Hawke's Bay District Health Board

AAU Histology Block Hawke's Bay Hospital

Detailed Seismic Assessment

Hawke's Bay District Health Board

AAU, Pharmacy, Dialysis Block HA34

Hawke's Bay Hospital

Detailed Seismic Assessment

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Executive Summary

Background

This report describes the Detailed Seismic Assessment (DSA) of the structure of the AAU Histology Block at the Hawke's Bay Hospital. The building is located within the Hawke's Bay Hospital, on Canning Road, on the lot with the following legal description:

Lot 3 DP2443 (0.5774Ha)

This building is a part of the Hawke's Bay Hospital and has a post-disaster function to society. As discussed with the Hawke's Bay District Health Board (HBDHB), the building will be considered as Importance Level 3, according to NZS1170.0. However, results for an IL4 building will also be provided and discussed to provide better information and options to the Board.

Building Description

The AAU Histology Block was originally built as the single storey with a basement "Pharmaceutical and Outpatients Block" and designed in 1979 by Edwards, Clendon & Partners Consulting Engineers. This original design considered an additional floor for a potential future expansion. The original structure is comprised of reinforced concrete moment frames in both directions for the ground floor and first floors with an in-situ concrete slab. The roof was just above the first-floor level and comprised of a steel structure supported on the portal frames. The original seismic design coefficient was 0.192g. The original concrete frames have adequate detailing to achieve ductility, although most of the beams have non-complying reinforcement splices located at the plastic hinges.

Due to the presence of uncontrolled fill over most of the plan area of the building, foundations are comprised of encased concrete piles of 600mm or 750mm diameter and 3m depth. It is understood that these piles will not have sufficient embedment into bearing strata to sustain significant tension loads arising from seismic demands.

In 2004, the originally intended expansion of the building was built. As part of this alteration, seismic strengthening with steel braces took place to replace the primary lateral load resisting system bracing the first floor. In a second stage, the strengthening scheme was completed by providing reinforced concrete block masonry shear walls on the ground floor level to transfer the actions from the braces on the ground floor to the foundations.

The current gravity structure for the ground and first storeys remains almost like original, with reinforced concrete frames founded on piles. For the first floor, precast beams were used as lost formwork with the addition of 100mm thick in-situ slab on top. Lateral load systems were enhanced during the 2004 alterations by providing steel concentrically braced frames (CBF) at some bays in line with architectural partitioning for the first storey and reinforced concrete masonry walls at the basement. Most of the walls in the basement are placed under the CBFs with some exceptions.

The first-floor slab has a seismic joint to the neighbouring building to the south, with a total gap of 35mm provided. The gap is covered by a sliding steel plate that during the inspection showed signs of distress. Shrinkage cracks were also visible at the first-floor slab during the inspection. Due to this seismic gap and discontinuity of frames, it will be considered that the AAU Histology block remains as a separate structure from other buildings. However, 35mm deflection may give room for potential pounding with the adjacent structure. At the ground floor level, no signs of a seismic gap were observable.

The roof was replaced and a plant room provided during the 2004 alterations. The gravity structure of the roof is comprised of steel portal frames spanning along numbered grid lines. Lateral bracing is provided via frame action along numbered gridlines and tension-only CBFs in the longitudinal direction (Reid braces). Part of the roof structure is shared with the plant room, which has a separate set of stiffer braces.

The gravity structure for the plant room consists of an in situ concrete slab on top of precast concrete beams as lost formwork. The concrete slab rests on a steel portal framed structure to provide a load path for gravity actions. The plant room is braced in both directions with CBFs and eccentrically braced frames (EBFs). Both the Plant Room roof and main roof are braced with tension only bracing in the longitudinal direction and steel portal frames in the transverse. These portal frames are the primary gravity structure for the roof.

The 2004 alterations were designed to an elastic lateral coefficient of 1.05g using the standard for Loading at the time, NZS4203:1992. This compares to the current demand for an IL4 structure based on NZS1170.5:2004 of 2.11 g for full code requirements and an elastic structure. The site specific probabilistic seismic hazard assessment (PSHA) prepared by GNS Science (McVerry & Buxton, 2012) defines an elastic seismic coefficient of 1.45g. By comparing these factors, it can be seen that the design target of the 2004 alterations would represent only 50% of NZS 1170.5 and 72% of the spectra provided by the PSHA for an IL4 structure. For an IL3 structure, the design loads used in 2004 are 70% for the NZS1170.5 spectra and 96% for the PSHA.

The building is currently used as an Acute Assessment Unit (AAU) on the ground floor, with a pharmacy and a dialysis unit as well. This area is open 24hours to the public and has beds to have patients under temporary supervision. A library, an auditorium and an education centre work are located on the First Floor. We understand that a future Histology laboratory may be located in a current vacant area at the first floor level.

The building has been assessed based on the specific seismic hazard study performed by GNS Science. The subsoil conditions for the site have been defined as site subsoil class D or deep soil. It is Opus' understanding that there is a pre-existing Initial Seismic Assessment (ISA) report on the building, rating it above 70%NBS IL4.

During this report, no geotechnical seismic hazards have been evaluated such as liquefaction and lateral spreading. It is recommended that an additional report and advice should be sought in these areas.

Current structural layouts can be seen in the following figures:

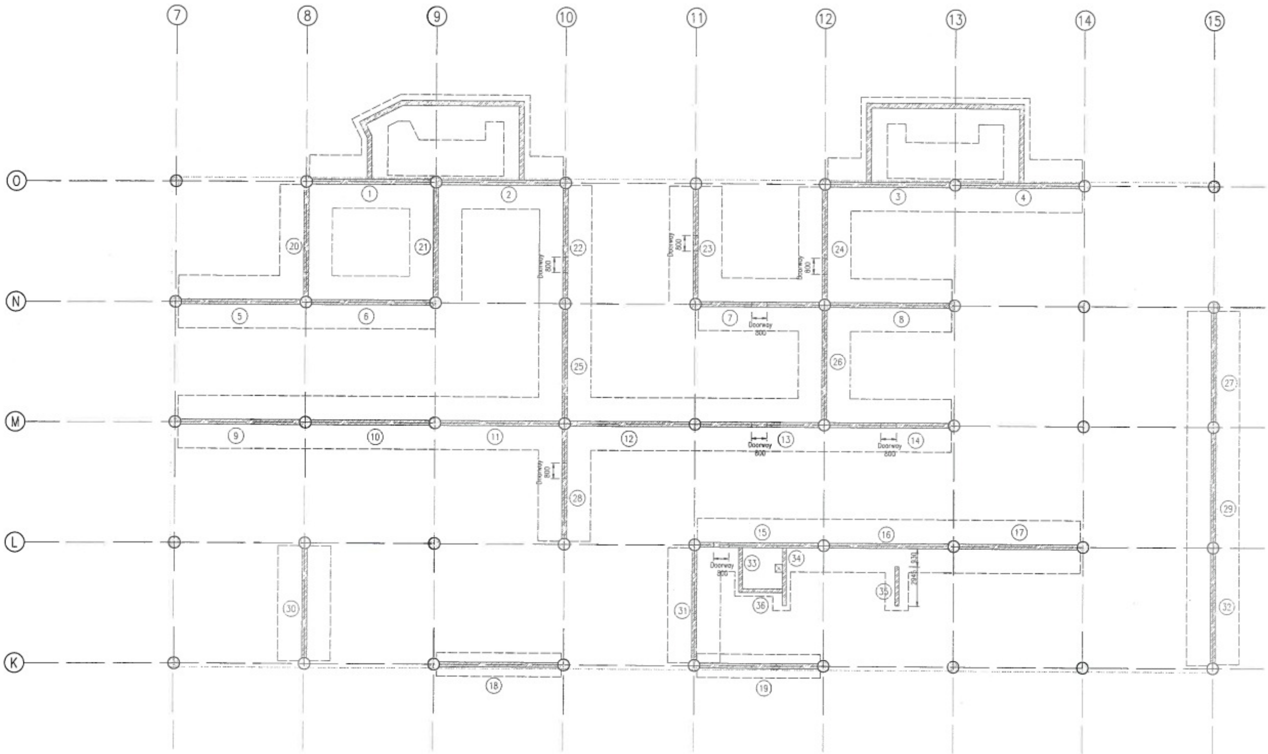


Figure 1: Layout of CMU shear walls under Ground Floor

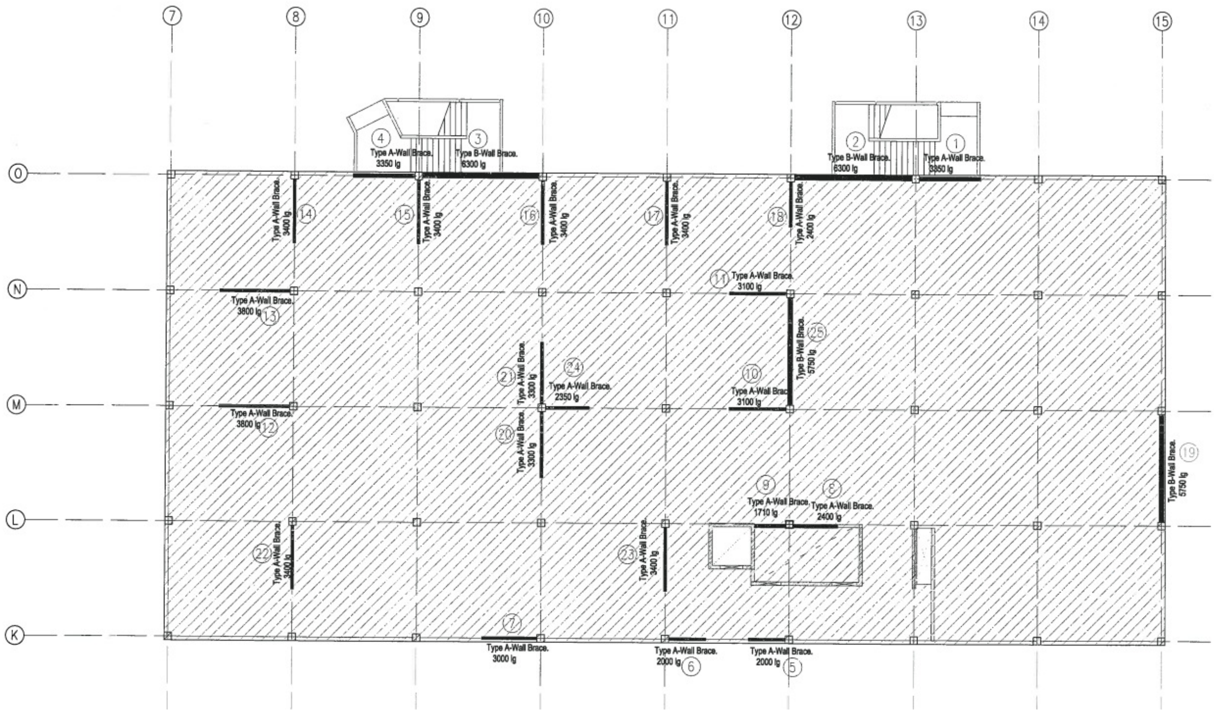


Figure 2: Layout of CBF to first floor after the 2004 alterations

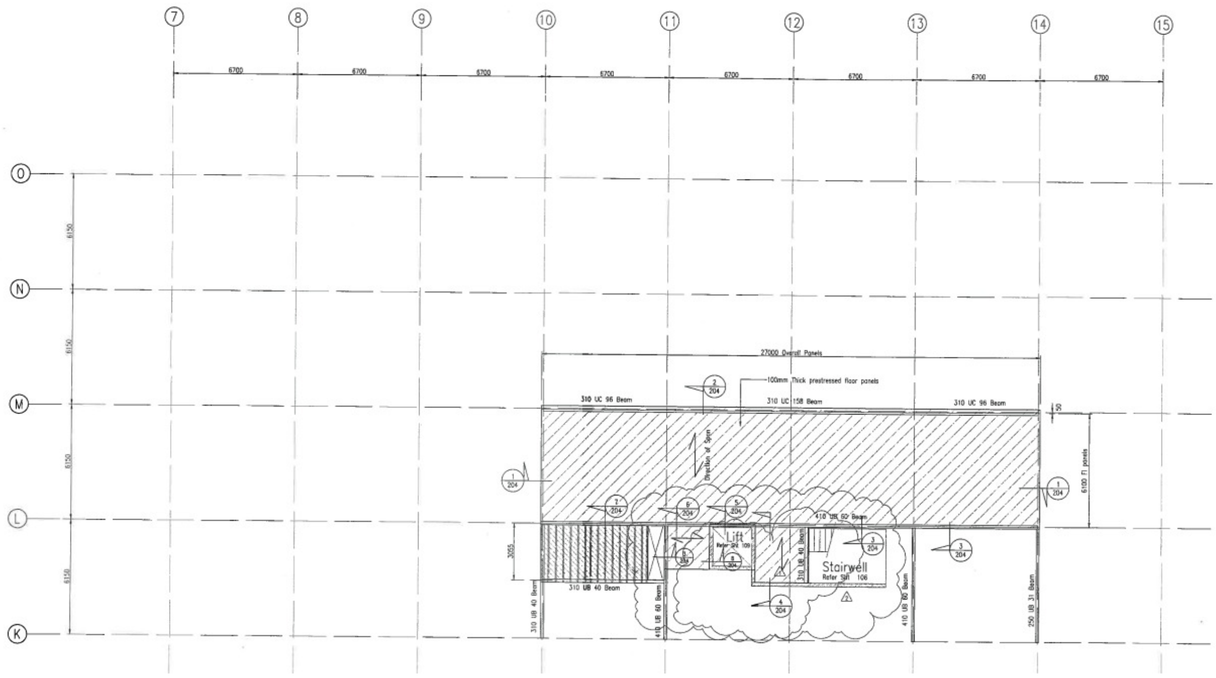


Figure 3: Current Plant Room Floor Plan

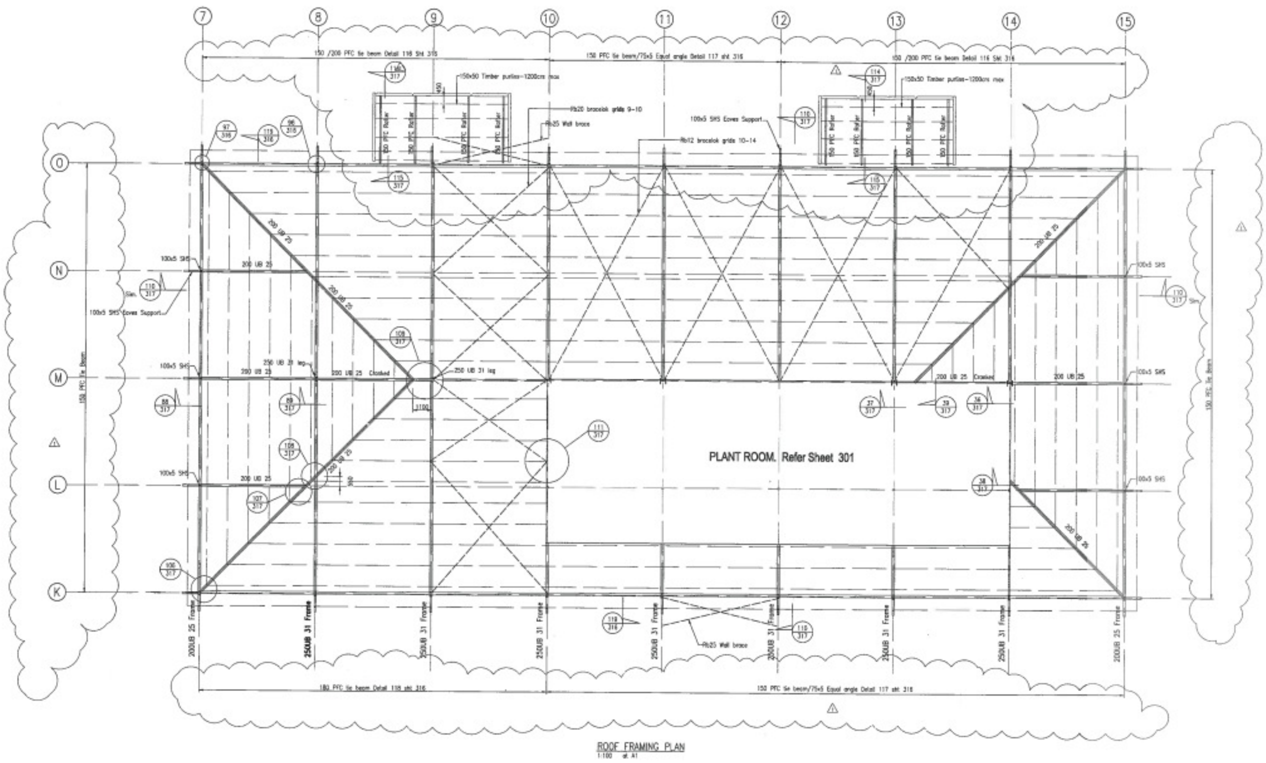


Figure 4: Current Roof Structure

Basis for the Assessment

This assessment has been based on the following information:

- Original Structural Drawings dated November 1980 by Edwards Clendon and Partners.
- Alterations Drawings from LHT Design, dated May 2004.
- Geotechnical Report dated 26/8/1974 by Brickell, Moss, Rankine and Hill Consulting Engineers.
- Two non-invasive walk through site visits performed during May and August by Opus Engineers Julian Benito and Noel Evans.
- "Seismic Design Spectra and Geotechnical Hazard Summary for Hastings Hospitals", R. McVerry et. Al., GNS Science Consultancy Report 2012/243, September 2012. (McVerry & Buxton, 2012)
- No intrusive investigations were completed.
- Limited material testing data from the 2004 design calculations with no additional material testing.
- The geotechnical report used in the 2004 alterations.

Assessed Earthquake Rating

The results of the DSA indicate the building's earthquake rating to be ***15%NBS (IL4)*** or ***15%NBS (IL3)*** assessed in accordance with the guideline document The Seismic Assessment of Existing Buildings-Technical Guidelines for Engineering Assessments, dated July 2017. The earthquake rating assumes that ***Importance Level 4 (IL4)*** and ***Importance Level 3 (IL3)***, in accordance with the Joint Australian/ New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002, is appropriate. Therefore, this is a ***Grade E*** building following the NZSEE grading scheme.

Grade E buildings represent a risk to occupants **greater than 25 times** that expected for a new building, indicating a ***very high*** risk exposure. A building with an earthquake rating less than 34%NBS fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-Prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67%NBS is considered as an Earthquake Risk Building (ERB) by the New Zealand Society for Earthquake Engineering.

The AAU-Pharmacy and Dialysis Block is not therefore categorised as an Earthquake Risk Building, and also it meets the criteria that could categorise it as an Earthquake Prone Building (EPB). The territorial authority will define whether the building is EPB or not.

A summary of the score of individual structural elements can be found in the following tables:

Table 12: Summary of Results for IL4 - ULS

Element	Direction	Rating [%NBS]	Mode of Failure/Comments
Main Roof Structure	Longitudinal	**<15**	Longitudinal reid-braces insufficient. Weld failure, anchor failure.
	Transverse	**<15**	Frames without sufficient capacity. Ductility and fully bracing required to achieve higher NBS.
Plant Room Roof	Longitudinal	**<15**	Tension-only bracing and detailing not sufficient. Elastic failure.
	Transverse	**35**	Frames not fully braced. Lateral torsional buckling likely.
Plant Room Floor	Longitudinal	**22**	Connection of braces fail, braces not in opposing pairs
	Transverse	**24**	Connection of braces fail, braces not in opposing pairs
First Floor	Longitudinal	**44**	CBFs connections with brittle failure.
	Transverse	**40**	CBFs connections with brittle failure.
Ground Floor	Longitudinal	**100**	Capacity of shear walls. Overall capacity limited by foundation.
	Transverse	**53**	Capacity of shear walls. The openings and the number of walls are the reason for the disparity of results with the longitudinal direction.
Foundations	Longitudinal	**87**	Sliding failure of foundations and failure of piles.
	Transverse	**80**	Sliding failure of foundations and failure of piles.
External Stairs (Non-Structural)	Longitudinal	**26**	Shear friction at base of walls.
	Transverse	**22**	Shear friction at base of walls. Global stability also an issue.
Results		**15%NBS (IL4)**	

Table 14: Summary of Results for IL3 - ULS

Element	Direction	Rating [%NBS]	Mode of Failure/Comments
Main Roof Structure	Longitudinal	**<15**	Longitudinal reid-braces insufficient. Weld failure, anchor failure.
	Transverse	**<15**	Frames without sufficient capacity. Ductility and fully bracing required to achieve higher NBS.
Plant Room Roof	Longitudinal	**27**	Tension-only bracing and detailing not sufficient. Elastic failure.
	Transverse	**63**	Frames not fully braced. Lateral torsional buckling likely.
Plant Room Floor	Longitudinal	**37**	Connection of braces fail, braces not in opposing pairs
	Transverse	**43**	Connection of braces fail, braces not in opposing pairs
First Floor	Longitudinal	**58**	CBFs connections with brittle failure.
	Transverse	**53**	CBFs connections with brittle failure.
Ground Floor	Longitudinal	**100**	Capacity of shear walls. Overall capacity limited by foundation.
	Transverse	**69**	Capacity of shear walls. The openings and the number of walls are the reason for the disparity of results with the longitudinal direction.
Foundations	Longitudinal	**100**	Sliding failure of foundations and failure of piles.
	Transverse	**87**	Sliding failure of foundations and failure of piles.
External Stairs (Non-Structural)	Longitudinal	**44**	Shear friction at base of walls.
	Transverse	**36**	Shear friction at base of walls. Global stability also an issue.
First Floor Cladding Panels	Out-of-plane	**90**	Failure in Bending of PFC, mechanism forced and fall of panel to the ground.
Results		**15%NBS (IL3)**	

Seismic Retrofit Options

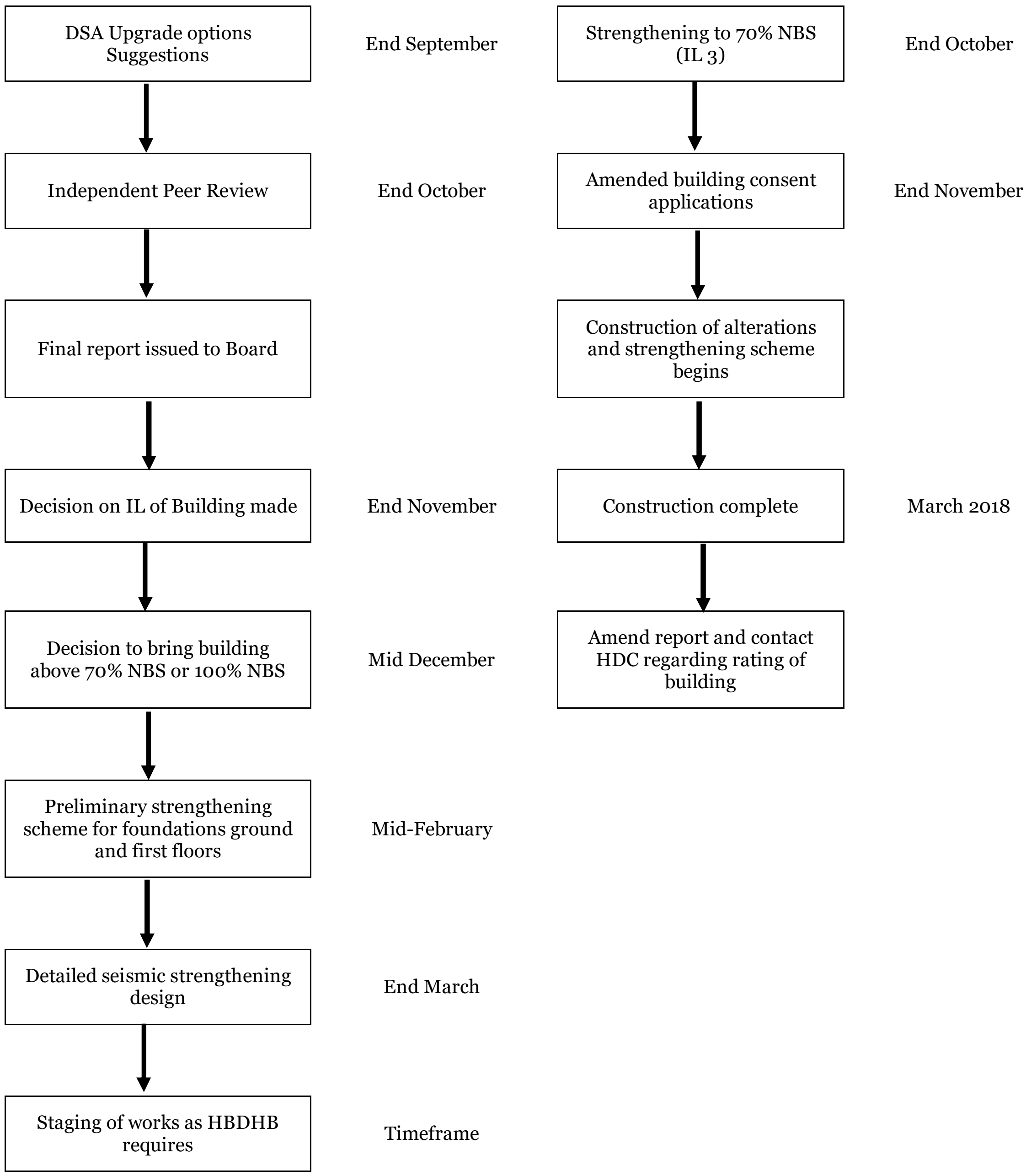
Seismic retrofit options are fully described in section 6 of this report. It includes options for 34%NBS (IL3) and 70%NBS (IL3). HBDHB has been working closely with Opus to expedite a strengthening scheme to 70%NBS which is currently under the design phase.

