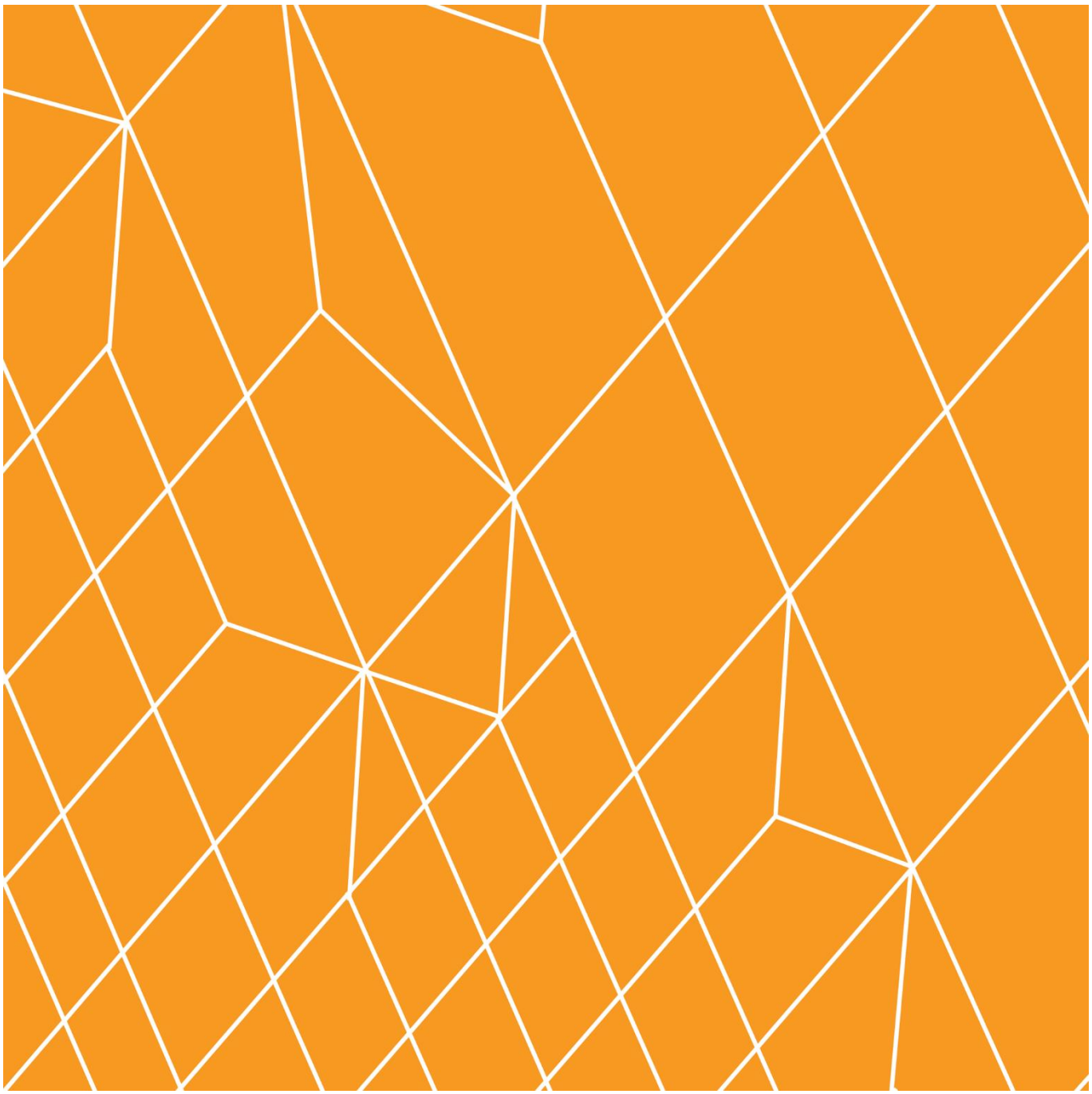

Michael Fowler Centre
Detailed Seismic Assessment

111 Wakefield Street
Te Aro, Wellington
New Zealand

Detailed Seismic Assessment Report

Holmes Australia Netherlands New Zealand USA

Revision 2
10 August 2023
137071.06

Level 2, Chartered Accountants House, 50 Customhouse Quay
PO Box 942
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Detailed Seismic Assessment Report
Michael Fowler Centre

Prepared For:
Wellington City Council

Date: 10 August 2023
Project No: 137071.06
Revision No: 2

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Report Issue Register

DATE	REV. NO.	REASON FOR ISSUE
21 December 2022	0	Draft for client review and peer review (pending close-out of internal review)
6 April 2023	1	Draft for peer review close-out
10 August 2023	2	Issue for Wellington City Council

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Executive Summary

This report presents the results of the Detailed Seismic Assessment completed for the Michael Fowler Centre, at 111 Wakefield Street, Te Aro, Wellington. We have assessed the seismic rating as 20%NBS$_{IL3}$.

As the Territorial Authority, the Wellington City Council have issued an Earthquake-prone building notice because the owner (also Wellington City Council) has not provided an engineering assessment. Therefore, the Territorial Authority is proceeding as if it had determined the building to be Earthquake-prone. The notice advises that building work is required to be carried out by 14 September 2029, to ensure the building is no longer earthquake prone. This Detailed Seismic Assessment will not serve as evidence to lift the notice.

The result of this Detailed Seismic Assessment indicates the building's seismic rating to be 20%NBS$_{IL3}$, governed by the following elements:

- the sloping transfer diaphragm below the Level 2 seating in the Auditorium (referred to as Level 1/2),
- the chord tie across the promenade at Level 2 of the Auditorium,
- the diaphragm over the double-tee flooring at Level 2 in the lobby area of the Auditorium,
- the diaphragm over the double-tee flooring at Level 4 around the promenades of the Auditorium,
- the roof diaphragm of the Auditorium, and,
- the Level 2 diaphragm in the Renouf Foyer.

These diaphragms provide a horizontal load path through the structures of the Auditorium and the Renouf Foyer and the primary vertical (gravity) load resisting systems are reliant on these load paths to maintain their integrity.

There are also localised elements with a similar seismic score (<34%NBS$_{IL3}$) identified in the following locations:

- the wall panels at the upper precast portion of the bay walls,
- the precast concrete cladding infill panels between the upper precast portion of the bay walls,
- the roof trusses of the Auditorium,
- the promenade roofs and facades around the building,
- the catwalk structure in the Auditorium, and,
- the wall reflectors in the Auditorium.

Background

Holmes NZ LP have been engaged by Wellington City Council to complete a Detailed Seismic Assessment (DSA) of the Michael Fowler Centre, at 111 Wakefield Street, Te Aro, Wellington. The DSA has been completed in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017), referred to as the "**Engineering Assessment Guidelines**" within this report.

The proposed revision of section C5 has been used for the assessment of reinforced concrete elements of the primary structure; this section of the Engineering Assessment Guidelines is referred to as the "**Yellow Chapter C5**" and was published in November 2018. This revision encompasses the learnings following the 2016 Kaikoura Earthquake event and follows the latest industry practices in assessing reinforced concrete structures. A rating produced using the Yellow Chapter C5 cannot be used for earthquake-prone rulings. In this case, since the building is already considered earthquake-prone, we have elected to use the latest technical knowledge to inform the rating of the building for purposes of understanding next steps and a potential strengthening design. An assessment in accordance with the requirements of the Red Chapter C5 has therefore not been undertaken. However, it is expected that it would not materially change the conclusions of the DSA.

In accordance with the Building Act 2004 (as amended by the Earthquake-prone Buildings Amendment Act 2016), the Ultimate Limit State (ULS) seismic demand is derived from the New Zealand Structural Design Actions Standard, NZS1170.5:2004 (Standards New Zealand, 2016).

This report builds on previous assessment and improvement works undertaken on the Michael Fowler Centre since 2014. Major changes and updates that form part of this report have resulted from new industry knowledge and assessment updates in the following areas:

- precast concrete flooring,
- diaphragms, including the treatment of cold-drawn (non-ductile) mesh reinforcing,
- foundation and soil-structure interaction, and,
- the inclusion of secondary and non-structural elements.

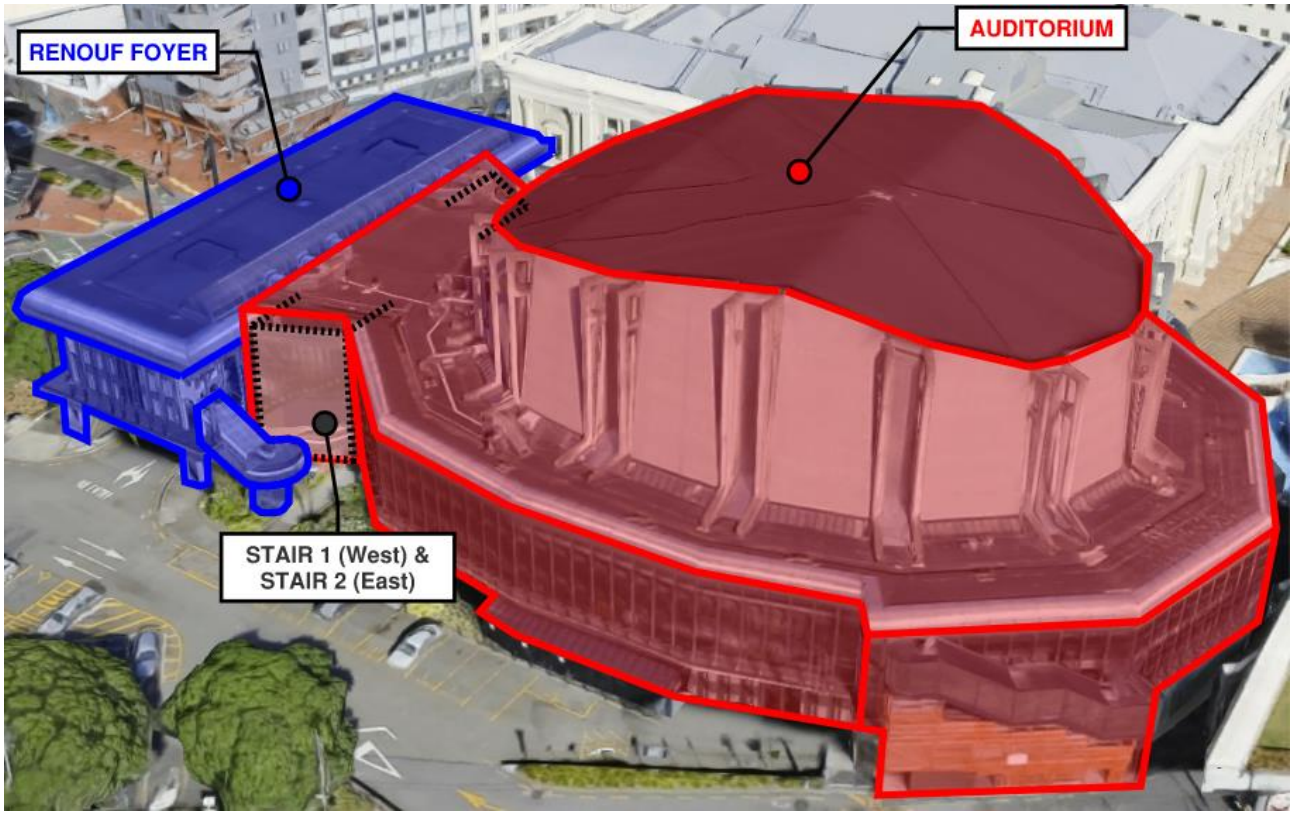

Figure ES-1: Illustration highlighting the seismically separated structures at the Michael Fowler Centre

Building Description

The Michael Fowler Centre is a performing arts centre in central Wellington. The building comprises four seismically separated structures as shown in Figure ES-1. The predominant structures at the site are the Auditorium and the Renouf Foyer. However, there are also two independent staircase structures that are connected to the main structures through the non-structural cladding and façade system but have an independent structural system.

Auditorium

The Auditorium is an elliptical shaped structure that is predominantly used as a venue for concerts, conferences, and other large scale public events. In plan, there are four main areas referenced around the structure. Refer to Figure ES-2, which designates the Inner Auditorium, Backstage area, Promenade, and Entry Lobby area on Level 4. These areas are generally consistent up the height of the structure; however, they occur in different combinations. Figure ES-3 presents longitudinal (North-South) and transverse (East-West) sections of the Auditorium and highlights the floor plate regions.

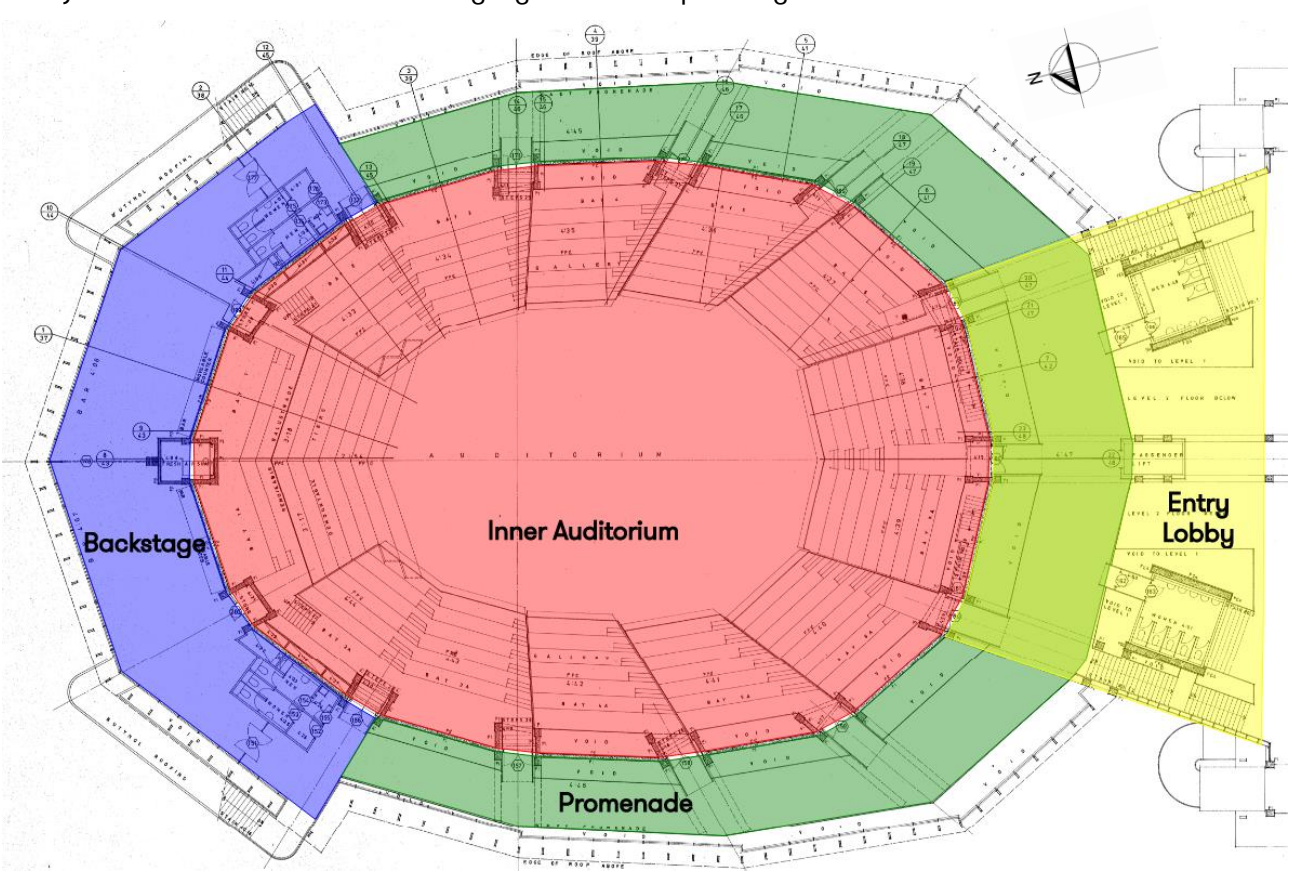

Figure ES-2: Regions of the Auditorium floor plate

The floors are discussed in further detail in Section 5.1.1.4, but to summarise the key floor plate locations:

- the Backstage area occurs Levels 1 to 4,
- the Inner Auditorium and Promenades have structure on Levels 1, 2, and 4,
- the Entry Lobby occurs on Levels 1 and 2,
- a localized sound booth occurs on the south end of Level 5 Inner Auditorium, and,
- a sloped diaphragm occurs in the southern Inner Auditorium from Levels 2 and ends at Level 1/2.

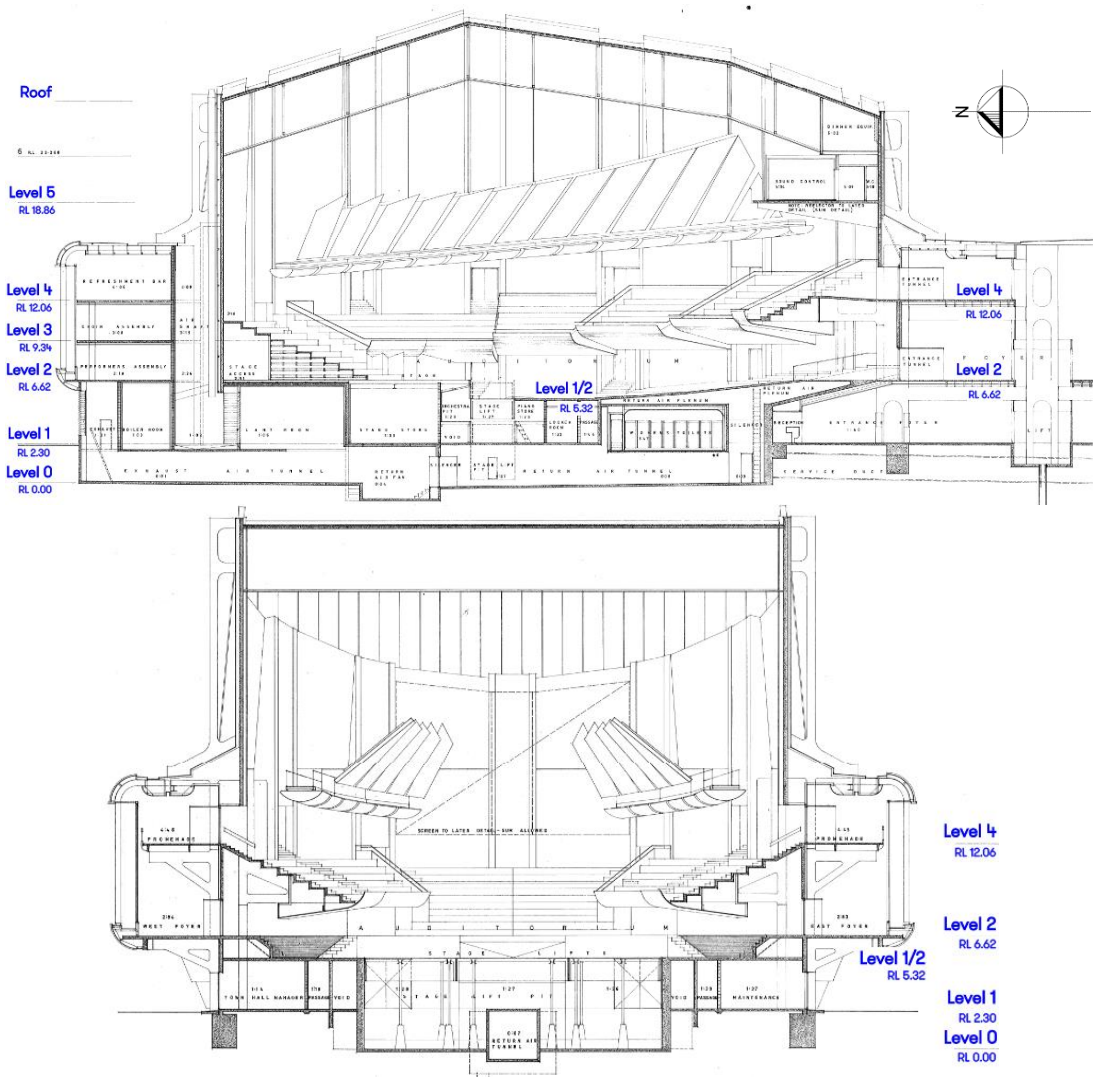

Figure ES-3: Longitudinal (top) and Transverse (bottom) architectural sections of Auditorium

The Inner Auditorium features sloped seating from Level 2 and Level 4 looking towards the stage at Level 2. The cantilevered seating accessed from Level 4 is tiered and changes in elevation around the ellipse; the seating at the south is higher than that towards the north. Radial seismic gaps occur around the floor plate at Level 4, allowing the individual cantilever walls to move independently at this level.

Hanging above the Inner Auditorium are timber reflector panels with two catwalk areas behind them for stage technicians. There is also a hanging 'Donut' shaped structure above the stage used for acoustics and lighting. These features are shown in Figure ES-4.

Around the outer ring of the Auditorium, the Promenade floors provide access to the inner seating areas. At the perimeter of the Promenade is a large, glass curtain wall spanning from above Level 4 and landing at Level 2. It is supported by stainless-steel mullions and glulam beams in the roof. The façade wraps around all the buildings at the site, continuing around the independent staircase structures and the Renouf Foyer to the south. At the northern end of the outer ring, the façade around Promenades continues around the Backstage area.

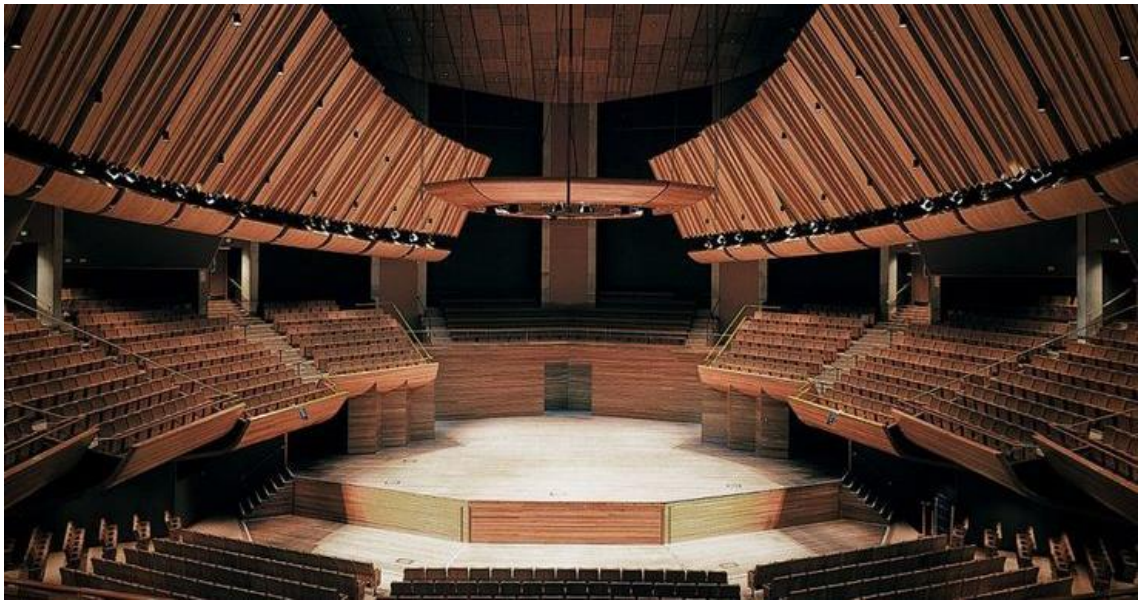

Figure ES-4: Photograph of the Inner Auditorium

The ground floor area at Level 1 provides rooms for offices, restrooms, and plant area underneath the stage and sloped Level 1/2 seating.

A series of roof trusses support a roof comprising precast concrete hollowcore (Dycore) units with a concrete topping. Between the bottom chords of the roof trusses spans a ceiling built-up of acoustic timber panels.

The primary lateral load resisting system comprises cantilevering C-shaped reinforced concrete walls around the perimeter of Inner Auditorium; these are referred to as the Bay Walls. The Auditorium is split into 14 radial Bays each comprising a single Bay Wall which have with two supporting buttress walls. The buttress walls are not perpendicular to the web area of Bay Walls but are oriented to a central origin point. The Bays are numbered 1-7 along the eastern side of the ellipse and are mirrored with the suffix (a) for the western Bays. All discussion of the eastern bay structure is structurally consistent to the western (a)-suffix bays. A deep ground beam forms a mostly continuous ring supporting the primary Bay Walls over their piled foundation.

The Bay Walls are continuous up the height of the building for most Bays. However, four Bays at the lobby (6, 7, 7a, 6a) stop at Level 2 and transition to precast column pairs or an isolated buttress for the ground floor (Level 1). The column pairs and isolated buttress resist the overturning of the walls above while the sloping diaphragm to Level 1/2 transfers the shear to the inner squat wall, Wall 21.

In addition to the localised cantilever wall discontinuity at Level 1, significant lateral resistance is added in the Inner Auditorium by the squat walls at the ground floor. The walls are a mix of reinforced concrete and concrete masonry units, with the most structurally significant inner walls labelled in Figure ES-5. The inner wall system is tied to the cantilever walls through concrete diaphragms at Level 1/2 and 2, as well as on the ground floor level. The single storey walls are supported by a series of slender ground beams, shallow strip footings, and isolated pile foundations.

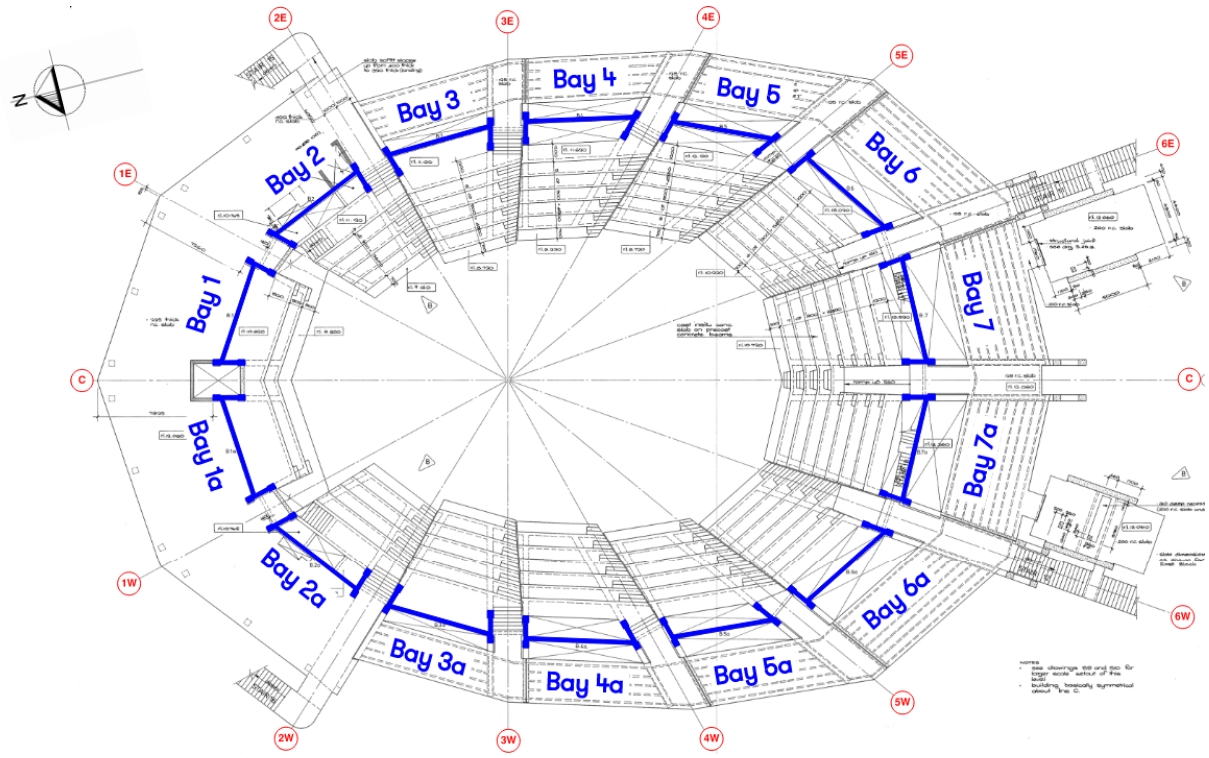

Figure ES-5: Primary lateral load resisting system at Level 4 (drawing S7 of original 1980's structural set)

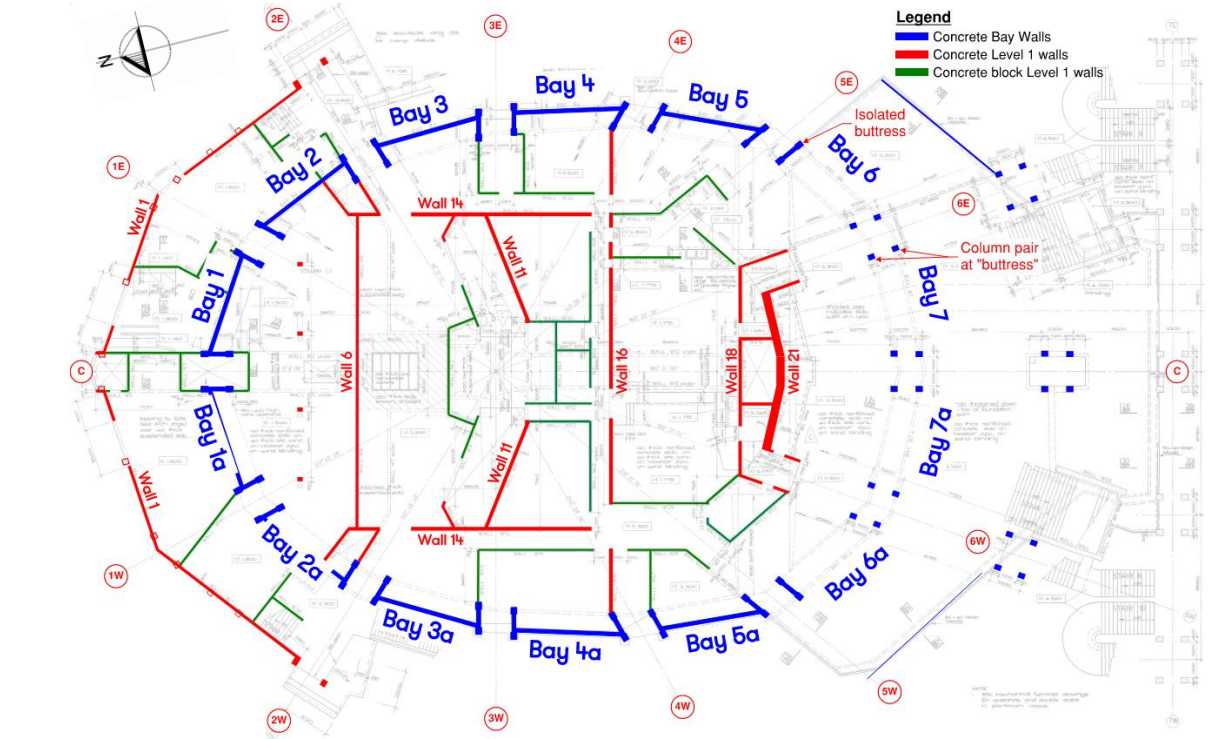

Figure ES-6: Primary lateral load resisting system at Level 1 (drawing S3 of original 1980's structural set)

Renouf Foyer

The Renouf Foyer is a two-storey, rectangular structure located on the southern end of the site. It comprises concrete frames that connect to a floor at Level 2 which is a double-height space up to the roof structure. A plan view of the Renouf Foyer is shown in Figure ES-8.

The ground floor provides an external pick-up and drop-off facility while the upper floor primarily consists of an open plan, double-height space. The upper level of the Renouf Foyer is clad with a large curtain wall consisting of glulam mullions which are curved at roof. The Renouf Foyer is seismically separated from the Auditorium with functional linkages via a suspended floor area above ground level. A series of stair structures either span through the Renouf Foyer or are externally supported by the Independent Staircase Structures.

The Renouf Foyer saw an extension in 1989, adding an additional suspended floor area, referred to as the 'kitchen extension'. An additional frame bay was added to the south of four central transverse frames in the Renouf Foyer. These frames span over the basement substructure.

Figure ES-7: Renouf Foyer at Level 2 (looking east)

The primary lateral load-resisting system of the Renouf Foyer consists of a two-way concrete moment frame system. The precast coupled column units form single-bay moment frames in the transverse (north-south) direction. In the longitudinal (east-west) direction, adjacent sets of coupled columns are connected via concrete coupling beams to provide frame action within each precast coupled column group. Each group of four columns is not connected to the adjacent precast column group.

The kitchen extension frames are precast concrete beams extending from the central frames of the Renouf Foyer in the transverse direction. In the longitudinal direction no additional vertical lateral framing was added. The precast floor units act as a laterally cantilevering diaphragm.

Independent Staircase Structures

The two independent staircase structures are located between the Auditorium and the Renouf Foyer. The staircase structures contain suspended floors at Levels 3 and Level 4 which are used for toilet facilities. The staircase structures also support a series of stairs (three flights each) on the external faces, which span between the Renouf Foyer and the Auditorium, between the ground floor level and the floors at Level 2, Level 3 and Level 4. The staircase structures are constructed on independent foundations from the Auditorium, though the frame extensions of the Bay Walls of Bays 6(a) and 7(a) also land on these foundations. Otherwise, they are seismically separated from the Auditorium and the Renouf Foyer. Therefore, they are expected to behave independently during seismic shaking.

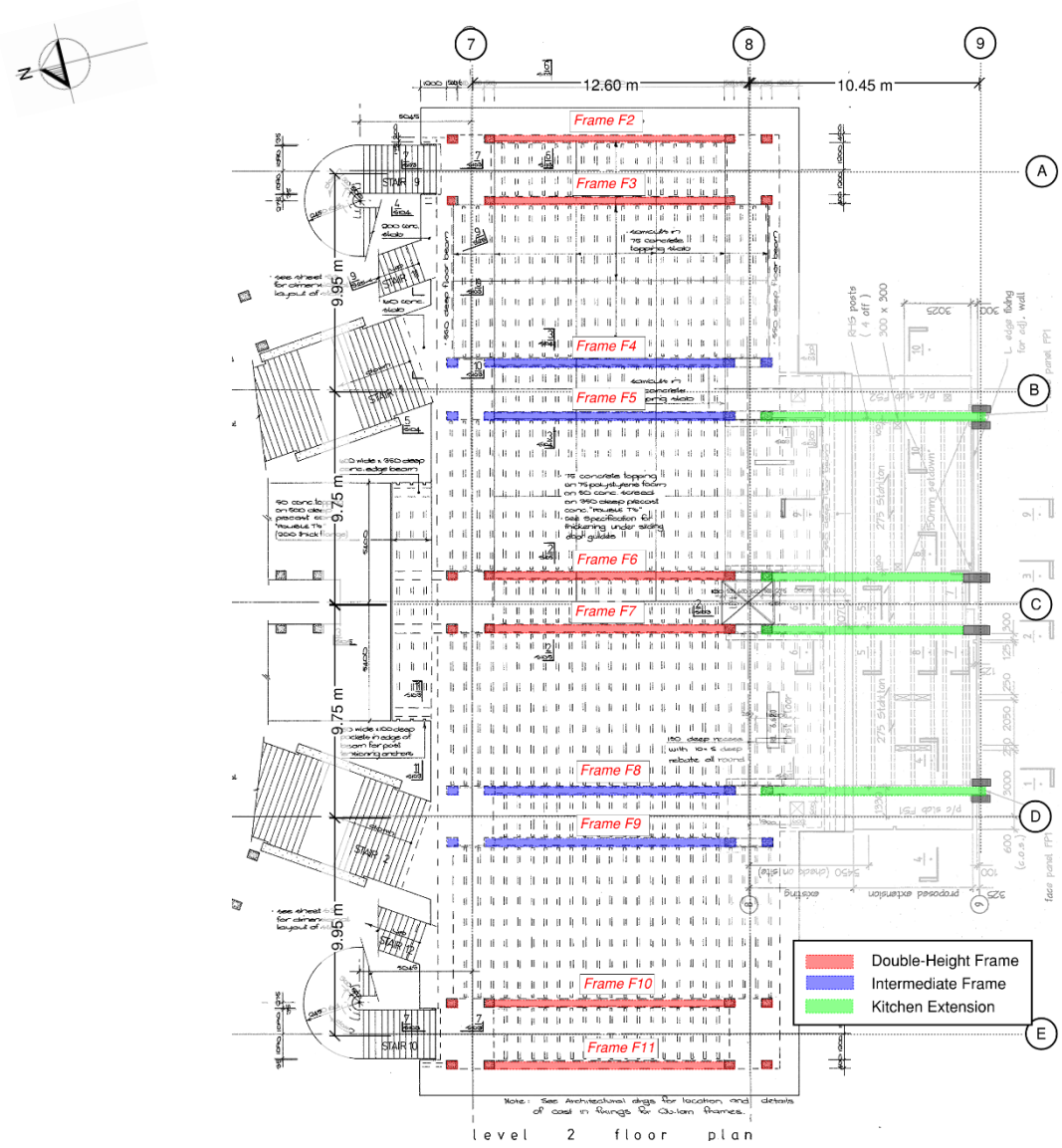

Figure ES-8: Level 2 plan highlighting the transverse frames layout within the Renouf Foyer (drawing S102 of original 1980's structural set supplemented with the drawings of the 1989 kitchen extension)

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Assessed Seismic Rating

The Assessed Seismic Rating is 20%NBS_{IL3}

The results of the DSA indicate the building's seismic rating to be 20%NBS_{IL3} assessed in accordance with the Engineering Assessment Guidelines. The seismic rating assumes that 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 (Standards New Zealand, 2011), due to its use as a public assembly space.

Relative earthquake risk compared with a new building

This rating equated to a Grade D building following the NZSEE grading scheme. Grade D buildings represent a risk to occupants 10-20 times that expected for a new building, indicating a high-risk exposure relative to a new building if a large earthquake occurs.

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 Michael Fowler Centre is therefore categorised as an Earthquake Risk Building and also it meets one of the criteria that could categorise it as an Earthquake-prone Building.

Actions required by the building owner in accordance with the Building Act

Following identification as a potentially Earthquake-prone building (EPB), building owners are required to provide the Territorial Authority with an engineering assessment of the building within twelve months (unless an extension is granted). When the Territorial Authority receives the engineering assessment they determine if a building is Earthquake-prone. If the Territorial Authority determine a building is Earthquake-prone, an Earthquake-prone building notice is issued which must be displayed in a prominent place on or adjacent to the building. The Territorial Authority may also restrict access to the building.

Engineering assessments can be sent to the Territorial Authority at any time for consideration in determining the Earthquake-prone building status.

The Territorial Authority has issued an earthquake-prone building notice on 14 March 2022, stating that seismic strengthening work, to ensure the building is no longer earthquake prone, must be completed before 14 September 2029. This Detailed Seismic Assessment is used to inform Wellington City Council of which elements of the building have shown a seismic score below <34%NBS_{IL3}.

Identified Structural Weaknesses - Auditorium

Structural weaknesses are those elements included in our assessment (the failure of which would constitute a significant life safety hazard per the Engineering Assessment Guidelines) which scored <100% NBS. The Critical Structural Weakness is the lowest scoring of these elements and governs the rating for the building.

The following structural weaknesses have been identified in the primary structure of the Auditorium.

Table ES-1: Structural Weaknesses identified in the assessment of the primary structure of the Auditorium

Building Element	Structural Weakness	%NBS$_{IL3}$
Transfer diaphragm at Level 1/2	Exceedance of tie capacities in the diaphragm (provided by deformed bars in the diaphragm) and ties to the flat diaphragm	20
Level 2 promenade diaphragm	Exceedance of chord tie capacities in the diaphragm (provided by deformed bars in the diaphragm).	20
Double-tee flooring at Level 2 in the lobby area	▪ Exceedance of friction capacity due to the lack of tie capacity in the diaphragm (665-gauge non-ductile mesh). ▪ Loss of seating due to excessive differential displacements and pounding between the Auditorium and the Renouf Foyer	25
Roof diaphragm	Lack of tie capacity in the diaphragm (661-gauge non-ductile mesh) leading to subsequent structural weaknesses in other elements, such as: ▪ Excessive axial loads in the top chords of the roof trusses ▪ Excessive loads in the connection between the roof trusses and the precast portion of the perimeter buttress wall system ▪ Excessive size of cracks in the concrete diaphragm across the supports of the DyCore roof elements ▪ Redistribution of seismic lateral loading to the perimeter buttress walls leading to excessive out-of-plane loading	20 – 30
Level 4 wedge-shaped diaphragm of promenade floor and stepped seating	▪ Exceedance of friction capacity due to the lack of tie capacity in the diaphragm (665-gauge non-ductile mesh). ▪ Loss of seating due to excessive differential displacements between the Auditorium and Independent Staircase Structures	25
Bay walls (upper precast wall panels)	Exceedance of shear capacity at the upper precast portion of the bay walls, at the horizontal interface between different precast wall panels	25
Bay walls (upper precast wall panels)	Exceedance of the out-of-plane flexural capacity of the precast concrete wall panels	35
Gravity columns at the extended frames of Bays 6(a) and Bays 7(a)	Lack of compatibility with the displacement demands from differential lateral spread between foundations of the Auditorium and the independent staircase structures ***(does not require a reduction for the consideration of a geotechnical step change)***	45

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Building Element	Structural Weakness	%NBS$_{IL3}$
Bay walls (connection of upper precast wall panels to the roof)	Exceedance of shear capacity at the interface between the precast wall panels and the roof	45
Foundation system	Exceedance of inelastic rotation capacity of the piles during the lateral spread scenario ***(does not require a reduction for the consideration of a geotechnical step change)***	45
Gravity columns in the backstage area	Exceedance of rotation limits for slab-column joints.	45
Bay walls (cast insitu portion above floor level 2)	Exceedance of inelastic rotation capacity, limited by bar buckling in the web area at Level 2 of Bays 7(a).	50
Bay walls (isolated columns in lobby area at Bays 6(a) and 7(a))	Exceedance of inelastic rotation capacity, which is significantly reduced due to the large axial load ratio. Because of the discontinuation of the Bay Wall system in the lobby area the flexural demands in the Bay Walls result in significant axial loads on the buttress columns at the ground floor level.	55
Level 2 Precast Cantilever Beams adjacent to double T slabs	Combined actions on beam reinforcing: gravity, lateral ties, vertical acceleration	55-70
Squat walls at Level 1	Exceedance of shear capacity in the walls. (the lowest score is assessed for Wall 18, other squat walls are achieving higher scores, see Section C4.1.2).	55-75
Level 4 Precast Cantilever Beams	Combined actions on beam reinforcing: gravity, lateral ties, vertical acceleration	75
Floor diaphragms at backstage area (Level 2, Level 3 and Level 4)	Exceedance of tie capacities in the diaphragm (provided by deformed bars in the diaphragm).	75-80
Floor diaphragm at director's area (Level 5)	Exceedance of tie capacities in the diaphragm (provided by deformed bars in the diaphragm).	85

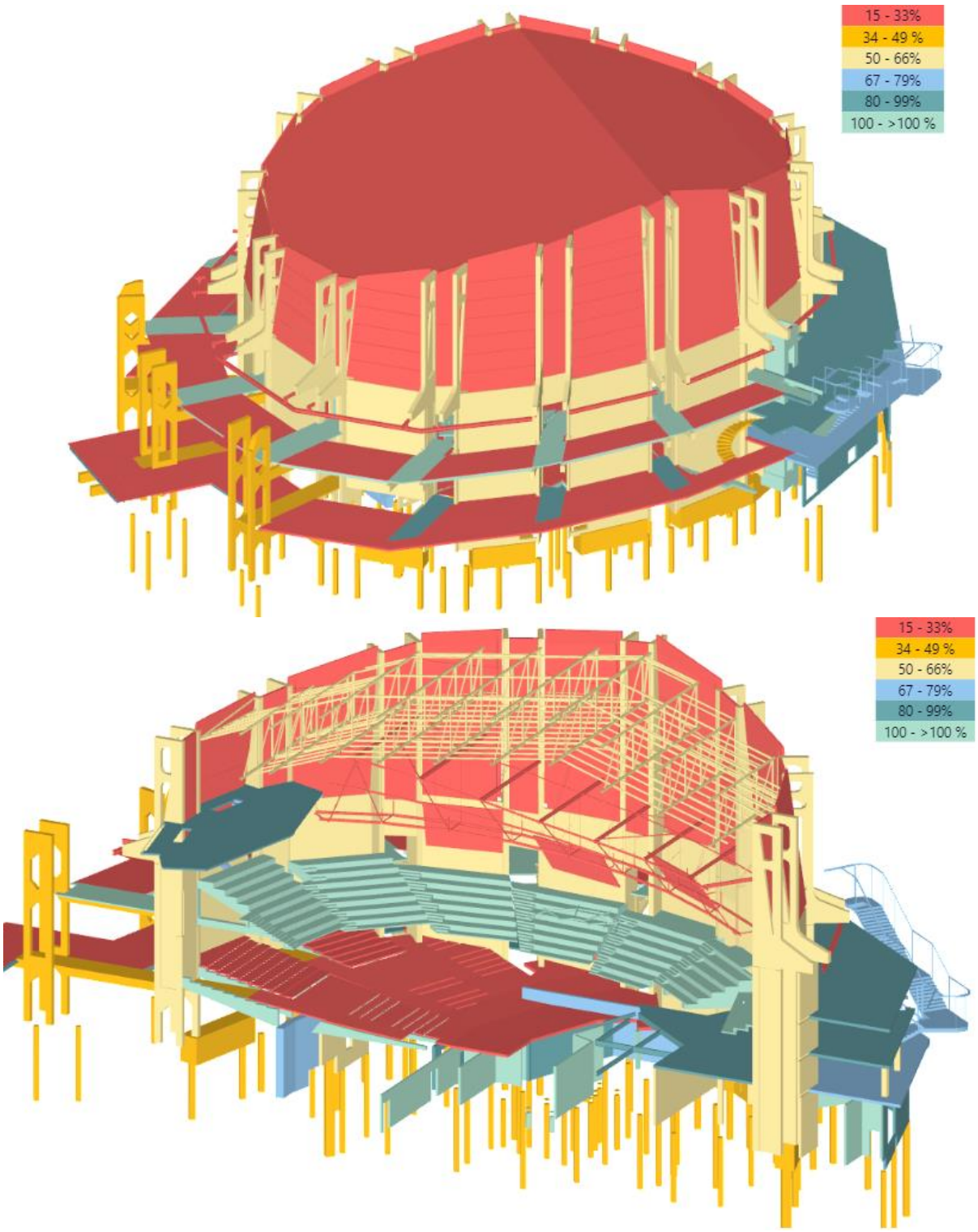

Figure ES-9: Visualisation of seismic scores of the Auditorium

This visual representation includes the stairs, however, excludes most of the other secondary and non-structural elements. It also excludes the Renouf Foyer and the two Independent Staircase Structures.

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Identified Structural Weaknesses – Renouf Foyer

The following structural weaknesses have been identified in the primary structure of the Renouf Foyer.

Table ES-2: Structural Weaknesses identified in the assessment of the primary structure of the Renouf Foyer

Building Element	Structural Weakness	%NBS$_{IL3}$
Level 2 diaphragm	Exceedance of tie capacities in the diaphragm (665-gauge non-ductile mesh)	25
Kitchen extension stitch joint	Insufficient displacement capacity of connection between original Renouf Foyer frames and the 1989 kitchen extension	55
Roof trusses	Both imposed displacements and inertial loads limit the capacity due to: ▪ Deformation/drift compatibility of transverse frames with Renouf Foyer columns ▪ Connections of transverse trusses to coupled column units under longitudinal seismic loading.	55
Coupled column units	Exceedance of inelastic rotation capacity of columns at the base, as a soft-storey forms at the ground floor level.	60
Foundation system	Flexure-shear of piles in undegraded ground conditions	70
Kitchen extension frames	Exceedance of inelastic rotation capacity of hanger beam-column connection when displacing southwards.	80

Identified Structural Weaknesses – Independent Staircase Structures

The following structural weaknesses have been identified in the primary structure of the Independent Staircase Structures.

Table ES-3: Structural Weaknesses identified in the assessment of the primary structure of the Independent Staircase Structures

Building Element	Structural Weakness	%NBS$_{IL3}$
Walls in out-of-plane direction	Exceedance of joint shear capacity as a beam-sway mechanism degrades the joint shear capacity	40
Foundations in in-plane direction of the walls	Exceedance of tension capacity of piles, resulting in the exceedance of 2.5% drift limit	70

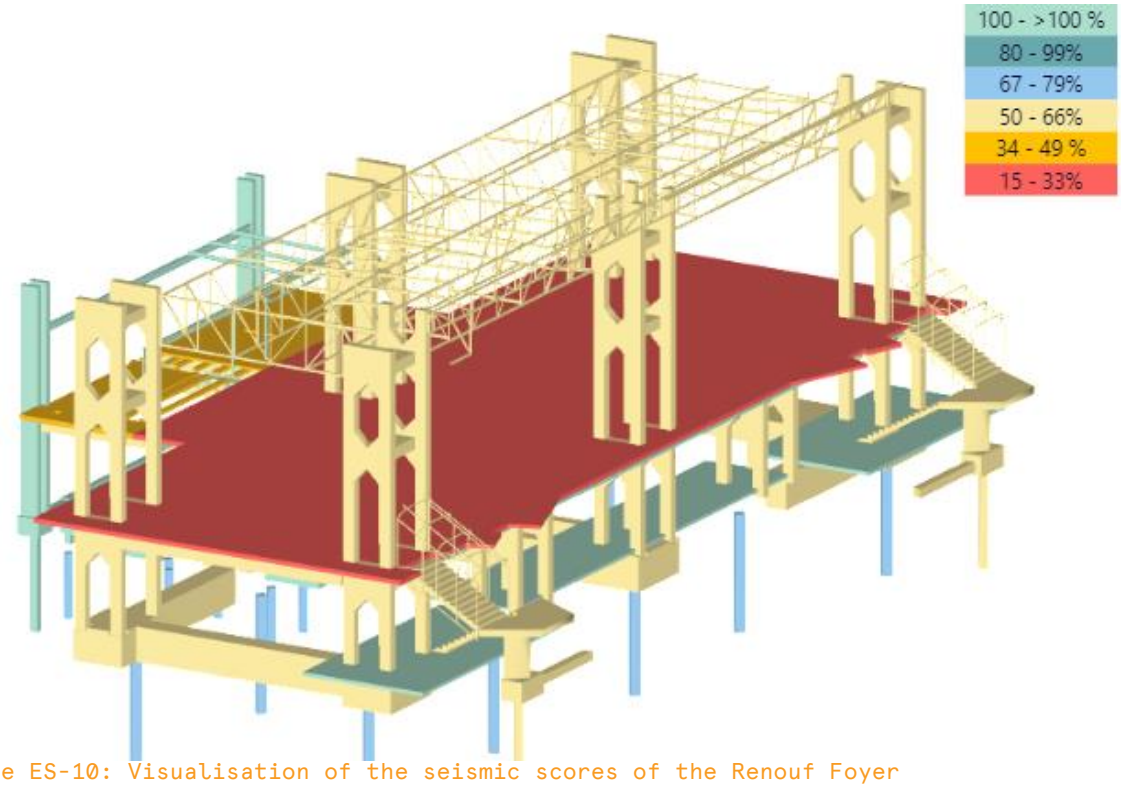

Figure ES-10: Visualisation of the seismic scores of the Renouf Foyer

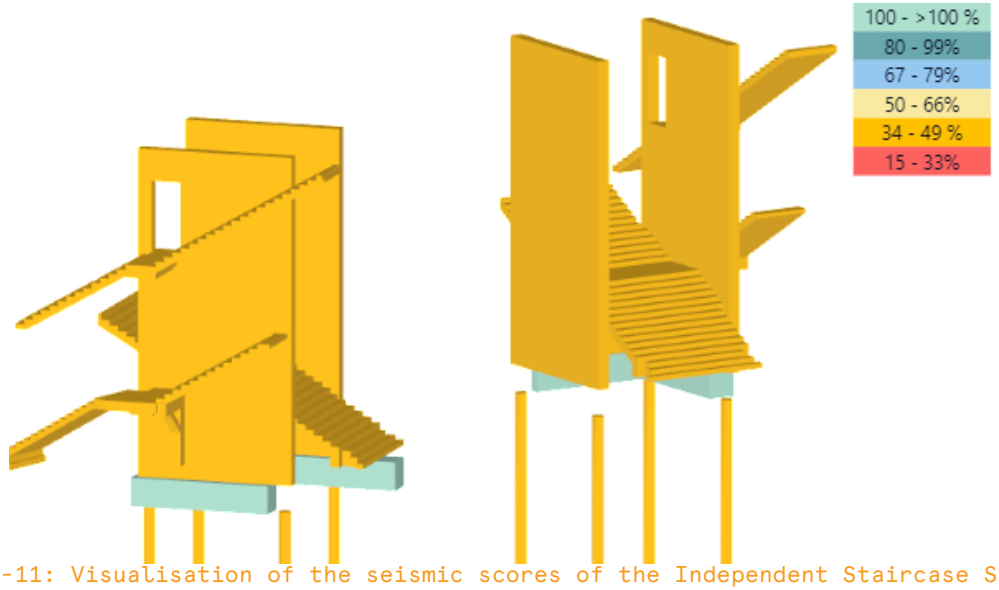

Figure ES-11: Visualisation of the seismic scores of the Independent Staircase Structures

This visual representation includes the stairs, however, excludes most of the other secondary and non-structural elements.

Identified Structural Weaknesses – Secondary and Non-Structural Elements

The following secondary structural and non-structural (SSNS) aspects were considered in the assessment of the seismic rating:

Table ES-4: SSNS included or not included in the Assessed Seismic Rating

Building Part	Included?	Reason for decision to include/exclude
Precast infill cladding panel at façade of Auditorium	Yes	Heavy element above Space Class I
Reinforced concrete masonry block walls	Yes	>25kg/m^2 partition walls bordering Space Class I, II and III
Light-weight façade and glazing system around promenade	Yes	Support frames that could fall into Space Class I
Lightweight arching roof above Renouf Foyer	No	Light-weight cladding elements
Catwalks	Yes	Considered a heavy (>25kg) horizontal cantilevering element above Space Class I
Wall reflectors	Yes	Considered a heavy (>25kg) 'sign or billboard' with a fall height greater than 3m onto Space Class I
Stage reflector ('Donut')	Yes	Considered a heavy (>25kg) vertical cantilevering element above Space Class I
Rigging points in the Auditorium and Renouf Foyer	Yes	Heavy plant that could lead to the element falling onto the building occupants
Stairs	Yes	Above Space Class I Providing emergency egress paths
Suspended portion of the light-weight timber stage	No	Above Space Class V
Suspended light-weight timber ceiling underneath Level 4 cantilever floor segments	No	Lightweight ceiling systems above Space Class III
Movable stage lift	No	Above Space Class V
Ceiling panels in Auditorium	Yes	Potentially unrestrained ceiling tile (>7.5kg) above Space Class I
Operable walls in Renouf Foyer	Yes	Considered a heavy (>25kg) element above Space Class I

The scope excludes the seismic (and condition) assessment of Mechanical Plant and other services on the site. This is understood to be undertaken by a separate consultant. This covers the equipment for mechanical ventilation, the hydraulic plant for the stage and the rigging, and any electrical plant in the building. The conclusions of this assessment are not captured in this report.

The following structural weaknesses have been identified in the secondary structural and non-structural elements.

Table ES-5: Structural Weaknesses identified in the assessment of the secondary and non-structural elements

Building Element	Structural Weakness	%NBS$_{IL3}$
Catwalks structures	Exceedance of capacity of in-plan bracing.	20-30
Wall reflectors	Concrete break-out from excessive forces on the post-installed anchors	20
Promenade roof & façade system	Exceedance of deformation capacity of existing glazing suite	25
Precast infill panels between the upper precast portion of the bay walls	Limited displacement compatibility with the primary structure of the Auditorium	25
Stairs 15/16	Loss of integrity of the landings at large inter-storey drifts.	35
Stairs 11/12	▪ Exceedance of flexural capacity from in-plane loading. ▪ Lack of displacement compatibility of lower and upper stair flight sliding connections. ▪ Differential ground movement from lateral spreading of the ground exceeding the seating capacity.	40-45
Stairs 7/8	Differential ground movement from lateral spreading of the ground exceeding the seating capacity	45
Stairs 13/14	Integral cast insitu connection leading to excessive flexure of stair flight under imposed displacements.	45-60
Concrete masonry block walls	Exceedance of out-of-plane capacity of block walls at Level 4	50
Stairs 9/10	Exceedance of both displacement capacity and structural capacity at comparable seismic demand due to: ▪ Lack of displacement compatibility of level 2 sliding connection. ▪ Exceedance of flexural capacity of lower stair flight due to significant mass eccentricity under inertial loads.	50
Rigging points in the Renouf Foyer	Exceedance of flexural capacity under 'Parts and Components' loading	70
Yellow Chapter C5 checks for precast flooring	Loss of seating due to excessive drifts of the primary structure (limited at the double-tee units in the lobby area at Level 2, all other locations have a score of >100%NBS$_{IL3}$)	80

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Recommended Next Steps

The detailed seismic assessment (DSA) of the Michael Fowler Centre has identified a number of structural weaknesses that rate the building as potentially earthquake prone. Considering the complexity of the structural system and the public significance of this assessment, we recommend that the outcome of this DSA should be subject to a peer review.

We also recommend that the findings of this DSA are considered in a concept strengthening scheme that addresses the structural weaknesses to achieve a rating of at least >34%NBS_{IL3}.

Basis for the Assessment

The Detailed Seismic Assessment (DSA) has been completed in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017). This document is referred to in this report as the **Engineering Assessment Guidelines**.

The performance levels of the reinforced concrete elements have been assessed in accordance with the revised version of Chapter C5 (Concrete Buildings, November 2018) of the Engineering Assessment Guidelines. This document is referred to as the **Yellow Chapter C5**.

The assessment has been based on the information listed in Section 6.1 of this report. It includes the original design drawings, specifications and structural calculations available from the original structural design and architect.

The as-constructed structure appears to be generally consistent with the available drawings. No significant modifications appear to have been made to the primary lateral or gravity systems, aside from the kitchen extension in the Renouf Foyer and subsequent earthquake securing works undertaken by Holmes in 2016-2020.

The concrete that was observed showed no significant cracking, spalling, corrosion, or decay that would affect the results of this assessment.

Engineering Assessment Summary Report

An Assessment Summary Report has been prepared in accordance with the Engineering Assessment Guidelines, and to fulfil the requirements of Section 2.5 of the Earthquake-prone Building Methodology. This can be found in Appendix A

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1 INTRODUCTION AND BACKGROUND

Holmes NZ LP have been engaged by Wellington City Council to complete a Detailed Seismic Assessment (DSA) of the Michael Fowler Centre, at 111 Wakefield Street, Te Aro, Wellington.

This seismic assessment is undertaken to provide to Wellington City Council the seismic rating of the buildings at the site in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017). This document is referred to as the "**Engineering Assessment Guidelines**" within this report. The assessment includes the provisions of the revised Section C5 for the assessment of reinforced concrete structures, commonly referred to as the **"Yellow Chapter C5"**. This revision encompasses the learnings following the 2016 Kaikoura Earthquake event and follows the latest industry practices in assessing reinforced concrete structures.

A rating produced using the Yellow Chapter C5 cannot be used for earthquake-prone rulings. In this case, since the building is already considered earthquake-prone, we have elected to use the latest technical knowledge to inform the rating of the building for purposes of understanding next steps and a potential strengthening design. An assessment in accordance with the requirements of the Red Chapter C5 has therefore not been undertaken. However, it is expected that it would not materially change the conclusions of the DSA.

The Michael Fowler Centre comprises of several seismically separated structures, including the Auditorium, the Renouf Foyer and two independent staircase structures. These structures have all been constructed in the early 1980's, and the Renouf Foyer had a kitchen extension in 1989.

The structures were previously subject to a detailed seismic assessment in 2014 to the "NZSEE Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, June 2006, including Corrigenda 1, 2 and 3" by Holmes Consulting Group Limited. The structures were categorised as Importance Level 3 (IL3). This assessment concluded that the structures typically had seismic ratings of 50-60%NBS_{IL3}. However, an assessment to the 2006 version of these guidelines primarily focussed on the primary lateral load resisting system. Therefore, the 2014 assessment did not include detailed considerations of the capacity of diaphragms, foundations, secondary structural elements, etc. Nonetheless, the 2014 assessment identified potential structural weaknesses with regards to the staircases, and the hollowcore (Dycore) roof structure of the Auditorium. In addition, it was noted that the building was potentially prone to the effects of liquefying ground layers under the building, for example through lateral spread of the ground.

Holmes has since been involved with the seismic strengthening of a number of the structural weaknesses identified in the 2014 assessment. Specifically, this includes modifications to the staircase structures (2016), the provision of seating brackets to secure the precast concrete Dycore units in the roof of the Auditorium and the precast concrete double-tee flooring in the Renouf Foyer and Auditorium promenades (2019). Furthermore, Holmes have provided specific studies of the roof diaphragm of the Auditorium (2019, 2022), the foundation system and its susceptibility to the effects of liquefying ground layers (2020, in collaboration with Tonkin+Taylor Ltd) and minor structural engineering services such as for the refurbishment of the external staircase systems of the Auditorium (2022).

