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Hastings District Library

Detailed Seismic Assessment

13 December 2023

CONFIDENTIAL

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Revision Details

Revision	Details
A	Draft Report for Client Comment
B	Updates for Peer Review Guidance
C	Updates for Peer Review Comments
D	Updates for Peer Review & Client Comments, change to IL2
E	Final Issue

Disclaimers and Limitations

This report ('**Report**') has been prepared by WSP exclusively for Hastings District Council ('**Client**') in relation to Detailed Seismic Assessment for the Hastings City Library ('**Purpose**') and in accordance with the signed agreement on 27 March 2023. WSP accepts no liability whatsoever for any reliance on or use of this Report, in whole or in part, for any use or purpose other than the Purpose or any use or reliance on the Report by any third party.

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

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

The structural assessment is based on the limited information provided by the building owner/representative and visual inspection of the building. No destructive investigations have been undertaken at this stage.

The analysis and assessment are based on the building in its current state as of 14th April 2023 (non-intrusive site inspection). However, there may be deficiencies in the building that could cause the capacity of the building to be reduced. In this case, the current capacity of the buildings may be lower than that stated in this report. Areas of existing damage or modification will be highlighted where identified in the site visit.

Despite the use of best national and international practice, the analysis outcomes are affected by a certain level of uncertainty due to some assumptions and simplifications made during the assessment process. These include:

- Simplifications introduced in the analysis and numerical modelling of the existing structure, including boundary conditions (e.g. foundation fixity, soil stiffness, etc.);
- Assessment of material strength based on the indication provided in "The Seismic Assessment of Existing Buildings guidelines".
- Approximation in the assessment of the post-yield behaviour of structural elements;
- Assessment is based on the undamaged condition of the building;
- Site soil condition and properties.

The assessment is limited to Section B1 Structure of the New Zealand Building Code (NZBC). No assessment of the compliance of other sections of the NZBC has been undertaken. Additionally, the assessment considers earthquake loads only; other loads such as wind and snow are not covered.

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Executive Summary

The Hastings District Council Library was originally built circa 1957 as the main library for the Hastings District. It houses the Hall of Memories, a First and Second World War memorial. The building was extended in 1992. The original structure comprises cast in-situ reinforced concrete frames and walls for the main roof, Hall of Memories tower and the lower roof that contains the administration offices. The original mezzanine area was also extended in 1992 and connected to the new extension, both built with structural steel beams and columns supporting timber joists and flooring. The new extension consists of three wings, two at a 45-degree angle and a central one, built using structural steel moment frames and concrete block shear walls. As there is a seismic gap between the two structures this assessment has been divided between the Original Library Building and the 1992 Extension Building.

The results of the DSA indicate the building's *Earthquake Rating* to be **20%NBS (IL2)** assessed in accordance with the Guidelines, for both the original portion built in 1957 and the extension built in 1992. Therefore, these are **grade D buildings** following the New Zealand Society for Earthquake Engineering (NZSEE) grading scheme. Grade D buildings represent a risk to occupants 10-25 times greater than expected for a new building, indicating a **High-Risk safety risk exposure**.

A building with an *Earthquake Rating* less than 34%NBS when assessed in accordance with the Version 1 Guidelines (the 'Red book') fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-Prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67%NBS is considered as an Earthquake-risk Building by the NZSEE. **The Hastings City Library therefore potentially falls within the criteria that could categorise it as an EPB.**

The critical structural weaknesses (CSW) were found to be the lack of strength of cast-in anchors supporting the steel roof trusses of the main roof in the original portion and the lack of strength of cast-in anchors in shear walls in the 1992 extension. For the original portion, the lack of strength and movement capacity of the connections can lead to loss of seating of the trusses and additional bending of the roof beams that could lead to a partial collapse of the roof structure, particularly at the central portion. For the 1992 extension, the lack of strength in the cast in anchors can lead to loss of a positive load path with increased deformations that can lead to a collapse of the steel frames.

The assessment identified the following structural weaknesses in the building:

*Table 1 – Assessment Results for Individual Building Components and/or Systems for the **Original Library Building 1957***

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
Concrete Moment Frames – Transverse Direction-Northern Elevation	25%	• Lack of strength and deformation capacity of beam-column joint failure and reinforcement anchorage failure due to the lack of transverse reinforcement in the joint panel zone and inadequate anchorage of bars. • Lack of deformation capacity can lead to softening and collapse of the frame.
Concrete Moment Frames – Transverse Direction – Southern Elevation	30-40%	• Lack of strength and deformation capacity of beam-column joint failure and reinforcement anchorage failure due to the lack of transverse reinforcement in the joint panel zone and inadequate anchorage of bars. • Lack of deformation capacity can lead to softening and collapse of the frame.
Concrete Moment Frames and Walls on Southeastern elevation	45-65%	• Lack of strength and deformation capacity of short column due to presence of infill resulted in shortening of clear shear span, reinforcement anchorage failure due to inadequate lapping length and foundation uplift. • Lack of deformation capacity can lead to softening and collapse of the frame.
Concrete Moment Frames and Walls on Northwestern elevation	45-75%	• Lack of strength in columns and small shear walls. Uplift of foundation. • Lack of strength and increased deformations from footing uplift can lead to excessive displacements with partial or total collapse of the frame.
Roof trusses and associated connections to the main roof and steel members	20%	• Lack of shear capacity of cast-in bolts into concrete. Lack of compression capacity at bottom chord due to insufficient section size and full lateral restraints. • This can lead to loss of seating of the trusses and additional bending of the roof beams that could lead to a partial collapse of the roof structure, particularly at the central portion.

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
Roof Perimeter Beam	25%	• Lack of out-of-plane bending capacity to span between main lateral restraints in both longitudinal elevations. • Lack of strength and deformation capacity can lead to increased displacements with partial collapse of the frame elevation.
Reinforced concrete walls and foundations at Hall of Memories	45-65%	• Lack of foundation uplift capacity due to insufficient ground beam/pad size, insufficient bearing capacity due to poor ground bearing condition. • Lack of foundation capacity can lead to increased deformations in the main structure. However, full instability of the Hall in the transverse direction is unlikely.
Internal Mezzanine steel structure 1992	40%-55%	• Lack of tensile capacity of end connections to transfer loads in the longitudinal direction. Lack of capacity of end columns to transfer demands to the foundation and perimeter roof beam. • Lack of strength can lead to secondary load paths with the risk of increased deformations for the gravity structure of the mezzanine that can lead to a partial collapse.
Internal Mezzanine steel structure 1957	65%	• Lack of tensile capacity of end connections to transfer loads in the longitudinal direction. Lack of end columns capacity to transfer demands to the foundation and perimeter roof beam. • Lack of strength can lead to secondary load paths with the risk of increased deformations for the gravity structure of the mezzanine that can lead to a partial collapse
Mezzanine timber diaphragm	75%	• Lack of shear capacity to span between bracing lines. • Lack of strength can lead to increased deformations that could affect gravity structure with localized collapse.
Lower roof concrete area	40-45%	• Lack of bracing capacity and diaphragm connections to southeastern elevation, interaction

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
		with infills and potential short column effects. • Shear failure of columns can lead to partial collapse of the gravity frames.
Perimeter masonry walls and brick cladding	20%	• Lack of out-of-plane capacity and ties to the structure. • Out-of-plane failure of wall or falling of brick cladding at height could potentially lead to a significant life safety risk.
External relocated canopy	45%	• No detailed assessment was carried out. • Lack of connections can lead to increased deformations and collapse
Extension canopy to workroom	35%	• Lack of tensile capacity of connections of roof elements back to the main concrete frames • Partial collapse of roof beams might be expected.
Entry Canopy	To be confirmed	• No information has been found, to be confirmed in subsequent stages.
Mezzanine stair	40%	• Insufficient shear capacity of base anchor connections. • Lack of bottom positive connection can also lead to damage of the upper connection, leading to a partial collapse and loss of egress route.

*Table 2 – Assessment Results for Individual Building Components and/or Systems for the **Library Extension Building 1992***

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
Steel Portal 310UB at wings	35%	• Lack of full lateral restraints to the bottom chord to guarantee ductile behavior. • Lack of deformation capacity can lead to instability and collapse.
Steel Portal 310UB at wings - foundations	75%	• Insufficient bearing capacity • Failure can lead to increased deformations and softening.
End block masonry wall at side wings	85%	• Lack of out of plane overturning capacity. Insufficient connection to tie back to the building. • Out-of-plane collapse could potentially lead to significant life safety risk.
Struts along knees in longitudinal direction at wings	<33%	• Lack of shear capacity of end connection to main wing concrete masonry walls. • Lack of strength and positive load path will lead to increased displacements for the wings with partial or total collapse of the portal frames.
Main 410UB/360UB Portal frames at central wing	40-75%	• Lack deformation and strength capacity due to lack of full lateral restraints to critical flange. Lack of foundation capacity. • Lack of capacity deformation and increased foundation demand can lead to reduced stiffness with increased deformations and partial or total collapse of the frames.
Connections of mezzanine beams to shear walls	20%	• Lack of shear capacity at end connections due to insufficient anchor edge distance. • Failure of beam end connections would lead to loss of primary load path with increased connections and potentially collapse of the steel frames.
Roof steel braces	35%	• Insufficient tensile strength with asymmetrical layout. • Failure of bracing will lead to increased deformation at roof level with potential to increase demands

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
		in portal frames that could lead to their partial collapse.
SHS portal frame for end elevation	45%	• Lack of strength and asymmetrical braces. • Lack of positive load path and strength will lead to increased deformation with risk of collapse of this elevation.
Elevated roof in central wing	>70%	• Welded connections and elements lack flexural capacity. • Lack of strength can lead to increased deformations with the risk of partial collapse of frames.
Deformations	80%	• Portal frames lack stiffness for full demands. • Excessive deformations can lead to instability.
Timber floor diaphragm	75%	• Timber diaphragm lacks connectivity to transfer loads through central portion of mezzanine in the central wing. • Lack of diaphragm action can lead to increased demands on frames and lack of positive load path in the longitudinal direction, increasing deformations in the central gravity frames that could lead to partial collapse.
Mezzanine Stair	40%	• Insufficient shear capacity of base anchor connections. • Lack of bottom positive connection can also lead to damage of the upper connection, leading to a partial collapse and loss of egress route.

*These results are considered draft until a final liquefaction assessment is carried out for the site.

Technical Summary

The table below meets the reporting requirements of Section 2.5 of the EPB methodology, as required by the Earthquake Prone Buildings Amendment Act 2016.

1. Building Information	
Building Name/ Description	Hastings Library
Street Address	201 Eastbourne Street, Hastings
Territorial Authority	Hastings District Council
No. of Storeys	Original library building in 1957 - two storeys with mezzanine floor and admin areas; Original war memorial tower in 1957 – single storey tower; AKA Hall of Memories Original library workroom in 1957 – single storey structure; Library extension in 1992 – single storey wing extension and two storeys central extension with mezzanine floor;
Area of Typical Floor (approx.)	1800m^2 (in total)
Year of Design (approx.)	Original library building, workroom and war memorial tower designed in 1957; Library extension and alterations designed in 1992;
NZ standards used for design	N.Z.S.S 95: 1955 Model Building Bylaw, Basic loads to be used in design and methods of application (loading standards). NZS 3404. 1&2: 1989 to be read in conjunction with AS 1250:1981, Steel structures standard, Standards Association of New Zealand, Wellington, NZ (material standards; structural steel). NZS 3101. 1&2: 1982, Concrete structures standard - Code of practice for the design of concrete structures, Standards Association of New Zealand, Wellington, NZ (material standards; reinforced concrete). NZS 3603: 1990, Timber Structures Standard, Standards Association of New Zealand, Wellington, NZ (material standards; structural timber).
Structural system including foundations	Library and War Memorial Tower (1957) - The building is a single storey concrete structure with reinforced concrete in-situ walls and frames. The roof structure is comprised of steel trusses with timber purlins. The War Memorial area is of walled construction with a lightweight steel and timber roof. The overall building footprint is approximately 1800 square metres in total and 6.0 metres in height to the apex from the ground level.

	There is a steel mezzanine structure where there is a kitchenette, toilets and storage. Shallow pad foundations are present throughout the building with beams. Library Wing Extension (1992) - The building wing portion to the east is a single storey steel portal frame structure and generally clad with styrocrete plaster system and glazed all around the building. The building is approximately 6.0 metres in height to the apex from the ground level. The primary structure of the building including steel purlins over steel portal frames, and shallow foundation ground beam and pad footings system. The central portion is a two-storey steel framed building with concrete masonry shear walls as bracing from the first-floor level to the foundation. The two single storey wings are tied back to this central portion. An additional mezzanine on the south & northwest side has been added and extend to the west of the original building and this mezzanine ties in with the first floor of the central wing.
Does the building comprise a shared structural form or shares structural elements with any other adjacent titles?	Share common corridor at southwest side of the original library with adjacent art gallery building under the same title. Both buildings are seismically isolated around the perimeter.
Key features of ground profile and identified geohazards	Due to the lack of detailed geotechnical investigation on ground surface condition, two types of subsoil classes have been adopted for the assessment depending on different circumstance. Subsoil Class D – 'Deep or Soft Soil Site' for load spectra; Subsoil Class C – 'Shallow Soil Site' for parts and portions; The site surface subsoils are determined on desktop geotechnical study, it was found that the subsoil comprised of Topsoil to 0.35m below ground level (bgl) underlying by fine to coarse SAND (Pumiceous Sand) to 3.5m bgl. Below this, the soils change to alluvial SILT to 8.0m bgl where becomes gravelly. Ground water level inferred at a depth of 2.0m. There is liquefaction potential at the site. More investigations and study are required. Refer to WSP geotechnical assessment report for more information.
Previous strengthening and/ or significant alteration	During the 1992 alterations, the original wing has been modified substantially.

Heritage Issues/ Status	The building is not listed in either the Historic Places Trust Register or the District Plan.
Other Relevant Information	None.