For 34%NBS (IL3) seismic strengthening will be concentrated at the roof structure. This will be achieved by replacing existing cross braces and providing additional ones. Lateral restraint of the portal frames will also be required via fly braces. This strengthening will cause low to medium disruption to the first floor.

For 70%NBS (IL4) seismic strengthening, additional members will be required at the roof level to brace it in both directions. Braces of the plant room will need to be replaced. Additional bracing will be necessary on the first floor, although these can be constructed from the outside limiting the disruption to the first-floor operations. Additional shear walls at the basement will also be required to brace the ground floor.

Recommended Next Steps

Discussions with the HBDHB are in place to develop a strengthening scheme to bring the building to 70%NBS. This strengthening scheme is currently in the design phase, and the HBDHB is expediting the resolution of the low scoring elements of the building. In this fashion, the following flowchart summarises the steps to be followed and the intent of the HBDHB in the short and medium term:



Technical Summary

1. Building Information	
Building Name/ Description	Acute Assessment Unit (AAU) – Histology – Pharmacy Building HA34
Street Address	210 Omahu Road., Hastings
Territorial Authority	Hastings District Council
No. of Storeys	2
Area of Typical Floor (approx.)	1320m^2
Year of Design (approx.)	1979 (original), 2004 (alterations and strengthening)
NZ Standards designed to	NZS4203:1976, NZS 4203:1992
Structural System including Foundations	Roof structure with tension-only CBFs and steel portal. The first floor braced in both directions with CBFs; ground floor braced via CMU walls.
Does the building comprise a shared structural form or shares structural elements with any other adjacent titles?	Seismic gap to the NE at the roof and first floor. No seismic gap at ground floor.
Key features of ground profile and identified geohazards	Liquefiable foundation soils, building founded on poor quality fills. Soil D.
Previous strengthening and significant alteration	2004: major alterations and strengthening to NZS 4203:1992.
Heritage Issues/ Status	None.
Other Relevant Information	Part of the Hawke's Bay Hospital

2. Assessment Information	
Consulting Practice	Opus International Consultants Ltd.
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings[1]	Dave Dekker, CPEng 1003026 Earthquake Engineering Principal with more than 20 years of experience in the field. Julian Benito, CPEng 1033106 Senior Structural Engineer with more than nine years of experience in the field.
Documentation reviewed, including: • date/ version of drawings/ calculations[2] • previous seismic assessments	Original construction drawings and calculations. 2004 calculations and drawings. No previous seismic assessments were reviewed, although an ISA has been performed before.
Geotechnical Report(s)	The original geotechnical report, report from 2004 alterations and GNS Report on the seismic hazard.
Date(s) Building Inspected and extent of inspection	Building inspected in May and August 2017. Visual inspection only.
Description of any structural testing undertaken and results summary	No testing required. 2004 alterations had one witness taken for the concrete strength.
Previous Assessment Reports	ISA
Other Relevant Information	Refer to GNS report under the references for seismic demands.

[1] This may include reference to the engineer's Practice Area being in seismic assessment, or commentary on experience in practice and recent relevant training, particularly if prior to re-assessment of practice area

[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	Assessed for IL4 and IL3. Hospital ward and laboratories with varying floor loads. Plant room at top storey.
Site Subsoil Class	D
For a DSA:	
Summary of how Part C was applied, including: • the analysis methodology(s) used from C2 • other sections of Part C applied	Sections A, C1, C2, C3, C4, C6 and C10 were used during this assessment. The force-based methodology has been applied with the equivalent static method used for representing earthquake demands.
Other Relevant Information	Elastic numerical model used to corroborate periods and deflections.

Contents

1 Introduction

1.1 Introduction

This report describes the Detailed Seismic Assessment (DSA) of the structure of the AAU Histology Block at the Hawke's Bay Hospital. The building is located within the Hawke's Bay Hospital, on Canning Road, as per Figure 1, on the lot with the following legal description:

Lot 3 DP2443 (0.5774Ha)

The building under study has two storeys above the surrounding ground level and a basement, used only for services. The approximate plan dimensions of the building are 53mx24,6m for a total area of 1.300 m². The total height of the building is approximately 12m. The geometry of the building can be seen in the following figures:

Figure 5: Building Aerial View (Source http://mapping.hdc.govt.nz)

Figure 6: AAU Histology Block (Source maps.google.com)

This building is a part of the Hawke's Bay Hospital. As discussed with the Hawke's Bay District Health Board (HBDHB), the building will be considered as Importance Level 3, according to NZS1170.5. However, analysis for an IL4 building will also be provided and discussed so as to provide better information and options to the Board.

The building is surrounded by other structures, with minimal seismic gaps or isolation. However, possible seismic interactions with adjoining structures is possible, and will be assessed in this report.

1.2 Scope of Work

The scope of work for this DSA includes the following:

- Walk through non-intrusive site inspection of the building.
- Quantitative structural assessment to determine the percentage of New Building Standard (%NBS) of the building based on Importance Level 4 (IL4) and Importance Level 3 (IL3) building.
- A desktop geotechnical investigation.
- Broad recommendations for improving the earthquake rating to 67%NBS(IL4) ULS if required and 100%NBS(IL4) SLS2 code performance, and 67%NBS(IL3) ULS.

The seismic performance of services and heavy equipment has not been assessed during this process. It is recommended that bracing of services should be assessed or improved as part of any remediation works. In addition, this report only considers earthquake loading, and therefore wind, gravity and other types of loading have not been included in this assessment.

Assessment of geotechnical hazards such as liquefaction, lateral spreading, soil instability and soil deformability have not been included fully in this report. An independent report by a suitably qualified geotechnical engineer will be required in this field.

1.3 Sources of Building Data

Research of the Hasting District Council records has drawn two sets of drawings that will be of importance for the assessment. However, there are some differences between the consent drawings and the final as-built condition. The following documents and references have been used:

- Original Structural Drawings dated November 1980 by Edwards Clendon and Partners.
- Alterations Drawings from LHT Design, dated May 2004.
- Geotechnical Report dated 26/8/1974 by Brickell, Moss, Rankine and Hill Consulting Engineers.
- Walk through non-intrusive site visit performed during May 2017 by Opus Engineers Julian Benito and Noel Evans.
- "Seismic Design Spectra and Geotechnical Hazard Summary for Hastings Hospitals", R. McVerry et. Al., GNS Science Consultancy Report 2012/243, September 2012. (McVerry & Buxton, 2012)

The drawings and a non-intrusive inspection have been used to confirm the structural systems, identify the mechanism type, any Severe Structural Weaknesses (SSW) and Structural Weaknesses (SW) associated with the structure and non-structural defects that may affect the performance of the structure. No original design calculations have been located besides a succinct design features report.

1.4 Building Regulations

The Building (Earthquake-prone Buildings) Amendment Act 2016 is the current amendment to the Building Act 2004 that sets the performance objectives for buildings, and provides a system for managing earthquake-prone buildings that include the MBIE guidelines. The intent of the act is to protect people and property and therefore performance limits are set in terms %NBS as an ultimate limit state (ULS). Although the Act sets a threshold of 33%NBS for earthquake-prone buildings, it does not set the minimum performance objectives for post-disaster function as the Hawke's Bay Hospital. It is the recommendation of the new MBIE guidelines that this type of building should provide at least 100%NBS for an SLS2 earthquake, with minimal damage to continue operations as normal. For the Ultimate Limit State (ULS) the only legal requirement is to comply with the 33%NBS within the legal time frame but, for an IL4 building, a recommended target of 67%NBS(IL4) should be achieved.

A hospital building is likely to be needed in an emergency, and therefore it is classified as a priority building by the Act, meaning that the building should be assessed to determine if it is earthquake-prone before 1 Jan 2020. If any seismic work is required to raise the rating of the building, the timeframe for a priority building in a high seismic risk area like Hawke's Bay is 7.5 years after the Territorial Authority has issued the EPB notice.

Opus International Consultants encourages the building owner to liaise with Territorial Authority after the results of the DSA report have been discussed in order to comply with the regulatory expectations and time frames.

2 Building Description

2.1 Site Geotechnical Conditions

The Hawke's Bay Hospital is located on the Heretaunga Plains, described as Holocene river deposits comprised of poorly consolidated alluvial gravel, sand and mud by GNS Science New Zealand's Geology Web Map. Hastings District Council mapping described the site as Holocene River Deposits, that were former river channels occupied during the last century and that have the presence of lenticular sands and gravels in main channels with interlayered silt deposits. The depth of the river deposits can be up to 1km deep at the location. This area has been assigned a moderate earthquake shaking hazard according to the Hasting District Council (HDC).

The original geotechnical report for the site was prepared by Brickell, Moss, Rankine and Hill Consulting Engineers and is dated August 1974. The report shows that the building under study is located on a fill of and old quarry zone. This borrow pit has been filled uncontrollably with various depths of slag and cinder refuse with large pieces of concrete and scrap metal. The report concludes that deep foundations are necessary to transfer the loads to the natural alluvial deposits past the fill materials up to 15ft (4.5m) in thickness. Council maps also show the extent of the uncontrolled fill areas, shown in the following figures:

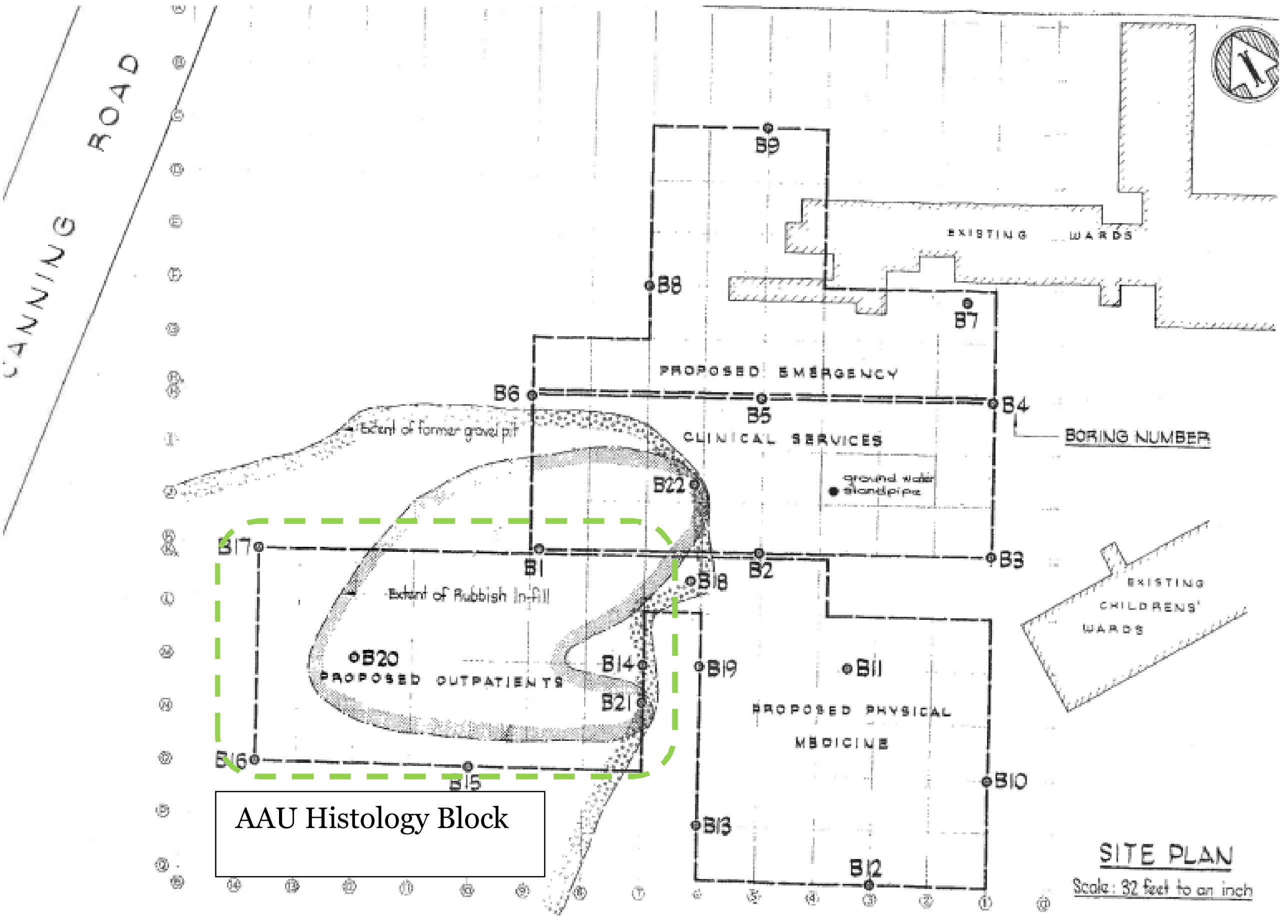


Figure 7: Extent of Uncontrolled Fill – From BMRH Report 1974

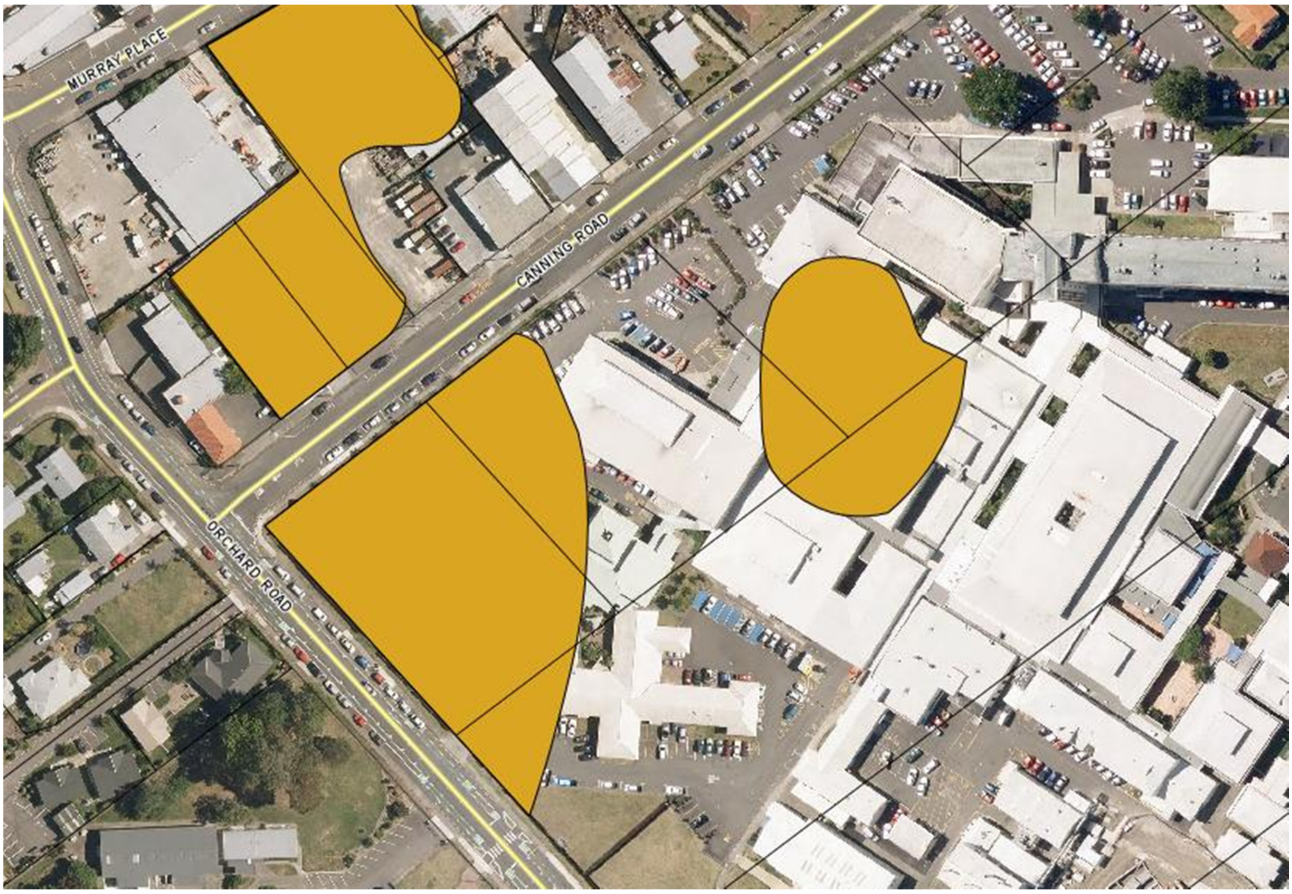


Figure 8: Suspected Fill areas by Hastings District Council – mapping.hdc.govt.nz

Elsewhere on site, there is a 1.5m layer of alluvial gravels with good bearing capacity. The water table was found to be 3m below the surface. However, this measurement has been taken 40 years ago and it is expected to have changed due to the further land development of the area.

In terms of subsoil classification, based on the above information on the original geotechnical report, HDC maps and area GNS report, we can assume the site subsoil class to be D. Shear wave velocity records define the site period to be 3.7 seconds at 300m depth where the bedrock lies. The site lies within 20km of six active faults, being the most significant the Napier Fault that was the cause of the 1931 Hawke's Bay earthquakes. It is believed that the superficial expression of this fault lies within 600m of the site.

In terms of liquefaction hazards, the subsoil conditions of fill materials, loose gravels, silts and sands with a high water table, makes the site susceptible to liquefaction. The GNS report states that there are present small amounts of liquefiable material in thin layers on the top 3m and a few layers between 5 and 8m. Therefore, it is believed liquefaction is likely in the event of a major earthquake, with the worst effects likely to be within the 75m wide strip bounding Omahu road (McVerry & Buxton, 2012).

Hasting District Council has categorized the liquefaction risk for the area as moderate. We encourage the client to engage a chartered professional Geotechnical Engineer to undertake a geotechnical assessment of the site in order to quantify the risks associated with these hazards.

As a conclusion, the site is comprised of layered alluvial deposits with the inclusion of a vast area of uncontrolled fill. The site can be catalogued as site subsoil class D for deep soil. Liquefaction risk is moderate for the site. For the purpose of this analysis, geotechnical properties of the underlying soils will be assumed from the data provided in the original geotechnical report. Additional investigations by LHT in 2004, has confirmed the use of Su=30kPa for earthquake loads.

2.2 Building Description and History

The AAU Histology Block was originally built as the single storey with basement “Pharmaceutical and Outpatients Block” and designed in 1979 by Edwards, Clendon & Partners Consulting Engineers. This original design considered an additional floor (roof) for a potential future expansion. The original structure is comprised of reinforced concrete moment frames in both directions for the ground floor and first floors. The roof was just above the first floor level and comprised of a steel structure supported on the portal frames. The design loads were those as per NZS4203:1976 with the following parameters I=1.6, S=0.8, M=1, R=1. These parameters reflect that the concrete frames were designed for ductile behaviour, with an Importance Factor of 1.6 (post-disaster). Thus, the original design seismic coefficient builds up to 0.192g. The original concrete frames have adequate detailing to achieve ductility, although most of the beams have reinforcement splices located at the plastic hinges, a detail not acceptable in current codes.

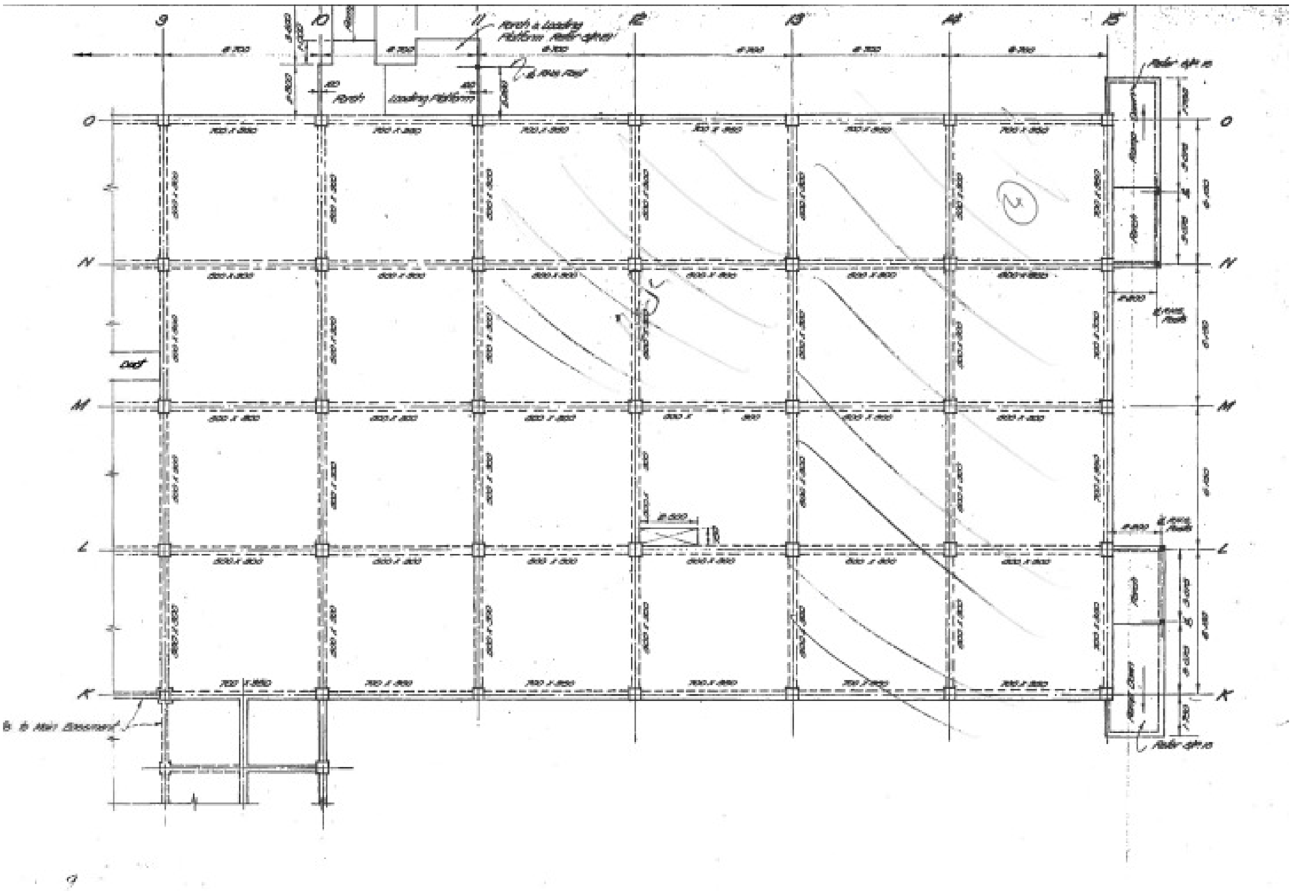


Figure 9: Original Ground Floor Plan 1979

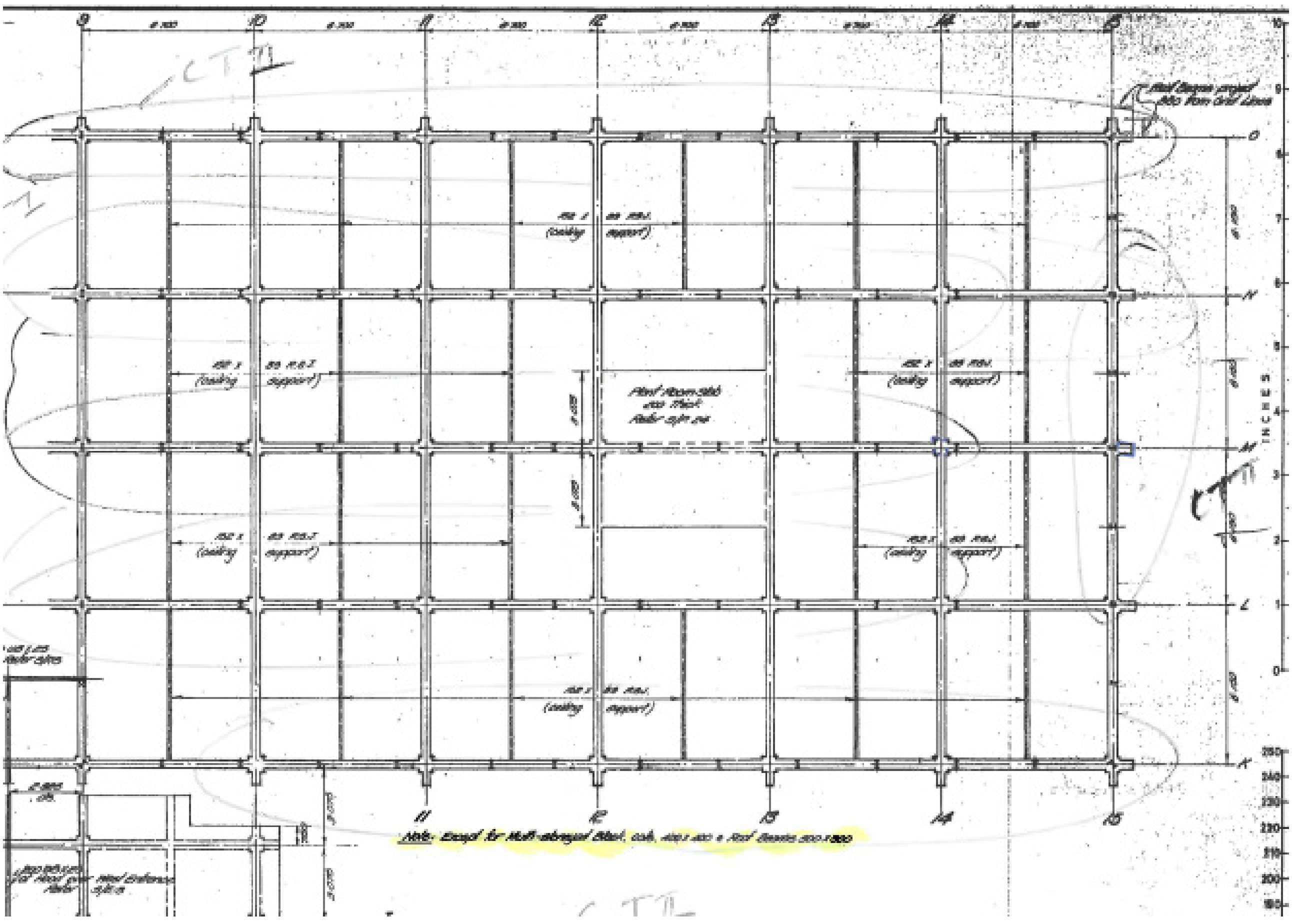


Figure 10: Original First Floor Structure with only steel roof on top.