A Detailed Seismic Assessment quantifies the risk posed to people by existing buildings from earthquakes

The objective of a DSA is to inform users about the risk posed to people by existing buildings under earthquake actions. A DSA specifically considers life safety, egress, and protection of adjacent property by assessing strength and deformation capacity. A DSA provides a more comprehensive assessment of the likely seismic performance of an existing building than an Initial Seismic Assessment (ISA).

A DSA quantifies the seismic behaviour of the building as a seismic rating. This is expressed as a percentage of the standard achieved from application of the building code requirements, or percent New Building Standard (%NBS). The rating provides a measure of the expected performance from a life safety point of view, compared with the minimum required by the Building Code for new buildings.

A DSA is also used to determine whether a building is Earthquake-prone in accordance with the definition for an Earthquake-prone building (EPB) in the New Zealand Building Act.

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2 SCOPE

Our Assessment has been completed in accordance with the Engineering Assessment Guidelines

The DSA for the Michael Fowler Centre has been completed in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017), referred to as the "**Engineering Assessment Guidelines**" within this report. The Engineering Assessment Guidelines are referenced in the Earthquake-prone Building methodology, for the identification of Earthquake-prone buildings, as set by the Chief Executive of the Ministry of Business Innovation and Employment under section 133AV of the Building Act 2004.

The proposed revision of section C5 has been used for the assessment of reinforced concrete elements of the primary structure, this section of the Engineering Assessment Guidelines is referred to as the "**Yellow Chapter C5**". This revision encompasses the learnings following the 2016 Kaikoura Earthquake event and follows the latest industry practices in assessing reinforced concrete structures.

The assessment has been undertaken to inform strengthening design and an assessment in accordance with the requirements of the Red Chapter C5 has therefore not been undertaken. However, it is expected that it would not materially change the conclusions of the DSA.

Specific guidance on damage limitation and the ability to occupy after an earthquake is not included in a DSA conducted to the minimum requirements of the guidelines, unless specifically appended to the scope of a normal DSA.

Our scope of work for the Detailed Seismic Assessment included the following:

- Review of available structural drawings and other historic information for the structures to determine the nature of the designs, primary structural characteristics, and adequacy of the lateral force-resisting systems.
- Visual inspection and review of the building to get familiar with the building, visual assessment of its condition, observation of important structural and seismic characteristics, noting obvious deficiencies or differences from the documentation.
- Assessment of the likely seismic performance of the buildings by completing a DSA in accordance with the Engineering Assessment Guidelines.
- Report of our findings and recommendations, outlining the Assessed Seismic Rating, commentary on seismic performance and risks, and describing analysis methodology and modelling parameters.

The following exclusions apply to our scope of work:

- The seismic (and condition) assessment of Mechanical Plant on the site is understood to be undertaken by a separate consultant. This covers the equipment for mechanical ventilation, the hydraulic plant for the stage and the rigging, and any electrical plant in the building. The conclusions of this assessment are not captured in this report.

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3 TERMINOLOGY AND KEY DEFINITIONS

Terminology used to communicate seismic risk and behaviour

The communication of seismic risk and assessed seismic behaviour of the building use the terminology defined in the Engineering Assessment Guidelines. Key terms and acronyms used in this report include:

- **Earthquake rating** or **Seismic rating** – The rating given to a building as a whole to indicate the seismic standard achieved in regard to human life safety compared with the minimum seismic standard required of a similar new building on the same site. Expressed in terms of %NBS.
- **Earthquake score** or **Seismic score** – The score given to an element (primary, secondary or non-structural) of a building to indicate the performance achieved in regard to the requirements of the Engineering Assessment Guidelines. Expressed in terms of %NBS
- **Ultimate Limit State (ULS) shaking demand** – the shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members for the same site.
- **Percent New Building Standard (%NBS)** – The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage.
- **Structural Weakness (SW)** – An aspect of the building structure and/or foundation soils that scores less than 100%NBS.
- **Severe Structural Weakness (SSW)** – A defined SW that is potentially associated with catastrophic collapse and for which the capacity may not be reliable assessed based on current knowledge.
- **Critical Structural Weakness (CSW)** – The lowest scoring structural weakness determined from a DSA.
- **Liquefaction** – The process which causes soil to behave more like a liquid than a solid during an earthquake. The shaking rearranges sand and silt grains in wet soil underground and the water between the grains is squeezed. Consequently, pressure builds up until the grains float and water is forced up to the surface through the easiest path it can find, often through cracks in the ground or concrete.
- **Lateral kinematic (cyclic) displacements of the ground** – When a portion of ground liquefies, the layers that sandwich it (typically the crust at the surface and the denser layers below it) will be able to displace relatively to each other during the seismic shaking.
- **Lateral spread of the ground** – The horizontal movement of ground towards the free-face or downslope as a result of the liquefaction of shallow underlying soil deposits.
- **Seismic separation** – Also known as *seismic gap*, is a clear distance between two buildings allowing both buildings to drift freely under lateral loading before hitting each other. A seismic separation of structural nature typically considers that any non-structural or architectural features within the gap will be smashed as the buildings hit each other.
- **Pounding** – The excitement of seismic loading within a structure when it hits a seismically separated neighbouring building. This typically happens when earthquake loading forces both buildings to drift toward each other, hence closing the seismic gap.

4 SITE DESCRIPTION

The Michael Fowler Centre is located at 111 Wakefield Street, Te Aro, Wellington. The building is bordered by the Wellington Town Hall building on the west and the temporary Royal New Zealand Ballet building on the east. Via the narrow laneway between the Wellington Town Hall and the Michael Fowler Centre, there is pedestrian access to the Civic Square, which is located northeast of the site, as well as vehicle access around the facilities of the Michael Fowler Centre. In between the Royal New Zealand Ballet building and the Michael Fowler Centre is a small carpark and vehicle access to the facilities at both buildings. North of the building is the Capital E building and the City-to-Sea Bridge, which provides pedestrian access between the Civic Square and the lagoon. To the south of the building is Wakefield Street. The site is on a reasonably flat ground profile.

The building comprises four seismically separated structures. The predominant structures at the site are the Auditorium and the Renouf Foyer. However, there are also two staircase structures that are connected to the main structures through the non-structural cladding and façade system but have an independent structural system.

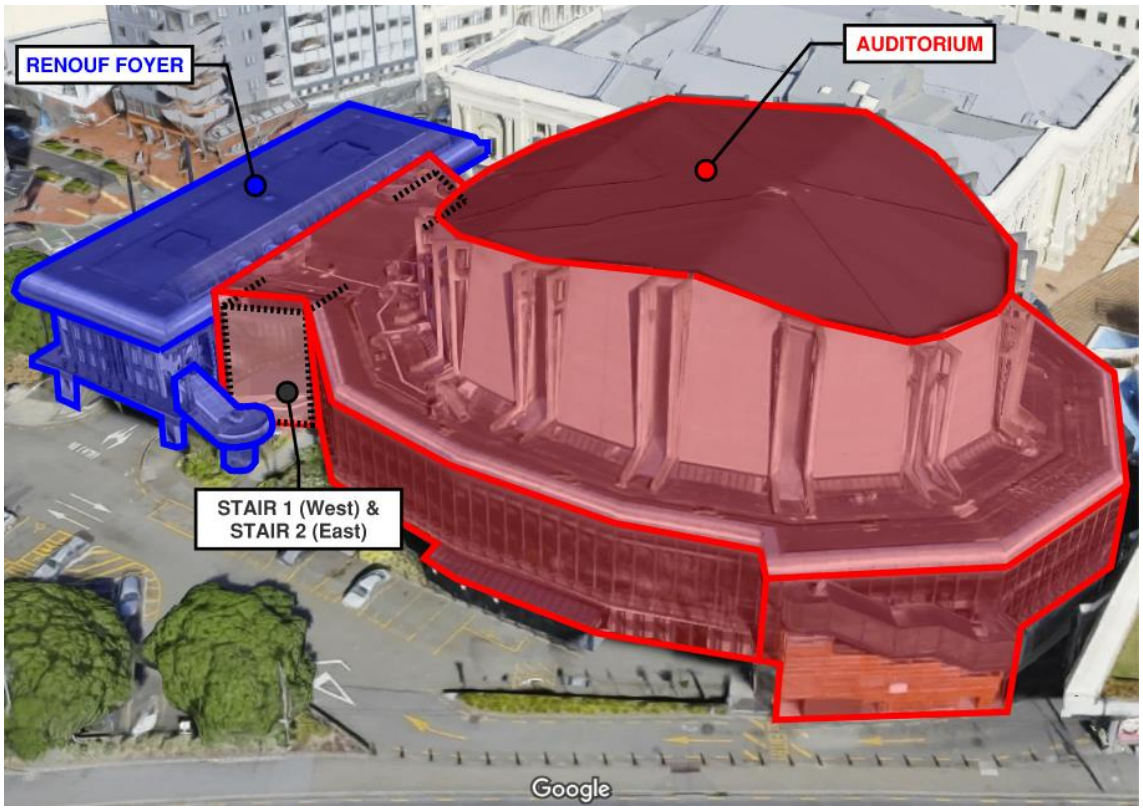

Figure 4-1: Snapshot from Google Street View of the Michael Fowler Centre looking south-West towards Cuba Street - highlighting the independent structures at the site

Until recently, there was a link bridge that provided access between the Wellington Town Hall and the Michael Fowler Centre at the first-floor level of the Auditorium. As part of the seismic strengthening works of the Wellington Town Hall, this link bridge has been removed and is not intended to be replaced.

4.1 Property Boundaries and Scope Exclusions

The Michael Fowler Centre is located at 111 Wakefield Street, Te Aro, Wellington. However, it is part of the property of the Civic Square area, listed on the Wellington City Council property map as 'Lot 1 DP 494594',

which is shown in Figure 4-2. For this DSA, the scope is limited to the building at 111 Wakefield Street, highlighted in yellow – which are the structures listed within this chapter of this report.

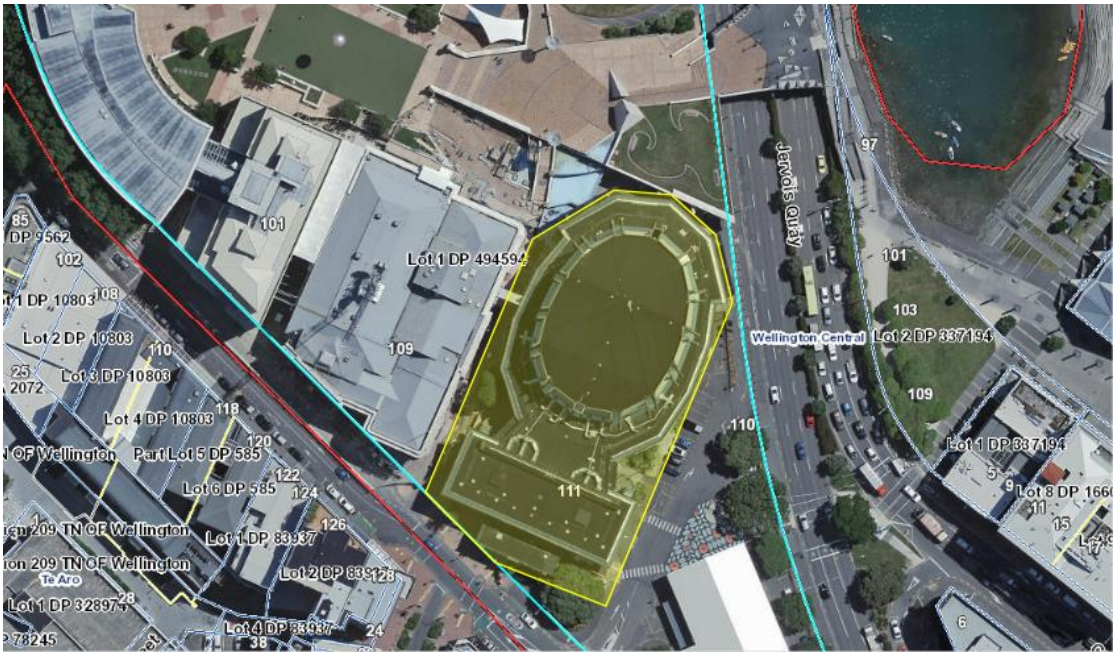

Figure 4-2: Snapshot from Wellington City Council property map (Wellington Maps / OneMap)

4.2 Ground Profile and Subsoil Conditions

A Geotechnical Assessment Report has been prepared by Tonkin+Taylor Limited (T+T) and is dated 25 November 2022. This report is appended to this report in Appendix B and summarised in this section.

The Michael Fowler Centre is founded on a flat ground profile which has been formed out of reclamation fill estimated to date from the 1890's. It consists of mixed loose silty/sandy gravels, sands, and gravelly silts up to a thickness of 4.4m. Below that there a thin ground layer of beach sand (medium dense gravelly sand) and a very thick layer of alluvium (dense to very dense silty sand and gravel) before bedrock is estimated at 40-60m below the ground floor level.

The available monitoring data indicates a current average groundwater level at RL+0.3m, ranging between RL+0.0m and RL+0.75m due to tidal effects. A groundwater level of RL+0.5m has been adopted for this assessment (current average tidal +0.2m average sea level rise over the next 50 years). The ground water level is higher than the base of the perimeter ring foundation beam of the Auditorium and the basement tunnel structure of the Auditorium.

The reclamation fill is prone to liquefaction, which is further discussed in Section 4.3. Towards the northeast of the building is the lagoon which provides a potential risk of lateral spread of the ground towards the lagoon following liquefaction.

The site is located approximately 1.5km east of the Wellington Fault.

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4.3 Potential for Liquefaction

The Geotechnical Assessment Report describes that liquefaction is expected to be triggered at seismic shaking that corresponds to 45%NBS_{IL3}. The consequences of the liquefaction are listed below:

- Lateral kinematic ground displacements, in the order of 100mm, which will impose lateral passive pressures onto the foundation beams, basement structure and piles during seismic shaking.
- Lateral spread of the ground, which will impose lateral passive pressures onto the foundation beams, basement structure and piles during seismic shaking. The quantity of lateral spread is expected to be largest on the northern end of the building near the free-face (the lagoon) and are expected to be smaller on the southern end of the building, at the Renouf Foyer. At the expected level of onset of liquefaction, at 45%NBS_{IL3}, the probable quantity of lateral spread is 315mm and 155mm at the northern and southern ends of the building, respectively. At larger levels of shaking the quantity of lateral spread increases by 100's of millimetres.
- Sand boils, which form as the pore pressure in the liquefied ground forces the groundwater to rise to the surface. This will likely lead to local tilting and sinking of the ground in the crust layer and the potential for ejecta. Particularly, affecting the slabs-on-grade and shallow foundations.
- Free-field settlements of the ground, in the order of 100mm, which is expected across the site of the Michael Fowler Centre.
- Buoyancy or uplift pressures on the elements of the structure that are buried below the ground water level, such as the basement tunnel that runs underneath the Auditorium.
- Much lower soil strength and softer lateral and vertical ground stiffness around the piles in the liquefied ground layers.
- Lower passive pressures (and resistance) that can be considered during liquefied ground conditions.
- Locally, piles may lose all axial load carrying capacity if founded above a liquefiable pocket of ground (considered unlikely as per the Geotechnical Assessment Report).

There is considerable amount of uncertainty in the assessment of the geotechnical parameters and therefore the Geotechnical Assessment Report also references upper and lower bounds for consideration. In understanding the global structural behaviour during seismic shaking, both the upper and lower bounds of stiffness have been considered in the assessment. This includes upper and lower bound stiffnesses in the vertical direction, the lateral direction, and for undegraded and liquefied ground conditions.

However, to determine the seismic scores of the foundation system and any considerations for structural 'step change', the assessment has been undertaken on the basis of most probable parameters.

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5 BUILDING DESCRIPTION

The Michael Fowler Centre is a performing arts centre in central Wellington. It was constructed in 1980 to a Warren & Mahoney architectural design with structural design undertaken by Holmes, Wood, Poole and Johnstone (a predecessor to Holmes Consulting and Holmes NZ). The building is also renowned to have undergone specific acoustic design by Professor A. Harold Marshall. The Auditorium seats up to 2209 people during a concert.

Figure 5-1: Historic photograph during the construction of the Michael Fowler Centre - taken from the Wellington City Recollect website

In accordance with the recommendations from NZS1170.0:2004, the Michael Fowler Centre is considered an Importance Level 3 (IL3) structure as a major structure accommodating crowds. It meets two specific criteria for this consideration as it allows more than 300 people to congregate in one area and it is considered a public assembly area (theatre) with a footprint larger than 1,000m^2.

The Michael Fowler Centre is not listed as a building on the New Zealand Heritage List/Rārangi Kōrero, however, the Civic Centre area (where the Michael Fowler Centre is a part of) is listed on the Wellington City Council (WCC) Heritage List as having significant value. The Michael Fowler Centre is referenced several times in the description of the area.

The Michael Fowler Centre comprises four distinct structures, namely the Auditorium, the Renouf Foyer and two independent staircase structures. The buildings are essentially symmetrical along the central axis of the footprint, aside from some minor detailing differences.

The material properties of the primary structure are taken from the design and construction documentation. Intrusive investigations were undertaken to verify the structural details of the Auditorium

promenade roof. No other intrusive investigations or testing were undertaken for this assessment. A summary of the site investigations is given in Section 6.2.

The primary structure of the Michael Fowler Centre comprises reinforced concrete elements, either formed cast insitu or as precast elements. The concrete strengths that were used in the design vary across the different building elements between 25-35 MPa and contain Grade 275 and Grade 380 (high-strength ('H')) reinforcement. The original technical specifications outline lap lengths and hook configurations of the typical reinforcement that, in accordance with the Engineering Assessment Guidelines, require no specific consideration (i.e., reduction) for the probable strength of the reinforced concrete elements.

5.1 Description of Auditorium

The Auditorium is an elliptical shaped structure that is predominantly used as a venue for concerts, conferences, and other large scale public events. In plan, there are four main areas referenced around the structure. Refer to Figure 5-2 which designates the Inner Auditorium, Backstage area, Promenade, and Entry Lobby area on Level 4. These areas are generally consistent up the height of the structure; however, they occur in different combinations. Figure 5-3 presents longitudinal (North-South) and transverse (East-West) sections of the Auditorium and highlights the floor plate regions.

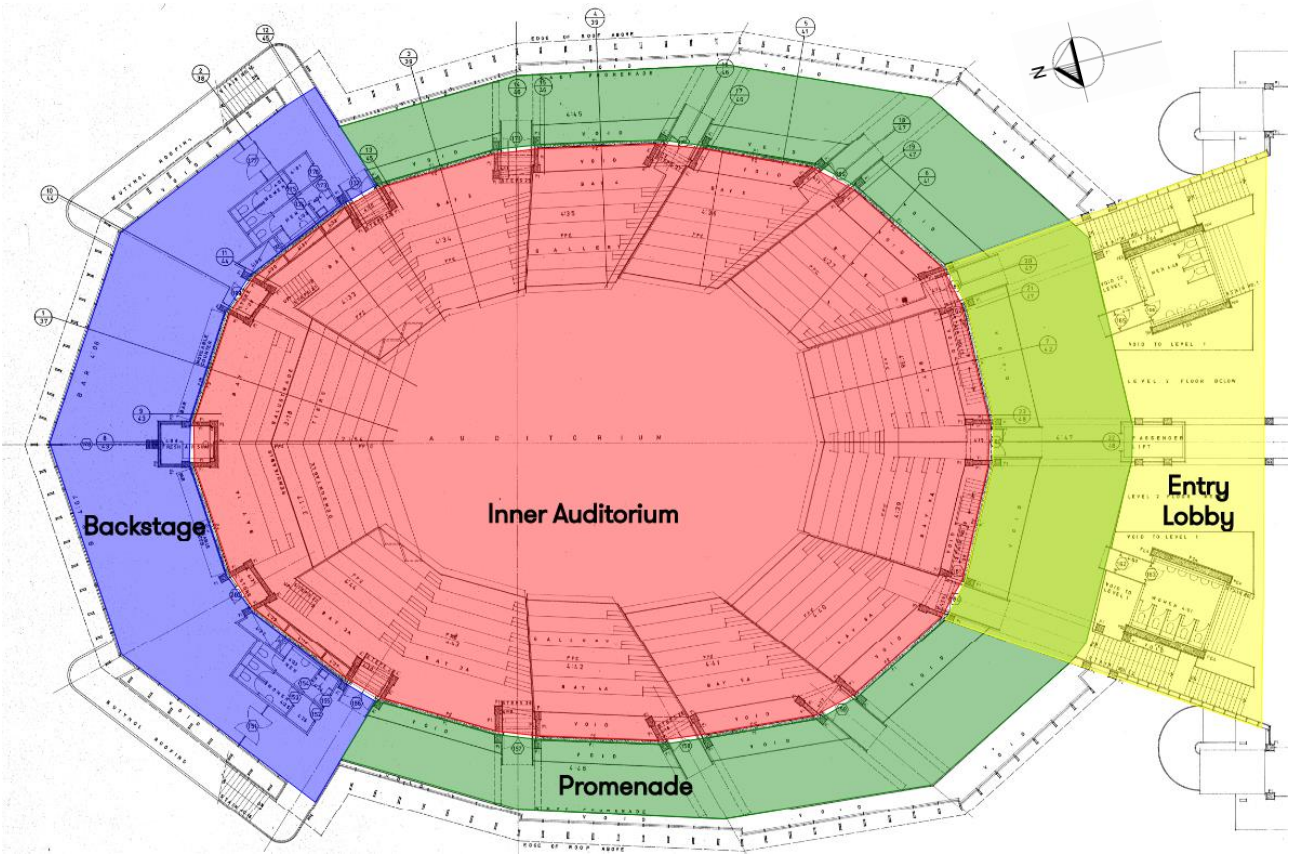

Figure 5-2: Regions of the Auditorium floor plate

The floors are discussed in further detail in Section 5.1.1.4, but to summarise the key floor plate locations:

- the Backstage area occurs Levels 1 to 4,
- the Inner Auditorium and Promenades have structure on Levels 1, 2, and 4,
- the Entry Lobby occurs on Levels 1 and 2,
- a localized sound booth occurs on the south end of Level 5 Inner Auditorium, and,

- a sloped diaphragm occurs in the southern Inner Auditorium from Levels 2 and ends at Level 1/2.

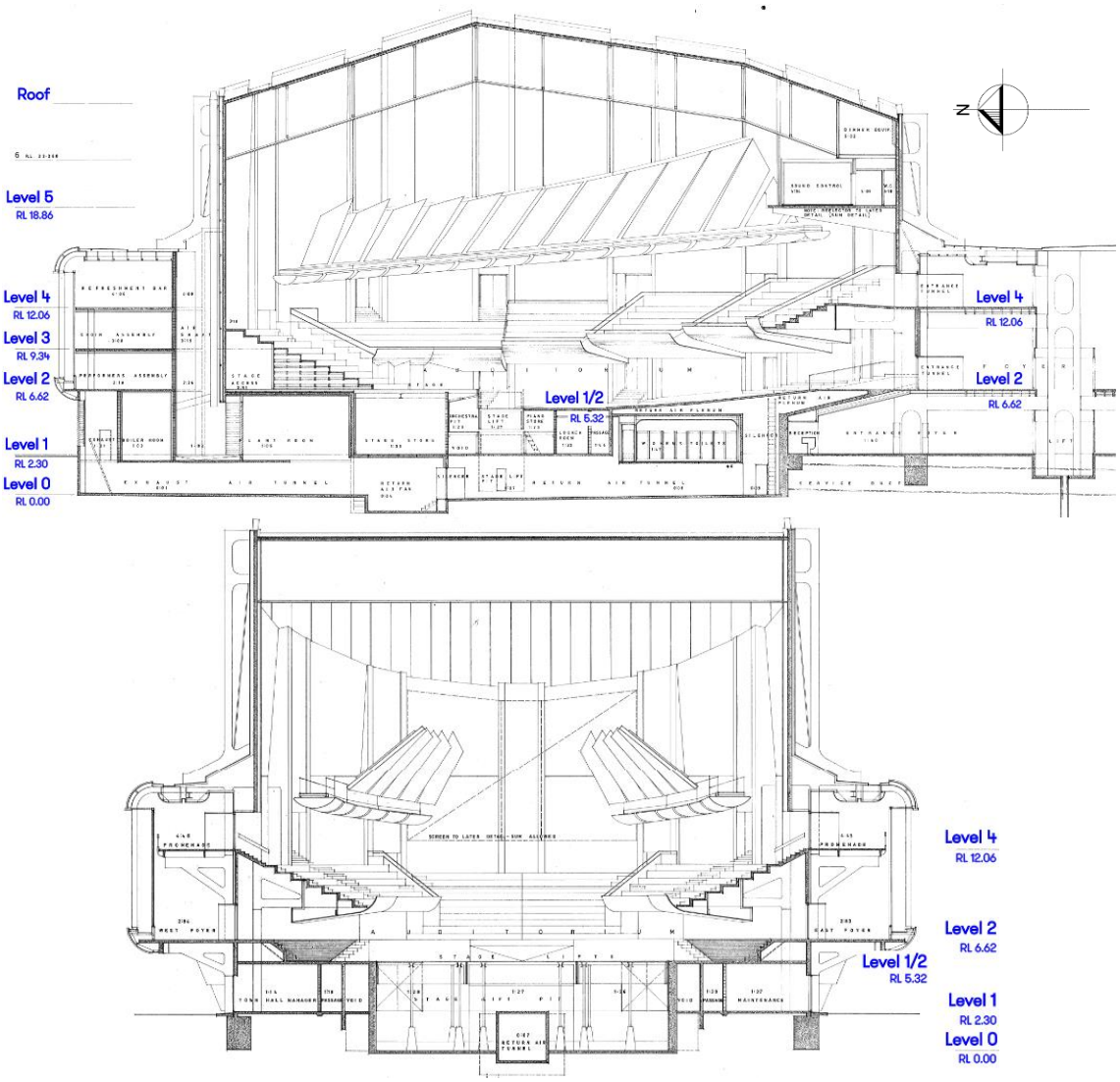

Figure 5-3: Longitudinal (top) and Transverse (bottom) architectural sections of Auditorium

The Inner Auditorium hosts sloped seating from Level 2 and Level 4 looking towards the stage at Level 2. The cantilevered seating accessed from Level 4 is tiered and changes in elevation around the ellipse; the seating at the south is higher than that towards the north. Radial seismic gaps occur around the floor plate at Level 4, allowing the individual cantilever walls to move independently at this level.

Hanging above the Inner Auditorium are timber reflector panels with two catwalk areas behind them for stage technicians. Not shown in Figure 5-3 but seen in Figure 5-4, is a hanging 'Donut' shaped structure above the stage also used for acoustics and lighting.

Around the outer ring of the Auditorium, the Promenade floors provide access to the inner seating areas. At the perimeter of the Promenade is a large, glass curtain wall spanning from above Level 4 and landing at Level 2. It is supported by stainless-steel mullions and glulam beams in the roof. The façade wraps around all the buildings at the site, continuing around the independent staircase structures and the Renouf Foyer

to the south. At the northern end of the outer ring, the façade around Promenades continues around the Backstage area.

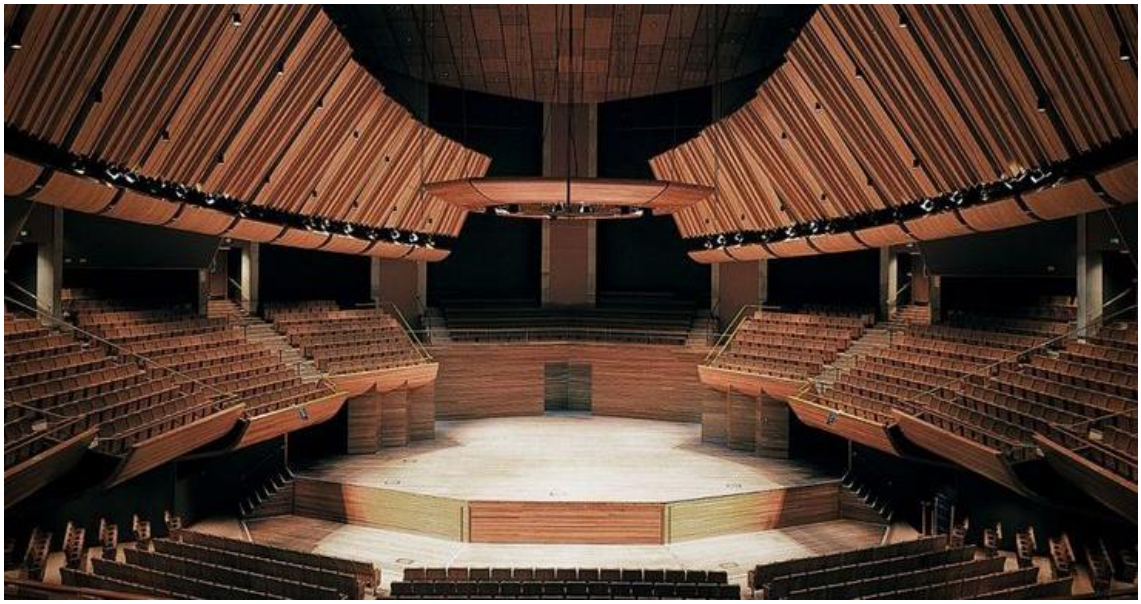

Figure 5-4: Photograph of the Inner Auditorium

The ground floor area at Level 1 provides rooms for offices, restrooms, and plant area underneath the stage and sloped Level 1/2 seating. The total footprint of the Auditorium has maximum dimension of 75m in the longitudinal direction and 49m in the transverse direction. The Auditorium has a total footprint of 2,500m^2, which includes the entry lobby around independent staircase structures. The area encapsulated by the main walls forming the Inner Auditorium is 1,400m^2.

A series of roof trusses support a roof comprising Dycore units with a concrete topping. They are 3.5m tall and provide access to a ceiling space. Between the bottom chords of the roof trusses spans a ceiling built-up of acoustic timber panels.

5.1.1 Lateral Load Resisting System (LLRS)

The primary lateral load resisting system comprises cantilevering C-shaped reinforced concrete walls around the perimeter of Inner Auditorium, these are referred to as the Bay Walls. The Auditorium is split into 14 radial Bays each comprising a single Bay Wall with two supporting buttress walls. The buttress walls are not perpendicular to the main Bay Walls, but are oriented to a central origin point, see Figure 5-5. The Bays are numbered 1-7 along the eastern side of the ellipse and are mirrored with the suffix (a) for the west side Bays. All discussion of the eastern bay structure is roughly identical to the western (a)-suffix bays. A deep, ground beam forms a mostly continuous ring supporting the primary Bay Walls over their piled foundation (refer Section 5.1.3).

The Bay Walls are continuous up the height of the building for most Bays. However, four Bays at the lobby (6, 7, 7a, 6a) discontinue at Level 2 and transition to precast column pairs or an isolated buttress for the ground floor (Level 1), see Figure 5-6. The column pairs and isolated buttress resist the overturning of the walls above while the sloping diaphragm to Level 1/2 transfers the shear to the inner squat wall, Wall 21.

In addition to the localised cantilever wall discontinuity at Level 1, significant lateral resistance is added in the Inner Auditorium by the squat walls creating rooms at the ground floor. The walls are a mix of reinforced concrete and concrete masonry units, with the most significant inner walls labelled in Figure 5-6.

The inner wall system is tied to the cantilever walls through concrete diaphragms at Level 1/2 and 2, as well as on the ground floor level. The single storey walls are supported by a series of slender ground beams, shallow strip footings, and isolated pile foundations (refer Section 5.1.3).

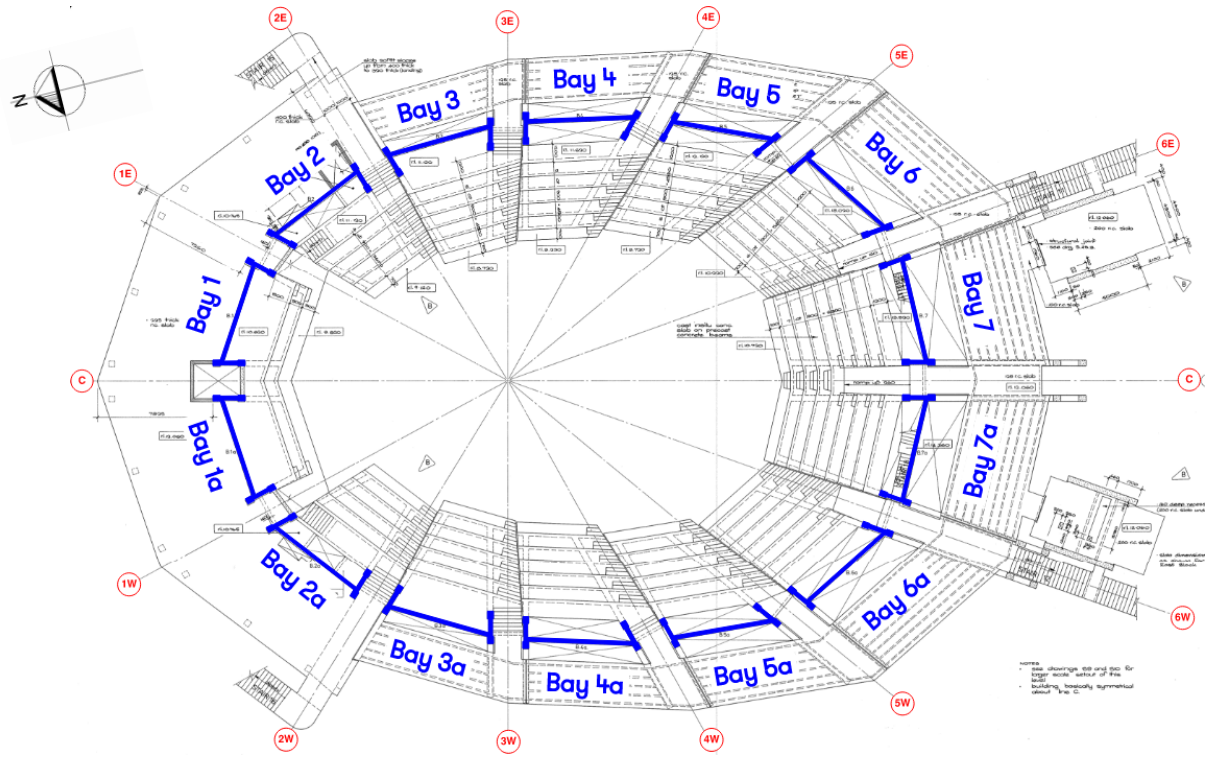

Figure 5-5: Auditorium Level 4 LLRS layout (drawing S7 of original structural set)

5.1.1.1 Cantilever Bay Wall Buttressed System

Typical cantilever walls

The typical cantilever walls span from the deep ring beam to the roof with a cantilevering height of 23.6m to 29.0m. The main Bay Walls are 7m long between their buttresses which are 2.2m long. The wall system is split into two construction methods up the height of the Auditorium. The top portion, approximately above RL 17.00, is constructed from precast concrete elements, while the bottom portion below is constructed as cast insitu concrete. A typical elevation of a Buttress Wall is shown on Figure 5-8 along with a section through the precast portion. This figure is colour-coded and represents the following structural description.

The upper precast buttress is cast as a frame element with two 400x500mm columns and beam segments between them. The precast buttress cantilevers from the insitu buttress below and is connected with rebar in grouted 'Uniflex' ducts. On the outer edge, a cantilever 'fin' supports the promenade timber roof and façade system on a hung structural steel beam (refer Section 5.1.4.2).

In between the precast buttresses are stacked precast concrete wall panels, forming the web of the Bay Walls, and stitched to the buttresses with and insitu joint up the height. These precast concrete wall panels are 315mm thick, of which 200mm is considered the structural thickness and the remainder is architectural built-up. The precast concrete Bay Wall panels slope inwards towards the Auditorium and end up almost flush with the inner column of the precast buttress at the roof level. The stacked panels are connected to each other through two weld-plate connections, formed out of a 350mm long and 25mm thick steel plate.

Between adjacent Bay Walls are precast concrete infill panels, in line with the inner column of the precast buttresses. These infill façade panels are 250mm thick and are connected to the precast buttresses through a bolted connection, with some allowance for the panels to drift independently from the rest of the structure. The infill panels are therefore considered to be a secondary structural element of cladding and are not part of the primary lateral load resisting system.

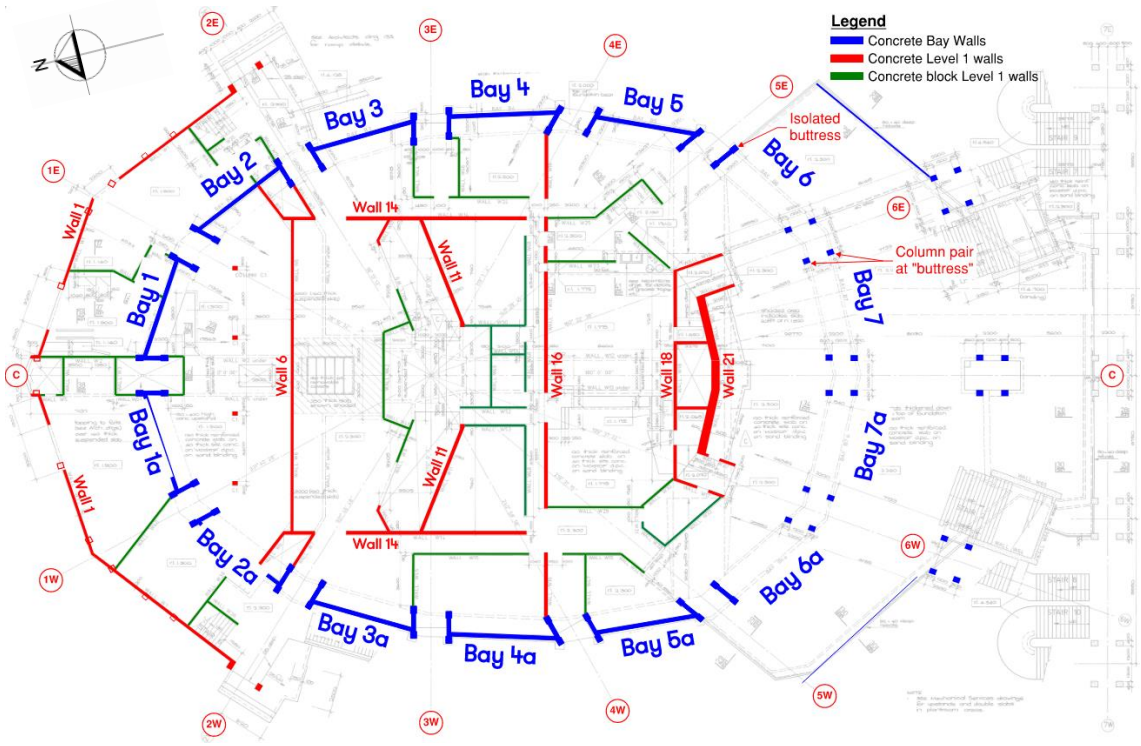

Figure 5-6: Auditorium Level 1 LLRS layout (drawing S3 of original structural set)

The lower cast insitu portion of the walls are 250mm thick with the same lengths as the precast portion above, refer Figure 5-7. The ends of the buttresses are either thickened to 400x500mm insitu columns or are built out with a precast capping element part of the Precast Cantilever Beams supporting the promenades and interior seating (see Section 5.1.2.2).

A 2.0m space exists between adjacent Bay Wall which is generally used for entrance pathways into the Inner Auditorium. At some ends of the Bay Walls, a deep cantilever beam is cast to support gravity loads; it does not tie to the adjacent beam and therefore does not act as a spandrel.

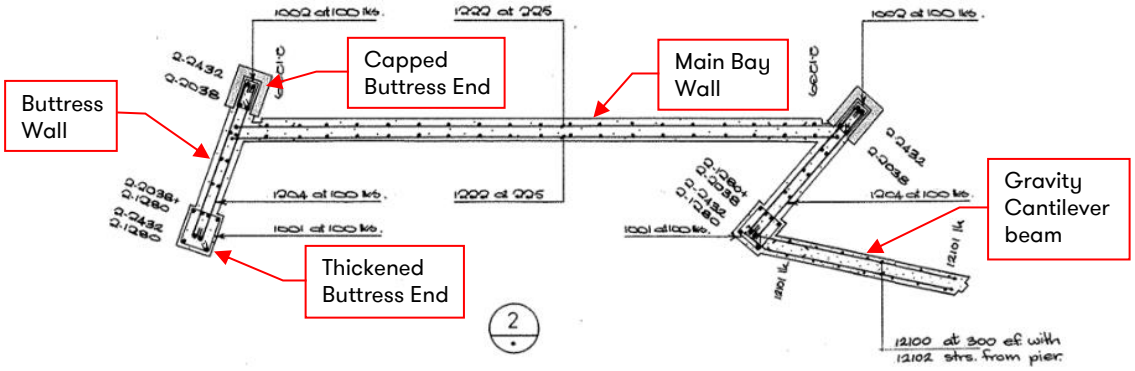

Figure 5-7: Typical insitu section of cantilever wall element

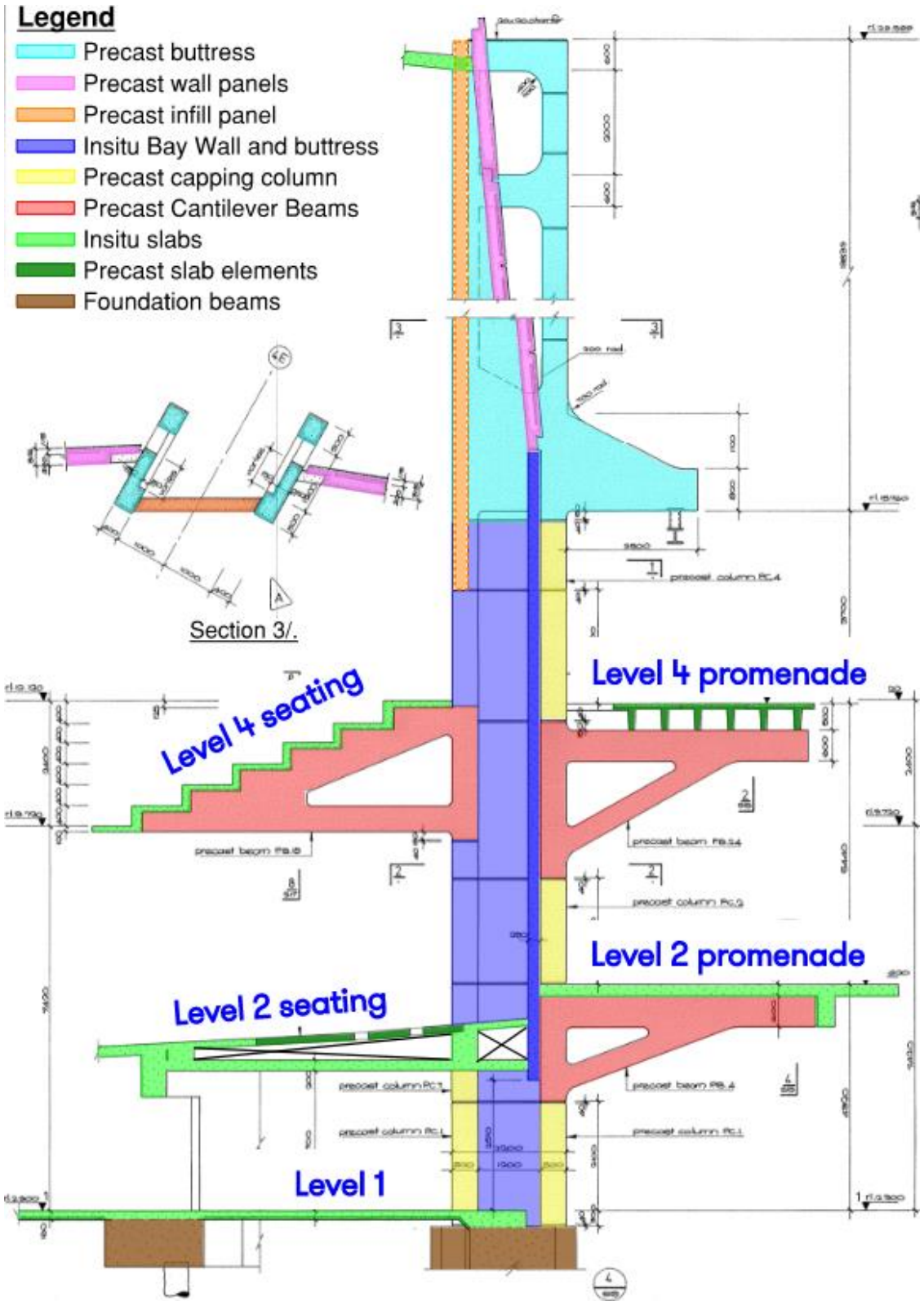

Figure 5-8: Elevation of buttress wall at Bay 5 (drawing S20 of original structural set)

Specifics of cantilever walls in the Backstage area

The Backstage area runs between the northern buttresses of Bays 3 and 3a on Levels 1 to 4. The semi-circular floors are cast integral with the Bay Walls and span to Wall 1 at the northern façade. On the Inner Auditorium face, the cantilever walls support the stage at Level 2 and is similarly cast integral to the wall face.

Various non-structural concrete block walls occur and are highlighted in Section 5.1.4.1. These walls between Level 2 to 4 have reinforcing hooked to the slab above and below, but no horizontal reinforcing hooking to any adjacent concrete Bay Wall. Below Level 2, horizontal reinforcing ties generally do occur to the adjacent concrete walls and are classified into the squat wall system at Level 1, see Section 5.1.1.3.

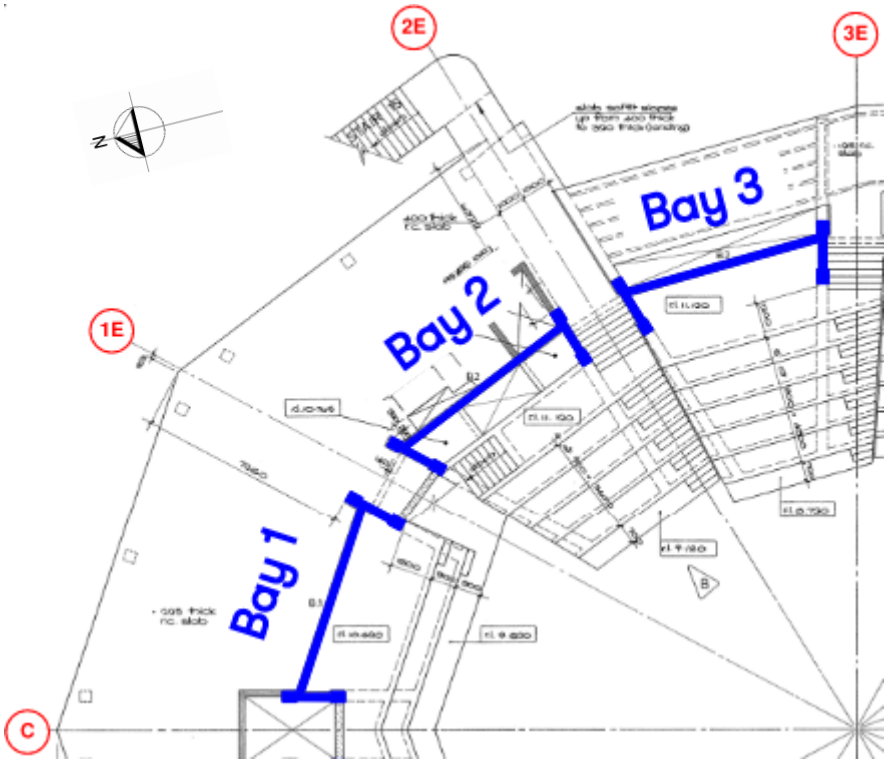

Figure 5-9: Partial plan of Level 4 Bay 1, Bay 2, and Bay 3 (drawing S7 of original structural set)

Specifics of cantilever walls in the Lobby area

The Lobby area Bays are considered to be Bays 6, 7, 7a, and 6a. In this region the cantilever walls do not continue to Level 1. For Bays 6 and 6a, the northernmost buttress continues to the ground floor level and referred to as an isolated buttress in Figure 5-6. The southern buttress of this bay lands on a precast arched spandrel over two precast 400x500mm column. For Bays 7 and 7a, both the buttress walls transition to the precast arched spandrel and column element. The buttress walls which change to a precast frame have a long precast concrete frame extension reaching outwards into the lobby; refer to Figure 5-10 and Section 5.1.1.2.

Additionally, the upper precast Bay Walls and buttresses support a small slab at Level 5 at the Inner Auditorium. Precast tapered beams cantilever off the ends of the precast buttresses, which support the slab.

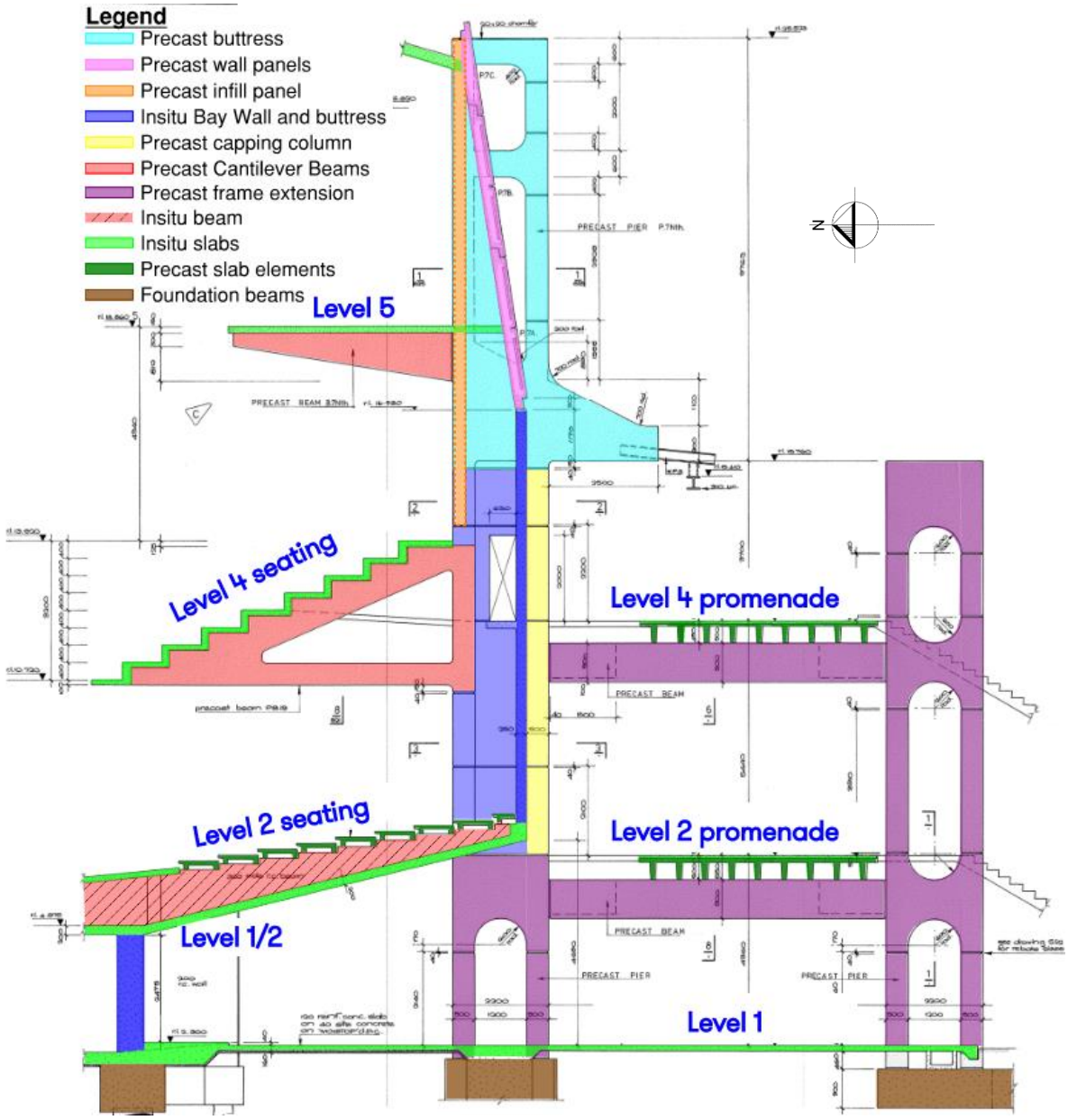

Figure 5-10: Elevation of frame extension at Bay 7 (drawing S23 of original structural set)

5.1.1.2 Lobby Frame Extensions

The Lobby frame extension in Figure 5-10 has 400x900mm precast beams spanning primarily in the building longitudinal direction to 400x500mm columns at Level 2 and Level 4. The southern pairs of precast columns extend upwards to support the lightweight timber roof structure (not shown). No beam element is present in the building's transverse direction between pairs of columns.

5.1.1.3 Squat Wall System at Level 1

The ground floor level (Level 1) has a system of squat concrete walls in both the longitudinal and transverse direction of the building. The most significant are named in Figure 5-6 above.

On the northern perimeter of the building is the wall and column system referred to as Wall 1. From the Level 1 to 2, this system consists of a 200mm thick concrete and a set of two columns at the Eastern and Western ends. Wall 1 extends to Levels 3 and Level 4 as a line of columns, supporting a two-way spanning insitu concrete slab at these levels. The columns at these upper levels are horizontally offset relative to the centreline of the wall at the ground floor level, creating an eccentric corbel connection to the exterior face. The upper columns are primarily gravity load-supporting and are not considered part of the primary lateral system.

Wall 6 is a 200mm thick concrete wall and is the primary transverse wall at the north end of the story level. The wall extends below the ground floor level and is also part of the basement structure.

Wall 11 occurs in two locations, one each at the eastern and western halves of the floor plan. They occur toward the centre of the structure and have the most pronounced plan skew. They are 250mm thick and extend below the ground floor level to be part of the basement structure.

Walls 14 also occurs in two locations, one each at the eastern and western halves of the floor plan. They are 250mm thick and are the primary longitudinal concrete walls at the eastern and western Inner Auditorium.

Wall 16 is located at the transition between the flat seating area and the upper sloping seating area at Level 1/2. It runs transversely across the full footprint of the Inner Auditorium, between Bay 4 and Bay 4a. This is a 250mm thick wall.

Further south, where the lower Level 1/2 transitions from the flat portion to the sloping portion there are the walls referenced as Wall 18 (including its flanking Walls 17 and 19) and Wall 21 (including its flanking Walls 20 and 22). Wall 18 is 200mm thick and the majority of Wall 21 is a 600mm thick triply reinforced concrete wall (see Figure 5-11). The central portion of the 600mm thick concrete wall extends below ground level as part of the basement structure. Wall 21 provides lateral shear resistance through the sloping Level 1/2 diaphragm where the lobby Bay Walls discontinue.

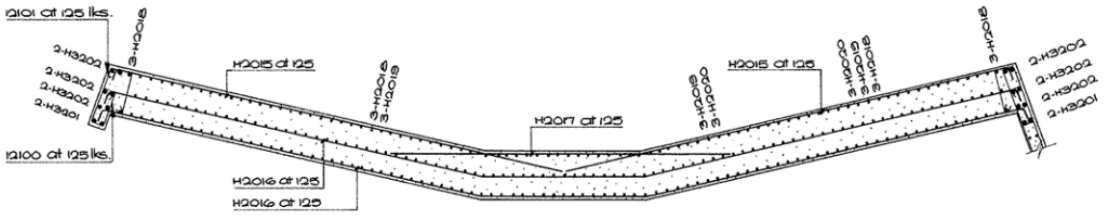

Figure 5-11: Cross-section of Wall 21 (drawing S48 of original structural set)

5.1.1.4 Floor Diaphragms

The floor systems in the Auditorium are a mix of cast insitu flooring and precast flooring systems. These various slab areas are outlined here.

Level 5

Supported by tapered Precast Cantilever Beams (Section 5.1.2.2) on Bay 6 through 6a buttresses, is a localized 150mm thick cast insitu concrete floor area at Level 5, see Figure 5-12. The slab has openings along the length of Bay 7 and 7a where two symmetrical staircases connect down to Level 4. Through another set of steel stairs there is also access from the Level 5 floor to the catwalks system that runs behind the acoustic reflector panels.

The Level 5 slab is one way spanning between the Precast Cantilever Beams where stirrups extend into the slab to provide an integral connection. The slab also connects to starters cast into the sloped precast Bay Wall panels.

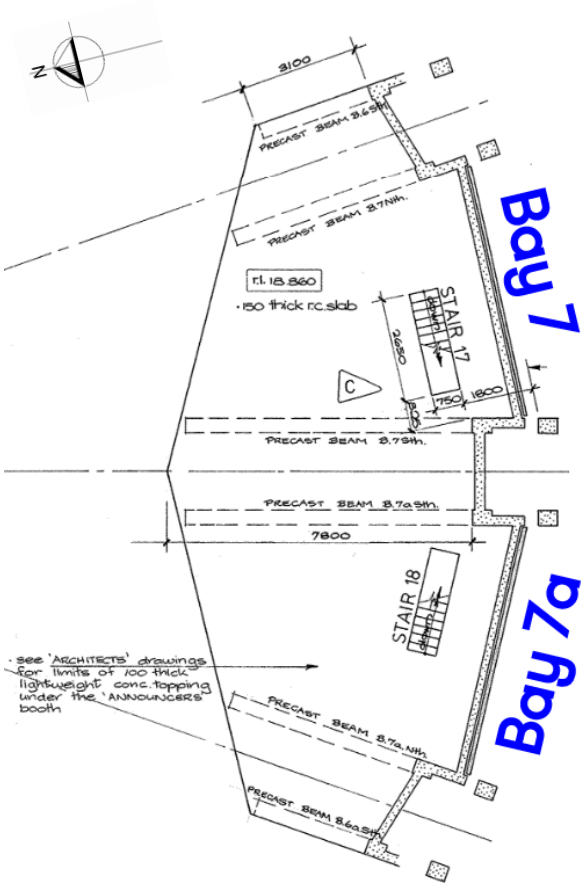

Figure 5-12: Level 5 slab (drawing S6 of original structural set)

Level 4 stepped seating

At the Level 4 Inner Auditorium, there is a ring of tiered insitu stepped seating 100mm or 200mm thick, see Figure 5-13.

Radial seismic gaps occur around the floor plate at Level 4, allowing the individual cantilever walls to move independently at this level. Individual bays of stepped seating are only connected to a single Bay Wall segment and a 100mm gap generally occurs to the adjacent seating slab. This creates discrete slab 'wedges' around the floor plan. The exception to this separation is at Bays 1 and 1a and Bays 7 and 7a, where the flooring is continuous across two Bay 'wedges'. See Figure 5-14 demonstrating the 'wedges' sections. Large plan voids exist on either side of the Bay Wall webs.

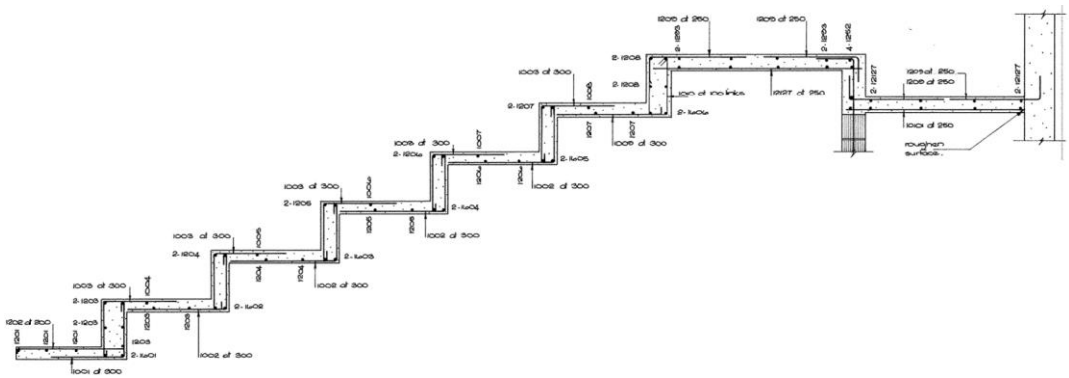

Figure 5-13: Cross-section of the Level 4 stepped seating at backstage region (drawing S65 of original structural set)

Level 4 promenade

The promenade has 450mm deep double-tee units with a 50mm topping reinforced with a cold-drawn mesh. The double-tee units span between the Precast Cantilever Beams (Section 5.1.2.2) and lobby frame extensions. The double-tee units are typically web-supported with 110mm seating, although some locations have a flange-hung detail with 75mm seating. The 2m wide spacing between Bays has a 125mm cast insitu slab which connects the promenade to the stepped seating of the Inner Auditorium.