2. Assessment Information	
Consulting Practice	WSP NZ Limited
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings[1]	Julian Basilio Benito CPEng 1033106 10 Years of experience in New Zealand performing seismic assessments and retrofits and 100+ hours of formal training on assessment and retrofit of existing buildings at the University of Canterbury and MBIE training courses on the guidelines. Specialization in Earthquake Engineering and total of 15+ years of experience in structural engineering. Practice area is the assessment and design of low to mid-rise buildings. .
Documentation reviewed, including: • date/ version of drawings/ calculations[2] • previous seismic assessments	(a) Davies Phillips and Chaplin Architectural Drawings & Hastings City Council Engineer's Department Drawings dated December 1957, Ref No. 925, titled, "Hastings and District War Memorial Library", Sheets 1-14 (architectural) & Sheets S1-S10 (structural). (b) Works Consultancy Services Napier drawings package dated 1992, Job No. 3/177/5, titled, "Central Library – Eastbourne Street Alterations and Additions". 80 sheets in total (architectural, structural, mechanical, and electrical). (c) Structural Calculation from Building Permit Application Form dated 1992, Central Library Extensions Structural Calculations. Building Permit No. ABAJ59645, 122 pages. (d) Works Consultancy Services Napier geotechnical report dated May 1992, Report No. V431D2U/92/1/R2, titled, "Hastings Library Extensions Geotechnical Report for Design Purpose", 11 pages. (e) Strata Group, 2011, J111130, "Structural Assessment of Hastings Memorial Library Eastbourne St., Hastings", Previous Seismic Assessment of the Original Library Building.
Geotechnical Report(s)	High-level geotechnical desktop study has been conducted for the existing site by WSP. The desktop study report is numbered: 2-S5600.5A, Date: 30 May 2023. A detailed liquefaction assessment is recommended.
Date(s) Building Inspected and extent of inspection	This report is based on a non-intrusive site inspection on 14th April 2023 by WSP Structural Engineers.

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

	Point cloud survey was carried out during the inspection with handheld scanning device to capture accurate real-life dimension and layout of the existing building.
Description of any structural testing undertaken and results summary	None.
Previous Assessment Reports	(a) WSP roof condition assessment report dated 19 September 2022, Job No. 2-S5600.5A, titled, "Hastings library roof condition assessment report", 14 pages. (b) Strata Group, 2011, J111130, "Structural Assessment of Hastings Memorial Library Eastbourne St., Hastings", Previous Seismic Assessment of the Original Library Building
Other Relevant Information	None.

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	Public library– Importance Level 2
Site Subsoil Class	Site subsoil Class as per NZS 1170.5: 2004; Subsoil Class D – 'Deep or Soft Soil Site' for spectra; Subsoil Class C – 'Shallow Soil Site' for parts and portions;
Summary of how Part C was applied, including: • the analysis methodology(s) used from C2. • other sections of Part C applied	Displacement based elastic procedure – Simplified nonlinear pushover analysis (SLaMA) Section C5 – Concrete Structures for primary structure to determine grades and probable strengths (capacities). Section C6 – Steel Structures for primary structure to determine grades, probable strengths (capacities) and failure mode of steel members and connections. Section C7 – Moment Resisting Frames with Infill Panels Structures for primary structure to determine grades and probable strengths (capacities). Section C8 – Unreinforced Masonry Buildings for secondary structure to determine grades and probable strengths (capacities). Section C2 – Forced Based approach to determine NBS of main building elements. 3D ETABS model to support hand calculations on capacities and overall rating.
Other Relevant Information	None.

4. Assessment Outcomes	
Assessment Status (Draft or Final)	Final Issue
Assessed %NBS Rating	20%NBS *(IL2)*
Seismic Grade and Relative Risk (from Table A3.1)	Grade D Buildings (High risk), both the Original Portion and the Extension. 10-25 times greater risk relative to a new building.
Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed	• Suspended lay-in tile ceiling system without seismic restraint to limit relative movement between ceiling and building elements. Not assessed. • Mezzanine floor stairs has insufficient shear capacity of base anchor connections. • Brick veneer cladding over concrete wall with potentially deteriorated wire ties. This is highlighted as a commonly risky element observed in existing buildings, intrusive investigation or condition assessment is recommended prior to future development to confirm the condition/location of veneer tie. • Lack of seismic restraint to the bookshelves. Not assessed. • Non-seismic glazing frames to accommodate relative movement; not assessed. • Potential lack of seismic restraint to large HVAC units at ceiling; not assessed. • External relocated canopy • Internal timber stairs. Not assessed. • External timber walls at end elevations of 45-degree wings. Not assessed.
Describe the Governing Critical Structural Weakness	Original 1957 Building: - The connections of the roof trusses to walls and beams can lead to increased displacement and loss of seating with potential partial collapse. Extension in 1992: - Lack of strength of knee connections of the steel strut back to the main walls at the central wing and connections of mezzanine beams to shear walls can lead to increased deformations and collapse due to loss of positive load path.
If the results of this DSA are being used for earthquake prone decision purposes, and elements rating <34%NBS have been	WSP are not aware of any letter from HDC identifying the building as potentially earthquake prone.

identified (including Parts[3]):	
Recommendations (Optional for EPB purposes)	Complete full geotechnical assessment without any further design for the redevelopment or strengthening of this building.

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

1 Introduction

WSP has been engaged by Hastings District Council to undertake a Detailed Seismic Assessment (DSA) of the existing library building at 201 Eastbourne Street, Hastings. This assessment will be used to determine the seismic risk and inform potential avenues for future use of the building within a site development. No previous Initial Seismic Assessment (ISA) has been carried out for this building. However, there is note of a previous detailed seismic assessment based in the previous NZSEE (New Zealand Society for Earthquake Engineering) guidelines.

The existing Hastings Library Building is mainly comprised of two main structural units, the 1957 Original Building and the 1992 Extension. Original drawings for the main library, alteration and extension drawings of the building were provided, together with research in Retrolens[4], it was found that (Figure 2):

- The green highlighted library and workroom was built between circa 1957-1959. The building is primarily comprised of in-situ reinforced concrete frames and walls with partial masonry infill panels. This part includes the Hall of Memories (also referred as the War Memorial Tower in this report) in blue.
- The yellow highlighted library extension was designed and built circa 1992 and is of steel portal frame construction.
- The brown highlighted workroom extension was designed and built circa 1992 and is of timber frame construction and steel.
- The red highlighted art gallery was built between circa 1969-1974 and is of concrete wall panel construction with steel truss roof (Included in a separate report).

Major alterations and extensions have been undertaken for the library building during circa 1992. During this stage, addition of new mezzanine floor to the library; wing shape extension was added to the Northeast side of the library; existing workroom was extended to the Southeast side; and other minor refurbishment of the internal building fittings throughout the library.

Seismic gaps between the yellow and green highlighted building (wing extension and library), red and green highlighted building (art gallery and library) were identified from the existing plans which indicates independent behaviour during seismic events and therefore a separate assessment was conducted for each building/structural units.

[4] Historical Imagery Resource. [Online] Available at: https://retrolens.co.nz/map

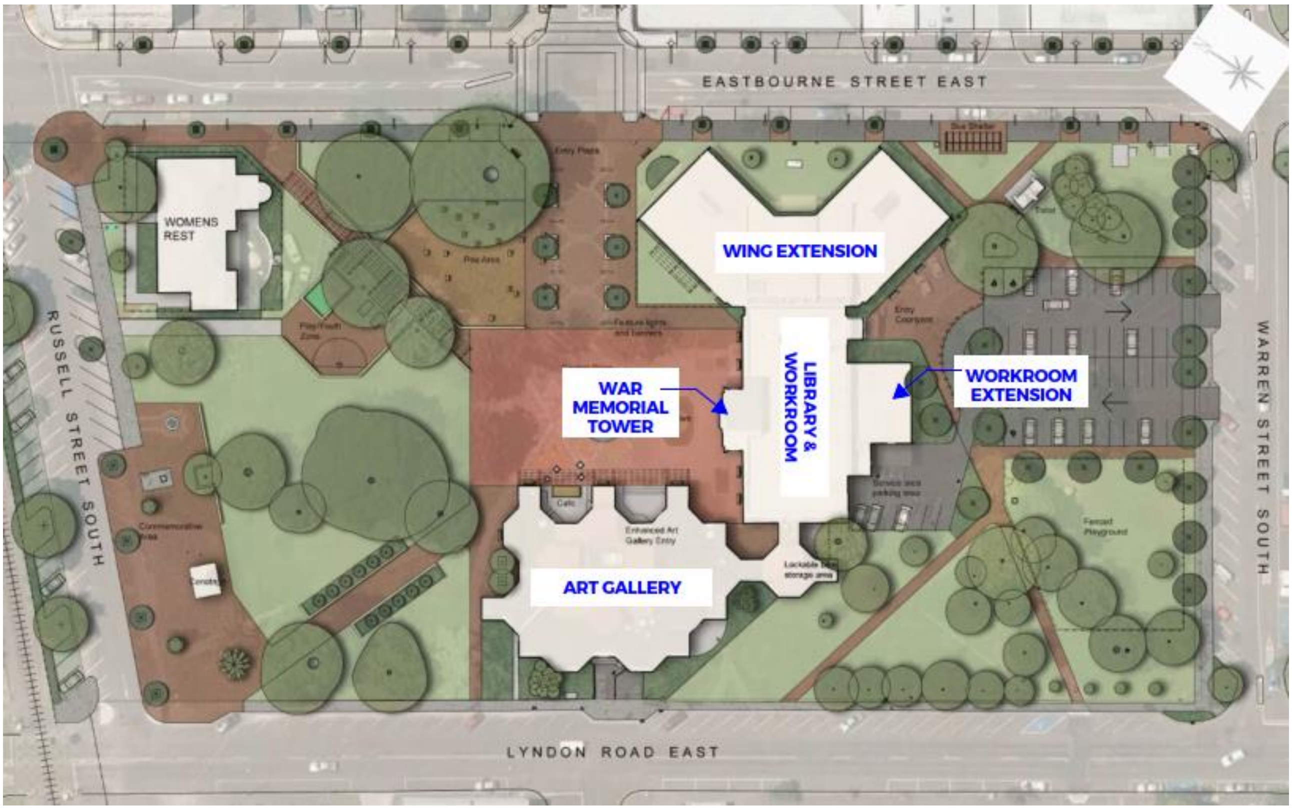

Figure 1: Site Plan of Hastings Library, Hastings - Courtesy of Matthews & Matthews Architects Ltd.

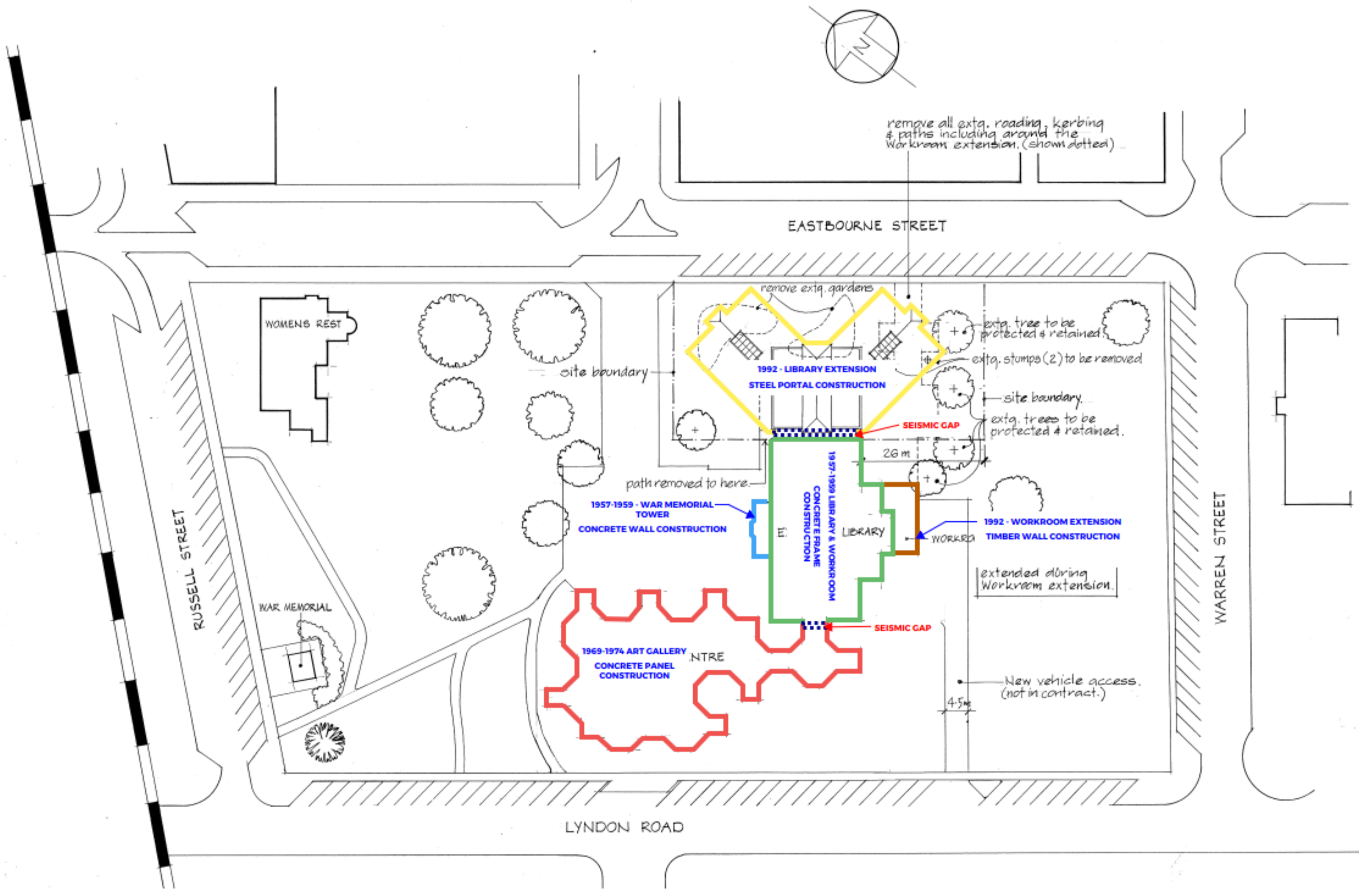

Figure 2: Site Plan Shown Different Construction Stages of Hastings Library - Courtesy of Works Consultancy Services Ltd drawing, Sheet No. 101, Job No, 3/177/5.