At the ground level, a suspended slab is supported between the main beams. Below the ground floor structure, there is a basement level, with limited access, used for services. Originally the basement is only depicted with limited height. During the 2004 alterations, it was dug down to a height of 2.5m.

Due to the presence of uncontrolled fill over most of the plan area of the building, foundations are comprised of encased concrete piles of 600mm or 750mm diameter and 3m depth. It is understood that these piles will not have sufficient embedment into bearing strata to sustain significant tension loads arising from seismic demands. The 2004 alterations exposed the top metre of the piles, removing that soil friction from its capacity for gravity or uplift forces.

The first-floor structure was designed to receive the future precast or in-situ floor slabs and allow the upgrade of the building to a full two-storey structure. This is reflected in the original design and calculations from 1979, where it can be noted that the design seismic mass comprised 1060kN for the Roof Level above the first floor, 3949kN for the Second floor and 6973 kN for the first floor level (ground floor). Refer to Figure 9 and Figure 10.

In 2004, the original intended expansion of the building took place. The current gravity structure for the ground and first storeys remains almost as original, with reinforced concrete frames founded on piles. For the first floor, precast beams were used as lost formwork with the addition of 100mm thick in-situ slab on top. Lateral load systems were enhanced during the 2004 alterations by providing concentrically steel braced frames (CBF) at some bays in line with architectural partitioning for the first storey and reinforced concrete masonry walls at the ground floor. Most of the walls under the ground floor are placed under the CBFs with some exceptions.

The first-floor slab has a seismic joint to the neighbouring building to the south, with a total gap of 35mm provided. The gap is covered by a sliding steel plate that during the inspection showed signs of distress. Shrinkage cracks were also visible at the first-floor slab during the inspection. Due to this seismic gap, it will be considered that the AAU Histology block remains as a separate structure from other buildings. However, 35mm deflection may give room for potential pounding.. At the ground floor level, no signs of a seismic gap was observable.

The roof was replaced and a plant room provided during the 2004 alterations. The gravity structure of the roof is comprised of steel portal frames spanning along numbered (north-south) gridlines. Lateral bracing is provided via frame action along numbered gridlines and tension-only CBFs in the longitudinal direction (Reid braces). Part of the roof structure is shared with the plant room, which has a separate set of stiffer braces.

The gravity structure for the plant room consists of an in-situ concrete slab on top of precast concrete beams as lost formwork. The concrete slab rests on a steel portal framed structure to provide a load path for gravity actions. The plant room is braced in both directions with CBFs and eccentrically braced frames (EBFs). Both the plant room roof and main roof are braced with tension only bracing in the longitudinal direction and steel portal frames in the transverse. These portal frames are the main gravity structure for the roof. The layouts of the 2004 alterations can be seen in the following figures:

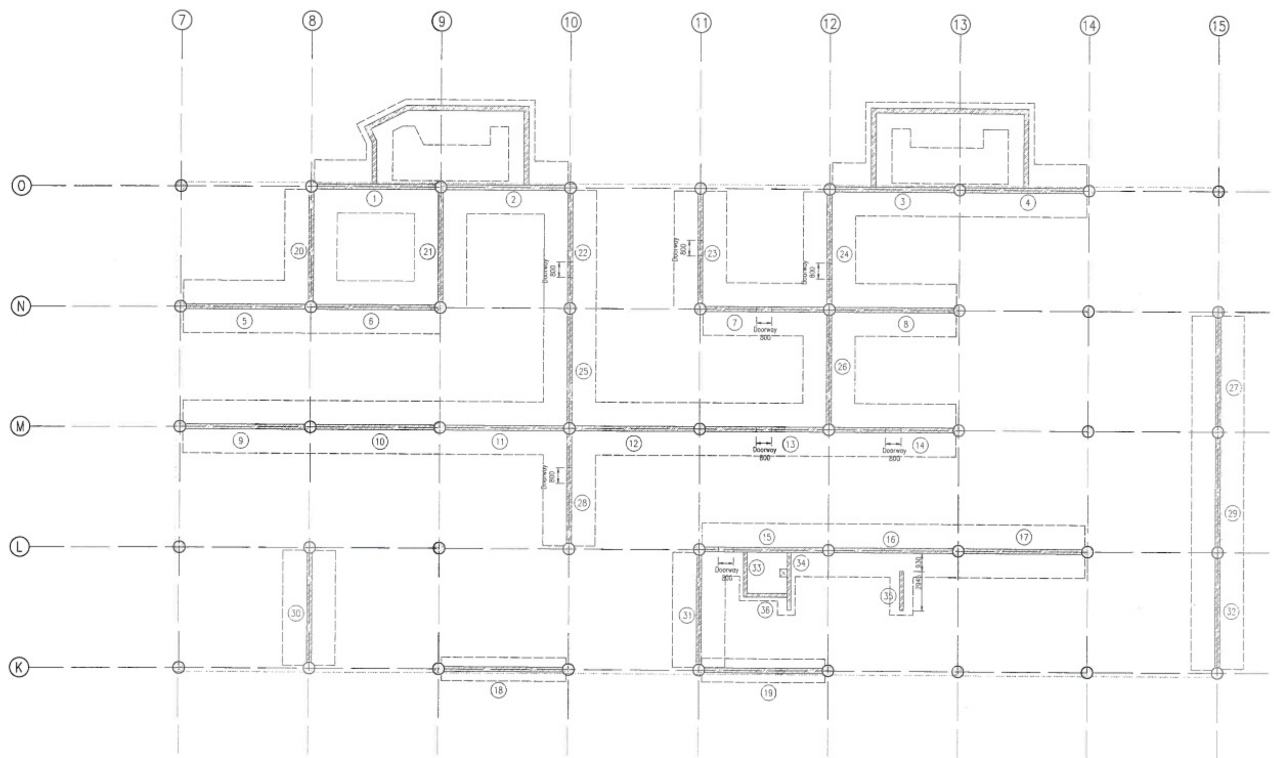


Figure 11: Layout of CMU shear walls under Ground Floor

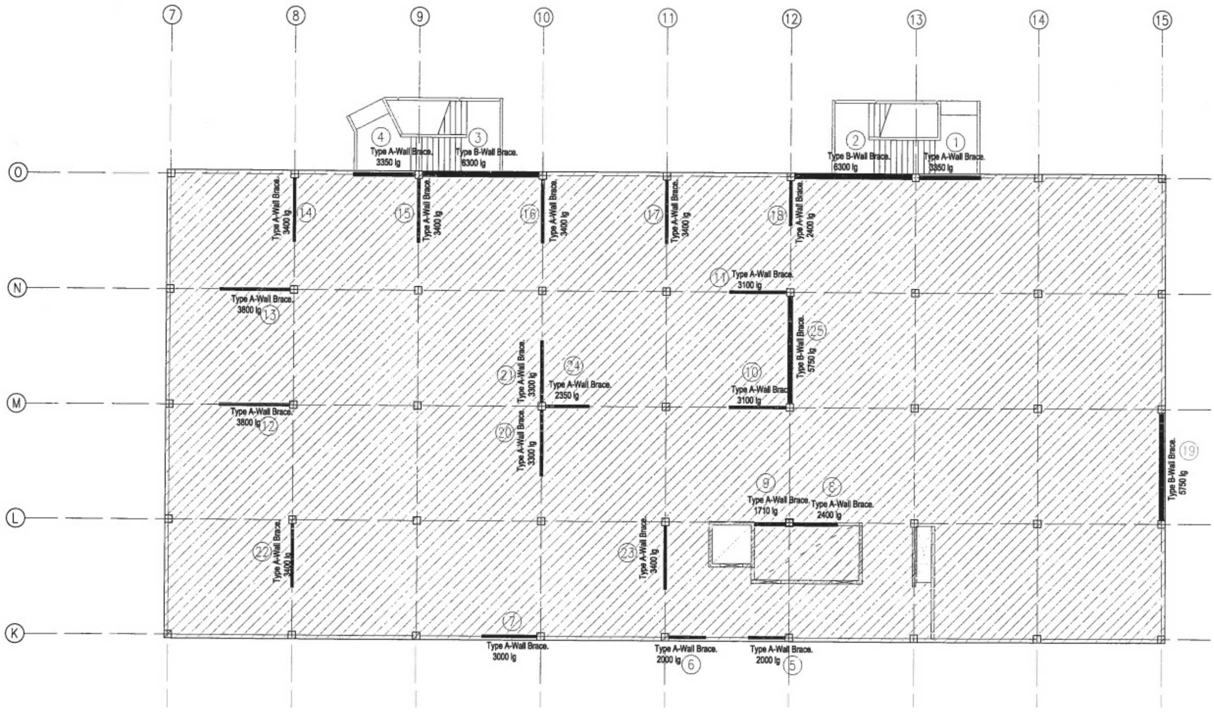


Figure 12: Layout of CBF to first floor after the 2004 alterations

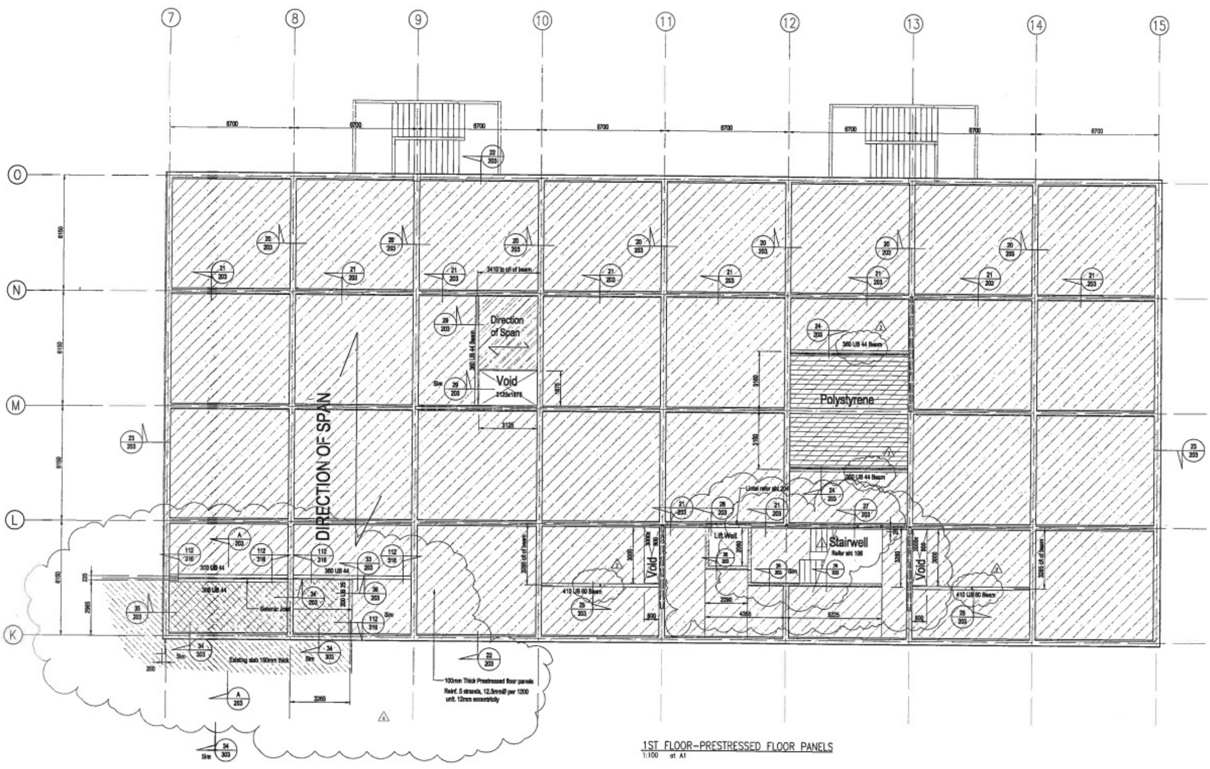


Figure 13: Current First Floor Plan

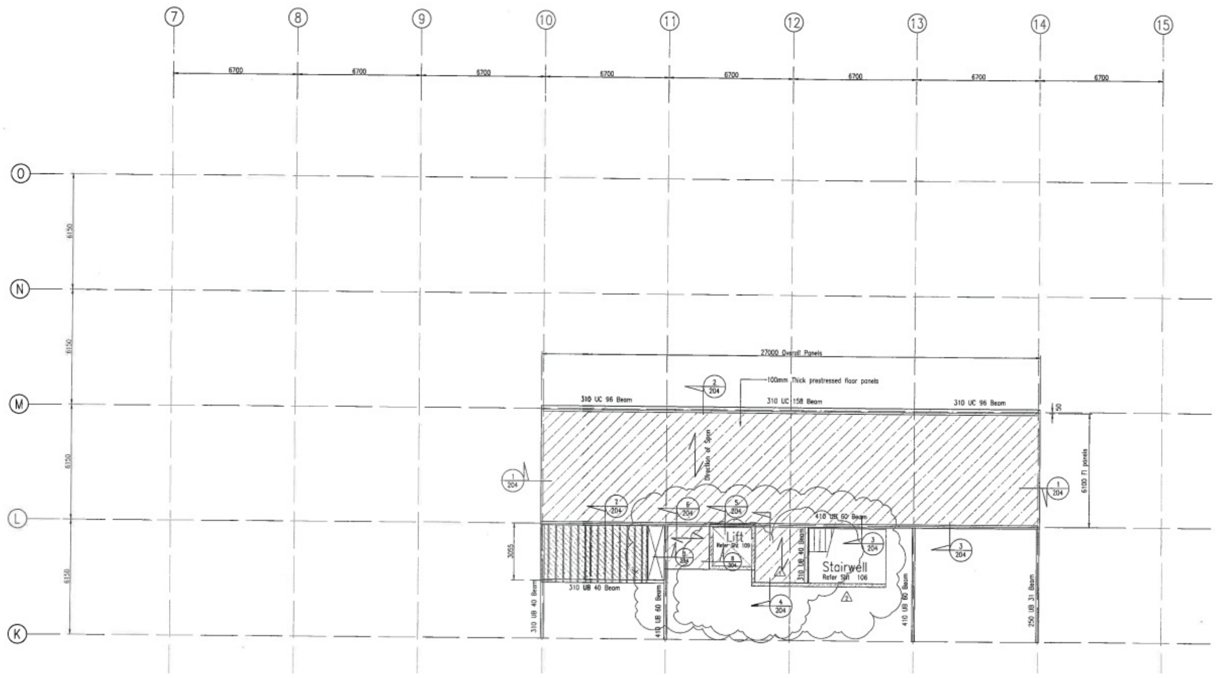


Figure 14: Current Plant Room Floor Plan

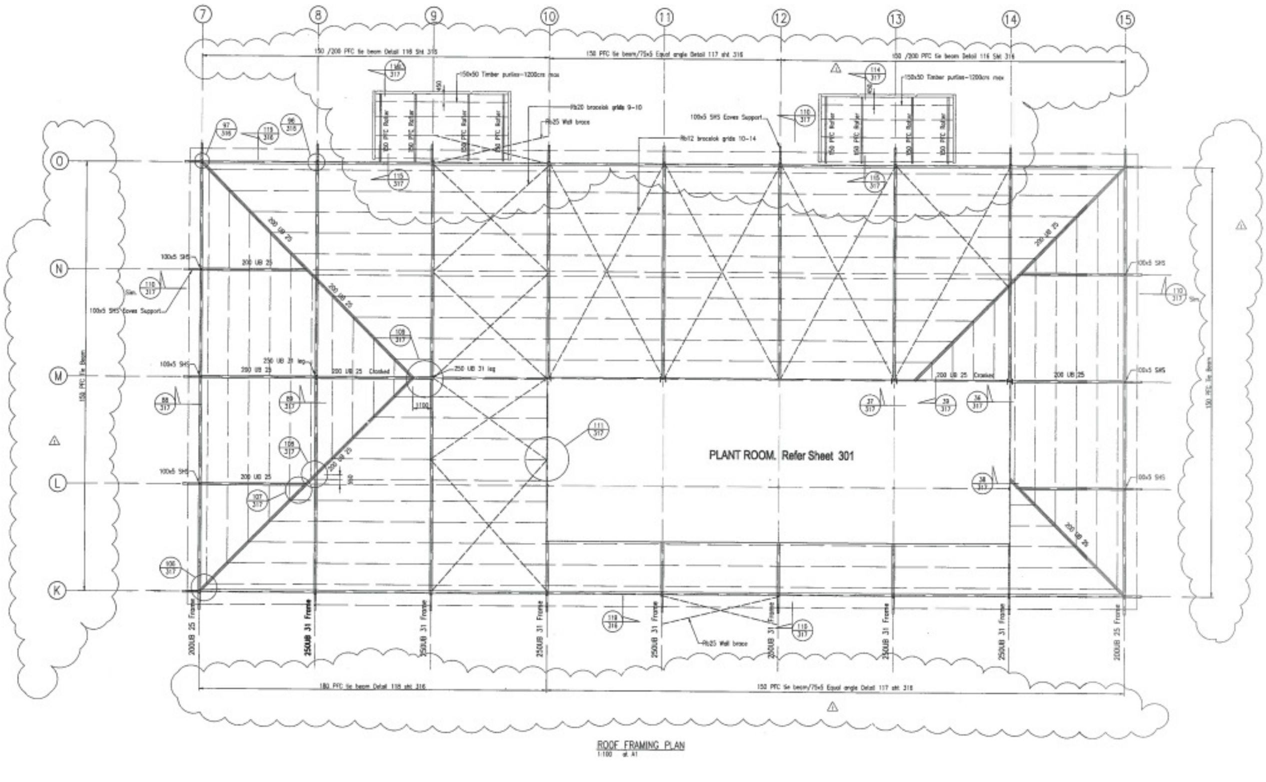


Figure 15: Current Roof Structure

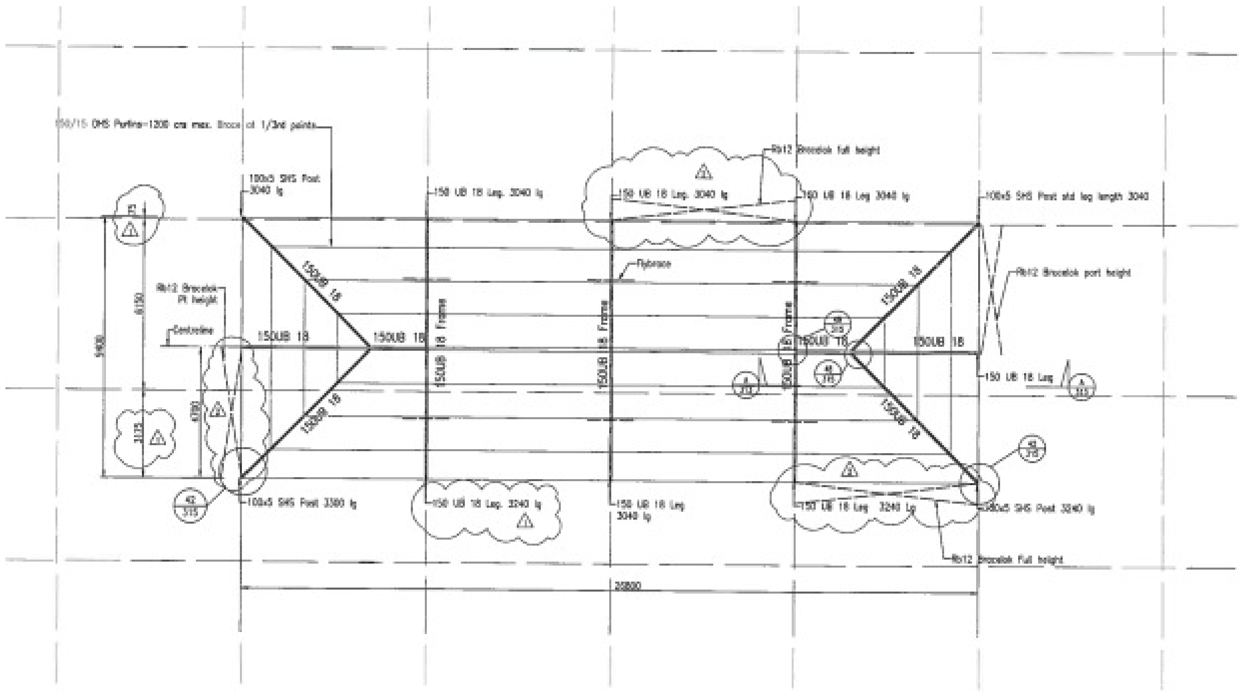


Figure 16: Current Plant Room Roof Structure

The 2004 alterations were designed to an elastic lateral coefficient of 1.05g using the standard for Loading at the time, NZS4203:1992. This compares to the current demand for an IL4 structure based on NZS1170.5:2004 of 2.11 g for full code requirements and for an elastic structure. The site specific probabilistic seismic hazard assessment (PSHA) prepared by GNS Science (McVerry & Buxton, 2012) defines an elastic seismic coefficient of 1.45g. By comparing these coefficients, it can be seen that the design target of the 2004 alterations would represent only 50% of NZS 1170.5 and 72% of the spectra provided by the PSHA for an IL4 structure. For an IL3 structure, the design loads used in 2004 are 70% for the NZS1170.5 spectra and 96% for the PSHA.

The new floor areas at the first floor and plant room have been designed to the following imposed loads:

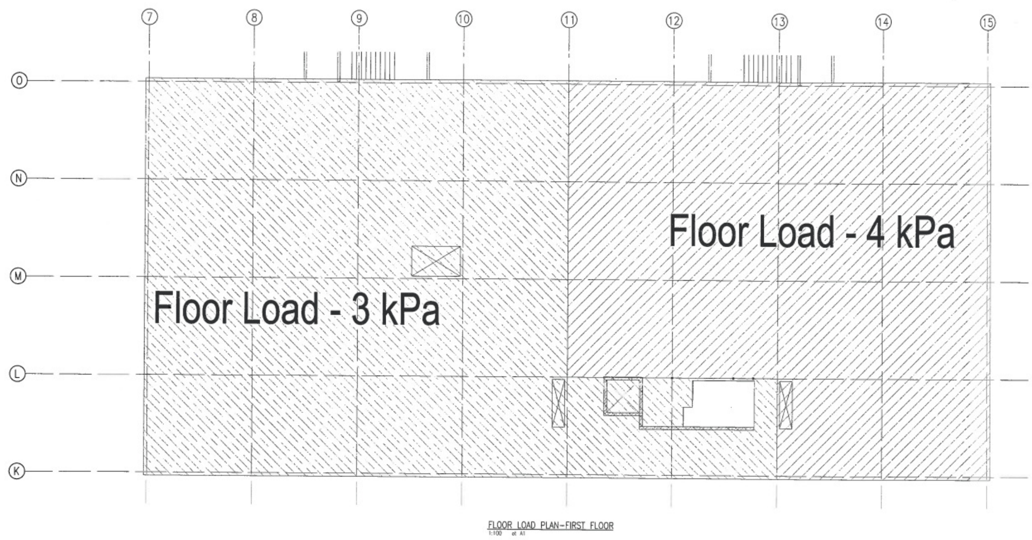


Figure 17: Imposed loads on First Floor

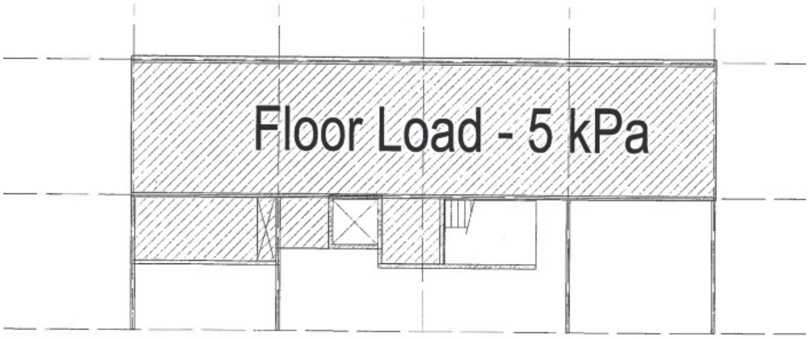


Figure 18: Imposed loads on Plant Room

These loads and their distribution have been considered when defining the seismic mass of the building.