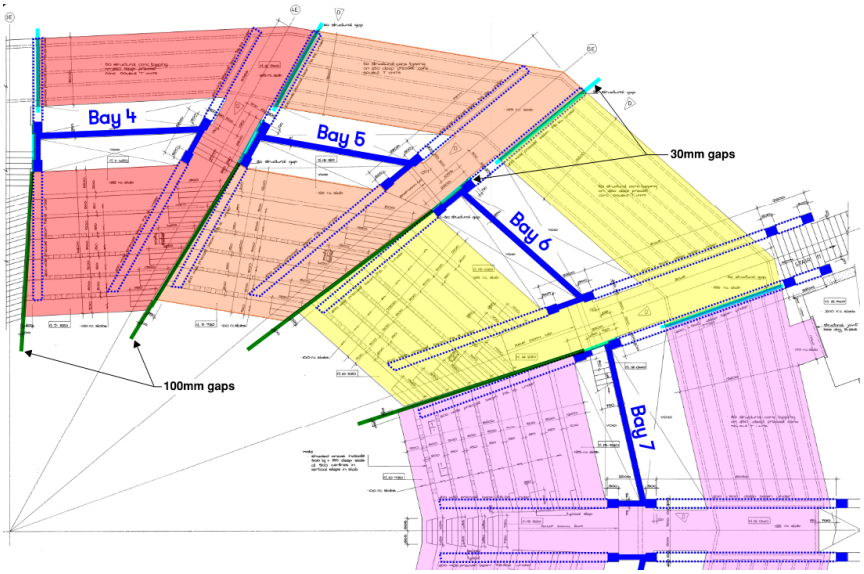

Figure 5-14: Level 4 partial plan highlighting the different bay 'wedges' (drawing S10 of original structural set)

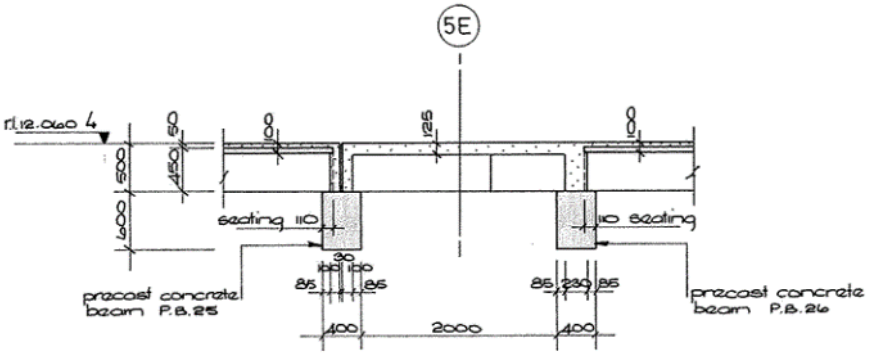

Figure 5-15: Level 4 transition between double-tee flooring and cast insitu with the 30mm structural gap (drawing S21 of original structural set)

Similar to the inner stepped seating, individual Bays are disconnected by a 30mm structural gap through the Promenades, continuing the 'wedge' behaviour. See Figure 5-15 for a section of the transition between the double-tee and insitu portion with seismic gap (highlighted in cyan on Figure 5-14). This has been equipped with a 'smooth surface', so it likely has limited friction resistance. Where the double-tee lands next to the insitu slab, the surface of the Precast Cantilever Beams is intentionally roughened with a starter bar protruding into the topping concrete, tying it to its Bay 'wedge'.

The lobby area of the promenade provides access the independent stair structures (see Section 5.3). The Auditorium and the independent staircase structure slabs are separated by seismic gaps at this location.

Level 2 promenade

Around the perimeter of the Auditorium a 300x900mm deep cast insitu beam occurs along the promenade east and west side edges. Typically, a 200mm thick cast insitu concrete slab spans two-way between the Precast Cantilever Beams and the perimeter beam to Bay Wall. The slab cantilevers 1150mm over the perimeter beam where it supports the mullions of the façade and roof system, refer Figure 5-16.

The Lobby region is different, as the promenade has 500mm deep double-tee units and a 50mm topping reinforced with a cold-drawn mesh. These double-tee units span between the supporting precast beam system. Large plan voids occur on the Promenade side of the Bay Wall. The double-tee units are mostly web-supported with 110mm seating, although some have a flange-hung support with 75mm seating. A 125mm cast insitu slab spans between Precast Cantilever Beams of adjacent bays. There are no specific structural gaps in this floor slab.

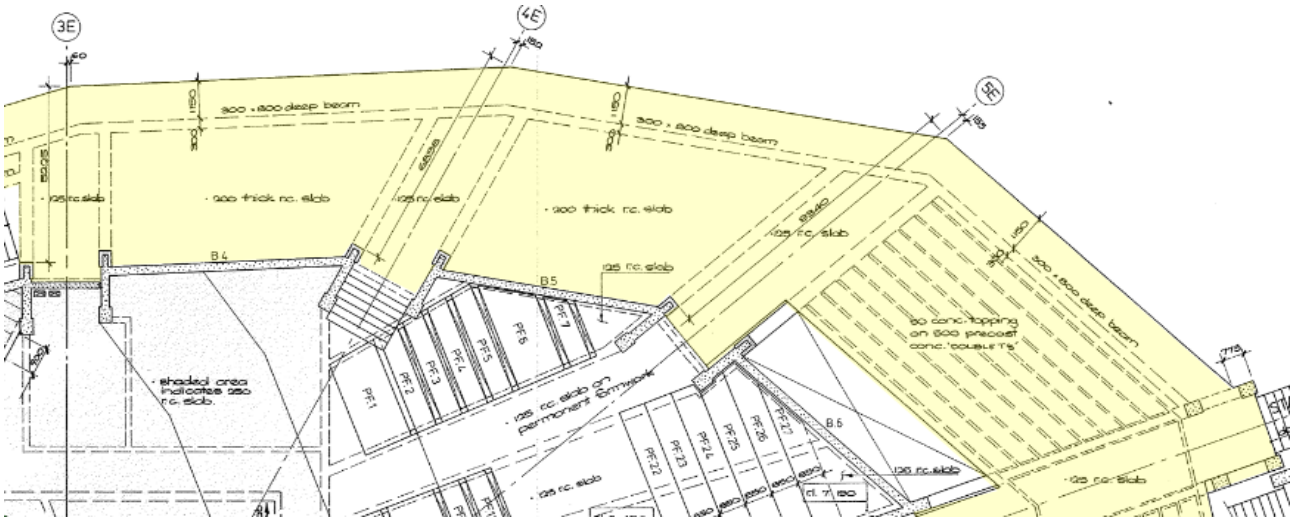

Figure 5-16: Level 2 promenade partial plan (drawing S5 of original structural set)

Level 2 lobby region

Between Bays 7 and 7a the flooring continues onwards towards the Level 2 slab of the Renouf Foyer, allowing people to move between the two structures. The floor consists of a few different configurations on the Auditorium side, which are referenced in Figure 5-17.

Firstly, the green highlighted 500mm deep double-tee units are web-supported on the precast frame extensions of Bays 7 and 7a and cantilever over both sides. A concrete edge beam is formed as a 600x350mm beam with a chamfered edge on the outer corner, shown in hatched red, which spans from the promenade (yellow) to the cantilevering double-tees (green). 'Pockets' are allowed for on the beam face to recess post-tensioning anchor heads. No specifications nor details are provided on the post-tensioning or the concrete edge beam.

Secondly, there are 350mm deep double-tee (light blue) central to the other floor units. These span between the precast frame extensions and the 600x350mm concrete edge beam. It is expected that the edge beam and post-tensioning in the 500mm deep double-tee units are designed to support these 350mm deep units.

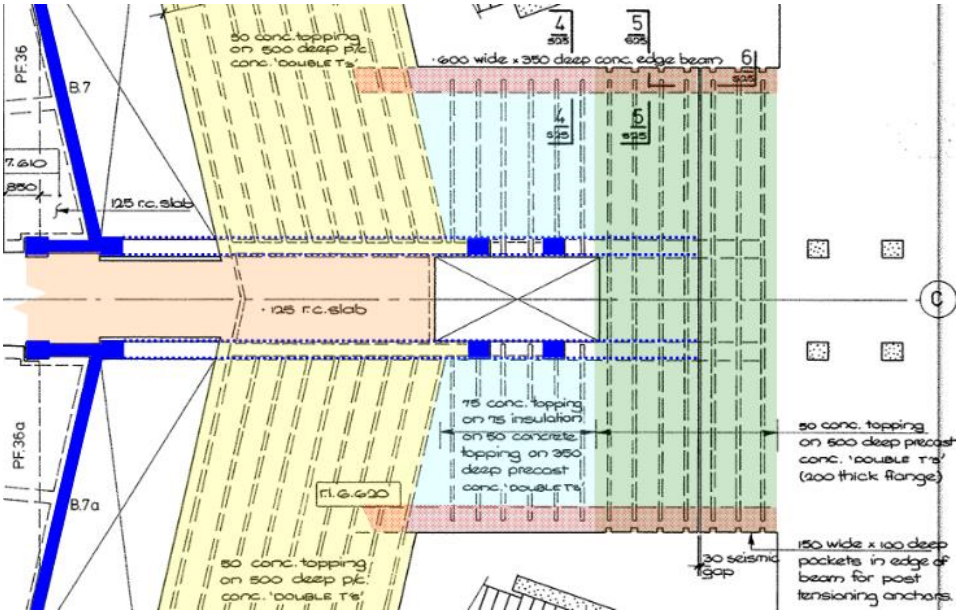

Figure 5-17: Level 2 lobby floor plan to the Renouf Foyer (drawing S5 of original structural set)

Level 1/2 and Level 2 of Inner Auditorium

The Level 2 Inner Auditorium consists of several separate areas and is shown on Figure 5-18 with colours referencing the different zones from north to south.

The stage is a 200mm cast insitu concrete slab, spanning between Bays 2 though 2a, four isolated concrete gravity columns, and the long squat Wall 6. The furthest east and west portions of slab overlap with the southern seating area of the Level 2 floor, creating a plenum space. The stage area also a lightweight timber framed floor supported by a grid of steel beams, south of Wall 6.

South of the stage is a movable lifting stage area for piano storage and an orchestra pit. A steel frame can be raised and lowered as required, but the member sizing and detailing is not available in documentation.

The southern half of the Inner Auditorium is used for seating: a flat portion in the middle of the Auditorium and a sloping portion that slopes upwards towards the south (shown in orange). The sloped portion is split into two floor slabs connected by tapered beams which span from the discontinued Bay Walls onto the squat walls of Wall 16, 18, and 21. See Figure 5-19 for a section of this built-up.

The 125mm thick upper slab is perforated and provides a walkway around precast seating units. The lower slab is referenced as the Level 1/2 floor and is the primary shear transfer element to Wall 21 and the rest of the squat wall system. The lower slab is a 200mm cast insitu concrete slab reinforced with deformed bars.

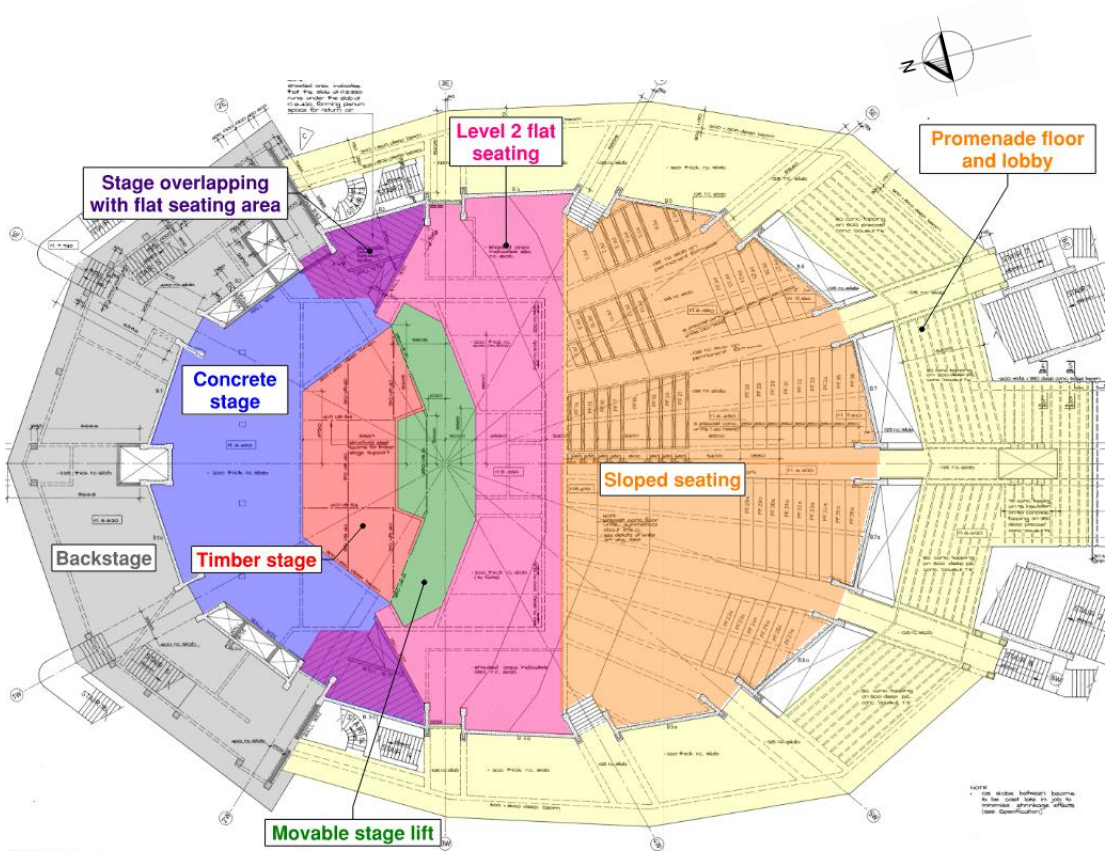

Figure 5-18: Level 2 floor plan area (drawing S5 of original structural set)

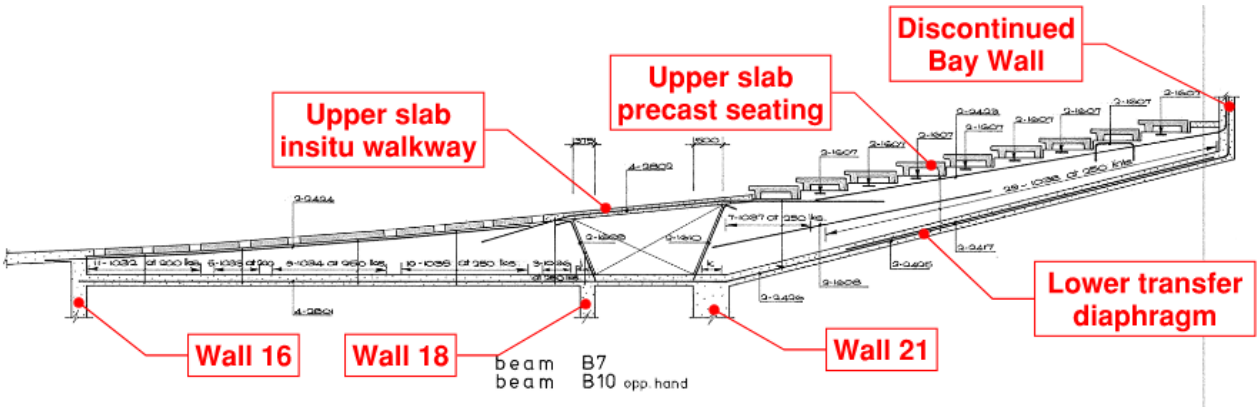

Figure 5-19: Section of sloped transfer diaphragm to Level 1/2 and tapered beams between Level 2 and Level 1/2 (drawing S59 of original structural set)

Level 2, 3, and 4 backstage area

On the northern end of the Auditorium, the backstage area slabs are similar at each level. These floor areas are accessible from the staircases on the western and eastern ends, labelled Stairs 5, 6, 13, 14, 15 and 16, as well as from the Promenades and the stage (refer Section 5.1.4.3). The slabs are 225mm thick cast insitu concrete floors that span two-way between the Bay Walls and northern Wall 1. There are no beams as part of Wall 1, forming slab-column joints at those locations.

There are a few locations noted as 400mm thick. These are typically 2m wide and located in line with the landings of Stairs 15 and 16. There are also two 1m wide strips at the southern extends of the Level 2 slab near the concrete masonry block walls 62 and 63.

Notably, a 20mm seismic gap is called out at the north end of Bay 3 and 3a on Level 3 and 30mm gap on Level 4. This gap allows for differential movement between the backstage area and Promenade slabs and creates a combined 'wedge' region. The gapping is continued around the beams and the block walls number 62 and 63 at this interface.

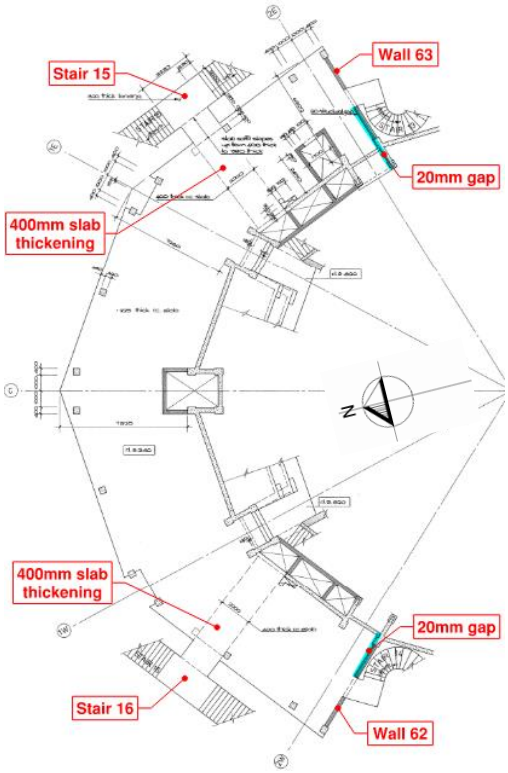

Figure 5-20: Level 3 backstage area slab (drawing S6 of original structural set)

Ground floor (Level 1)

The ground floor level provides access to all areas of the Auditorium. The ground floor is generally at the same elevation, except for the backstage area and portion of the inner Auditorium which are lower by approximately 0.4m and 0.5m, respectively. A number of voids also exist to provide access to the basement tunnel (refer Section 5.1.3.1).

There are three reinforcing areas for the slab-on-grade, shown in Figure 5-21, with colours designating each. The lobby is a 100mm slab-on-grade reinforced with cold-drawn mesh. The outer perimeter has a thickened slab edge and connects to the deep ring beam at the north face.

The remainder of the ground slab is a 120mm thick and is typically reinforced with deformed bars (blue and grey in Figure 5-21). The portion shown in red contains a higher percentage of reinforcing, presumably to tie the squat Wall 21 to the deep ring beam. In general, this slab acts as a rigid diaphragm to tie together the Inner Auditorium squat walls and the perimeter ring foundation beam (see Section 5.1.3.2).

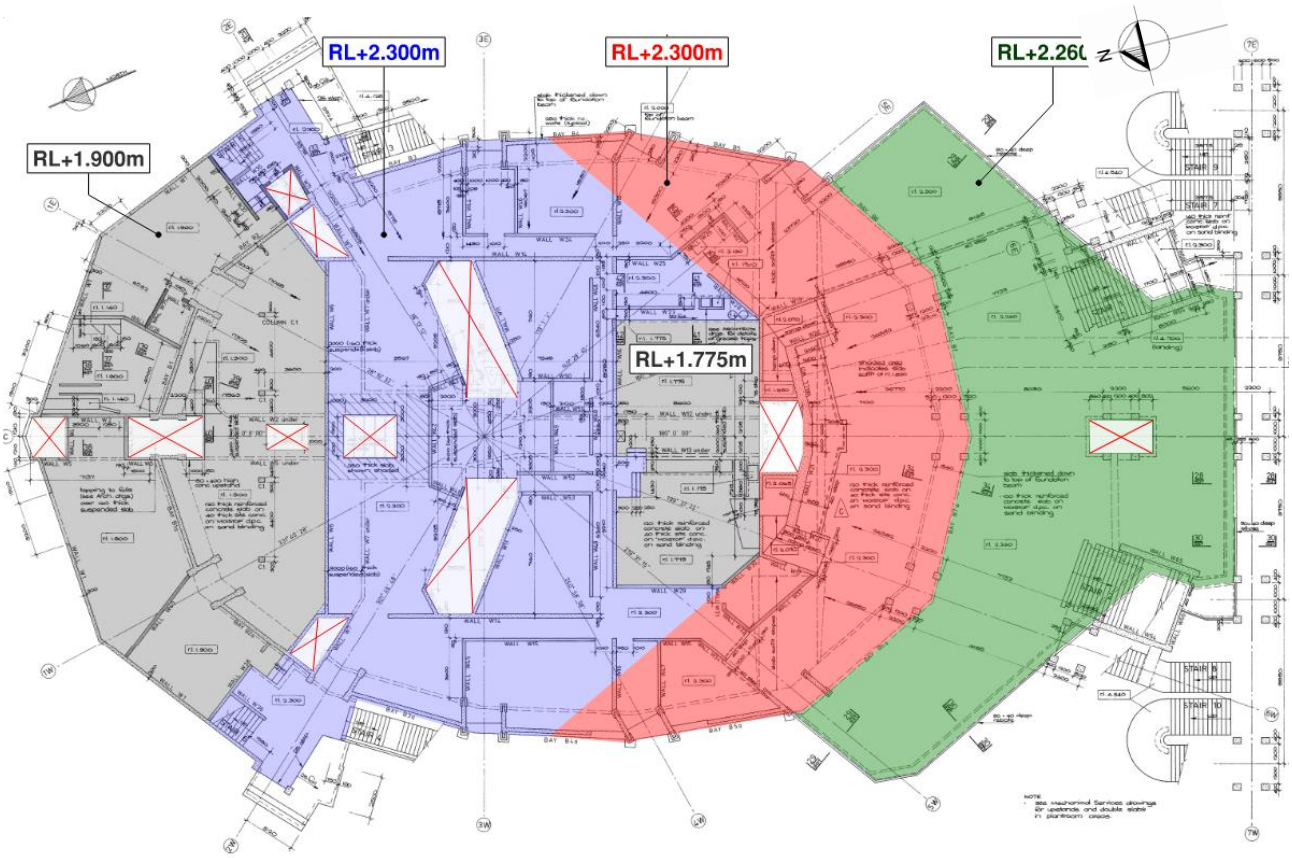

Figure 5-21: Level 1 slab on grade (drawing S3 of original structural set)

5.1.1.5 Roof Diaphragm

The roof of the Auditorium consists of 150mm thick Dycore hollowcore units topped with a 50mm insitu concrete reinforced with cold-drawn mesh. The Dycore units span in the longitudinal direction of the building to the deep roof trusses which span in the transverse direction. The Dycore units only provide gravity support, it is reliant on the 50mm topping to act as a diaphragm for lateral loads from seismic shaking.

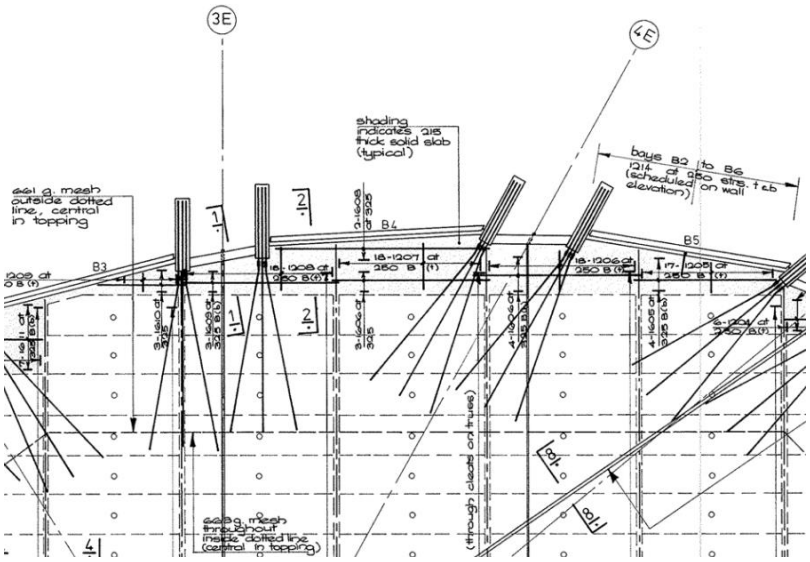

Figure 5-22: Partial roof plan showing diaphragm reinforcing (drawing S82 of original structural set)

Dycore units are precast hollowcore concrete units that are extruded sections with circular shaped voids to reduce their weight. The 1.2m wide panels were typically reinforced with prestressing strands that provide a constant (long-term) compression stress in the units. This stress allows them to span relatively large distances compared to conventionally reinforced concrete slabs. No shop drawings of the Dycore units for Michael Fowler Centre could be retrieved for this assessment, although typical designs for the time of construction have been consulted. Dycore units typically do not contain any vertical shear reinforcement and this assumption has been maintained. The original seating condition of the Dycore floor units (seen in Figure 5-23) has been improved during the seismic strengthening works described in Section 5.1.5.

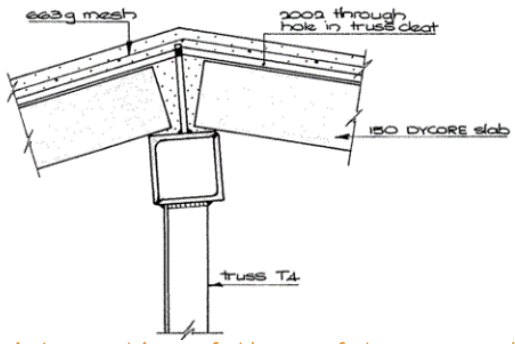

Figure 5-23: Detail of typical intersection of the roof trusses and the Dycore units (drawing S82 of original structural set)

At the edges of the elliptical roof, a 215mm cast insitu slab is formed to stitch the roof to the vertical elements, see Figure 5-22. The roof diaphragm is connected to the precast buttresses through 3-Ø28mm starter bars. This insitu edge is tied to the sloped precast Bay Walls through starter bars in the upper panel.

5.1.2 Gravity Load Resisting System

The dead loading and live loading that is considered in this seismic assessment is described in Appendix C, which has been determined in accordance with NZS1170.1:2002. The majority of structural elements act as part of both the lateral load and gravity resisting system, including the cantilever Bay Walls and diaphragms.

Elements in this section are predominantly gravity-only elements and not part of the primary lateral load resisting system discussed in Section 5.1.1.

5.1.2.1 Roof Trusses

The roof trusses are 3.5m deep and are formed from mild steel angle profiles. The top and bottom chord of the trusses are formed out of two angles that are longitudinally welded together to make a box section. The chords span half the width of the Auditorium and connect in the middle. The connection at the top chord is a bolted end plate while the bottom chord has a true pin (see Figure 5-24). The roof trusses connect to the precast concrete structure both at the precast Bay Walls and the precast buttresses, depending on the layout (Figure 5-26). Cast in anchors support built-up steel corbels and cradles, on which the trusses sit.

The roof slopes down towards the north and the south, forming the highest point approximately in the middle, refer to Figure 5-25. The total change in height between Bay 7 and the apex is approximately 6.5m, with a maximum slope of 20 degrees. This change in roof height is also reflected in the top elevation of the precast wall elements around the Auditorium.

The roof trusses are braced to each other with members spanning between the top chords and between the bottom chords. These are 152x76 channel sections typically spaced 4.8m apart and are bolted to the chords. The chord bracing ultimately anchors to the precast portion of the Bay Walls. See Figure 5-26 for a roof plan showing the truss layout.

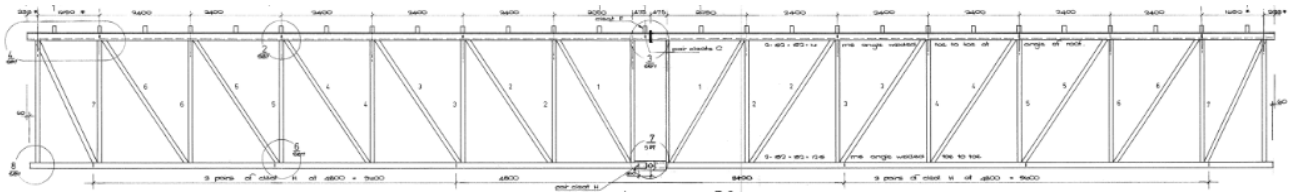

Figure 5-24: Truss T6 elevation, similar to all trusses (drawing S86 of original structural set)

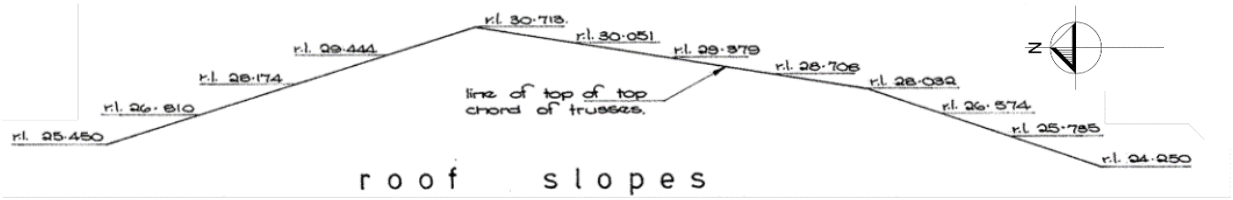

Figure 5-25: Diagram of roof slope (drawing S86 of original structural set)

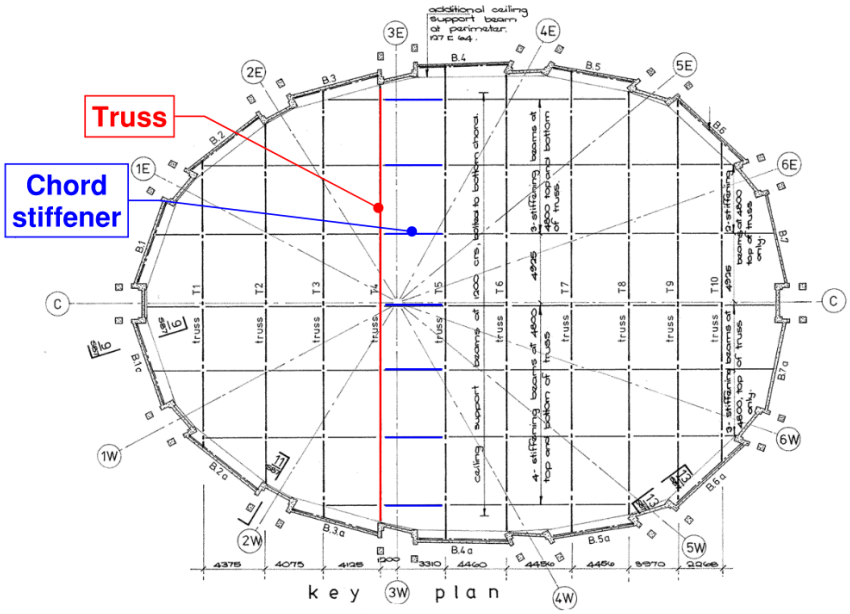

Figure 5-26: Auditorium roof truss plan (drawing S86 of original structural set)

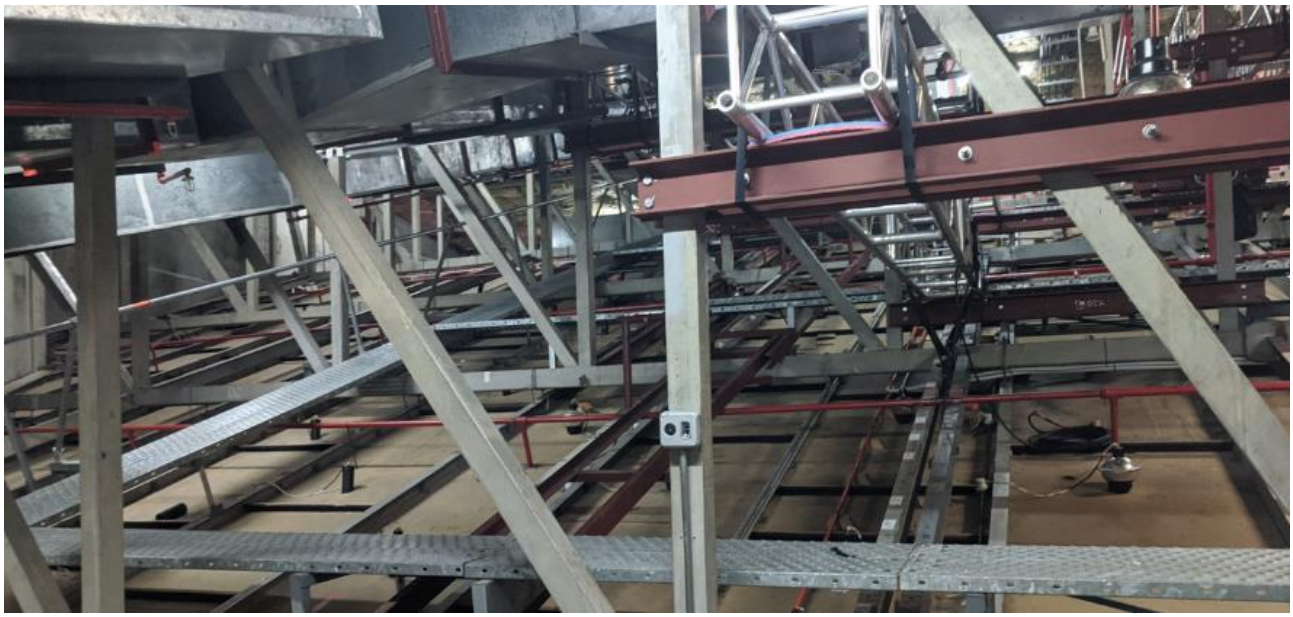

Figure 5-27: Photograph from inside the ceiling space

There is access within the ceiling space through a series of walkways between the roof trusses for maintenance and support of Auditorium activities. Timber acoustic ceiling panels are fixed to the underside of the chords with a custom rail profile. The ceiling space contains a series of mechanical ventilation ducts, a sprinkler system, lighting fixtures, and a rigging system. These are all tied to the roof trusses. See Figure 5-27 showing the mix of elements in the ceiling space.

5.1.2.2 Precast Cantilever Beams

The majority of the building floor weight is hung off the buttresses of the Bay Walls with Precast Cantilever Beams. The beams are shown in Figure 5-8 and in Figure 5-28 below. The beams are precast truss frames which support the Promenades and the Level 4 interior seating. The connecting edge of the beams is C-shaped and caps onto the end of the buttress walls. The main longitudinal reinforcement in the horizontal beam and diagonal strut extends into the length of the buttress wall, forming an integral connection. Figure 5-28 shows the indicative reinforcing placement of the Precast Cantilever Beams connecting to the Bay 4 buttress.

Between story heights, precast columns are similarly C-shaped and cap onto the cast insitu buttresses with small link reinforcing, refer Figure 5-29. These sections may provide a compression load path or some confinement to the cast insitu ends; however, this is expected to be nominal. Their strength and stiffness contribution is generally excluded from the assessment.

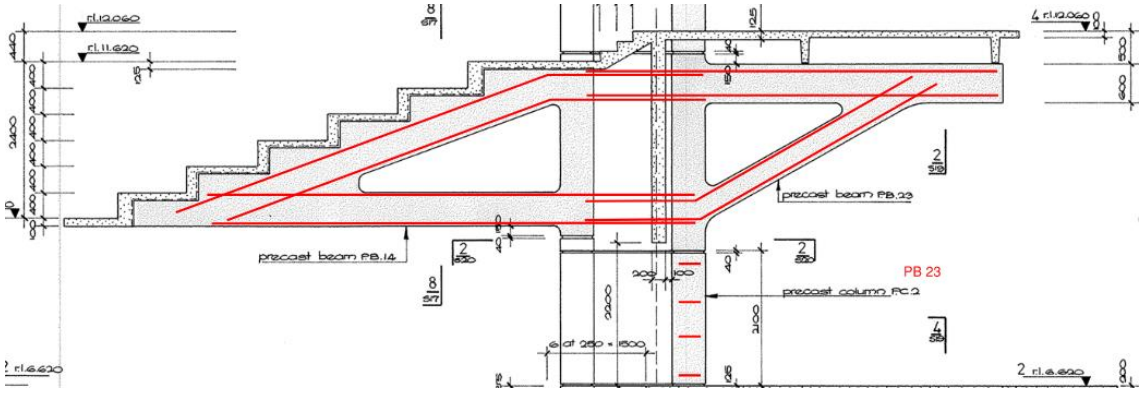

Figure 5-28: Typical Precast Cantilever Beam and column connectivity to buttress wall (drawing S18 of original structural set)

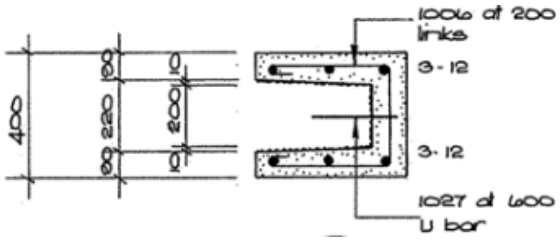

Figure 5-29: Section of precast capping column (drawing S72 of original structural set)

The Level 5 slab is also supported on Precast Cantilever Beams of a different type, see Figure 5-30. These are solid, tapered beams and are also supported off the buttress walls. The top and bottom reinforcing bars have a welded splice to bars extending from the face of the precast buttress. A weld plate occurs in the centre of the beam end for shear support, also shown in Figure 5-30. This end connection was then cast in concrete.

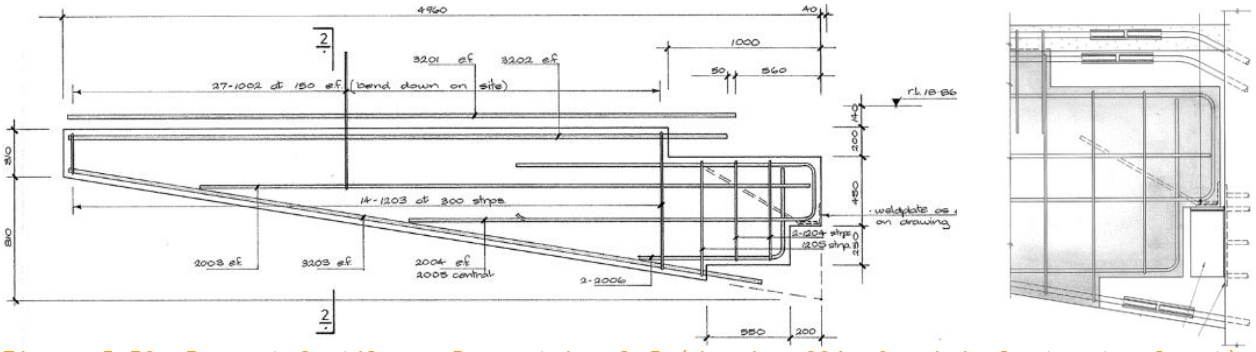

Figure 5-30: Precast Cantilever Beam at Level 5 (drawing S94 of original structural set)

5.1.3 Foundations and Substructure

5.1.3.1 Basement Structure

Below the ground floor, a basement tunnel structure runs in a crucifix shape along the length of the Auditorium and along the width of the stage. Figure 5-31 references the various uses of the tunnel: connecting services throughout the building (green) and housing the supporting structure for the movable stage area (red). The area used for the services in the building connects to a Ø1600mm precast concrete duct (shown in yellow) that runs underneath the Renouf Foyer to connect to main city services. Furthermore, there are two small lift pits, shown in blue.

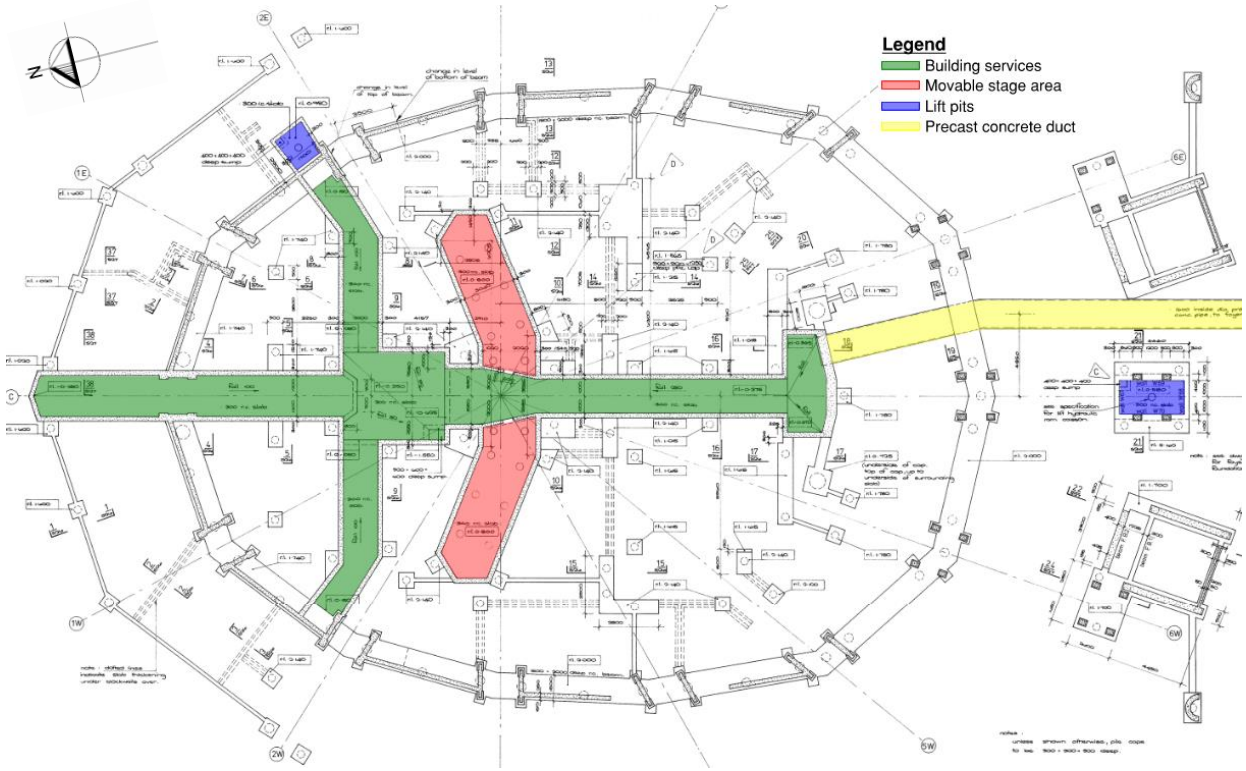

Figure 5-31: Foundation plan highlighting the basement structure (drawing S2 of original structural set)

The basement walls are 300mm thick insitu concrete and floor base slab. Along the length of the basement walls there are several pile caps that tie into the walls and connect them to the piled foundation system. The structural drawings indicate a damp proof course 'moistop' being applied around the base of the

slab-on-grade and the basement structure. There are also waterstop profiles 'hydrofoil' and 'rearguard' applied at the construction joints of the basement structure.

5.1.3.2 Piled Foundation System

The foundation system of the Auditorium is a mix of reinforced concrete beams and spread footings over piles. The piles are Ø450mm or Ø560mm 'Franki' piles, some of which have a double bulb at the bottom, intended to provide additional tension capacity. There are also four Ø900mm bored piles with a belled base. The reinforcing cages vary within pile types, creating seven total variations noted as A-G in Figure 5-32.

The cantilever walls sit on a 1500x2000mm deep reinforced concrete ring beam, see Figure 5-33. This ring beam is interrupted at the northern end when it is bisected by the basement structure. There are three Ø560mm piles under each Bay Wall, one under each buttress and one under the web. Where the cantilever walls are discontinued at Level 2 (Bays 6, 7, 7a, and 6a), they have three grouped piles at the locations under precast column groups or the isolated buttresses.

The Inner Auditorium concrete squat walls are supported by the varying configurations of Ø450mm and Ø560mm piles. The base elevation of the concrete squat walls extends to the bottom of the 900x900 pile caps, creating their own ground beam and rotational fixity in the plane of the wall. The concrete masonry block walls are supported by Ø450mm piles with only shallow strip footings.

The large shear transfer wall, Wall 21, is supported by two Ø900mm bored piles and one Ø450mm pile at the midspan. At the location where the ring beam is bisected there are Ø900mm piles at each end, mirroring this large increase on pile resistance at the north of the building.

The movable stage area and the lifts in the building are supported on caisson piles. These caissons are not considered to contribute to the lateral resistance and are provided to resist operational vertical reactions.

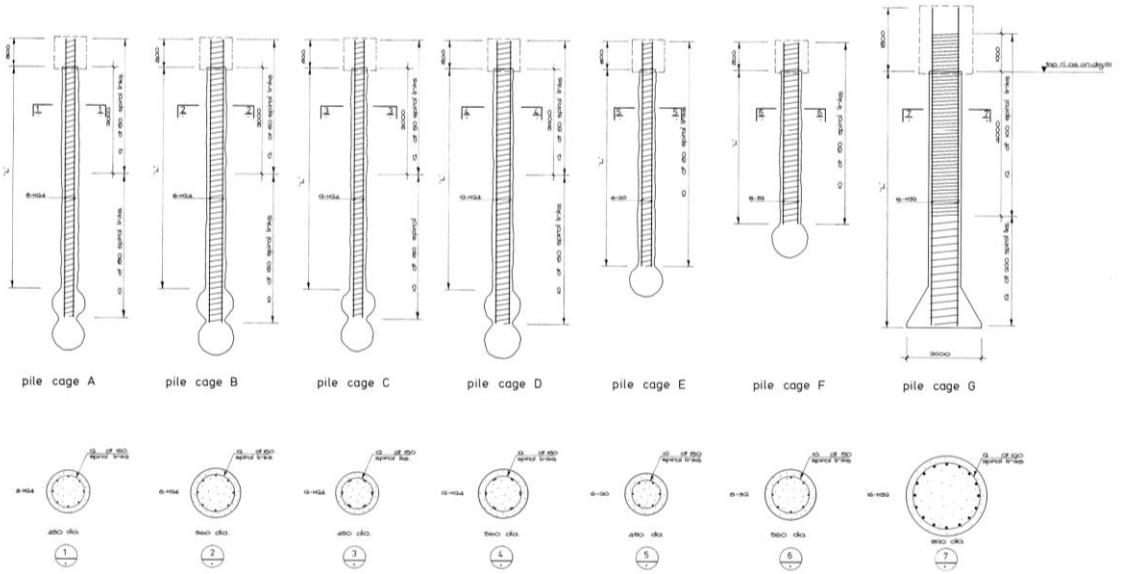

Figure 5-32: Pile types in Auditorium (drawing S29 of original structural set)

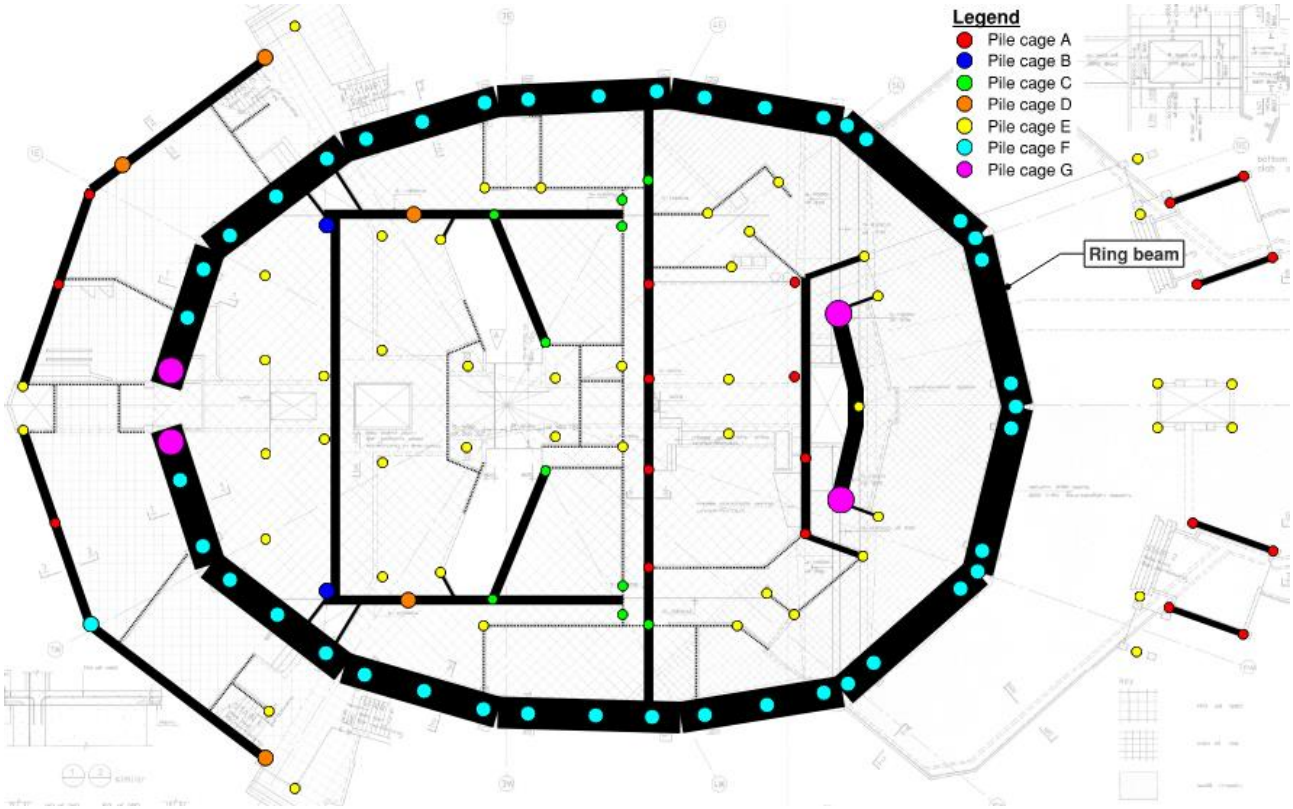

Figure 5-33: Pile plan showing foundation beams and pile types (drawing S30 of original structural set)

5.1.4 Secondary Building Elements

Elements which do not significantly contributed to the lateral load or gravity resisting systems, or which only contribute to the operation of the Auditorium are summarized in this section.

5.1.4.1 Concrete Masonry Infill Block Walls

The concrete masonry block walls located in the Level 1 Inner Auditorium are part of the gravity and lateral resistance of the structure. There are other block walls used as partitions and their locations are summarized in this section. According to the structural specifications, all walls have Class A Vibrapac blocks and typically are reinforced with Ø12mm bars at 600 centres each way. The walls are fully grouted with Mix No.1, with a compressive strength of 12.5 MPa. The vertical reinforcement in the concrete masonry block walls typically protrudes into the floor slabs above and below it. This is also true for these walls classified as “infill”; however, they are determined to not significantly contribute to the stiffness or resistance of the structure.

At the northernmost buttresses of Bay 1 and Bay 1a, an air ventilation shaft is formed by block walls. The walls extend past Level 4 and ties in with the external ‘fins’ of the precast buttresses.

At Level 2, spanning up to Level 4, there are concrete masonry block walls that form service ducts and elevator shafts near the Backstage area. Infill walls 62 and 63 occur at the southern extents of the Backstage area region between Level 2, 3 and 4.

5.1.4.2 Light-weight Roof and Façade System

The building is wrapped in a light-weight stainless steel and glazing façade system which transitions to a timber framed roof above floor Level 4. A typical section of this, approximately at Bay 5, is shown in Figure 5-34.

A 310UC97 structural steel section is hung from the external 'fins' of the precast buttresses, following the elliptical shape around the promenades and the backstage area. This steel section supports 88x350mm glulam timber beams which span to the façade. Here, the glulam beams curve and are bolted to a stainless-steel box mullion, which runs uninterrupted from the promenade roof level down to Level 2. At the Level 2 edge of slab, the stainless-steel mullion is supported on two structural steel box sections.

The glulam beams support a plywood roof with timber joists and a hung ceiling feature. The light-weight roof slopes towards a gutter profile, creating a discontinuity on the plywood diaphragm. As such, the plywood is not continuous across the width of the roof. Where the ceiling is exposed architecturally, the plywood is replaced with timber tongue-and-groove boards following the curved profile of the glulam.

The façade glazing is inset approximately 1m from the structural mullions. Aluminium RHS sections brace the aluminium window frames and glazing back to the mullions. This space also supports a grating panel walkway used for window cleaning.

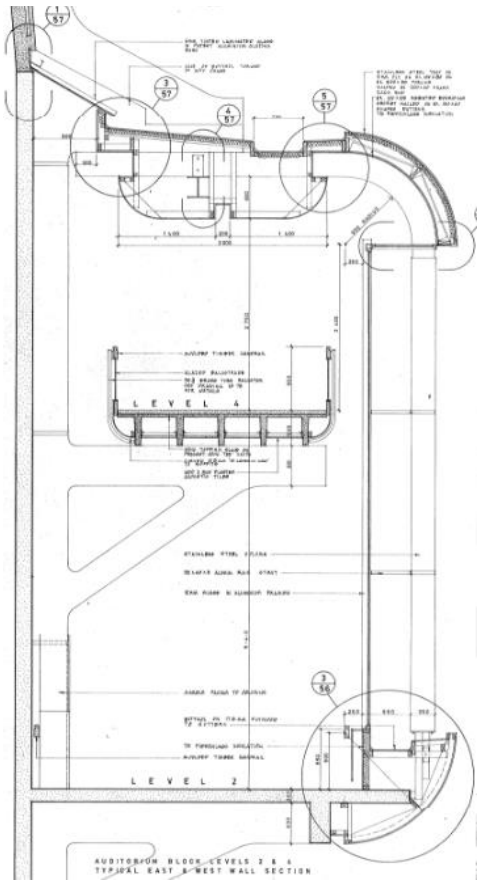

Figure 5-34: Typical section of promenades (drawing 40 of original architectural set)

On the southern end of the Auditorium, the promenade provides access to the independent staircase structures and the Renouf Foyer. Here, the roof glulam beams are continuous, connecting all three structures (see Figure 5-35). At the Auditorium they are supported by the same fin-hung UC as elsewhere in the promenades. A steel frame connects the independent staircase structures and the top of the frame extension beam (Figure 5-36). The glulam beams are hung from this central steel framing and continues to a steel truss at the Renouf Foyer with a similar hanger detail (Figure 5-37).

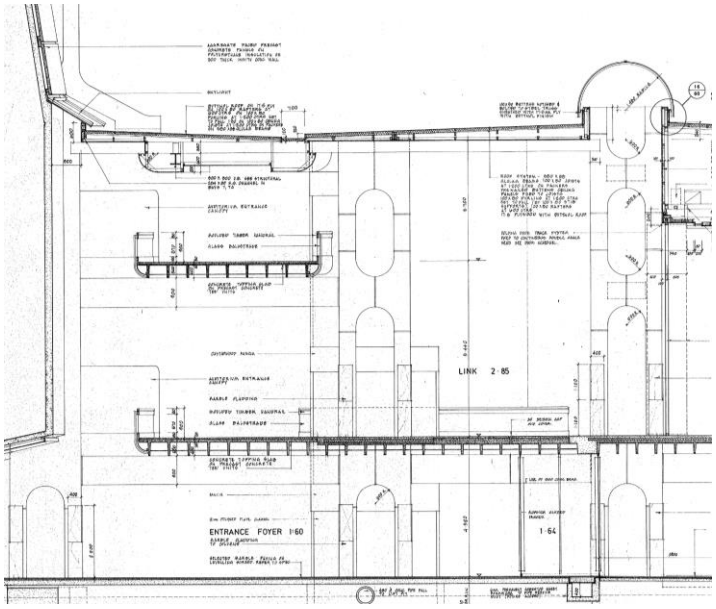

Figure 5-35: Section of the promenade roof spanning over the independent staircase structures and Renouf Foyer (drawing 49 of original architectural set)

The main steel member, SB10 (refer Figure 5-36), is a 254UB31. It is bolted to the top of the central frame extensions of Bays 7 and 7a. The connection to the top of the independent staircase structures is a sliding connection in both principal directions, see Figure 5-37. This assessment assumes the detail is centralised, boxed in by two steel RHS-sections, giving the steel beam 35-40mm of movement in the Auditorium longitudinal direction. In the building transverse direction, there is more than 200mm of seating.

At the Renouf Foyer a steel truss (T3) connects the primary structural frames. This truss is formed out of 102x6.3 SHS sections as the chords and verticals, and with 76x4.0 SHS sections as the diagonals. Figure 5-38 provides the connection detail of the truss to the Renouf Foyer frames. A 152x102x6.3 RHS cantilevers from a cast-in weld plate and provides seating for the steel truss. There is no direct connection from this cantilevered stub to the truss. This assessment assumes the detail is centralised as drawn and allows approximately 95-100mm of movement along the length of the truss (Auditorium transverse), and 50-55mm movement perpendicular to the truss (Auditorium longitudinal). These two sliding details, along with the glulam hangers, allows for the independent movement of the primary roof framing all four structures. Secondary element detailing such as continuous the glulam beams and façade supports, cross over these points of relative flexibility in the primary steel framing.

SB12 is a PFC305 and provides support of the facade between the Auditorium and the Renouf Foyer. At the Auditorium frame extension, there is a cast-in weld plate from which the channel is anchored. At the Renouf Foyer, the channel is hung from the truss (T3) with a 76x4.0 SHS. This hanger is relatively flexible along the axis of the steel channel, which allows for some independent movement between the Auditorium and the Renouf Foyer. Perpendicular to SB12 is a second diagonal SHS member bracing it to the truss, and any movement allowance comes from the detail in Figure 5-38.

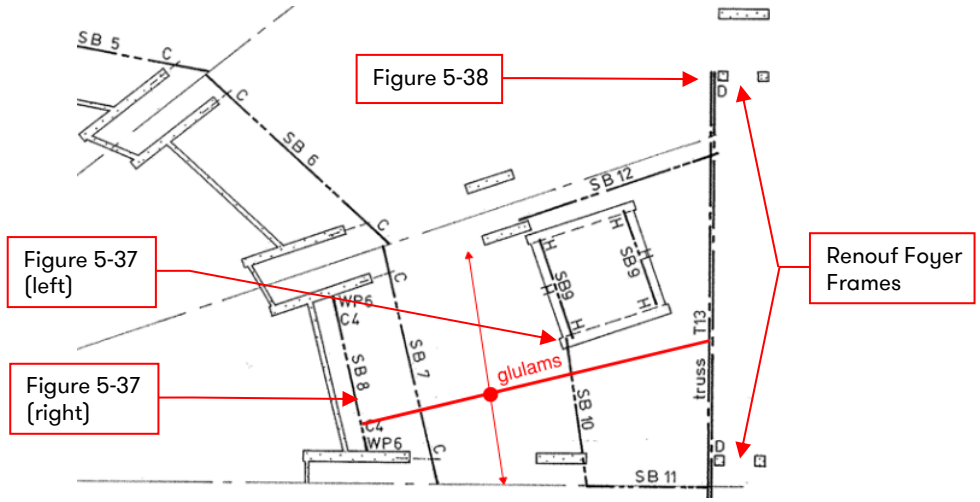

Figure 5-36: Partial plan of lobby roof showing roof framing (drawing S89 of original structural set)

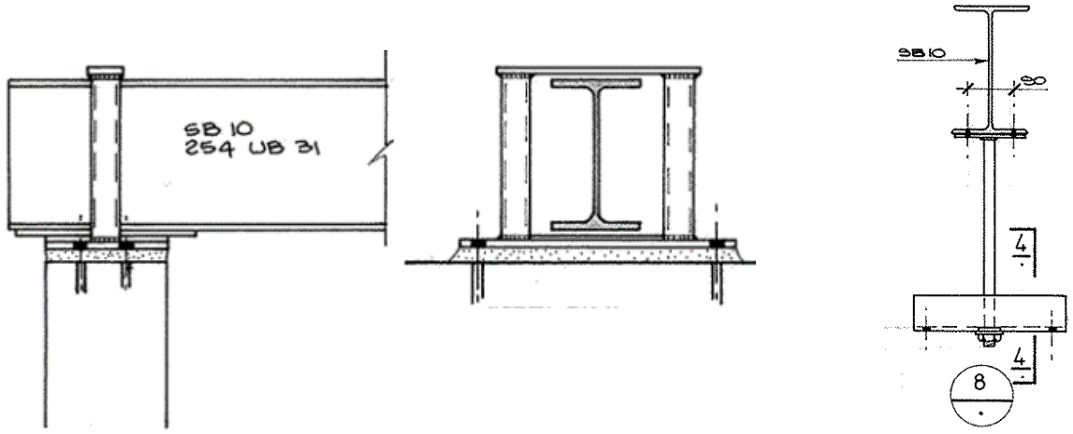

Figure 5-37: Detail of sliding connection at roof of the independent staircase structures (left) and glulam hanger connection (right) (drawing S91 of original structural set)

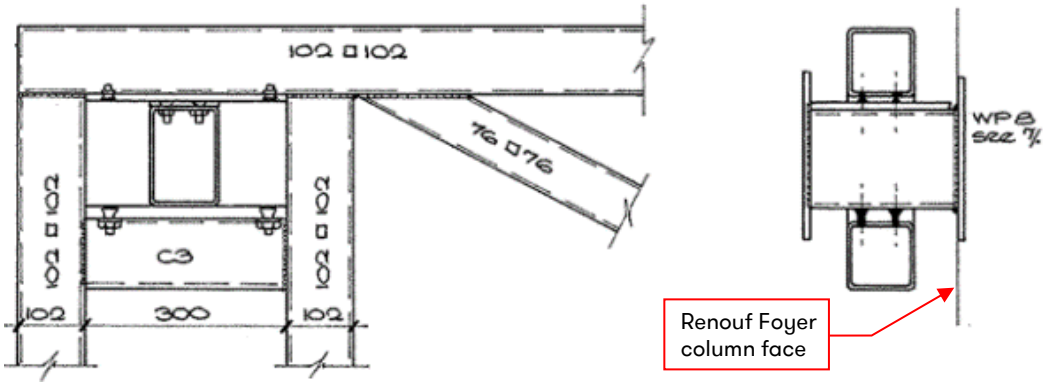

Figure 5-38: Detail of sliding connection of the structural steel truss at the Renouf Foyer (drawing S91 of original structural set)

5.1.4.3 Stairs

The Auditorium has a range of stairs connecting the different floor levels of the building. They all are cast insitu reinforced concrete elements. Figure 5-39 shows all the staircases at the Michael Fowler Centre, including those at the Renouf Foyer and independent staircase structures, at the various floor levels. The stairs are mirrored along the central axis of the building.