Figure 3: Elevation of Hastings Library, Looking East (Showing West Elevation)

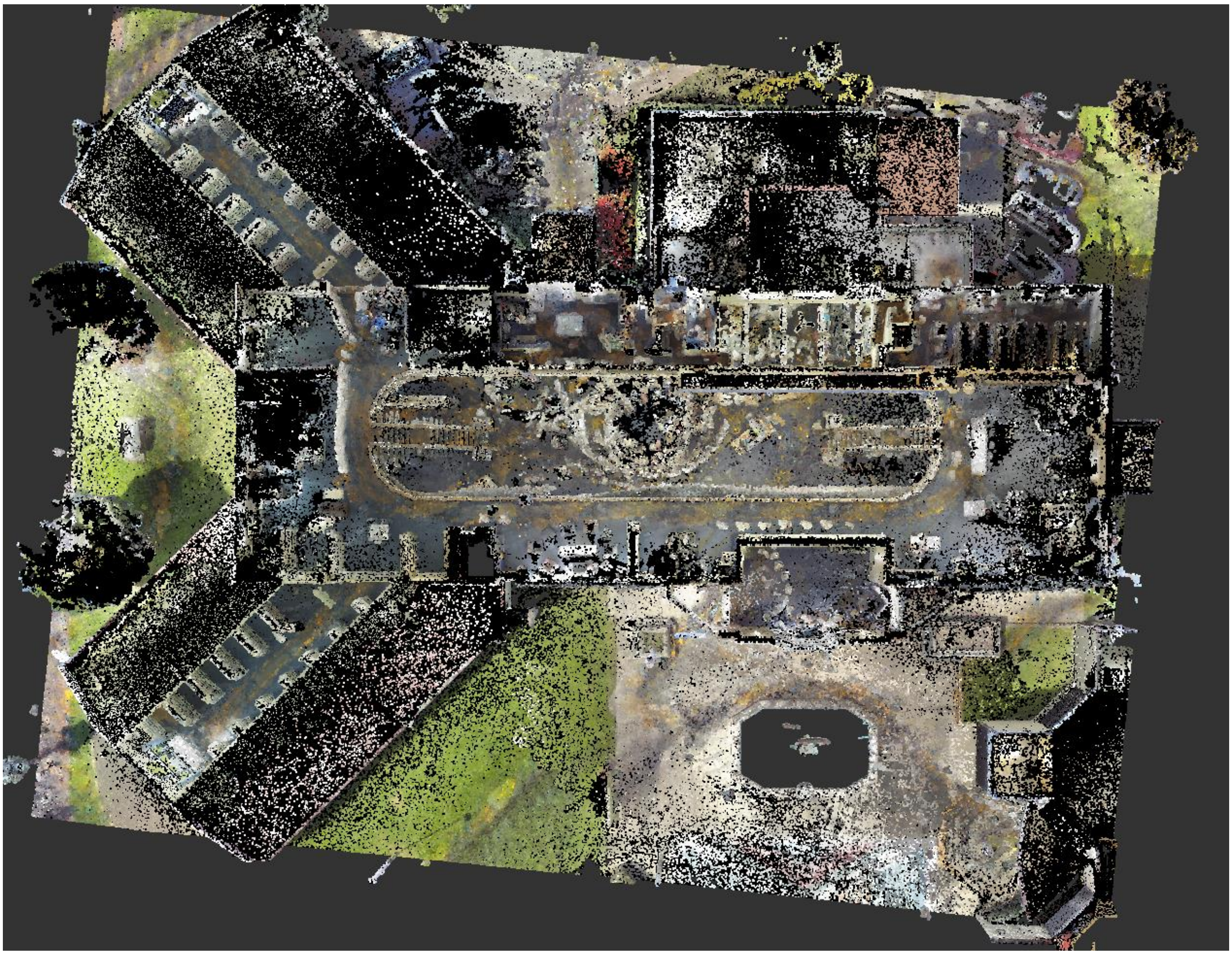

Figure 4: Point cloud of Hastings Library & Art Gallery – Plan View

2 Scope of Work

The purpose of this assessment is to establish the seismic risk of the buildings, to explore compliance with the Building Act 2004 Minimum Standards for Existing Buildings and the Building (Earthquake Prone Building) Amendment Act 2016 and assists with decision making for the proposed redevelopment to the Hastings Library and Art Gallery and.

The assessment and reporting have been undertaken in accordance with The Guidelines[5], published by MBIE, July 2017. The Guidelines have been produced by New Zealand engineering technical societies in conjunction with the Ministry of Business, Innovation and Employment (MBIE) and the Earthquake Commission. The Guidelines came into force on 1 July 2017 and supersede the previous guidance published in 2006 by the New Zealand Society of Earthquake Engineering (NZSEE).

The New Zealand Seismic Hazard Model (NZSHM) was updated and released in 2022. The new NZSHM has resulted an increase of circa 40% to the seismic spectral acceleration demand in the Hawkes Bay Region. Interim advice from the Engineering communities, technical societies and regulator is to continue using the loads as of July 2017. This report has followed this guidance.

The scope of work for this engineering assessment includes the following:

- Review available drawings of the original construction and any improvements undertaken.
- Undertake a non-intrusive site inspection of the existing structure. This includes cross checking critical dimensions on original plans with site measurements.
- Undertake a site-specific geotechnical desktop study to support the assessment (appended to this report.
- Undertake calculations of the seismic capacity of the main structural elements and connections of the building to determine a result for the seismic capacity of each element and hence the seismic capacity of the building as a whole, in percentage of new building standard (%*NBS*)
- Produce a DSA report which will include:
  - A description of the building and its structural composition.
  - Seismic ratings for the various structural elements of the building.

The assessment relates to the overall structural performance of the building. The non-structural elements and other building contents have not been assessed for their seismic performance.

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

3 Sources of Building Data

3.1 Documents

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

- Davies Phillips and Chaplin Architectural Drawings & Hastings City Council Engineer's Department Drawings dated December 1957, Ref No. 925, titled, "Hastings and District War Memorial Library", Sheets 1-14 (architectural) & Sheets S1-S10 (structural).
- Works Consultancy Services Napier drawings package dated 1992, Job No. 3/177/5, titled, "Central Library – Eastbourne Street Alterations and Additions". 80 sheets in total (architectural, structural, mechanical, and electrical).
- Original architectural drawings of alteration of some roof and external canopy by Morgan Flynn Architects (dated 1989)
- Structural Calculations from Building Permit Application Form dated 1992, Central Library Extensions Structural Calculations. Building Permit No. ABAJ59645, 122 pages.
- WSP roof condition assessment report dated 19 September 2022, Job No. 2-S5600.5A, titled, "Hastings library roof condition assessment report", 14 pages.

3.2 Previous Assessments

The following previous assessments have been found:

- Seismic Assessment carried out by Strata Group in 2011.

This assessment was carried out based on the previous guidelines and with a ductility assumption, from which a rating of 52%NBS(IL3) was obtained.

3.3 Geotechnical Information and site geotechnical testing

A site-specific geotechnical report was completed by WSP for this assessment. The report will be issued in the appendices. This report is used as the main source of geotechnical information for this assessment. No specific site investigations were carried out.

Desktop study review was done based on surrounding soil conditions of adjacent buildings.

4 Building Description

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

4.1 Site Geotechnical Conditions

A site-specific geotechnical report[6] was completed by WSP. This report is used as the main source of geotechnical information for the seismic assessment. The report is a desktop study which reviewed all readily available historical data for the site and neighbouring area.

The geotechnical report recommended that the site be assessed Site Class D for structural purposes. Due to the lack of certainty in the soil profile, Site Class C has been used for Parts and Portions accelerations.

The liquefaction potential and other geohazards are discussed in detail in the geotechnical report. Given the potential risk for liquefaction, further site investigations are recommended as there is risk for bearing failure if liquefaction occurs within the influence zone of the footings.

The geotechnical report states that an ultimate bearing capacity of 100kpa should be used and spring values are recommended for the analysis.

Staff has verbally communicated to the structural engineer that there are records of settlement or floor damage in the building to the northwest, but no formal information has been obtained regarding this.

4.1.1 Step Change

The results are inconclusive to determine whether a step change can occur. However, there is a high risk of liquefaction occurring within the footing influence area.

4.1.2 Soil structure interaction

Soil-structure interaction has only been partially considered via the modelling of compression only springs with lower and upper bound stiffness properties.

The report followed the recommendations from chapter C4. A this point in time, it is inconclusive whether the assessment can be categorised structurally dominated, interactive or geotechnically dominated. Further geotechnical investigations and assessment is recommended.

4.2 Building History

The original library building has had the following alterations throughout its life as follows:

- 1969: Connection of building to the new museum room (Art Gallery Stage 1) through the southwest elevation. Opening created through the masonry cladding for new corridor.
- 1992: Overall refurbishment to the library with construction of extension, addition of mezzanine area to the southwest and northeast elevations in steel and timber. Extension of the workroom area in light timber and steel, and relocation of existing roof. New Entrance roof to the main library. Demolition of walls.

[6] “Hastings Library and Art Gallery – Detailed Seismic Assessment Geotechnical report, June 2023”

4.3 Building Description

The Original Hastings Library building is a two storey in-situ reinforced concrete frame structure, combined with shear walls in all the four elevations. Within the library, there is an eccentric concrete shear wall structure (war memorial tower or Hall of Memories) at the northwest side. The complex was designed by Engineer's Department at Hastings City Council (Structural Engineer) in 1957.

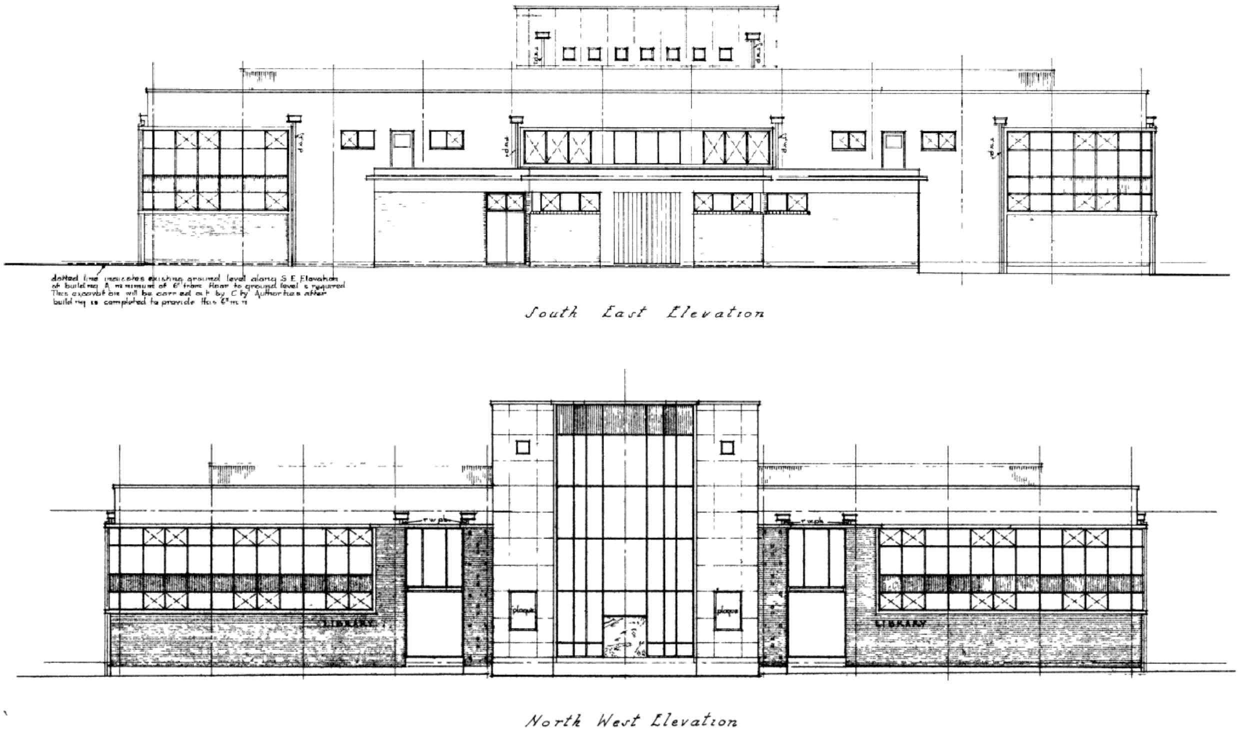

Figure 5: Original Elevations to the Main Entry and Hall of Memories respectively