The current structure presents a high level of redundancy at every storey as the lateral strength is provided by several items and does not rely on a few elements. As can be seen in the above plans , bracing to the ground floor comprises of more than 15 walls in each direction, bracing to the first floor comprises of more than 10 braces in each direction, and bracing to the roof structure is shared by all the frames due to the rigid roof diaphragm. The only exception to this is the bracing of the roof itself in the longitudinal direction, where it relies on tension only bracing at a single bay at each side of the building. Below this new lateral structure built in 2004, the original concrete moment frames still remain, with good ductile detailing but without sufficient strength to resist current code loading or stiffness to share lateral loads with the added bracing elements. As the concrete masonry shear walls and steel CBFs provide a stiffer load path, it will be considered that all the lateral loads are transferred through the new elements and that the original moment frames remain only as gravity frames with no practical contribution to the base shear due to their greater flexibility. A numerical model has been prepared as an independent check and confirms this behaviour, with more than 85% of the lateral loads being transferred by the first floor CBFs.

In terms of building irregularities, the following have been found in line with the requirements of NZS1170.5:2004:

- Vertical Irregularities:
    - Weight mass irregularity: Exists between the Roof level and the first Floor.
    - Vertical Stiffness Irregularity: Exists Between the Roof level and the first floor and between the first floor and the ground floor.
    - Discontinuity in capacity: exists between each storey although not critical.
    - Vertical geometric irregularity: Not present.

- Plan Irregularities:
    - Horizontal Plan Irregularity not present.
    - Out-of-plane offset of lateral force resisting walls: not present, although there is offset between the walls and braces.
    - Torsional Sensitivity: Structure torsionally sensitive.
    - Torsional instability of ductile buildings: Not present.

The building has a vertical irregularity at the roof level. Therefore, the roof will be considered as a part, and the main building as the main structure. As the seismic height of the structure is less than

12m, it will be analysed with the equivalent static method, despite other irregularities existing within the structure.

The seismic base is at the ground floor level, at the top of the basement walls. This is because the shear walls bracing the ground floor exhibit a very rigid behaviour with a natural frequency of less than 0.05s. It is expected that this stiff behaviour will not amplify earthquake shaking and therefore it will be only subjected to the peak ground accelerations (PGA) and the shear demands from the dynamic response of the structure above. Therefore, the first-floor level is to be considered the first mass of the building, and the roof and plant room, the second.

It is worth mentioning that during the 2004 alterations two external staircases were erected using precast concrete walls. As it will be further discussed in the results section, these elements will also be analysed but do not form part of the lateral load resisting system of the building as a whole.

2.2.1 Diaphragms

The diaphragm for the ground floor remains as original, with a two-way in situ reinforced concrete slab. When changing from a moment resisting frame to a walled structure, it is expected that forces will concentrate at the walls and therefore there will be need for collectors and chords on the diaphragm. However, the even distribution of walls around the building will mitigate this and minimise stress concentrations.

For the first floor, the diaphragm will be represented by the 100mm thick in-situ concrete topping on top of the precast concrete beams, reinforced with non-ductile 665 mesh. Starter bars have been provided on the first floor at almost every beam, with a typical D16 dowel every 600mm, dowelled into the existing concrete beams. As above, it is expected that due to the number of braces provided, loads will not be highly concentrated and therefore, diaphragm demands will be kept to acceptable values.

The main roof diaphragm will behave as rigid in both directions due to the presence of tension only cross bracing as shown on Figure 15. The floor of the Plant Room has an in-situ diaphragm with non-ductile mesh in a similar fashion to the first floor. The roof of the Plant Room will behave as flexible as there are no cross-bracing present.

Due to our understanding of the different diaphragms of the building, a diaphragm assessment has not been carried out as it will not bring further value at this stage. However, a diaphragm assessment and design may be required if a detailed seismic strengthening scheme is developed.

2.2.2 Building Condition

The building, as inspected, shows a good overall condition with no visible structural damage due to any durability issues. As mentioned before, there are some cracks present in both the Plant Room and the first floor slabs. It is believed this cracking is shrinkage related as no construction or shrinkage joints were observable during the site visit.

2.3 Assumptions

The following assumptions were made during the process of this assessment. These assumptions are due to the lack of information on the building and are based on the best judgement of the authors at this point in time. The results of this report are subject to the accuracy of these assumptions:

- Undrained shear strength of soils is 30kPa (as per 2004 design features report). Soils in undrained condition under earthquake loadings.
- Material properties based on historical records as per table below.
- Bracing layout and connections as per structural drawings. Correctness of drawings could not be confirmed fully during the site inspections performed by Opus.
- No bond failure in chemical anchors as no information is present for the epoxy characteristics or installation records.
- Existing concrete frames do not contribute to the lateral strength of the building.
- Internal stair walls not considered in the analysis as they do not continue to the ground floor.
- Piles have sufficient capacity to withstand earthquake demands.

In terms of the sensitivity of the results to these assumptions, the assumption that has the most severe impact on the results is the undrained shear strength of the foundation soils, which is based on the original 2004 design features report. This assumes that the foundation soils are exhibiting an undrained behaviour under rapid loading. However, no confirmation of a high water table or saturated soils has been made on site.

3 Detailed Seismic Assessment

3.1 Assessment Methodology

The New Zealand standard methodology for assessing the earthquake performance of existing buildings is specified in guidelines that were prepared by the Ministry of Business, Innovation and Employment and the New Zealand Society for Earthquake Engineering[3] jointly with SESOC and NZGS. The force-based methodology described therein will be followed during this report. However, structural capacities of block masonry walls will be used based on current standards, adjusted to reflect the probable material strengths. Sections A, C1, C2, C3, C4, C6 and C10 were used during this assessment.

Results will be presented for an IL4 and IL3 structure, bearing in mind that the client will review the building importance level (and hence future building post-diaster functionality) based on these results.

3.2 Earthquake Demands

The expected displacement ductility of the system and the chosen analysis technique were taken into consideration to determine the best approach to modelling the earthquake demands. As the force-based procedure has been used, the acceleration response spectra with the equivalent-static methodology have been applied to determine the seismic demands. The low period of the building, low height, regularity and expected ductility demands make this selection suitable. Acceleration spectra from the GNS site-specific PSHA has been used in this assessment instead of those from NZS1170.5. As detailed below, the Plant Room and roof structure have been analysed separately, using the demands as a part from NZS1170.5:2004 combined with the PGA values from the PSHA (McVerry & Buxton, 2012).

3.2.1 Seismic Mass

To determine the seismic mass of the building, a probable value of the imposed loads will be used in addition to all the self-weight and superimposed dead loads. The imposed loads that were included in the seismic mass are shown in Table 1. As described earlier in this document, the seismic base will be considered at the ground floor, therefore the original ground floor will not be considered as an additional mass or storey in the analysis. However, its mass has been calculated to take into account its additional demand at the base of the basement walls.

[3] The Seismic Assessment of Buildings, July 2017.

Table 1: Imposed Seismic Mass

Floor Level	Imposed Weight Q_i	Combination Factor Ψ_E
Roof	0.25 kPa	0.0
Floor Live 1 (Firs Floor)	3.0kPa	0.3
Floor Live 2 (First Floor)	4kPa	0.3
Ground Floor	3kPa	0.3
Plant Room	5kPa	*1,0.3

*For the plant room, an estimate of the weight of the equipment present has been performed independently.

An area reduction factor of 0.5 has been used for the floor imposed loads. From the Occupancy loads on Table 3.1 NZS 1170.1, a recommended load of 2kPa is suggested for hospital wards and 3 kPa for operating theatre, X-Ray rooms and laboratories. Therefore, a value of 3kPa will be used as imposed loads for the seismic mass. In the 2004 design, approximately 40% of the first-floor area was designed for an imposed load of 4kPa. This additional imposed loads, has been considered in the seismic mass of the building.

The total weight of the building is 24854kN, with a distribution of 10782kN, 11804kN and two 268kN for the ground, first floors and plant floor/roof respectively. By considering the seismic base at the ground floor, the total seismic mass of the building is approximately 14MN.

3.2.2 Horizontal Seismic Coefficient

The following parameters have been considered to define the acceleration spectra from NZS1170.5:2004:

Table 2: Parameters for Seismic Loads - ULS

Parameter	Value	Comments
Site Subsoil Class	D	Deep Soil Deposits, can be E, but will not change spectra for the building periods involved
Z	0.39	Seismic hazard factor for Hastings
R_u (ULS)	1.8 1.3	Importance Level 4 – 1/2500 yr Importance Level 3 – 1/1000 yr
N(T,D)	1.0	<20 km from nearest major fault, but T<1s.

With the above parameters, an elastic seismic coefficient of 2.11g is obtained at IL4. The total base shear that the building will be subjected to, will depend on the dissipating characteristics of its lateral load resisting mechanisms and the type of construction.

However, the site-specific design spectra from the GNS Science report recommends an elastic coefficient of 1.45g for IL4 and 1.05g for IL3. This seismic demand will be used in this assessment.

As a reference value, the 2004 extension and strengthening design employed a seismic coefficient of 0.86g for nominally ductile loading, and 1.06 for an elastic loading, which leads to the following comparison table:

Table 3: Comparison of Design Forces

Period	Elastic Seismic Coefficient used in the design [g]	%NBS when compared to current loading [%]
Original Construction (1980)	0.192	13 (IL4) 18 (IL3) 22 (IL2)
2004 Alterations (NZS 4203:1993)	1.05 (0.86 nominally ductile)	72 (IL4) 96 (IL3) >100 (IL2)
Current (PSHA made by GNS for the site)	1.45 (IL4) 1.09 (IL3) 0.86 (IL2)	N/A

A first conclusion can be drawn from the above table: the building elastic design spectra in 2004 reaches 72%NBS of the current GNS site-specific design spectra. However, as a nominally ductile level of action has been used for the 2004 design and our assessment shows that the building currently exhibits no ductility, this reduces to 59%NBS. The situation is better if we consider an IL3 building, where it is expected that the building would be more than 70%NBS. If we compare the same design forces to the spectra on NZS1170.5:2004, these values are even lower. It must be understood that these values represent the design intent at the time of design. However, the presence of structural weaknesses will reduce the values of %NBS to be achieved. Part of the objective of this report is to find these structural weaknesses and obtain an accurate %NBS that represents the current structure performance and status of the building. The values presented above are provided as an example only (to show the effect on %NBS rating of the development of design code loading over the life of this building) and do not represent the current calculated %NBS rating of the building.

If the building is considered IL4, an SLS2 level of performance needs to be evaluated. This performance objective means that the building should withstand a design earthquake for an IL2 structure with limited damage of structural and non-structural elements to guarantee that the building can maintain its function following a major earthquake. The following parameters for SLS2 seismic loads under NZS1170.5:2004 apply:

Table 4: Parameters for Seismic Loads – SLS2

Parameter	Value	Comments
Site Subsoil Class	D	Deep Soil Deposits, might be E, but will not change spectra for the building periods involved
Z	0.39	Seismic hazard factor for Hastings
R_u (ULS)	1.0	SLS2=ULS (IL2) – 1/500yr
N(T,D)	1.0	<20 km from nearest major fault but T<1s.

The elastic seismic design spectra for a 500yr return period recommended by GNS is 0.86g, and this value will be used in the assessment. If we consider an Sp=0.9 and a nominally ductile level of ductility, the total horizontal design coefficient reduces to 0.68g. NZS1170.5 allows a maximum ductility of 2 to be exhibited during an SLS2 event. Again, the part coefficient for SLS2 will be in the order of 1.37g.

As part of this study, and as explained below, the Plant Room and roof structure will be analysed as a separate part, as per NZS1170.5 Chapter 8. The building has a vertical irregularity in terms of tributary mass between the roof and the first floor. However, the highest building period is less than 0.4s, therefore, the equivalent static distribution can be used to model the earthquake demands (see 6.1.3 NZS1170.5). As the roof and Plant Room is lighter than the rest of the building, only 16% of the mass, an analysis of the demands at each level was undertaken, comparing two scenarios:

- Distribution #1: Considering the Roof as a Part, therefore subjected to accelerations from Chapter 8 NZS1170.5.
- Distribution #2: Considering the Roof as second mass and storey, and providing the equivalent static distribution with the 8% additional mass distributed at the top.

The above analysis draws the following results for the distribution of seismic demands at each mass:

Table 5: Distribution of Seismic Demands for ULS (IL4) – Sp=1, μ=1.

Floor Level	Distribution #1 [kN]	Distribution #2 [kN]
First Floor	20404*	13651
Roof + Plant Room	8165	6753
Total	**20404**	**20404**

*: Accumulated base shear from Roof and First Floor masses.

As it can be seen from the above results, the distribution No. 2 produces 1400kN less demand onto the roof and plant room. Therefore, the roof and plant room structures will be considered as a separate part and analysed independently as per distribution #1.

Now, if we consider that the structure is an IL3 building, the following distribution of seismic demands are obtained:

Table 6: Distribution of Seismic Demands for ULS (IL3) – Sp=1, μ=1.

Floor Level	Distribution #1 [kN]	Distribution #2 [kN]
First Floor	15338*	10261
Roof + Plant Room	4508	5077
Total	**15338**	**15338**

*: Accumulated base shear from Roof and First Floor masses.

It is worth commenting that the building is mostly of regular shape and that there are no higher modes expected to develop that will change the distribution of seismic demands from what it is shown on the above tables. Also, based on the 2004 design features report, the overall ductility demands for the ULS will be deemed to be low for an acceptable performance, justifying the use of the equivalent static procedure.

Accidental eccentricity and structural eccentricity have not been considered in the hand calculations prepared. However, due to the regular shape of the building it is expected that the scores obtained for each element will not vary considerably. Material Properties

The following materials properties were used during the assessment:

Table 7: Material Properties

Material	Lower Characteristic Strength	Probable Strength
Concrete (1980)	f'c=30MPa	f'cp=45 MPa (E=29171MPa)
Steel (1980)	f'y.1=300MPa	f'y.1p=345MPa
Reinforcement (1980)	f'y.r1=275MPa f'y.r2=380MPa	fy.p.r1=297MPa fy.p.r2=410MPa
Concrete (2004)	f'c2=17.5MPa	f'cp2=25 MPa
Reinforcement (2004) HD	Fy.r3=500MPa	Fyp.r3=540MPa
Reinforcement (2004) R	Fy.r4=300MPa	Fyp.r4=324MPa
Reinforcement (1985)	f'y.2=275MPa	fy.p1=297MPa
Structural Steel (1985)	f'y2=227MPa	fy.p2=245Mpa
Concrete Masonry	f'c.cb=12MPa	f'p.cb=18MPa

A core sample taken from 1980's concrete, shows a 50MPa compressive strength. However, this is only one sample and no statistical conclusion can be taken. A 45MPa probable strength will be used.

4 Results of the Detailed Seismic Assessment

Most of the analysis done to establish the current rating of the structure was undertaken via hand calculations, identifying the different load paths and capacities for the new lateral elements constructed during the 2004 alterations. The total capacity and ductility of the system has been based on the available redundancy and the different failure mechanisms that their elements represent. The description of the analysis begins with the shear walls at the basement, then the analysis of the different braces and finally the analysis of the roof structure and Plant Room. As an additional assessment, the staircases to the exterior of the original building have been analysed.

4.1 Basement Shear walls – Ground Floor Bracing

During the 2004 works, a number of RCM (Reinforced Concrete Masonry) walls were constructed at the basement level, in order to provide a direct load path for the shear forces from the steel CBFs above and also to contribute to the bracing of the ground floor. The typical elevation for the walls is represented in the following figure:

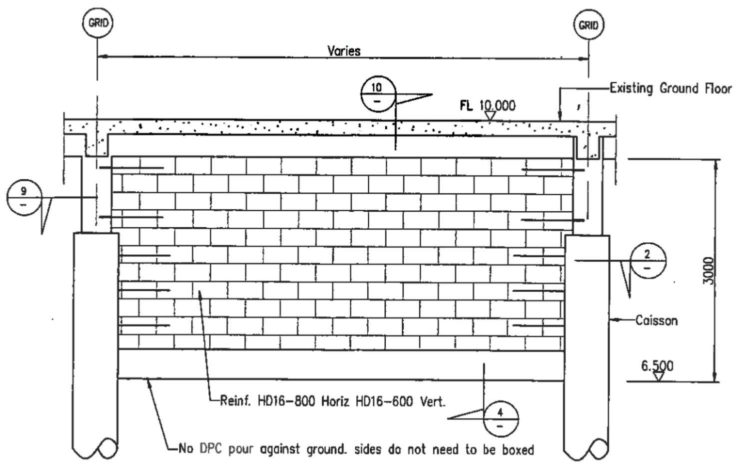


Figure 19: Typical Basement RCM Wall elevation

Most of the walls at the basement are similar to the detail above. However, some of them have doorways in order to provide access to the full extent of the basement plan. Also, in many cases, there are openings for ducts and services that reduce the capacity of the walls. Most of these walls have an in-situ concrete element at the top course, cast in-situ to dowel in the wall against the ground floor beams or columns. This dowelling and concreting of the wall means that the shear wall can act as an infill due to its confinement by the surrounding frame.

The foundations of these walls are in situ concrete elements, cast between the piles. These footings are shallow and overlay site concrete used as a ground improvement. The typical dimensions of the foundations are 400mm deep by 2100mm wide.

From the analysis and original design features report, it is understood that the walls were designed as shear walls. For this mechanism to develop, the following load path is required:

Floor slab and beams -> dowels -> wall -> foundation -> piles

The design intends to transfer shear to the RCM walls by means of a shear friction mechanism provided by dowel action at the interface between the new walls and the original in-situ concrete beams by reinforcement epoxy grouted into the beams and cast into the walls. From our analysis, we have determined that the weak link in the load path is the dowels at the top of the wall. In many cases, the cast concrete infill at the top of the wall is incomplete, and therefore these dowels are not properly encased in concrete and are inefficient. Where the dowels are encased, the shear friction mechanism requires that the bars should be properly anchored at each side of the critical plane (wall to beam interface) so they can develop their yield stress. However, our analysis shows that the dowels may fail prematurely and in a brittle fashion, through concrete break-out failure of the epoxy grout. Therefore, dowel action at the top of the wall can be considered non-ductile for the analysis as it will have a quickly degrading capacity once the failure has occurred.

Once the original mechanism fails, the shear walls can be still effective to transfer shear by compressive strut action of the infill wall from corner to corner, provided they are fully confined within the existing concrete frame. Walls with door openings, large opening for services, and walls not extending for the full width between piles cannot develop this mechanism. In this case, shear will be transferred by bearing of the columns and beams on the walls, without the need for the top dowels. The load path will then be:

Floor slab and beams>concrete column, beam and node>strut action through wall >concrete column beam and node>foundation>piles

As in the previous case, the weakest link in the chain will dictate the amount of shear that can be withstood by the system and the maximum level of displacement ductility that can be achieved. In most of the cases where the strut is developed, shear failure of the beams and columns does not occur, but the foundations are ineffective in transferring the total amount of shear to the ground by cohesion at their bases and sides. Therefore, in each case, sliding of the footing is the prevailing mechanism with the consequent loading of partially exposed piles. This loading of piles is limiting the strength of the walls in most of the cases. More detail on these findings can be seen in each individual wall calculation for this mechanism found in the appendix. However, this mode of failure will not be considered as the whole foundation system needs to fail in order to generate a collapse. This total capacity was calculated to be in the order of 23MN which is more than the capacity of the walls themselves.

Also, it has been found that the wall foundations do not have enough mass to counteract the vertical demands generated from the struts. Therefore, these additional forces, compression at one end and additional tension at the other end must be materialised by the piles. Tensile pile capacity has not been assessed but, due to the shallow embedment of these piles and partial exposure by 2004 works, it is expected to be insufficient. As a result, the rating has not been limited to this behaviour, which is reflected in one of the assumptions of the detailed seismic assessment.

A list of each one of the shear walls with their characteristics, geometry, opening and comments can be found in the appendix. These properties were the input data for each one of the individual calculation sheets.

Figure 20: Example of RCM wall with incomplete infill and lack of dowel action at top.

The concrete lift shaft was also analysed. In this case, the lift shaft wall is poorly connected to the surrounding slabs, and therefore there is limited capacity for load transfer to these walls and pounding is also likely to occur. Also, the wall longitudinal reinforcement is not fully lapped to the foundation dowels, which means that limited flexural capacity is expected to develop at the base of the wall (classical bending capacity allows for yielding of reinforcement along almost of 50% of the wall length). Therefore, contribution of this wall to the overall strength has not been considered. In addition to this, there is potential for pounding between the different floors and the lift shaft, but the consequences of this are minimal with probable concrete spalling around the opening only.

The following table summarises the capacities obtained for each one of the walls in the longitudinal and transverse directions. The analysis is detailed in the appendix. The first value is the shear capacity that arises as a result of strut action (when available), top dowels, shear friction and pure shear of the block wall. The second analysis determines whether the maximum capacity of the shear wall can be taken by the foundation pad under it or the balance of the shear is transferred to the column where the column should resist that balance of shear, which is expressed as a percentage. The percentage values for the piles are an interpretation that the balance of the shear from the foundation will be resisted by the piles. If the percentage is less than 100%, it means that the pile will be stressed past its full bending capacity, leading to a local plastic hinge at the wall base. Also, a third analysis for the impact of the strut action on columns and beams has been done to account for any undesirable failures. This analysis follows a capacity design rationale as it is foreseen that due to the nature of the return period of the actions, it is expected that the walls will exceed their capacities for IL4 levels of loading.

Table 8: Assessment Results for Basement Walls bracing the ground floor – Longitudinal Direction

Wall	Shear Capacity (kN)	Mode of Failure	Shallow Foundation Capacity (kN)	Foundation (%)	Comment
1	1602	Strut Action – Some ductility	922	>100	
2	403	Dowel Failure – Brittle	922	>100	
3	1602	Strut Action – Some ductility	461	34	
4	1602	Strut Action – Some ductility	461	34	
5	1602	Strut Action – Some ductility	263	25	
6	377	Strut Action – No ductility	263	>100	Shear failure of column due to strut
7	406	Top dowels, brittle	461	>100	If strut happens, shear failure of column and beam
8	1602	Strut action – some ductility	461	34	
9	360	Top Dowels - Brittle	461	>100	
10	1602	Strut action – some ductility	461	34	
11	1526	Top Dowels – Brittle, strut action around opening also possible	461	>100	Shear failure of column possible
12	1900	Strut Action	461	34	
13	450	Dowel Action – Brittle	461	100	Shear failure of column may happen
14	1900	Strut action – some ductility	461	34	
15	460	Dowel Action – Brittle	461	100	Shear failure of column may happen
16	1900	Strut action – some ductility	461	34	
17	1900	Ditto	461	34	
18	1900	Ditto	461	25	
19	1900	Ditto	461	25	
Total	**17502**		**9220***		

*Total foundation capacity is represented by the foundation capacity in all the foundation pads beneath the walls under consideration only. However, some capacity of the foundations of the walls in the perpendicular direction will be mobilised as well (9220+0.6*5810) plus shear capacity of the piles under the walls (14x177kN) for a total of 15184 kN.

Table 9: Assessment Results for Basement Walls bracing ground floor– Transverse Direction

Wall	Shear Capacity (kN)	Mode of Failure	Shallow Foundation Capacity (kN)	Foundation (%)	Comment
20	227	Dowel-Brittle	415	>100%	
21	0				
22A	142	Dowel – Brittle	415	>100	Shear failure of column possible.
22B	254	Dowel – Brittle	415	>100	
23A	134	Dowel – Brittle	415	>100	
23B	295	Dowel – Brittle	415	>100	
24A	134	Dowel – Brittle	415	>100	
24B	295	Dowel – Brittle	415	>100	
25	1533	Strut action – some ductility	415	35	Shear failure of beam, non critical.
26	1533	Strut action – some ductility	415	35	
27	513	Dowel – Brittle	415	>100	
28	1533	Strut action – some ductility	415	35	
29	513	Dowel – Brittle	415	>100	
30	1533	Strut action – some ductility	415	35	
31A	134	Dowel – Brittle	415	>100	
31B	194	Dowel – Brittle	415	>100	
32	513	Dowel – Brittle	415	>100	
33	0				No action
34	0				No Action
35	0				No Action, dowels exposed
Total	**9523**		**5810****		

**Total foundation capacity is represented by the foundation capacity in all the foundation pads beneath the walls under consideration only. However, some capacity of the foundations of the walls

From the above results, it can be seen that the capacity in the transverse direction is lower than in the longitudinal. This is due to the fact that there are more walls with openings in the transverse direction, meaning that the masonry infill compression strut is not always materialised and therefore the shear capacity of the wall depends on the limited shear capacity of the dowels.

Results also show that when no strut action is possible, shear failure of the dowels will occur with the consequence of breaking out the concrete, which is a brittle failure. This means that most of these walls will have degrading strength once this capacity has been achieved. Therefore, the addition of the shear capacities at the bottom of the table is an unconservative estimate of the maximum capacity that the system can have.

The low values of pile capacity mean that the piles will be subjected to lateral loading. A plastic hinge is expected to form in the pile below the wall shallow foundations. If we consider that the piles between the walls under study for each direction contribute to the lateral load, in addition to all the shallow footings, the foundation maximum capacity may be around 15MN in the longitudinal direction and 14MN in the transverse. This means that in the transverse direction, the capacity is dominated by the capacity of the wall but in the longitudinal direction the capacity is limited by the foundations. If the piles were to fail due to excessive curvature ductility demands or P-Delta effects, it is unlikely to affect life safety as there will be some support from the new RCM walls and their foundations. However, the building will then be supported on the uncontrolled fill and potentially subject to increased global and differential settlement, affecting building damage and post disaster function.

In some of the walls, the strut action will lead to a shear failure of beams and columns. Failure in shear of the columns might not be critical as the shear walls will still be in place, albeit cracked, and could provide some support for any lost column.