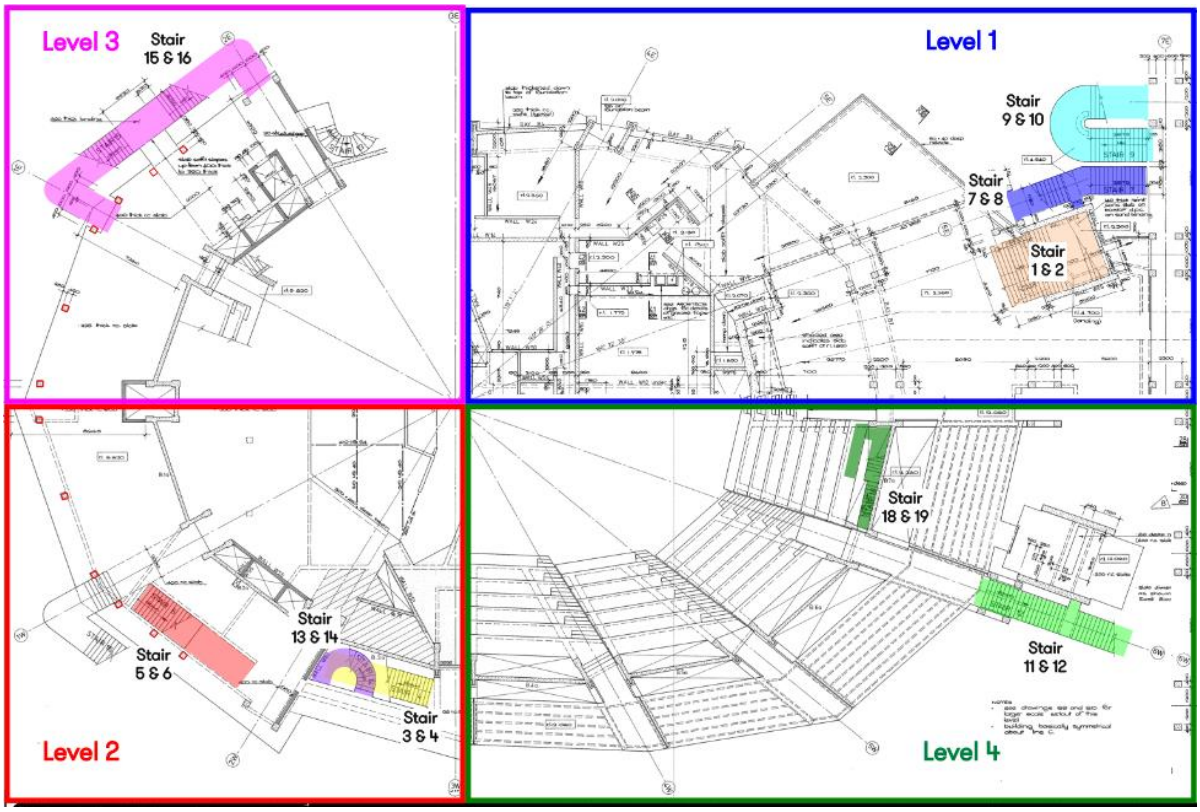

Figure 5-39: Combined floor plans showing all stairs on the MFC site

Stairs 1 & 2 are shown at Level 1 as brown. They are part of the independent staircase structures and are further discussed in Section 5.3.

Stairs 3 & 4 are shown at Level 2 as yellow. They run from the Backstage area at ground floor to the promenade structure at Level 2. They primarily cantilever from the face of the Bay Wall at Bays 3 and 3a. Reinforcing is also provided along the span, tying it to the top and bottom slab with no sliding connection.

Stairs 5 & 6 are shown at Level 2 in red. They run from the Backstage area from the ground floor to Level 2. They primarily cantilever from the face of Wall 1. Reinforcing is also provided along the span, tying it to the top and bottom slab with no sliding connection.

Stairs 7 & 8 are shown at Level 1 in dark blue. They are located outside of the building, primarily used as emergency egress routes. They run from the ground floor level underneath the Renouf Foyer up to Level 2 at the Auditorium. They have an intermediate support which cantilevers from the independent staircase structures and an independent footing at the base. They primarily act as simply supported spans between landing levels. The support at the mid-height landing was modified in 2016, which is further discussed in Section 5.1.5. In principle, the alterations formed a sliding joint to seismically separate the independent staircase structures from the Auditorium.

Level 2, Chartered Accountants House, 50 Customhouse Quay
PO Box 942
Wellington 6140
holmesanz.com

Stairs 9 & 10 are shown at Level 1 in light blue. They are located outside of the building, primarily used as emergency egress routes. They run from the ground floor level underneath the Renouf Foyer to a mid-height landing and return to Level 2 at the Renouf Foyer. The stairs primarily as simply supported structures between the floor levels and the mid-height landing which is supported by a column. The column has a pad footing which is tied back to the independent stair core foundation. The flights of stairs originally had no sliding connections and were cast integral with the floor levels and the landing. However, the modifications in 2016 created a sliding joint, to allow the Renouf Foyer to drift independently of these stairs.

Stairs 11 & 12 are shown at Level 4 in light green. They are located alongside the independent staircase structures and run from the Level 2 Renouf Foyer to Level 4 of the Auditorium, with a mid-height landing cantilevered from the independent staircase structures. They primarily act as simply supported spans between landing levels. There are sliding joints between the upper flight of stairs at the mid-height landings, and the bottom flight at the Level 2 Renouf Foyer. Therefore, the Auditorium, independent staircase structures, and the Renouf Foyer are seismically separated. Where the upper flight follows the movement of the Auditorium, and the lower flight follows the movement of the independent staircase structures.

Stairs 13 & 14 are shown at Level 2 in light purple. They are a half circle-shaped stair connecting the Level 2 promenade to the Backstage area at Level 3. The stair runs are primarily simply supported and are cast integral with Level 2 and Level 3.

Stairs 15 & 16 are shown at Level 3 in pink. They are located on the Eastern and Western ends of the Backstage area. They run between cantilevering landings from Level 2 to Level 3 and to Level 4. The stair flights are primarily simply supported between these landing. The flights of the stairs have no sliding connections and were remediated in 2016 with a steel catch-frame. This work is further described in Section 5.1.5 and intends to support the flights of stairs, should they break off in a seismic event.

Stairs 17 & 18 are shown at Level 4 in dark green. The stairs are in the Inner Auditorium and lead up to the Level 5 slab from Level 4. They primarily cantilever from the face of the Bay Wall at Bays 7 and 7a. Reinforcing is also provided along the span, tying it to the top and bottom slab with no sliding connection.

5.1.4.4 Catwalks, Reflectors, and Donut

On the inside of the Auditorium is a network of catwalks and two types of acoustic reflectors that are hung from the Bay Walls and roof trusses. The lower reflectors are located just above the stepped seating at Level 4 and hung from the Bay Walls alone. Independent from these reflectors are upper reflectors hung from the Bay Walls and trusses. The catwalks are accessible from Level 5 and supported within the upper reflector structure.

The primary piece of permanent rigging in the Auditorium is an octagonal element clad with timber panels and referred to as the 'Donut'.

Lower Reflectors

The lower reflectors have a steel structure of 75x50x4 hollow sections and clad with timber panels. The base is cantilevered from the Bay Wall and the top edge is tied back with rod hangers (Figure 5-40).

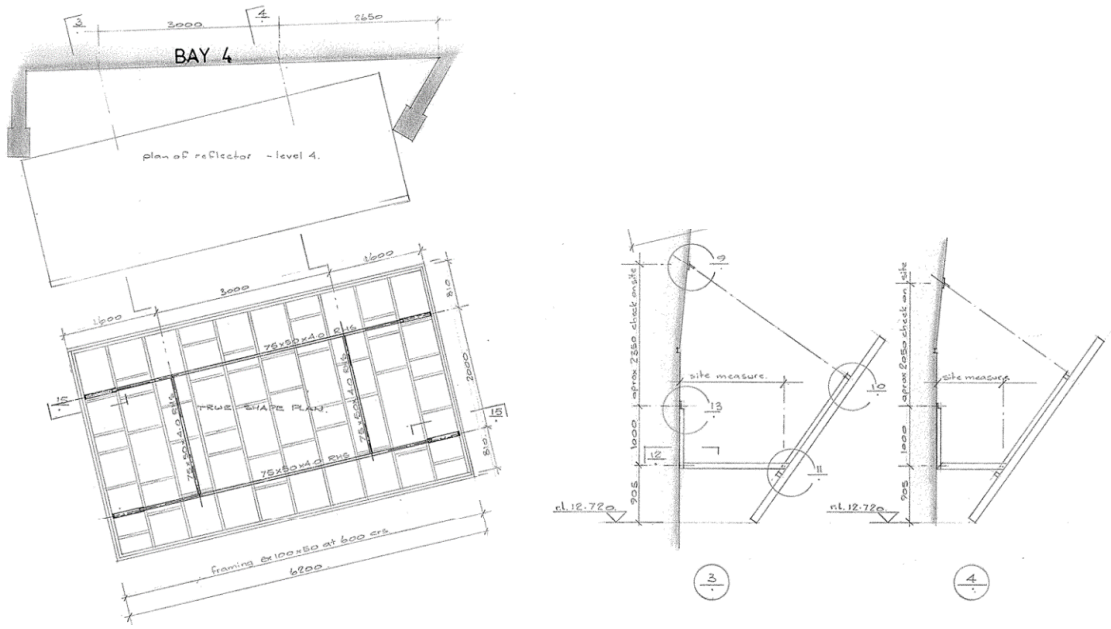

Figure 5-40: Details of lower reflector frame (drawing S99 of original structural set)

Catwalk and upper reflectors

The upper reflectors are timber framed elements attached to a system of RHS and SHS beams which are hung from a 250UB and two 20mm rods (Figure 5-41 left). The 250UB spans from the Bay Wall connection of post installed mechanical anchors to diagonal 20mm tension rods generally hung off the roof truss bottom chord.

The catwalks are located behind the reflectors and hung with two rods from the 150UB. Along the axis of the catwalks, the reflector frames are connected with tube and channel sections.

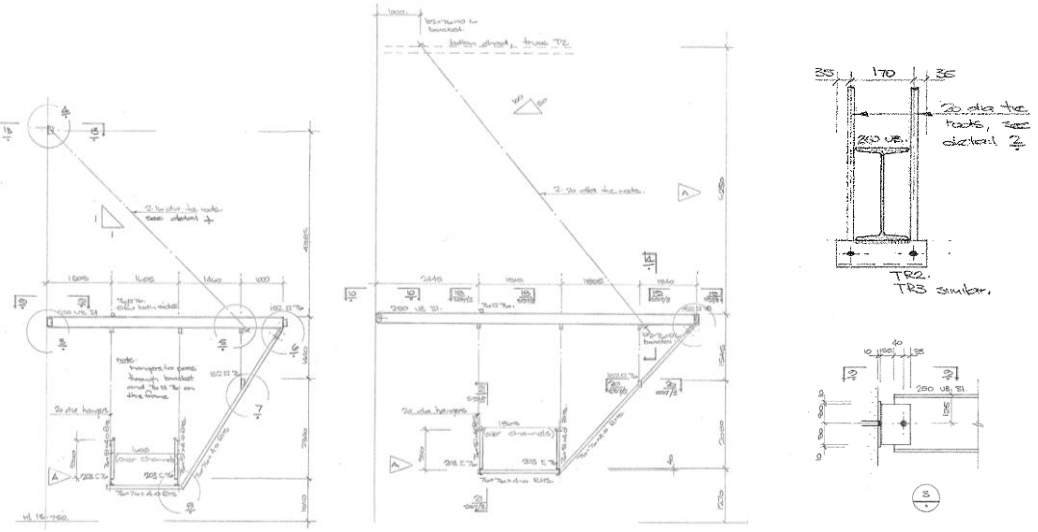

Figure 5-41: Section (left) and details (right) of typical upper reflector frames (drawing S97/2 of original structural set)

Donut

In the centre of the Auditorium over the stage is a 'Donut' shaped reflector. It is hung through a series of steel cables which are connected to proprietary F44P square rigging trusses. This rigging is then tied to a system of channel sections bolted to the roof trusses.

5.1.5 Alterations to the Existing Structure of the Auditorium

There were local alterations made to the structural elements of the Auditorium in 2016, 2020, and 2022. These changes influence the seismic performance of the building and are further described in this section.

Stairs (2016)

Following the seismic assessment in 2014, issues were raised with the lack of deformation compatibility around the stairs. These were constructed integrally with the floor levels at both ends of the flights, meaning that they would act as a diagonal brace when the two levels move relative to each other. This would likely lead to significant damage in a seismic event and alterations were done at Stairs 7 & 8, 9 & 10, and 15 & 16. Further description of these stairs is provided in Section 5.1.4.3.

For Stairs 7 & 8 new supporting structures were built next to the mid-height landings. This provided additional seating once a saw cut was added to the upper flight. The new supporting structures were founded on small foundation beams and tied into the mid-height landings for lateral support.

Stairs 9 & 10 received saw-cuts at the Level 2 slab interface and new steel support frames were provided to increase the seating of the stair flight. This way, the Renouf Foyer can drift independently of these stairs.

No sliding connections were added at Stairs 15 & 16. Instead, steel catch-frames were provided underneath the flights of stairs, see Figure 5-42. The intent of the frames is to catch the flights should they break off from the landings and lose support in a seismic event. The cantilevered concrete landings extending from Levels 3 and 4 were supplemented with steel plates and dowels to increase the cantilevering capacity. At Level 2, the landing was captured by a steel structure hanging from the Level 3 slab above.

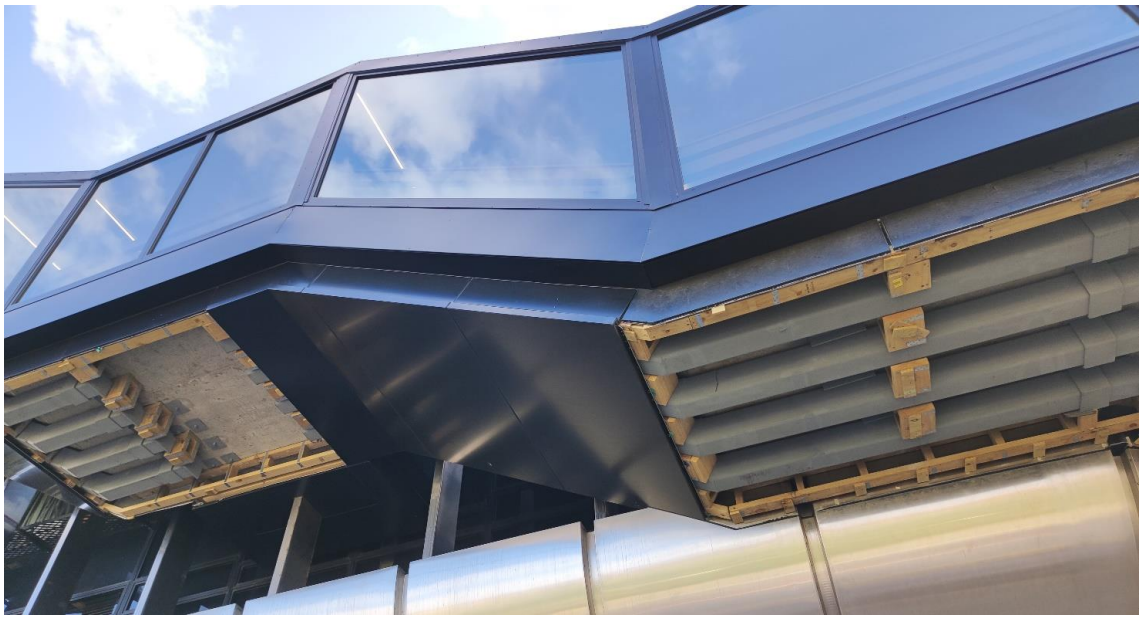

Figure 5-42: Photo of the securing works under Stairs 15 (Stair 16 similar)

Seating of Precast Units (2019)

There were various securing works added to the precast roof system in 2019. These included providing additional seating to the Dycore units and addressing potential issues with 'alpha' slabs, torsion, and negative moment failure in the northern and southern ends of the Auditorium.

There were also improvements made to the double-tee units in the Auditorium and Renouf Foyer and rib-and-timber infill units in the Renouf Foyer. A typical detail used as part of these securing works is shown in Figure 5-43.

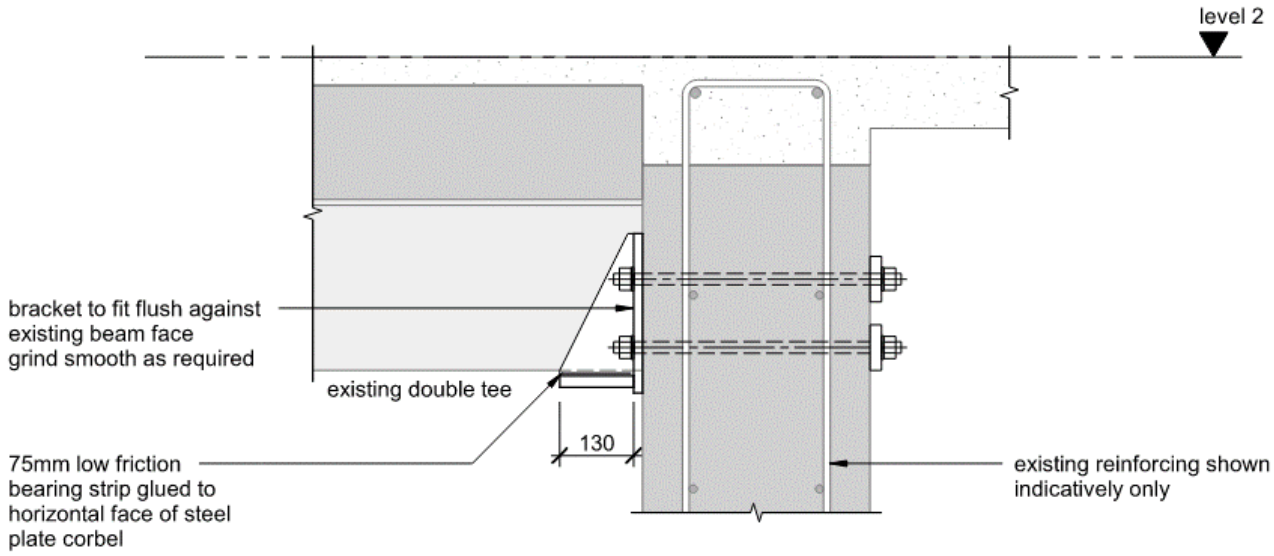

Figure 5-43: Typical detail providing additional seating to the flange-hung double-tee units

Envelope Improvement of Stairs 15 & 16 (2021)

The waterproofing envelope around Stairs 15 & 16 required replacement. Structurally, the existing timber roof was replaced with a lightweight metal deck. The new roof built-up has a similar weight to the existing and the existing lateral and vertical load paths were maintained. No seismic strengthening was triggered as part of this scope of work.

During construction of this roof replacement work, it was observed that the stair roof connections back to the Auditorium differed from the original drawings and represented an unreliable load path. Various site sketches and instructions were issued to remediate these details (refer Figure 5-44) in an attempt to strengthen the area ahead of this Detailed Seismic Assessment.

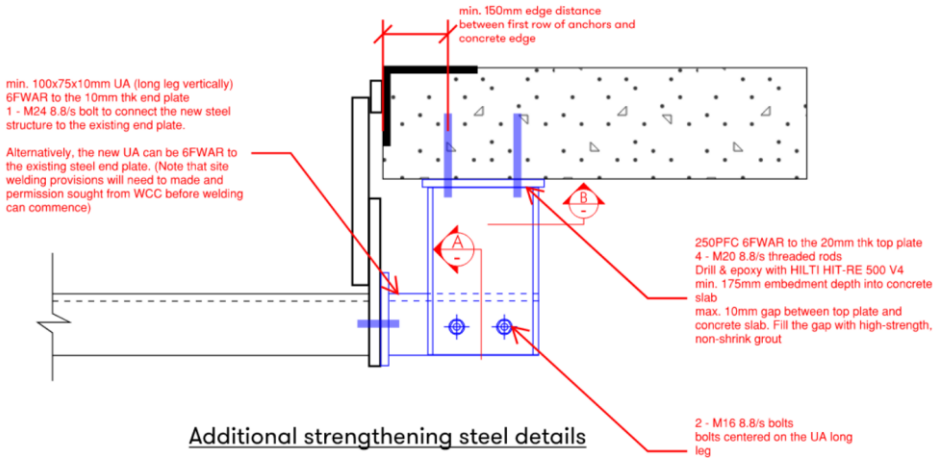

Figure 5-44: Stair 15 & 16 construction monitoring strengthening detail

Catwalks, wall reflectors, and 'donut' (overhead stage reflector)

During visits to the site, variations from the original drawings were seen in the catwalk structure. There was no formal documentation available for these alterations. The catwalk has been assessed in consultation with the original drawings and the modifications observed on site. Some of the alterations identified are:

- On the eastern reflectors, there are two sets of X-configuration cables in plan (Figure 5-45, left) rather than two diagonal SHS braces as seen in the drawings and the western set of reflectors (Figure 5-45, right).
- The two vertical SHS braces are replaced with two cable braces, on both the east and west reflectors.
- Some of the vertical rods have a welded splice connection.

Figure 5-45: Photos of upper reflector and catwalk structure

The 'Donut' was designed to be hung by four 8mm cables at an angle from the roof trusses. Over the course of reviewing the loading on the Auditorium roof truss through 2018-2020, a few alterations were made to the Donut and its support to the truss. The key alterations are:

- The Donut has been equipped with new larger cables which are now hung vertically.
- The Donut is now supported on proprietary F44P trusses instead of directly loading the bottom chord of the auditorium truss.

5.2 Description of the Renouf Foyer

The Renouf Foyer is a two-storey, rectangular structure located on the southern end of the site with floor dimensions of approximately 45.0x18.0m. The structure has a floor-to-floor height of 4.62m between Level 1 and Level 2. The double-height frames span another 8.1m upwards to the centreline of the uppermost coupling beams which are aligned with the roof trusses.

The ground floor provides an external pick-up and drop-off facility while the upper floor primarily consists of an open plan, double-height space (refer Figure 5-46). The upper level of the Renouf Foyer is clad with a large curtain wall consisting of glulam mullions which are curved at roof. The Renouf Foyer is seismically separated from the Auditorium with functional linkages via a suspended floor area above ground level. A series of stair structures either span through or are externally supported by the Independent Staircase Structures.

The Renouf Foyer saw an extension in 1989, adding an additional suspended floor area, referred to as the 'kitchen extension' and floor dimensions of approximately 10.0x21.0m. An additional frame bay was added to the south of four central transverse frames in the Renouf Foyer. These frames span over the basement substructure.

Figure 5-46: Picture taken inside the Renouf Foyer at floor Level 2 (looking East).

5.2.1 Lateral Load Resisting System (LLRS)

The lateral load-resisting system of the Renouf Foyer consists of a two-way concrete moment frame system, refer Figure 5-47. The precast coupled column units form single-bay moment frames spanning ~10.0m in the transverse (north-south) direction. In the longitudinal (east-west) direction, adjacent sets of coupled columns are connected via concrete coupling beams to provide frame action within each precast coupled column group. Each group of four columns, is not connected to the adjacent precast column group.

The kitchen extension frames are precast concrete beams extending from the central frames of the Renouf Foyer in the transverse direction. In the longitudinal direction no additional vertical lateral framing was added. The precast floor units act as a laterally cantilevering diaphragm.

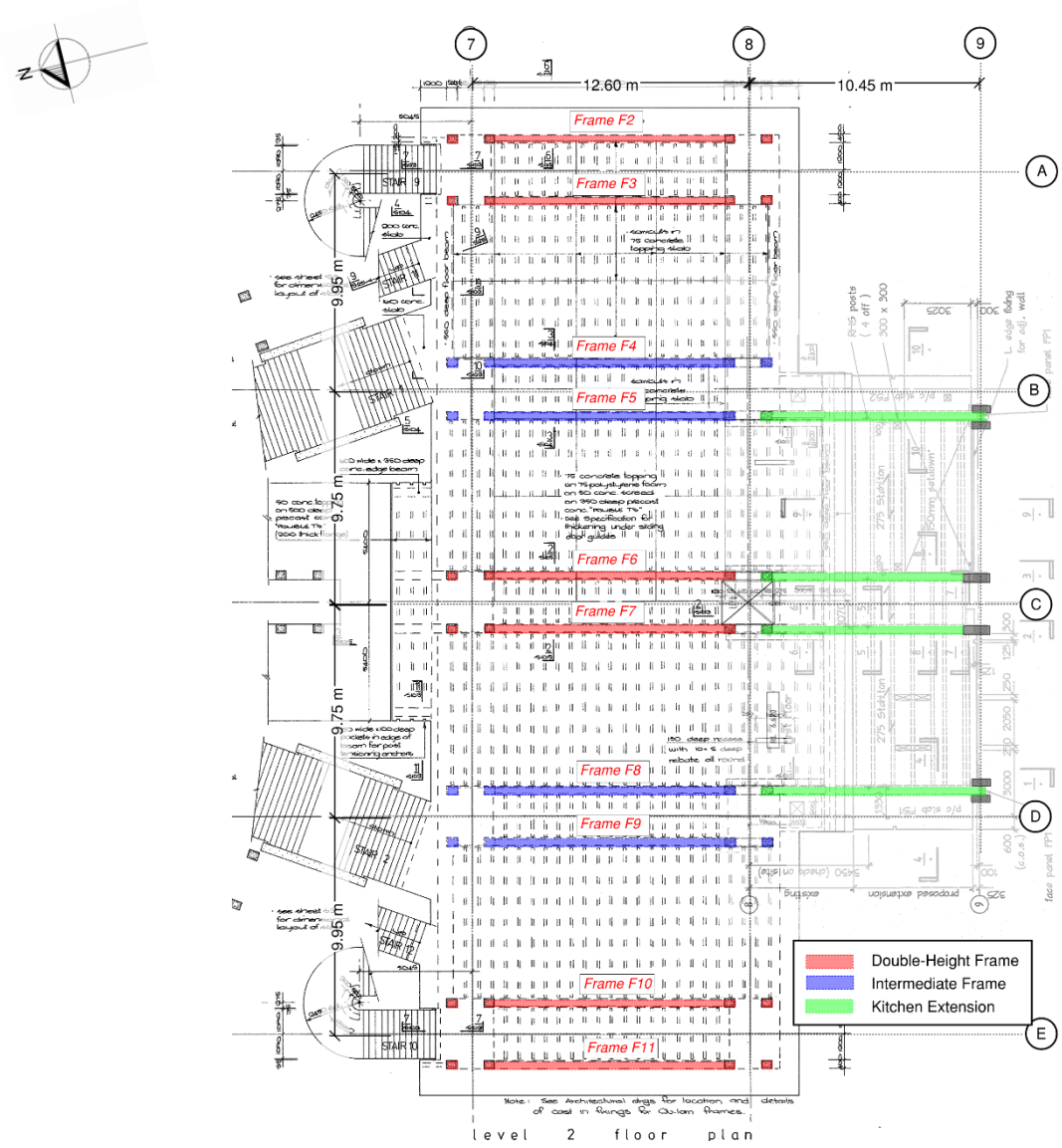

Figure 5-47: Level 2 plan highlighting the transverse frames layout within the Renouf Foyer (drawing S102 of original 1980's structural set supplemented with the drawings of the 1989 kitchen extension)

5.2.1.1 Precast Concrete Frame System

The primary supporting structure of the Renouf Foyer consists of a series of precast coupled column units. In the building transverse direction, beams frame into the coupled columns to form a frame at Level 2; above Level 2 the column pairs act as cantilevers. Some transverse frames only support the suspended floor at Level 2 and do not continue to the roof, these are referred to as the 'intermediate frames.' Other precast coupled column units span past Level 2 to support the roof structure, these are referred to as the 'double-height frames'. Refer to Figure 5-47 for the location of each frame type and Figure 5-49 for elevations of each. They feature arched coupling beam elements spanning between column pairs, similar to the frame extensions in the Auditorium. An image of the double height frames is shown in Figure 5-48.

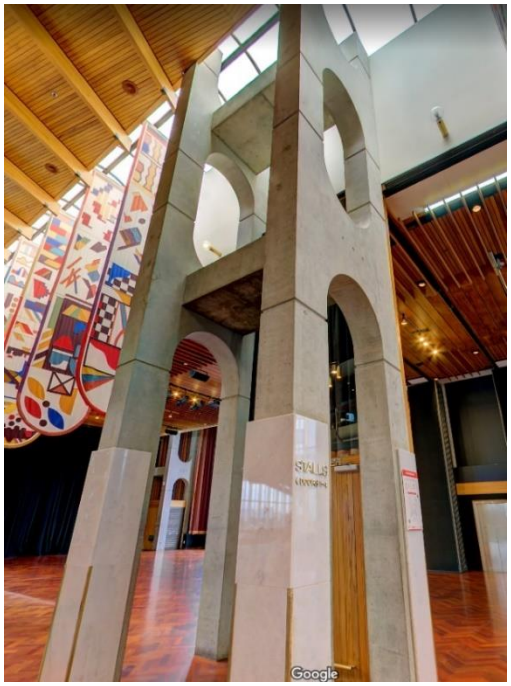

Figure 5-48: Picture of precast coupled double height frames in the Renouf Foyer

In the longitudinal direction, there are cast insitu link elements which connect two sets of two closely spaced double-height frames. This forms a four-column group above Level 2 with six such column groups in the Renouf Foyer. Typical double-height precast columns have three link slab elements between Level 2 and the roof. At Level 2, the precast floors become a deep cast insitu slab section to couple column forces in this direction. This is in contrast to the discrete link elements above.

The kitchen extension consists of two typical types in the transverse direction, refer Figure 5-47. Central frames (at gridline C) have a traditional beam-column connection to the kitchen extension columns where the beam is in the same plane as the column. The end frames (at gridlines B and D) use a slotted beam-column connection where the beam is encapsulated between a column on either face of the beam. No beam element was created on gridline 9 to create a longitudinal frame for this kitchen extension.

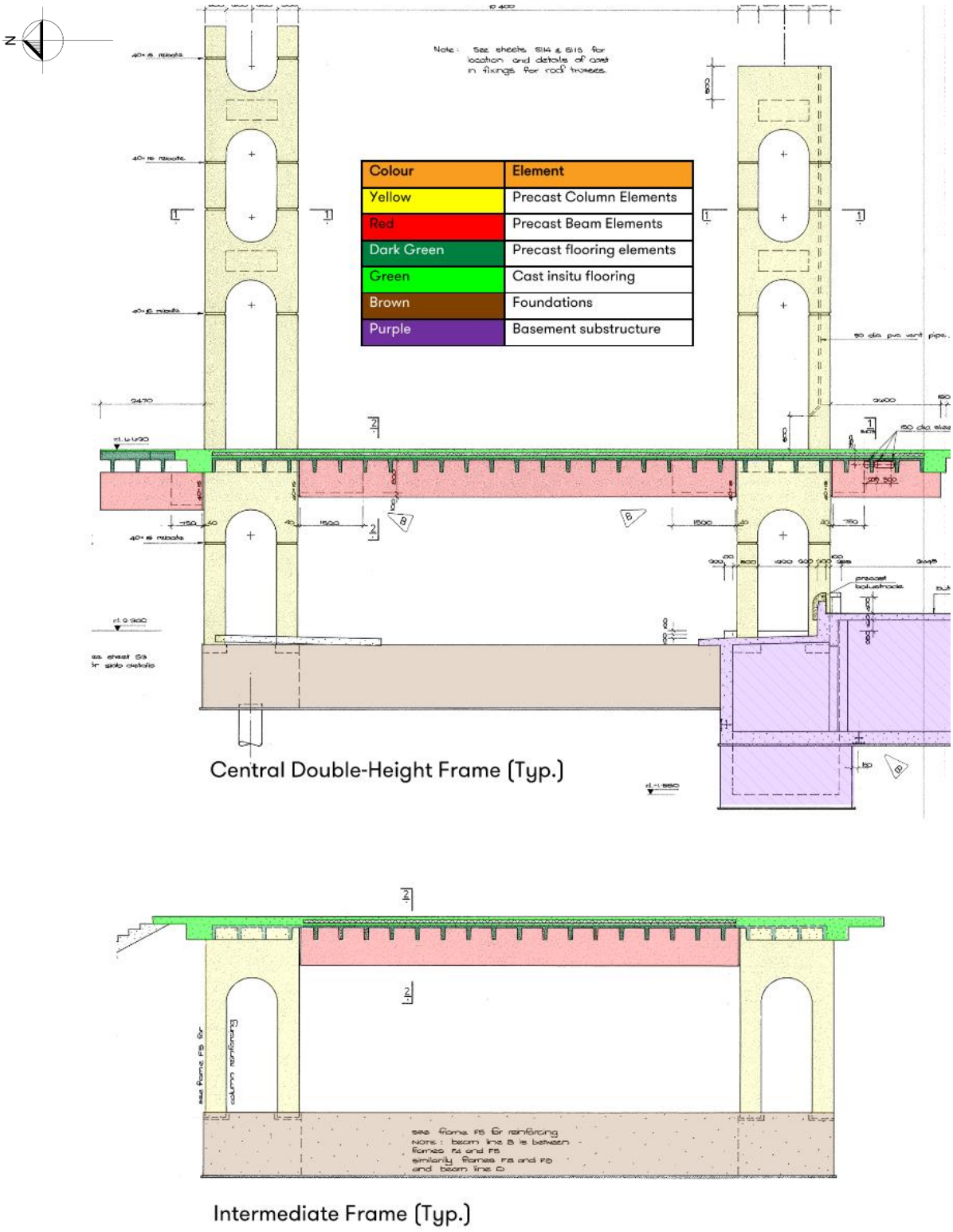

Figure 5-49: Elevations of transverse frames in the Renouf Foyer (drawing S105 of original structural set)

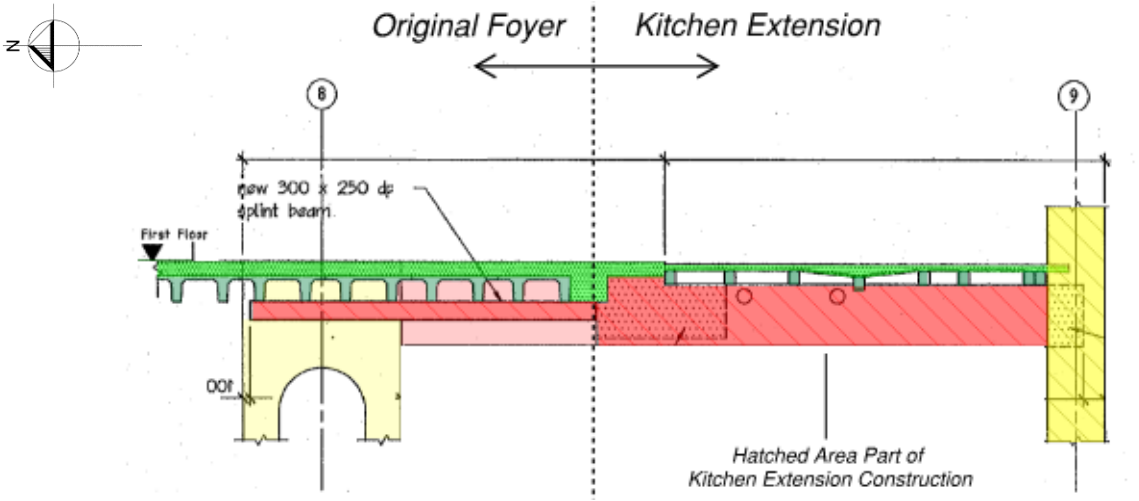

Figure 5-50: Section of added kitchen extension frame (drawing W1748-S2-1)

5.2.1.2 Floor Diaphragms

The single suspended floor level in the Renouf Foyer at Level 2 spans east to west and typically consists of precast double tee flooring units with an insitu concrete topping. The insitu concrete topping is reinforced with cold-drawn (non-ductile) mesh. There are also cast insitu slab edge beams around the perimeter of the floor plan.

The kitchen extension suspended floor is constructed with a precast rib and timber infill flooring system with an insitu concrete topping. The insitu concrete topping is reinforced with cold drawn mesh with saddle bars over the supporting precast beams. The kitchen extension slab also has precast flat slabs cantilevered from the eastern and western most frames.

The suspended floor uses a double-tee pre-cast flooring system that spans in the East-West direction between the supporting beams. The kitchen extension uses a precast concrete rib flooring system with timber infills. The precast flooring units are connected through a concrete topping reinforced with cold-drawn mesh and insitu edge beams around the perimeter of the Renouf Foyer.

5.2.2 Gravity Load Resisting System

The majority of structural elements act as part of both the lateral load and gravity resisting system, including the precast frames and floor structure. Elements in this section are predominantly gravity-only elements, not discussed in Section 5.2.1.

5.2.2.1 Roof Structure

The roof structure consists of a series of steel roof trusses which span between the Renouf Foyer columns in both principal directions of the structure refer (Figure 5-51).

The roof trusses are typically constructed using square hollow-sections (SHS) ranging from 50 to 100mm in width, with a total truss depth of approximately 2.0 m. The transverse trusses (trusses 1 to 4), at gridlines A, C and E, connect directly to the columns. The longitudinal trusses (trusses 5 to 12) on gridlines 7 and 8 are supported by trusses 1 to 4.

The roof trusses support a lightweight roof system on the top chord consisting of plywood over 'Brownbuilt' cold-formed purlins and timber battens. Within the depth of the roof trusses sits a service space including two plant rooms supported by trusses 7/8 and 9/10. The bottom chord of the roof trusses also supports a timber batten suspended ceiling. A typical section through the roof space of the Renouf Foyer is shown in Figure 5-52.

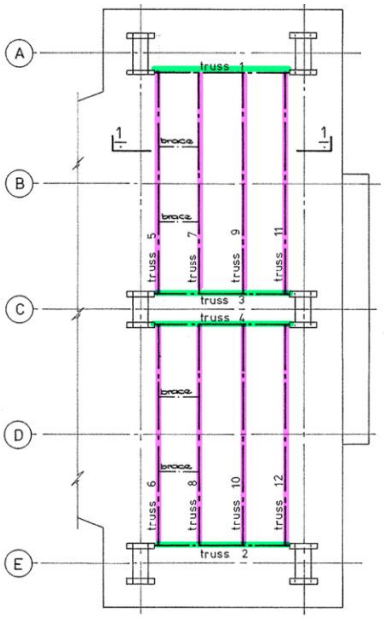

Figure 5-51: Roof plan of truss layout (drawing S114 of original structural set)

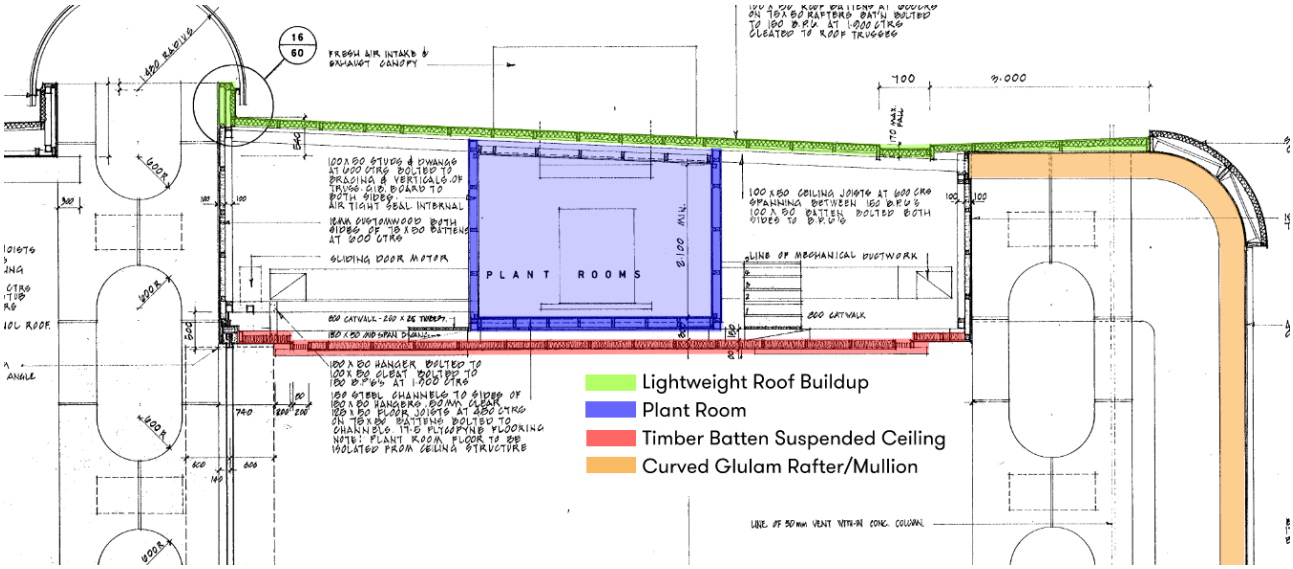

Figure 5-52: Section of Renouf Foyer roof space (looking east) (drawing 49 of original architectural set)

5.2.3 Foundations and Substructure

5.2.3.1 Basement and Ground Slab

A basement substructure is located underneath the kitchen extension on the south side of the Renouf Foyer. The basement is fully enclosed with a series of piles supporting a cast insitu slab. Cast insitu walls act as propped retaining walls with a cast insitu concrete roof slab which connect to the ground floor level.

The basement structure is accessed through a ramp on the south-east side of the Renouf Foyer. The basement also houses a service lift which sits between one of the precast coupled column units and runs into a lift pit set down from the basement structure itself. As part of the kitchen extension, precast columns were connected to the basement structure either directly or via additional foundation beams aligned with the south wall of the basement.

The Renouf Foyer does not have a full ground floor slab as there is road access running beneath the foyer between frames. Instead, there are two sections of 150 mm cast insitu slab either side of the road spanning from gridlines B to D, shown in green on Figure 5-53.

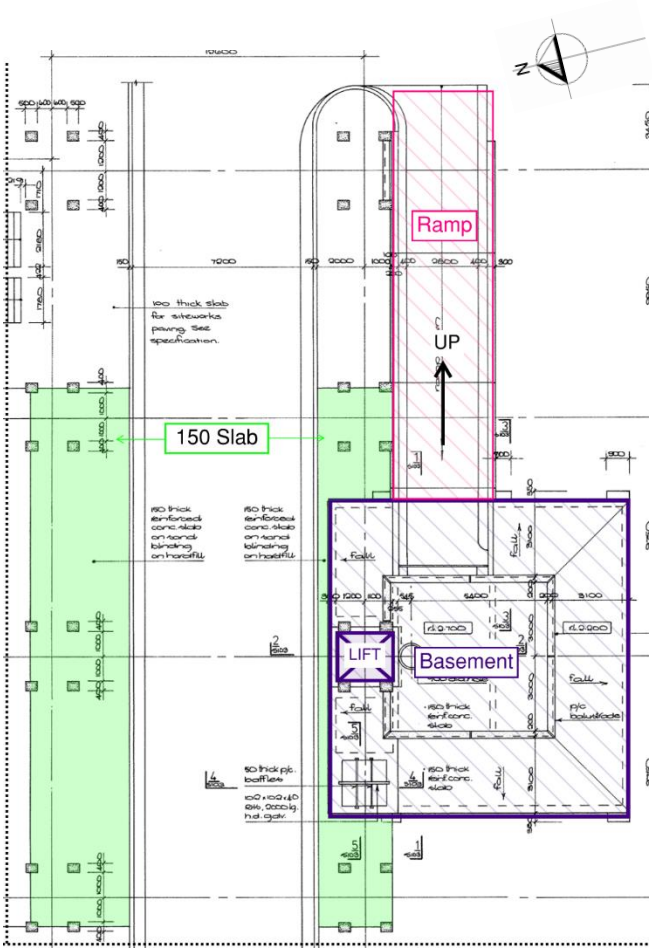

Figure 5-53: Ground floor plan showing the basement and ground slab layout (drawing S102 of original structural set)

- For the kitchen extension, the central frame columns are founded on the wall of the existing basement structure. The end frame columns are supported by a single pile and tied back to the basement structure through foundation beams in the East-West direction.

5.2.3.2 Piled Foundation System

The foundation system of the Renouf Foyer consists of a series of foundation beams between precast concrete column units, supported on piles. The kitchen extension utilises a similar combination of foundation beams and piles.

At the Eastern and Western end, four sets of precast concrete column groups are tied together by four foundation beams. At the central column groups, at gridline C, the southern end lands on the basement structure and the northern end has an isolated pile cap tied back to the basement. This creates three 'islands' of foundations not tied to each other.

The pile types under the Renouf Foyer are the same as the Auditorium (Section 5.1.3.2). Under each group of four columns is a single 560mm pile with ple cage B. Under the basement structure are 450mm piles with pile cage E.

The lift between the central frames is supported by a caisson pile.

5.2.4 Stair Structures

Eight stairs in total connect to the Renouf Foyer (Stairs 1-2 and 7-12). For a detailed description of the staircase structures connecting to the Renouf Foyer, see Auditorium Section 5.1.4.3.

5.2.5 Alterations to the Existing Structure of the Renouf Foyer

Seating of double-tee units

Securing works were designed and constructed in 2019 to provide additional seating for the flange-hung double-tee units and the rib infill floors. These are similar to the Auditorium work and are presented in Section 5.1.5.

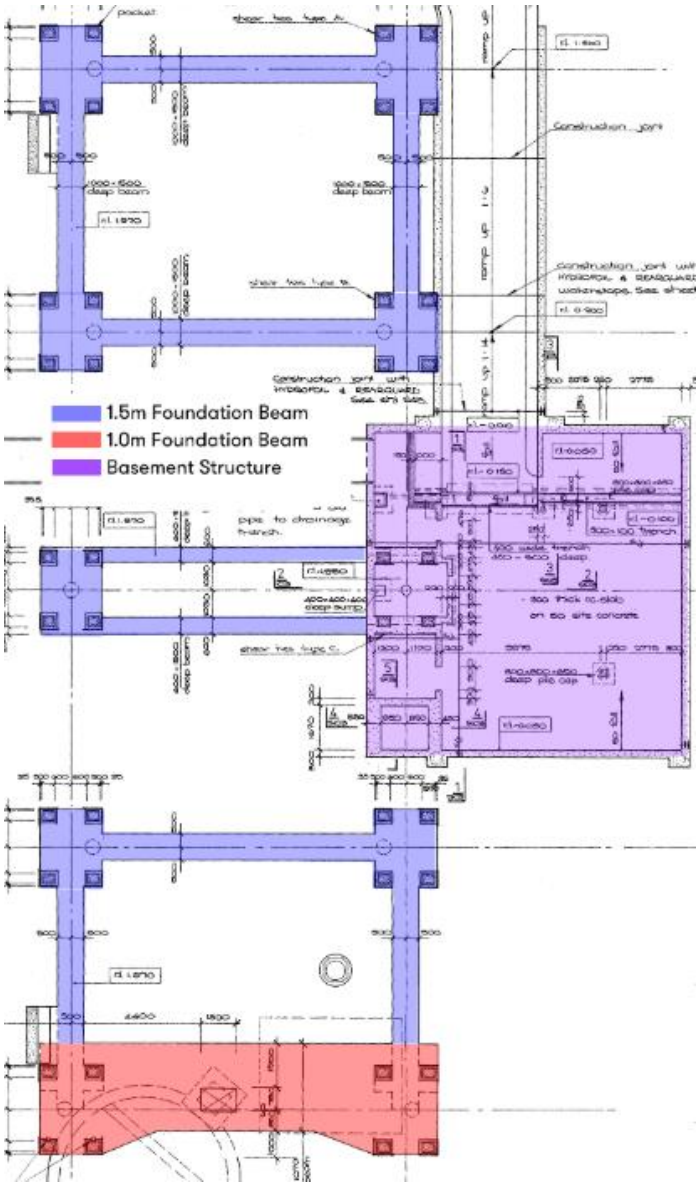

Figure 5-54: Foundation plan of Renouf Foyer (drawing S102 of original structural set)

5.3 Description of the Independent Staircase Structures

The independent staircase structures are located between the Auditorium and the Renouf Foyer, shown in Figure 5-55. The staircase structures contain suspended floors at Levels 3 and Level 4 which are used for toilet facilities. The staircase structures also support a series of stairs (three flights each) on the external faces, which span between the Renouf Foyer and the Auditorium, between the ground floor level and the floors at Level 2, Level 3 and Level 4. The staircase structures are constructed on independent foundations and are seismically separated from the Auditorium and the Renouf Foyer, though the frame extensions of the Bay Walls of Bays 6(a) and 7(a) also land on these foundations. Otherwise, they are seismically separated from the Auditorium and the Renouf Foyer. Therefore, they are expected to behave independently during seismic shaking.

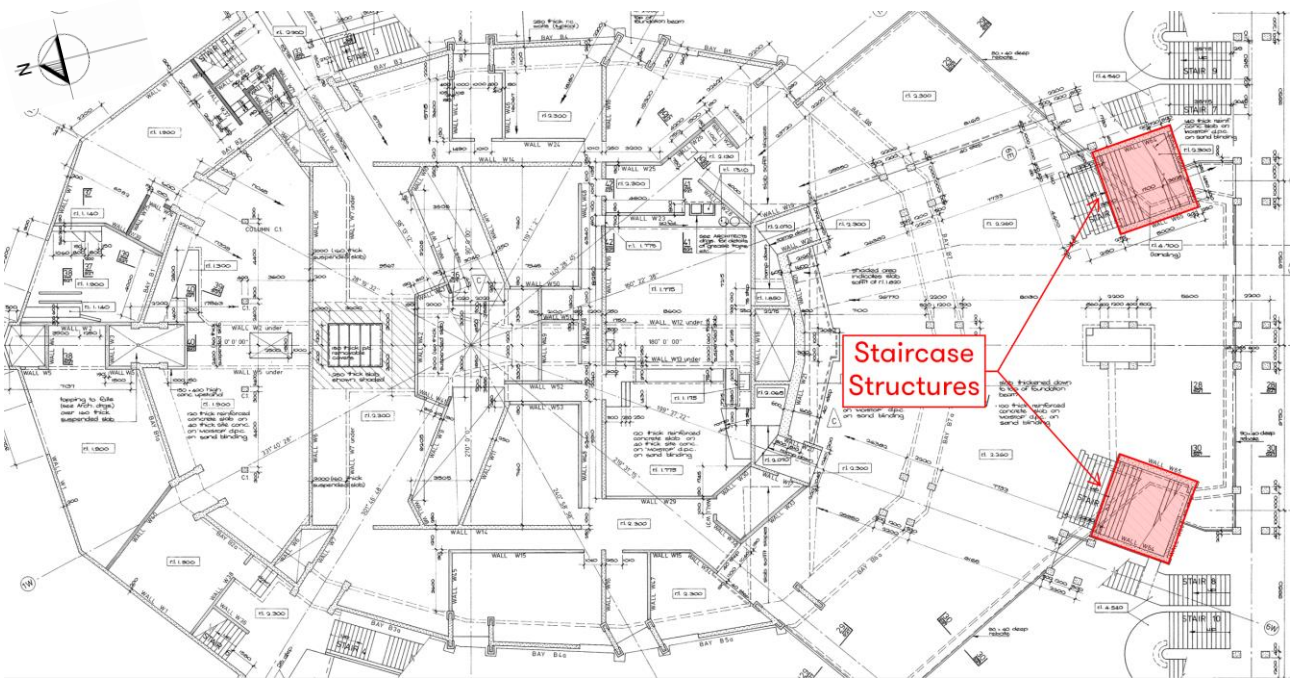

Figure 5-55: Level 1 Auditorium plan highlighting the independent staircase structures (drawing S3 of original structural set)

5.3.1 Lateral and Gravity Load Resisting System

All structural elements of the independent stair structures contribute to the lateral and gravity resistance. Refer to Figure 5-55 for plans and elevation of the rectangular box structure. The staircase structures resist lateral loads through the primary stair walls under in-plane loading. In out-of-plane loading the wall segments act through frame action, in combination with the floor slabs and interior stair run. A secondary concrete wall (W66) is located only at the southern end of the structures, spanning between the ground floor level and Level 2.

Concrete floor slabs occur at Level 3, Level 4, and at the timber roof level. The top of the staircase structures occurs just below the glulam framing. Stairs 1 & 2 run between the primary stair walls and are tied in and similarly act as a floor slab.

5.3.2 Primary Structure

The staircase structures house a cast insitu stair which spans between the ground floor level in the Auditorium to Level 2 in the Renouf Foyer. The staircase structures support intermediate landings for two external stairs each (stairs 7/8 and 11/12) which find support on a small cantilever ledge of the external face of the primary walls. The upper external stair (stair 11/12) has a similar cantilever ledge, which provides access to the level 3 toilet blocks. The floor plan layout of the staircase structures and a typical section of can be seen in Figure 5-56.

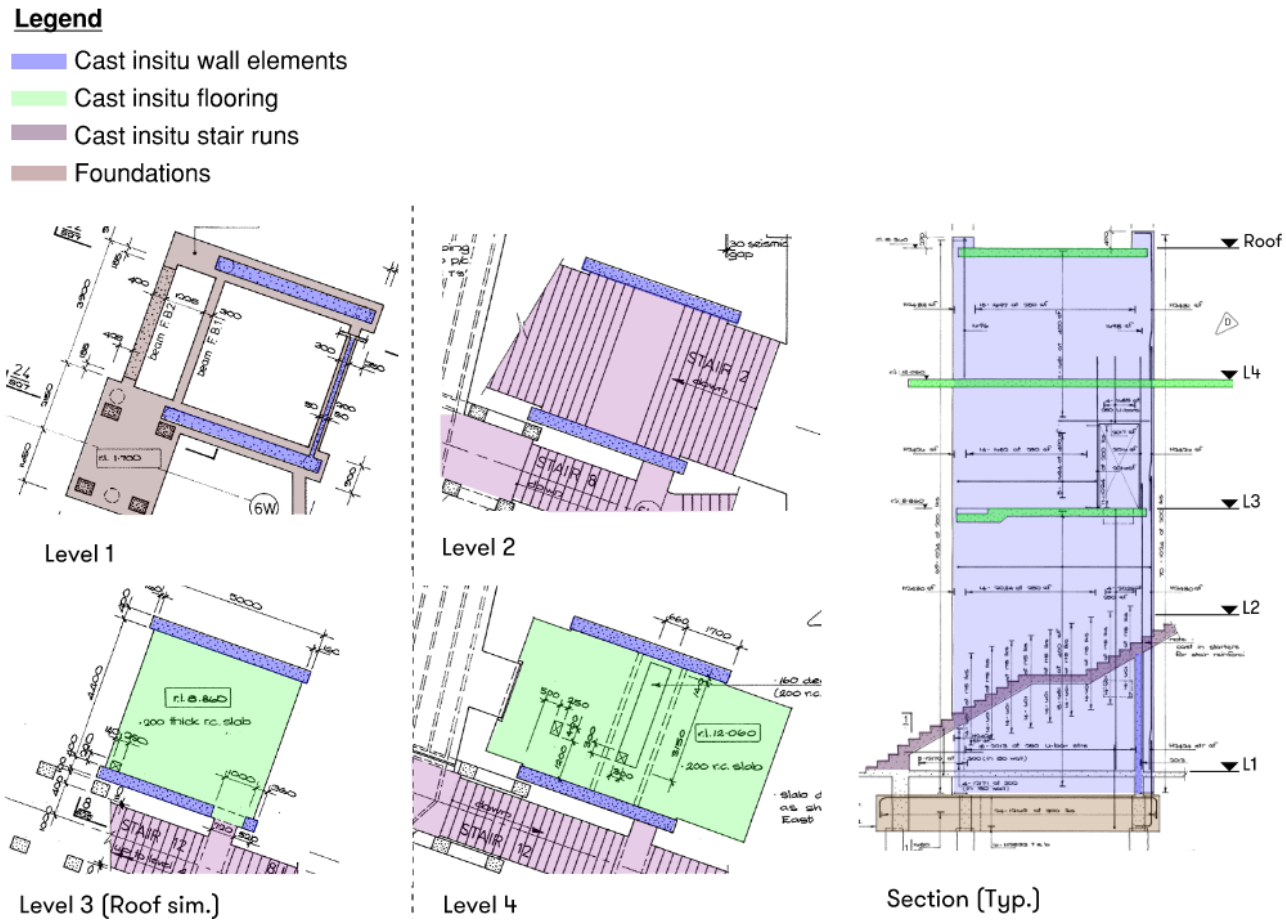

Figure 5-56: Partial floor plans of the independent staircase structure (drawings S3, S5, S6 & S7 of original structural set)

5.3.2.1 Roof system

The staircase structures also support the gravity loads for a portion of the lightweight roof. The primary steel structure in the roof allows for relative seismic movement between the independent stairs and the other structures, while some secondary components are rigidly connected. For a detailed description of the timber roof system see the Auditorium Section 5.1.4.1.

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6 BASIS OF THIS REPORT

The Detailed Seismic Assessment (DSA) has been completed in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017). This document is referred to in this report as the **Engineering Assessment Guidelines**.

The performance levels of the reinforced concrete elements have been assessed in accordance with the revised version of Chapter C5 (Concrete Buildings, November 2018) of the Engineering Assessment Guidelines. This document is referred to as the **Yellow Chapter C5**.

6.1 Information Available for the Assessment

The assessment has been based on the following information listed in Table 6-1. Documents that are part of the same set have been lumped for clarity.

Specific references are made to this documentation throughout this report.

Table 6-1: Information used in the assessment

#	Document	Date	By
1	Architectural drawings	1980	Warren & Mahoney Architects
2	Structural drawings	1980	Holmes, Wood, Poole & Johnstone
	Structural specifications		
	Site reports (1 – 5) *		
	Structural design calculations		
3	Piling report	1982	Holmes, Wood, Poole & Johnstone
4	Structural drawings of the Kitchen Extension	1989	Holmes Consulting Group
	Structural calculations for the Kitchen Extension		
5	Seismic Assessment of the Michael Fowler Centre	2014	Holmes Consulting Group Limited
6	Structural drawings for the seismic strengthening of staircase structures	2016	Holmes Consulting LP
	Structural Project Features Report and calculations for the seismic strengthening of staircase structures		
7	Site report documenting the rapid visual assessment of the Michael Fowler Centre following the 2016 Kaikoura Earthquake event	2016	Holmes Consulting LP
8	Assessment report of precast flooring and roof to the requirements of the Yellow Chapter (C5)	2018 - 2019	Holmes Consulting LP
	Structural drawings for the seismic strengthening of precast flooring and roof		
	Structural Project Features Report and calculations for the seismic strengthening of precast flooring and roof		
9	High-level assessment report of the foundations and soil-structure interaction	2020	Holmes Consulting LP Tonkin+Taylor Limited
10	Photographs taken of the condition of the floors during the placement of new carpet flooring	2021	Holmes Consulting LP Wellington City Council

#	Document	Date	By
11	Roof Truss Service Loading Report	2018 – 2022	Holmes Consulting LP
	Structural calculations for the service loading and stage rigging in the Auditorium and the Renouf Foyer		
12	Structural drawings for envelope remediation of Stairs 15/16	2022	Holmes Consulting LP
	Structural Project Features Report and calculations for envelope remediation of Stairs 15/16		
13	High-level assessment report of the roof diaphragm	2022	Holmes Consulting LP
14	Geotechnical Assessment Report	2022	Tonkin+Taylor Limited

** Only a small selection of site reports was able to be retrieved and used in this seismic assessment*

6.2 Site Observations and Investigations

Visits to the site have been carried out on the dates summarised in this section of the report. The purpose of these visits was to visually observe important structural and seismic characteristics, assess their condition and note obvious deficiencies or differences from the documentation. A summary of observations can be found in Appendix J. Observations have been visual only, unless stated otherwise. Material testing has not been completed. Material strengths based on the values used in the original design documentation were considered sufficient for this assessment

The as-constructed structure appears to be generally consistent with the available drawings. No obvious significant modifications appear to have been made to the primary lateral or gravity systems as far as could be determined from the visual inspections, other than those explicitly noted in sections 5.1.5 and 5.2.5. No obvious significant cracking, spalling, corrosion, or decay was observed that would affect the results of this assessment.