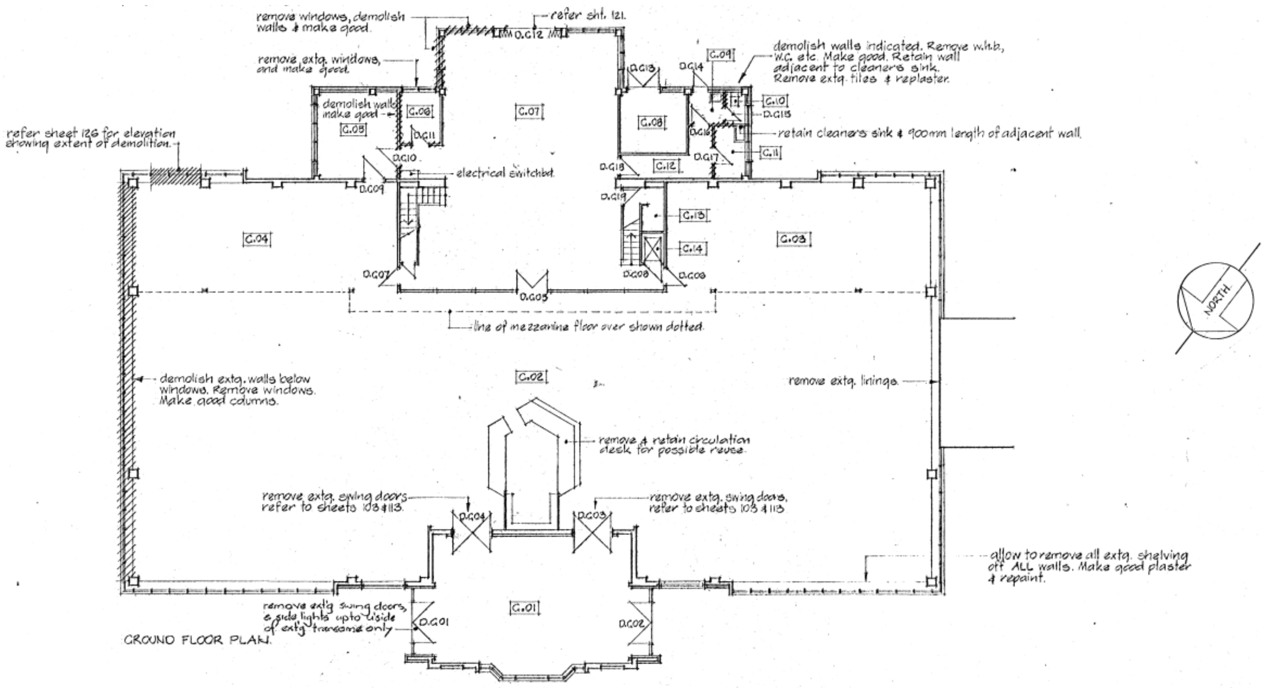

Figure 6: Ground Floor Plan 1957 - Library

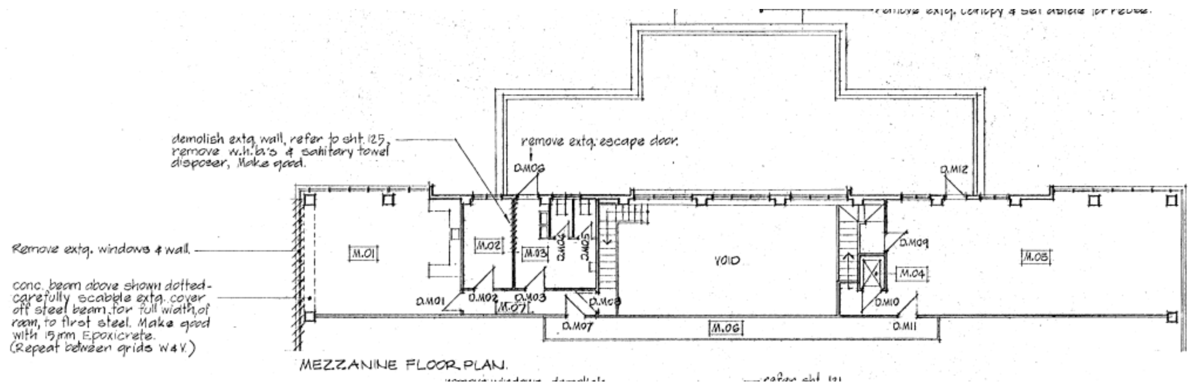

Figure 7: Mezzanine Floor Plan 1957 - Library

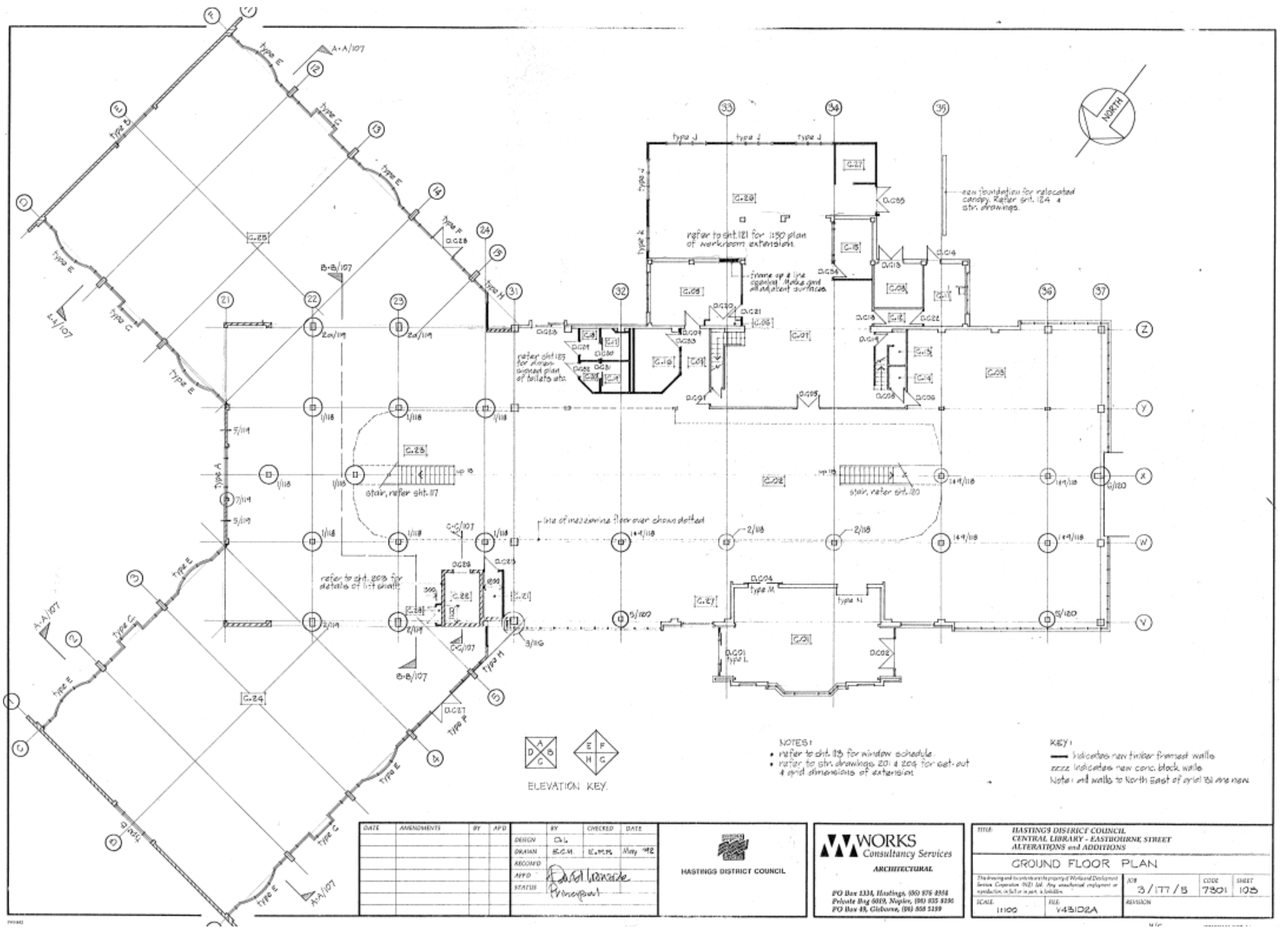

Figure 8: Ground Floor Plan 1992 - Library

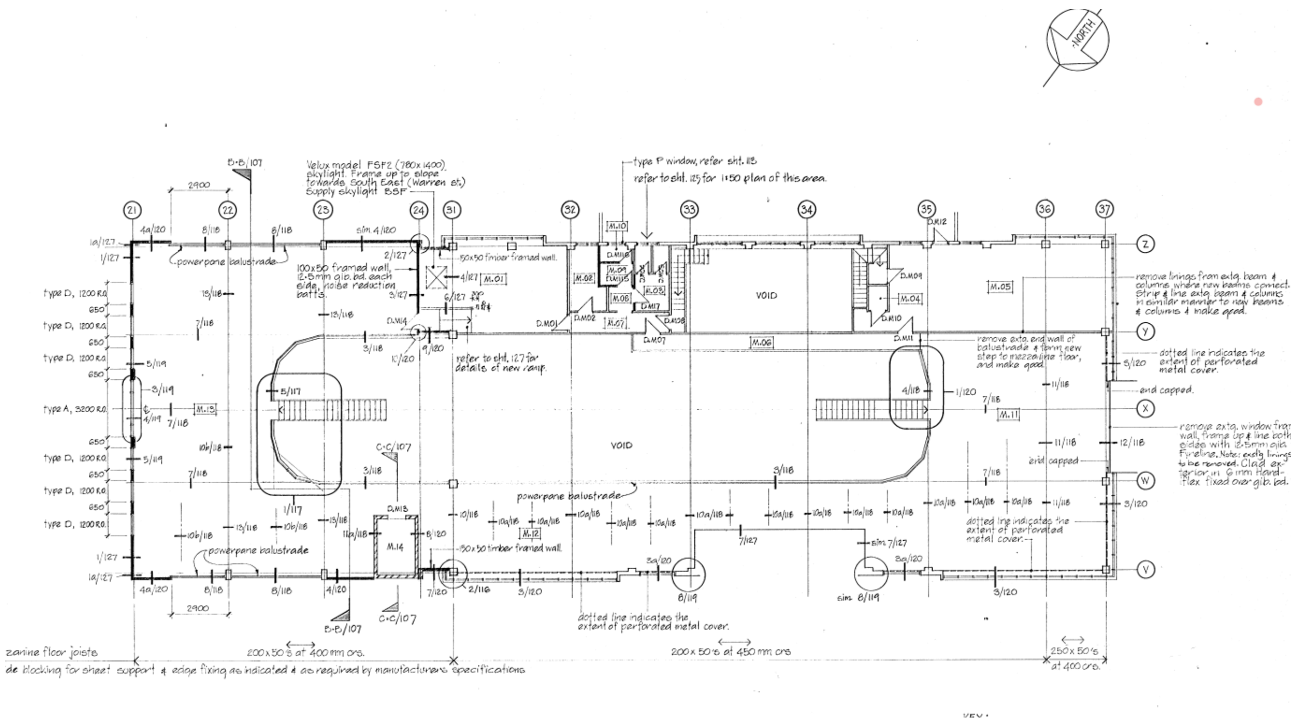

Figure 9: Mezzanine Floor Plan 1992 - Library

The original wing comprises a mezzanine area where toilets, kitchen and a storage area are located, built in steelwork and connected via a corridor. There is a workroom space for administrative offices on the lower roof area to the east, built also in reinforced concrete frames with some cavity walls and masonry infills. This workroom space was also part of the original entrance to the building. Refer to figure 6 & 7 for original ground floor and mezzanine floor plan in 1957.

The building is generally clad with plaster system and glazed on all sides. Brick veneer cladding is present at the lower level on Northwest side only.

Major extensions and alterations have been designed by Works Consultancy Services Ltd in 1992. During this stage, the original building was changed radically. An addition of new mezzanine floor area was added to the south and northwest that connects with the new extension. The original entry and workrooms were closed to the public and extended to the southeast side. A new entry was placed by the last bay to the right on the southeast elevation above.

New wing extensions were added to the northeast of library building, which comprises of a central 2-storey wing that connects to the library at mezzanine floor level. In addition, two new wings at 45 degrees from the central wing and library were added as additional library floor area. The new central wing is comprised of steel moment frames with concrete masonry shear walls, while the two wings are comprised of steel portal frames and concrete masonry walls. The new addition is generally clad in lightweight timber plastered cladding and glazing. Refer to figure 6 & 7 for new ground floor and mezzanine floor plan in 1992.

Seismic gaps between the yellow and green highlighted buildings (wing extension and library), red and green highlighted building (art gallery and library) were identified from the existing plans which indicates independent behaviour during seismic event and therefore separate assessment was conducted for each building (refer to Figure 2).

The overall building footprint is approximately 1800 square metres and 6.0 metres in height to the apex from the ground level. The tallest parts of the building are the war memorial tower and central raised section of the wing extension with a total height of approximately 9.5m.

4.4 Site Visits and Investigations

A non-intrusive site inspection on 14th April 2023 by WSP Structural Engineers. This was undertaken to check critical dimensions of the structure and gain better understanding of the existing structure, layout and identify any potential discrepancies. The inspected areas were found to generally conform to the existing drawings. Site observation was limited due to the coverage of ceiling and wall lining over building elements.

Point cloud survey was carried out during the inspection with handheld scanning device to capture accurate real-life dimension and layout of the existing building. A full orientated point cloud 3D building information model (BIM) was produced to assist with structural assessment.

The block walls were not scanned for reinforcement during the inspection. It was therefore assumed that the block walls reinforcing layouts conform with the existing drawings.

4.5 Original Library Building

4.5.1 Gravity Resisting System

The primary gravity structure of the original library building consists of timber purlins spanning between steel trusses. The steel trusses span the width of building are supported on top of the perimeter beams and partially in the concrete walls of the Hall of Memories.

The gravity load path for the main roof is comprised of the reinforced concrete in-situ frames as walls that are present around the perimeter. For the original main entry area and current offices, the gravity structure is comprised of a cast in-situ reinforced concrete slab supported on reinforced concrete frames. Plain round bars with standard hooks form the reinforcement of these elements.

To the southeast elevation of the main hall, the original mezzanine area and walkway is comprised of steel rolled joists, steel angles and steel channel sections as gravity support. The floor system is comprised of timber joists and particle board which was refurbished during 1992.

The new mezzanine area is supported by steel beams and square hollow sections that form gravity portal frames. The mezzanine floor is comprised timber joists with particle board as flooring substrate. New concrete pads were laid where necessary to provide support to the new columns.

In 1992, the workroom was also extended to the southeast side of library during the same stage. Extension to the workroom was achieved with the provision of steel beams and posts connected to the original concrete beams and the interior and exterior was completed in light timber frame construction and attached to the library building without seismic gap. Some of the external masonry partitions were also removed.

The foundation system at the library and workroom extension is comprised of a concrete slab on grade with pad footings below concrete /steel columns and continuous ground beam around perimeter of building with some internal ground beams.

In both existing and new mezzanine, bolted/anchored connections to the existing structure provide support to the floor beams for gravity support.

At roof and mezzanine floor level, the timber rafters/joists span between the library and new extension with gravity only connection with insufficient capacity to transfer lateral seismic load, therefore, a seismic gap was considered to present between the two buildings.