By considering the capacity of the walls only, the rating will likely be 53%NBS (IL4) in the transverse direction. Most of the modes or failure of these walls have no ductility and are brittle. In the longitudinal direction, 87%NBS (IL4) is the likely rating. In terms of the SLS2 performance objective, it is limited to a 93%NBS (SLS2) in the transverse direction, with over 100%NBS (SLS2) in the longitudinal direction.

In terms of the results considering the building to be an IL3 structure, the rating of the ground floor is expected to be 69%NBS (IL3) in the transverse direction and 100%NBS (IL3) in the longitudinal direction.

Drifts obtained at the ground level proved to be minimal and therefore these will not affect the above results.

The conclusion of this analysis is that the behaviour of the below ground floor structure will be complex. The above stated numbers are maximum values as it assumes that all the walls will find their capacities at the same time. Due to eccentricities of earthquake loading and relative location within the building, it will be likely that walls will not be loaded to the same extent. After every pulse, the walls that show a brittle behaviour will have degrading strength. Therefore, the rating is expected to be less than the numbers stated above as this brittle behaviour of some walls does not contribute to redistribution and redundancy.

As a summary, the following structural weaknesses (SWs) were identified during the assessment for the shear walls bracing the ground floor:

- Lift Shear walls: longitudinal reinforcement not lapped at foundation, yielding capacity likely not to be reached. Connections to floors very weak, with limited capacity.

- Reinforcement dowels connecting basement shear walls to ground floor beams have insufficient anchoring to the existing concrete. Concrete break-out is likely to occur before bars reach their yield stress. This failure is force-controlled and therefore it exhibits low ductility.

- Strut behaviour is likely to occur in the basement concrete masonry infill panels from corner to corner. Some of these struts can provide shear forces to surrounding beams and columns that can induce failure of these elements.

- When the basement infill walls are not perfectly surrounded by the concrete frame, strut action is not likely to develop and then shear dowel behaviour prevails. Again, shear capacity of the walls is limited by the capacity of the dowels against the existing ground floors.

- New foundation beams have limited capacity against sliding. The balance of shear is then transferred to the piles that will take the shear through bending to foundation soils. This balance can exert high plastic demands on piles with the subsequent loss of gravity support. In the longitudinal direction, the capacity of the system is likely to be limited by the foundation.

- Brittle overall behaviour of basement level with no ductility to allow for redistribution of forces and for maximum capacities to happen at the same time. Therefore, the sum of capacities stated above are a maximum result. This is considered the severe structural weakness of the building (SSW).

- Shear forces through basement infill walls via compression struts will generate tensile uplift forces on piles. Pile embedment can be shallow and may not be able to take these uplifts.

- Some walls do not have a continuous concrete infill and connection to the beams above and are constructed in a poor manner.

- Some walls are not effective as they do not continue to the underside of the beams due to large services penetrations. In these cases, short column effect with subsequent shear failure may occur.

- Internal staircase wall does not continue to the lower level.

4.2 First Floor CBFs

The first-floor mass is braced down to ground floor through an array of steel concentrically braced frames or CBFs. These frames were added during the 2004 alterations, and have steel members to all four sides as well as the diagonal braces. These elements are dowelled into the original concrete beams and columns, with the use of chemical anchors. The slenderness of these elements means that global buckling of the brace in compression is likely to occur, and therefore the system will exhibit a pinched hysteresis with a reduced dissipation capacity. There are two typical braces: Brace Type B extends from column to column and connects to the surrounded frame; Brace Type A does not extend full width of the structure bay at each location, meaning that a vertical strut is placed at some point between columns. These two typical geometries are shown in the following figures:

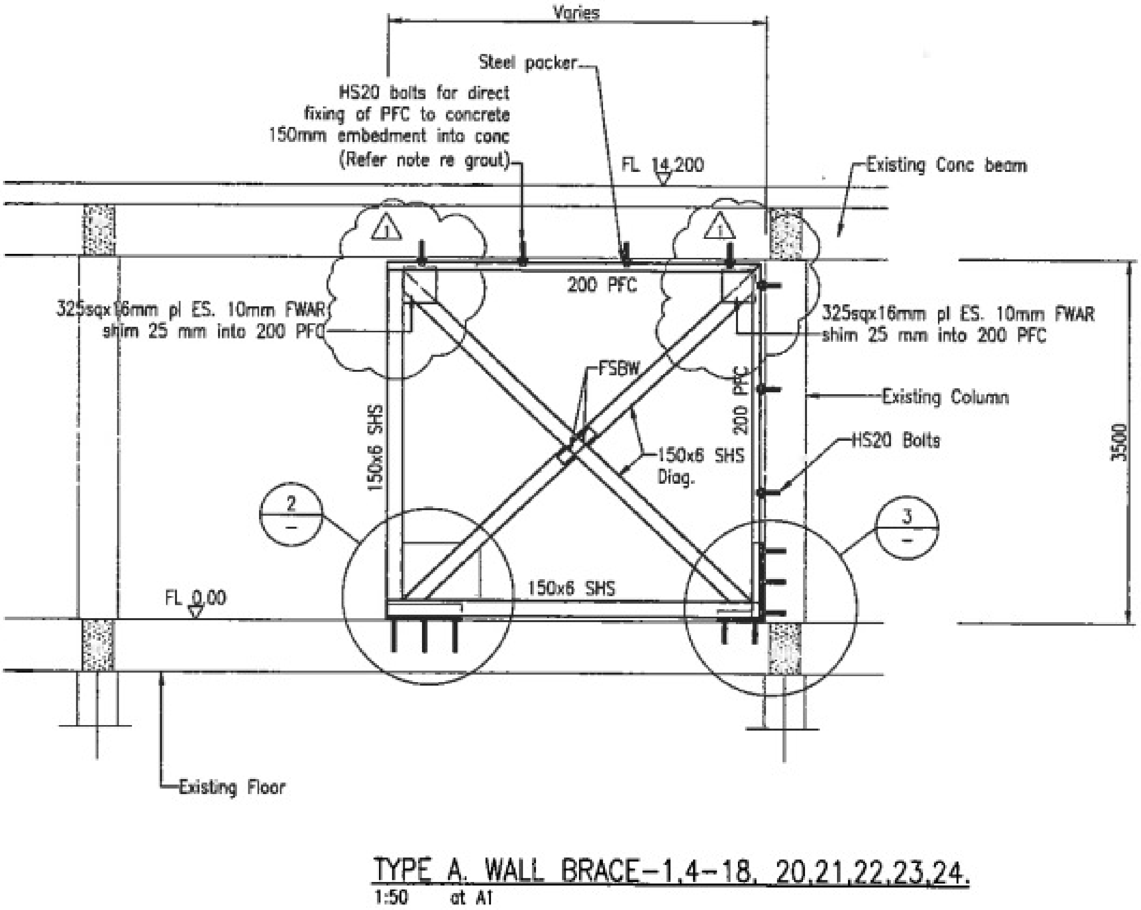


Figure 21: Type A CBF

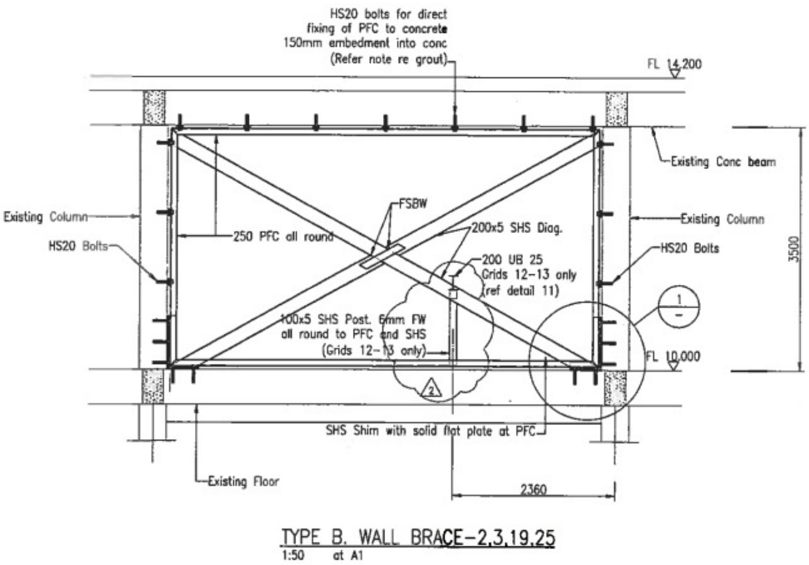


Figure 22: Type B CBF

All of these braces are currently hidden from view behind architectural linings. It is our assumption that these linings haven't been designed to restrain the braces against global buckling and therefore the buckling lengths for the braces will be 0.5L (L is the diagonal length) in the in-plane and L in the out-of-plane directions. Based on the above details, the load path for a load applied at the top of the braces for brace Type A will be:

Floor slab and beams>Bolts to Top Collector and Columns and bearing on Columns>Brace (Global Buckling) >Brace Splice >Details 1, 2 and 3 connection to Beams and Columns > Ground Floor Diaphragm

As can be seen, any failure on this load path will limit the capacity of the brace. Again, a capacity based approach was followed as it is expected that the demands will be greater than the global capacities of the braces for an 2500yr event, with the need for ductility to be developed if a high level of %NBS needs to be achieved.

During our analysis, we have calculated the following capacities for the braces in the longitudinal and transverse directions respectively:

Table 10: Assessment Results for First Floor CBFs – Longitudinal Direction

CBF	Global Tensile Capacity [kN]	Global Compressive Capacity [kN]	Maximum Horizontal Shear due to detailing [kN]	Mode of Failure
4	1282	812	600	Bolts/Weld of connection 2
3	1467	877	800	Anchor Bolts, Concrete Break-out.
2	1467	877	800	Anchor Bolts, Concrete Break-out
1	1282	741	627	Anchor Bolts, Concrete Break-out
13	1282	723	810	Anchor Bolts, Concrete Break-out
11	1282	817	750	Anchor Bolts, Concrete Break-out
12	1282	723	810	Anchor Bolts, Concrete Break-out
24	1282	905	500	Bolts/Weld of connection 2
10	1282	817	750	Anchor Bolts, Concrete Break-out
9	1282	963	425	Bolts/Weld of connection 2
3*	1282	900	550	Bolts/Weld of connection 2
7	1282	859	600	Anchor Bolts, Concrete Break-out
6	1282	970	450	Anchor Bolts, Concrete Break-out
5	1282	970	450	Anchor Bolts, Concrete Break-out
Total	**18318***	**11951***	**8922**	

*=Sum of axial capacities, not horizontal shear.

Table 11: Assessment Results for First Floor CBFs – Transverse Direction

CBF	Global Tensile Capacity [kN]	Global Compressive Capacity [kN]	Maximum Horizontal Shear due to detailing [kN]	Mode of Failure
14	1282.05	777.24	750	Anchor Bolts, Concrete Break-out.
15	1282.05	777.24	750	Anchor Bolts, Concrete Break-out.
16	1282.05	777.24	750	Anchor Bolts, Concrete Break-out
17	1282.05	777.24	750	Anchor Bolts, Concrete Break-out
18	1282.05	900.00	550	Bolts/Weld of connection 2
21	1282.05	790.65	750	Anchor Bolts, Concrete Break-out
25	1466.85	935.31	800	Anchor Bolts, Concrete Break-out
20	1282.05	790.65	750	Anchor Bolts, Concrete Break-out
19	1466.85	951.04	800	Anchor Bolts, Concrete Break-out
22	1282.05	777.24	750	Anchor Bolts, Concrete Break-out
23	1282.05	777.24	750	Anchor Bolts, Concrete Break-out
Total	**14472***	**9031***	**8150**	

*=Sum of axial capacities, not horizontal shear.

Again, these results show that the braces will exhibit a non-ductile behaviour due to their connections to the surrounding structure. Albeit some of the connections have more capacity than the global compression capacity of the braces, it is when the braces will be subject to tension that the connections are expected to fail in a brittle manner. Most of the failures are due to the connection of the braces to the existing concrete via chemical anchors. The mode of failure of these anchors is concrete break-out, which is a brittle failure. It is worth noting that concrete shear failure or face blow-out is expected on those connections close to the edge of the building.

According to the 2004 design features report, connections were designed for the MCE earthquake of NZS4203:1992, and not through a capacity design analysis. This is most likely the reason why connections are limiting the behaviour for an increased earthquake demand based on current codes.

The overall behaviour in both directions is brittle. Once the connection capacities are exceeded, these braces will not be capable of transferring shear or tensile reactions to the concrete frames. Therefore, it is expected that most of the braces will be lost progressively, and it is not clear at this point to what extent. Thus, the sum of capacities expressed in the table above is a maximum capacity for the system in each direction, and it is the lack of ductility of these connections that will lead to degrading strengths with increased displacements. Therefore, we can affirm that the maximum probable capacity for the first floor CBFs is 8922kN in the longitudinal direction and 8150kN in the transverse direction.

The brittle behaviour of the system means that an elastic earthquake demand is suitable for this case. For an IL4 structure, a rating of 40%NBS (IL4) is expected in the transverse direction and 44%NBS (IL4) in the longitudinal direction. In terms of the SLS2 performance objective, a 86%NBS (SLS2) is expected in the transverse direction and 94%NBS (SLS2) in the longitudinal.

If the structure were to be considered IL3, a rating of 53%NBS (IL3) will be obtained in the transverse direction and a rating of 58%NBS (IL3) in the longitudinal direction.

Again, the values above represent a maximum as it assumes that at the capacities of the CBFs will be mobilised at the same time, which is unlikely. Regarding drifts, the numerical model has experienced a drift of 0.5% for the elastic ULS IL4 loading, meaning that the ratings stated will not be downrated due to flexibility of the structure.

In summary, the following Structural Weaknesses (SWs) have been found during the assessment:

- Braces exhibit brittle behaviour through limited capacity of connections to the surrounding concrete frame. Concrete break-out of anchors is the predominant mechanism of failure, although weld failure has also been noted in some cases.
- Some braces do not have a shear wall beneath them, giving rise to high shear demands on beams below and transfer forces in the ground floor diaphragm.
- Precast floor beams at the first floor have a limited seating of 50mm (only a weakness if ductility demands occur at the frame beams with its consequent frame elongation).
- Lack of overall redundancy as progressive failure of braces will eliminate the capacity of those failed. Overall strength by adding individual capacities is a maximum value.

4.3 Plant Room

As part of the 2004 alterations, the original Plant Room was demolished and a new area was added. The new Plant Room is located between gridlines 10 to 14 and M to K. It has a main floor structure comprised of precast concrete beams with an in situ reinforced concrete slab, resting on a steel framed structure. Access to the plant room is reached via the internal stairwell and it is understood that the lift engines and machinery is also accessed at this floor. The roof structure is comprised of steel portals with cold-formed steel purlins and girts.

In terms of the lateral load paths, the roof level is braced via tension-only bracing in the longitudinal direction and steel frames in the transverse. The main floor of the Plant Room is braced via CBFs resisting compression and tension. One of these braces is an EBF (Eccentrically Braced Frame), which will have a slightly different behaviour than the rest. It is worth noting that the CBFs (and EBFs) are not placed in opposing pairs, under the same line of bracing. This arrangement indicates that the bracing will not be effective when the braces buckle, leading to a non-ductile behaviour. Indeed, these braces are in the same orientation, making it a non-desirable layout, as all the bracing will reduce its capacity once the buckling load has been reached, leaving only to a residual post-buckling capacity. A ratechting index of more than 1.15 is expected.

The roof diaphragm can be considered flexible in both directions as there are no bracing elements. The concrete floor diaphragm can be considered rigid as it has an in-situ topping with a non-ductile mesh. The arrangement of the bracing elements may introduce local stress concentrations to the diaphragm. No diaphragm assessment has been performed at this point as it will not modify the rating of the building, nor it its believed by the authors that will be critical.

It is worth noting that drawings show that a small portion of the internal lift RCM wall continues to the roof level. However, it is not described on the drawings how this portion of the wall is connected to the roof level and onto the first-floor diaphragm and therefore it has not been considered in the analysis. In addition, in order to simplify the assessment, an independent evaluation of the seismic capacity of the Plant Room has been made without considering any influence from the main roof structure.

In the following figures, the main structural elements are depicted to understand the general layout of the Plant Room.

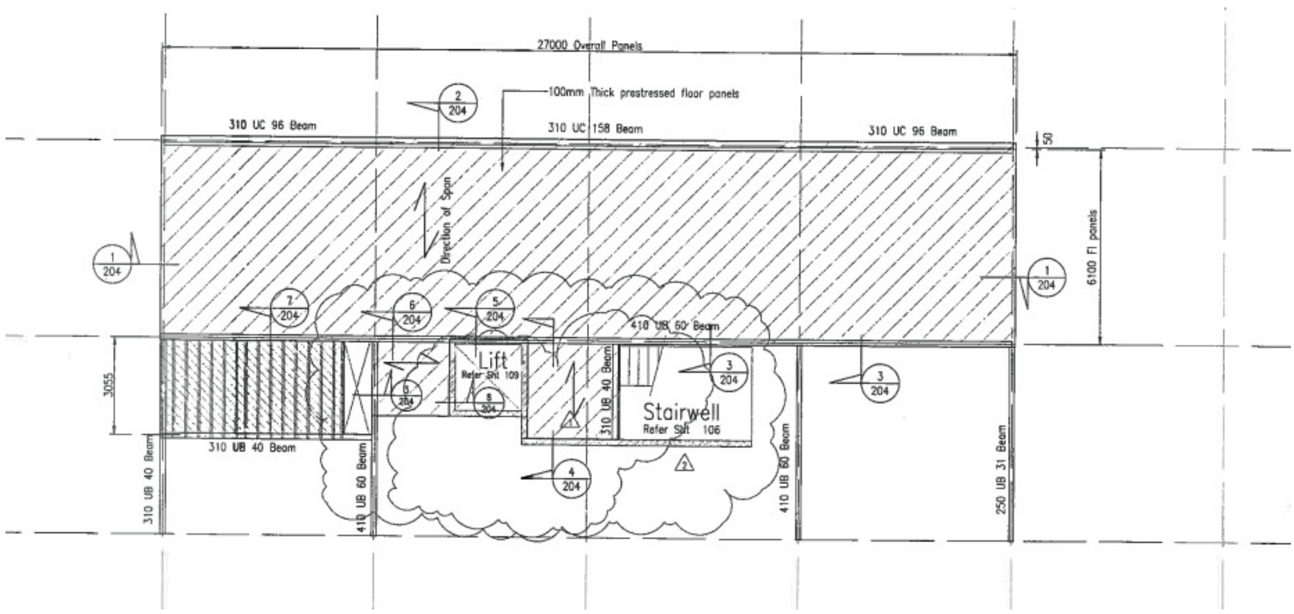


Figure 23: Plant Room Gravity Structure – Main floor level

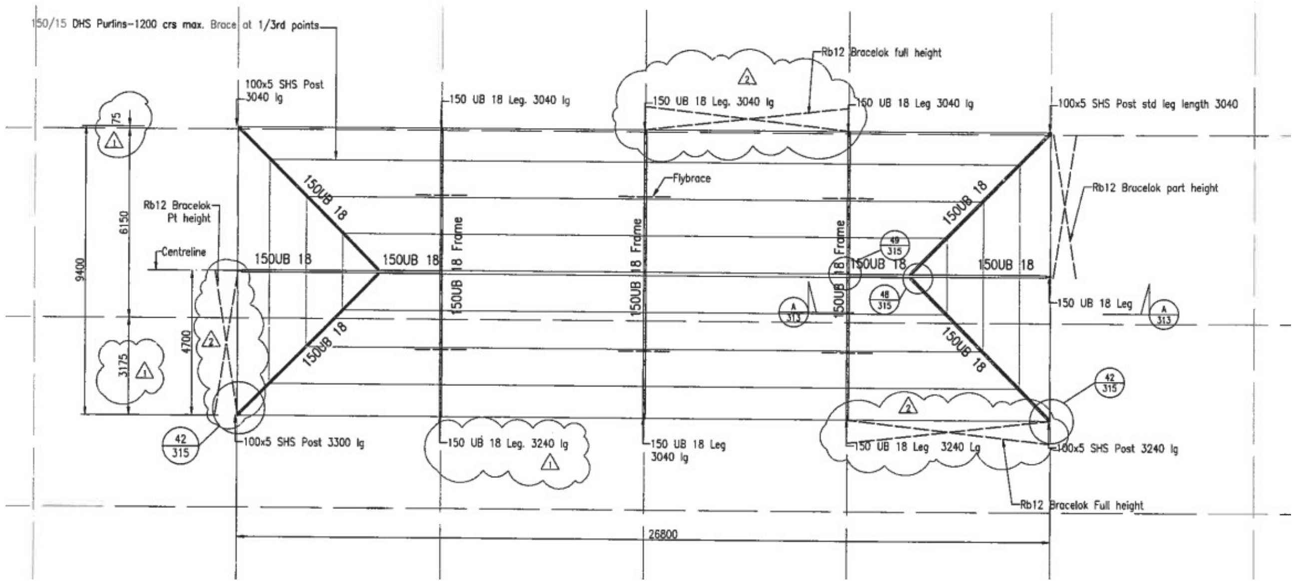


Figure 24: Plant Room Structure – Roof Plan

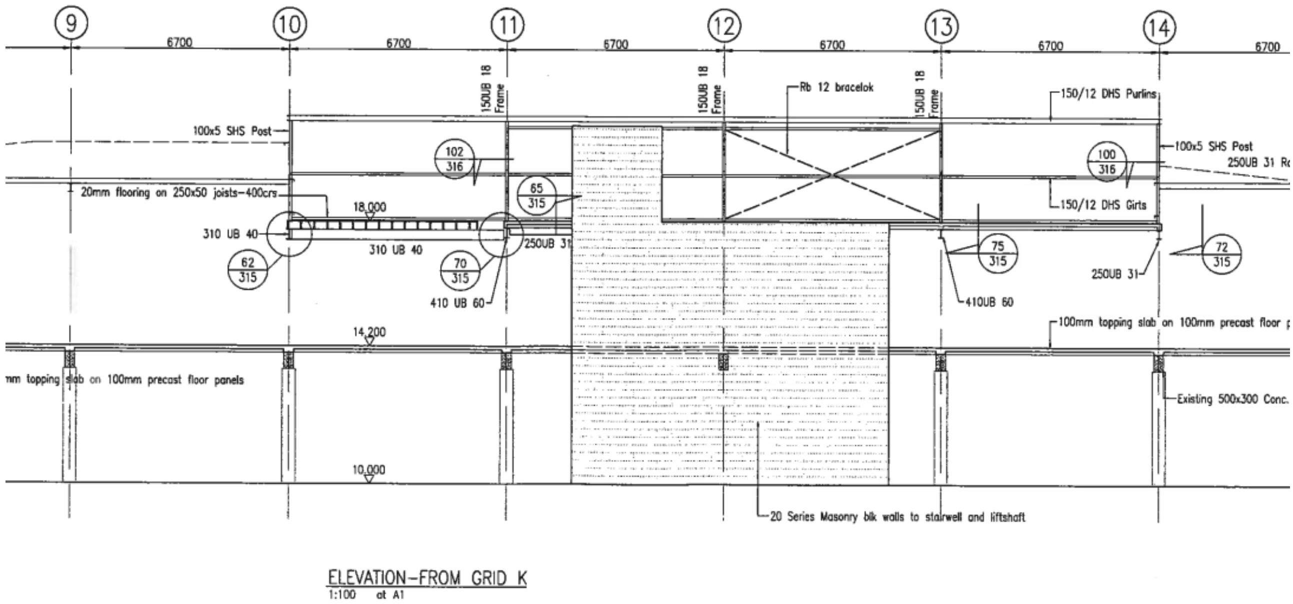


Figure 25: Elevation from Grid K – Connection of wall to structure not confirmed.

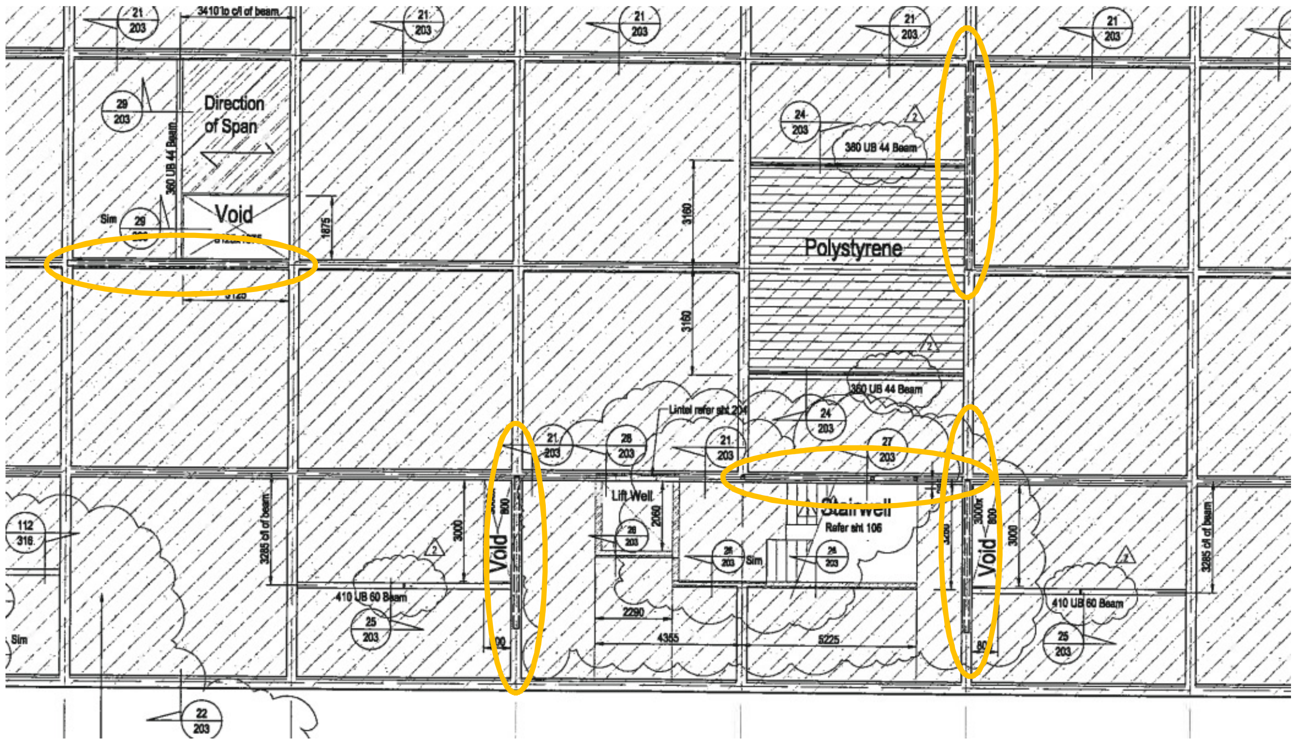


Figure 26: Bracing Layout of Plant Room main floor

The assessment shows that the roof rating is 15%NBS (IL4) in the longitudinal direction, being limited by the formation of plastic hinges in the minor axis of the portal legs as a consequence of the eccentric connection of the braces. Yielding of the longitudinal tension-only braces is expected to occur at a similar level of loading. In the transverse direction, the roof structure is deemed to be 35%NBS (IL4), with the portal failing prematurely in lateral torsional buckling due to their lack of lateral restraint. For IL3 levels of shaking, the roof structure scores 27%NBS (IL3) in the longitudinal direction and 63%NBS (IL3) in the transverse direction.