The site visit of 25 May 2022 was a general walk-around of the site and through the buildings in all publicly accessible areas.

The site visit of 24 August 2022 included visual inspections of the ceiling space of the Auditorium and the catwalk system.

The site visit of 30 September 2022 was used to complete the audit of the secondary structural and non-structural (SSNS) elements within the Michael Fowler Centre. It included visual inspections in the ceiling spaces of the Auditorium and Renouf Foyer, and a general walkover through the publicly accessible areas.

On 24 November 2022, intrusive investigations were undertaken, which allowed the visual inspection of the ceiling spaces inside the promenade roof. The primary objective of this site visit was to verify the extent of the plywood diaphragm, the overall structural detailing. There was also consideration given to the quantity and spacing of nails, though no specific measurements were taken. A description of the observations is captured in Section G.3.1 of Appendix G.

During this site visit, there was also concrete scanning undertaken of the double-tee floor seating at the Level 4 promenade. This was used to verify the detailing of drawing S78 and S79 of the original structural drawings by Holmes, Wood, Poole & Johnstone.

The site observations from previous engagements have also been included in this DSA. This includes the observations during the securing works of the stairs, the securing works of the precast units, the placement of new carpet flooring, and the envelope remediation works of Stairs 15/16.

6.3 Earthquake Demand Used in the Assessment

In accordance with the Building Act 2004 (as amended by the Earthquake-prone Buildings Amendment Act 2016), the Ultimate Limit State (ULS) seismic demand is derived from the New Zealand Structural Design Actions Standard, NZS1170.5:2004 (Standards New Zealand, 2016).

Factors that affect the earthquake demand are described below.

The earthquake demand does not cover any updates that are part of the recent (October 2022) review of the National Seismic Hazard Model (NSHM). Though it is noted that the NSHM has highlighted that the earthquake demands in Wellington are generally higher than those currently captured by NZS1170.5:2004.

Building Importance Level

We have assumed Importance Level 3 (major structure accommodating crowds) with a 50-year design life, in accordance with the New Zealand Structural Design Actions Standard NZS1170.0:2002 (Standards New Zealand, 2011), for the analysis of the Michael Fowler Centre.

The New Zealand Structural Design Actions Standard NZS1170.5:2004, requires the use of a higher level of load for Importance Level 3 buildings for the design of new structures. Typical buildings are regarded as Importance Level 2 and are designed for an earthquake with and annual probability of exceedance of 1/500. An Importance Level 3 building is designed for a load level approximately 30% greater (a 'risk factor' of R=1.3). This equates to a design-level earthquake with and annual probability of exceedance of 1/1000.

Seismic Hazard Zone

Wellington CBD is located in a zone of high seismicity. The Wellington Fault passes within 1.5km of the site. We have used a hazard factor of Z=0.40 and near-fault factors in accordance with NZS1170.5:2004.

Similarly, for the Nonlinear Time-History Analysis (NLTHA), the NZS1170.5:2004 hazard spectrum is adopted for the scaling the ground motion records. Further information on ground motion selection and scaling is found in Section D.3.1.

Site Subsoil Class

Seismic loads are also dependent on the soil type a structure sits on. According to the Geotechnical Assessment Report by Tonkin+Taylor Ltd., a Site Soil Class of D has been used.

Comparison between original design requirements (NZS4203:1976) and design requirements for a new building (NZS1170.5:2004)

The original design calculations from the 1979 design documentation were available for review during the assessment process. This provided valuable insights in the level of seismic actions that were used for the original design of the structure for comparison against current design loading.

The Michael Fowler Centre was structurally designed in accordance with the seismic design actions specified in NZS4203:1976. Current seismic design actions are specified in NZS1170.5:2004.

Although seismic design procedures were deemed "modern" in 1976, there have been developments in the intervening years which have resulted in a better understanding of seismic performance, increased levels of design load, and also more restrictive requirements for the detailing of reinforced concrete components – in particular for buildings of such complex form and geometry.

The difference between the seismic design coefficient from each standard are illustrated in Table 6-2 and Figure 6-1. This concludes that the Michael Fowler Centre was originally designed for seismic loads which

are approximately 33% - 40% of the current seismic design loadings, assuming a structural period of T≤0.4s and a system ductility of μ_{sys}=1.25.

Table 6-2 : Comparison of parameters used to determine the seismic coefficient between NZS4203:1984 and NZS1170.5

Parameter	NZS 4203:1976			NZS1170.5:2004		
Ground conditions	C	0.15	Flexible soil T < 0.45s Zone A	$C_h(T)$	3.0	Sub-soil class D T < 0.4s
Structure's fundamental period						
Location				Z	0.4	Wellington
Risk factor	R	1.1	Category 2 [1]	R_u	1.3	Importance Level 3
Importance factor	I	1.3	Class 2			
Structure type	S	1.0	Case 4 [2]			
	S	1.6	Case 6 [3]			
Structure materials	M	1.0	Reinforced non-prestressed concrete			
Near-fault factor				N	1.0	T < 1.5s
Structure ductility				k_μ	1.143	Structural ductility factor approximated at μ_{sys} = 1.25
Structural response factor				S_p	0.925	
Near-fault factor				N	1.0	T < 1.5s
Structure ductility				k_μ	1.571	Structural ductility factor approximated at μ_{sys} = 2.0
Structural response factor				S_p	0.70	
Seismic design coefficient	C_d	**0.21** [2] **0.34** [3]	C · I · S · R · M	C(T)	**0.69** [4] **1.26** [5]	$\frac{C_h(T) \cdot Z \cdot R_u \cdot N \cdot S_p}{k_\mu}$

[1] In NZS4203:1976, category 2 buildings included 'buildings accommodating more than 1,000 people'.

[2] In NZS4203:1976, case 4 buildings included 'two or more parallel and approximately symmetrically arranged ductile cantilever shear walls'.

[3] The concrete squat wall system below Level 2 was designed for higher lateral load from seismic shaking, corresponding to case 6, which was intended for 'shear walls not designed for ductile flexural yielding but having the ability to dissipate a significant amount of seismic energy'.

[4] Considering a system ductility of μ_{sys} = 2.0

[5] Considering a system ductility of μ_{sys} = 1.25

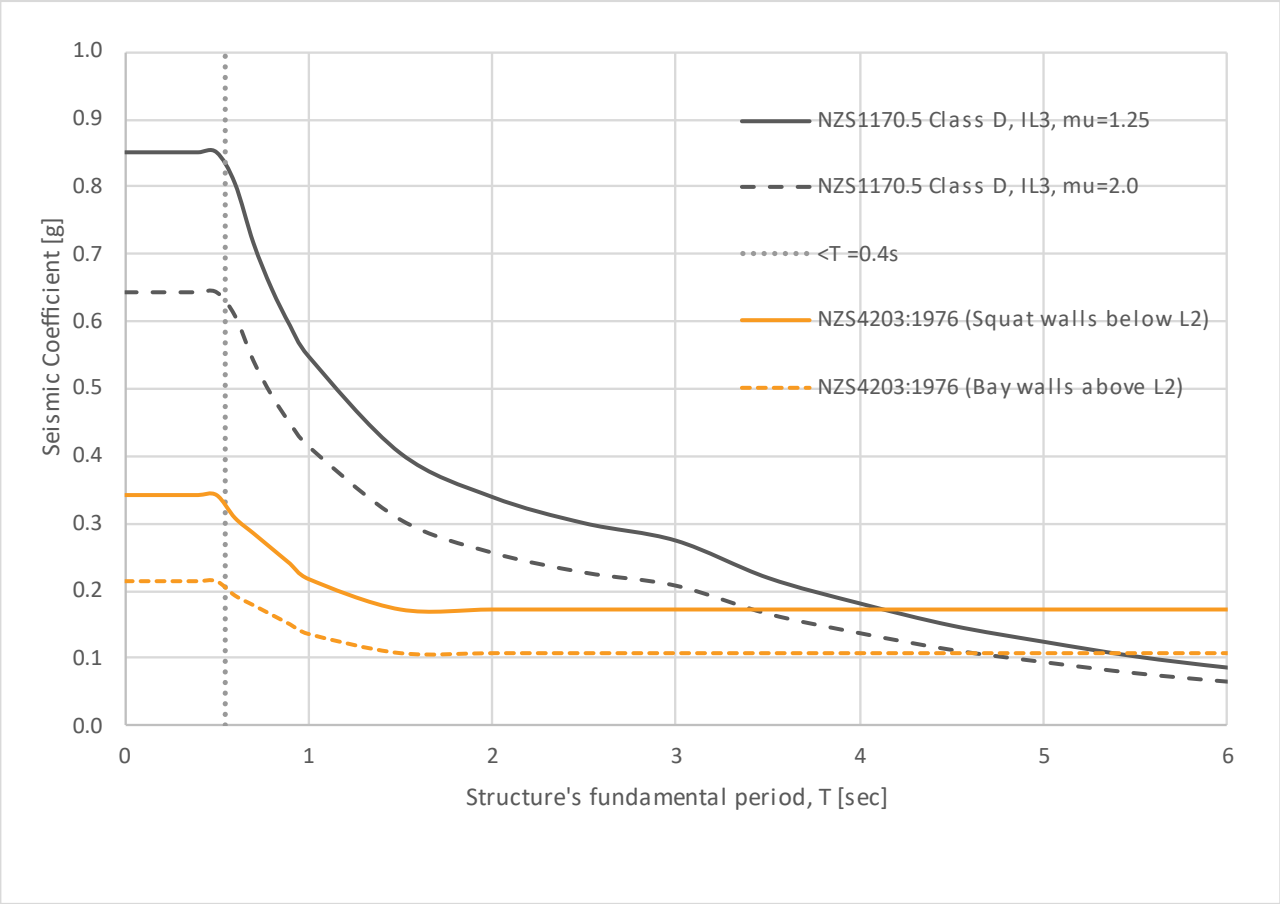

Figure 6-1: Seismic spectra comparison between NZS4203:1984 and NZS1170.5:2004

6.4 Previous (Seismic/Structural) Assessments

Detailed Seismic Assessment (2014)

The Michael Fowler Centre was previously seismically assessed in 2014 to the "NZSEE Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, June 2006, including Corrigenda 1, 2 and 3" by Holmes Consulting Group Limited. The structures were categorised to be Importance Level 3 (IL3). This assessment concluded that the primary structures of the Auditorium, the Renouf Foyer and the independent staircase structures typically had seismic ratings of 50-60%NBS_{IL3}. An assessment to the 2006 version of these guidelines primarily focussed on the primary lateral load resisting system. Therefore, the 2014 assessment did not include detailed considerations of the capacity of diaphragms, foundations, secondary structural elements, etc. Nonetheless, the 2014 assessment identified potential structural weaknesses with regards to the staircases, and the hollowcore (Dycore) roof structure of the Auditorium. In addition, it was noted that the building was potentially prone to the effects of liquefying ground layers under the building, for example through lateral spread of the ground.

It is specifically noted that this seismic assessment excluded detailed analysis into the following areas:

- Assessment of the roof diaphragm. Though it was noted that the roof diaphragm is 'lightly' reinforced with cold-drawn (non-ductile) mesh and qualitative comments were made on this being a potential structural weakness. A sensitivity study was undertaken in the analysis model to test

whether the primary lateral load resisting structure would have a similar performance without the roof structure. This was mimicked by reducing the structural thickness of the diaphragm by a factor of 4. The 2014 assessment report then concluded that the roof structure does not carry any major transfer loads.

- Assessment of precast flooring elements with regards to loss of seating, torsion, negative moment failure and other failure modes of Dycore and double-tee units. Only a qualitative assessment was done to highlight the potential associated risks.
- Assessment of the upper precast portion of the Bay Walls. A sensitivity study was undertaken using a sub-assembly model, which concluded that the upper precast portion of the Bay Walls were not considered a limiting factor on the panel performance.
- Assessment of the foundation system. A qualitative comment was made that the probable lateral displacements of the structure are small enough that the piles were not considered to render the building earthquake prone. There was also a comment that the effects of liquefaction and lateral spread would need to be considered if any strengthening would be proposed.
- Assessment of the effects of pounding.
- Assessment of other secondary and non-structural elements, such as the catwalks and reflectors inside the Auditorium, the light-weight façade structure around the building, the precast infill panels, etc.

Assessment of roof structure (2018)

In 2018, Holmes Consulting LP were engaged to undertake a review of the roof of the Auditorium for the purpose of providing high-level recommendations and options to reduce seismic risk. It included a visual inspection in the voids of the Dycore units (a representative selection was taken) and an analysis model was setup of the roof diaphragm (65mm topping concrete). The analysis model was set up in isolation and made simplified assumptions on the behaviour of the global structure.

The conclusions were as follows:

- No significant cracking was observed in the Dycore units. Additional supplementary angles were recommended to provide improved seating width for the hollowcore units, which was implemented in 2019. The primary aim was to reduce the potential for loss of support to these units should the diaphragm mesh capacity be locally exceeded.

- The assessment showed that a score of 34%NBS_{IL3} could likely not be achieved in a detailed seismic assessment as the roof diaphragm would exceed the capacity of the reinforcing. The diaphragm was assessed to the 2017 version of the Engineering Assessment Guidelines, incorporating the Red Chapter C5, which allowed the reliance on the capacity of the cold-drawn (non-ductile) mesh. The report subsequently concluded that there were localised areas that would require strengthening to achieve a rating above 34%NBS_{IL3}.

Assessment of precast flooring (2019)

In 2019, Holmes Consulting LP were engaged to undertake a review of the Auditorium promenade and Renouf Foyer precast floors, including the seating of the double-tee units at Level 2 and Level 4. The assessment included a visual inspection of the double-tee flooring and calculations in accordance with Yellow Chapter C5 of the Engineering Assessment Guidelines.

The report concluded that the flange-hung double-tee units were particularly prone to “bird’s mouth” failure or unseating at large displacements of the Auditorium. The web-supported double-tee units were concluded to achieve acceptable seismic scores >100%NBS_{IL3}.

In 2020, securing works were undertaken, which are described in Section 5.1.5.

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Assessment of the potential effects of liquefaction and lateral spread (2020)

In 2020, Holmes Consulting LP and Tonkin+Taylor Limited were engaged to provide high-level recommendations and options to reduce the seismic risk due to the performance of the foundations of the Michael Fowler Centre.

A Geotechnical Desktop Assessment was completed by Tonkin+Taylor Limited which determined that liquefaction was predicted to occur at 45-55% NBS_{IL3}. At the onset of liquefaction, cyclic displacements of up to 150mm were estimated and lateral spread up to 500mm at 50%NBS_{IL3}.

The high-level structural assessment determined that the elastic base-shear take-out capacity of the structures was 35%-40% NBS_{IL3}. The assessment found that the foundations were likely to have a lower seismic score than the triggering of liquefaction in the ground. The assessment further identified that there are areas which may be subject to differential lateral spread and that those areas have limited tolerance to that movement.

Several remediation options were presented in the report to mitigate the seismic risk associated with liquefaction and lateral spread of the ground.

(Re)assessment of the roof (2022)

In 2022, Holmes Consulting LP were engaged to review the earlier undertaken assessment of the roof diaphragm to the recommendations of the more recent Yellow Chapter C5 which specifically states that the capacity of cold-drawn (non-ductile) mesh shall not be relied on to provide capacity in tension ties. The reassessment was undertaken to estimate whether the roof diaphragm is likely to achieve a score of less than 34%NBS_{IL3} in a more comprehensive DSA.

The assessment considered 'strut and tie' calculations which attempted to show that the inertia of the roof could land the compressive struts on the Bay Walls and that the roof trusses could act as ties to keep the buttresses from spreading apart. Ultimately, this concluded that, when viewed in isolation, the roof diaphragm could achieve a score of >34%NBS_{IL3}.

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7 METHOD OF ASSESSMENT

The Michael Fowler Centre comprises of four structures, intended to behave independently with separate lateral force resisting systems, and having seismic gaps at the interfaces with each other. Each structure has been analysed and assessed independently in acknowledgment of the primary structural intent, as outlined in Sections 7.1 to 7.3.

Where the different structures interact, the assessment included consideration of the provisions of Section C2B of the Engineering Assessment Guidelines for pounding.

The foundations have been assessed in isolation of the superstructure's response.

7.1 Auditorium

Two levels of analysis are used to interrogate the Auditorium structure. A Modal Response Spectrum Analysis (MRSA) is performed in accordance with Section C2.7.2 of the Engineering Assessment Guidelines to determine the seismic rating of the superstructure and substructure. A supplementary Nonlinear Time History Analysis (NLTHA) model was built to rationalise the Ultimate Limit State (ULS) floor acceleration used for the diaphragm analyses. Although this NLTHA model has allowed for review and validation of some of the other primary building load paths (including primary buttress wall inelasticity), it is only used to define floor accelerations for the assessment and not to determine element demands of the primary lateral load resisting system.

Modal Response Spectrum Analysis (MRSA)

We have used the Modal Response Spectrum Analysis method to determine the seismic scores of the primary lateral load resisting system of the Auditorium. The Auditorium is expected to behave in a generally nominally ductile manner, with some potential for limited ductility in flexure or shear in lightly axially loaded perimeter buttress walls; therefore, it was deemed suitable for an MRSA approach. The primary lateral load resisting system is generally regular in plan around the structure, with the exception of the discontinuities at the Bay Walls of Bays 6 through 6a Lobby region.

An MRSA was chosen over an Equivalent Static Analysis (ESA) procedure due to the irregularity of floor levels and diaphragms in the Auditorium—and also the potential for the ESA to misrepresent the distribution of accelerations and inertial forces in a structure which is stiff relative to its foundations. Some diaphragms only exist on portions of the building and story heights are irregular. A triangular distribution in accordance with NZS1170.5:2004 was therefore not considered appropriate. MRSA base-shears and forces were scaled above grade level to be at least equal to ESA base shears. See Appendix C for the technical summary of the MRSA model, which was built in ETABS 20.3.0.

Additionally, the initial (elastic) lateral stiffness of the piled foundation and foundation beams was included in determining the superstructure modal response and vertical force distribution. This demonstrated that the Auditorium is likely behaving close to a large rigid block on flexible foundations with limited higher mode effects.

The inclusion of nonlinear foundation stiffness was used to perform sensitivity studies on the interaction between the foundation system and the superstructure. The upper and lower bound flexibility of the foundations was combined with the initial and cracked stiffnesses of the diaphragms to understand the ability of the superstructure to find secondary load paths. A summary of these findings is captured in C.3.4.

Nonlinear Time-History Analysis (NLTHA)

Nonlinear time-history analysis involves subjecting the computer model of the building to a large and variable suite of real earthquakes. The earthquake records are carefully selected and scaled so that they

represent the types of earthquakes which contribute to a given seismic load level and the local seismic hazard. The actual time-varying response of the structure to these records is analysed. As elements yield or become damaged, the analysis continually updates their properties so that the effect of that damage on response is directly incorporated into the assessment.

The evaluation was conducted in accordance with the NZ Engineering Assessment Guidelines, with a mean ground motion scaling process used (specifically, ASCE 41-17). Further information and technical details on seismic loads, ground motion selection and scaling, analysis procedures and evaluation can be found can be found in Appendix D. The NLTHA model and analysis was used for part of the evaluation only.

The nonlinear model was built in Holmes' in-house analysis program ANSR-II and was built to a level to enable upper bound acceleration response of the diaphragms to be determined, and to allow some validation and checking of the primary structure evaluation (reviewing shear and flexural behaviour of perimeter buttress walls and the distribution of nominal or limited ductility).

Diaphragm accelerations from the Yellow Chapter C5 (similar to the NZS1170.5:2004 pseudo–Equivalent Static Analysis or pESA approach) were reviewed but not considered to be appropriate. They are intended for use in other structural types and for this short period structure they give overly conservative estimates of accelerations at height for a given level of ground motion input. The evaluation of diaphragms themselves was not conducted in the time-history model. Acceleration envelopes from NLTHA were used to construct a specific static pESA distribution for diaphragm evaluation.

The horizontal stiffness of the piled foundation stiffness was limited to the initial elastic stiffness. No soil or pile nonlinearity has been included. There is likely to be value in including a representation of hysteretic damping in a strengthened scheme, depending on the form of any foundation interventions that might be proposed. With the exception of a few particular vulnerabilities (such as diaphragms), the present lateral capacity of the foundations is broadly comparable to that of the primary superstructure and thus soil damping as the foundation capacity mobilises could have a high influence on overall behaviour.

However, at this time, inclusion of small levels of hysteretic damping at the levels of shaking associated with assessed diaphragm ultimate capacity are unlikely to materially change the reported %NBS range of those components which are clearly a significant vulnerability. Similarly, the effects of liquefaction and kinematic displacements were not investigated in this analysis—it is used to investigate upper bound inertial superstructure response in the non-liquefied or undegraded case.

7.2 Renouf Foyer

The Simple Lateral Mechanism Analysis (SLaMA) method was used for the complete structural assessment of the Renouf Foyer to determine the seismic score. This method involves identifying the likely elastic and inelastic deformation mechanisms, and the lateral strength and displacement capacity of these mechanisms.

SLaMA is an appropriate form of assessment for the Renouf Foyer, as the structure consists of only two suspended levels and has several similar frames of regular layout. This indicates the overall seismic response should be dominated by the fundamental vibration modes in each principal direction and higher modes will not be critical.

The simple lay-out of the frames means that any possible inelastic mechanism could be readily identified and assessed in accordance with the Engineering Assessment Guidelines. Once the SLaMA has been completed, potential inelastic mechanisms of the structure were identified. This considers element overstrength, to capture the variety of potential mixed beam/column sway mechanisms and flexure-shear type response as the structure transitions to inelastic behaviour.

The SLaMA method was also supplemented by linear elastic analysis in a 3D Microstran model.

7.3 Independent Staircase Structures

The Simple Lateral Mechanism Analysis (SLaMA) method was also used for the complete structural assessment of the independent staircase structures.

SLaMA is an appropriate form of assessment for the independent staircase structures as the structure has a regular layout and well-defined lateral systems acting in each direction. The primary walls act in parallel motion in the in-plane direction. In the out-of-plane direction they form a single-bay, 4-storey moment frame system with the Level 1 internal stairs and the Level 2 and Level 4 floor slabs. Based on the lack of significant vertical irregularities in the lateral systems, the overall seismic response is dominated by the fundamental vibration modes in each principal direction and higher modes are not critical.

The simple lay-out of the frames means that any potential inelastic mechanisms could be readily identified and assessed in accordance with the Engineering Assessment Guidelines.

7.4 Software Version Summary

The following computer applications were used for the assessment:

Table 7-1. Software Versions used for Design and Validation

Building	Analysis Type	Software Used	Software Version	Archive Files
Auditorium	MRSA	ETABS Plus	20.3.0	MFC_Auditorium_v15
Auditorium	NLTHA	ANSR [1]	4E	ModelA4e_MFC_DSA_v10a
Renouf Foyer	ESA	Microstran	10.1.0.3	Renouf_Foyer_Full (V2, V4 and V5)
Independent Staircases	ESA	Microstran	10.1.0.3	MFC_StairBlock_OOP_V1

[1] Holmes' in-house nonlinear modelling software

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8 ASSESSED SEISMIC RATING

We have assessed the seismic rating for the Michael Fowler Centre as 20%NBS$_{IL3}$

The results of the DSA indicate the building's seismic rating to be 20%NBS$_{IL3}$ governed by the following elements:

- the sloping transfer diaphragm below the Level 2 seating in the Auditorium (referred to as Level 1/2),
- the chord tie across the promenade at Level 2 of the Auditorium,
- the double-tee flooring at Level 2 in the lobby area of the Auditorium,
- the double-tee flooring at Level 4 around the promenades of the Auditorium,
- the roof diaphragm of the Auditorium, and,
- the Level 2 diaphragm in the Renouf Foyer.

These diaphragms provide a horizontal load path through the structures of the Auditorium and the Renouf Foyer and the primary vertical (gravity) load resisting systems are reliant on these load paths to maintain their integrity.

There are also localised elements with a similar seismic score (<34%NBS$_{IL3}$) identified in the following locations:

- the wall panels at the upper precast portion of the levels of the bay walls,
- the precast concrete cladding infill panels between the upper precast portion of the bay walls,
- the roof trusses of the Auditorium,
- the promenade roofs and facades around the building,
- the catwalk structure in the Auditorium, and,
- the wall reflectors in the Auditorium

The vertical elements (walls) of primary lateral load resisting system of the Auditorium are governed by the wall panels in the upper precast portion of the Bay Walls, which are rated at 25%NBS$_{IL3}$. The lower cast insitu portion of the Bay Walls has a seismic score of 50%NBS$_{IL3}$.

The foundation system is limited by the lateral spread towards the lagoon at 45%NBS$_{IL3}$. The effects of liquefaction and lateral spread of the ground will exceed the capacity of the foundations at higher levels of shaking. The provisions for the geotechnical step change reduction in seismic rating have only been applied to elements where a subsequent structural step change results in a significant life-safety hazard.

The vertical elements (frames) of the primary lateral load resisting system of the Renouf Foyer are rated at 60%NBS$_{IL3}$. This rating is controlled by the columns on the ground floor level of the building.

The independent staircase structures are rated at 45%NBS$_{IL3}$. This rating is controlled by the out-of-plane capacity of the walls acting as a frame structure.

Table 8-1: Seismic scores (%NBS$_{IL3}$) of the different structures

Structure	Primary lateral load resisting system	Diaphragms	Foundations	Secondary and non-structural elements
Auditorium	25%	**20%**	45%	25%
Renouf Foyer	60%	**25%**	70%	50%
Independent Staircase Structures	**45%**	>100%	95%	n/a

The precast flooring inside the Auditorium and Renouf Foyer have been assessed in accordance with the provisions of Yellow Chapter C5 and are considered as secondary structural elements (Section 8.4). These have found no governing structural weaknesses. The assessment has considered the securing works undertaken in 2019, which provided seating angles and catch-frames to support the precast flooring at critical locations. There are a few locations achieving seismic scores of 80%NBS_{IL3}; however, these are located on Level 2 in the lobby area of the Auditorium. This area of flooring is already assigned a lower score due to structural weaknesses in the diaphragm (topping concrete) and from pounding with the Renouf Foyer.

No severe structural weaknesses were identified in the assessment.

The building is potentially Earthquake-prone

The assessment has been undertaken to inform strengthening design and an assessment in accordance with the requirements of the Red Chapter C5 has therefore not been undertaken (which would be required to alter its Earthquake-prone status). However, it is expected that it would not materially change the conclusions of the DSA.

A building with an earthquake rating less than 34%NBS_{IL3} 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_{IL3} is considered as an Earthquake Risk Building by the New Zealand Society for Earthquake Engineering.

The Michael Fowler Centre is categorised as an Earthquake Risk Building and also it meets one of the criteria that could categorise it as an Earthquake-prone Building.

Relative earthquake risk compared with a new building

The building is classified as a Grade D building following the NZSEE grading scheme, as shown in Table 8-2. Grade D buildings represent a risk to occupants 10-25 times that expected for a new building, indicating a high-risk exposure relative to a new building if a large earthquake occurs.

Table 8-2: Grading system for earthquake risks relative to a new building from Engineering Assessment Guidelines

Percentage of New Building Standard (%NBS_{IL3})	Grade	Approx. 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
35-66	C	5-10 times greater	Medium risk
20-34	**D**	**10-25 times greater**	**High risk**
<20	E	25 times greater	Very high risk

Actions required by the building owner in accordance with the Building Act

The Territorial Authority has notified Wellington City Council that the Michael Fowler Centre needs to complete seismic strengthening before 14 September 2029. This Detailed Seismic Assessment informs which elements of the building have shown a seismic score below <34%NBS_{IL3}.

More information of regulatory requirements for existing buildings can be found in Section 10.

Structural Weaknesses

A Structural Weakness (SW) is any aspect of the building structure and/or foundation soils that scores less than 100%NBS. Other elements that could present a significant life safety hazard were assessed as having strength and deformation capacities above 100% NBS_{IL3}.

The Critical Structural Weakness was found to be the sloping transfer diaphragm below the Level 2 seating of the Auditorium (referred to as Level 1/2)

The Critical Structural Weakness (CSW) is the lowest scoring structural weakness, the failure of which would result in a significant life safety hazard. The CSW was found to be the sloped transfer diaphragm below the Level 2 seating of the Auditorium (referred to as Level 1/2), which connects bay walls at Bays 6 through 6a to the squat Walls 18 and 21 at the ground floor level.

This diaphragm provides a primary lateral load path between these structural elements that is integral to the lateral load resisting system of the Auditorium. Exceedance of the capacity of this diaphragm could require the structure to re-establish global stability through secondary load paths. This will be paired with larger displacements across structural elements that are sensitive to those displacements.

8.1 Auditorium

The following structural weaknesses have been identified in the primary structure of the Auditorium.

Table 8-3: Structural Weaknesses identified in the assessment of the primary structure of the Auditorium

Building Element	Structural Weakness	%NBS_{IL3}
Transfer diaphragm at Level 1/2	Exceedance of tie capacities in the diaphragm (provided by deformed bars in the diaphragm) and ties to the flat diaphragm	20
Level 2 promenade diaphragm	Exceedance of chord tie capacities in the diaphragm (provided by deformed bars in the diaphragm).	20
Double-tee flooring at Level 2 in the lobby area	▪ Exceedance of friction capacity due to the lack of tie capacity in the diaphragm (665-gauge non-ductile mesh). ▪ Loss of seating due to excessive differential displacements and pounding between the Auditorium and the Renouf Foyer	25
Roof diaphragm	Lack of tie capacity in the diaphragm (661-gauge non-ductile mesh) leading to subsequent structural weaknesses in other elements, such as: ▪ Excessive axial loads in the top chords of the roof trusses ▪ Excessive loads in the connection between the roof trusses and the precast portion of the perimeter buttress wall system ▪ Excessive size of cracks in the concrete diaphragm across the supports of the DyCore roof elements Redistribution of seismic lateral loading to the perimeter buttress walls leading to excessive out-of-plane loading	20 – 30
Level 4 wedge-shaped diaphragm of promenade floor and stepped seating	▪ Exceedance of friction capacity due to the lack of tie capacity in the diaphragm (665-gauge non-ductile mesh). Loss of seating due to excessive differential displacements between the Auditorium and Independent Staircase Structures	25

Building Element	Structural Weakness	%NBS$_{IL3}$
Bay walls (upper precast wall panels)	Exceedance of shear capacity at the upper precast portion of the bay walls, at the horizontal interface between different precast wall panels	25
Bay walls (upper precast wall panels)	Exceedance of the out-of-plane flexural capacity of the precast concrete wall panels	35
Gravity columns at the extended frames of Bays 6(a) and Bays 7(a)	Lack of compatibility with the displacement demands from differential lateral spread between foundations of the Auditorium and the independent staircase structures ***(does not require a reduction for the consideration of a geotechnical step change)***	45
Bay walls (connection of upper precast wall panels to the roof)	Exceedance of shear capacity at the interface between the precast wall panels and the roof	45
Foundation system	Exceedance of inelastic rotation capacity of the piles during the lateral spread scenario ***(does not require a reduction for the consideration of a geotechnical step change)***	45
Gravity columns in the backstage area	▪ Exceedance of rotation limits for slab-column joints.	45
Bay walls (cast insitu portion above floor level 2)	Exceedance of inelastic rotation capacity, limited by bar buckling in the web area at Level 2 of Bays 7(a).	50
Bay walls (isolated columns in lobby area at Bays 6(a) and 7(a))	Exceedance of inelastic rotation capacity, which is significantly reduced due to the large axial load ratio. Because of the discontinuation of the Bay Wall system in the lobby area the flexural demands in the Bay Walls result in significant axial loads on the buttress columns at the ground floor level.	55
Level 2 Precast Cantilever Beams adjacent to double T slabs	Combined actions on beam reinforcing: gravity, lateral ties, vertical acceleration	55-70
Squat walls at Level 1	Exceedance of shear capacity in the walls. (the lowest score is assessed for Wall 18, other squat walls are achieving higher scores, see Section C4.1.2).	55-75
Level 4 Precast Cantilever Beams	Combined actions on beam reinforcing: gravity, lateral ties, vertical acceleration	75
Floor diaphragms at backstage area (Level 2, Level 3 and Level 4)	Exceedance of tie capacities in the diaphragm (provided by deformed bars in the diaphragm).	75-80

Building Element	Structural Weakness	%NBS$_{IL3}$
Floor diaphragm at director's area (Level 5)	Exceedance of tie capacities in the diaphragm (provided by deformed bars in the diaphragm).	85

8.1.1 Commentary on seismic performance

There is significant plan irregularity at Level 2

The primary lateral load resisting system of the Auditorium are the Bay Walls and the squat walls at Level 1. These elements collect the lateral seismic loading from the diaphragms and take this out through the foundations. The system is typically regular, except for the Bay Walls at Bays 6(a) and 7(a). Here, the web area of the walls terminates above Level 2 and only the twin columns of the buttress continue down to the foundations. This feature of the structure creates a large open plan area at the ground floor of the Auditorium; however, it is directly related to the Critical Structural Weakness of the Auditorium.

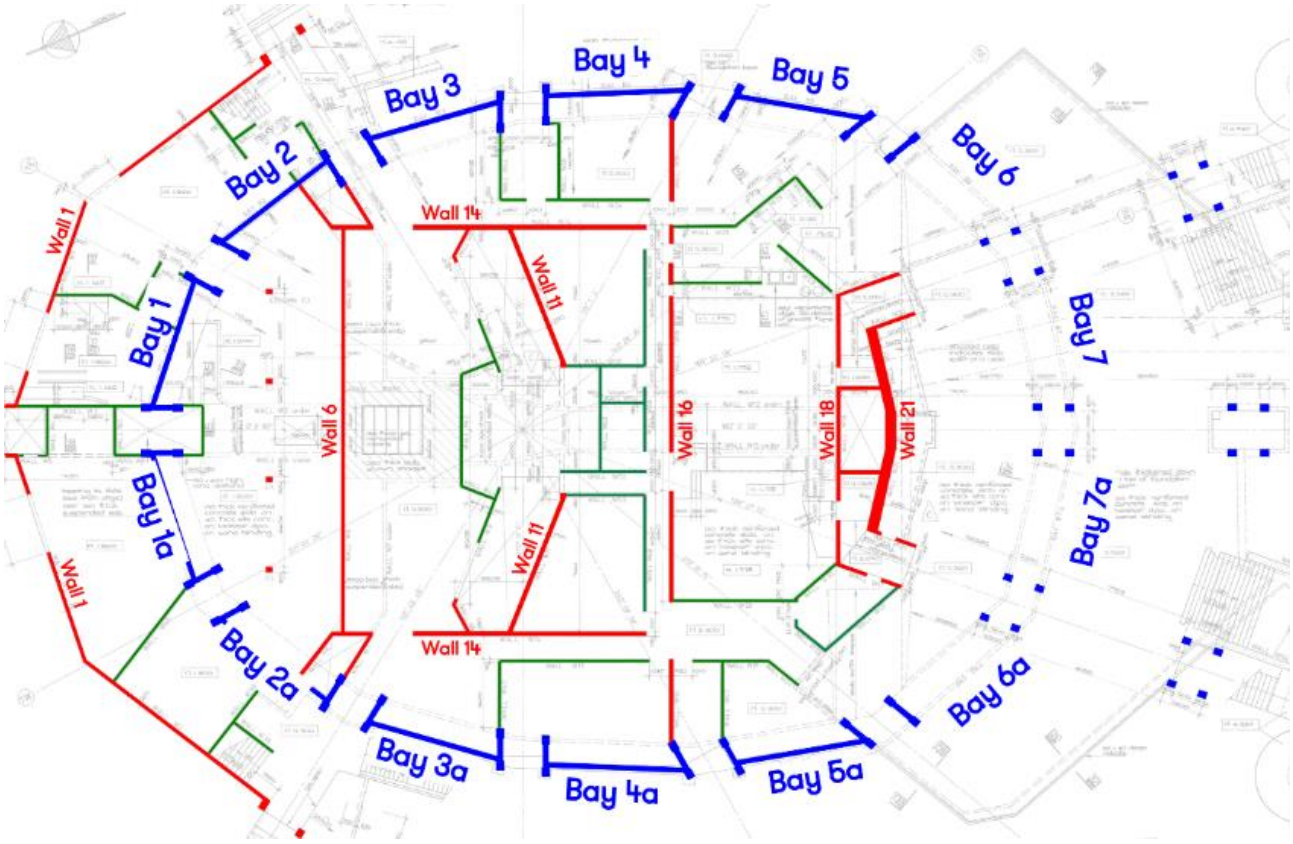

Figure 8-1: Snapshot taken from drawing S3C of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing the wall system at the ground floor level (Level 1)

To deal with the irregularity in plan, the primary lateral load resisting system is reliant on the sloping transfer diaphragm below the Level 2 seating of the Auditorium (referred to as Level 1/2). This diaphragm spans between the bay walls of Bays 6(a) and 7(a) down to the squat walls 16, 18 and 21. There is no other lateral load resisting system nearby.

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The only other lateral load resisting systems in the transverse direction of the Auditorium are located at Bays 1(a) and at squat Wall 6. However, these are located relatively far away and require the lateral load from seismic shaking to travel around the building, through the promenade floors. This is a much more flexible load path and may be paired with large lateral displacements – larger than the precast flooring at the Level 2 and Level 4 promenade floors could accommodate. As such, the sloping transfer diaphragm at Level 1/2 is a critical element in the lateral load resisting system of the Auditorium and exceedance of its capacity is considered a significant life-safety hazard.

The assessment of diaphragms for buildings with plan irregularity has seen a lot additional attention since the 2010/2011 Canterbury earthquakes and the 2016 Kaikoura earthquake, as damage has been observed in critical diaphragms. Generally, this has been observed to be resulting in subsequent damage to other related building elements that are reliant on the diaphragm. Whilst the original design has accounted for additional reinforcing in the sloping transfer diaphragm Level 1/2 (compared to other floor elements), it is the Critical Structural Weakness of the Auditorium.

There is cold-drawn mesh in several floor elements of the building

The diaphragms at the roof level of the Auditorium and the double-tee flooring in the promenade floors of Level 2 and Level 4 of the Auditorium comprise the precast flooring element with a 50/65mm topping. This topping is reinforced with 661-, 663- or 665-gauge mesh reinforcing.

There is also 665-gauge mesh reinforcing in the 100mm slab-on-grade floor between the southern end of the Auditorium and the independent staircase structures.

This mesh is classified as cold-drawn mesh with a lower bound tensile strain capacity of less than 5% (non-ductile) and as such is recommended by Section C5.6.3.1 of Yellow Chapter C5 to not be relied on when calculating the capacity of tension ties. The capacity of this mesh is known to be reduced materially by strains induced from long-term effects (such as shrinkage of concrete) and by concentration of strain arising from eccentric anchorage of forces by cross wires. By shrinkage alone, the strain capacity can be exceeded, and the mesh can already be fractured prior to a seismic event. On this basis, the use of cold-drawn mesh in the New Zealand construction industry is currently prohibited from structural elements that need to act as a diaphragm and from primary flexural reinforcement where there is a potential for yielding.

There are alternative methods and testing exercises which may be used to demonstrate there is a residual amount of strain capacity in the mesh. However, considering the overall dimensions of the floor and roof elements that are considered in the assessment, these alternatives are not deemed likely to result in different conclusions.

The presence of this cold-drawn (non-ductile) mesh is directly associated with the seismic scores of the roof diaphragm, the Level 2 diaphragm in the lobby area and the Level 4 promenade floor of the Auditorium. It also means that there is no reliable tie between the foundations of the Auditorium and Independent Staircase Structures. When exposed to the effects from lateral spread of the ground, the lateral stretch in the frame extensions of the bay walls of Bays 6(a) and 7(a) result in a structural weakness at this location.

The double-tee precast flooring typically needs to rely on the cold-drawn (non-ductile) mesh to resist its own inertia and transfer these into the supporting members. If the mesh has no capacity left, then the inertia of the double-tees needs to be transferred by other (less conventional) means.

The assessment has found that the double-tees are partially keyed into the cast insitu block-out area, as shown in Figure 8-2. This has been dimensioned on the original 1980's structural drawings to have a dimension of (110-85=) 25mm. However, this localised key may be subject to construction tolerances and is considered to be in the concrete cover area. When subject to lateral loading, the key may therefore be prone to premature spalling and provide an unreliable magnitude of resistance.

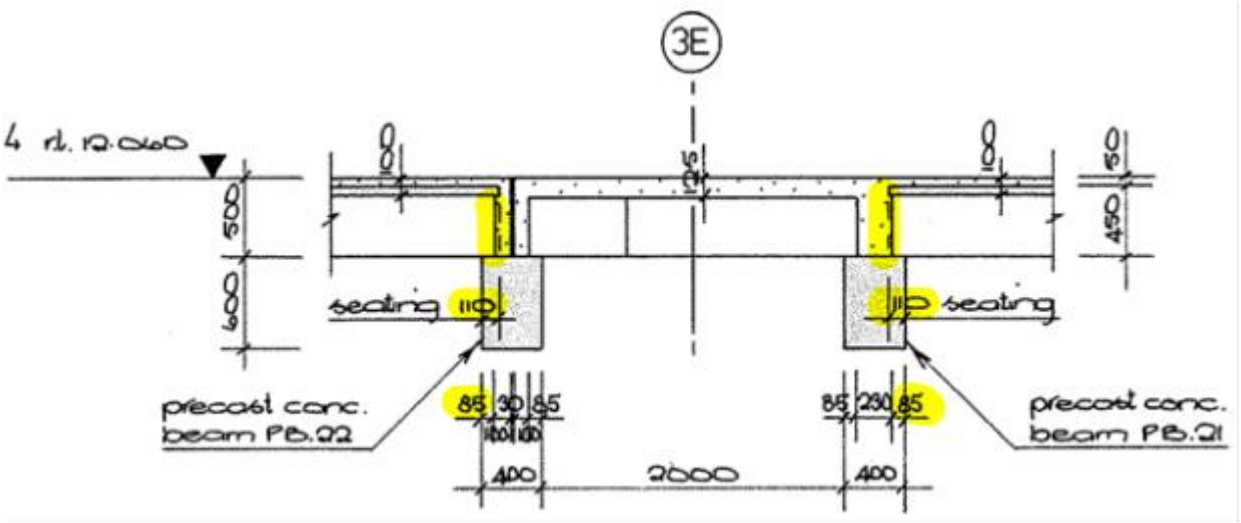

Figure 8-2: Example of keyed in situation of double-tee units at floor Level 4

The only manner of resistance that can be quantified, is a friction mechanism between the seating underneath the webs of the double-tees and the supporting beams. This mechanism has formed the basis of the assessment of these load paths and has resulted in a score of <33%NBS$_{IL3}$. However, this is likely to be a conservative lower bound, as there are other non-quantifiable mechanisms that could provide additional capacity. Also, the double-tee units have to displace quite a distance in the direction of sliding before the web areas lose their seating. The actual life-safety risk of these elements may therefore be underpredicted.

The seismic score of the bay walls is governed by the upper precast portion

The bay walls comprise two distinct portions. The upper portion is constructed from precast buttresses and precast wall panels. Between those elements are precast infill panels, which are considered as a non-structural component (see Section 8.4). The lower portion of the bay walls are cast insitu C-shaped walls that are typically regular across its height and regularly reinforced.

The lower portion between the ground floor level (Level 1) and the promenade roof (above Level 4) can form a plastic hinge in flexural behaviour – either located just above Level 2 or at the base at Level 1. This allows the bay walls to behave in a ductile manner to dissipate energy from seismic shaking. However, the potential for bar buckling limits the system to a relatively low ductility for cantilever walls of μ_{sys} = 2.0. The lower portion therefore has seismic scores of 50%NBS$_{IL3}$ at Bays 7(a) and higher at other locations.

The upper precast portion however is constructed of several parts, such as the precast buttresses and the precast wall panels. These parts rely on the connecting joints between them to resist the seismic loading. The joints are typically limited by shear capacities or other mechanisms of low ductility. They are found to be locally unable to deal with the overstrength demand of the ductile mechanism in the lower cast insitu portion of the bay walls and, as such, they were assessed using nominal ductile behaviour.

The weakest link is the horizontal interface between different precast wall panels. The panels are not connected along their entire horizontal interface. Instead, the load path is required to engage the two stitch zones which runs vertically along the precast buttresses and the two weld plates. The area that is engaged in the stitch zone is relatively small, which limits the shear capacity significantly. The weld plates are configured in a very brittle way, which does not provide a large contribution to the shear capacity either.

The loss of integrity of the upper precast portion of the Bay Walls mainly affects primary the lateral load resisting system, which results in excessive displacements to other structural elements and significant spalling and cracking of the concrete elements.

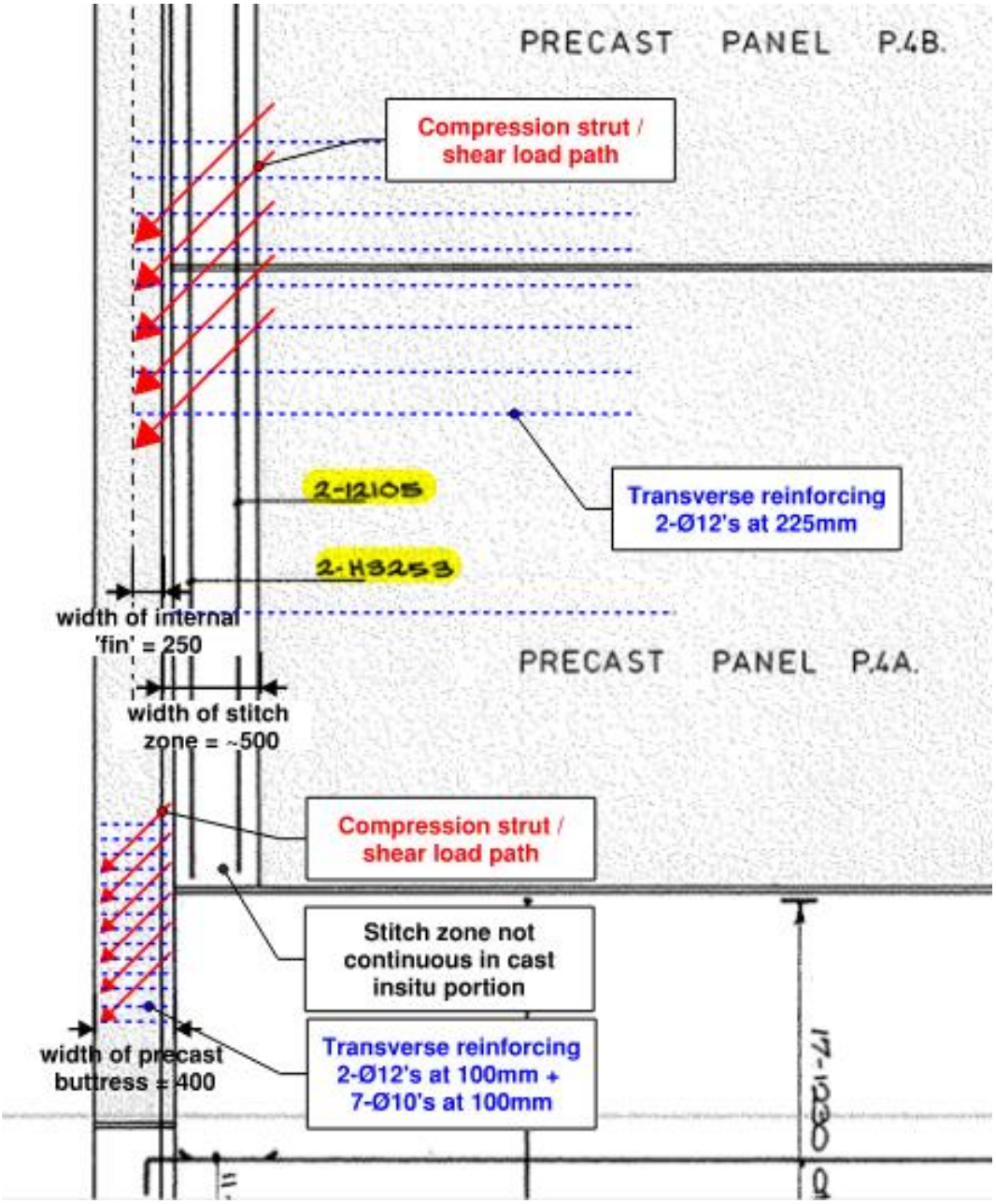

Figure 8-3: Extract from drawing S35B of the original 1980's structural drawing, highlighting the shear load path through the stitch zone and the precast buttress

The seismic score of the foundations is governed by the effects from lateral spread

The foundations of the Auditorium are relatively flexible compared to the mass of the superstructure, as the piles are relatively slender. Therefore, the structure is expected to displace as a rigid block above the ground. The flexibility of the foundation system results in a nonlinear behaviour which is assessed using a displacement-based assessment method. The foundations were assessed in isolation of any ductile behaviour of the primary structure as both systems have yield strengths of similar order.

The foundation system is potentially fragile at the four Ø900mm bored piles, as these are governed by a flexure-shear mechanism in both undegraded and liquefied ground conditions. Exceedance of the capacity of these bored piles in the foundation system may result in a loss of integrity of the primary lateral load resisting system at large levels of seismic shaking. These bored piles support Wall 21, which attaches to the sloped diaphragm at Level 1/2, and they also support the portion of the perimeter ring beam where it is interrupted by the basement tunnel structure. Wall 21 is a particularly critical element in the primary lateral load resisting system.

During initial undegraded ground conditions the piles and foundation beams can develop sufficient strength and displacement capacity to deal with the overstrength of the primary lateral load resisting system.

In liquefied ground conditions, the foundation system is more reliant on its displacement capacity as the strength of ground drops. A small number of the piles are potentially not founded in competent alluvium ground layers. Therefore, they can only nominally provide lateral capacity in liquefied ground conditions.

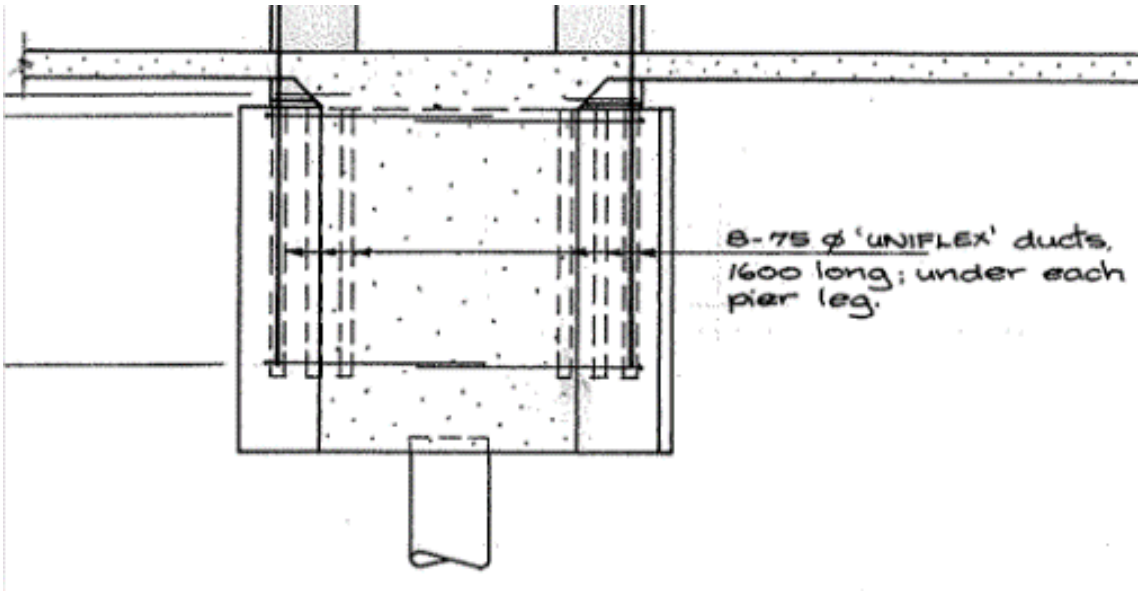

Figure 8-4: Snapshot taken from drawing S39B of the Holmes, Wood, Poole & Johnstone drawing set - showing a typical section of the perimeter ring beam and the pile

Effectively, they will drift with the displacements of the structure or see vertical settlement as a consequence of liquefaction. The remaining lateral capacity of the foundation system and the piles requires the structure to displace >200mm within the ground to take out the required base-shear of the superstructure. In liquefied ground conditions, combined with cyclic displacements and 80% of the base-shear, the foundation system is scored at 45%NBS_{IL3}.

For the assessment to the displacements imposed by lateral spreading of the ground, the piles are required to displace >250mm towards the free face which is near the lagoon (across Jervois Quay). These deformations exceed the displacement capacity of some of the piles, whilst others may still have a nominal lateral capacity. However, the vast majority of the piles are considered to retain their gravity load carrying capacity at these displacements. Any base-shear demand from the superstructure that is imposed on the foundation system can still be taken out by passive pressures against the foundation system. The assessment concluded that only a small additional displacement (~20mm) would be sufficient to generate the passive pressures to take out the base-shear demand.

During the probable ground displacements associated with the lateral spreading of the ground, the Ø900mm bored piles are likely to exceed their capacity and behave in flexure-shear. However, the lateral spread scenario only requires ground displacements to be combined with a relatively low base-shear demand of 25%. Under seismic loading at 45%NBS_{IL3} in liquefied ground conditions, this corresponds to ~10%ULS seismic loading in the undegraded ground conditions. This load may be tolerable by the structure without the participation of Wall 21, which may have exceeded the capacity of its piles at this level of shaking. A sensitivity study has been undertaken to compare the displacements and stresses in the structure without Wall 21 at 10%ULS seismic loading, and these are relatively similar (or less) to those corresponding to 45%ULS shaking with the inclusion of Wall 21. As such, for the low levels of seismic shaking during the lateral spread scenario, the bored piles Ø900mm can exceed their capacity, without leading to a significant life safety hazard. The gravity support of Wall 21 is maintained by a Ø450mm 'Franki' pile which is located centrally under the wall.

Therefore, the assessment scores the foundation system at 45%NBS_{IL3}, at which the life-safety criteria are met. However, the structure may use all its inelastic displacement capacity to achieve this score, leaving it damaged and likely unfit for further use.

Frame extensions of the bay walls of Bays 6(a) and 7(a)

The columns of the frame extensions of the bay walls at Bays 6(a) and 7(a) are part of the Auditorium structure and connected through the beams at Level 2 and Level 4. However, they are founded on the same foundations as the independent staircase structures, and they are only tied back to the Auditorium through the 100mm slab-on-grade which is reinforced with cold-drawn (non-ductile) mesh. This reinforcing cannot be relied upon to provide a tie force in accordance with the Engineering Assessment Guidelines. The differential lateral spread between the Auditorium and the independent staircase structures can therefore cause the effects of lateral stretch to have the two foundation elements displace independently relative to each other.

This displacement is expected to be taken out by flexure of the frames. The structural hierarchy of this frame was determined to be controlled by a column-sway mechanism, similar to the frames in the Renouf Foyer. A soft-storey mechanism may form, and the columns' inelastic displacement capacity will be exceeded following the lateral stretch between the structures. Considering that liquefaction and lateral spread could be considered a geotechnical step change in the behaviour of the ground, an additional assessment is required to determine if it also results in a structural step change.

The Engineering Assessment Guidelines provide additional requirements if a geotechnical step-change, such as a trigger of liquefaction, leads to a structural step-change. There is particular emphasis on structural step change that could lead to a significant life-safety hazard – requiring the seismic score of the element that causes this significant life-safety hazard to be half of the level of shaking (or corresponding displacement) that causes the step-change behaviour.

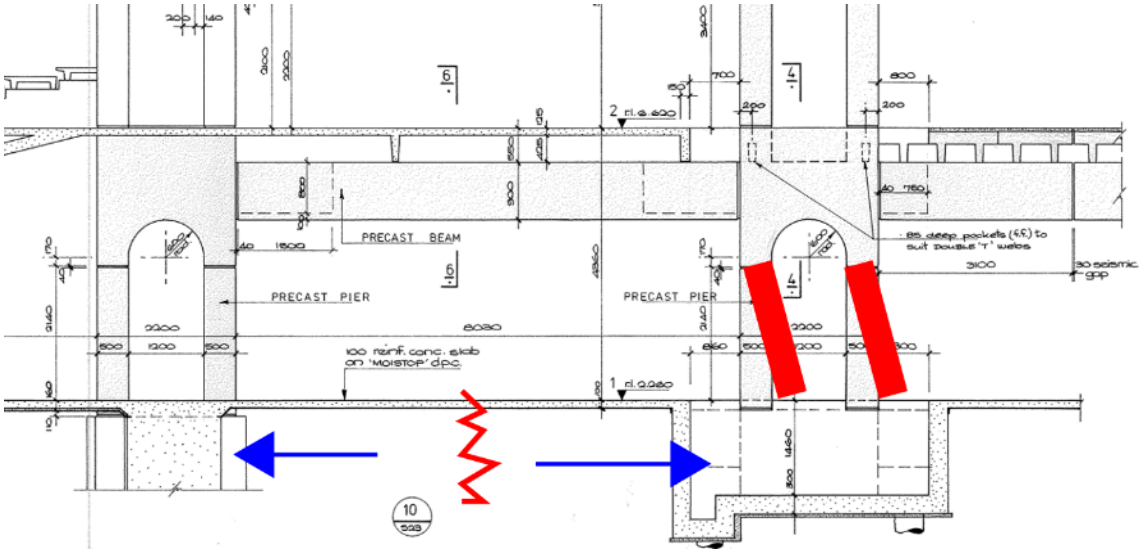

Figure 8-5: Illustration of the lateral stretch at the frame extensions of the bay walls at Bays 7(a), where the stretch at the ground floor level requires the columns at the independent staircase structures to take the differential movement in flexure

To assess whether an element is triggering a step-change behaviour in the structure, reference is made to the recommendations in the paper "*Application of the Geotechnical Step Change Provisions of the 2017 MBIE Guidelines for the Seismic Assessment of Buildings*" (Whitehurst, 2021). In this paper a simplified representation to assess the margin between a life-safety hazard risk and a collapse risk is discussed. This approach has been adopted for this DSA to demonstrate which structural elements (primary and secondary) are required to consider the halving of the seismic score. Figure 8-6 is used to illustrate this potential margin.

It is implicit (and in some instances explicit) in the Engineering Assessment Guidelines that there is a margin between the life-safety risk and collapse risk. This paper discusses that the intend of the step-change reduction (halving of the seismic score) is to quantify the risk for elements that lose that inherent margin at a geotechnical step-change.

For the purpose of this DSA, this margin between a life-safety risk and the collapse risk is considered as a factor of 2 consistently across all structural elements. Following discussions with Tonkin+Taylor, this approach was considered to align with the 'uncertainty factor described in their Geotechnical Assessment Report. It is our opinion that this an acceptable approach. However, it is noted that the Engineering Assessment Guidelines do not specifically cover this in detail for each mechanism that each structural element could form.

The lateral spread is considered to drift the Auditorium as an 'island' toward the lagoon. As the structure is unable to sustain the passive pressures acting on the Auditorium, it is expected that the structure moves along with the probable lateral spread on the northern end of the building – quantified by the Geotechnical Assessment Report (Appendix B) as 250mm at 45%NBS_{IL3}. The Renouf Foyer, the independent staircase structures and the foundation of the frame extensions of the bay walls of Bays 7(a), are also all expected to drift as independent 'islands'. However, these are expected to move to a lesser extent. For example, at the independent staircase structures, the probable lateral spread is only 175mm at 45%NBS_{IL3}.

The solid grey line shows the probable quantity of lateral stretch in the ground between the Auditorium and the foundation elements at the independent staircase structures. It shows that at the onset of liquefaction, at 45%NBS_{IL3}, the differential lateral spread between the foundations of the Auditorium and the

independent staircase structures is 250mm – 175mm = 75mm. At larger levels of seismic shaking, this quantity is larger.

The dashed grey lines indicate the level of uncertainty in the predictions from the Geotechnical Assessment Report, showing a factor of ½ and 2 on the probable quantities of lateral spread on the site.

The columns of the frame extensions of the bay walls at Bays 6(a) and 7(a) were concluded to have a total displacement capacity of 68mm. This is indicated by the bottom orange line and by the term 'δ_{SLSH}' at the intersection with the solid grey line. This point indicates the displacement associated with a significant life-safety hazard risk.

Through the implicit margin to collapse risk within the Engineering Assessment Guidelines (factor of 2 assumed, as per description above), the upper orange line can be determined. This gives a second point at the intersection with the grey line, shown by the term '$\delta_{collapse}$'.