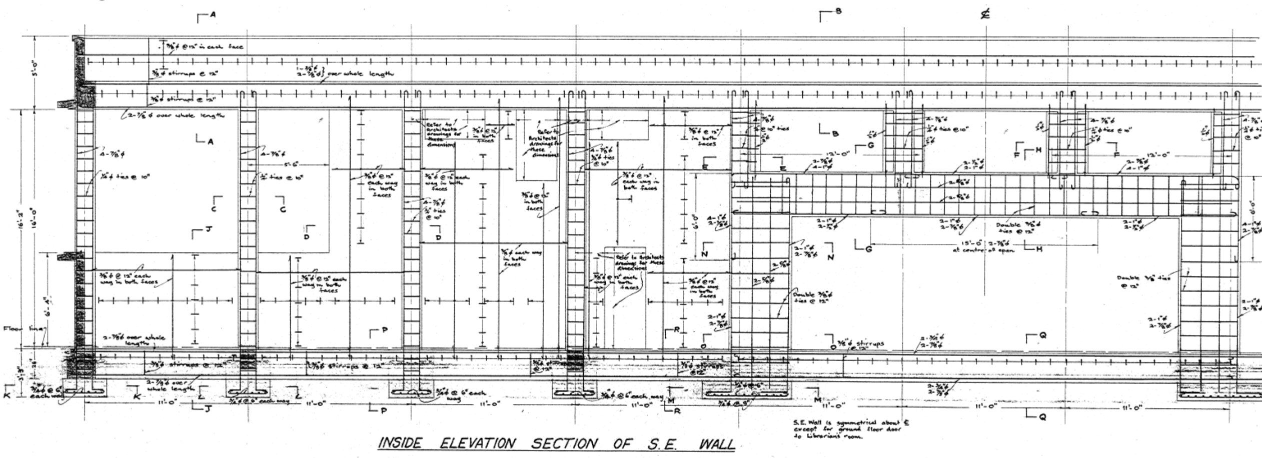

Figure 10: Typical Concrete Moment Frame with Infill Elevation on Library (Grid Line Z)

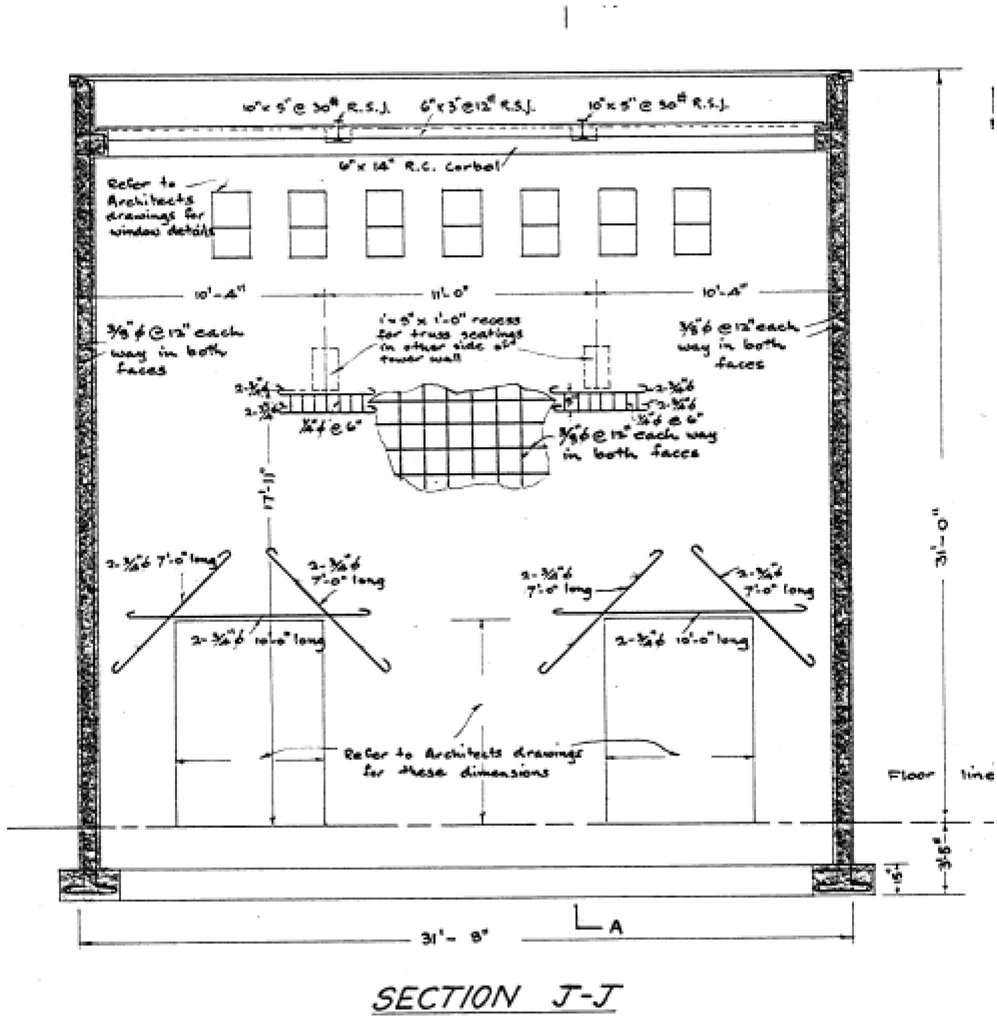

Figure 11: Concrete Shear Wall Elevation on War Memorial Tower

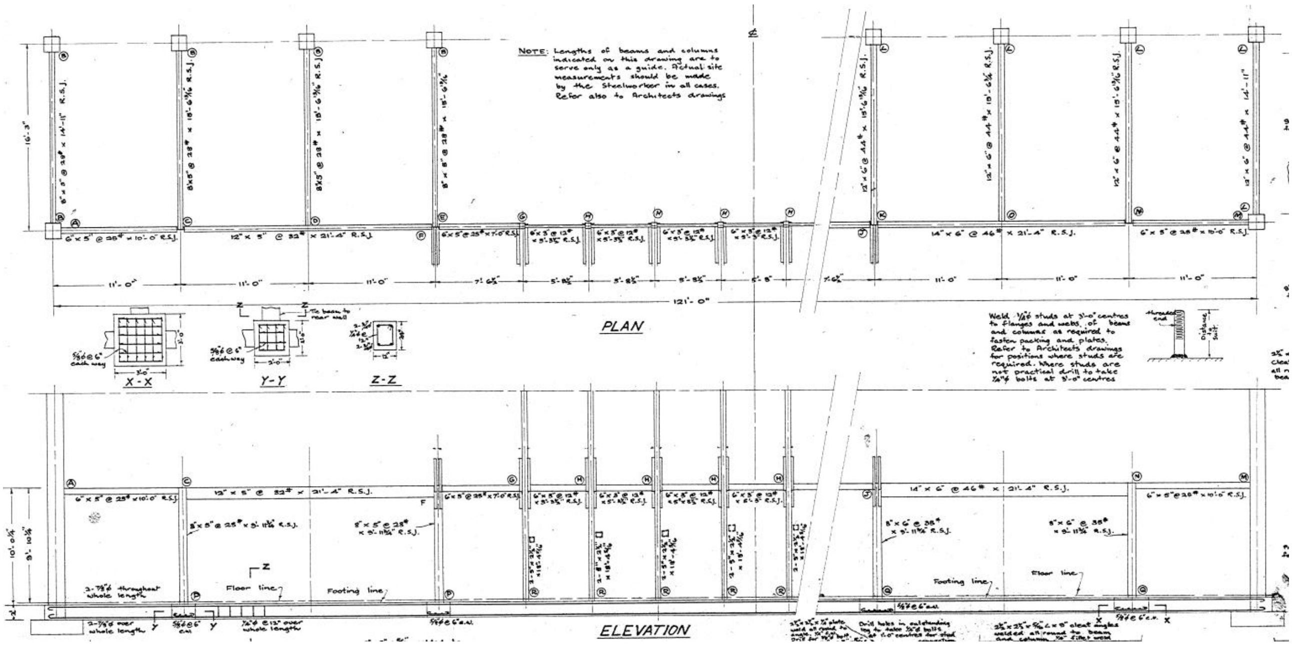

Figure 12:: Original Mezzanine Area Steel Structure Plan and Elevation Respectively

4.5.2 Lateral Load Resisting System

The lateral load resisting system of library and workroom, in both the longitudinal and transverse directions, is provided by the concrete moment frames around the building perimeter and by the concrete shear walls in line with these frames.

For the lower workroom area, unreinforced masonry infill panels were provided as cavity walls around the external elevations. These will interact with the concrete frames to provide lateral support and the presence of openings might create short column effects.

The original steel frame mezzanine is provided to mainly act as gravity support frame, with cantilever action from the footings. However, the connections to the concrete frames will provide stability in both directions as this will be a stiffer load path. The walls at workroom extension are lined with plasterboard bracing system for lateral load resistance but given the main steel roof beams are connected back to the original concrete frames, these walls might not be efficient in resisting lateral forces and most of the workroom roof inertia will be transferred to the concrete frames at the roof level.

There is no reliable roof diaphragm present at the main library open space where load will be transferred to lateral force resisting system through out of plane action of the portal frames or through the perimeter concrete beams via the timber ceiling joists in compression/tension on the longitudinal direction. In the transverse direction, the bottom chords will act as direct struts transferring load at each side of the perimeter beam. For simplicity of assessment, it is reasonable to assume load is distributed by tributary width to each concrete frame/wall through the load path described previously. In contrast, the roof system at war memorial tower is comprised of steel rafters in orthogonal directions with timber joists span them where fibrous plaster ceiling was present underneath; therefore, it is assumed to act as a flexible roof diaphragm and transfer loads via direct axial load or minor axis bending.

Steel bracing was provided on top of timber rafters at workroom extension to distribute loads to lateral resisting system. At the original workroom area, a concrete slab will act as a diaphragm to brace this portion back to the library in the longitudinal direction.

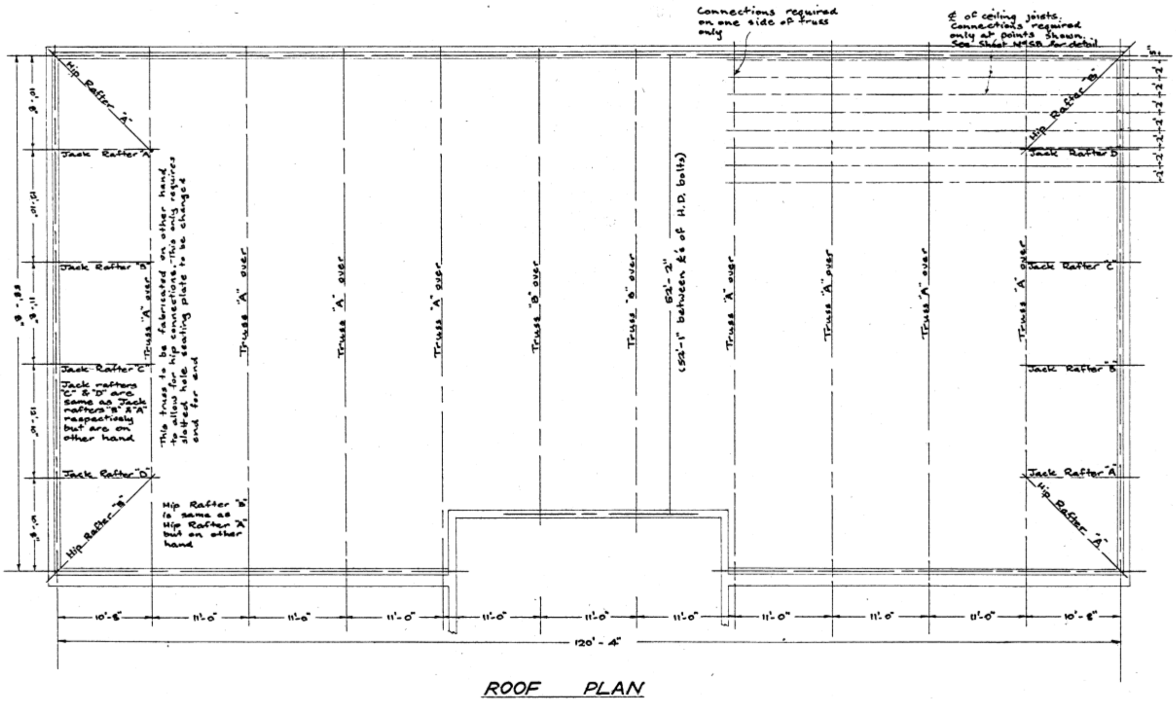

Figure 13: Original Library Main Roof plan

At the new mezzanine floor level, nominal cantilever column action and some frame actions are main lateral resisting mechanism. Nominal diaphragm action with the timber flooring will exist but only as simple supported beam and not cantilevering out the end elevations. Therefore, lateral load is assumed to distributed by tributary width to each steel moment frame and lateral support is relied upon the concrete bolts at the ends of the frames.

The library and war memorial building is founded on a shallow foundation system comprising of concrete slab with individual concrete pads below the concrete and steel columns. Above these are ground beams around the building perimeter to tie separate pads together. Lateral loads at ground floor level will be resisted by frictional resistance between the shallow concrete foundation and ground. New extension to the workroom follows similar type of foundation construction and lateral resisting mechanism.