For the Plant Room floor structure, the assessment shows that in the longitudinal direction, the capacity of the braces is limited by the connection details to the structure, either at the top or the bottom. These details are limiting the capacity of the braces to 560-620kN, and their mode of failure is brittle (i.e., concrete break-out or weld failure). The global compressive buckling strength of the braces is in the order of 620kN, and therefore global buckling may happen before failure of the connections. However, when the braces are pulled in tension, the connections will fail as the tensile capacity is larger and it is likely to be exhibited due to the high earthquake demands. In the longitudinal direction, the Plant Room main floor has a rating of 22%NBS (IL4) and 37%NBS (IL3).

For the main floor structure in the transverse direction, the behaviour is similar to the above stated for the longitudinal direction. The capacity of the braces is limited to 600kN, and again, the connections will fail in a brittle manner when the braces are in tensile action. The total rating for the floor in the transverse direction is 24%NBS (IL4) and 43%NBS (IL3). In both IL4 and IL3 cases, drifts are expected to be in the order of the ratings hereby stated for the roof structure, while drifts for the plant room floor are expected to be higher than the %NBS stated for this element.

In terms of the SLS2 performance objective, the plant room scores 33%NBS, being the roof structure in the longitudinal direction the limiting factor.

In summary, the following Structural Weaknessess (SWs) have been identified during the assesment:

- For the Plant Room roof structure, braces are eccentric to the frames. Minor bending of portal frames may occur in the longitudinal direction, instead of the intended yielding of the tension-only CBFs. This is the severe structural weakness for the plant room roof (SSW).

- For the roof structure in the transverse direction, portal frames are not fully braced and therefore they do not exhibit a ductile behaviour.

- For the Plant Room floor, braces are not arranged in pairs. The structure is likely to have an asymmetrical behaviour, with ratcheting effects.

- For the Plant Room floor, brace capacities are limited by the connections to the existing structure. The connection to the concrete beams underneath or welding to the steel gravity structure of the floor show deficiencies and brittle behaviour. This is likely to be the limiting structural weakness and therefore the severe structural weakness for the plant room floor.

4.4 Main Roof Structure

The main roof structure is comprised of steel portal frames that provide the main support for gravity loads and lateral loads in the transverse direction. In the longitudinal direction, tension-only bracing provides resistance to lateral loads. Most of the transverse frames are similar, with the exception of the ones at each end. Tension-only roof bracing and hip beams provide sufficient rigidity to consider the diaphragm rigid in both directions. The roof structure is depicted in the following figures:

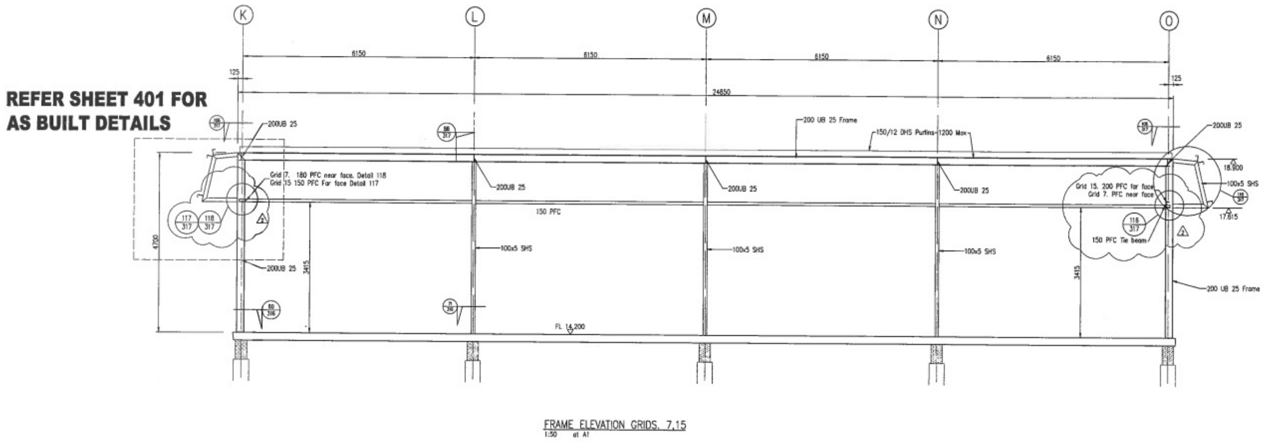


Figure 27: Typical End Frame at roof level.

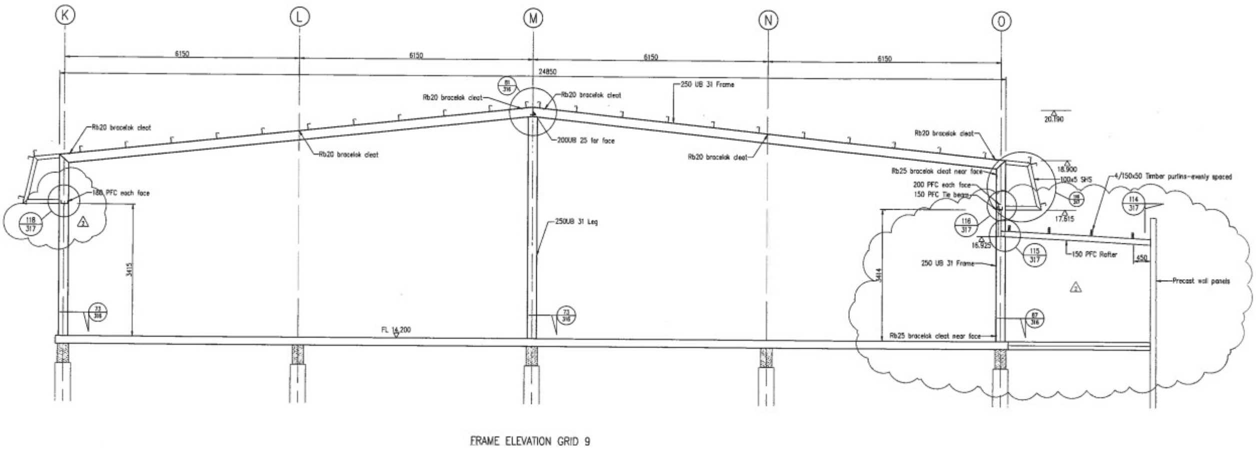


Figure 28: Typical intermediate frame at roof level

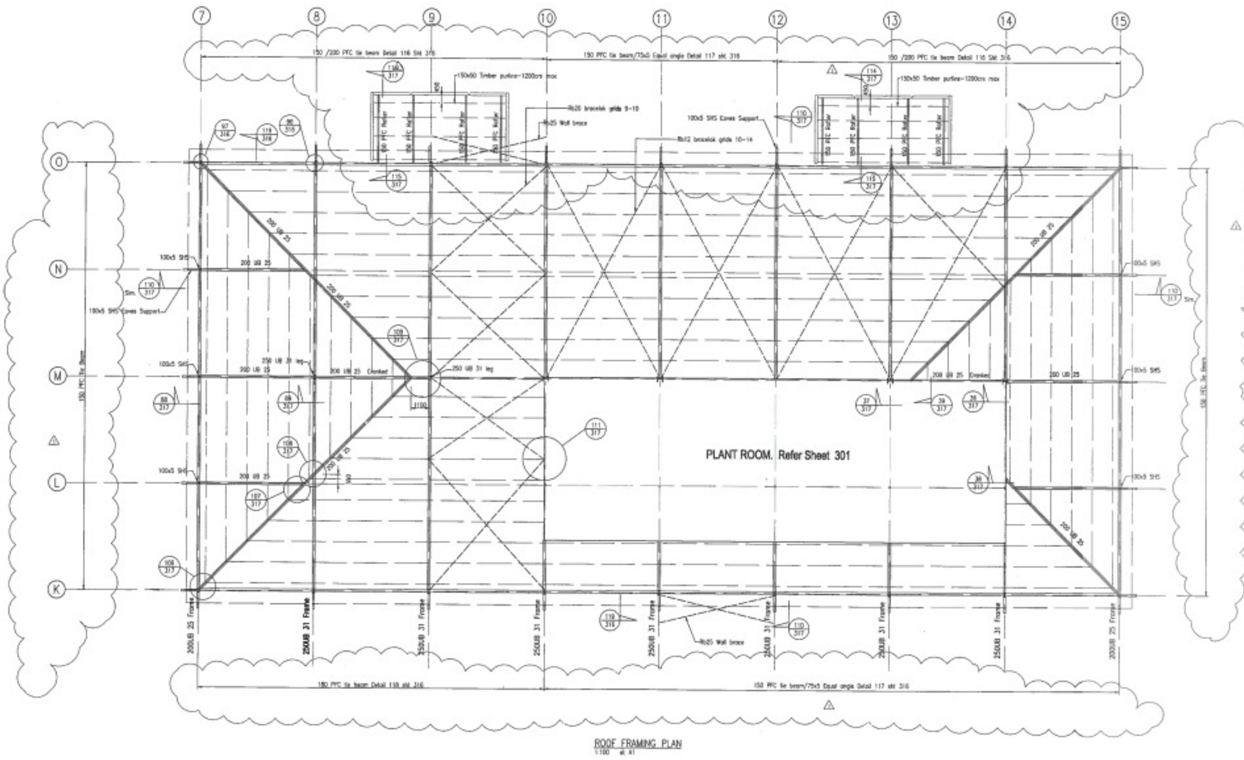


Figure 29: Roof Framing Plan

Results from the assessment show that in the longitudinal direction, the rating is limited by either the roof diaphragm (compression of purlins) or by the connections of the longitudinal bracing to the side frames. The analysis shows that the vertical braces are limited by their connection to the main portal frames that are eccentric and by their connection to the base plates. In the latter case, shear capacity is limited by concrete break-out, which is a non-ductile failure. Also, the cleat weld has a big limitation on the load that can be withstood. The rating obtained is less 15%NBS for this item for both IL3 and IL4 importance levels.

In the transverse direction, thanks to the hip beams and the roof bracing, all the frames contribute to the lateral capacity. The assessment shows that the frames aren't fully laterally braced, and therefore lateral torsional buckling is likely to occur. The rating obtained in the transverse direction is less than 15%NBS, again for the IL3 and IL4 importance levels.

In both cases for IL4 and IL3, drifts are expected to be in the order of the ratings hereby stated.

The following Structural Weaknesses (SWs) have been identified for this element:

- Roof bracing is limited by the compressive capacity of the purlins, which in addition, are not contained in the same plane.
- Longitudinal vertical bracing is limited by the connection details to the columns. A compression element is also needed at the top of the frames to adeqautely distribute loads. Either bolt break-out failure or weld failure are likely to occur.
- Transverse frames are not fully braced and therefore they cannot exhibit ductility in order to obtain a higher rating (SSWs).

- Connections of hip beams to main portals are minimal.

In terms of the SLS2 performance objective, it is expected that the roof will less than 30%NBS (SLS2). This means that for a 500yr event (100%NBS SLS2), the roof structure is likely to sustain severe damage and therefore it will not be operative after this event.

4.5 External lightweight Panels (SSNS)

An analysis of the cladding panels at the first floor was carried out to determine the risk of the panels falling onto external areas of the building. A simple analysis of the connections and demands as carried out and a rating in excess of 90%NBS (IL3) was obtained.

4.6 External Stairwells (SSNS)

The external stairwells represent a relevant secondary structural and non-structural (SSNS) element. These staircases were added to the building during the 2004 alterations and they are comprised of reinforced precast concrete walls, resting on top of an in-situ concrete foundation. The stair's flights and landings are comprised of precast concrete slabs connected to the walls. The layout of these two staircases can be seen in the following figures:

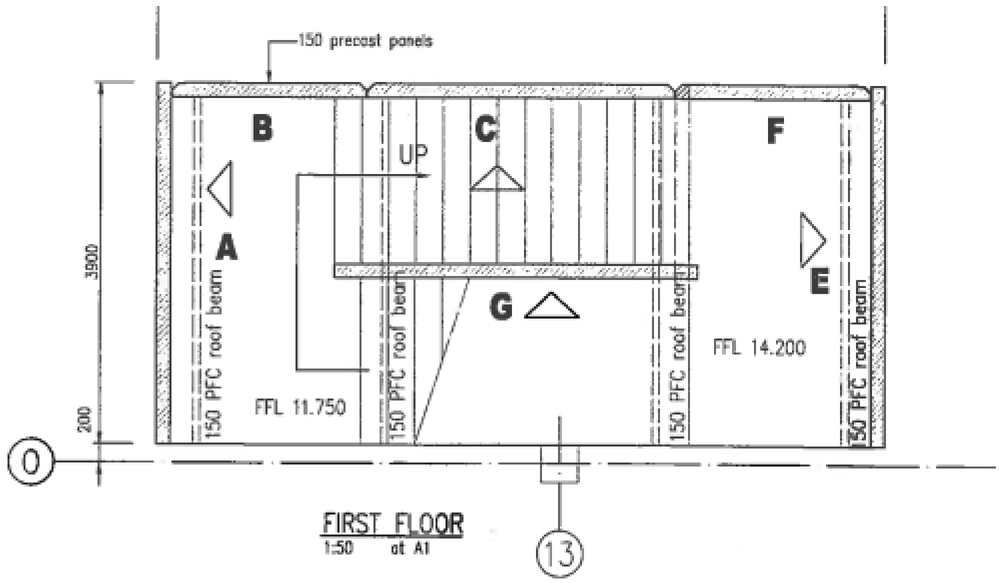


Figure 30: First Floor Plan of external stairwell

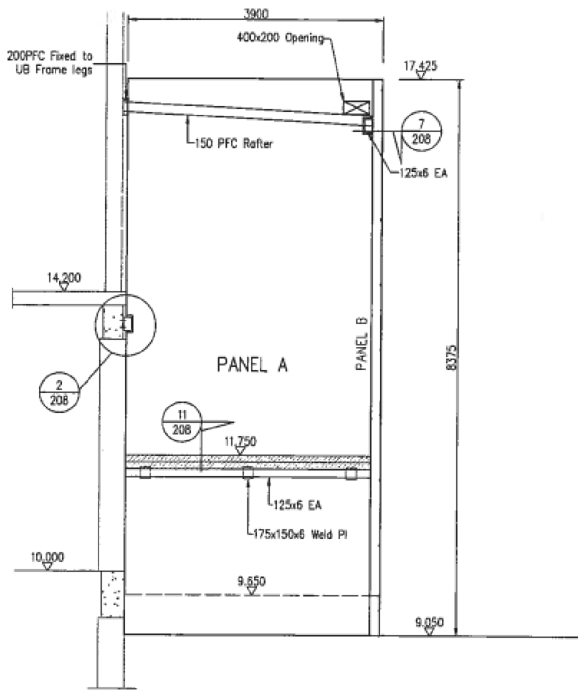


Figure 31: Elevation of Stairwell

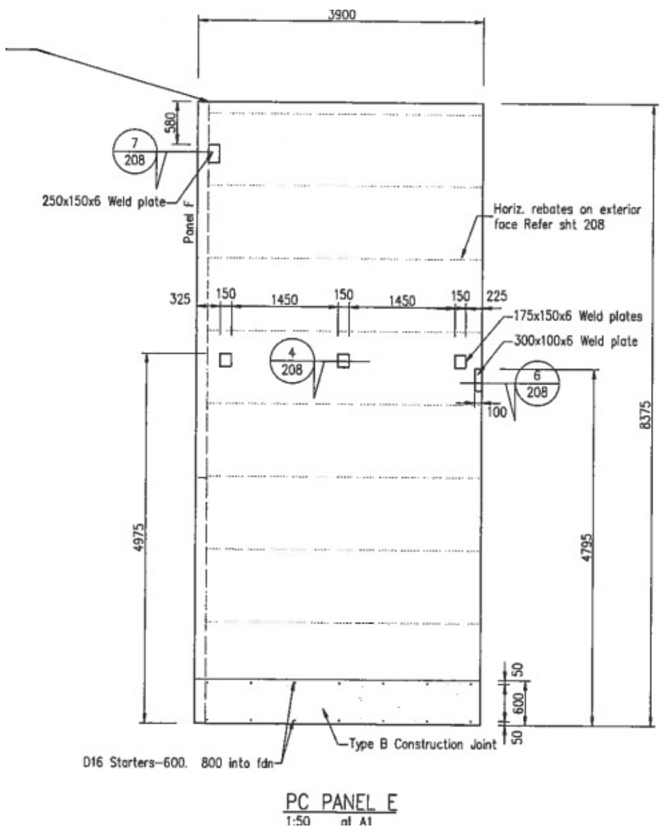


Figure 32: Typical wall detail (notice foundation thickness and reinforcement)

As can be seen, the panels are connected to each other through weld plates and are connected to the foundation on their side with D16 starters at 600crs. The gravity structure of the stair is comprised of steel members, welded to cast-in weld plates, with precast floor slabs for the landings and flights. All these elements are resting on top of the walls, which in turn take the load to the foundation. The lateral structural system is achieved through the in-plane action of the walls. At the roof level, the roof is connected to a collector on the roof bulkhead. The side panels also have a welded connection to an angle bolted into the existing first-floor beams.

The assessment shows that this element has several structural weaknesses. The first one is the connection to the first floor beams. This connection will have its capacity exceeded at very low demands, meaning that the stairwell will be disconnected and behave separately from the main building. Therefore, in our analysis, we have considered that the stairwells are free-standing, without any interaction with the main building. In fact, the roof of the stairwell is connected to the roof of the main structure, but this connection is via a flexible 150PFC, and therefore no major interaction is achieved here.

The analysis shows that the shear capacity of the stair structure is limited by shear friction of the bottom connection of the walls to the foundation and by the overall stability. A rating of 22%NBS(IL4) has been achieved in the transverse direction and 26%NBS(IL4) in the longitudinal direction. If the building is considered IL3, then the ratings are 44%NBS in the longitudinal direction and 36%NBS in the transverse direction. Pounding between the stairwell and the building is likely, but no severe damage is expected. For global stability, the short direction is the critical with 48%NBS(IL4) and 81%NBS(IL3).

In summary, the following structural weaknesses (SWs) have been found during the analysis:

- Connection of the stairwell to the building is non-ductile (concrete break-out), the stairwell will be essentially disconnected after first pulse of the earthquake. The ratingfor the connection is 9%NBS(IL4). After the disconnection, the structure by itself will perform better under earthquake loadings.
- Pounding between stairwell and first floor mass is likely, with minor damage occurring.
- Foundation starters to the bottom of the wall are not connected to the main reinforcement. Foundation splitting is likely.
- Shear friction at foundation of dowels limits the bending and shear capacities of the walls. This is the severe structural weakness (SSW) as it limits the rating of the element.
- Global stability in the short direction can be an issue to be considered for strengthening.

In terms of serviceability, it is expected that the building will rank around 50%NBS (SLS2). Therefore, in the case of a 100%NBS SLS2 event, it is likely that these stairs will sustain damage and will not be accessible for evacuation or operation under this return period of seismic demand.

4.7 Existing Gravity Frames

The existing reinforced concrete gravity frames will provide some strength to the overall behaviour of the main floors. Based on a basic analysis, it has been assessed that the frames do not have sufficient stiffness to contribute to lateral load resistance by the CBFs or the RCM walls. Even so, a quick calculation showed that the lateral capacity of the mechanism averages 2MN, which means that high level of ductility capacity will be required even to obtain 70%NBS(IL3). These frames are well detailed for their age and will exhibit a ductile behaviour. However, lap splices in plastic hinge regions and partial fulfilment of ductile detailing requirements will mean that the overall displacement ductility that can be materialised will be only up to a limited level of 2. Therefore, the capacity of the frames is insufficient to contribute to the overall rating of the building once the other elements have failed. For this reason the existing gravity frames have not been used in assessing the capacity of the building.

Due to their ductile detailing it is expected that the frames will accommodate the drifts imposed by earthquake actions and resisted by the CBFs and RCM walls. However, some local column failures may occur near the joints where the RCM walls are present. These failures might not be critical as the presence of the wall will create a secondary path for the gravity loading. Therefore, this individual concrete column failures have not been classified as critical from a life safety perspective.

4.8 Assessment Result Summary

The following tables summarise the results obtained for the different structural systems and non-structural systems for an Importance Level 4 and Importance Level 3 structures, respectively:

Table 12: Summary of Results for IL4 - ULS

Element	Direction	Rating [%NBS]	Mode of Failure/Comments
Main Roof Structure	Longitudinal	**<15**	Longitudinal reid-braces insufficient. Weld failure, anchor failure.
	Transverse	**<15**	Frames without sufficient capacity. Ductility and fully bracing required to achieve higher NBS.
Plant Room Roof	Longitudinal	**<15**	Tension-only bracing and detailing not sufficient. Elastic failure.
	Transverse	**35**	Frames not fully braced. Lateral torsional buckling likely.
Plant Room Floor	Longitudinal	**22**	Connection of braces fail, braces not in opposing pairs
	Transverse	**24**	Connection of braces fail, braces not in opposing pairs
First Floor	Longitudinal	**44**	CBFs connections with brittle failure.
	Transverse	**40**	CBFs connections with brittle failure.
Ground Floor	Longitudinal	**100**	Capacity of shear walls. Overall capacity limited by foundation.
	Transverse	**53**	Capacity of shear walls. The openings and the number of walls are the reason for the disparity of results with the longitudinal direction.
Foundations	Longitudinal	**87**	Sliding failure of foundations and failure of piles.
	Transverse	**80**	Sliding failure of foundations and failure of piles.
External Stairs (Non-Structural)	Longitudinal	**26**	Shear friction at base of walls.
	Transverse	**22**	Shear friction at base of walls. Global stability also an issue.
Results		**15%NBS (IL4)**	

Table 13: Summary of Results for IL4 – SLS2

Element	Direction	Rating [%NBS]	Performance for SLS2 after a 500yr event
Main Roof Structure	Longitudinal	**30**	Likely collapse of roof structure, top storey unavailable.
	Transverse	**30**	Likely collapse of roof structure, top storey unavailable.
Plant Room Roof	Longitudinal	**33**	Services at plant room no longer operative
	Transverse	**33**	Services at plant room no longer operative
Plant Room Floor	Longitudinal	**33**	Services at plant room no longer operative, first floor inaccessible.
	Transverse	**33**	Services at plant room no longer operative, first floor inaccessible.
First Floor	Longitudinal	**86**	Some damage to CBFs, ground and first floor unsafe after event.
	Transverse	**94**	Some damage to CBFs, ground and first floor unsafe after event.
Ground Floor	Longitudinal	**100**	Plant room will remain operative if earthquake is in this direction. .
	Transverse	**93**	Minor damage and repairs required after event.
Foundations	Longitudinal	**100**	Foundations likely to withstand SLS2 earthquake without damage.
	Transverse	**100**	Foundations likely to withstand SLS2 earthquake without damage.
External Stairs (Non-Structural)	Longitudinal	**50**	Damage sustained after earthquake, stairs probably un-usable.
	Transverse	**50**	Damage sustained after earthquake, stairs probably un-usable.
Results		**Large extent of damage to the building, being not able to continue operations after 500yr earthquake.**	

Table 14: Summary of Results for IL3 - ULS

Element	Direction	Rating [%NBS]	Mode of Failure/Comments
Main Roof Structure	Longitudinal	**<15**	Longitudinal reid-braces insufficient. Weld failure, anchor failure.
	Transverse	**<15**	Frames without sufficient capacity. Ductility and fully bracing required to achieve higher NBS.
Plant Room Roof	Longitudinal	**27**	Tension-only bracing and detailing not sufficient. Elastic failure.
	Transverse	**63**	Frames not fully braced. Lateral torsional buckling likely.
Plant Room Floor	Longitudinal	**37**	Connection of braces fail, braces not in opposing pairs
	Transverse	**43**	Connection of braces fail, braces not in opposing pairs
First Floor	Longitudinal	**58**	CBFs connections with brittle failure.
	Transverse	**53**	CBFs connections with brittle failure.
Ground Floor	Longitudinal	**100**	Capacity of shear walls. Overall capacity limited by foundation.
	Transverse	**69**	Capacity of shear walls. The openings and the number of walls are the reason for the disparity of results with the longitudinal direction.
Foundations	Longitudinal	**100**	Sliding failure of foundations and failure of piles.
	Transverse	**87**	Sliding failure of foundations and failure of piles.
External Stairs (Non-Structural)	Longitudinal	**44**	Shear friction at base of walls.
	Transverse	**36**	Shear friction at base of walls. Global stability also an issue.
First Floor Cladding Panels	Out-of-plane	**90**	Failure in Bending of PFC, mechanism forced and fall of panel to the ground.
Results		**15%NBS (IL3)**	

As it can be seen from the above tables, if the building is to be considered Importance Level 4, the expected result will be 15%NBS (IL4), governed by the roof structure and the Plant Room. The mode of failure is collapse of the roof structure due to the lack of lateral capacity in both directions.. This result means that the building is currently a very high earthquake risk and has an Grade E risk category. Also, the building will not be able to fulfil its post-disaster function, as the level of damage for the SLS2 event will be extensive.