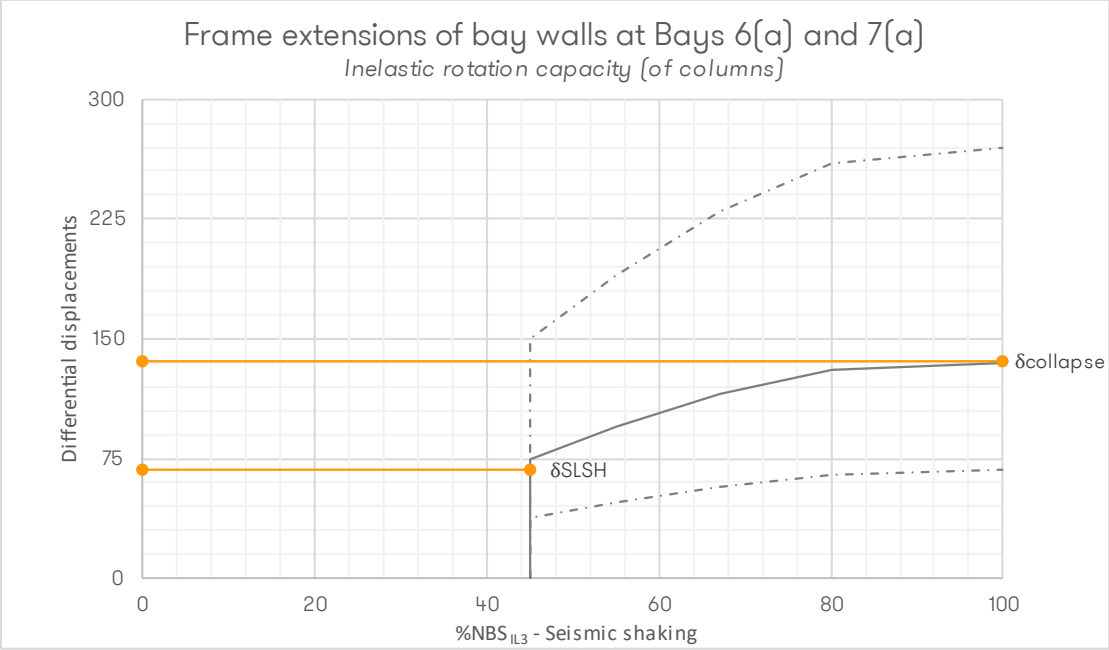

Figure 8-6: Assessment of extended frames at Bays 6(a) and 7(a) against the effects of lateral spread

To assess each structural element of the Michael Fowler Centre that is subject to the effects of lateral spread, this graphical representation can then result in three possible conclusions:

- Both the displacement capacity of the life-safety criteria 'δ_{SLSH}' and the collapse criteria '$\delta_{collapse}$' intersect the graph of the lateral spread past the initial vertical portion. This indicates the geotechnical step-change does not affect this structural element. The seismic score of the structural element does not need to be halved.
- The displacement capacity of the life-safety criteria 'δ_{SLSH}' meets the graph of the lateral spread amount in the vertical portion, but the displacement capacity of the collapse risk '$\delta_{collapse}$' intersects in the diagonal portion. This indicates that there is a residual margin between the life-safety risk and the collapse risk. As such, there is a geotechnical step-change, but this does not result in a structural step-change. The seismic score of the structural element does not need to be halved.

- Both the displacement capacity of the life-safety criteria 'δ_{SLSH}' and the collapse criteria '$\delta_{collapse}$' intersect the graph of the lateral spread amount in the vertical portion. This indicates that there is no nominal margin between life-safety risk and the collapse risk. This indicates a structural step-change which could cause a significant life-safety hazard following a geotechnical step-change, and, therefore, the seismic score of the structural element must be halved.

As can been seen in Figure 8-6, the lateral stretch on the ground floor may induce displacements that provide a margin between the displacement capacity for a life-safety hazard and that for a (partial) collapse scenario. The lateral spread associated with seismic shaking at 45%NBS_{IL3} therefore does not provide the lateral stretch to exceed the displacement capacity, and there is a margin (factor of 2) to collapse. Consequently, the seismic score of the extended frame does not need to be halved, however the score is limited at 45%NBS_{IL3}.

A similar approach is used to assess the consideration of potential step change at other interfaces in the building; however, this location at the frame extensions of the bay walls at Bays 6(a) and 7(a) were found to be the only locations where the lateral spread governs the seismic rating, other than the foundation system.

8.1.2 Commentary on elements that score less than 34%NBS_{IL3}

In accordance with the EPB Methodology, the following statements elaborate on the mode of failure and physical consequences of the structural weaknesses which result in the building being classified as potentially Earthquake-prone.

Sloping transfer diaphragm below the Level 2 seating (referred to as Level 1/2)

The primary lateral load resisting system is reliant on the sloping transfer diaphragm at Level 1/2 to redistribute the lateral load from seismic shaking from the discontinuities at the bay walls of Bays 6(a) and 7(a) down to the squat walls 16, 18 and 21 at Level 1. At large levels of seismic shaking this sloping floor diaphragm could see the ties exceed their tensile capacity at which the floor may crack and soften, becoming relatively more flexible as a result.

If this occurs, the primary lateral load resisting system will attempt to redistribute the lateral load from seismic shaking to other (still relatively stiff) load paths throughout the structure. However, it needs to do so through the Level 2 promenade floor in the lobby area of the Auditorium. Whilst this is initially a stiff load path, it does not have a lot of strength as the diaphragm is reinforced with cold-drawn (non-ductile) mesh. It could make the global structural response unpredictable and result in excessive displacements in areas that are not capable of dealing with these displacements (i.e., precast flooring).

Therefore, the assessment has concluded that the sloping transfer diaphragm at Level 1/2 is an integral part of primary lateral load resisting system and exceedance of the capacity, at a seismic score of 20%NBS_{IL3} could lead to a life-safety hazard.

Level 2 promenade floor diaphragm

In the transverse direction, the analysis model of the Auditorium shows that the building wants to act as a deep beam, bending between the walls at Bays 1(a) in the north and the squat walls 16, 18 and 21 in the south. The system relies on the cast insitu slab around the promenade at Level 2 to restrain that as a large tie. Due to the plan eccentricity (the discontinuity of the bay walls at Bays 6(a) and 7(a), there is an additional tie demand on this area.

The promenade has significant tie capacity across Bays 3(a) and 4(a) where they are a series of at least 14-H32 diameter bars of the higher reinforcing grade, which provide the tie capacity. Further to the south, across Bays 5(a), the reinforcing reduces significantly to a series of Ø24 bars of the lower reinforcing

grade. These provide a much lower tie capacity and this transition area is considered to be the governing structural weakness of the Level 2 diaphragm.

Exceedance of this tie capacity will force the structural system of the Auditorium to redistribute the lateral load to secondary load paths throughout the structure. However, it needs to do so through elements with low or unreliable capacity, such as the Level 2 promenade floor in the lobby area of the Auditorium. Whilst this is initially a stiff load path, it does not have a lot of strength as the diaphragm is reinforced with cold-drawn (non-ductile) mesh. It could make the global structural response unpredictable and result in excessive displacements in areas that are not capable of dealing with these displacements (i.e., precast flooring).

Roof trusses

During seismic shaking, the bay walls impose drifts on the roof trusses as they displace. The roof trusses have a tolerance against this movement, as there is some space between them and the walls. However, at large levels of seismic shaking, these tolerances may be fully utilised. The drifts could then result in additional forces in the roof trusses as they try to stay compatible with the deformed shape of the bay walls. When these additional loads start to exceed the capacity of the roof trusses, they may lose their ability to carry the gravity loads and the support of the Dycore roofing units.

Roof diaphragm

Section 6.4 describes some of the recent high-level assessments that have been undertaken in 2018 and early 2022 on the capacity of the roof diaphragm. Through micro and macro strut and tie analysis, the high-level assessments have attempted to demonstrate that the roof diaphragm could be scored above 34%NBS_{IL3}. However, these analyses have been undertaken in isolation and did not consider the global behaviour of the structure. Also, the analysis relied on assumptions associated with the capacity of the cold-drawn (non-ductile) mesh, the roof trusses or the rigidity of the perimeter buttress walls from the 2014 DSA to justify the simplifications made in the analysis.

The assessment of the roof diaphragm for this DSA used the output of the global analysis model as a starting point and supplemented this with more detailed understanding of the roof trusses (and their connections) and the perimeter buttress wall systems. In general, the assessment of the roof diaphragm identified three potential structural weaknesses:

- Exceedance of axial loads in the top chords of the roof trusses, especially at the connections with the precast concrete wall panels. The roof trusses could be relied upon to provide a tie capacity, however, there is a limit to that capacity, especially if the system relies on the capability of the roof trusses to transfer any loads in the wall panels.

- Excessive 'stretch' of the diaphragm as it tries to 'open up' from the flexural behaviour – relying on the perimeter buttress walls' in-plane strength to form a tie. There is some tolerance to allow strains in the topping, however this should be limited to ensure the Dycore floor units don't lose their seating and the chord stiffeners do not exceed the total displacement capacity within the bolted connection (ply bearing and bolt slip).

- Excessive out-of-plane loads on the Bay Walls (in-plane for the buttresses) as this would likely lead to other load path related issues down the walls. As the diaphragm would crack and soften, there would be more reliance on the precast buttress columns and the buttress walls to provide restraint. However, this is paired with large displacements which would result in additional demands imposed on the Level 4 promenade floor. Also, there would be excessive tie forces in the Level 2 and Level 1/2 diaphragms to try and hold the system together.

The assessment has considered various sensitivity studies in the analysis of the roof diaphragm. However, we have been unable to demonstrate a probable scenario which accounts for all these three potential structural weaknesses achieving a seismic score above 34%NBS$_{IL3}$ without engaging a tie capacity in the cold-drawn (non-ductile) mesh. Qualitatively, there are a number of global load paths available, meaning that a sudden loss of lateral capacity overall – from a single point of failure – is unlikely. However, collectively, none of the load paths have sufficient reliability to conclude a rating above 34%NBS$_{IL3}$ without risk of localised or larger collapses from loss of lateral capacity or inability of the hollowcore slabs to accommodate the implied strains and displacements.

On this basis, the assessment has concluded a seismic score of the roof diaphragm of 20-30%NBS$_{IL3}$.

Double-tee flooring at Level 2 in the lobby area and at the Level 4 promenade

The flooring in the lobby area at Level 2 and in the Level 4 promenade comprises double-tee units with a concrete topping, where the topping is required to function as a diaphragm. However, the topping is reinforced with cold-drawn (non-ductile) mesh. Therefore, there are many locations in these floor areas where no reliable tie forces can be formed through the reinforcing. The flooring will essentially be reliant on the individual concrete elements (i.e., double-tee units) to bear against each other to transfer the lateral loads from seismic shaking. At large levels of seismic shaking, this could cause the double-tee units to rattle loose.

Some movement may be allowed; however, the quantity of the subsequent movement is hard to predict. Therefore, this friction interface is typically not used in design of new buildings as a reliable method to secure precast flooring.

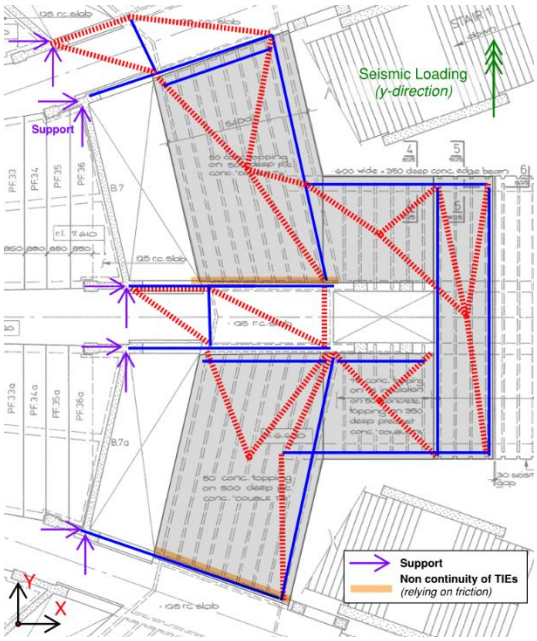

Figure 8-7: Sketch of the macro strut and ties forming across the lobby area at Level 2

The assessment has attempted to evaluate the probable behaviour of these floor areas through a macro strut and tie approach, which has predominantly considered that compression struts can be landed at various interfaces where elements are bearing directly on other elements. However, all attempts relied on areas where the only tie capacity can be generated by friction between the webs of the double-tee units and the supporting beams.

The friction capacity is generated by the self-weight of the double-tee units. However, the assessment has found that this friction capacity is only capable to sustain levels of seismic shaking at a seismic score of 25-30%NBS$_{IL3}$. Beyond that, the double-tee units are likely to move independently. The movement puts internal forces into the double-tee units that they are not designed for, and the friction interface can cause spalling to the units which may result in loss of seating.

These friction load paths will also need to accommodate the lateral loading from the adjacent structures when the Auditorium is subject to pounding with the Renouf Foyer (lobby area at Level 2) and with the Independent Staircase Structures (Level 4 promenade). Both these interfaces are expected to experience pounding at levels of seismic shaking between 10-20%NBS$_{IL3}$.

As a result, the seismic score of these flooring is 20%NBS$_{IL3}$.

8.2 Renouf Foyer

The following structural weaknesses have been identified in the primary structure of the Renouf Foyer.

Table 8-4: Structural Weaknesses identified in the assessment of the primary structure of the Renouf Foyer

Building Element	Structural Weakness	%NBS$_{IL3}$
Level 2 diaphragm	Exceedance of tie capacities in the diaphragm (665-gauge non-ductile mesh)	25
Kitchen extension stitch joint	Insufficient displacement capacity of connection between original Renouf Foyer frames and the 1989 kitchen extension	55
Roof trusses	Both imposed displacements and inertial loads limit the capacity due to: ▪ Deformation/drift compatibility of transverse frames with Renouf Foyer columns ▪ Connections of transverse trusses to coupled column units under longitudinal seismic loading.	55
Coupled column units	Exceedance of inelastic rotation capacity of columns at the base, as a soft-storey forms at the ground floor level.	60
Foundation system	Flexure-shear of piles in undegraded ground conditions	70
Kitchen extension frames	Exceedance of inelastic rotation capacity of hanger beam-column connection when displacing southwards.	80

8.2.1 Commentary on seismic performance

The lateral load resisting system has a complex hierarchy within beam-column joints

The unique detailing of the beam-column joints of the precast concrete coupled column units provides several mechanisms at approximately the same seismic score. It is complex to identify which plastic mechanisms within the structure would form first under lateral load from seismic shaking.

In the north-south direction, energy is expected to be dissipated primarily through plastic deformations concentrated between the ground floor level (Level 1) and Level 2 due to the large weight of the suspended floor dominating the response. Effectively, this constitutes soft-storey behaviour. The detailing of the reinforcement and the unconventional internal load paths are combined with variability in material properties, foundation stiffness and effects such as strain-hardening. This meant that three possible mechanisms are possible with a similar likelihood:

- Column hinging
- Beam hinging – locally initiated by foundation rotation
- Localised failure of joint reinforcement.

Any of these three, or a combination thereof, could develop in the frames of the Renouf Foyer. All three mechanisms were found to have a ductility capacity of between μ=2.0-3.0 before a significant life safety hazard could occur.

In the east-west direction, the columns act in groups of four, where pairs of coupled column units are connected by cast insitu link slabs. The link slabs located above Level 2 cannot be relied upon to dissipate much energy from seismic shaking because of their unique eccentric configurations. They connect directly to coupled column units, as is expected in a more conventional frame system. The link slabs will instead twist the coupling beams spanning in their perpendicular direction. This load path was found to be a potential structural weakness during seismic shaking. With similar variability as the north-south direction, two possible mechanisms are possible:

- Ground floor column-sway – leading to a soft-storey mechanism.
- Failure of the mid-height link slab elements – forming a beam-sway mechanism with low ductility from the torsion/twist of the link slab.

For both the transverse (north-south) and the longitudinal (east-west) direction, the lowest scoring elements of the primary structure are identified as the ground floor column-sway mechanism (ground floor soft-storey). This resulted in the mechanism with the lowest displacement capacity and, subsequently, the lowest seismic score.

The foundation system can accommodate the differential lateral stretch

The foundation system of the Renouf Foyer comprises effectively three 'islands' that could move independently, if they are exposed to large passive pressures from lateral spreading of the ground. There are 'islands' on the eastern and western ends of the Renouf Foyer, that tie together four groups of coupled column units. Then there is also a third 'island', along the central gridline C, that supports only two of the intermediate coupled column units.

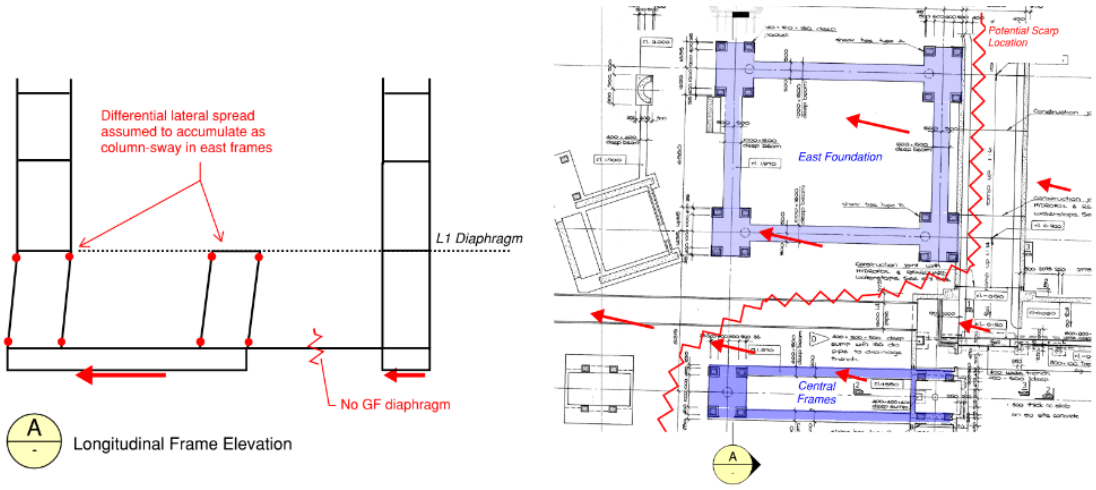

Figure 8-8: Potential location of surface scarp due to lateral spreading that could impose displacements on the Renouf Foyer superstructure

The differential lateral spread across the site could drag these 'islands' apart as passive pressures build up against the sides of the foundation beams and the basement structure. The piles are unable to resist these loads. The Renouf Foyer does not have a ground floor diaphragm that could limit differential ground movement between foundation elements if liquefaction leads to lateral spreading or ground lurch in the liquified soil beneath the structure.

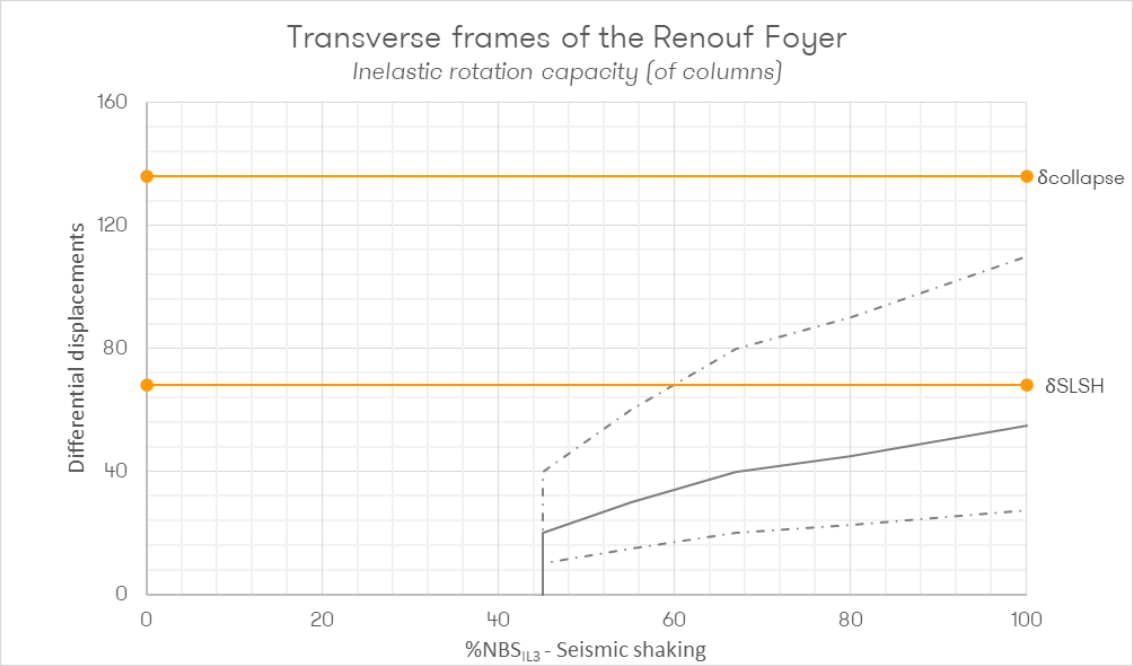

Figure 8-9: Assessment of Renouf Foyer columns, due to differential lateral spread

Because the system is restraint at Level 2 by a rigid diaphragm, the imposed actions from the lateral stretch require the system to form a column-sway soft-storey. These columns have an inelastic rotation capacity of 68mm. This is considered the displacement capacity associated with a significant life-safety hazard 'δ_{SLSH}'.

The assessment against a potential structural step change follows the same methodology as outlined in Section 8.1.2 for the frame extensions of the bay walls at Bays 6(a) and 7(a). However, for the location of the Renouf Foyer, the differential lateral stretch is 20mm at the onset of liquefaction at 45%NBS_{IL3}, shown by the solid grey line on Figure 8-9.

It shows that the foundations can accommodate the expected amount of differential lateral spread, even when looking at larger levels of seismic shaking.

The connection of the Kitchen Extension frames has low resilience against seismic shaking

The kitchen extension provides limited additional capacity to the primary lateral load resisting system of the Renouf Foyer. Therefore, it must have displacement compatibility to displace along with the Renouf Foyer under lateral load from seismic shaking which will develop forces through the stitch connection.

The kitchen extension frames are connected into the existing Renouf Foyer using epoxied reinforcing bars which extend the existing cantilever beams on the southern side of the Renouf Foyer to form the additional kitchen extension frame bay. This connection into the existing Renouf Foyer frames was found to have limited capacity which could result in a localised structural weakness at the connection as the structure displaces in the north-south direction. The ability of the epoxied bars to undergo localised deformation, either due to bar yielding or concrete pull-out, whilst still supporting gravity loads, is limited and could pose a significant life-safety hazard.

Roof trusses have limited displacement compatibility

The roof trusses were originally designed for gravity and wind actions only. As such, no consideration was made for the for the displacements in the precast coupled column units that may be experienced from seismic shaking. The roof trusses spanning in the north-south direction connect directly into the coupled column units and will be forced to similar rotations as the structure displaces. The trusses ultimately exceed their buckling capacity and were found to pose a significant life safety hazard when drifts exceeded 1.0%. The connections between the roof trusses and the coupled column units were found to be a structural weakness at similar drifts.

8.2.2 Commentary on elements that score less than 34%NBS$_{IL3}$

In accordance with the EPB Methodology, the following statements elaborate on the mode of failure and physical consequences of the structural weaknesses which result in the building being rated at less than 34%NBS$_{IL3}$.

Level 2 diaphragm

The Renouf Foyer originally had a section of floor between gridlines B and D adjacent to the kitchen extension on gridline 8, cantilevering around 2.5m. As part of the kitchen extension, the perimeter slab edge beam was offset to accommodate the cantilevered floor spans. This has created an eccentric load path for any tie forces required to transfer lateral load into the adjacent intermediate frames in longitudinal (east-west) direction.

The central double height frames support approximately half the total weight of the roof structure in addition to the self-weight of the surrounding floor slab. Lateral forces occurring due to the seismic mass under earthquake shaking will largely have to be transferred through the Level 2 diaphragm to be resisted by the adjacent intermediate frames due to the following structural features:

- At the northern end of the central double height frames, there is no foundation beam running in the east-west direction. This means that the set of coupled column units are essentially free to rotate on the pile cap – ultimately, reducing the amount of lateral load they will resist.
- At the southern end of the central double height frames, there is a penetration in the floor at Level 2 for the service lift. This means the columns at the southern end see a discontinuation of 550m deep cast insitu slab which is located at the other coupled column units.

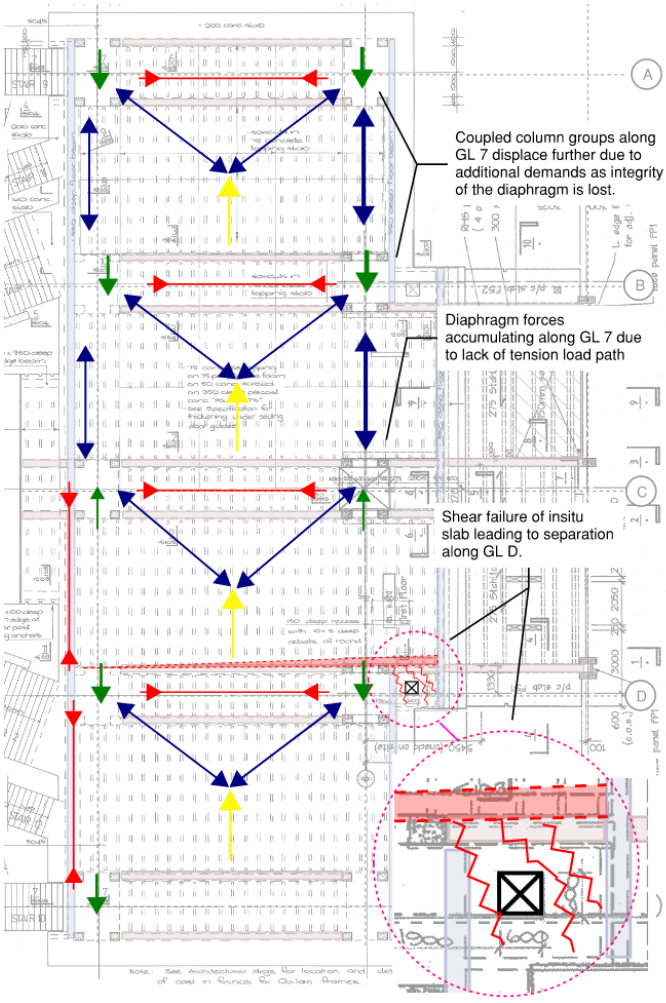

Figure 8-10: Qualitative strut & tie analysis showing failure mechanism along gridline 8 due to offset slab edge beam.

The estimated load paths are shown in Figure 8-10 for inertia from seismic shaking towards the east. Though the building is symmetrical and will act similar in the opposite direction.

For this direction, only a nominal tie can form along gridline 8 back to gridlines D and E, as the lateral load must be transferred through an eccentric load path. And this load path also has an opening in it. The capacity of this tie would be exceeded at seismic shaking of 25%NBS_{IL3}.

Instead, the lateral load would have to be taken solely through compression struts into the adjacent frames on gridline 8 (at gridlines A and B).

The exceedance of this tie would result in unequal displacements of the coupled column units at each end of the Renouf Foyer. For the loading in this direction, the four coupled column units on the eastern end would see more mass than those at the western end. The diaphragm is otherwise reinforced with cold-drawn (non-ductile) mesh, and, therefore, no other ties can form.

It would result in a separation (large crack) along gridline D (and gridline B upon load reversal). And the diaphragm would lose its overall integrity.

The double tee flooring units have been previously retrofitted with catch brackets; however, the original seating capacity of the units is expected to be exceeded which would lead to the flooring units dropping onto the catch brackets. This could lead to 'frame elongation' effects, where upon load reversal the flanges of the double tee units will begin to bear against the supporting beam leading to crushing of the precast units.

Due to limited capacity and lack of a redundancy in the diaphragm to accommodate such a mechanism, the diaphragm was given a seismic score of **25%NBS_{IL3}**.

8.3 Independent Stair Structures

The following structural weaknesses have been identified in the primary structure of the Independent Staircase Structures.

Table 8-5: Structural Weaknesses identified in the DSA

Building Element	Structural Weakness	%NBS_{IL3}
Walls in out-of-plane direction	Exceedance of joint shear capacity as a beam-sway mechanism degrades the joint shear capacity	40
Foundations in in-plane direction of the walls	Exceedance of tension capacity of piles, resulting in the exceedance of 2.5% drift limit	70

The Critical Structural Weakness was found to be walls

The Critical Structural Weakness (CSW) is the lowest scoring structural weakness, the failure of which would result in a significant life safety hazard. The CSW was found to be the beam-sway mechanism that forms in the walls of the Independent Staircase Structure.

8.3.1 Commentary on seismic performance

Beam-sway mechanism in transverse direction

In the transverse direction of the independent staircase structures, a beam-sway mechanism forms through flexure in the floors.

The structure has a relatively high yield drift capacity as it is relatively flexible and plastic hinges are expected to form in the floor near the walls. However, as the plastic hinges are formed, the wall-floor joints degrade rapidly as plastic hinges are formed.

The wall-floor joints are minimally reinforced, with no specific joint reinforcement. They are external joints with the reinforcement configured as in 'Case 2' of Figure C5.27 of the Engineering Assessment Guidelines. The subsequent section of the Engineering Assessment Guidelines then prescribes that there is only 0.15% inelastic drift capacity for external joints with hooked bars bent away from the joint.

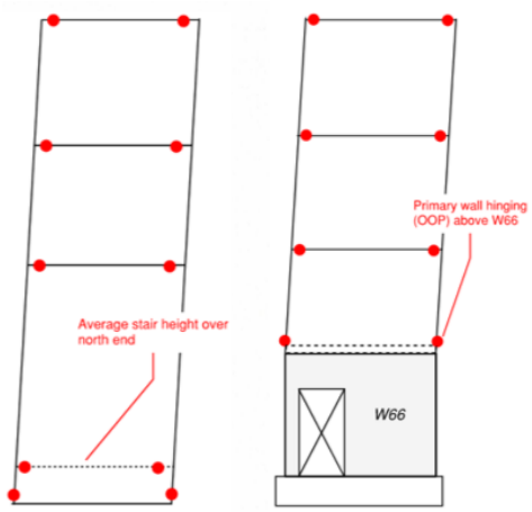

Figure 8-11: Illustration of beam-sway mechanism in the walls of the independent staircase structures

8.4 Secondary Structural and Non-Structural Elements

The secondary structural and non-structural (SSNS) aspects that were considered in this assessment are listed in Table 8-6. Consideration includes a decision to include or exclude these elements from an overall seismic score. The intent of the EPB Methodology is to identify and rate those building parts which, should they lose support or collapse, would present an unavoidable danger that a number of people are exposed to. Parts and Components loading was calculated per Chapter 8 of NZS1170.5:2004, refer Appendix G.

We have applied Section 2.4.1 of the EPB Methodology, using the guidance in Section A4.3.2 of the Engineering Assessment Guidelines, to determine which parts are included or excluded from the rating.

The scope of this assessment has excluded the assessment of the mechanical plant and other services.

Building contents are not considered in this assessment. The following building contents were observed on site:

- seating in the Auditorium
- office furniture at ground floor level
- kitchen equipment at ground floor level
- light fixtures and decorations
- Māori Pouwhenua sculptures

Table 8-6: SSNS included or not included in the Assessed Seismic Rating

Building Part	Included?	Reason for decision to include/exclude
Precast infill cladding panel at façade of Auditorium	Yes	Heavy element above Space Class I
Reinforced concrete masonry block walls	Yes	>25kg/m^2 partition walls bordering Space Class I, II and III
Light-weight façade and glazing system around promenade	Yes	Support frames that could fall into Space Class I

Building Part	Included?	Reason for decision to include/exclude
Lightweight arching roof above Renouf Foyer	No	Light-weight cladding elements
Catwalks	Yes	Considered a heavy (>25kg) horizontal cantilevering element above Space Class I
Wall reflectors	Yes	Considered a heavy (>25kg) 'sign or billboard' with a fall height greater than 3m onto Space Class I
Stage reflector ('Donut')	Yes	Considered a heavy (>25kg) vertical cantilevering element above Space Class I
Rigging points in the Auditorium and Renouf Foyer	Yes	Heavy plant that could lead to the element falling onto the building occupants
Stairs	Yes	Above Space Class I Providing emergency egress paths
Suspended portion of the light-weight timber stage	No	Above Space Class V
Suspended light-weight timber ceiling underneath Level 4 cantilever floor segments	No	Lightweight ceiling systems above Space Class III
Movable stage lift	No	Above Space Class V
Ceiling panels in Auditorium	Yes	Potentially unrestrained ceiling tile (>7.5kg) above Space Class I
Operable walls in Renouf Foyer	Yes	Considered a heavy (>25kg) element above Space Class I

The following structural weaknesses have been identified in the secondary structural and non-structural elements.

Table 8-7: Structural Weaknesses identified in the assessment of the primary structure of the Auditorium

Building Element	Structural Weakness	%NBS$_{IL3}$
Catwalks structures	Exceedance of capacity of in-plan bracing.	20-30
Wall reflectors	Concrete break-out from excessive forces on the post-installed anchors	20
Promenade roof & façade system	Exceedance of deformation capacity of existing glazing suite	25
Precast infill panels between the upper precast portion of the bay walls	Limited displacement compatibility with the primary structure of the Auditorium	25
Stairs 15/16	Loss of integrity of the landings at large inter-storey drifts.	35

Building Element	Structural Weakness	%NBS$_{IL3}$
Stairs 11/12	▪ Exceedance of flexural capacity from in-plane loading. ▪ Lack of displacement compatibility of lower and upper stair flight sliding connections. ▪ Differential ground movement from lateral spreading of the ground exceeding the seating capacity.	40-45
Stairs 7/8	Differential ground movement from lateral spreading of the ground exceeding the seating capacity	45
Stairs 13/14	Integral cast insitu connection leading to excessive flexure of stair flight under imposed displacements.	45-60
Concrete masonry block walls	▪ Exceedance of out-of-plane capacity of block walls at Level 4	50
Stairs 9/10	Exceedance of both displacement capacity and structural capacity at comparable seismic demand due to: ▪ Lack of displacement compatibility of level 2 sliding connection. Exceedance of flexural capacity of lower stair flight due to significant mass eccentricity under inertial loads.	50
Rigging points in the Renouf Foyer	Exceedance of flexural capacity under 'Parts and Components' loading	70
Yellow Chapter C5 checks for precast flooring	Loss of seating due to excessive drifts of the primary structure (limited at the double-tee units in the lobby area at Level 2, all other locations have a score of >100%NBS$_{IL3}$)	80

8.4.1 Commentary on elements that score less than 34%NBS$_{IL3}$

Catwalk structures

The catwalk structures that run along the bay walls of the Auditorium have minimal bracing in plan that would restrain them during seismic shaking. The system relies on two bays of bracing profiles. This bracing is insufficient to deal with the seismic shaking in the longitudinal direction of the building.

As the bracing capacity of the catwalk structures is exceeded, the system may be subject to excessive displacements as there is no secondary load path that could restrain the system. This could lead to elements of the catwalk pounding into each other, which could result in damage and potentially loss of vertical support.

Following several site inspections, the catwalk structure was visually audited, and it observed that the catwalk structures had been altered relative to the original 1980's drawings. There are several changes to the bracing system, to the vertical hangers that carry the walkway and to the framework that supports the reflector screens. These changes have not improved the overall strength of the catwalk system as some of the new detailing has introduced eccentricities in the connections. Photos of these observations are included in Section G.5.1

A selection of the new connection details has been considered in the seismic score of the catwalk system and resulted of a score of 20-30%NBS$_{IL3}$. A range has been given for the seismic score, as there are uncertainties in the post-yield performance of the catwalk system.

Wall reflectors

From seismic shaking in the transverse direction of the wall reflectors, the connections into the buttresses of the bay walls may be subject to concrete break-out. The system relies on a steel framework at the bottom and steel threaded rods at the top to provide gravity support and to restrain it against movement during seismic shaking.

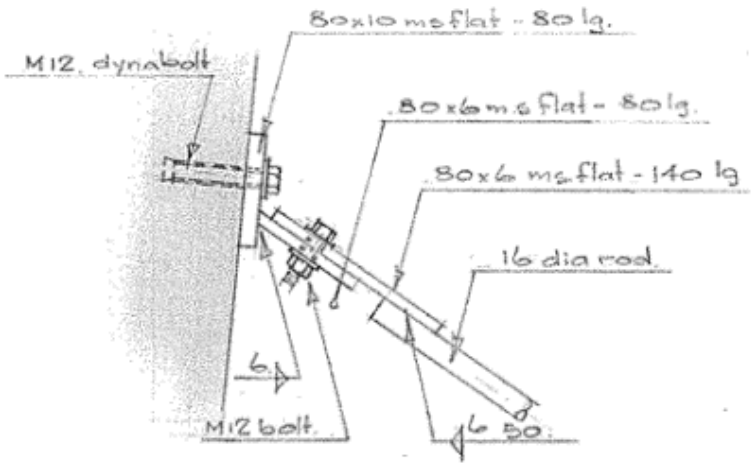

Figure 8-12: Connection detail of upper tie rod of the wall reflectors. Where it connects to the cast insitu buttresses of the bay walls

The structural hierarchy of the system predicts that the steel framework at the bottom of the wall reflectors yields early during seismic shaking; however, this does not result in a life safety hazard. The critical structural weakness is the upper connection of the threaded rod, shown in Figure 8-12. Here the connection exceeds the concrete break-out capacity of the shallow embedded anchor at 20%NBS_{IL3}, and there are similar ratings for the shear in the M12 bolt and in the 6mm fillet weld.

Promenade roof and façade system

Along the perimeter of the building is a light-weight façade and roof system comprising glulam timber beams, a curved stainless steel cladding panel, stainless steel mullions and a glazing suite. The structural system of this roof relies on the integrity provided by the plywood diaphragm on the top of the roof. This drags any loads from seismic shaking back to the bay walls of the Auditorium or the coupled column units of the Renouf Foyer. It provides stability to the curtain wall glazing suite in the circumferential direction and restrains the roof in the radial direction.

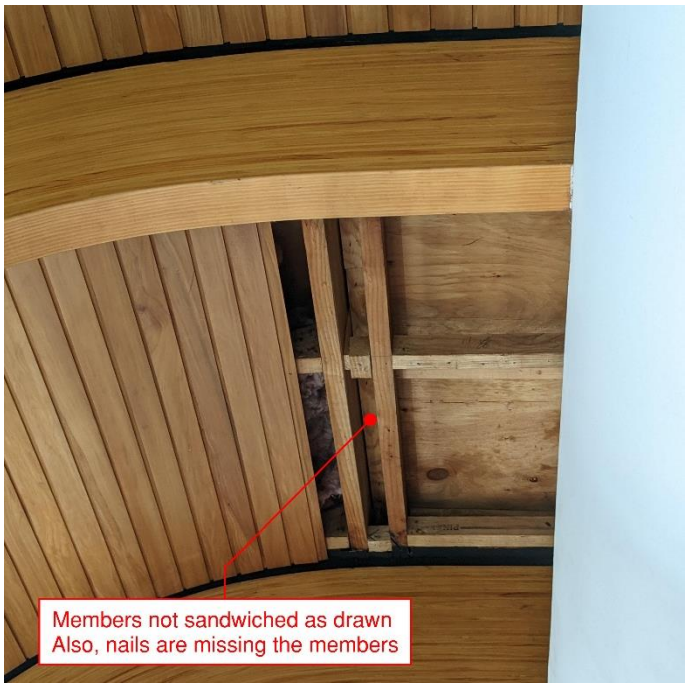

Figure 8-13: Photograph from site visit - showing the discontinuity in the plywood diaphragm

The plywood diaphragm is not continuous across the roof, as it undergoes various steps at the gutters and at the curved transition to the stainless-steel mullions. Intrusive investigations were undertaken to visually observe the built-up of the plywood diaphragm. The original 1980's structural and architectural drawings do not specify the structural design of the timber elements of the building and so there are some uncertainties on the exact configuration.

The investigations verified that the plywood diaphragm is not continuous across the different steps. It also showed that the timber nailing is at relatively large spacing, and, in some cases, missing the specified members. This was particularly observed at the transition to the curved portion of the light-weight roof.

As a result, the assessment has considered the plywood diaphragm to terminate at the start of the curved portion of the roof. This removes considerable stability from the system and the only structural member that could resist any displacement in the circumferential direction is the curved glulam beam, in torsion. This is a relatively flexible system and timber in torsion can also be very brittle.

The critical structural weakness is found in the curtain wall glazing suite. The curved glulam beams are so flexible in torsion that their displacement under 25%NBS$_{IL3}$ will exceed the displacement capacity of the glass panels.

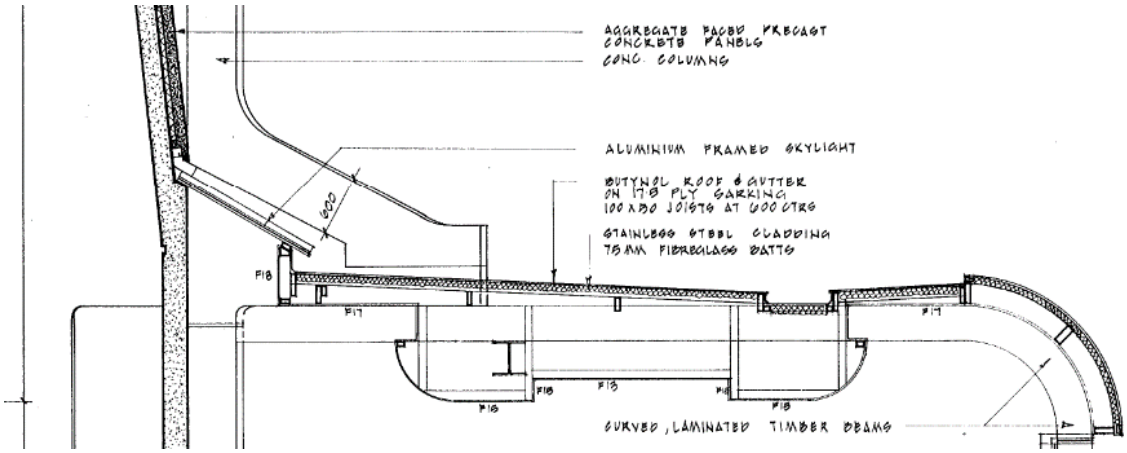

Figure 8-14: Built-up of light-weight roof system around the building

The curtain wall glazing suite is assessed as a secondary structural element, as it encompasses a large portion of the building, spans across multiple stories and is located over a Space Class I. Therefore, it does not meet the criteria of a 'lightweight cladding element' in accordance with Table A4.1 of the Engineering Assessment Guidelines and needs to be considered as a potential significant life safety hazard.

Precast cladding infill panels

Around the upper precast sections of the Auditorium, several large precast cladding panels are located. They fill the spaces between the bay walls. The infill panels are attached to the precast buttress at the ends of the bay walls using connections with bolts in slotted holes. The bolts provide the gravity support to the infill panels.

During seismic shaking, the bay walls impose drifts on the infill panels as they displace. The slotted holes allow some tolerance against this movement. However, at large levels of seismic shaking, these tolerances may be fully utilised. The drifts could then result in excessive forces in the bolted connections, which may cause them to exceed their capacity. This may result in the bolts losing their capacity to provide gravity support to the infill panels.

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9 RECOMMENDED NEXT STEPS

The Detailed Seismic Assessment (DSA) of the Michael Fowler Centre has identified a number of structural weaknesses that rate the building as potentially earthquake prone.

This Detailed Seismic Assessment has been peer reviewed, and the reviewer's comments have been incorporated in this issue of the report.

We refer to the Concept Strengthening design that has been issued to Wellington City Council on 24 March 2023.

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10 STATUTORY REQUIREMENTS FOR EXISTING BUILDINGS

When working with existing buildings, there are statutory requirements that must be considered, these vary depending on whether the building is considered to be potentially Earthquake-prone.

The Michael Fowler Centre is currently notified as Earthquake-prone. This DSA has confirmed that it should be considered potentially earthquake-prone and will not serve as evidence to lift the existing notice. Therefore, the relevant sections of the New Zealand Building Act 2004 and the Building (Earthquake-prone Buildings) Amendment Act 2016 that need to be considered in relation to the building's structure and strength are set out as follows.

10.1 Requirements under the Building Act 2004 (as amended by the Earthquake-prone Buildings Amendment Act 2016)

The Building (Earthquake-prone Buildings) Amendment Act 2016 took effect on the 1 July 2017. This Act provides a national policy framework for managing Earthquake-prone buildings. The three main steps of the framework are:

- Territorial Authorities (TA's) must identify potentially Earthquake-prone buildings
- Building owners of potentially Earthquake-prone buildings must commission an engineering assessment
- TA's must use this information to determine whether or not a building or part is Earthquake-prone.

Key features of the act are described below:

Earthquake-prone Buildings

An Earthquake-prone building is a building (or part of a building) that will have its ultimate capacity exceeded in a moderate earthquake (have an earthquake rating less than 34%NBS) **and** if the building (or part) were to collapse, the collapse would likely cause injury or death to persons, or cause damage to any other property.

- The Territorial Authority (TA) in whose district the building is situated, determines if a building is Earthquake-prone. The earthquake rating (%NBS), mode of failure and physical consequence identified by an accepted engineering assessment informs this decision. However, the TA's decision will also depend on occupancy, accessibility, neighbouring buildings, and their proximity.
- A moderate earthquake is an earthquake at the site that is the same duration, but one-third as strong as the earthquake shaking that would be used to design a new building at that site (33%ULS loading).
- An Earthquake-prone building can be one that poses a risk to people on adjoining properties and not just those within the building itself.
- Excluded from the definition of 'Earthquake-prone building' are certain residential housing, farm buildings, retaining walls, wharves, bridges, tunnels, and monuments.
- Included in the definition of 'Earthquake-prone building' are hostels, boarding houses and residential housing that is more than two storeys and contains three or more household units.

Seismic Risk Locations

Different locations are assigned different 'seismic risk'. There are three different categories defined by the seismic hazard factor (Z) in the New Zealand Loadings Code (NZS 1170)

- High Seismic Risk – Z factor greater than or equal to 0.3
- Medium Seismic Risk – Z factor between 0.15 and 0.3
- Low Seismic Risk – Z factor less than 0.15

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The seismic risk relates to timeframes for strengthening and identification of potentially earthquake-prone buildings. **Wellington** is considered a **High Seismic Risk region**.

Priority Buildings

Priority buildings are defined as buildings in areas of high or medium seismic risk that:

- Are generally used for health or emergency services or as educational facilities, or,
- Contain unreinforced masonry (such as parapets) that could fall onto busy thoroughfares in an earthquake, or,
- The Territorial Authority has identified as having the potential to impede strategic transport routes after an earthquake.

Priority buildings have shorter timeframes for identification and strengthening of earthquake-prone buildings.

The Michael Fowler Centre has been classed as a Priority Building.

Alterations and Change of Use

In addition to legislation covering consideration of Earthquake-prone buildings, other relevant sections of the Building Act are:

- **Section 112:** Alterations to existing buildings. Section 112 of the Building Act requires that a building subject to an alteration continue to comply with the relevant provisions of the Building Code to at least the same extent as before the alteration. Essentially this section means that the building may not be made any weaker than it was as a result of any alteration.
- **Section 115: Change of Use.** Section 115 of the Building Act requires that the territorial authority (Wellington City Council) be satisfied that the building in its new use will comply with the relevant sections of the Building Code "as nearly as is reasonably practicable". In relation to building earthquake strength, this does not necessarily require the building to comply in full with the current Building Code, provided that it can be shown that full compliance is impractical, or that the cost (by any relevant measure of value) is unreasonable under the circumstances. Interpretations would relate to the specific circumstance and would need to be agreed in dialogue with the territorial authority on a case-by-case basis.
- **Section 133AT: Alterations to buildings subject to EPB notice**. Specifically with respect to seismic work, if any alteration to a building subject to an EPB notice is "substantial", then the alteration must include the necessary seismic work which results in the building no longer being classified as earthquake-prone. There are also other requirements for alterations specific to earthquake prone buildings (substantial or not), relating to means of escape from fire, and access/facilities for those with disabilities. The Building Act and regulations should be referred to for the specific requirements, including the definition of what constitutes a substantial alteration (NZ Govt, 2016) (NZ Govt., 2017).

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11 LIMITATIONS

Findings presented as a part of this project are issued pursuant to our contract with Wellington City Council dated 2 November 2021 and for the sole use of Wellington City Council in its evaluation of the subject property. The findings are not intended for use by other parties and Holmes NZ assumes no liability to any party other than Wellington City Council.

Our observations have been visual only and are limited to representative samples. Our observations have been restricted to structural aspects only. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been inspected or reviewed, and secondary elements such as windows and fittings have not generally been reviewed.

Our findings relate to the structural performance of the building under earthquake actions. We have not reviewed other loading conditions such as the live load capacity of the floors or wind loading.

Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

12 REFERENCES

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Appendices

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Appendix A

Assessment Summary Report

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Appendix A Engineering Assessment Summary Report

The following table summarises the key points from the seismic assessment and is provided in accordance with Section A8.5 of the Engineering Assessment Guidelines.

For engineering assessments being undertaken for potentially Earthquake-prone buildings, this summary meets the requirements of Section 2.5 of the EPB methodology.

Building Information	
Building Name/ Description	Michael Fowler Centre
Street Address	111 Wakefield Street, Te Aro, Wellington
Territorial Authority	Wellington City Council
No. of Storeys	(up to) 4 stories
Area of Typical Floor	2,500m^2 – Auditorium and independent staircase structures 800m^2 – Renouf Foyer
Year of Design	1980
NZ Standards Designed to	NZ4203:1976
Structural System including foundations	Auditorium: Cantilever flanged wall system, supplemented with concrete squat walls on the ground floor level Renouf Foyer: Concrete moment frames with arch-shaped joint zones. Foundations: Ø450mm and Ø560mm Franki piles, and Ø900mm bored piles – linked by foundation beams and a ground floor slab.
Does the building comprise a shared structural form or share structural elements with any other adjacent titles?	n/a
Key features of ground profile and identified geohazards	Founded on a subsoil of mixed loose silty/sandy gravels, sands and gravelly silts up to a thickness of 4.4m. Below that there a thin ground layer of beach sand (medium dense gravelly sand) and a very thick layer of alluvium (dense to very dense silty sand and gravel) before bedrock is estimated at 40-60m below the ground floor level. The reclamation fill is prone to liquefaction.
Previous strengthening and/ or significant alteration	▪ In 1989, the Renouf Foyer was extended labelled as the 'kitchen extension' ▪ In 2018, securing works were undertaken to the staircases. ▪ In 2020, securing works were undertaken to provide seismic resilience to the precast DyCore units in the roof of the Auditorium, the flange-hung double-tee units in the promenades and the Renouf Foyer, and the rib infill floor in the Renouf Foyer.
Heritage Issues/Status	n/a
Other relevant information	-

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Assessment Information	
Consulting Practice	Holmes NZ LP
CPEng Responsible	Laura Whitehurst CPEng No.: 1159151
Documentation reviewed	▪ Original design documentation (architectural and structural) from 1980 ▪ Design documentation of the 1989 kitchen extension ▪ Previous seismic assessment report from 2014 ▪ Design documentation of Securing works (2016, 2019, 2020)
Geotechnical Report(s)	Geotechnical Assessment Report, by Tonkin+Taylor Limited, dated December 2022
Date(s) Building Inspected and extent of inspection	Visual inspections have been carried out on the following dates: ▪ 25 May 2022 ▪ 24 August 2022 ▪ 30 September 2022 ▪ 24 November 2022
Description of any structural testing undertaken and results summary	n/a
Previous Assessment Reports	2014 Seismic Assessment, by Holmes Consulting LP
Other Relevant Information	

Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	The Michael Fowler Centre is a performing arts centre. It can house up to 2,200 people inside the Auditorium, whilst also accommodating other people in other areas of the building. It is considered an Importance Level 3 (IL3) structure.
Summary of how Part C was applied,	Auditorium: Modal Response Spectrum Analysis (MRSA) Auditorium (floor and roof diaphragms only): Nonlinear Time-History Analysis (NLTHA) to establish floor accelerations Renouf Foyer: Simplified Lateral Mechanism Analysis (SLaMA) Independent staircase structures: Simplified Lateral Mechanism Analysis (SLaMA) Foundations: Displacement-based procedure Concrete elements were assessed in accordance with the revised Yellow Chapter C5.
Other Relevant Information	-

<table>
<tr><th colspan="3">Assessment Outcomes</th></tr>
<tr><td>Assessment Status</td><td colspan="2">Draft for client review and peer review</td></tr>
<tr><td>Assessed Seismic Rating</td><td colspan="2">20%NBS_{IL3}</td></tr>
<tr><td>Seismic Grade and Relative Risk</td><td colspan="2">Grade D, corresponding to a high risk</td></tr>
<tr><td>Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed</td><td colspan="2">The following secondary and non-structural elements were considered in the detailed seismic assessment:
Precast infill cladding panel
Reinforced concrete masonry walls
Light-weight roof and façade glazing suite
Catwalks and wall reflectors
’Donut’
Rigging points in the Auditorium and Renouf Foyer
Stairs
Ceiling panels in Auditorium
The following secondary and non-structural elements were excluded:
Light-weight timber stage
Mechanical plant (to be undertaken by separate consultant)
Operable wall
Local lightweight arching skylight in the Renouf Foyer
Timber stage
Suspended light-weight timber ceiling underneath Level 4 cantilever floor segments
Movable stage lift</td></tr>
<tr><td>Describe the Governing Critical Structural Weakness</td><td colspan="2">The sloping transfer diaphragm underneath the Level 2 seating (referred to as Level 1/2)</td></tr>
<tr><td rowspan="4">If the results of this DSA are being used for Earthquake-prone decision purposes, and elements rating <34%NBS have been identified (including Parts):</td><td>Engineering Statement of Structural Weaknesses and Location</td><td>Mode of Failure and Physical Consequence Statement(s)</td></tr>
<tr><td>Sloping transfer diaphragm at Level 1/2</td><td>Exceedance of tie capacities in the diaphragm resulting in excessive displacements of the Auditorium, leading to (partial) collapse.</td></tr>
<tr><td>Diaphragm over double-tee units at the Level 2 floor in Auditorium lobby area and Level 4 promenade</td><td>Loss of load path through the diaphragm over the double-tee sections.
Highest demand when considering the effects of pounding between the Auditorium and Renouf Foyer, leading to the double-tee floor units losing their seating and falling inside the building.
Reinforced with cold-drawn (non-ductile) mesh</td></tr>
<tr><td>Level 2 floor in Renouf Foyer</td><td>Exceedance of tie capacities in the diaphragm, resulting in excessive size of cracks in the concrete diaphragm across the supports of the rib and infill floor elements, leading to loss of support and falling of the floor elements.</td></tr>
</table>

Assessment Outcomes		
	Level 2 floor in promenades of the Auditorium	Exceedance of tie capacities in the diaphragm resulting in excessive displacements of the Auditorium, leading to (partial) collapse.
	Roof diaphragm	Lack of tie capacity in the diaphragm leading to subsequent structural weaknesses in other elements, such as: ▪ Excessive axial loads buckling the top chords of the roof trusses, leading to loss of gravity support by the roof trusses and falling inside the building. ▪ Excessive loads in the connection between the roof trusses and the precast portion of the perimeter buttress wall system, leading to the roof trusses losing their seating and falling inside the building. ▪ Excessive size of cracks in the concrete diaphragm across the supports of the DyCore roof elements, leading to the DyCore roof losing their seating and falling inside the building. ▪ Redistribution of excessive seismic lateral loading to the perimeter buttress walls leading to loss of integrity of the primary structure of the Auditorium and partial collapse.
	Roof trusses	Lack of displacement compatibility with the primary lateral load resisting system, leading to excessive forces in the steel members of the roof truss. This could lead to the roof trusses losing their gravity load carrying capacity.
	Precast cladding infill panels	Lack of displacement compatibility with the primary lateral load resisting system, leading to excessive forces in the bolted connections that provides the gravity support of the infill panels.
	Catwalk system and wall reflectors	Exceedance of bracing capacity and capacity of shallow embedded anchors. Leading to loss of integrity and loss of support.
	Precast concrete wall panels in the upper precast portion of the Bay Walls	Excessive shear demands leading to loss of integrity of the primary lateral load resisting system. Leading to cracking and spalling of the concrete section and (partial) collapse of the
	Light-weight roof and façade glazing suite around the promenade and Renouf Foyer	Loss of load path between the glazing suite and the primary lateral load resisting system. Leading to loss of integrity of the façade system.
Recommendations	This Detailed Seismic Assessment has been peer reviewed, and the reviewer's comments have been incorporated in this issue of the report. The peer review is complete. Holmes NZ LP have issued a Concept Strengthening design to Wellington City Council on 24 March 2023. Please refer to that document for further details.	

Appendix B

Geotechnical Assessment Report

11 April 2023
Job No: 85841.0050 Rev 2

Wellington City Council
PO Box 2199
Wellington 6140

Attention: Tane Dunne

Dear Tane

Michael Fowler Centre
Geotechnical Desktop Assessment Rev 2

DRAFT

1 Introduction

This report relates to the Michael Fowler Centre building (MFC/the Site), located at 11 Wakefield Street, Wellington.

The report presents a summary of Tonkin & Taylor Ltd's (T+T's) geotechnical input to the detailed seismic assessment carried out by Holmes NZ Limited Partnership (Holmes), who is the project structural engineer. This report supersedes T+T's 2020 Geotechnical Desktop Assessment[1]. This work was undertaken by T+T at the request of Wellington City Council (WCC) in accordance with our proposal[2].

2 Scope of work

The scope of this geotechnical seismic assessment was to:

- Review and update the ground and groundwater model using available geotechnical data in the area obtained since 2020.
- Assess liquefaction and associated consequences. This includes assessment of total and differential lateral spread across the Site at different levels of earthquake shaking.
- Review as-built pile records, and update pile founding and vertical capacity/stiffnesses.
- Review and update base shear take-out mechanisms, including lateral pile analysis.
- In conjunction with Holmes, assess soil-structure interactions, and collaborate to determine if a geotechnical or structural "step change" could be applicable to this building.
- Update T+T's Geotechnical Desktop Assessment[1] to include the points above (this report).

[1] T+T (27 August 2020). Letter Report. *Michael Fowler Centre. 111 Wakefield Street, Wellington. Geotechnical Desktop Assessment Rev 1.* (T+T ref: 85841.0030).
[2] T+T (20 April 2022). Letter of Engagement. *Michael Fowler Centre. Variation Order VO2. Geotechnical Support to Detailed Seismic Assessment*. (T+T ref: 85841.0050).

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3 Basis of assessment of existing buildings

This geotechnical assessment has been undertaken generally in line with Section C4: Geotechnical Considerations, The Seismic Assessment of Existing Buildings Guideline, July 2017 (the Guideline). In accordance with that guideline, this assessment is focused on geotechnical aspects which could influence the behaviour of the structure with respect to life safety. Serviceability aspects are not considered.

If it is determined that modifications to the existing foundations are required, then the concept design for these works should be developed in consultation with a geotechnical engineer. Information presented in this report is not intended for the design of building or foundation modifications.

4 Site description

Conclusion	Information reviewed
• Site plans are included in **Appendix A**. These show the MFC building location, along with reclamation areas by year (Figure A1), and geotechnical data used in the assessment (Figure A2). • The Site is located at 111 Wakefield Street, Te Aro, Wellington. • Property legal description: Lot 1 DP 494594. • Property/parcel area (total): approximately 34,000 m^2. • MFC building footprint area (total): approximately 4,000 m^2. • The MFC building is located near the centre of the property. Further detail on the MFC building (including foundations) is presented in **Section 6**. • The Site is flat (current ground surface level approx. RL +2.15 m). • The Site is located approximately: – 50 m Northeast (seaward) of the pre-1900 shoreline. – 40 m Southwest (landward) of the current shoreline, i.e. at Whairepo Lagoon. • The Site is bound by: Wellington Town Hall building (west), MFC Carpark and Jervois Quay (east), Wakefield Street (south), and Civic Square (north). • A basement carpark is located adjacent to the north-western corner of the building, extending north beneath the Civic Square.	• Aerial photograph and property details (refer Figures A1 and A2 included in **Appendix A**). • LiDAR data sourced from LINZ data service (NZVD2016 datum). • Reported elevations are in terms of the Wellington 1953 datum = NZVD2016 + ~0.4m.

