 FOUNDATION DESIGN CALCULATIONS (Rev.A Dated 02 March 2020)