If the building is considered Importance Level 3 as it is now, the rating of the building is expected to be 15%NBS (IL3), meaning again that the building has a high earthquake risk and rated as seismic risk category Grade E.

These low values are in part because the building exhibits an elastic behaviour under earthquake loadings governed by non-ductile connection failures. These connection failures do not dissipate energy in a dependable way and therefore the seismic actions cannot be dissipated by any mechanisms. If those mechanisms were possible, and ductility achieved, the rating of the overall building will improve significantly. The roof structure in the longitudinal and transverse directions are the lowest scores, being the eccentric brace connections and lack of restraint of frame members the severe structural weaknesses (SSWs) that control the rating of the building.

Another severe structural weakness worth of mentioning is the lift shaft walls and internal stair core walls. These walls lack adequate reinforcment lap detailing at their foundations and therefore cannot develop their full strength. Also, their connection to the floor slabs are minimal and pounding between the floors and the lift core and stairs is expected to occur.

At the foundation level there is a strut present taking gravity loads to the nearest pile. It might be possible that during the earthquake drifts, this strut may buckle, leading to a sudden loss in the support of the load. Direct support of the load against the foundation ground is recommended.

It is worth mentioning that despite these low score values, the main structure for the ground and first floors rates as earthquake risk (<67%NBS). However, to bring the building to the 67%NBS threshold, works will be required to the first floor and roof structures as detailed in the following pages.

5 Commentary on Seismic Risks

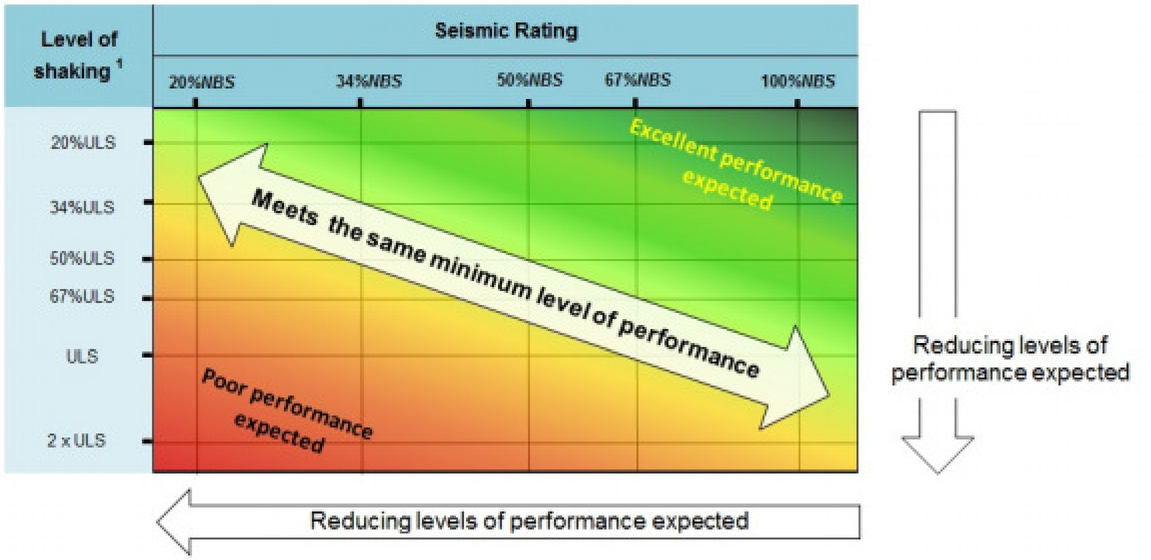
The above results intend to materialise the relative seismic risk when compared to a new building if it were designed today. A lower %NBS means a higher seismic risk, which follows a non-linear relationship with the rating. The following table prepared by the New Zealand Society for Earthquake Engineering intends to describe this relative risk for different levels of %NBS achieved:

Table 15: Assessment outcomes and relative risk

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

As can be seen, an earthquake prone building is expected to have 10 times more risk than a new building in terms of seismic hazards.

The following graph shows the expected performance for a given %NBS achieved, versus an earthquake demand equivalent to %ULS shaking, for a given Importance Level:



Notes:

1. The diagram represents the tolerable (minimum) outcomes for buildings of any earthquake rating, at any level of shaking.
2. For any earthquake rating, the expected seismic performance represented on the vertical axis should be achieved over the full range of possible shaking levels.

Figure 33: Indicative relationship between seismic performance, earthquake rating and level of shaking (MBIE guidelines)

Therefore, when a building is subjected to an earthquake shaking comparable to a greater %NBS, a poor performance is expected, with subsequent increased risk of collapse or loss of life. When analysing results, it is always wise to compare the relative risks to ascertain what the consequences of the stated ratings are.

6 Seismic Retrofit Options

In this section, seismic retrofit options are described to achieve 34%NBS and 70%NBS, considering the building as Importance Level 3. All these seismic retrofit options are at a basic level. To confirm that the proposed seismic strengthening schemes fully achieve the performance objectives, further analysis and design needs to be carried out. As discussed in the next section, a staged approach with basic engineering, preliminary design and detailed design to the most suitable option has always drawn the best outcomes.

Currently, the HBDHB has engaged Opus to develop strengthening options for the building for a complete strengthening scheme above 70%NBS. At this point in discussion, the intended scheme will be achieved in two stages, with the low ratings being dealt with urgency.

6.1 Above 34%NBS (IL3)

The below options for strengthening the building above 34%NBS will only bring disruption to the first floor. These options will increase the overall capacity and will provide redundancy to the structure, reducing the risk of damage and loss of function.

6.1.1 Roof Structure

The roof structure rates less than 15%NBS (IL3) in both directions. To improve this rating, tension only braces in the longitudinal and transverse directions are provided. For both cases, the roof bracing will likely be replaced and additional compression members used, especially between grids 9 and 10.

6.1.1.1 Longitudinal Direction

Existing bracing will need to be replaced with tension-only bracing. In addition, new bracing elements will need to be provided to increase the overall capacity. Four bays with tension only cross bracing via EA100x6 are proposed. These braces shall connect the knee of the portal with the floor slab. A top element will be needed to connect the different bays and to collect the demands from the roof structure. We propose an 100SHS to be this knee element.

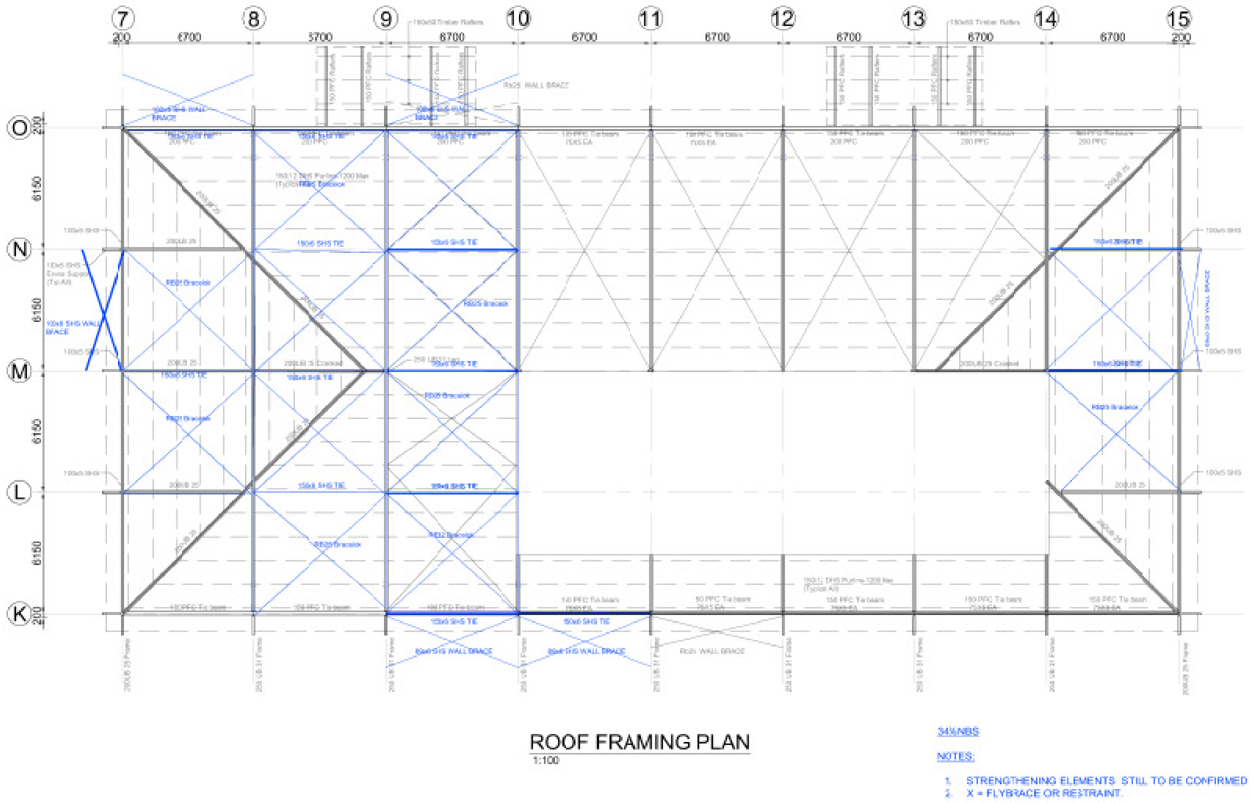


Figure 34: Location of strengthening for roof structure

New braces can be placed at any location, along the bays. However, a symmetrical layout is always recommended. Connections to the concrete slab and to the frame will need to be provided.

Refer to the appendix for a complete detailed set of elements to be provided.

6.1.1.2 Transverse Direction

In the transverse direction, tension only braces at the end bays 7 and 15 plus a CBF frame on grid 9 are encouraged. Welding of plates to the columns might be required. Additional restraints will be required for the central column in the intermediate frames. The restraint of the portal rafter will occur at every portal.

Refer to the appendix for a complete detailed set of elements to be provided.

6.1.2 Plant Room

The plant room will need to be strengthened above 34%NBS at the roof level in the longitudinal direction. In this case, in similar fashion as with the roof structure, the cross braces shall be replaced with equal angles and a knee SHS100 should be provided.

Refer to the appendix for a complete detailed set of elements to be provided.

6.2 Above 70%NBS (IL3)

6.2.1 Roof Structure

To achieve 70%NBS (IL3) the roof must be strengthened as for 34%NBS but to a greater extent as follows.

6.2.1.1 Longitudinal Direction

Existing bracing will need to be replaced with ductile tension-only bracing. Also, new bracing elements will need to be provided to increase the overall capacity. Six bays with tension only cross bracing via EA100x6 are proposed with a 100x6SHS strut at the knee of the portal.

The roof cross bracing will need to be enhanced as per the drawings, by welding connection plates to the bottom flange of the portals.

6.2.1.2 Transverse Direction

In the transverse direction, compression and tension steel braced frames (CBF's) will be needed. Two bays are needed in total, with a 200x5SHS at each. New border elements and connections will be needed for this element. Please note that one of the suggested locations is in line with one of the existing plant room braces. This current bracing will not be needed if new bracing is supplied as per the recommendations below.

6.2.2 Plant room

6.2.2.1 Roof

In the longitudinal direction, similarly as before for 34%NBS, the tension-only bracing via an EA100x5 will need to be replaced and provided. Connections to be improved. A knee compression member will also be needed along each side of braced lines.

In the transverse direction, fly braces will be required to portal frames to provide ductility. These fly braces will be located at the knee of the portal and at the first purlin and girt.

6.2.2.2 Floor

For the floor of the plant room, replacement of the existing braces to the floor below will be required. One option is proposed:

- Option 1: Tension and compression CBF steel frames. A 200x5 SHS cross brace will be needed at two locations in each direction (4 total). Connections and frame elements will need to be improved.

Please refer to the drawings on the appendix.

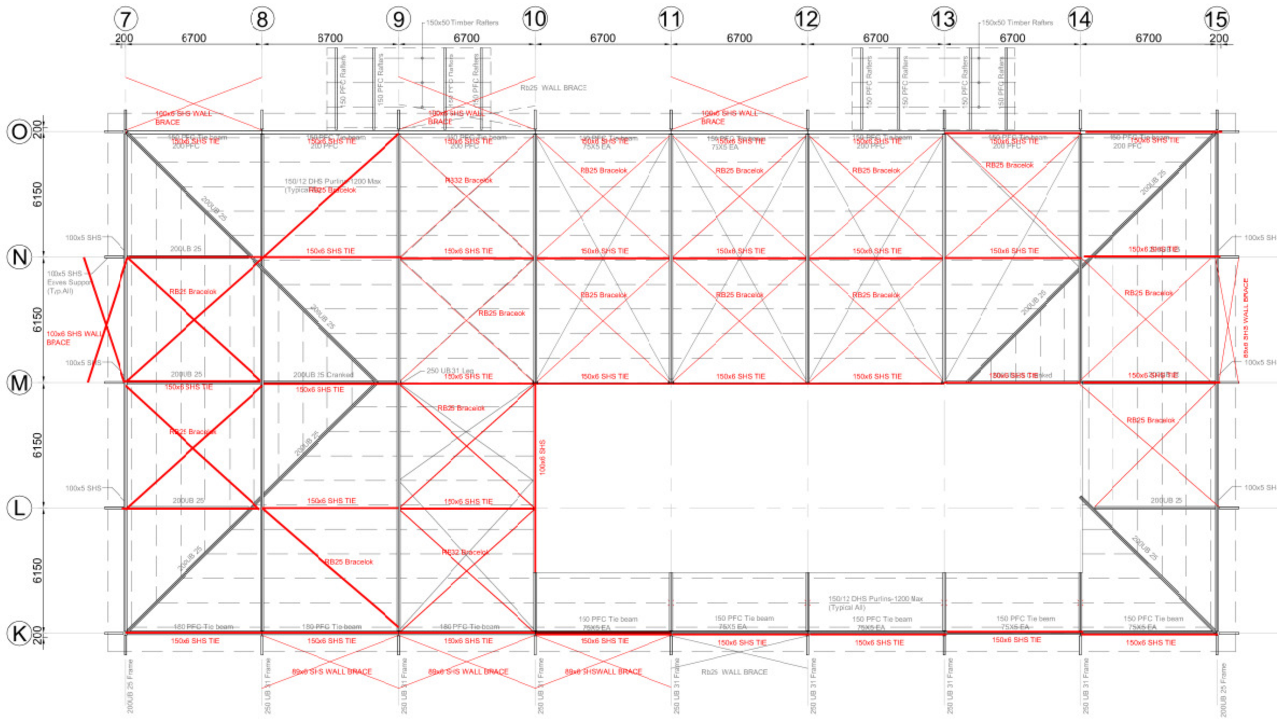


Figure 35: Location of main roof strengthening to 70%NBS: EA cross bracing (green) and existing bracing (blue), with SHS100 (Triangle) and 150SHS CBF (red)

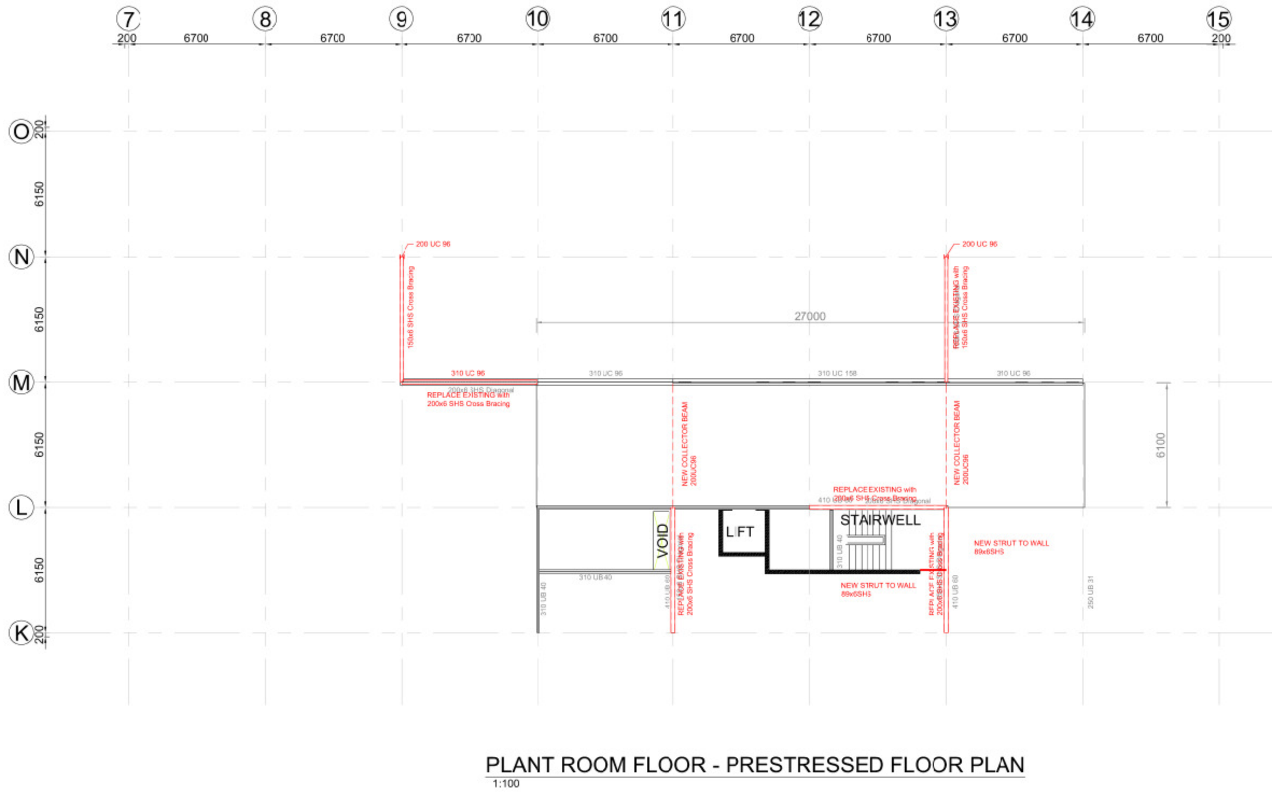


Figure 36: Location of strengthening elements to plant room floor to 70%NBS: CBFs 200x6SHS

6.2.3 First Floor

To achieve 70%NBS (IL3), the bracing capacity of the first floor needs to be increased. Due to the brittle behaviour of the existing braces, one approach might be to replace them but this is going to be extremely disruptive. The approach followed here adds new braces in line with the exterior of the building so works can be done from the outside with little disturbance to the AAU operations.

Therefore, the proposed strengthening scheme provides two new steel CBFs in each direction (blue full lines on Figure 37). These CBFs will have a 300PFC framing them to provide a continuous support for the reactions. These frames will be designed following a capacity design approach; however, it is likely that again the connections will be the limitation for the strength. This limitation to transfer the forces from the braces to the concrete frames has led to the provision of two additional frames in each direction, conditional to the connections capacities (dotted blue lines on xxxx). The reason for this approach is that the connections might not be able to materialise the demands from a single CBF and therefore additional bays will be needed to spread these actions. Sizes of the bracing elements range from 200x16SHS to 200x5 SHS if four bays are used in each direction. All steel grade for the SHSs is 450MPa, and 300MPa for the framing PFCs.

New RCM walls or in-situ concrete walls are encouraged under each one of the locations. In addition, a new foundation for these walls with micro piles might be needed. The need for micropiles is not confirmed at this stage yet.

Another point of consideration is that the first floor diaphragm (floor slab) will need to be assessed to confirm that the transfer of forces to the bracing lines is achievable through the cast insitu topping. There might be additional strengthening requirement for this item. However, the number of braces and their location may not concentrate forces and therefore the diaphragm will be able to sustain the demands from earthquake actions.

In due course of this strengthening scheme, testing of existing anchors will be needed just to confirm that the capacities stated in this report are achieved. Several number of anchors will need to be tested in order to obtain statistical data to correlate to design strength values. Location of existing braces and their connections will also need to be inspected visually in order to confirm these strengths.

As mentioned before, the %NBS achieved in the DSA is a maximum expectation as individual CBFs may fail before others reach their capacity. Therefore, further analysis with the strengthening element in place will be required in order to confirm that the 70%NBS can be achieved. If the addition of the new braces is not enough to achieve this, replacement of existing braces might be required. However, this is likely to performed as well on the outside frames.

A sketch of the strengthening layout for the first floor can be seen in the following image:

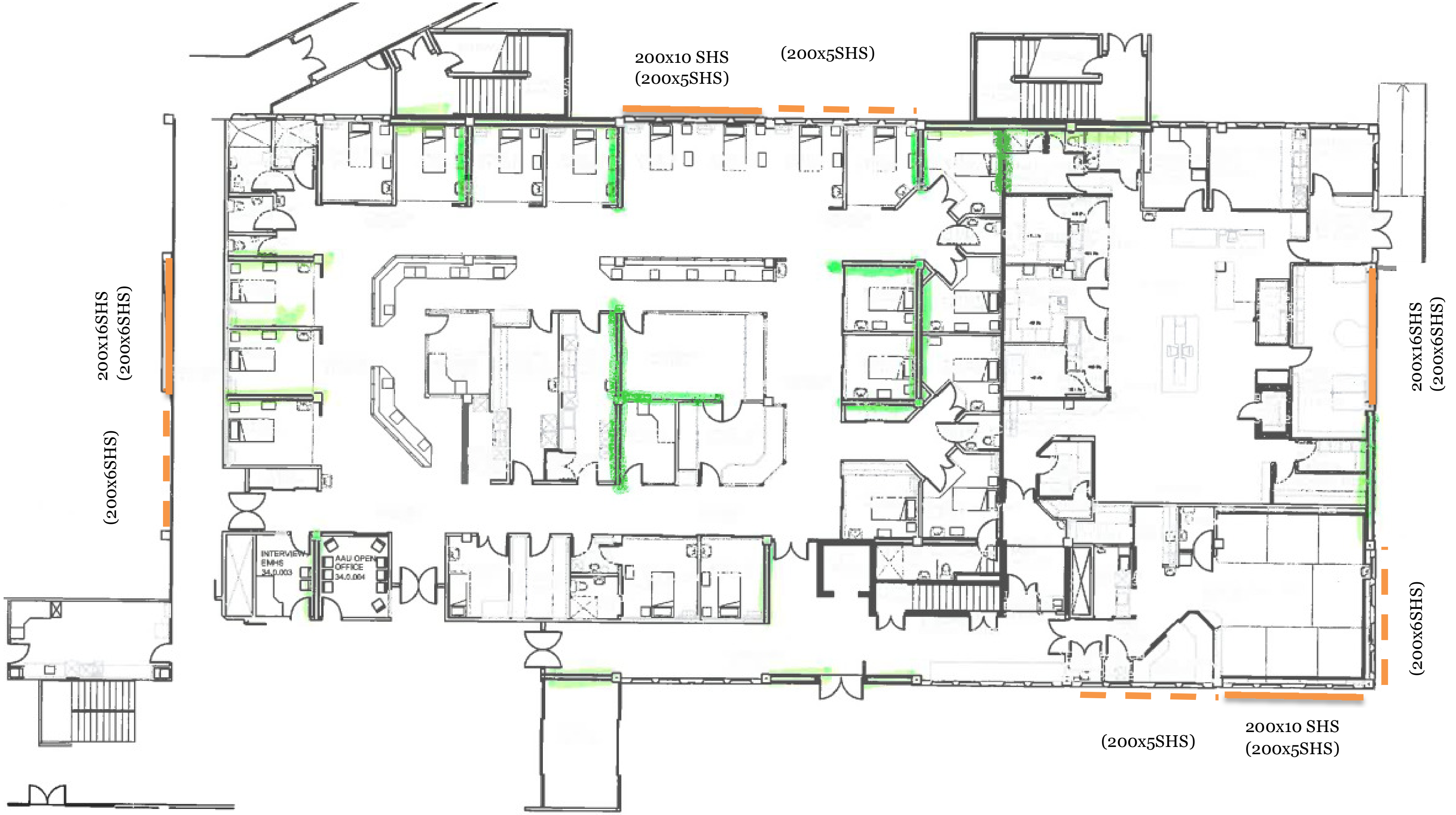


Figure 37: Location of strengthening elements to first floor to achieve 70%NBS: Orange full and dotted lines represent CHS CBFs frames – Existing CBF frames in green

6.2.4 Foundations

It is not clear at this point in the analysis if the existing piles will be able to materialise the uplift reactions from the walls required to achieve 70%NBS. It might be possible that tensile reactions might need to be achieved with the use of micropiles or additional mass, specially at the corners or the perimeter of the building. This is an additional risk to the strengthening cost that will need to be considered during the following design phases.