5 Assessment and interpretation of site conditions

5.1 Ground and groundwater conditions

Conclusion	Information reviewed
• Available geotechnical data considered in this assessment are listed below (also shown on Figure A2 in **Appendix A)**: – Five site-specific boreholes (B1 to B5) up to a max. depth of 18.2 m. Borehole logs are included in **Appendix C**. – At Wellington Town Hall building, located west of the Site: o Five boreholes (BH201, BH202, BH203, BH305 and BH306) up to a max. depth of 20.4 m. o Groundwater monitoring data from BH201 to BH203 (standpipe piezometer and level logger). o Five Cone Penetrometer Tests (CPT201, CPT202, CPT203, CPT305 and CPT312) up to a max. depth of 9.9 m. – At other areas, located within 50 m of the Site: o Five boreholes (BH1, BH2, BH4, BH9, and BHA2) up to a max. depth of 20 m. o Four CPT's (CPT1003, CPT1004, CPT1005, and CPT1006) up to max. depth of 13.8 m.	• Depths reported are measured from the ground surface at the time of drilling/testing. • 1:50,000 geological map 22 (Begg, J.G.; Mazengarb, C., 1996). • MFC BH logs by Brickell, Moss, Rankine & Hill (1978), provided by WCC (refer **Appendix C**). • T+T Wellington Town Hall geotechnical factual report (August 2017) • ME (2017). Guidance document. Coastal

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Conclusion	Information reviewed
• The inferred soil profile at the Site is summarised below. A geological cross-section is presented in Figure A3 in **Appendix A.** Further detail of the soil profile is presented in Table B1 in **Appendix B**. **Layer 1: Reclamation fill (1889)** Typically 4.5 m thick. Mixed loose silty/sandy GRAVEL, SAND and gravelly SILT. **Layer 2: Beach deposits** Typically 1.5 m thick. Medium dense gravelly SAND. Upper 0.5 m typically loose. **Layer 3: Alluvium** Inferred to be 35 m to 55 m thick. Dense to very dense silty SAND and GRAVEL. Interbedded with medium dense SAND/GRAVEL, and firm to very stiff SILT/CLAY. **Layer 4: Bedrock** Inferred to be 40 m to 60 m deep. SANDSTONE, SILTSTONE and MUDSTONE. • An approx. 6.5 m high, mass concrete retaining wall (seawall) is located along Whairepo Lagoon and is inferred to be founded in the beach deposits. • Groundwater level: – Available monitoring data indicates a current average groundwater level at RL +0.3 m, ranging between RL +0.0 m and RL +0.75 m due to tidal effects. – A groundwater level of RL +0.5 m has been adopted for this assessment (current average tidal + 0.2 m average sea level rise over the next 50 years).	hazards and climate change. • "Updated 3D basin model and NZS 1170.5 subsoil class and site period maps for the Wellington CBD". (Kaiser, A.E. et al, May 2019) • NZGD and T+T internal files.

5.2 Active faults

Conclusion	Information reviewed
• The Wellington Fault lies approximately 1.5 km northwest of the Site. The Wellington Fault is included in Table 3.6 of NZS 1170.5:2004 as a major fault requiring near fault factors when assessing structural design actions. • There are numerous other active and inactive faults mapped nearby in Wellington City. Bathymetric survey of the Wellington Harbour identified the active Aotea Fault. The Aotea Fault is inferred to project onshore and extend southward beneath Te Aro and Aro Valley. Based on the inferred onshore location, the alignment of the fault is approx. 400 m southeast of the Site.	• GNS Online database of active faults. • NZS 1170.5:2004, Section 3.1.3 & Table 3.6. • "The structure and seismic potential of the Aotea and Evans Bay Faults, Wellington, New Zealand" (Barnes, P.M. et al, 2018).

5.3 Previous earthquakes

Conclusion	Information reviewed
The following recent earthquakes were felt at the Site: • Kaikoura Earthquake (14 November 2016 at 0.02am) Location: 15 km northeast of Culverden, approx. 215 km from the Site Magnitude: M_L 7.8 Focal depth: 15km Intensity felt at the Site: PGA 0.16g recorded at Frank Kitts Park, approx. 700 m north of the Site PGA 0.18g recorded at Te Papa Museum, approx. 400 m east of the Site • Lake Grassmere Earthquake (16 August 2013 at 2.31pm) Location: Lake Grassmere, approx. 72km from the Site Magnitude: M_L 6.6 Focal depth: 8km Intensity felt at the Site: PGA 0.11g recorded at Frank Kitts Park, approx. 700 m north of the Site	• Earthquake magnitude source of data: http://geonet.org.nz/

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Conclusion	Information reviewed
• There is no known evidence of ground damage at the Site as a consequence of these earthquakes.	

6 Existing building foundations

Conclusion	Information reviewed
The MFC building comprises an open auditorium (the Auditorium), surrounded at its extents by two access levels (ground and first level). Several peripheral foyer buildings (Renouf Foyer) are located at the southern end of the Auditorium. In between the Auditorium and Renouf Foyer are two staircase structures and a lift pit structure. Staircases also connect the Auditorium to the Renouf Foyer. Existing building foundations comprise: • Approximately 150 no. 'Franki' type concrete piles either 450 mm or 560 mm diameter. Founding depths ranging between 4 m and 13 m. Depths to pile head ranging between 0.85 m to 3 m. Includes 6 no. reinforcing cage types (Cage A to F). • 4 no. 900 mm Diameter bored, belled piles. Founding depths ranging between 8 m and 14 m. Pile head located at 1.35 m or 2.35 m depth. Includes single reinforcing cage type (Cage G). • Approximately 20 screw-jack caisson piles (not assessed in this study). • 1 m to 1.5 m wide concrete capping beams, founded between depths of approximately 1 m and 3 m. • There are a series of basement tunnels 2 m to 5 m width falling from the perimeter to a central pit. Pit and tunnel floor founded typically 2.5 m to 3 m depth.	• Holmes Wood Poole & Johnstone Structural Drawings No. 666/$S1_F$, 666/$S101_D$, 666/S29, 666/$S26_C$, and 666/$S27_B$ (refer **Appendix E**). • Available pile construction records/logs (we note some piles did not have records). • Depths reported are measured from the current ground surface level.

7 Geotechnical engineering considerations

7.1 Seismic site subsoil class

Conclusion	Information reviewed
• Micro-tremor testing undertaken by GNS at the Wellington City Town Hall (located just west of the Site) indicate a natural site period of 0.7 seconds. • Based on "Map 2, Central Wellington NZS1170.5:2004, Site Subsoil Class (V2.0)" from Kaiser et al (2019), the Site is indicated as a deep or soft soil site, i.e. Class D. • Accordingly, the Site Subsoil class is assessed as being *Class D – Deep soil site*. • Level of certainty in the above assessment is *high*. Further investigations are not expected to change this assessment.	• Refer **Section 5.1**. • NZS1170.5:2004. • "Updated 3D basin model and NZS 1170.5 subsoil class and site period maps for the Wellington CBD". (Kaiser, A.E. et al, May 2019)

7.2 Ground shaking hazard

The seismic hazard in terms of peak ground acceleration (PGA) and magnitude (M_{eff}) for the Site has been assessed based on MBIE/NZGS Earthquake Geotechnical Engineering Practice Module 1 (2016): Overview of the Guidelines, Section 5, Method 1. **Table 7-1** below presents the return periods for earthquakes with various 'unweighted' PGA's with a corresponding earthquake magnitude. The derived PGA is used to determine liquefaction potential at the Site.

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Table 7-1: Ground shaking hazard

NZS 1170.5 Limit State	PGA (g)	M_{eff}	Return period (years)
Ultimate limit state (ULS)	0.45	7.1	1000
Serviceability limit state (SLS)	0.09	6.2	25

Note:

PGA and effective magnitude has been assessed based on Bridge Manual SP/M/022 Third Edition for the following:

Building design life	50 years – agreed with Holmes
Building importance level	IL3 (NZS 1170.0:2004, Table 3.2) – agreed with Holmes
Return period factor, Ru	1.3 for 1000yr and 0.25 for 25yr return period (NZS 1170.5:2004, Table 3.5)
Subsoil class	D (Deep soil)
Return period PGA coefficient, $C_{0,1000}$	0.45 (Bridge Manual Table 6A.1)
Site subsoil class factor, f	1.0 (Bridge Manual Section 6.2)
PGA	$C_{0,1000}$ x Ru/1.3 x f x g (Bridge Manual Section 6.2)
Effective Magnitude, Meff	7.1 for 1000yr and 6.2 for 25yr return period (Bridge Manual Table 6A.1)

Note all levels/magnitudes of earthquake shaking reported herein (as a % of ULS shaking) are in terms of Module 1 (2016) values for an IL3 building, unless indicated otherwise. For example, 100% ULS earthquake shaking represents PGA of 0.45g, M_{eff} 7.1.

The Kaikoura M7.8 earthquake recorded a PGA of 0.18g at Te Papa Museum (see **Section 5.3**). Applying the magnitude weighting procedure of Idriss and Boulanger (2014)[3] to this record, yields a PGA of 0.2g at M_w = 7.1. This magnitude weighted intensity represents 45% of the ULS shaking reported in **Table 7-1** (0.2/0.45 = 45% ULS).

7.3 Impact of design standard changes

In October 2022, GNS Science released the revised National Seismic Hazard Model (NSHM)[4]. This represents the latest scientific knowledge of earthquake hazard in New Zealand and is an important factor for understanding and managing earthquake risk in the built environment. This is likely to result in changes in the level of shaking to be allowed for in design of new buildings in Wellington. Refer **Appendix D** for more detail, including a comparison of seismic shaking parameters and associated likelihoods at the Site. We note the following in relation to these expected changes:

- These changes will not affect the classification of a building as earthquake-prone. Earthquake-prone classification is defined in legislation as the documents current in July 2017. Changes in legislation in the future may be possible.
- It is not known if these changes will be applied in reporting of %NBS of an existing building, i.e. if reporting of %NBS will be in terms of the previous standard/guideline or in terms of the revised/new documents.
- If applied, the changes would apply to all buildings in Wellington.

7.4 Liquefaction assessment

The assessment of liquefaction and associated consequences has been based on a 50% probability of exceedance (PE), which is considered an appropriate representation of the probable ground behaviour. This assessment is based on the ground shaking parameters presented in **Section 7.2**.

[3] Boulanger, R.W and Idriss, I.M., 2014. CPT and SPT based liquefaction triggering procedures." Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.

[4] https://nshm.gns.cri.nz/

7.4.1 Liquefaction potential

The triggering of liquefaction, for each soil layer identified as being susceptible to liquefaction, has been assessed in accordance with the procedure of Idriss and Boulanger (2014)[3]. This method is based on empirical relationship with the SPT 'N'/ CPT 'q_c' and fines content. SPT data from the five site-specific boreholes, and all CPT's listed in **Section 5.1** have been used in our assessment. The conclusions are summarised in **Table 7-2**.

Table 7-2: Liquefaction potential

Layer no.	Geological unit	Conclusion
1	Reclamation Fill (1889)	• Above groundwater level: Not expected to liquefy. • Below groundwater level: Widespread liquefaction is possible in earthquake shaking with PGA > 0.2g, M7.1 (> 45% ULS earthquake shaking).
2	Beach Deposit	• Within upper (looser) deposits, widespread liquefaction is possible in earthquake shaking with PGA > 0.2g, M7.1 (> 45% ULS). • Within lower (denser) deposits, localised zones of liquefaction is possible in earthquake shaking with PGA > 0.23g, M7.1 (> 50% ULS earthquake shaking).
3	Alluvium	• Generally dense and widespread liquefaction is unlikely. • Localised pockets of liquefaction, i.e. liquefaction within layers/lenses of non to low plasticity SILT, and medium dense SAND is possible in earthquake shaking with PGA > 0.25g, M7.1 (> 55% ULS earthquake shaking).
4	Bedrock	• N/A.

7.4.2 Liquefaction consequences

Considering the potential for liquefaction described in **Section 7.4.1**, associated consequences of liquefaction at the Site have been identified as listed in **Table 7-3** below.

Table 7-3: Liquefaction consequences

Consequence	Comments
Cyclic displacement (ground lurch)	• Of the order of 100 mm has been estimated over the entire building footprint at > 45% ULS earthquake shaking.
Lateral spread	• Expected to be large (100's mm) and laterally spreading towards the Whairepo Lagoon. • Estimated to occur at > 45% ULS earthquake shaking. • Level of certainty to calculate lateral spread is low. Refer **Section 9.3**.
Reduced soil strength and stiffness	• Will result in loss or substantial reduction in support to foundations if founded in liquefied soils.
Post-liquefaction settlement	• Of the order of 100 mm free-field settlement is estimated at > 45% ULS earthquake shaking.
Sand boils	• Possible as thickness of crust is generally < 2 m. Ground settlement is likely as a result of sand boil(s). This is in addition to free-field settlement described above. • Building is founded on piles so unlikely to directly result in settlement of the building. Elements not founded on piles could settle. Elements designed as supported on grade may be 'suspended' and increase foundation loadings.
Uplift pressure	• Expected for buried elements (e.g. tunnels) founded below groundwater within reclamation fill or upper beach deposits.

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8 Geotechnical issues identified

Key geotechnical issues that may affect the building's seismic performance are listed in **Table 8-1**.

Table 8-1: Geotechnical issues identified

Issue	Comments
Lateral aspects	
Cyclic displacement and lateral spreading	• These ground movements will impose kinematic soil loads onto the foundations (ground beams and piles) and cause damage to piles. • Differential lateral spread (lateral stretch) across the building can be very damaging. • Refer **Sections 9.2** and **9.3**.
Reduced resistance (passive and sliding) to base shear	• Refer **Section 9.2**.
Vertical aspects	
Reduced uplift (tension) and compression capacity of piles	• Refer **Sections 10.1** and **10.2**.
Differential settlement of foundations	• The building is supported on piles. Liquefaction can cause a loss or reduced bearing capacity in some piles (soft springs) and cause differential settlement. Holmes should consider the effect of soft and stiff springs for the liquefaction condition. • Refer **Section 10.1**.
Reduced support to ground beams	• Provides limited bearing capacity during liquefaction and not required by Holmes. Therefore, not assessed further.
Uplift pressure on buried elements (e.g. tunnels)	• Refer **Section 10.3**.

9 Assessment of existing foundations – lateral aspects

9.1 Estimation of structural base shear loading

In the event of widespread liquefaction (refer **Section 7.4**), it is expected that the seismic response of the ground and structure could be influenced as follows:

- Limit the cyclic shear stress (and PGA) that can be transmitted to the surface, particularly at higher levels of earthquake shaking.
- Increase the natural period of earthquake shaking of the ground.
- Change in natural period of earthquake shaking of the structure, depending on the foundation system of the building.

Holmes should consider these factors in assessing the base-shear of the building.

9.2 Base shear take-out

As discussed in **Section 7.4**, liquefaction-induced lateral ground movements (cyclic displacement and lateral spreading) is possible at > 45% ULS earthquake shaking.

In an earthquake where widespread liquefaction of the reclamation fill and beach deposits occurs, four scenarios should be considered to represent the ground behaviour expected during different stages of an earthquake. These are summarised in **Table 9-1**.

Geotechnical parameters or outputs referred to in **Table 9-1** below are included in **Appendix F** (relating to piles) and **Appendix G** (relating to ground beams).

Table 9-1: Ground lateral behaviour during earthquake causing widespread liquefaction

Scenario No.	Description	Comments on Base Shear Take-out
1	Start of earthquake. No liquefaction.	100% Base shear resisted by: • Passive resistance on outside of building perimeter ground beam(s) (in non-liquefied conditions). • Lesser of: 1) Sliding resistance of encapsulated soil (in non-liquefied conditions) between ground beams; and 2) Passive resistance on inside of ground beams (in non-liquefied conditions). • Lateral capacity of piles (no liquefaction case)[1, 2]. Actions in piles have been provided for various increments of pile head lateral displacement.
1a	Zones of liquefaction triggered across the Site, but no lateral ground movement.	As above, with the following exceptions: • 100% Base shear (estimated using non-liquefied ground conditions) is to be resisted; and • Lateral capacity of piles: 20% of piles in liquefied conditions at critical locations, and the remaining 80% of piles in non-liquefied conditions.
2	Liquefaction triggered, but no lateral ground movement.	100% Base shear resisted by: • Passive resistance on outside of ground beam (in liquefied conditions). • Lesser of: 1) Sliding resistance of encapsulated soil (in liquefied conditions) between ground beams. 2) Passive resistance on inside of ground beams (in liquefied conditions). • Lateral capacity of piles (liquefaction only case)[1, 2, 3]. Actions in piles have been provided for various increments of pile head lateral displacement.
3	Cyclic displacement occurs. During shaking. (Allow for 100 mm cyclic displacement at > 45% ULS)	• 80% Base shear to be resisted. • Relative movement between building and the ground needs to be considered to determine if ground beam passive resistance and sliding resistance of encapsulated soil is contributing to base shear resistance or are additional soil loads on the building (in addition to base shear). • To consider passive resistance and sliding resistance of the encapsulated soil to resist base shear, the building needs to move more than cyclic displacement (i.e. more than 100 mm). To consider full passive resistance the building needs to move ~140mm to 220mm (depending on depth of ground beam). • If the building moves less than the cyclic displacement, the ground beam passive resistance and sliding resistance of encapsulated soil are additional soil (kinematic) loads on the building (in addition to base shear). • Lateral capacity of piles [1, 2]. Actions in piles have been provided for various increments of pile head lateral load. When the building moves less than cyclic displacement, the pile head lateral load modelled should represent both soil (kinematic) loads and base shear loads. When the building moves more than the cyclic displacement, the pile head lateral load modelled should represent the base shear loads only.

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Scenario No.	Description	Comments on Base Shear Take-out
4	Lateral spreading occurs. Towards end of shaking, possibly post-shaking. Refer **Section 9.3** for discussion on the magnitude and likelihood of lateral spread.	• 25% Base shear to be resisted. • Relative movement between building and the ground needs to be considered to determine if ground beam passive resistance and sliding resistance of encapsulated soil is contributing to base shear resistance or are additional soil loads on the building (in addition to base shear). • To consider passive resistance and sliding resistance of the encapsulated soil to resist base shear, the building needs to move more than lateral spread displacement. • If the building moves less than the lateral spread displacement, the ground beam passive resistance and sliding resistance of encapsulated soil are additional soil loads on the building (in addition to base shear). • Lateral capacity of piles [1, 2]. Actions in piles have been provided for various increments of pile head lateral load. When the building moves less than lateral spread displacement, the pile head lateral load modelled should represent both soil (kinematic) loads and base shear loads. When the building moves more than the lateral spread displacement, the pile head lateral load modelled should represent the base shear loads only. Note only selected pile types and levels have been assessed as agreed with Holmes.

Note 1: Sensitivity on the lateral spring and pile group effects to be considered as stated in **Appendix F.**

Note 2: Lateral analysis carried out for the pile head and toe levels requested by Holmes. Pile structural parameters (stiffness, moment capacity, axial compression load and pile head fixity) are provided by Holmes.

Note 3: It was found that when a pile is becoming unstable (pile head shear load becomes negative), it will require support from the building (adds to the base shear loading on building). Structural assessment to consider if this is a feasible mechanism. Otherwise, piles beyond that head displacement may not be relied on to carry vertical load.

Furthermore, some Cage E and F piles were assessed to have negligible lateral capacity. Where the pile head displacement is greater than 75 mm for Cage E piles and 90 mm for Cage F piles, the pile may not be relied on to carry vertical load.

9.3 Total and differential lateral spread

9.3.1 Uncertainties associated with lateral spread

The possibility of liquefaction-induced lateral spread has been identified at the Site (refer **Section 7.4**). The magnitude of total and differential lateral spread will be influenced by:

- The magnitude and intensity of earthquake shaking.
- The thickness and nature of the liquefied layer.
- Three-dimensional effects of the limited width of the Whairepo Lagoon, and the lateral restraint provided by the land either side of the lagoon.
- Degree of lateral restraint provided by the seawall on the landward side of the lagoon.
- Distance from the lagoon. The magnitude of potential displacement will reduce with increasing landward distance.
- The degree of lateral restraint provided by the MFC piles.

It must be appreciated that there is extreme difficulty in predicting the magnitude of lateral spread. Actual lateral spread could be an order of magnitude difference from that estimated. Lateral spread displacements at 100% ULS earthquake shaking could range from 100 mm to 1 m at the seaward end of the building and 0 mm to 300 mm at the landward end of the building. For the purpose of undertaking a DSA of the building, probable lateral displacements have been estimated based on available methods and presented in subsequent sections. An 'uncertainty factor' of half or double of the reported probable lateral displacements should be considered by Holmes as a sensitivity.

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9.3.2 Lateral spread – free field assessment (no building influence)

This assessment of 'free-field' lateral spread has considered:

- Assessed liquefaction potential (refer **Section 7.4**).
- Two-dimensional evaluation of lateral spread potential based on methods proposed by Zhang et al (2004)[5] and Gillins & Bartlett (2014)[6], including sensitivity analyses.
- Three-dimensional effects of the limited width of the Whairepo Lagoon, and the lateral restraint provided by the land either side of the lagoon.

Probable lateral spread plots across the Site for different levels of earthquake shaking are presented in Figure H2 included in **Appendix H**.

9.3.3 Lateral spread – with consideration of building influence

Any parts of the building which are well tied together is likely to move together as an "island" (single unit) and therefore averaging the lateral spread relative to the adjoining ground. This assumes that the structural elements are able to resist the passive earth pressure (soil) loading imposed by differential lateral spread ("lateral stretch"). This passive loading has been modelled and reported to Holmes (refer **Appendix G**). Holmes undertook their structural analysis to assess the ability of structural elements to resist the passive loading and concluded that:

- Individually, the Auditorium and the two staircase structures are adequately tied together such that they could be expected to move as three separate "islands" as a result of lateral spread. We understand that the Renouf Foyer is not well tied together. However, the probable differential lateral spread across the Renouf Foyer is unlikely to be large.
- Tying between the above elements are limited such that differential lateral movement ("lateral stretch") between these individual "islands" could be expected.

The probable lateral spread has been estimated for the Auditorium and Renouf Foyer acting as "islands" are presented in **Table 9-2** below.

Table 9-2: Probable lateral spread displacements of Auditorium and Renouf Foyer

% ULS earthquake shaking	Probable lateral spread estimate (mm)[1,2]		
	Auditorium (total)	Renouf Foyer (total)	Auditorium/Renouf Foyer (differential)
45	250	150	100
100	650	450	200

Note 1: Holmes advised that the existing piles are able tolerate the lateral displacements reported above, while maintaining vertical load carrying ability. Holmes also advised the structures will move more than the displacements reported above, under base shear loading.

Note 2: An 'uncertainty factor' of half and double of the reported probable lateral displacements should be considered by Holmes as a sensitivity.

[5] Zhang, G., Robertson, P.K. and Brachman, R.W.I., 2004. Estimating liquefaction-induced lateral displacements using the standard penetration test or cone penetration test. Journal of Geotechnical and Geoenvironmental Engineering, 130(8), pp.861-871.

[6] Gillins, D.T. and Bartlett, S.F., 2014. Multilinear Regression Equations for Predicting Lateral Spread Displacement from Soil Type and CPT Data. Journal of Geotechnical and Geoenvironmental Engineering, 140(4).

9.4 Step change

The Guideline (refer **Section 3**) requires resilience of the structure to be considered in assessing the %NBS score for structural weaknesses as a consequence of liquefaction. Liquefaction and the associated sudden change in soil behaviour is referred to in the Guideline as a geotechnical "step change".

We understand Holmes have identified structural weaknesses, which are associated with liquefaction-induced lateral spread. Accordingly, the %NBS score of the structural weaknesses will need to consider if a "step change" in the interactive geotechnical-structural behaviour could be expected.

Lateral spread with increasing earthquake shaking at selected locations along the structure has been assessed, refer Figure H3, **Appendix H.** This should be considered by Holmes in assessment of resilience, and to determine whether a structural "step change" applies, as defined by the Guidelines.

10 Assessment of existing foundations – vertical aspects

10.1 Pile axial compression

Table 10-1 below summarises the probable geotechnical vertical compression capacities and spring stiffnesses of the existing piles.

Table 10-1: Vertical compression capacity and stiffness of piles

Pile type	Shaft dia. (m) [Note 1]	Assumed base dia. (m) [Note 2]	Assumed pile founding condition	Probable limiting compression capacity (kN)		Compression springs (kN/mm) [Note 5, 6, 7, 8]
				No liquefaction [Note 3]	With liquefaction [Note 4, 7]	
Franki	0.45	1.0	Founded on/within denser beach or alluvium deposits	1,950	1,550	No Liquefaction Case: 40 to 200 Liquefaction Case: 20 to 200
Franki	0.56	1.2		2,800	1,750	No Liquefaction Case: 45 to 280 Liquefaction Case: 20 to 280
Bored Belled	0.90	2.0		7,850	2,700	No Liquefaction Case: 80 to 400 Liquefaction Case: 20 to 400

Note 1: Shaft diameter obtained from Holmes Wood & Johnstone Structural Drawings No. $S1_F$, $S101_D$, and S29.

Note 2: The volumes of concrete to develop the Franki "bulb" are derived from reviewing as-built pile construction records. These volumes have been applied to assess base diameters.

Note 3: Bearing capacity provided accounts for possible weaker silt layers located within pile founding zone of influence, and also the possibility of limited pile embedment into denser ground.

Note 4: Bearing capacity provided accounts for possible localised liquefaction (within upper alluvium) at some depth (but not directly) below the pile founding level, and possible liquefaction above the pile founding level.

Note 5: Pile stiffnesses are presented as a range due to the limited degree of certainty to which these values can be predicted.

Note 6: Due to the variability of the ground and associated uncertainty, it is recommended that for the structural analysis, both softer springs and stiffer springs and a combination are considered to understand the behaviour of the foundation system and to obtain displacements and loads. In particular, the behaviour should be checked from the most adverse combination of springs at critical locations.

Note 7: Although considered unlikely, it is possible that a pile is founded immediately above a liquefiable pocket. Structural analysis should consider the effect on the behaviour of the structure (and consequences) should a single pile at critical locations lose all support (i.e. no capacity).

Note 8: See also **Section 10.3**.

10.2 Pile axial uplift (tension)

Table 10-2 below summarises the probable geotechnical vertical tension capacities and spring stiffnesses of the existing piles.

Table 10-2: Vertical tension capacity and stiffness of piles

Pile cage	Shaft dia. (m) [Note 1]	Assumed base dia. (m) [Note 2]	Probable limiting tension capacity (kN) [Note 3 and 4]		Tension springs (kN/mm) [Note 6 and 7]
			No liquefaction	With liquefaction	
A	0.45	1.00	300	160	15 to 150
B	0.56	1.20	250	130	15 to 150
C	0.45	1.00	350	180	15 to 150
D	0.56	1.20	450 [Note 5]	300 [Note 5]	15 to 150
E	0.45	0.45	20	0	2 to 10
F	0.56	0.56	60	20	5 to 20
G	0.90	2.00	700	400	15 to 150

Note 1: Shaft diameter obtained from Holmes Wood & Johnstone Structural Drawings No. $S1_F$, $S101_D$, and S29.

Note 2: Holmes Wood & Johnstone Structural Drawings No. S29 and as-built records indicated that, for pile cages A to D, 800 mm was drilled into the bulb before lowering reinforcement. Accordingly the bulb can be depended on for tension/uplift capacity.

Note 3: Capacities does not consider self weight of pile.

Note 4: Capacities vary due to variation in as-built pile lengths; lower probable estimates provided. Discuss with T+T any specific piles where these values are exceeded.

Note 5: Pile Cage D piles are typically embeded deeper into dense alluvium and/or are longer compared to Cage B piles. Accordingly, the estimated tension capacity for Cage D piles are larger.

Note 6: Due to the varaibility of the ground and associated uncertainty, it is recommended that for the structural analysis, both softer springs and stiffer springs and a combination are considered to understand the behaviour of the foundation system and to obtain displacements and loads. In particular, the behaviour should be checked from the most adverse combination of springs at critical locations.

Note 7: Pile stiffnesses relate to the geotechnical tension capacity only, and does not consider pile structural elongation (i.e. axial extension).

10.3 Uplift pressure on foundation slabs

Buried elements (e.g. tunnels) with the base located at or within widespread liquefaction soils will be subject to liquefaction-induced uplift pressures. This is expected to affect buried structures with base located between RL +0.5 m and RL -2.85 m. For such buried elements, total uplift pressure (kPa) of: [19 kN/m^3 x depth of element below ground surface level] should be considered during liquefaction.

Structural assessment should consider uplift pressure on buried elements for critical load cases to assess the impact on the building's behaviour.

11 Mitigation

The potential mitigation measures would depend on the findings of the structural assessment, identifying issues governing the NBS rating of the building. However, the following possible options can be considered for further development:

a To provide foundations offering reliable lateral and vertical support to the building
   - Underpin the building with new foundations possibly in conjunction with structural modification e.g. base isolation. The new foundations would be required to be designed to resist the vertical loads and base shear and lateral ground movement loads; or
   - Ground improvement (jet grouting or possibly resin injection) beneath the footprint of the building and 4 m beyond to mitigate the effects of liquefaction.

b To mitigate the potential for lateral spread (some foundation strengthening for the building may still be required)
   - Large diameter bored pile wall adjacent to Michael Fowler Centre; or
   - Large diameter bored pile wall along seawall. Advantage of providing protection to Jervois Quay.

12 Further work

It is understood from Holmes that some geotechnical foundation strengthening may be required. The following further stages of work are recommended in collaboration between T+T and Holmes for the strengthening works:

- Optioneering and concept design.
- Preliminary design.
- Developed design.
- Detailed design.
- Construction monitoring.

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13 Applicability

This report has been prepared for the exclusive use of our client Wellington City Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Recommendations and opinions in this report are based on data from boreholes and CPT's. The nature and continuity of subsoil away from the investigation/test locations are inferred and it must be appreciated that actual conditions could vary from the assumed model.

Tonkin & Taylor Ltd

Report prepared by:

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Appendices

Appendix A **Site plan and cross-section**

Appendix B **Inferred soil profile at the Site**

Appendix C **Site-specific investigation data**

Appendix D **Impact of design standard changes**

Appendix E **Selected existing foundation drawings**

Appendix F **Lateral pile assessment**

Appendix G **Lateral resistance of ground beams**

Appendix H **Lateral spread assessment**

11-Apr-23
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Appendix A Site plan and cross-section

- **Figure A1 – Site plan (1:2000) with site location and waterfront reclamation history**
- **Figure A2 – Site plan (1:1000) with available geotechnical investigation data**
- **Figure A3 – Geological cross-section AA'**

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NOTES:

1. AERIAL PHOTO SOURCED FROM LINZ DATA SERVICE <-https://data.linz.govt.nz/layer/95524-wellington-010m-urban-aerial-photos-2017-> , LICENSED BY LINZ FOR RE-USE UNDER THE CREATIVE COMMONS ATTRIBUTION 4.0 NEW ZEALAND LICENCE (CC BY 4.0). ACCESSED 31/07/2019.
2. RECLAMATION BOUNDARIES ARE APPROXIMATE ONLY. BASED ON SEMMENS (2010) - AN ENGINEERING GEOLOGICAL INVESTIGATION OF THE CENTRAL WELLINGTON AREA.

Tonkin+Taylor

Exceptional thinking together www.tonkintaylor.co.nz

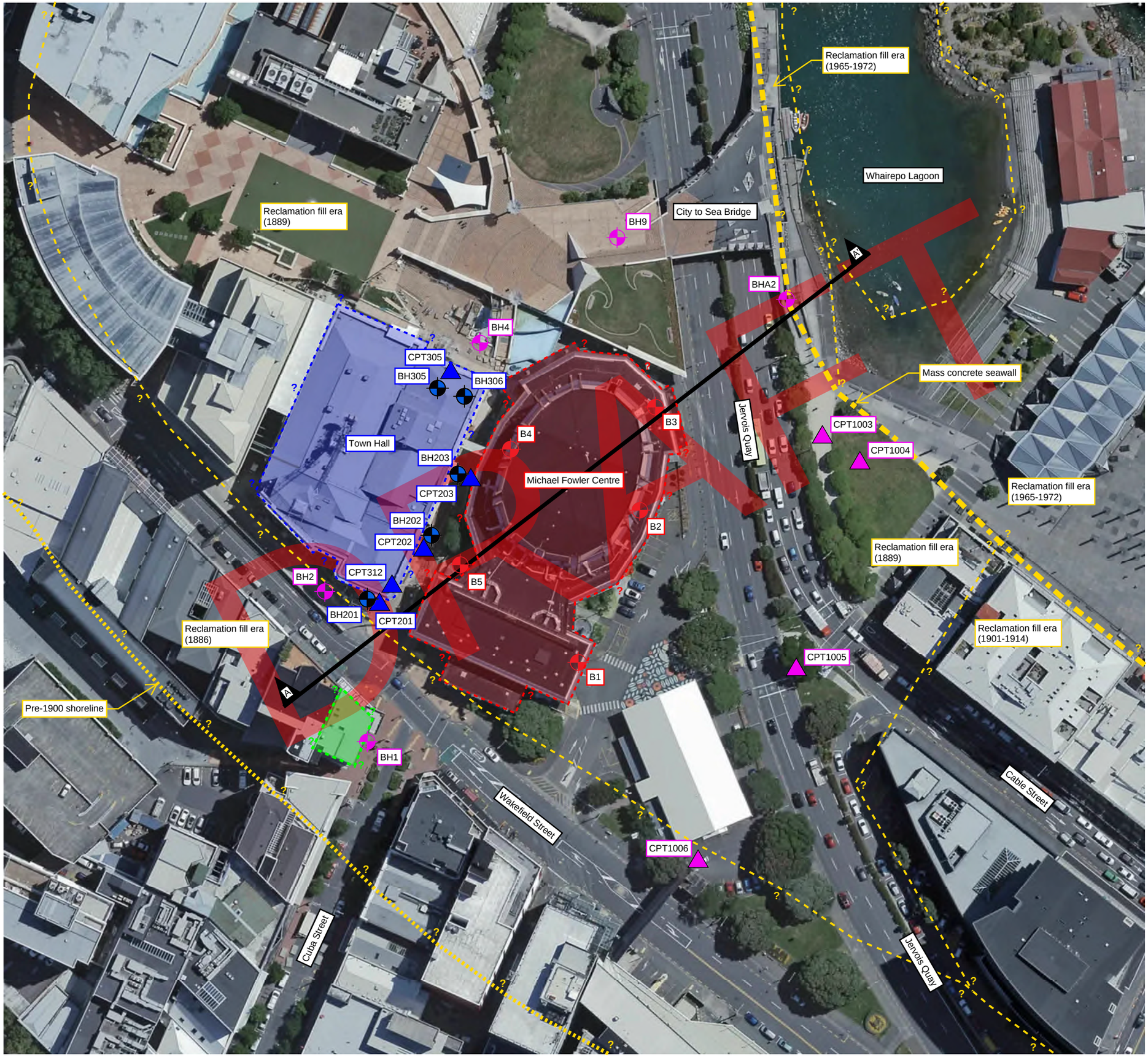
PROJECT No.	85841.0050	
DESIGNED	ZHYU	Aug.19
DRAWN	KAH	Aug.19
CHECKED		
APPROVED		DATE

CLIENT **WELLINGTON CITY COUNCIL**

PROJECT **MICHAEL FOWLER CENTRE**

TITLE GEOTECHNICAL SEISMIC ASSESSMENT
SITE LOCATION AND RECLAMATION AREAS

SCALE (A3) 1:2000 FIG No. FIGURE A1 REV 0



LEGEND

Cross-section AA'

Buildings (approximate extent)

- Michael Fowler Centre (the Site)
- Town Hall
- Greenwich

Approximate boundaries

- Reclamation fill era boundaries
- Pre-1900 shoreline
- Mass concrete seawall & reclamation fill boundary

Approximate geotechnical investigation locations

- Borehole at Michael Fowler Centre (1978)
- Borehole at Town Hall building (2017), from NZGD
- CPT at Town Hall building (2017)
- Other borehole (from NZGD or others)
- Other CPT (from NZGD or others)

Not all geotechnical investigation data available in the area is shown, only that considered in the assessment.

A3 SCALE: 1:1,000

0 10 20 30 40 50 (m)

1. LINZ Aerial Basemap Copyright LINZ CC BY 4.0. www.linz.govt.nz/linz-copyright Imagery Basemap Contributors. https://www.linz.govt.nz/data/linz-data/linz-basemaps/data-attribution
2. Reclamation boundaries are approximate only. Based on Semmens (2010) - An Engineering Geological Investigation of the Central Wellington Area.
3. Investigation locations are approximate only.

Created On:	30/06/2022
Created By:	ARolfe
Approved By:	
TT Proj Ref:	85841.0050
TT Map Ref:	TTMAPREF1433463001.902

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Wellington City Council
Michael Fowler Centre (MFC)
Geotechnical Seismic Assessment
Site Plan

FIGURE No. A2

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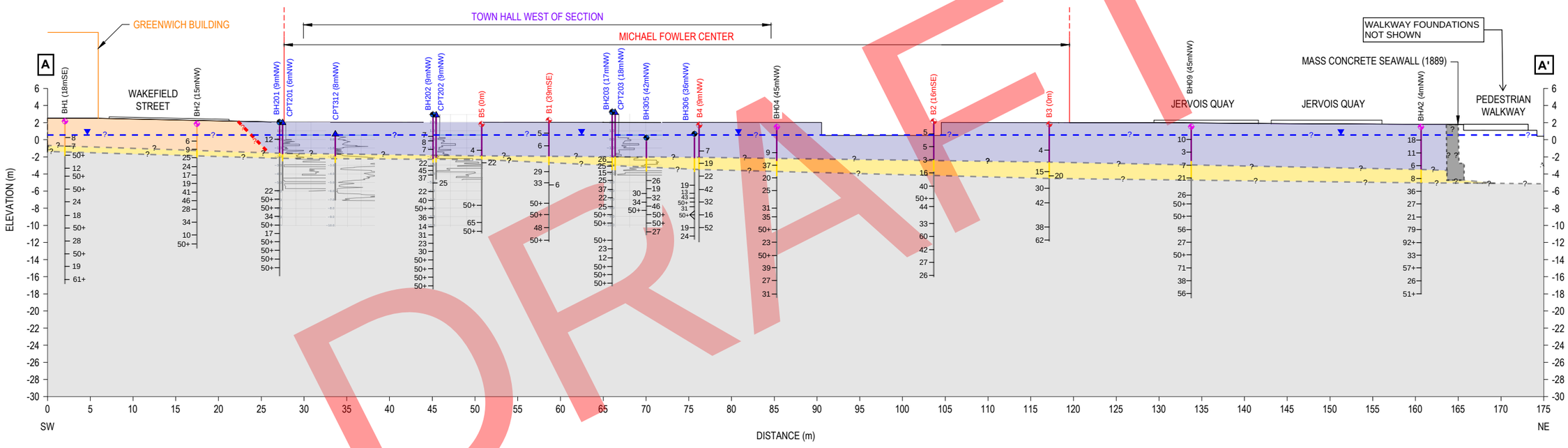

Legend

- **Reclamation Fill (1886)** - GRAVEL with sand and silt.
- **Reclamation Fill (1889) -** sandy GRAVEL with silt.
- **Beach Deposits -** SAND with silt, shells and gravel.
- **Alluvium Deposits -** Interbedded SAND, GRAVEL and SILT with some CLAY layers/lenses.
- Approximate reclamation fill boundary (1886 and 1889 fill).
- Existing ground surface (based on available LiDAR data)
- Site boundary/building extent
- Approximate groundwater level adopted for assessment
- Inferred geological boundary.
- Borehole at Michael Fowler Centre (1978)
- Borehole at Town Hall Building (2017), from NZGD
- CPT at Town Hall Building (2017)
- Other borehole (from NZGD or others)
- Mass concrete seawall (constructed 1889)

A3 SCALE 1:500
0 5 10 15 20 25 (m)

ORIGINAL IN COLOUR

NOTES:
1. Elevation levels are in terms of Wellington 1953 (New City) Vertical Datum.
2. SPT N values shown are raw (uncorrected) data.
3. CPT plots indicate q_c values (horizontal axis, 5MPa grid/intervals) with depth.
4. Investigation locations are approximate only.

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PROJECT No.	85841.0050	
DESIGNED	-	-
DRAWN	ANRO	Jun.19
CHECKED		
APPROVED	DATE	

CLIENT **WELLINGTON CITY COUNCIL**
PROJECT **MICHAEL FOWLER CENTRE**
TITLE GEOTECHNICAL SEISMIC ASSESSMENT
GEOLOGICAL CROSS-SECTION AA'
SCALE (A3) 1:500 FIG No. FIGURE A3 REV 0

Appendix B Inferred soil profile at the Site

Table B1: Inferred soil profile at the Site

Layer no.	Geological unit and description	Depth to top of layer (m)	Layer thickness (m)	SPT 'N' value	CPT 'q_c' (MPa)
1	Reclamation Fill (1889) (1) Mixed loose silty/ sandy GRAVEL, SAND and gravelly SILT.	0	3 to 5.5 *(typ. 4.5)*	4 to 7 (2)	1 to 5 *(typ. 2 to 3)*
2	Beach Deposit Medium dense gravelly Sand. Trace shell. Upper 0.5 m typically loose.	3 to 5.5 *(typ. 4.5)*	0.5 to 2.0 *(typ. 1.5)*	15 to 20	3 to 15 *(with depth)*
3	Alluvium Deposit Dense to very dense silty SAND and GRAVEL.	4.1 to 6.3 *(typ. 5.5)*	Approx. 35 to 55, *(proven 12.3)*	30 to 50+	15 to 25
	Interbedded with medium dense SAND/GRAVEL, and			15 to 30	5 to 15
	Firm to very stiff SILT/CLAY.			5 to 25	1.5 to 3.0
4	Bedrock SANDSTONE (Greywacke), SILTSTONE and MUDSTONE (Argillite)	Approx. 40 to 60 (3)	N/A	No data available.	No data available.

Note 1: The 1889 reclamation fill is retained by a mass-concrete seawall constructed in 1889 along Whairepo Lagoon. Based on a nearby borehole investigation through this wall, it is inferred to be founded in the beach deposits, i.e. at approximately 6.5 m depth.

Note 2: There is limited SPT N data within reclamation above the groundwater level, only two SPT's with SPT N value = 5.

Note 3: Appendix 3 Map 1 of "Updated 3D basin model and NZS 1170.5 subsoil class and site period maps for the Wellington CBD". (Kaiser, A.E. et al, May 2019).

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Appendix C Site-specific investigation data

- **Records of site-specific borehole no. 1 to 5 by Brickell, Moss, Rankine & Hill (1978).**

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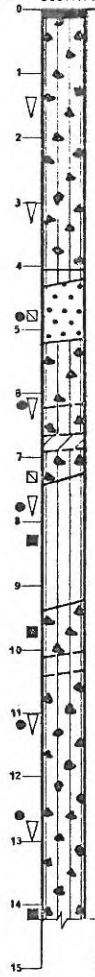
CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		FIELD DATA
% FINES (-№ 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, kPa	TEST MOISTURE CONTENT, %	SHEAR STRENGTH kPa	NATURAL WATER CONTENT, %	DRY DENSITY, kg/m³	PENETRATION RESISTANCE
									N = 5
									N = 6
							17.1		PUSH
							12.4		N = 29
							39.7		S = 33 N = 6
84	34	9	D.S. D.S. D.S.	100 150 200	25.4 24.4 22.8	115 140 180	22.6	1650	PUSH
							13.5	1966	S = 30/ 100mm
							13.6		N = 50/ 250mm
							14.8		N = 48
							12.1	1990	S = 25/ 100mm

BORING 1

ELEVATION : 2.29 metres



Seal
Brown SILTY GRAVEL (GM)
(loose; coarse; angular)

RECLAMATION FILL

Dark grey SANDY SILT (ML)
Grey SAND (SP)
(loose; medium grained; shells; random GRAVEL)
el. -2.8

BEACH

Brown SILTY GRAVEL (GM)
(dense; highly weathered; medium grained sub-rounded)

(grades grey)

(200 mm layer greyish brown firm CLAY (CL)

Grey SANDY SILT (ML)
(firm; sand fine; some rotted vegetation)

ALLUVIUM

Greyish brown SILTY GRAVEL (GM)
(very dense; medium grained; sub-rounded)

(300 mm layer brown, stiff, SILT (ML)

(grades brown)

(grades grey)

BORING DEPTH : 14.2 METRES
CASING DEPTH : 5.9 METRES
WATER LEVEL ENCOUNTERED BUT NOT MEASURED

NOTES, ABBREVIATIONS AND SYMBOLS

1. The borings were drilled between November 22 and 29, 1978 with truck mounted rotary wash equipment.

   100 mm size casing was used in the borings.

2. ■ Undisturbed sample recovered in split barrel, 60 mm I.D. ring lined sampler.

   ▨ Undisturbed sample attempted, but missed or disturbed in recovery.

   S Number of blows to drive sampler 300 mm with SPT hammer.

   PUSH Sampler driven by hydraulic pressure.

   ▽ Standard Penetration Test (SPT): performed in accordance with standard ASTM D1586-67

   N SPT blowcount per 300 mm.

   ● Disturbed observation sample.

   ▽ Observed static ground water level.

3. The elevations of the borings are referenced to the Wellington New City Datum.

4. Indicated depths are in metres below the existing ground surface.

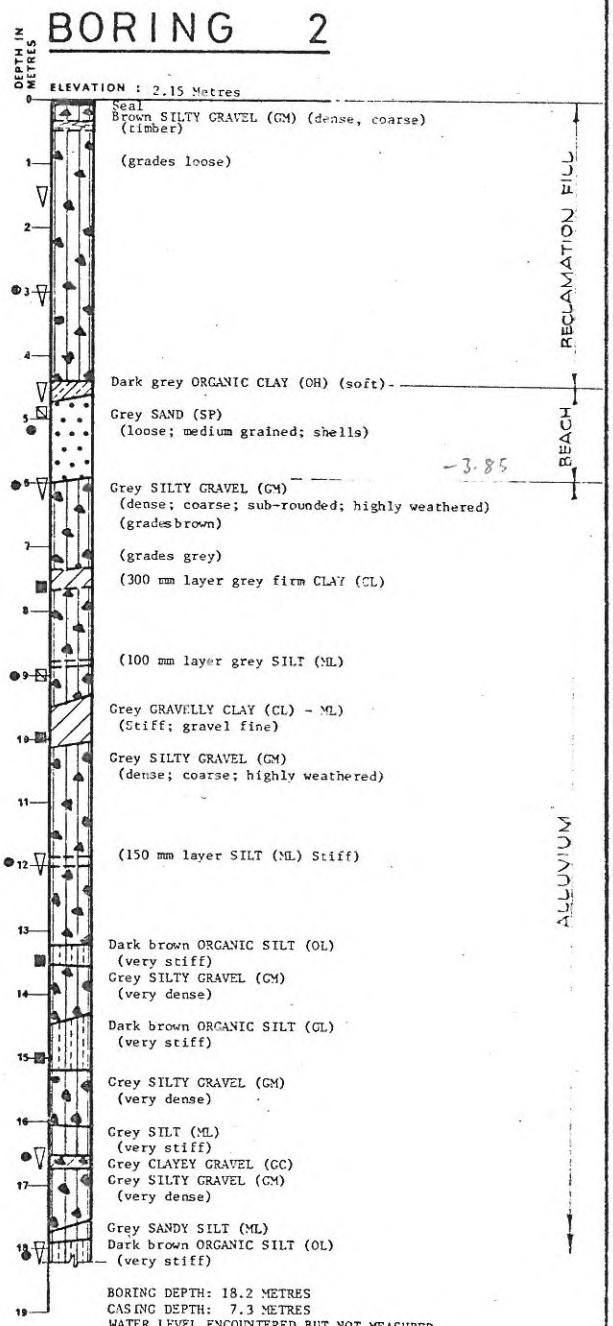
5. DS = strain controlled, direct shear test at natural moisture content.

BORING LOGS

BRICKELL, MOSS, RANKINE & HILL

PLATE B-1

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		FIELD DATA
% FINES (-№ 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, kPa	TEST MOISTURE CONTENT, %	SHEAR STRENGTH kPa	NATURAL WATER CONTENT, %	DRY DENSITY, kg/m³	PENETRATION RESISTANCE
							10.9		N = 5
							12.5		N = 5
									N = 3 PUSH
							20.0		
							12.7		N = 16
							26.2	1610	S = 40
							9.5		S = 50/ 200 mm
83	27	7	D.S.	100	16.7	161	17.7	1830	S = 44
			D.S.	170	17.7	156			
			D.S.	240	18.0	196			
							23.9		N = 33
							33.5	1410	S = 60
							23.8	1662	S = 42
							14.6		N = 27
							24.3		N = 26



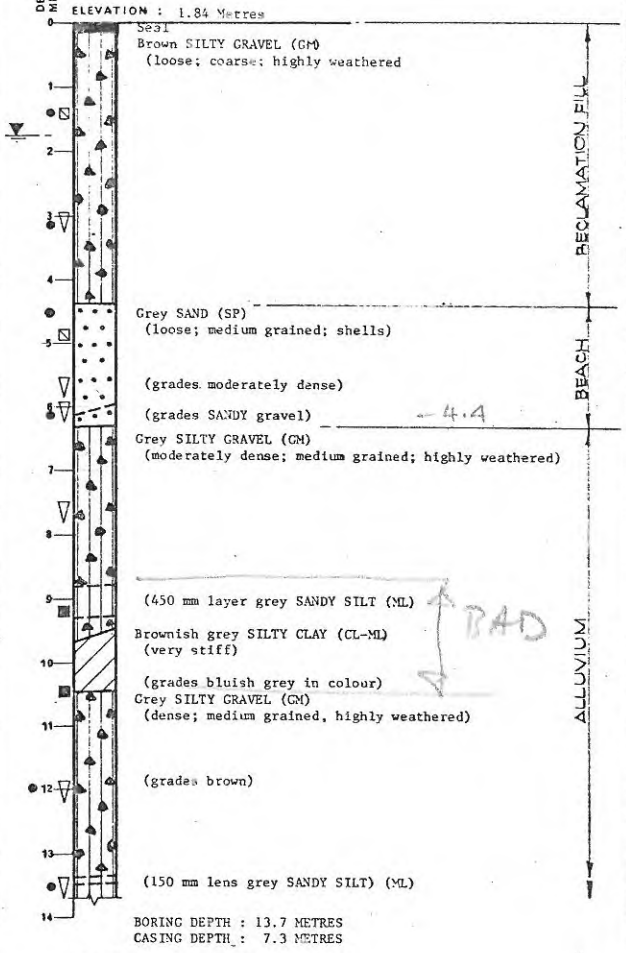
BORING LOGS

BRICKELL, MOSS, RANKINE & HILL

PLATE B-2

BORING 3

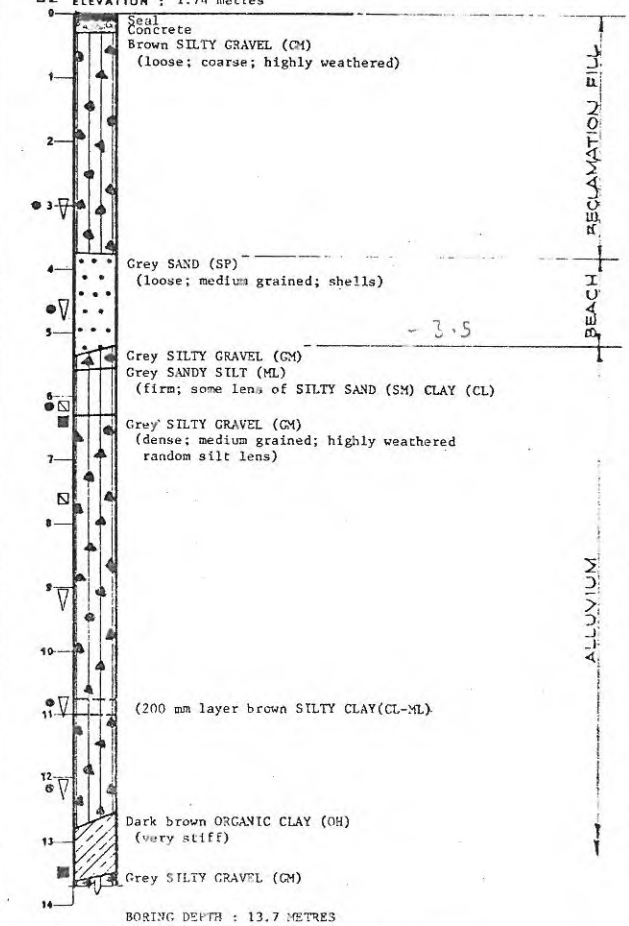
CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		FIELD DATA
% FINES (-№ 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, kPa	TEST MOISTURE CONTENT, %	SHEAR STRENGTH kPa	NATURAL WATER CONTENT, %	DRY DENSITY, kg/m³	PENETRATION RESISTANCE
							13.9		PUSH
									N = 4
							21.1		
									PUSH/ 700 mm
							12·1		N = 15
									N = 20
									N = 30
74	28	4	D.S.	100	22.3	109	25.2	1620	S = 42
			D.S.	160	22.8	142			
			D.S.	220	22.3	197			
							21.5	1700	PUSH
							12.9		N = 38
							12.9		N = 62



BORING DEPTH : 13.7 METRES
CASING DEPTH : 7.3 METRES

BORING 4

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		FIELD DATA
% FINES (-№ 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, kPa	TEST MOISTURE CONTENT, %	SHEAR STRENGTH kPa	NATURAL WATER CONTENT, %	DRY DENSITY, kg/m³	PENETRATION RESISTANCE
							9.0		N = 7
							19.1		N = 19
							21.0		S = 22
							27.8	1660	PUSH/ 200mm
									S = 42
							18.5		N = 32
							15.6		N = 16
							10.2		N = 52
							31.0	1460	PUSH



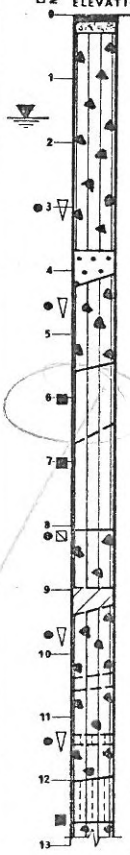
BORING DEPTH : 13.7 METRES
CASING DEPTH : 5.8 METRES
WATER LEVEL ENCOUNTERED BUT NOT MEASURED

BORING LOGS

BRICKELL, MOSS, RANKINE & HILL

BORING 5

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		FIELD DATA
% FINES (-№ 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, kPa	TEST MOISTURE CONTENT, %	SHEAR STRENGTH kPa	NATURAL WATER CONTENT, %	DRY DENSITY, kg/m³	PENETRATION RESISTANCE
							15.7		N = 4
							12.7		N = 22
63	23	2	D.S. D.S. D.S.	70 140 210	22.7 18.1 22.0	70 180 166	22.0	1630	PUSH
									PUSH
							14.6		PUSH/ 200mm
							14.5		N = 35/ 150 mm
							10.7		N = 65
							15.1		S = 38/ 150 mm



ELEVATION : 1.82 Metres

Seal Concrete

Brown SILTY GRAVEL (GM)
(loose; coarse; highly weathered)

RECLAMATION FILL

Grey SAND (SP) — -2.2
(loose; medium grained; shells)

BEACH

Grey SILTY GRAVEL (GM)
(moderately dense; fine; highly weathered)

Grey SANDY SILT (ML)
(firm; some lens of sand (SM & SP)

Greyish brown SILTY GRAVEL (GM)
(dense; coarse; highly weathered)

(300 mm layer brown SILTY CLAY (CL)

(200 mm layer brown SILTY (ML)

(100 mm layer ORGANIC SILT (OL)

Dark brown ORGANIC SILT (OL)
(very stiff)

ALLUVIUM

BORING DEPTH : 12.8 METRES
CASING DEPTH : 5.8 METRES

consol. test

BORING LOGS

BRICKELL, MOSS, RANKINE & HILL

PLATE B-4

JOB 2727/1 CLIENT Building Design Partnership LOCATION Wellington

DRN _____ BY KHG CHECKED _____ DATE 2.11.78

MERCER STREET

kerb & channel

Existing Municipal Chambers

Existing Town Hall

line of existing building to be demolished

CUBA STREET

JERVOIS QUAY

channel

kerb

B1

B2

B3

B4

B5

Existing Building

EXTENT OF PROPOSED STRUCTURE

BORING NUMBER

WAKEFIELD STREET

kerb & channel

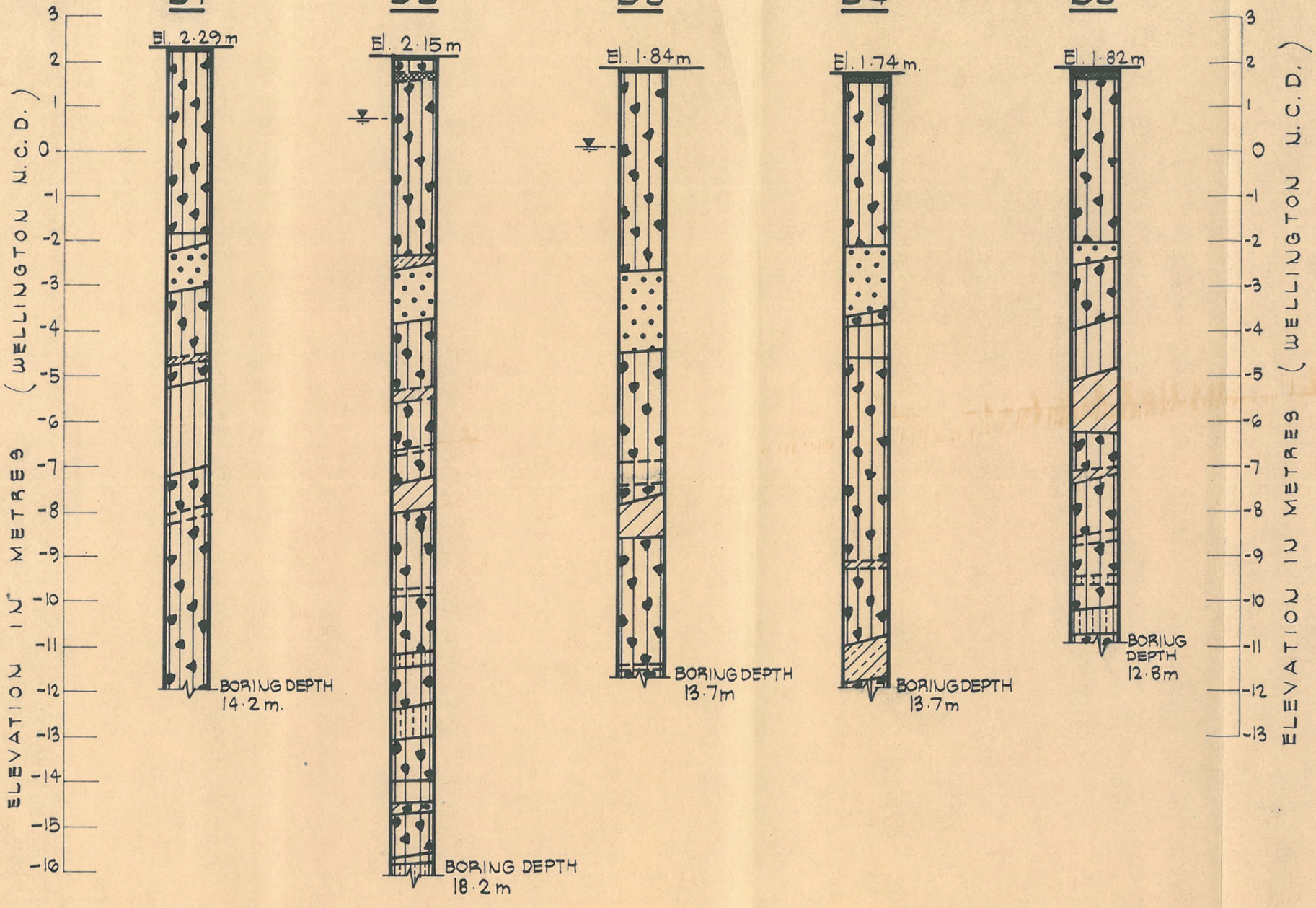
SITE PLAN

Scale : 1 : 480 (40 feet to an inch)

BRICKELL, MOSS, RANKINE & HILL

PLATE 1

JOB 2472/1 CLIENT Hames, Wood, Boam, Sound & Johnstone LOCATION Wellington BY SMH CHECKED DATE 5.12.78



PRELIMINARY BORING LOGS

BRICKELL, MOSS, RANKINE & HILL

PLATE 2

Appendix D Impact of design standard changes

In October 2022, GNS Science released the revised National Seismic Hazard Model (NSHM)[7]. This represents the latest scientific knowledge of earthquake hazard in New Zealand and is an important factor for understanding and managing earthquake risk in the built environment.

While the NSHM will inform future design standards, it does not provide information that can be directly applied in design or assessment applications. Consequently, the current minimum compliance pathway within the Building Code has not changed[8]. However, important updates to Building Code compliance documents that will be informed by the NSHM are expected to be released between 2023 and 2025. It is unknown at this stage, if and how this will affect the assessment of existing buildings, and the %NBS rating of buildings already assessed.

We have undertaken an initial appraisal of the implications of the 2022 NSHM for geotechnical design at the Site (refer seismic hazard comparisons in **Table D1** and **D2** below). It is uncertain how the updated NSHM will be reflected in future design standards, however it is possible that the code minimum seismic design loadings will increase in some situations.

Seismic hazard models carry an inherent amount of uncertainty, but more important is the uncertainty in what shaking a particular site or building will be subject to during its actual life. This depends on which specific earthquakes actually occur over that time. Therefore, designers and building owners are strongly encouraged to focus on resilient design practices, rather than the specific code minimum demand[9].