The lateral support for original external canopy is provided by anchored connection back to the main workroom area:

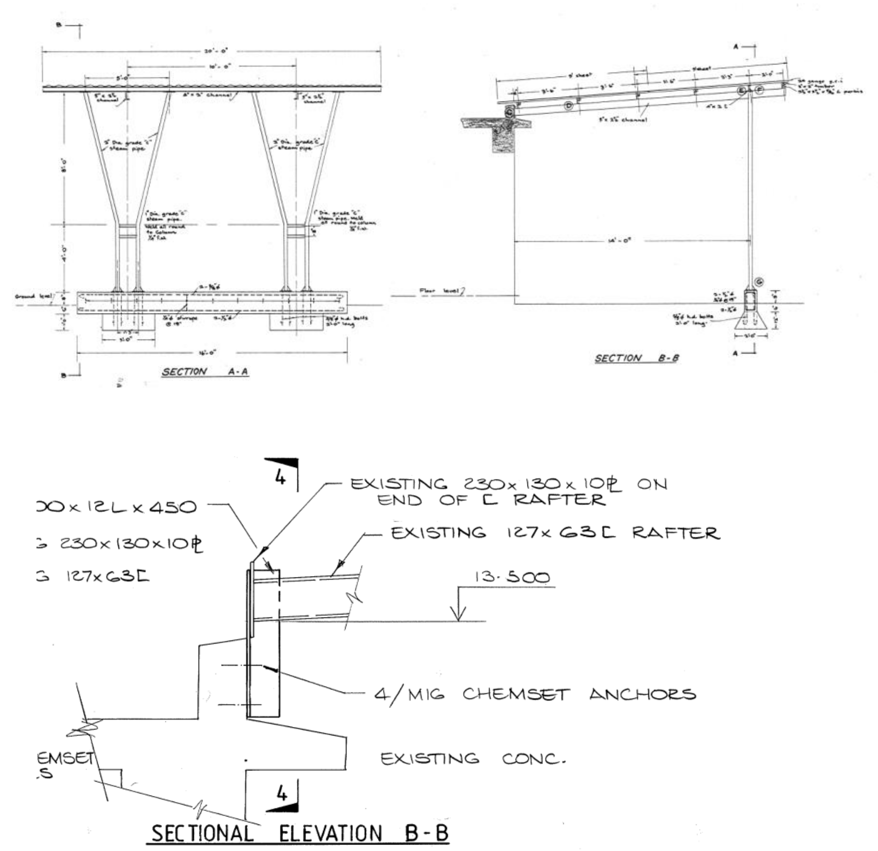

Figure 14: Original Exterior Canopy and Anchored Connection Back to the Workroom

4.6 1992 Library Extension

4.6.1 Gravity Resisting System

The primary gravity structure of the extension wings roofs consists of timber purlins spanning between steel portal frames (red on figure below). The steel rafters of these frames span the width of building and are supported by concrete encased steel columns which in turn supported by concrete pad foundations. Some of these frames are supported also on the main wing through a channel section that spans between the main columns of the central wing The end walls at each wing extension are comprised of cantilevered concrete block wall and founded on shallow reinforced concrete ground beams and footings.

The central wing roof structure is comprised of timber purlins that are supported on top of steel portal frames made from universal beam sections or square hollow section. The first floor is also supported by a continuation of these frames and the universal beams are supported on square hollow section columns. Reinforced concrete masonry walls provide also additional supports for the steel frames in this area. Foundations are shallow reinforced concrete pads and beams for the individual columns and walls. The lift shaft is comprised of reinforced concrete masonry walls with a shallow pad foundation.

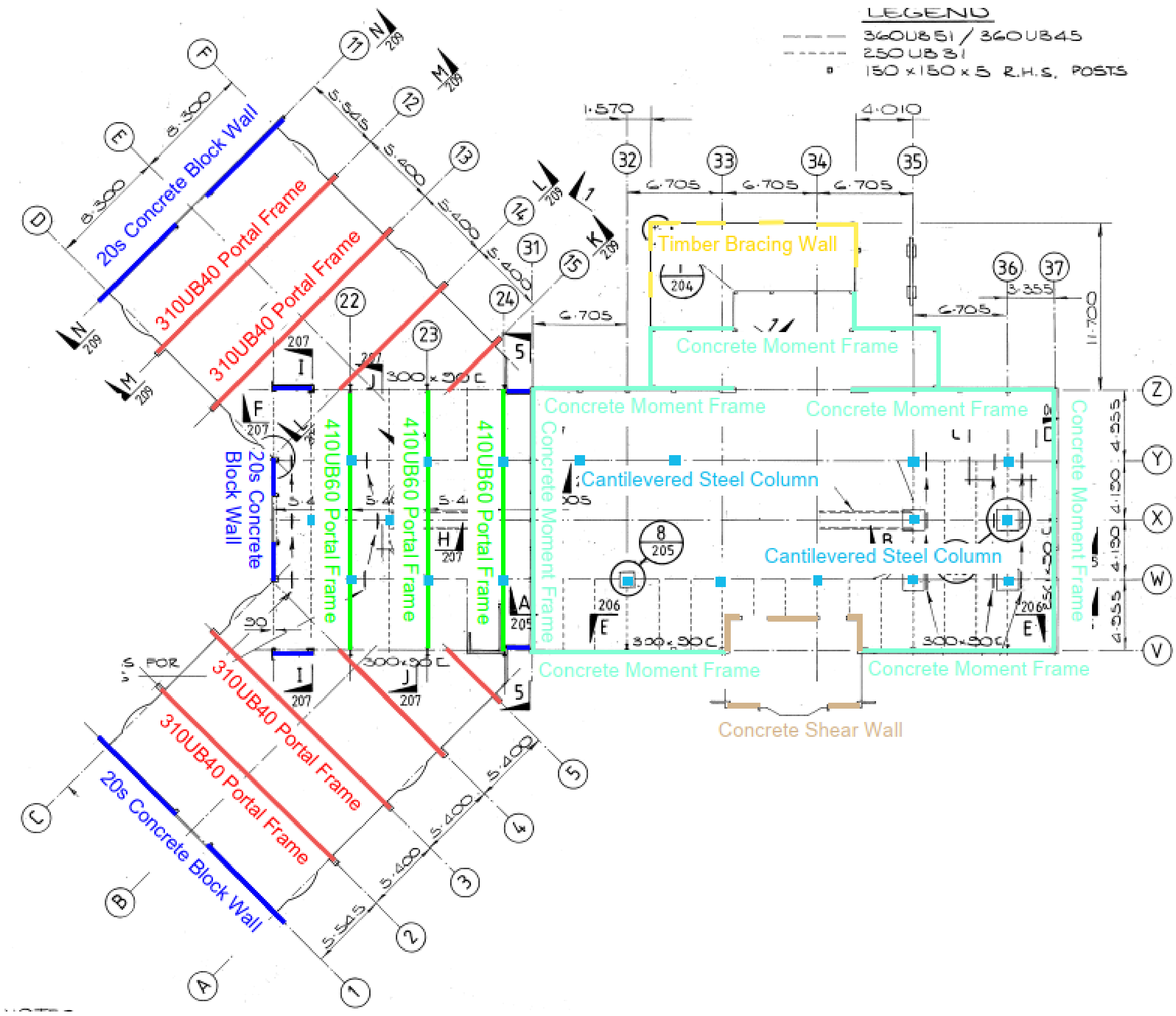

Figure 15: Plan View Showing Types of Lateral Bracing Element in the Longitudinal and Transverse Direction (Schematic Layout Produced for Assessment Only)

4.6.2 Lateral Load Resisting System

The lateral load resisting system of the library extension wings comprises steel portal frames in the transverse direction. In the longitudinal direction, restraint is provided by out-of-plane stability of the end masonry walls (although flexible) and by tie backs to the main wing along the knees of the portal frames. These tie backs connect at the floor level near the concrete masonry walls of both the central walls of the central wing and partially the lift shaft on the northern wing. Some of these portal frames are not continuous and rely on the main wing to transfer shear and axial demands as part of the lateral system.

There is no roof diaphragm and roof continuity at the apex due to the skylight at the centre of the wings. Load transfer longitudinally relies partially in minor axis bending of the portal frames and partially on the connections between the purlins and the portal frames and the end steel channel section at the central wing.

For the central wing, tension-only roof diagonal braces were provided only at the lower roof on Grid line 21 -24 and Grid line V-Z. The central portion of the roof transfers inertial loads to the lower roof level via portal frame action. The lower roof is then braced transversely by steel portal frames. The end elevation to the northeast has minimal frame actions through SHSs and additional tension only braces have been provided in an asymmetrical fashion. The lower first floor mezzanine area is then braced transversely and longitudinally via portal steel portal frames and reinforced concrete masonry walls.

The first floor of the central wing is comprised of timber joists with particle board as flooring that acts as a flexible diaphragm, distributing the loads between the adjacent bracing lines. In the longitudinal direction, the central portion of this floor depends on diaphragm action and/or minor axis bending of the steel main joists to transfer the loads to the main bracing lines at the edges of the building.

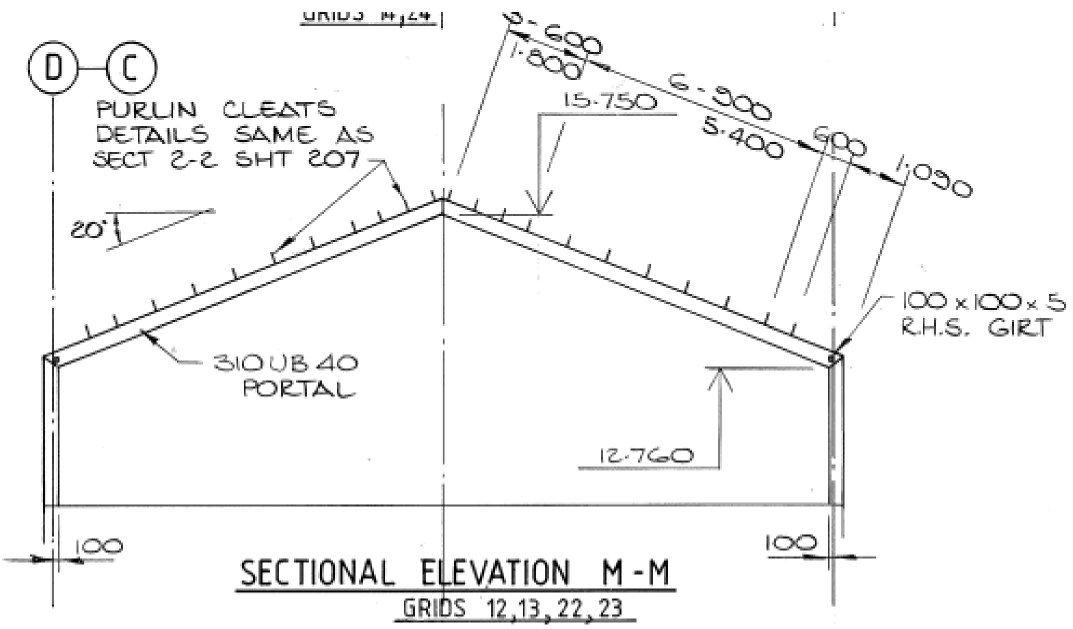

Figure 16: Typical Steel Portal Frame Elevation Extension Wings (Grid Line 2-3 & 12-13)

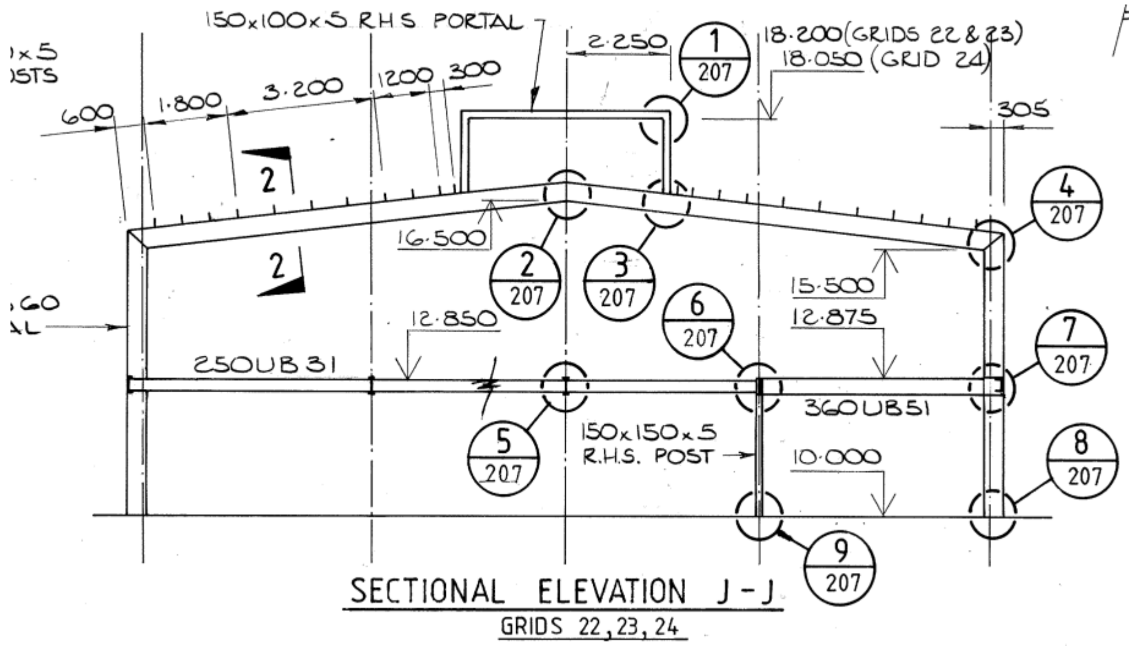

Figure 17: Typical Steel Portal Frame Elevation on Extension Central Wing (Grid Line 22-24)

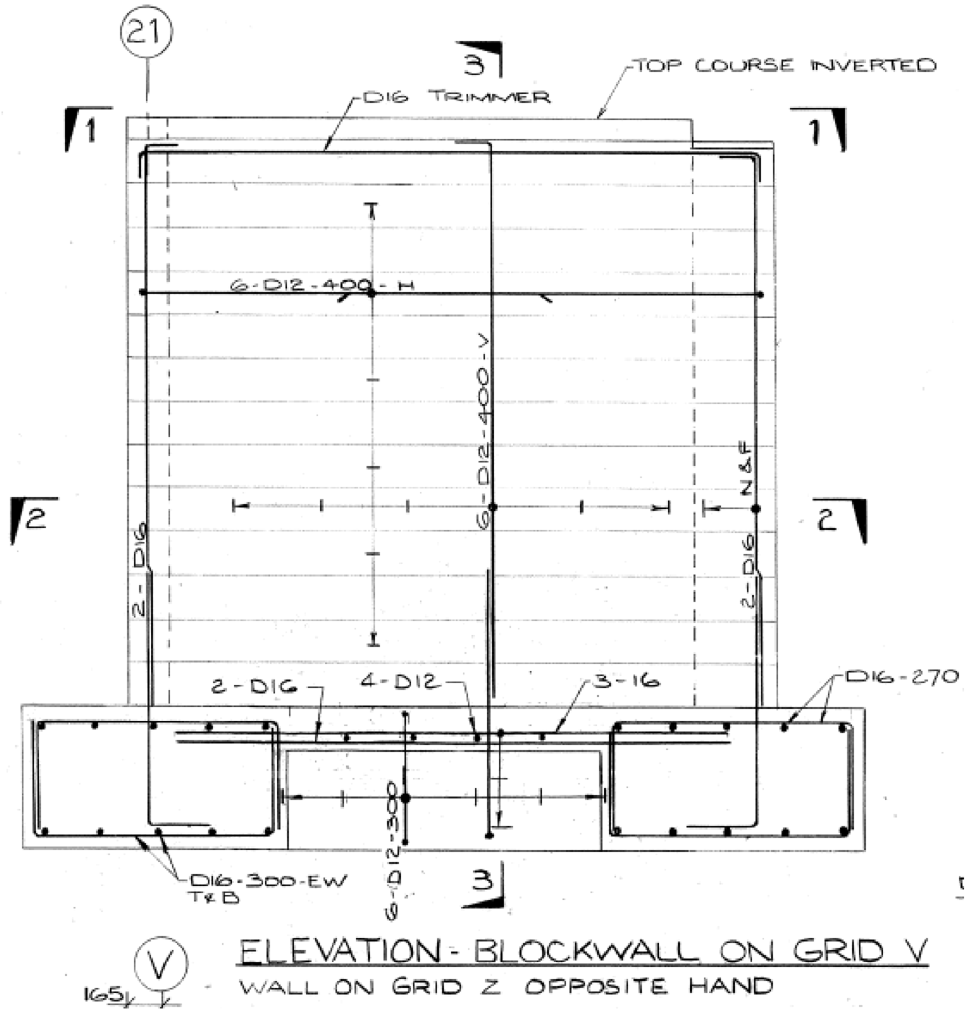

Figure 18: Typical Concrete Block Wall Elevation on Extension Central Wing (Grid Line 21, Z and V)

4.7 Secondary Structural and Non-Structural Elements

The following types of secondary structural and non-structural elements were found during the site visit and drawings and are explained further below.