6.2.5 External Stairs

The external stairs will need to be improved in order to achieve 70%NBS. An improvement of the connection of the walls to the foundation is required. This might be achievable via an equal angle connected to both the wall and the foundation. We may also recommend to disconnect the wall from the building to avoid any unnecessary damage. Improvement of the foundation will be also needed and could easily achieved through the provision of a new foundation beam and micropiles. The foundation works can be done from the outside of the building, minimising any disruption to the buildings.

6.3 Summary

The strengthening schemes presented are a basic engineering exercise and further design, investigation and analysis will be required to confirm that the 34%NBS and 70%NBS performance objectives are met. Further intrusive investigations to confirm the assumptions on this report will be needed during design and construction phases. Pull-out tests of the anchors are likely to be a requirement during the design phase to confirm the performance of the first floor levels.

Both strengthening levels to 34%NBS (IL3) and 70%NBS(IL3) is going to be disruptive to the first floor structure. The location of the braces can be discussed with the client to suit better future alterations and programme. To bring the building up the 34%NBS level, the strengthening works can be concentrated at the current vacant area. However, a minor disruption at the auditorium and lecture areas will be required.

To bring the roof structure to 70%NBS, more extensive additional works will be required to the roof structure.

The strengthening of the first floor to 70%NBS can be concentrated in the outside of the building, minimising disruption to the operations. However, noise and dust will be generated and therefore works might need to be staged during late hours or during programmed closures of the wing.

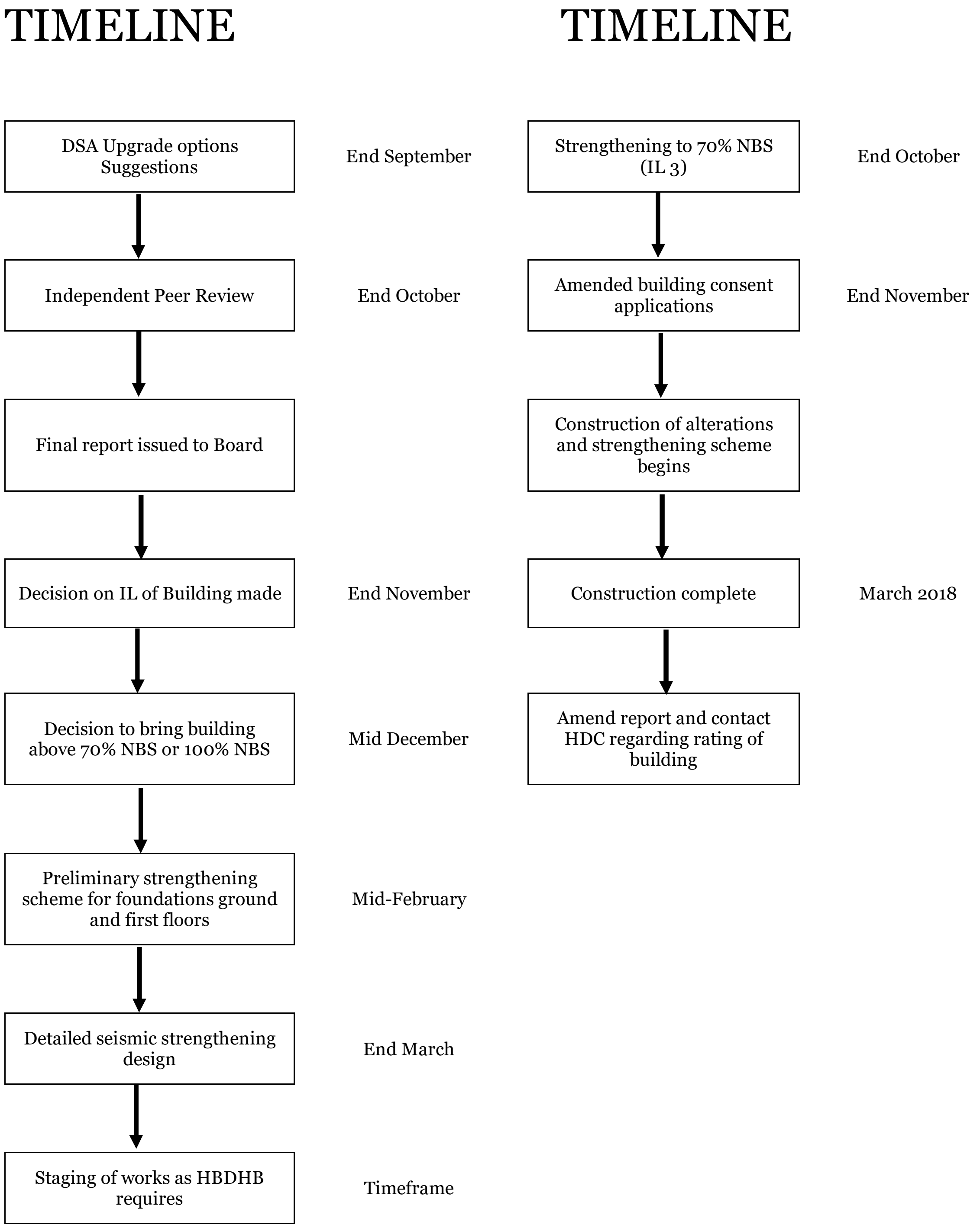
Currently, Opus is designing the strengthening scheme to bring the building up to 70%NBS, expediting the works at the future histology laboratory area.

Strengthening buildings is usually a complex procedure and therefore close liaison and frequent discussions and meetings are encouraged to report any progress and keep track of the intended design. Again, it is Opus recommendation that a staged approach should be sought. Ideally the design for strengthening will take between 3 and 6 months.

Strengthening schemes for Importance Level 4 have not been provided to the client based on the latest meeting held. With the knowledge at this point in time, strengthening above 70%NBS(IL4) can proof to be impossible due to the several structural weaknesses (SWs) that the building has and therefore an 100%NBS(SLS2) cannot be guaranteed.t

7 Recommended Next Steps

Discussions with the HBDHB are in place to develop a strengthening scheme to bring the building to 70%NBS. This strengthening scheme is currently in the design phase, and the HBDHB is expediting the resolution of the low scoring elements of the building. In this fashion, the following flowchart summarises the steps to be followed and the intent of the HBDHB in the short and medium term:



8 Conclusions

The results of the DSA indicate the building's earthquake rating to be ***15%NBS (IL4)*** or ***15%NBS (IL3)*** assessed in accordance with the guideline document "The Seismic Assessment of Existing Buildings-Technical Guidelines for Engineering Assessments", dated July 2017. The earthquake rating assumes that ***Importance Level 4 (IL4)*** and ***Importance Level 3 (IL3)***, in accordance with the Joint Australian/ New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002, is appropriate. Therefore, this is a ***Grade E*** building following the NZSEE grading scheme.

Grade E buildings represent a risk to occupants **greater than 25 times** that expected for a new building, indicating a ***very high*** risk exposure. A building with an earthquake rating less than 34%NBS fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-Prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67%NBS is considered as an Earthquake Risk Building (ERB) by the New Zealand Society for Earthquake Engineering.

The AAU-Pharmacy and Dialysis Block is not therefore categorised as an Earthquake Risk Building and also it meets the criteria that could categorise it as an Earthquake Prone Building. In accordance with the provisions of the Earthquake Prone Building requirements of the Building Act 2004 the determined earthquake rating requires the following actions for this building:

1. Notify the Hastings District Council of the results of this report.

2. If the building is considered an IL4 structure, strengthening above 34%NBS (IL4) is required before March 2025 (7.5 years).

3. If the building is considered an IL3 structure, without post-disaster functions, strengthening above 34%NBS (IL3) is required before September 2032 (15 years).

As part of this assessment we also noted the following:

- The external staircases may pose an individual risk to life and might not be available for evacuating the building after a 36%NBS earthquake unless strengthening occurs.

9 Appendices

9.1 Foundation Walls

9.1.1 Wall #1

Wall No. 1 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=1400mm
- No Openings
- Not confirmed during site inspection
- No pictures Available.
- Assumptions: fully grouted against frame, foundations as per as built drawings, properly dowelled.
- Structural Weaknesses: shear failure of beam expected due to infill demands.

9.1.2 Wall #2

Wall No. 2 has the following geometry and structural weaknesses:

- Length= 4200mm
- Height=1400mm
- Opening to side of wall.
- Not confirmed during site inspection
- No pictures Available:
- Assumptions: properly dowelled.
- Structural Weaknesses: none.

9.1.3 Wall #3

Wall No. 3 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=2000mm (2200 mm from as-built drawings)
- No Openings
- Assumptions: none.
- Structural Weaknesses: not properly doweled to beams. No dowel action, no strut action. Ineffective wall?

Figure 38: Wall No. 3

9.1.4 Wall #4

Wall No. 4 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=2000mm (2200 mm from as-built drawings)
- No Openings
- Assumptions: none.
- Structural Weaknesses: not properly doweled to beams. No dowel action, no strut action. Ineffective wall?

Figure 39: Walls 3 and 4

9.1.5 Wall #5

Wall No. 5 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=2000mm (1600 mm from as-built drawings)
- No Openings
- Assumptions: Fully grouted against frame. Properly dowelled.
- Structural Weaknesses: None.

Figure 40: Wall No. 5 (left) and 6 (right)

9.1.6 Wall #6

Wall No. 6 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=2000mm (1600 mm from as-built drawings)
- Opening to one side: 2200mmx500
- Assumptions:
- Structural Weaknesses: Opening, Eccentric strut action, deficient dowelling and grouting at opposite top corner from opening. Short column effect leads to shear failure of column.

9.1.7 Wall #7

Wall No. 7 has the following geometry and structural weaknesses:

- Length= 2400mm+Doorway+2900mm
- Height=2000mm (1800 mm from as-built drawings)
- Opening: door through middle
- Assumptions: Strut can be formed.
- Structural Weaknesses: Opening, diminishing shear strength of wall. Failure mechanism by dowel failure in non-ductile mode. Probable shear failure of columns if strut action takes place.

Figure 41: Wall No. 7, Right, with opening.

9.1.8 Wall #8

Wall No. 8 has the following geometry and structural weaknesses:

- Length= 6100
- Height=2000mm (2000 mm from as-built drawings)
- Opening: no openings
- Assumptions: Strut can be formed.
- Structural Weaknesses: Doweling ok.

Figure 42: Wall #8

9.1.9 Wall #9

Wall No. 9 has the following geometry and structural weaknesses:

- Length= 6100
- Height=2000mm (2000 mm from as-built drawings)
- Opening: Opening 800x2400
- Assumptions: Strut can be formed at lower part of wall.
- Structural Weaknesses: Opening.

Figure 43: Wall 9 (left) and 10 (right)

9.1.10 Wall #10

Wall No. 10 has the following geometry and structural weaknesses:

- Length= 6100
- Height=2000mm (2000 mm from as-built drawings)
- Opening: No Openings
- Assumptions: Strut can be formed.
- Structural Weaknesses: None.

9.1.11 Wall #11

Wall No. 11 has the following geometry and structural weaknesses:

- Length= 6100
- Height=2000mm (2000 mm from as-built drawings)
- Opening: Duct Opening At top corner (1100mmx2400mm)
- Assumptions:

- Structural Weaknesses: Strut cannot be formed except at lower part of column. Reduced shear capacity.

Figure 44: Wall 11 and 28 (right)

9.1.12 Wall #12

Wall No. 12 has the following geometry and structural weaknesses:

- Length= 6100
- Height=2000mm (1900 mm from as-built drawings)
- Opening: None.
- Assumptions: None.
- Structural Weaknesses: None.

Figure 45: Wall No. 12 (left) and 15 (right).

9.1.13 Wall #13

Wall No. 13 has the following geometry and structural weaknesses:

- Length= 2600mm+opening 800mm+2900mm
- Height=2000mm (1900 mm from as-built drawings)
- Opening: Door
- Assumptions: None.
- Structural Weaknesses: Door, dowels 70%grouted. Probable shear failure of column.

Figure 46: Wall 13

9.1.14 Wall #14

Wall No. 14 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=2000mm (1900 mm from as-built drawings)
- Opening: No Openings
- Assumptions: None.
- Structural Weaknesses: None.

Figure 47: Wall 14

9.1.15 Wall #15

Wall No. 15 has the following geometry and structural weaknesses:

- Length= 1600mm (only effective (lift core outside beam, not dowelled))
- Height=2000mm (1800 mm from as-built drawings)
- Opening: No Openings, but non continuous.
- Assumptions: Wall not Dowelled on Step
- Structural Weaknesses: Non-continuous? Step on wall and gap. Wall not dowelled on step.

Figure 48: Wall 15 - Step

9.1.16 Wall #16

Wall No. 16 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=2000mm (1800 mm from as-built drawings)
- Opening: No Openings.
- Assumptions: Wall dowelled correctly.
- Structural Weaknesses: None.

Figure 49: Walls 15, 16 and 17

9.1.17 Wall #17

Wall No. 17 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=2000mm (1800 mm from as-built drawings)
- Opening: No Openings.
- Assumptions: Wall dowelled correctly.
- Structural Weaknesses: None.

9.1.18 Wall #18

Wall No. 18 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=800mm
- Opening: No Openings.

- Assumptions: none
- Structural Weaknesses: Not properly dowelled, bond not continuous.

Figure 50: Walls 18

9.1.19 Wall #19

Wall No. 19 has the following geometry and structural weaknesses:

- Length= 6100mm
- Height=800mm
- Opening: No Openings.
- Assumptions: none
- Structural Weaknesses: Not properly dowelled, stack bond not continuous.

Figure 51: Wall #19

9.1.20 Wall #20

Wall No. 20 has the following geometry and structural weaknesses:

- Length= 2000 mm
- Height=1400 mm
- Opening: No Openings.
- Assumptions: Height.
- Structural Weaknesses: Not properly dowelled, Only small portion of wall, no framing action, limited shear action.

Figure 52: Wall #20 (left) and #5 (right)

9.1.21 Wall #21

Wall No. 21 has the following geometry and structural weaknesses:

- Length= 3200 mm+Opening+1900mm
- Height=2000 mm (1400 from drawings)
- Opening: Doorway.
- Assumptions: None.
- Structural Weaknesses: 1900mm length portion not to full height. Not taken into account.

Figure 53: Wall #21 – Uncomplete height on left, lack of dowels on right

9.1.22 Wall #22

Wall No. 22 has the following geometry and structural weaknesses:

- Length= 1400mm+Opening+3000mm
- Height=2000 mm (1400 from drawings)
- Opening: Doorway
- Assumptions: Well dowelled.
- Structural Weaknesses: No strut action.

Figure 54: Wall #22 – Rear

9.1.23 Wall #23

Wall No. 23 has the following geometry and structural weaknesses:

- Length= 1300mm+Opening+3200mm
- Height=2000 mm (1800 from drawings)
- Opening: Doorway
- Assumptions: Well dowelled.
- Structural Weaknesses: No strut action.

Figure 55: Wall #23 – Rear

9.1.24 Wall #24

Wall No. 24 has the following geometry and structural weaknesses:

- Length= 1300mm+Opening+3200mm
- Height=2000 mm (2200 from drawings)
- Opening: Doorway
- Assumptions: Well dowelled.
- Structural Weaknesses: No strut action.

Figure 56: Wall #24 – Rear left

9.1.25 Wall #25

Wall No. 25 has the following geometry and structural weaknesses:

- Length= 5500mm
- Height=2000 mm (1400 from drawings)
- Opening: Duct Opening (2000x400mm)
- Assumptions: Well dowelled.
- Structural Weaknesses: Weakened strut action

Figure 57: Wall #25 – With Opening

9.1.26 Wall #26

Wall No. 26 has the following geometry and structural weaknesses:

- Length= 5500mm
- Height=2000 mm (2200 from drawings)
- Opening: No Opening
- Assumptions: Well dowelled.
- Structural Weaknesses: None

Figure 58: Wall #26 – Wall 26 (right)

9.1.27 Walls #27, #29 and #32

Walls No. 27, 29 and 32 have the following geometry and structural weaknesses:

- Length= 5500mm
- Height=400 mm (400 from drawings)
- Opening: No Openings.
- Assumptions: Geometry as per drawings, no weaknesses. Not confirmed on site.
- Structural Weaknesses: None.

9.1.28 Wall #28

Wall No. 28 has the following geometry and structural weaknesses:

- Length= 5500mm
- Height=2000 mm (400 from drawings)
- Opening: Small duct opening at top corner. (150mmx600mm)
- Assumptions:
- Structural Weaknesses: None.

Figure 59: Wall #28

9.1.29 Wall #30

Wall No. 30 has the following geometry and structural weaknesses:

- Length= 5500mm
- Height=2000 mm (1800 from drawings)
- Opening: Small pipe through
- Assumptions: Well dowelled.
- Structural Weaknesses: None.

Figure 60: Wall #30

9.1.30 Wall #31

Wall No. 31 has the following geometry and structural weaknesses:

- Length= 1300mm+opening+2000mm
- Height=2000 mm (1800 from drawings)
- Opening: Doorway +duct opening left top corner
- Assumptions: Well dowelled.
- Structural Weaknesses: None.

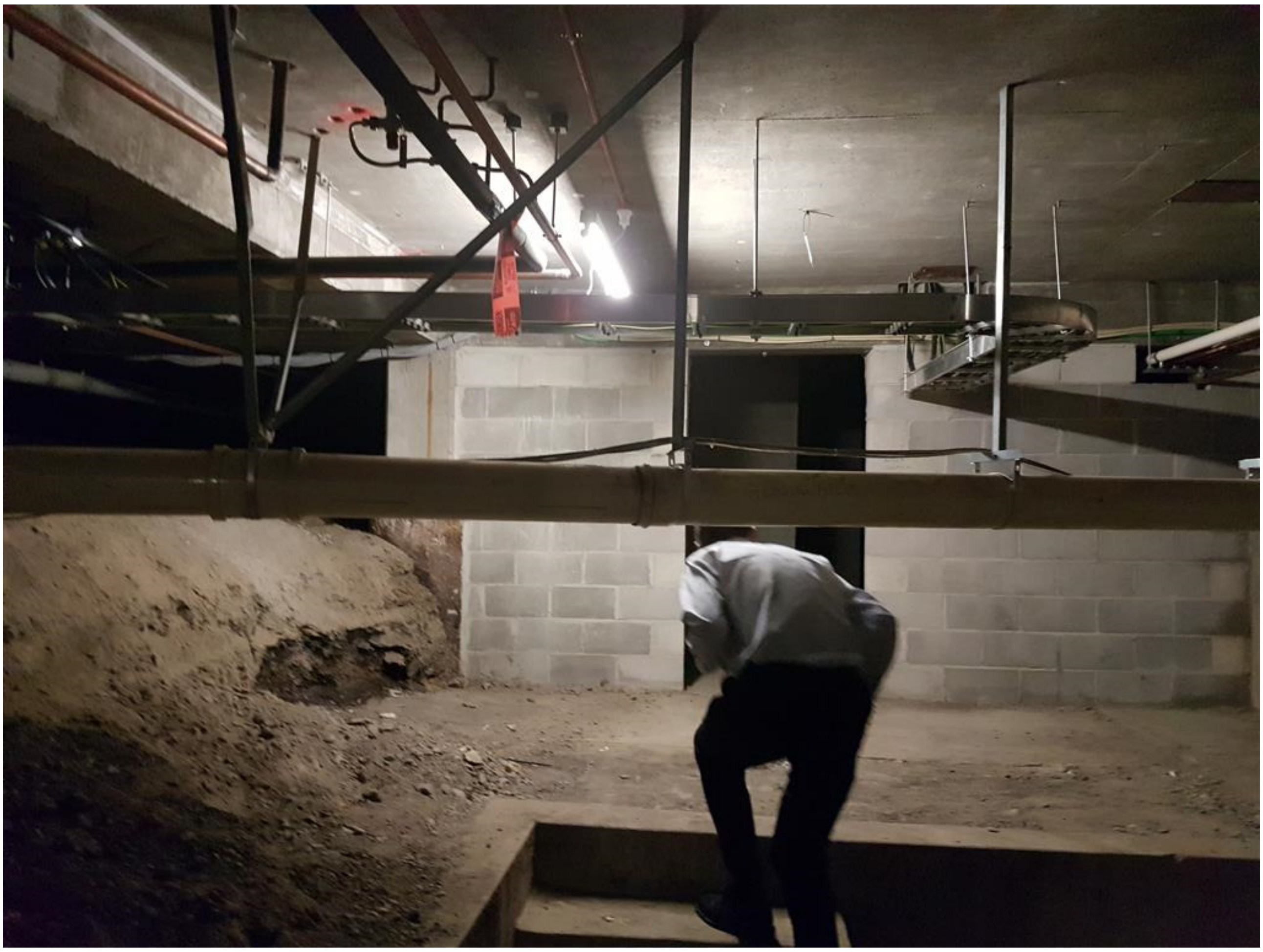


Figure 61: Wall #31

9.1.31 Walls #33, #34 and #36

These walls represent the lift core together with the offset on wall 15.

- Length= Varies
- Height=2000 mm (1800 from drawings)
- Opening: None
- Assumptions: Well dowelled.
- Structural Weaknesses: Weakly jointed to concrete floor and beam at ground floor.

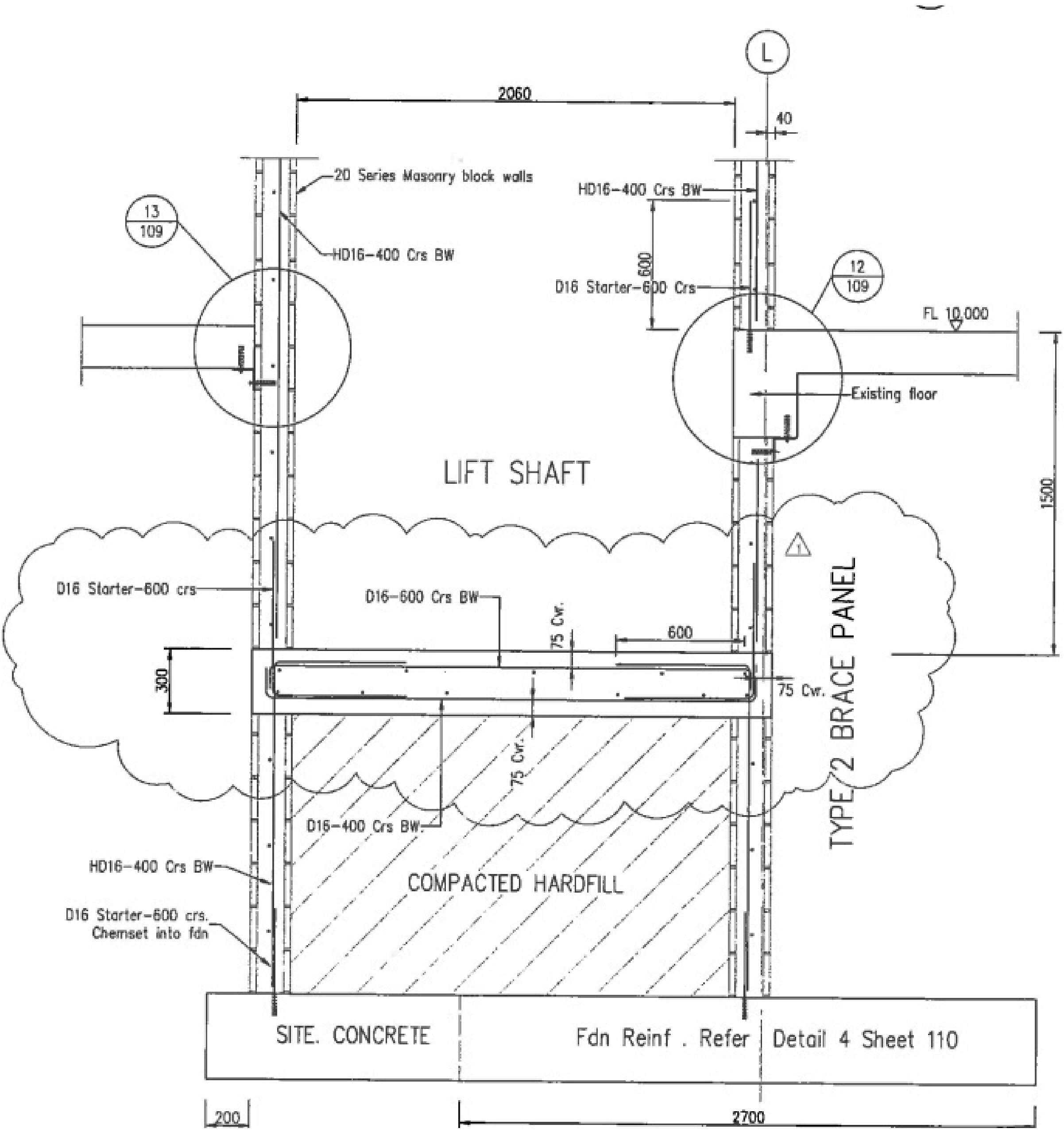


Figure 62: Sketch of lift core

9.1.32 Wall #36

Isolated wall dowelled to slab.

- Length= 2.5m
- Height=2200 mm (1800 from drawings)
- Opening: None
- Assumptions: None.
- Structural Weaknesses: Dowels un-grouted. No dowel action.

9.2 Basic Strengthening Options to 34%NBS (IL3)

9.3 Basic Strengthening Options to 70%NBS (IL3)

9.4 Calculations

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