4.7.1 Internal partition walls in the original building

There are timber internal partition walls at the ground level, first floor level and the workroom area. The assessment of the internal partition walls was excluded as it was considered a low risk compared to the other findings of the assessment in the main building.

4.7.2 Mechanical plant on roof (external)

From the aerial photos, it is possible to observe that there is a mechanical plant in the middle of the hexagonal space roof. There is also a previous report with information about the possibility of adding several units on top of the roof steel portal trusses in the original wing. We have not been able to confirm if this work has been carried out.

4.7.3 Ceilings

Ceilings in the new wings and the original building are in general light weight and of suspended construction. Given the age of the building, it is likely that bracing is minimal, therefore, detailed assessment has not been carried out for these elements.

4.7.4 External end walls (at extension wings)

The external walls at the end of the two 45-degree wings consist of timber framing supported laterally by the timber purlins back to the main roof and partially by the main end concrete masonry walls. Without sufficient information about the connections of this wall back to the building, it will be hard to assess the possibility of its damage during an earthquake. If additional strengthening will be carried out, it is recommended that these details are investigated prior to construction.

4.7.5 Mezzanine Stairs

The access to the first level mezzanine floor is via two steel stairs connected to the mezzanine and anchored to the ground slab. Given the limited capacity of these connections, it is likely that these elements will suffer damage from the imposed deformation by the main building movement due to incompatibility of displacements.

4.7.6 Internal timber stairs to original mezzanine

The internal timber stairs to the original mezzanine area are considers lightweight and therefore have not been assessed in this exercise. Alternatives means of evacuation exist if these were to fail during an earthquake.

4.7.7 Unreinforced cavity veneer walls

The original building has unreinforced masonry walls around its perimeter. Some of these walls are fixed back to a concrete wall but in many cases are freestanding or captive within a concrete frame. These has been considered in the assessment as independent wythes as existing ties cannot be relied upon due to durability.

4.7.8 External canopy

The original external canopy was relocated to the side of the work room during the 1992 alterations. This canopy is tied back to the main concrete building.

4.7.9 Entry canopy

A glassed double entry to the library is the main entry and evacuation route. We could not find any information on this structure and thus has exclude this from the assessment.

4.8 Building Irregularities

For the original 1957 building, the following irregularities exist as per the classification in NZS1170.5:

- Potential weak storey due to short-column effect on the south-eastern elevation.
- Weight mass irregularities

For the 1992 extension, the following irregularities exist as per the classification in NZS1170.5:

- Weight mass irregularities
- Vertical stiffness irregularity
- Discontinuity in capacity
- Torsional sensitivity

4.9 Building Condition

The 1992 extension showed no signs of deterioration. The original portion is in good condition and no apparent durability issues have been identified during the site visit. However, it is known that there is a leak in the workroom area of the original building and water ingress has been constant in this area. A closer inspection by the WSP team is planned in the near future.

5 Assessment Criteria

5.1 Material Properties

The following material properties were used during the process of this assessment. The assessment makes provision for possible variability of materials in a qualitative manner on how these will affect the numerical results obtained.

Table 3 – Material Properties Used for The Assessment

Material	Nominal Strength	Probable Strength	Justification
Concrete	Unknown	30MPa	From C5
Reinforcement	Unknown	f_y= 280MPa f_u= 475MPa ε_{su}=0.1 ϕ_O=1.25	Table C5.4 of C5 Guidelines.
Structural steel (1958)	247MPa	284MPa	Table C6B.5 of C6 Guideline
Structural Steel (Post 1990)	260MPa	300MPa	Table C6B.5 of C6 Guideline
Weld materials (1958)	Unknown	265MPa	
Weld materials (Post 1990)	Unknown	410MPa	

5.2 Seismic Loading

5.2.1 Design life

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

5.2.2 Importance level

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

5.2.3 Seismic loading spectrum

The structure has been assessed following the seismic actions prescribed on earthquake loading standard NZS 1170.5:2004. The parameters used for the assessment is as per Table 4.

Table 4 – Seismic Loads Parameters

Parameter	Value	Comments
Site subsoil class	C/D	Insufficient information available. • Subsoil Class D – 'Deep or Soft Soil Site' for spectra; • Subsoil Class C – 'Shallow Soil Site' for parts and portions;
Z	0.39	Seismic hazard factor for Hastings
Return period factor, R_u	1.0	Importance Level 2 for 50 years design life
Near fault factor, N (T, D)	1.0	> 20 km from nearest major fault.

The Sp factor has been considered as 1, considering that brittle behaviours exist within the building for both the steel and concrete portions.

5.3 Seismic Weight

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

The floor-imposed loads are not shown below as these are directly taken by the subgrade.

Table 5 – Imposed Seismic Mass

Floor Level	Imposed Weight, Q_i	Combination Factor, ψ_E
Floor Reading rooms	2.5kPa	0.3
Storage rooms	5	0.6
Roof	0.25kPa	0.0
Kitchen, toilets, etc.	2.5kPa	0.3

Super dead loads such as screed, ceiling, services, partitions, etc. have been included in the seismic mass.

5.4 Assessment Methodology

In general, the assessment was undertaken as a forced based assessment with assumption of elastic behaviour or very limited capacity of energy dissipation for the main structural systems comparing the demands from non-linear ETABS analysis model to the structural element capacities using the current standards of NZS3101:2006 for concrete and NZS3404:1997 for steel members.

A non-linear analysis was undertaken to consider a line spring under the bearing strip footing to allow for any uplift. A lower and upper bound value of $5MN/m^3$ and $20MN/m^3$ was used respectively to consider the uncertainty of the existing ground condition. The result was as expected with lower bound spring giving more demands to the structure with lower soil bearing pressures; compared to upper bound spring giving lower demands to the structure with higher soil bearing pressures. For the original wing, demands under elastic loads proved to rock the structure introducing deformed shapes and demands that were not in line with expected non-rocking behaviour. Therefore, for the original portion we have also analysed the results under a fixed foundation scenario as an upper bound of force distribution if uplift of the foundations is prevented.

In ETABS, automatic seismic load with distributed mass was considered given the different elements and levels in the original portion of the building and between the wings of the extension.

For concrete elements and walls, sections capacities and deformation capacities have been calculated as a first step for a SLaMA analysis. Connections and load paths have been checked against demands from the numerical analysis.

6 Detailed Seismic Assessment Results

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

(*These results are considered draft until a final liquefaction assessment is carried out for the site).

Table 6 – Assessment Results for Individual Building Components and/or Systems for the Original Library Building 1957

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
Concrete Moment Frames – Transverse Direction- Northern Elevation	25%	• Lack of strength and deformation capacity of beam-column joint failure and reinforcement anchorage failure due to the lack of transverse reinforcement in the joint panel zone and inadequate anchorage of bars. • Lack of deformation capacity can lead to softening and collapse of the frame.
Concrete Moment Frames – Transverse Direction – Southern Elevation	30-40%	• Lack of strength and deformation capacity of beam-column joint failure and reinforcement anchorage failure due to the lack of transverse reinforcement in the joint panel zone and inadequate anchorage of bars. • Lack of deformation capacity can lead to softening and collapse of the frame.
Concrete Moment Frames and Walls on Southeastern elevation	45-65%	• Lack of strength and deformation capacity of short column due to presence of infill resulted in shortening of clear shear span, reinforcement anchorage failure due to inadequate lapping length and foundation uplift. • Lack of deformation capacity can lead to softening and collapse of the frame.
Concrete Moment Frames and Walls on Northwestern elevation	45-75%	• Lack of strength in columns and small shear walls. Uplift of foundation. • Lack of strength and increased deformations from footing uplift can lead to excessive displacements with partial or total collapse of the frame.
Roof trusses and associated connections to the main roof and steel members	20%	• Lack of shear capacity of cast-in bolts into concrete. Lack of compression capacity at bottom chord due to insufficient section size and full lateral restraints.

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
		• This can lead to loss of seating of the trusses and additional bending of the roof beams that could lead to a partial collapse of the roof structure, particularly at the central portion.
Roof Perimeter Beam	25%	• Lack of out-of-plane bending capacity to span between main lateral restraints in both longitudinal elevations. • Lack of strength and deformation capacity can lead to increased displacements with partial collapse of the frame elevation.
Reinforced concrete walls and foundations at Hall of Memories	45-65%	• Lack of foundation uplift capacity due to insufficient ground beam/pad size, insufficient bearing capacity due to poor ground bearing condition. • Lack of foundation capacity can lead to increased deformations in the main structure. However, full instability of the Hall in the transverse direction is unlikely.
Internal Mezzanine steel structure 1992	40%-55%	• Lack of tensile capacity of end connections to transfer loads in the longitudinal direction. Lack of capacity of end columns to transfer demands to the foundation and perimeter roof beam. • Lack of strength can lead to secondary load paths with the risk of increased deformations for the gravity structure of the mezzanine that can lead to a partial collapse.
Internal Mezzanine steel structure 1957	65%	• Lack of tensile capacity of end connections to transfer loads in the longitudinal direction. Lack of end columns capacity to transfer demands to the foundation and perimeter roof beam. • Lack of strength can lead to secondary load paths with the risk of increased deformations for the gravity structure of the mezzanine that can lead to a partial collapse
Mezzanine timber diaphragm	75%	• Lack of shear capacity to span between bracing lines. • Lack of strength can lead to increased deformations that could

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
		affect gravity structure with localized collapse.
Lower roof concrete area	40-45%	• Lack of bracing capacity and diaphragm connections to southeastern elevation, interaction with infills and potential short column effects. • Shear failure of columns can lead to partial collapse of the gravity frames.
Perimeter masonry walls and brick cladding	20%	• Lack of out-of-plane capacity and ties to the structure. • Out-of-plane failure of wall or falling of brick cladding at height could potentially lead to a significant life safety risk. • Intrusive investigation or condition assessment is recommended prior to future development to confirm the condition/location of veneer tie.
External relocated canopy	45%	• No detailed assessment was carried out. • Lack of connections can lead to increased deformations and collapse
Extension canopy to workroom	35%	• Lack of tensile capacity of connections of roof elements back to the main concrete frames • Partial collapse of roof beams might be expected.
Entry Canopy	To be confirmed	• No information has been found, to be confirmed in subsequent stages.
Mezzanine stair	40%	• Insufficient shear capacity of base anchor connections. • Lack of bottom positive connection can also lead to damage of the upper connection, leading to a partial collapse and loss of egress route.

Table 7 – Assessment Results for Individual Building Components and/or Systems for the Library Extension Building 1992

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
Steel Portal 310UB at wings	35%	• Lack of full lateral restraints to the bottom chord to guarantee ductile behavior. • Lack of deformation capacity can lead to instability and collapse.
Steel Portal 310UB at wings - foundations	75%	• Insufficient bearing capacity • Failure can lead to increased deformations and softening.
End block masonry wall at side wings	85%	• Lack of out of plane overturning capacity. Insufficient connection to tie back to the building. • Out-of-plane collapse could potentially lead to significant life safety risk.
Struts along knees in longitudinal direction at wings	<33%	• Lack of shear capacity of end connection to main wing concrete masonry walls. • Lack of strength and positive load path will lead to increased displacements for the wings with partial or total collapse of the portal frames.
Main 410UB/360UB Portal frames at central wing	40-75%	• Lack deformation and strength capacity due to lack of full lateral restraints to critical flange. Lack of foundation capacity. • Lack of capacity deformation and increased foundation demand can lead to reduced stiffness with increased deformations and partial or total collapse of the frames.
Connections of mezzanine beams to shear walls	20%	• Lack of shear capacity at end connections due to insufficient anchor edge distance. • Failure of beam end connections would lead to loss of primary load path with increased connections and potentially collapse of the steel frames.
Roof steel braces	35%	• Insufficient tensile strength with asymmetrical layout. • Failure of bracing will lead to increased deformation at roof level with potential to increase demands

Structural Component/System	Seismic Score (%NBS – IL2)	Remarks
		in portal frames that could lead to their partial collapse.
SHS portal frame for end elevation	45%	• Lack of strength and asymmetrical braces. • Lack of positive load path and strength will lead to increased deformation with risk of collapse of this elevation.
Elevated roof in central wing	>70%	• Welded connections and elements lack flexural capacity. • Lack of strength can lead to increased deformations with the risk of partial collapse of frames.
Deformations	80%	• Portal frames lack stiffness for full demands. • Excessive deformations can lead to instability.
Timber floor diaphragm	75%	• Timber diaphragm lacks connectivity to transfer loads through central portion of mezzanine in the central wing. • Lack of diaphragm action can lead to increased demands on frames and lack of positive load path in the longitudinal direction, increasing deformations in the central gravity frames that could lead to partial collapse.
Mezzanine Stair	40%	• Insufficient shear capacity of base anchor connections. • Lack of bottom positive connection can also lead to damage of the upper connection, leading to a partial collapse and loss of egress route.

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

A structural weakness (SWs) is any part or portion of the structure that does rate less than 100%NBS. Thus, all the items on Table 6 and Table 6 represent a structural weakness. Some of the small walls at the main elevations of the original building have been identified as potentially a Severe Structural Weakness (SSW), although the presence of columns in line with them will mitigate this condition. The Critical Structural Weakness (CSW), this is, the lowest scoring structural weaknesses are considered to be:

- For the Original Library Building, the connections of the roof trusses to walls and beams. This lack of strength can lead to loss of seating of the trusses and additional bending and deformation of roof concrete beams that could lead to a partial collapse of the roof structure, particularly at the central portion.
- For the Extension Building (1992), the knee connections of the steel strut back to the main walls at the central wing and connections of mezzanine beams to shear walls. Lack of strength and positive load path could lead to increased displacements for the wings with partial or total collapse of the portal frames.

Part A of the Engineering Assessment Guidelines recommends that the %NBS earthquake rating given needs to reflect the reliability/accuracy implied. For this reason, earthquake ratings should only be quoted as a whole number. It is further recommended that the whole number scores be rounded to the nearest 5%NBS (up or down). Therefore, the above results in a rating of 20%NBS (IL2) for Hastings Library, Hastings.

6.2 Analysis Summary

6.2.1 Reinforced Concrete Moment Frames – Original Library Building 1957

Our analysis show that the reinforced concrete frames lack strength and deformation capacity. Most of the columns are likely to have anchorage failure and beam-column joint failure at external joints. Wall infill contribution enhances the response and stiffness at short periods, but they have limited deformation capacity.

The weakest elevation is the northeastern, where the original reinforced concrete walls were demolished. Because of this, this elevation is more flexible, contributing to the central war memorial tower attracting more load, meaning that the roof perimeter beams need to span further out-of-plane. Its counterpart, the Southwestern elevation still benefits from the presence of infill walls and scores higher. Beam column joint failure at exterior joints might pre-empt column capacity development in a column sway mechanism. Therefore, we expect a mixed mechanism with softening due to beam-column joint failure and anchorage failure of the column top connections.

The main longitudinal elevations show better performance than the short directions, although rating is limited by the potential soft storey at the higher portions of the southeast elevation. Lack of connection strength for the existing trusses will also indicate that the top concrete beams could fail progressively in the out of plane direction if connection and load transfer to the war memorial tower through the trusses is lost.

6.2.2 Roof Trusses to the Main Roof and Steel Members - Original Library Building 1957

The roof steel truss and associated connections of the library building were rated at 20%NBS(IL2) due to the insufficient shear capacity and lack of full lateral restraint to truss chords.

The roof truss acts both in plane and out of plane to transfer lateral loads due to the lack of effective roof diaphragm and acts as a critical load path for the lateral resisting system at both transverse and longitudinal directions. Lack of strength of these connections will mean that transfer of loads to the central memorial tower will cease, increasing out of plane deformation that could lead to a loss of seating and or partial collapse of the roof structure, as noted in the previous point.

6.2.3 Roof Perimeter Beams - Original Library Building 1957

The roof perimeter beams were found to be 25%NBS(IL2) due to insufficient out-of-plane (weak axis) bending capacity between main lateral resisting systems to transfer load. This deficiency is exacerbated when rocking of the foundations at the war memorial tower/hall occur. Lack of capacity can lead to an increased softening and increased deformations, leading to out of plane partial collapse of the roof.

6.2.4 Memorial Hall Concrete Walls - Original Library Building 1957

The concrete shear walls at war memorial hall have sufficient strength but suffer global stability problem due to high slenderness ratio and resulted in rocking and foundation uplift/bearing defects.

The rocking failure mechanism has resulted in 45-65%NBS(IL2) as a structural weakness.

6.2.5 Mezzanine floor – Steel Structure – 1992

The added mezzanine floor area to the original area footprint relies partially on the connections to the concrete main structure due to an incompatibility of displacements. These connections are likely to attract undesirable loads from the lateral system of the mezzanine (portal frames and cantilever columns), resulting in a loss of integrity of the end connections. The rating of these elements is between 40-55%NBS(IL2).

6.2.6 Mezzanine floor – Steel Structure – 1957

The original mezzanine floor area relies on cantilever action of the columns and on the end reactions to the portal frames via cast-in situ bolts. These connections are the limiting factor, rating about 65%NBS(IL2).

6.2.7 Mezzanine Floor Diaphragm - Original Library Building 1957

Floor diaphragm for the mezzanine floor is governed by the tensile capacity of end connections to transfer loads in the longitudinal direction and insufficient end columns capacity to transfer demands to the foundation and perimeter roof beam.

Some damage will be likely to occur on particle board flooring due to insufficient shear capacity between bracing lines.

Due to the above-mentioned mechanism, the mezzanine floor system has a rating between 75%NBS(IL2) as a critical structural weakness.

6.2.8 Lower Roof Concrete Area - Original Library Building 1957

The lower roof concrete area at the workroom is rated as 40-45%NBS(IL2) as a critical structural weakness. It is governed by beam-column joint failure and diaphragm connection to vertical resisting elements. Short column effect is also likely to occur due to the interaction of bounding frame and wall infill.

6.2.9 Perimeter Masonry Walls - Original Library Building 1957

It shows that the external masonry infills are rated as 20%NBS(IL2). The main reason for this is that the wire ties between internal and external wythes are potentially heavily deteriorated and both wythes are likely to act independently. Plus, arching to the bounding frame is minimum due to the presence of openings. Intrusive investigation or condition assessment is recommended prior to future development to confirm the condition/location of veneer tie.

The potential failure mechanism are out-of-plane failure and toppling failure with probable collapse outside the building to the nearby pathway.

6.2.10 Extension Canopy to Workroom - Original Library Building 1957

The canopy extension to the workroom lacks tensile capacity of roof element connection to transfer load back to the main concrete frame. This item was therefore rated to be about 45%NBS(IL2).

6.2.11 External relocated canopy - Original Library Building 1957

We have rated this element to be about 45%NBS(IL2) due to the lack of strength of the connections back to the lower roof area.

6.2.12 Mezzanine Stairs - Original Library Building 1957 and Library Extension Building 1992

The internal mezzanine stairs at the original library and extension are expected to rate at approximately 40%NBS(IL2) due to the lack of sliding joint and insufficient shear capacity on base plate connection. The stairs are expected to act as unintended braces and will be damaged as building drift increases. Once the bottom connection is damaged, a mechanism will be formed for lateral loads.

6.2.13 Foundations in General

Embedded in the results above, there are some deficiencies involved in the foundations. Uplift capacity under the end of shear walls is minimal and this is limiting the rating of the building. Foundation strengthening is recommended in future strengthening works.

6.2.14 Steel Portal 310UB Frames at Wings and Their Foundations – Library Extension Building 1992

These frames lack restraints at their critical bottom flange to provide ductile behaviour. Flexibility can also be an issue if a 70%NBS(IL2) strengthening target is sought. The bolted cast-in situ bolts to the base plates also lack sufficient shear strength.

6.2.15 Wing End Block Walls – Library Extension Building 1992

The block walls at the end of the wings are rated at about 85%NBS(IL2), being limited by out of plane stability and lack of strength of the corner connection to the longitudinal struts.

6.2.16 Struts along knees – Library Extension Building 1992

The SHS struts along the side elevations ties portal frames and end block walls together, it provides bracing for the wing extension in the longitudinal direction to the central wing. The end connections of these elements to the block walls are the limiting factor, with less than 33%NBS(IL2) rating. Member capacity also sits around that level of rating.

6.2.17 Main 410UB/360UB Portal Frames at Central Wing – Library Extension Building 1992

These elements rate about 40-75%NBS(IL2), being the limiting factor the lack of restraints to the critical flanges (top and bottom) to provide dependable ductility.

6.2.18 Connections of Mezzanine Beams to Shear Walls

These cast-in situ bolted connections to the masonry concrete walls are very weak. The steel beams serve as a collector and transfer element between walls and therefore attracts heavy loading. These connections have insufficient capacity, being the weakest link in the building.

6.2.19 Roof Braces – Library Extension Building 1992

These tension-only straps lack strength with about 35%NBS(IL2). The asymmetrical layout also creates issues that need to be remediated.

6.2.20 SHS Portal Frame for End Elevation – Library Extension Building 1992

This end bay of the frames is different from others and has two braced bays with straps which are eccentric to the elements. Thus, the rating of this elevation is around 45%NBS(IL2) due to lack of strength and load transfer efficiency in its current arrangement.

6.2.21 Elevated Roof in Central Wing – Library Extension Building 1992

The elevated portion at the central wing roof rates greater than 70%NBS(IL2), with the welded connections being the limiting factor.

6.2.22 Deformations – Library Extension Building 1992

The steel portal frame wings are flexible, and deformations exceed code limits to about 80%NBS(IL2).

6.2.23 Timber Floor Diaphragm – Library Extension Building 1992

Lack of strength and connectors indicate that this floor rating is limited. Higher ratings can be expected if more information is provided around connections and layout.

6.2.24 Mezzanine Star – Library Extension Building 1992

Same as per the original building, the stair connecting the extension building has the same deficiency with insufficient strength on the base plate shear connections to allow for building movement.

6.3 Secondary Risks

In the building, the safety of the occupants and the operational continuity may be affected if heavy items are not adequately restrained. These items include suspended ceilings, services restraints, storage cabinets, equipment racks, pipework, and liquid containment tanks. This DSA does not specifically assess the performance and effects of these non-structural elements.

During our site inspection, we have observed some items which could potentially pose a risk to life safety of the occupants. These are highlighted below:

- Suspended lay-in tile ceiling system without seismic restraint to limit relative movement between ceiling and building elements.
- Lack of seismic restraint to the bookshelves.
- Non-seismic glazing frames to accommodate relative movement;
- Potential lack of seismic restraint to large HVAC units at ceiling.

6.4 Assumptions and Limitations

Below are the limitations and assumptions made during the assessment of all structures.

- All concrete frames and walls are reinforced as per drawings.
- Assume all welding is structural purpose (SP) grade and were carried out under supervision and conform to the standard and specification at the time.
- Due to the frame construction, regular building plan and low-rise nature of the structure, the Simplified Lateral Mechanism Analysis (SLaMA) has been undertaken to assess the deformation capacity of the building.
- Grillage and frame action at roof and floor level has been assumed due to the ineffective diaphragm where load has been distributed by tributary width to each steel portal frames, concrete frames and shear walls.
- Ductility of 1.0 was adopted to steel portal frames as the members do not meet the ductile detailing requirements. Therefore, the portal frame has an elastic behaviour. This was also adopted for the 1992 extension due to the limitation with connections to the concrete that are brittle in nature and may occur early in the seismic shaking.
- Ductility of 1.0 was adopted to concrete frames and shear walls due to the presence of inadequate development length and plain round reinforcing bars and lack of beam column joint confinement.
- It is assumed that distributed cracking is expected to occur on concrete shear walls.
- Assume reduced tensile capacity of anchorage reinforcing bars due to inadequate development length of plain round bars in concrete frames/walls.
- A seismic gap was identified between the library and extension/art gallery based on original drawings.

- The mezzanine steel columns are assumed to act as nominal cantilever column to resist lateral load.
- Assume pin base connection for portal frames stability checks.
- Torsion effects have been allowed with the additional consideration of accidental eccentricity.
- The concrete encasement is assumed to suppress local buckling of the encased steel columns and lateral buckling for moment.
- The assessment does not cover any non-structural components within the buildings unless stated otherwise.
- The reinforcement corrosion in workroom roof slab has minor influence on structural integrity.
- The opinions in this document are based on the conditions and information available at the time the document was published and assume that the structures were built as per the materials, reinforcement sizes, etc. for its assumed building age.

7 Conclusions

The results of the DSA indicate the building's *Earthquake Rating* to be **20%NBS (IL2)** assessed in accordance with the Guidelines, for both the original portion built in 1957 and the extension built in 1992. Therefore, these are **grade D buildings** following the New Zealand Society for Earthquake Engineering (NZSEE) grading scheme. Grade D buildings represent a risk to occupants 10-25 times greater than expected for a new building, indicating a **High-Risk safety risk exposure**.

WSP are not aware of any letter from Hastings District Council stating that they have identified these buildings as Potentially Earthquake Prone. Therefore, we have carried out the assessment with the considerations and methodologies updated in the "yellow book" of Chapter C5 of the guidelines.

A building with an *Earthquake Rating* less than 34%NBS when assessed in accordance with the Version 1 Guidelines (the 'Red book') fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-Prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67%NBS is considered as an Earthquake-risk Building by the NZSEE. **The Hastings City Library therefore potentially falls within the criteria that could categorise it as an EPB.**

The critical structural weaknesses (CSW) were found to be the lack of strength of cast-in anchors supporting the steel roof trusses of the main roof in the original portion and the lack of strength of cast-in anchors in shear walls in the 1992 extension. For the original portion, the lack of strength and movement capacity of the connections can lead to loss of seating of the trusses and additional bending of the roof beams that could lead to a partial collapse of the roof structure, particularly at the central portion. For the 1992 extension, the lack of strength in the cast in anchors can lead to loss of a positive load path with increased deformations that can lead to a collapse of the steel frames.

These results are considered draft until a final liquefaction assessment is carried out for the site.

8 Next Steps

8.1 Condition Assessment

It is recommended that a condition assessment be carried out to inform any strengthening and repair (e.g., brick veneer ties, overall carbonation depth, etc.). A condition assessment for the lower roof area is already scheduled to occur this year.

8.2 Liquefaction assessment

As discussed in the geotechnical section, a liquefaction assessment is recommended to be carried out to complete this report in a final version.

8.3 Peer review

We recommend an independent peer review of this report due to the public nature of the building and in an effort to provide best practice and value to Hastings District Council.

Appendix A

Geotechnical Report

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