Liquefaction triggering and associated consequences are non-linear. Our liquefaction analysis has considered a range of seismic loadings, however limited to values between the Module 1 (2016) minimum limit states of SLS and ULS, values beyond ULS were not assessed. However, significant changes to the conclusions in the assessment of the behaviour of the ground, for this Site, are not expected. I.e. liquefaction is generally fully developed at the level of earthquake shaking considered in this assessment. Greater shaking would not significantly increase the thickness of soils liquefying at the Site. The magnitude of lateral spread can be expected to increase with greater levels of shaking, as demonstrated in **Appendix H**.

We recommend that building owners liaise closely with their geotechnical and structural professionals to understand the potential impacts of uncertainty and how this can be managed for the Site, and any proposed strengthening scheme.

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7 https://nshm.gns.cri.nz/

8 Current relevant compliance documents to meet *Clause B1: Structure* of the Building Code are as shown in Verification Method B1/VM1. For structural seismic design this is *NZS 1170.5:2004 – Structural Design Actions Part 5: Earthquake Actions – New Zealand.* For geotechnical design, although not directly referenced in B1/VM1, the Section 175 MBIE/NZGS guidance document *Earthquake Geotechnical Engineering Practice: Module 1 (November 2021)* is to be continued to be used for seismic design loadings.

9 NZSEE, SESOC, NZGS (August 2022). *Earthquake Design for Uncertainty: Advisory.* Revision 1. https://www.nzsee.org.nz/db/PUBS/Earthquake-Design-for-Uncertainty-Advisory_Rev1_August-2022-NZSEE-SESOC-NZGS.pdf

Table D1: Comparison of design seismic shaking parameters (PGA and M) for the Site

NZS 1170.5 Limit State	Return period (years)	PGA (g), M		
		Module 1 (2016)[1], this assessment	Module 1 (2021)[1]	NSHM2022[2]
Ultimate limit state (ULS)	1000	0.45g, M7.1	0.91g, M7.7	1.10g, M?
Serviceability limit state (SLS)	25	0.09g, M6.2	0.13g, M6.5	0.16g, M?

Note 1: Using Method 1 detailed in NZGS Module 1.

Note 2: Based on an assumed V_{s30} = 275 m/s, using GNS online tool.

Table D2: Comparison of assessed PGA likelihood for the Site

PGA (g), this assessment	Likelihood in terms of Module 1 (2016), this assessment		Likelihood in terms of NZHM2022[2]	
	Return period (years)	Probability of exceedance over a 50 year period (%)	Return period (years)	Probability of exceedance over a 50 year period (%)
0.45[1] (100% $ULS_{Module\ 1\ (2016),\ IL3}$)	1000	~5	100 to 150	25 to 40
0.2 (45% $ULS_{Module\ 1\ (2016),\ IL3}$, **liquefaction trigger**)	~80	~45	30 to 40	70 to 80
0.09[1] (100% $SLS_{Module\ 1\ (2016),\ IL3}$)	25	~87	10 to 15	95 to 100

Note 1: Using Method 1 detailed in NZGS Module 1 (2016).

Note 2: Based on an assumed V_{s30} = 275 m/s, GNS online tool. Only approximate ranges are provided, which do not represent the larger uncertainties associated with the hazard model.

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Appendix E Selected existing foundation drawings

Drawings by Holmes, Wood, Poole and Johnstone Consulting Civil and Structural Engineers.

- **Relating to existing piles (marked-up by T+T with details from as-built pile records):**
    - **Drawing no. 666/S1$_F$. *Wellington Town Hall. Pile Layout. (11 June 1980).* [Auditorium]**
    - **Drawing no. 666/S101$_D$. *Wellington Town Hall. Carcass – Pile Layout. Foundation-basement Layout.* (5 June 1980). [Renouf Foyer].**
    - **Drawing no. 666/S29. *Wellington Town Hall. Pile Reinforcing.* (21 September 1979). [Pile cage types].**
    - **As-built pile records are not included in this report, and can be provided upon request.**
- **Relating to ground beams and other shallow foundation elements:**
    - **Drawing no. 666/S26$_C$. *Wellington Town Hall. Foundation Carcass Sections.* (5 June 1981).**
    - **Drawing no. 666/S27$_B$. *Wellington Town Hall. Foundation Carcass Sections.* (5 June 1981).**

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Top = elevation of pile top
Toe$_s$ = elevation of pile reo. base, assumed pile shaft base
Toe$_b$ = elevation of assumed pile base bearing on ground

Cage A
Cage B
Cage C
Cage D
Cage E
Cage F
Cage G

NORTH

LC1 N = 11980 E = 14793 T = rl. 0·650 L = 10000

SjC1 SjC2 SjC3 SjC4 SjC5 SjC6 SjC7 SjC8 SjC9 SjC10 SjC11 SjC12 SjC13 SjC14 SjC15 SjC16 SjC17 SjC18

SjC 17 N = 110 E = 7315 T = rl. 0·850 L = 4400

SjC 0 N = 598 W = 854 T = rl. 0·850 L = 4400

SjC 12 S = 1110 W = 4944 T = rl. 0·850 L = 4400

LC2 S = 38100 W = 9000 T = rl. 0·280 L = 10000

reaction beam for pile test

reaction beam for pile test

NOTE
- see drawing 5101 for pile setout for foyer block

see revised specification [illegible] is issued 11.4.80

- lift caisson (temporary)
- screwjack caisson. (permanent)

PILE LEGEND		ULTIMATE LOAD CAPACITY (tonnes)		
		COMPRESSION	TENSION	
			CAGE A or B	CAGE C or D
	· 450 dia. single bulb ('FRANKI' type pile)	205	–	–
	· 450 dia. double bulb ('FRANKI' type pile)	205	120	170
	· 560 dia. single bulb ('FRANKI' type pile)	450	–	–
	· 560 dia. double bulb ('FRANKI' type pile)	450	120	170
	· 900 dia. with 2000 dia. bell (drilled type pile)	505	390	

NOTES :
- piles generally symmetrical about line C, unless noted otherwise
- references on piles
  N = north of line 3
  S = south of line 3
  E = east of line C
  W = west of line C
  T = rl. of top of pile
  L = length of pile below top rl.
  (see drawing 529)
- see drawing 529 for pile reinforcing cages.
- pile 9E removed
  piles 45E, 47E, 51E, 52E, 53E moved
  piles 45W, 47W, 48W, 49W moved
  piles 101E – 108E added
  piles 101W – 108W added

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WELLINGTON TOWN HALL

PILE LAYOUT

Scale 1 : 100
Drawn R. I. S.
Traced S. L. S.
Approved

F 11.6.80 : screwjack caissons SjC17 & SjC18 located
E 5.5.80 : caissons LC2 & SjC1 to SjC16, top level altered.
D 11.4.80 : 141 and screwjack caissons located.
C 27.2.80 : pile removed; piles moved; extra piles added (see notes)
B 21.1.80 : pile positions under 4th wall altered; pile references added.
A 15.1.80 : some pile positions altered.
21.9.79 Contract

666 / S1. F.

The Contractor shall verify all dimensions prior to commencing work

Top = elevation of pile top
Toe$_s$ = elevation of pile reo. base, assumed pile shaft base
Toe$_b$ = elevation of assumed pile base bearing on ground

Cage A
Cage B
Cage C
Cage D
Cage E
Cage F
Cage G

62E Top = RL 1.3m Toe$_s$ = RL -2.7m Toe$_b$ = RL -3.2m
63E rl. 0.545 L = 5.000 Top = RL 0.5m Toe$_s$ = RL -7.7m Toe$_b$ = RL -8.3m
66E rl. 0.545 L = 5.000 Top = RL 0.5m Toe$_s$ = RL -6.8m Toe$_b$ = RL -7.4m
64E rl. 0.545 L = 5.000 Top = RL 0.5m Toe$_s$ = RL -6.3m Toe$_b$ = RL -6.9m
67E rl. 0.545 L = 5.000 Top = RL 0.5m Toe$_s$ = RL -5.7m Toe$_b$ = RL -6.3m
68E rl. -0.275 L = 2.800 Top = RL -0.3m Toe$_s$ = RL -3.1m Toe$_b$ = RL -3.6m
71E rl. -0.275 L = 2.800 Top = RL -0.3m Toe$_s$ = RL -3.1m Toe$_b$ = RL -3.6m
73E rl. -0.275 L = 2.800 Top = RL -0.3m Toe$_s$ = RL -3.1m Toe$_b$ = RL -3.6m
69E rl. 0.545 L = 2.150 Top = RL -0.9m Toe$_s$ = RL -3.1m Toe$_b$ = RL -3.6m
70E rl. 0.545 L = 2.150 Top = RL -0.9m Toe$_s$ = RL -3.1m Toe$_b$ = RL -3.6m
72E rl. 0.545 L = 2.150 Top = RL -0.9m Toe$_s$ = RL -3.1m Toe$_b$ = RL -3.6m
65E rl. 0.545 L = 5.000 Top = RL 0.5m Toe$_s$ = RL -6.2m Toe$_b$ = RL -6.8m
LC3 rl. -1.950 L = 6700
65W rl. 0.545 L = 2.150 Top = RL -0.9m Toe$_s$ = RL -4.0m Toe$_b$ = RL -4.5m
64W rl. -0.275 L = 2.800 Top = RL -0.3m Toe$_s$ = RL -3.1m Toe$_b$ = RL -3.6m
66W rl. -0.275 L = 2.800 Top = RL -0.9m Toe$_s$ = RL -3.1m Toe$_b$ = RL -3.6m
60W rl. 0.545 L = 5.000 Top = RL 0.5m Toe$_s$ = RL -6.2m Toe$_b$ = RL -6.8m
62W rl. 0.545 L = 5.000 Top = RL 0.5m Toe$_s$ = RL -5.4m Toe$_b$ = RL -6.0m
58W rl. 1.075 L = 4.400 Top = RL 1.3m Toe$_s$ = RL -4.2m Toe$_b$ = RL -4.7m
61W rl. 0.075 L = 5.500 Top = RL 1.0m Toe$_s$ = RL -8.7m Toe$_b$ = RL -9.3m
63W rl. 0.075 L = 5.500 Top = RL 1.0m Toe$_s$ = RL -6.2m Toe$_b$ = RL -6.8m

M.H.
Ejector Station No.6
M.H. cover rl. 1.75
Pumping Station No.6
conc. slab rl. 0.85
M.H. cover rl. 1.83

560 dia. pile double bulbed reinf. cage B
450 dia. pile reinf. cage E
lift caisson (temporary) see revised specification clause 4.13 issued 11.4.80
see sheet S29 for reinforcing detail

pile layout

610 × 510 × 150 deep pocket.
shear ties type A.
rl. 2.000
rl. 1.660
ramp up 1:14
Construction joint
400 × 400 deep beam
1000 × 1500 deep beam
rl. 1.700
rl. 1.970
shear ties type B.
Construction joint with HYDROFOIL & REARGUARD waterstops. See sheet S43.
rl. 0.900
rl. -0.010
rl. -0.050
rl. -0.150
800 × 800 × 650 pile cap
rl. -0.100
300 × 100 trench c/y
400 × 300 × 600 deep sump with 150 dia pipe to drainage trench.
400 wide trench 450 – 500 deep.
rl. 1.970
rl. -1.550
400 × 400 × 400 deep sump
300 thick rc. slab on 50 site concrete
shear ties type C.
800 × 800 × 650 deep pile cap
rl. -0.050
see dtg. S2 for Auditorium details
rl. 1.700
rl. 1.870
rl. 2.000
see on site consultant for location and details of vent pipes for pumping station.

foundation - basement layout

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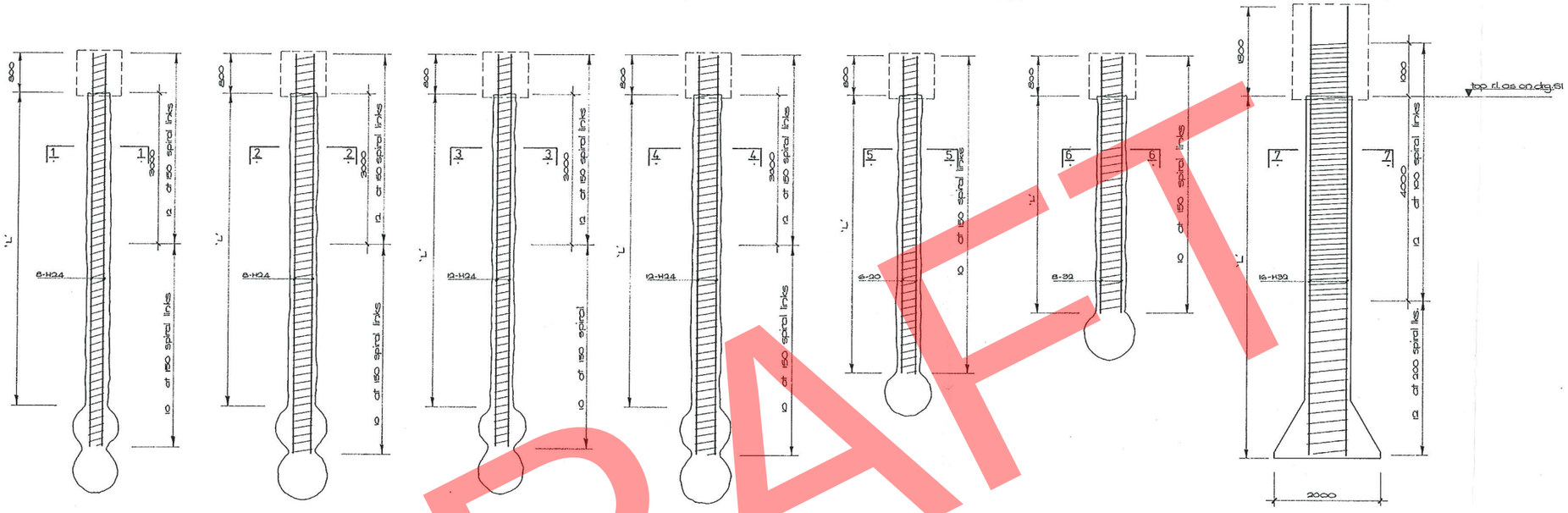
CARCASS - PILE LAYOUT
FOUNDATION-BASEMENT LAYOUT

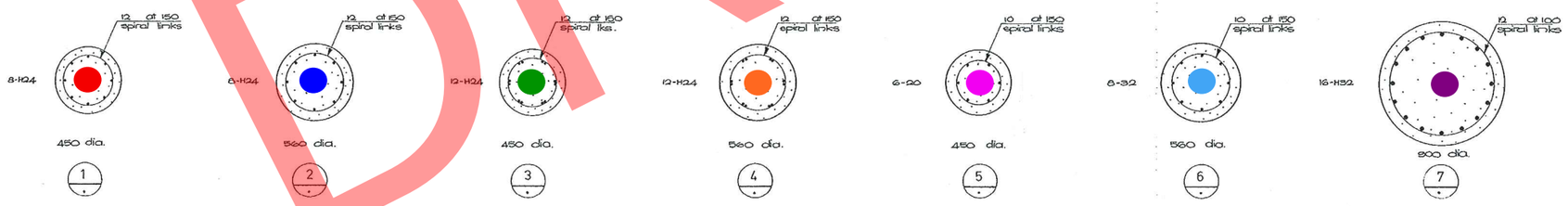
Scale 1 : 100
Drawn R. J. P.
Traced S. L. S.
Approved

D 5. 6. 80. spotcheckpo levels layout amended.
C 29. 4. 80. foundation beam line E. altered.
B 11. 4. 80. lift caisson located
A 28. 4. 80. single pile at C7. basement walls thickened, columns grouped.
21. 9. 79. Contract

666/S101 D

The Contractor shall verify all dimensions prior to commencing work





NOTE
- length 'L' varies, see dtg. S1. & S101.
- all piles to have 75 cover to links

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WELLINGTON TOWN HALL

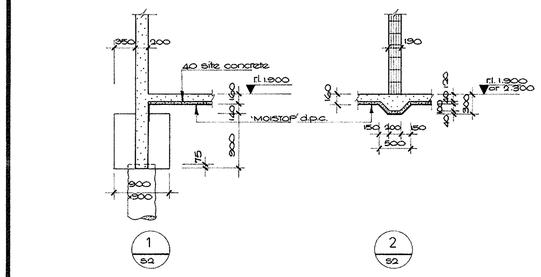
PILE REINFORCING

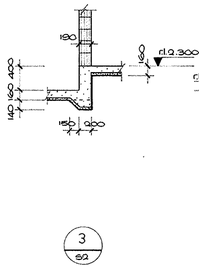
Scale 1 : 50, 1 : 20
Drawn H. M. F.
Traced S. L. S.
Approved

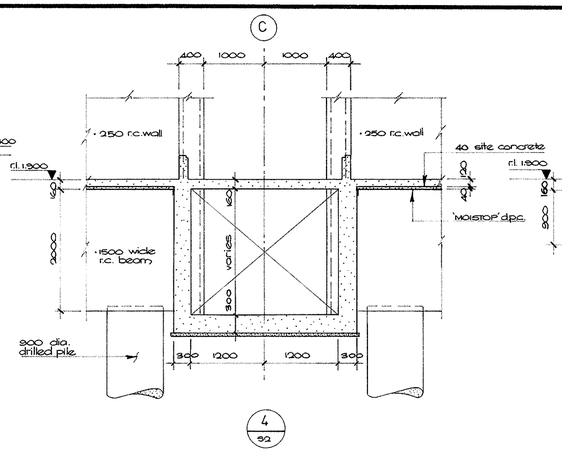
D
C
B
A
21.9.79 Contract

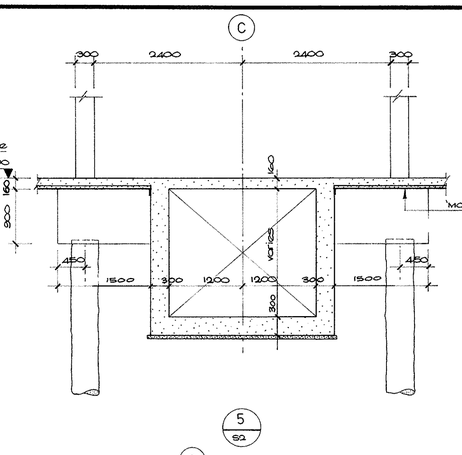
666/S29

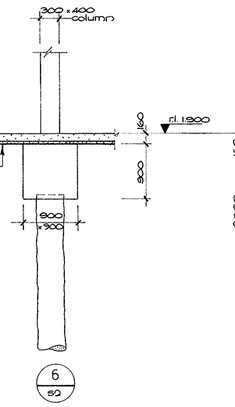
The Contractor shall verify all dimensions prior to commencing work

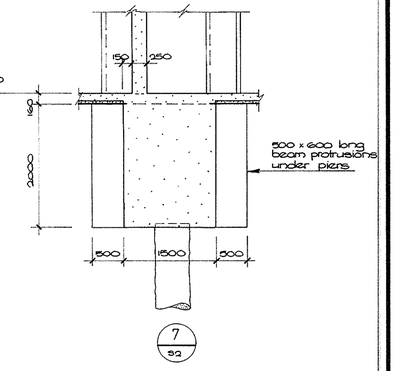


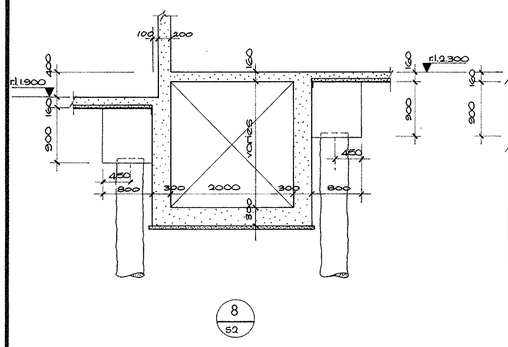


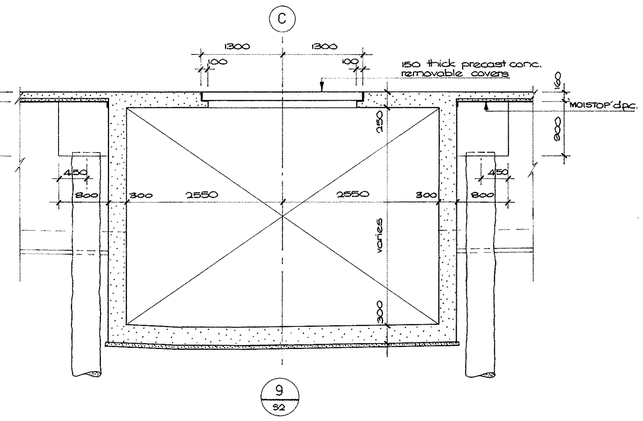


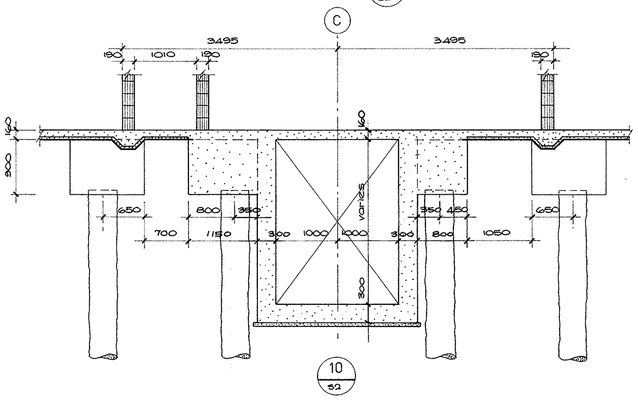


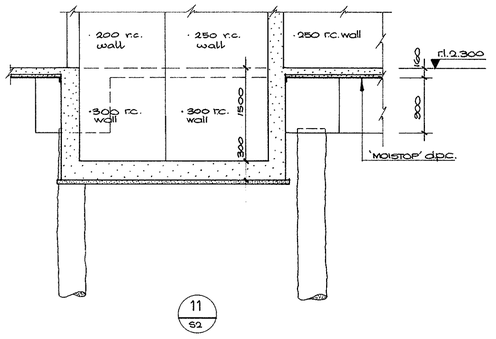


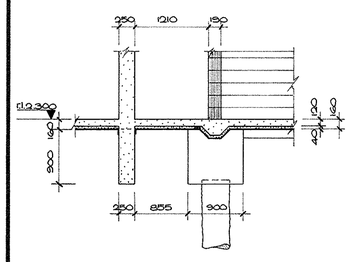


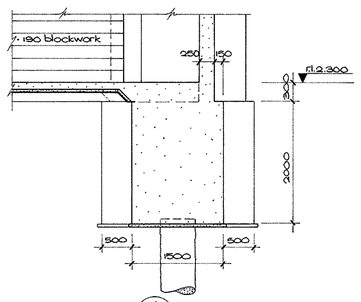


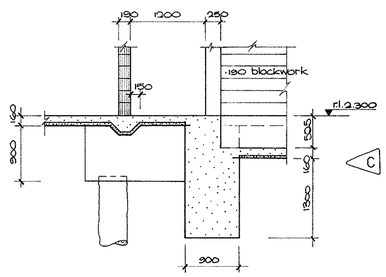


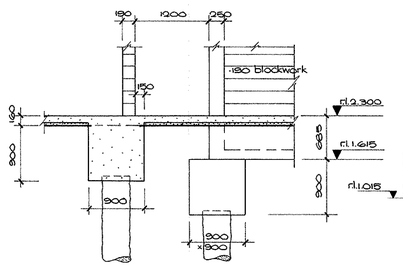


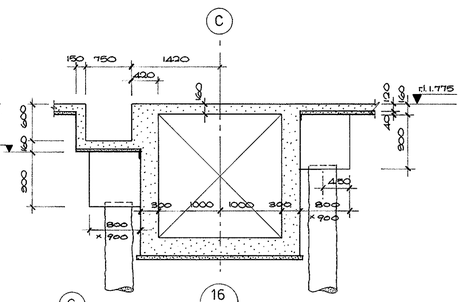


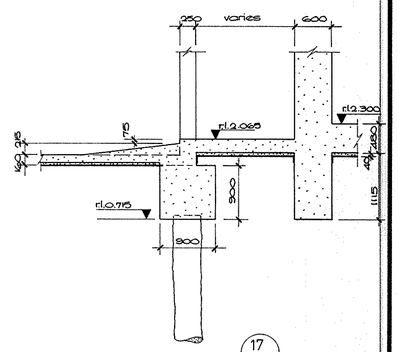


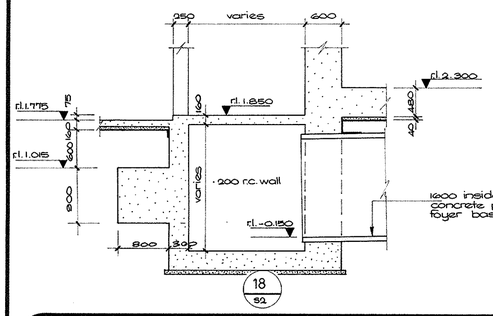


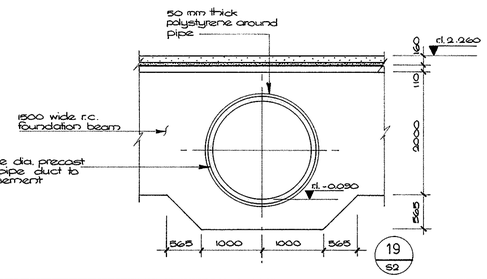


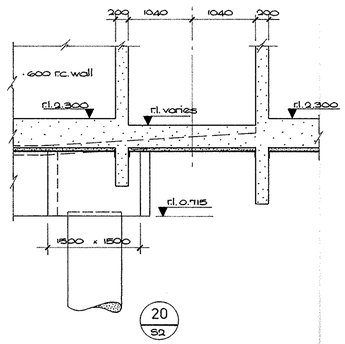


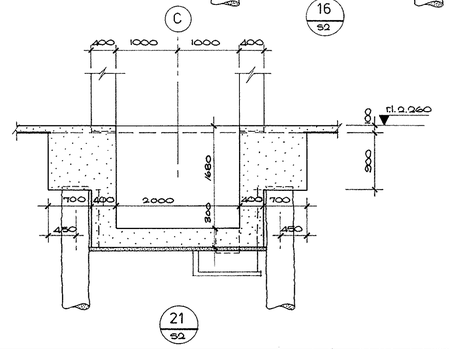












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WELLINGTON TOWN HALL

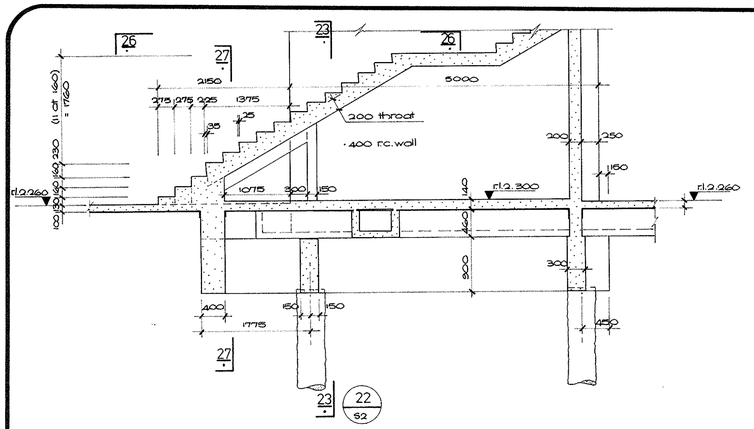
FOUNDATION
CARCASS SECTIONS

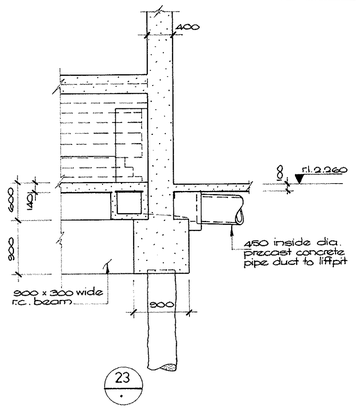
Scale 1 : 50
Drawn R. I. S.
Traced S. L. S.
Approved

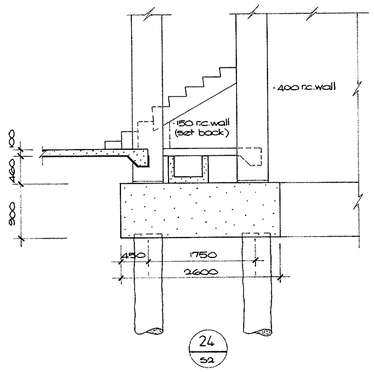
D		
C	5. 6. 81	pilecap/floor slab detail 14/52 modified
B	4. 6. 80	'lift pit increased in depth 'stage pits reduced in depth
A	28. 4. 80	'walls and slabs below L1 level changed to 300 thick & tanking removed
	21. 9. 79	**Contract**

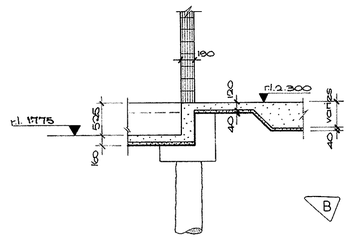
666 / S26 C.

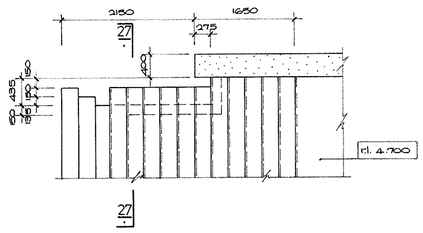
The Contractor shall verify all dimensions prior to commencing work

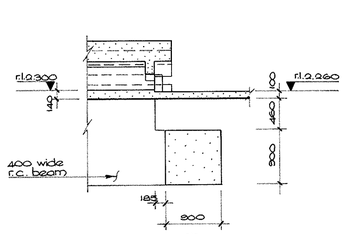


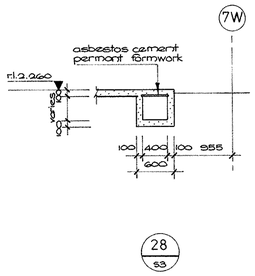


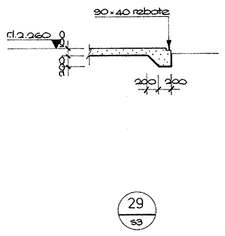


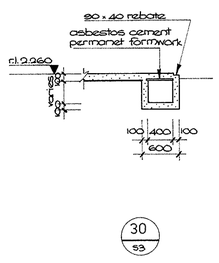


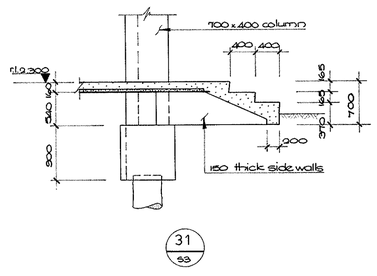


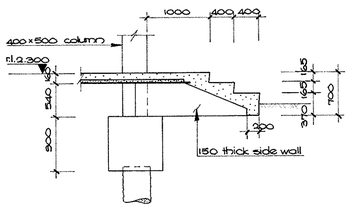


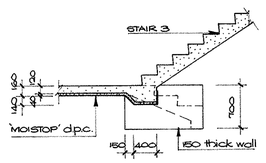


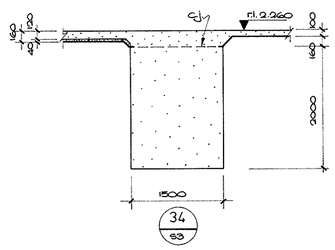


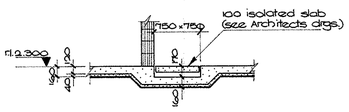


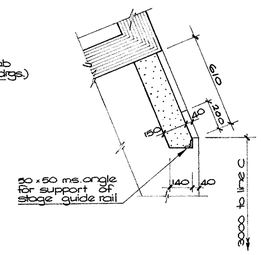


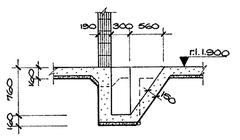


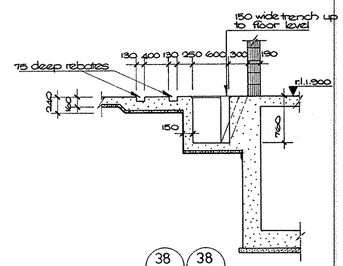


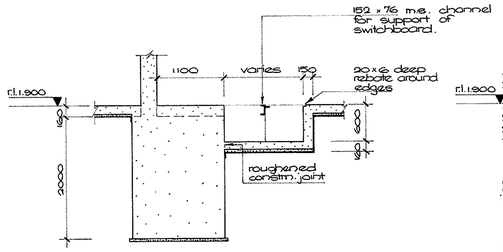


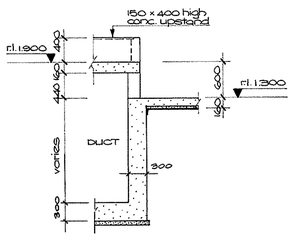


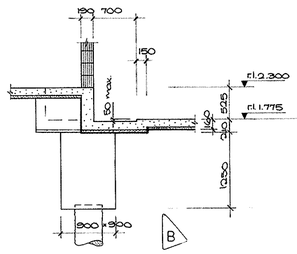


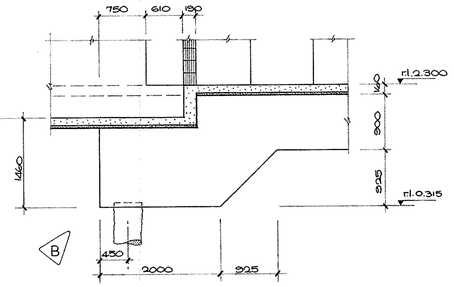












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WELLINGTON TOWN HALL

FOUNDATION
CARCASS SECTIONS

Scale 1 : 50, 1 : 20
Drawn R. I. S.
Traced S. L. S.
Approved

D
C
B 5.6.81 : duct modified details 22, 41 & 42, 52, 53 & 53
A 28.4.80 : walls & stairs below 'level 1' changed to 300 thick & 'tanking' removed.
21.9.79 **Contract**

666 / S27.B.

The Contractor shall verify all dimensions prior to commencing work

Appendix F Lateral pile assessment

Due to the significant amount of analysis output, only summaries of analysis are included in this report. Outputs can be provided upon request.

- **Summary Table F1 – Analysis for no liquefaction case.**
- **Summary Table F2**
    - **A – Analysis for liquefaction only case.**
    - **B – Analysis for no liquefaction case without axial compression loads, to investigate p-delta effects.**
    - **Marked-up foundation plan indicating piles assessed to have negligible lateral capacity (relevant for all cases with liquefaction).**
- **Summary Table F3 – Analysis for cyclic displacement case.**
- **Summary Table F4 – Analysis for lateral spread case.**

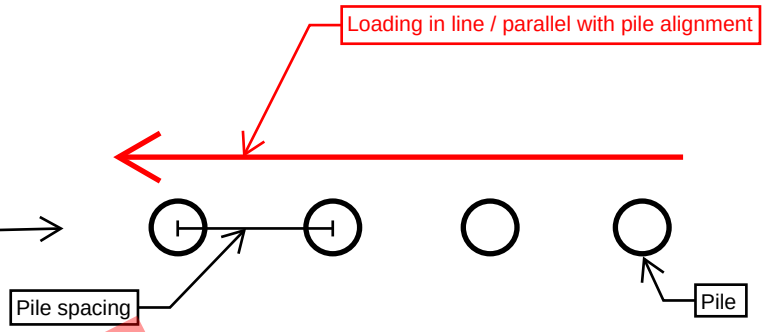
DRAFT

Michael Fowler Centre (MFC) - Geotechnical input to DSA (T+T Ref. 85841.005)
Pile lateral analysis - no liquefaction case

Summary Table F1

To account for the potential variability in the ground lateral stiffness, y-multipliers of 0.75 and 1.25 are to be considered for all lateral springs provided (factors applied to the displacement component of spring)

Pile lateral group effects:
- Applicable to piles loaded in line / loaded parallel to pile alignment (refer example) sketch.
- Piles with > 8D spacing, have no group effects
- Piles with < 4D spacing, have group effects
- Pile with 4D < spacing < 8D, have group effects only if pile head deflection > 15% of pile shaft diameter.



Example sketch of pile loading associated with lateral pile group effects

Summary of LPile analysis:

Cage ID	Pile levels case No.	Top of pile RL (m)	Base of pile reo. RL (m)	Additional analysis for lateral pile group effects	Shaft diameter	Eeff (GPa)	Mcap (kNm)	Axial compression	Pile head condition	Lateral load (kN) applied at pile	Displacement (mm) applied at pile head
A	1	1.3	-6.2	NO	450	12.87	276	954	Free + Fixed	Varies, for pushover analysis	Load case 1 = 20mm; Load case 2 = 40m; Load case 3 = 60mm; Load case 4 = 80mm; Load case 5 = 100mm; Load case 6 = 200mm for BM, SF and BC
	2	1.3	-10								
	3	0.2	-6.2								
	4	0.2	-10								
B	1	1.0	-5.4	NO	560	11.58	500	1108	Free + Fixed		
	2	1.0	-8.7								
	3	0.5	-5.4								
	4	0.5	-8.7								
C	1	1.3	-6.9	NO	450	12.87	355	954	Free + Fixed		
	2	1.3	-7.7								
D	1	1.3	-7.6	NO	560	11.58	597	1108	Free + Fixed		
	2	1.3	-8.4								
	3	0.8	-7.6								
	4	0.8	-8.4								
E	*1*	*1.3*	*-2.6*	NO	450	9.01	219	239	Free + Fixed		
	2	1.3	-10.1								
	3	*-0.9*	*-2.6*								
	4	*-0.9*	*-10.1*								
	5	1.3	-3.8								
	6	1.3	-4.6								
	7	0.2	-4.2								
F	*1*	*0.8*	*-3.3*	YES - expected in fixed head direction for piles supporting ring beam	560	11.58	554	1108	Free + Fixed		
	2	0.8	-6.4								
	3	*-0.2*	*-3.3*								
	4	*-0.2*	*-6.4*								
	5	-0.2	-3.4								
	6	-0.2	-5.2								
	7	0.1	-5.2								
G	1	0.8	-6.7	NO	900	9.01	2612	954	Free + Fixed		
	2	0.8	-11.7								
	3	*-0.2*	*-6.7*	YES - expected in fixed head direction for piles supporting ring beam							
	4	-0.2	-11.7								

DRAFT

Pile level case unchanged

Pile level case not assessed further

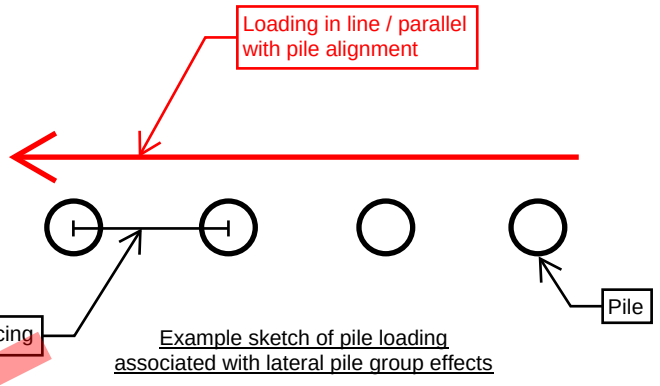
Additional pile level case assessed (requested by Holmes Consulting)

Michael Fowler Centre (MFC) - Geotechnical input to DSA (T+T Ref. 85841.005)
Pile lateral analysis - liquefaction only case (no lateral ground movement)

Summary Table F2a

To account for the potential variability in the ground lateral stiffness, y-multipliers of 0.75 and 1.25 are to be considered for all lateral springs provided (factors applied to the displacement component of spring)

Pile lateral group effects:
- Applicable to piles loaded in line / loaded parallel to pile alignment (refer example) sketch.
- Piles with > 8D spacing, have no group effects
- Piles with < 4D spacing, have group effects
- Pile with 4D < spacing < 8D, have group effects only if pile head deflection > 15% of pile shaft diameter.



Example sketch of pile loading associated with lateral pile group effects

Summary of LPile analysis:

Cage ID	Pile levels case No.	Top of pile RL	Base of pile reo.	Additional analysis for lateral pile group effects	Shaft diameter	E_{eff} (GPa)	M_{cap} (kNm)	Axial compress	Pile head condition	Lateral load (kN) applied	Displacement (mm) applied at pile head
A	1	1.3	-6.2	NO	450	12.87	276	954	Free + Fixed	Varies, for pushover analysis	Load case 1 = 20mm; Load case 2 = 40m; Load case 3 = 60mm; Load case 4 = 80mm; Load case 5 = 100mm; Load case 6 = 200mm; for BM, SF and BC
	2	1.3	-10								
	3	0.2	-6.2								
	4	0.2	-10								
B	1	1.0	-5.4	NO	560	11.58	500	1108	Free + Fixed		
	2	1.0	-8.7								
	3	0.5	-5.4								
	4	0.5	-8.7								
C	1	1.3	-6.9	NO	450	12.87	355	954	Free + Fixed		
	2	1.3	-7.7								
D	1	1.3	-7.6	NO	560	11.58	597	1108	Free + Fixed		
	2	1.3	-8.4								
	3	0.8	-7.6								
	4	0.8	-8.4								
E	*1*	*1.3*	*-2.6*	NO	450	9.01	219	239	Free + Fixed		
	2	1.3	-10.1								
	3	*-0.3*	*-2.6*								
	4	*-0.3*	*-10.1*								
	5	1.3	-3.8								
	6	1.3	-4.6								
	7	0.2	-4.2								
F	*1*	*0.8*	*-3.3*	YES - expected in fixed head direction for piles supporting ring beam	560	11.58	554	1108	Free + Fixed		
	2	0.8	-6.4								
	3	*-0.2*	*-3.3*								
	4	*-0.2*	*-6.4*								
	5	-0.2	-3.4								
	6	-0.2	-5.2								
	7	0.1	-5.2								
G	1	0.8	-6.7	NO	900	9.01	2612	954	Free + Fixed		
	2	0.8	-11.7								
	3	*-0.2*	*-6.7*	YES - expected in fixed head direction for piles supporting							
	4	-0.2	-11.7								

Pile level case unchanged

Pile level case indicating negligible lateral capacity. Case not assessed further. Refer separate foundation plan indicating these piles.

Additional pile level case addressed (requested by Holmes Consulting)

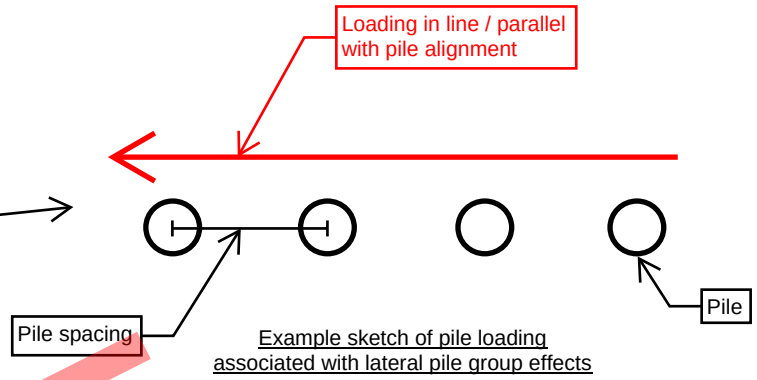
DRAFT

Michael Fowler Centre (MFC) - Geotechnical input to DSA (T+T Ref. 85841.005)
Pile lateral analysis - liquefaction only case (no lateral ground movement)_p-delta check

Summary Table F2b

To account for the potential variability in the ground lateral stiffness, y-multipliers of 0.75 and 1.25 are to be considered for all lateral springs provided (factors applied to the displacement component of spring)

Pile lateral group effects:
- Applicable to piles loaded in line / loaded parallel to pile alignment (refer example) sketch.
- Piles with > 8D spacing, have no group effects
- Piles with < 4D spacing, have group effects
- Pile with 4D < spacing < 8D, have group effects only if pile head deflection > 15% of pile shaft diameter.



Example sketch of pile loading associated with lateral pile group effects

Summary of LPile analysis:

Cage ID	Pile levels case No.	Top of pile RL (m)	Base of pile reo. RL (m)	Additional analysis for lateral pile group effects	Shaft diameter	Eeff (GPa)	Mcap (kNm)	Axial compression	Pile head condition	Lateral load (kN) applied at pile	Displacement (mm) applied at pile head
A	1	1.3	-6.2	NO	450	12.87	276	0	Free + Fixed	Varies, for pushover analysis	Load case 1 = 20mm; Load case 2 = 40m; Load case 3 = 60mm; Load case 4 = 80mm; Load case 5 = 100mm; Load case 6 = 200mm; for BM, SF and BC
	2	1.3	-10								
	3	0.2	-6.2								
	4	0.2	-10								
B	1	1.0	-5.4	NO	560	11.58	500	1108	Free + Fixed		
	2	1.0	-8.7								
	3	0.5	-5.4								
	4	0.5	-8.7								
C	1	1.3	-6.9	NO	450	12.87	355	0	~~Free~~ + Fixed		
	2	1.3	-7.7								
D	1	1.3	-7.6	NO	560	11.58	597	1108	Free + Fixed		
	2	1.3	-8.4								
	3	0.8	-7.6								
	4	0.8	-8.4								
E	*1*	*1.3*	*-2.6*	NO	450	9.01	219	239	Free + Fixed		
	2	1.3	-10.1								
	3	*-0.9*	*-2.6*								
	4	*-0.9*	*-10.1*								
	5	1.3	-3.8								
	6	1.3	-4.6								
	7	0.2	-4.2								
F	*1*	*0.8*	*-3.3*	YES - expected in fixed head direction for piles supporting ring beam; **NOT assessed for Run 3, as requested by Holmes.**	560	11.58	554	0	Free + Fixed		
	2	0.8	-6.4								
	3	*-0.2*	*-3.3*								
	4	*-0.2*	*-6.4*								
	5	-0.2	-3.4								
	6	-0.2	-5.2								
	7	0.1	-5.2								
G	1	0.8	-6.7	NO	900	9.01	2612	954	Free + Fixed		
	2	0.8	-11.7								
	3	*-0.2*	*-6.7*	YES - expected in fixed head direction for piles supporting							
	4	-0.2	-11.7								

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Pile level case assessed

Pile level case not assessed (as requested by Holmes Consulting)

Top = elevation of pile top
Toe$_s$ = elevation of pile reo. base, assumed pile shaft base
Toe$_b$ = elevation of assumed pile base bearing on ground

Cage A
Cage B
Cage C
Cage D
Cage E
Cage F
Cage G

Piles with negligible lateral capacity in liquefaction only case

This Figure relates to Summary Table F2.

NORTH

3E

C

3W

LC1 N = 11980 E = 14793 T = rl. 0.650 L = 10000

LC2 S = 38100 W = 0000 T = rl. 0.280 L = 10000

SjC1 SjC2 SjC3 SjC4 SjC5 SjC6 SjC7 SjC8 SjC9 SjC10 SjC11 SjC12 SjC13 SjC14 SjC15 SjC16 SjC17 SjC18

SjC 17 N = 110 E = 7315 T = rl. 0.850 L = 4400

SjC 9 N = 420 W = 3005 T = rl. 0.850 L = 1500

SjC12 S = 110 W = 4244 T = rl. 0.850 L = 4400

SjC13 N = 1509 W = 5884 T = rl. 0.850 L = 4400

SjC15 N = 2420 W = 8102 T = rl. 0.850 L = 4400

SjC16 N = 712 W = 8708 T = rl. 0.850 L = 4400

SjC18 S = 159 W = 6018 T = rl. 0.850 L = 4400

reaction beam for pile test

reaction beam for pile test

NOTE
- see drawing S101 for pile setout for foyer block

NOTES
- piles generally symmetrical about line C, unless noted otherwise
- references on piles
  N = north of line 3
  S = south of line 3
  E = east of line C
  W = west of line C
  T = rl. of top of pile
  L = length of pile below top rl. (see drawing S29)
- see drawing S29 for pile reinforcing cages.
- pile 9E removed
  piles 45E, 47E, 51E, 52E, 53E moved
  piles 45W, 47W, 48W, 49W moved
  piles 101E - 108E added
  piles 101W - 108W added

see revised specification [illegible] is issued 11.4.80

		ULTIMATE LOAD CAPACITY (tonnes)		
	• lift caisson (temporary)	COMPRESSION	TENSION	
	• screwjack caisson. (permanent)		CAGE A or B	CAGE C or D
PILE LEGEND				
	• 450 dia. single bulb ('FRANKI' type pile)	205	–	–
	• 450 dia. double bulb ('FRANKI' type pile)	205	120	170
	• 560 dia. single bulb ('FRANKI' type pile)	450	–	–
	• 560 dia. double bulb ('FRANKI' type pile)	450	120	170
	• 900 dia. with 2000 dia. bell (drilled type pile)	505	390	

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Christchurch & Wellington

WELLINGTON TOWN HALL

PILE LAYOUT

Scale 1 : 100
Drawn R. I. S.
Traced S. L. S.
Approved

F 11. 6. 80 : screwjack caissons SjC17 & SjC18 located
E 5. 5. 80 : caissons LC2 & SjC1 to SjC16, top level altered.
D 11. 4. 80 : lift and screwjack caissons located.
C 27. 2. 80 : pile removed; piles moved; extra piles added. (see notes)
B 21. 1. 80 : pile positions under 4th wall altered; pile reference added.
A 15. 1. 80 : some pile positions altered.
21. 9. 79 **Contract**

666 / S1. F.

The Contractor shall verify all dimensions prior to commencing work

Top = elevation of pile top
Toe$_s$ = elevation of pile reo. base, assumed pile shaft base
Toe$_b$ = elevation of assumed pile base bearing on ground

Cage A
Cage B
Cage C
Cage D
Cage E
Cage F
Cage G

Piles with negligible lateral capacity in liquefaction only case

This Figure relates to Summary Table F2.

DRAFT

pile layout

foundation - basement layout

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CARCASS - PILE LAYOUT
FOUNDATION-BASEMENT LAYOUT

Scale 1 : 100
Drawn R. J. P.
Traced S. L. S.
Approved

D 5.6.80. Construction joints layout amended, drainage trench/sump revised.
C 29.4.80. foundation beam line E altered.
B 11.4.80. lift caisson located
A 28.4.80. single pile at C7, basement walls thickened, columns precast.
21.9.79. Contract

666/S101 D

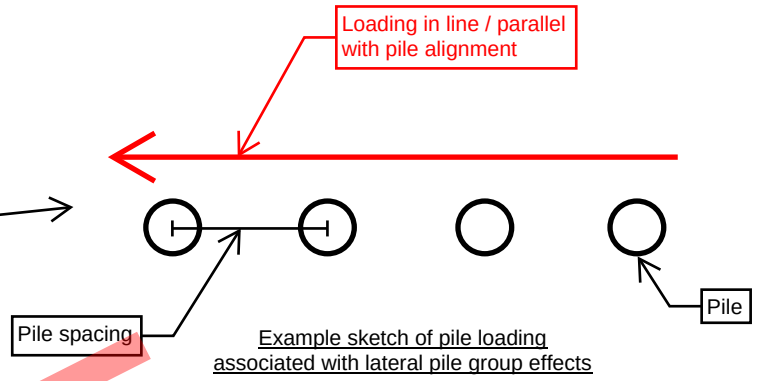
The Contractor shall verify all dimensions prior to commencing work

Michael Fowler Centre (MFC) - Geotechnical input to DSA (T+T Ref. 85841.005)
Pile lateral analysis - cyclic displacement case

Summary Table F3

To account for the potential variability in the ground lateral stiffness, y-multipliers of 0.75 and 1.25 are to be considered for all lateral springs provided (factors applied to the displacement component of spring)

Pile lateral group effects:
- Applicable to piles loaded in line / loaded parallel to pile alignment (refer example) sketch.
- Piles with > 8D spacing, have no group effects
- Piles with < 4D spacing, have group effects
- Pile with 4D < spacing < 8D, have group effects only if pile head deflection > 15% of pile shaft diameter.



Example sketch of pile loading associated with lateral pile group effects

Summary of LPile analysis:

Lateral load includes: load from passive earth pressure on ground beam above + building base shear load

Cage ID	Pile levels case No.	Top of pile RL	Base of pile reo.	Additional analysis for lateral pile group effects, refer note 1	Shaft diameter	E$_{eff}$ (GPa)	M$_{cap}$ (kNm)	Axial compressio	Pile head condition	Lateral load (kN) applied at pile head
A	1	1.3	-6.2	NO	450	12.87	276	954	Free + Fixed	Load case 1 = 0kN; Load case 2 = 25kN; Load case 3 = 50kN; Load case 4 = 75kN; Load case 5 = 100kN; Load case 6 = 150kN; Load case 7 = 200kN; Load case 8 = 300kN; Load case 9 = 400kN; Load case 10 = 500 kN
	2	1.3	-10							
	3	0.2	-6.2							
	4	0.2	-10							
B	1	1.0	-5.4	NO	560	11.58	500	1108	Free + Fixed	
	2	1.0	-8.7							
	3	0.5	-5.4							
	4	0.5	-8.7							
C	1	1.3	-6.9	NO	450	12.87	355	954	Free + Fixed	
	2	1.3	-7.7							
D	1	1.3	-7.6	NO	560	11.58	597	1108	Free + Fixed	
	2	1.3	-8.4							
	3	0.8	-7.6							
	4	0.8	-8.4							
E	*1*	*1.3*	*-2.6*	NO	450	9.01	219	239	Free + Fixed	
	2	1.3	-10.1							
	3	*-0.9*	*-2.6*							
	4	*-0.9*	*-10.1*							
	5	1.3	-3.8							
	6	1.3	-4.6							
	7	0.2	-4.2							
F	*1*	*0.8*	*-3.3*	YES - expected in fixed head direction for piles supporting ring beam	560	11.58	554	1108	Free + Fixed	
	2	0.8	-6.4							
	3	*-0.2*	*-3.3*							
	4	*-0.2*	*-6.4*							
	5	-0.2	-3.4							
	6	-0.2	-5.2							
	7	0.1	-5.2							
G	1	0.8	-6.7	NO	900	9.01	2612	954	Free + Fixed	Load case 1 = 0kN; Load case 2 = 200kN; Load case 3 = 400kN; Load case 4 = 600kN; Load case 5 = 800kN; Load case 6 = 1000kN; Load case 7 = 1200kN; Load case 8 = 1400kN; Load case 9 = 1600kN; Load case 10 = 1800 kN
	2	0.8	-11.7							
	3	*-0.2*	*-6.7*	YES - expected in fixed head direction for piles supporting ring beam						
	4	-0.2	-11.7							

Some Cage F piles with lower lateral capacity analyzed for adjusted lateral loads:

Load case	Lateral loads (kN)	
	Pile F5, pinned, with and wihtout group effects	Pile F6 and F7, pinned, with group effects
1	0	0
2	5	50
3	10	100
4	15	150
5	20	200
6	25	250
7	-	275

Pile level case unchanged

Pile level case not assessed further

Additional pile level case assessed (requested by Holmes Consulting)

DRAFT

Michael Fowler Centre (MFC) - Geotechnical input to DSA (T+T Ref. 85841.005)
Pile lateral analysis - lateral spread case

Summary Table F4

To account for the potential variability in the ground lateral stiffness, y-multipliers of 0.75 and 1.25 are to be considered for all lateral springs provided (factors applied to the displacement component of spring)

Lateral load includes: load from passive earth pressure on ground beam above + building base shear load

Summary of LPile analysis:

Cage ID	Pile levels case No.	Top of pile RL (m)	Base of pile reo.	Comments	Shaft diameter	E_{eff} (GPa)	M_{cap} (kNm)	Axial compressio	Pile head condition	Lateral load (kN) applied at pile
A	1	1.3	-6.2		450	12.87	276	954	Free + Fixed	
	2	1.3	-10							
	3	0.2	-6.2							
	4	0.2	-10							
B	1	1.0	-5.4		560	11.58	500	1108	Free + Fixed	
	2	1.0	-8.7							
	3	0.5	-5.4							
	4	0.5	-8.7							
C	1	1.3	-6.9		450	12.87	355	954	Free + Fixed	
	2	1.3	-7.7							
D	1	1.3	-7.6		560	11.58	597	1108	Free + Fixed	Load case 1 = 0kN; Load case 2 = 25kN; Load case 3 = 50kN; Load case 4 = 75kN; Load case 5 = 100kN; Load case 6 = 150kN
	2	1.3	-8.4	Pinned case did not run.						
	3	0.8	-7.6							
	4	0.8	-8.4							
E	*1*	*1.3*	*-2.6*		450	9.01	219	239	Free + Fixed	
	2	1.3	-10.1							
	3	*-0.9*	*-2.6*							
	4	*-0.9*	*-10.1*							
	5	1.3	-3.8							
	6	1.3	-4.6							
	7	0.2	-4.2							
F	*1*	*0.8*	*-3.3*		560	11.58	554	1108	Free + Fixed	Load case 1 = 0kN; Load case 2 = 25kN; Load case 3 = 50kN; Load case 4 = 75kN; Load case 5 = 100kN; Load case 6 = 150kN
	2	0.8	-6.4	Pinned case did not run.						
	3	*-0.2*	*-3.3*							
	4	*-0.2*	*-6.4*							
	5	-0.2	-3.4	Pinned case did not run.						
	6	-0.2	-5.2							
	7	0.1	-5.2							
G	1	0.8	-6.7		900	9.01	2612	954	Free + Fixed	
	2	0.8	-11.7							
	3	*-0.2*	*-6.7*							
	4	-0.2	-11.7							

DRAFT

Pile level case assessed (note that pinned head cases did not run/pile unstable)

Pile level case unchanged

Pile level case not assessed further

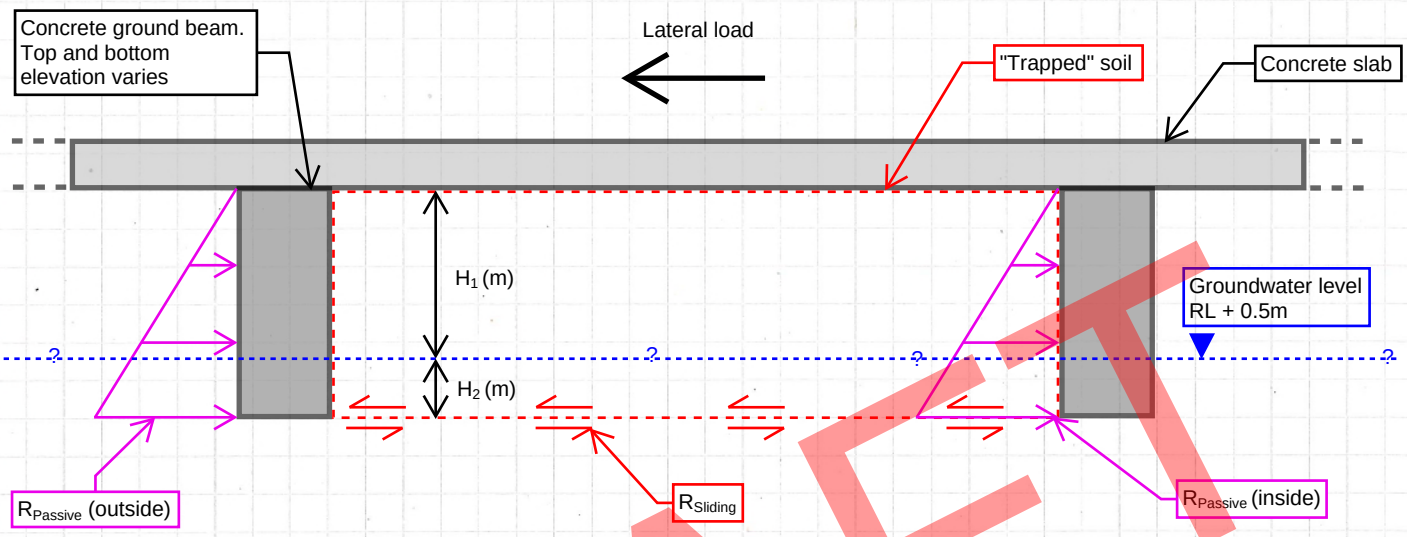
Additional pile level case assessed (requested by Holmes Consulting)

Appendix G Lateral resistance of ground beams

- **Sketch G1 – Ground beam lateral resistance, passive vs. sliding.**
- **Plot G1 – Probable lateral passive resistance for ground beams, without liquefaction.**
- **Plot G2 – Probable lateral passive resistance for ground beams, with liquefaction.**

DRAFT

Tonkin+Taylor	Project: MFC DSA	Computed: ANRO 02/08/2022	Office: Wellington
		Checked: 20	Job No: 85841.0050
Description: Lateral ground resistance from ground beams (passive earth) vs. sliding resistance of ground "trapped" between beams.		Revised: 20	File: -
		Checked: 20	Sheet No. **Sketch G1**



Elevation view of ground beams and soil "trapped" in-between

DRAFT

Geotechnical parameters:

R = Resistance from ground.

Adopted groundwater level = average tidal groundwater level + probable sea level rise.

$R_{Passive}$ (kN/m) = refer lateral load-displacement plots previously provided for with and without liquefaction.
-> to be applied along the ground beam length in direct contact with the ground.

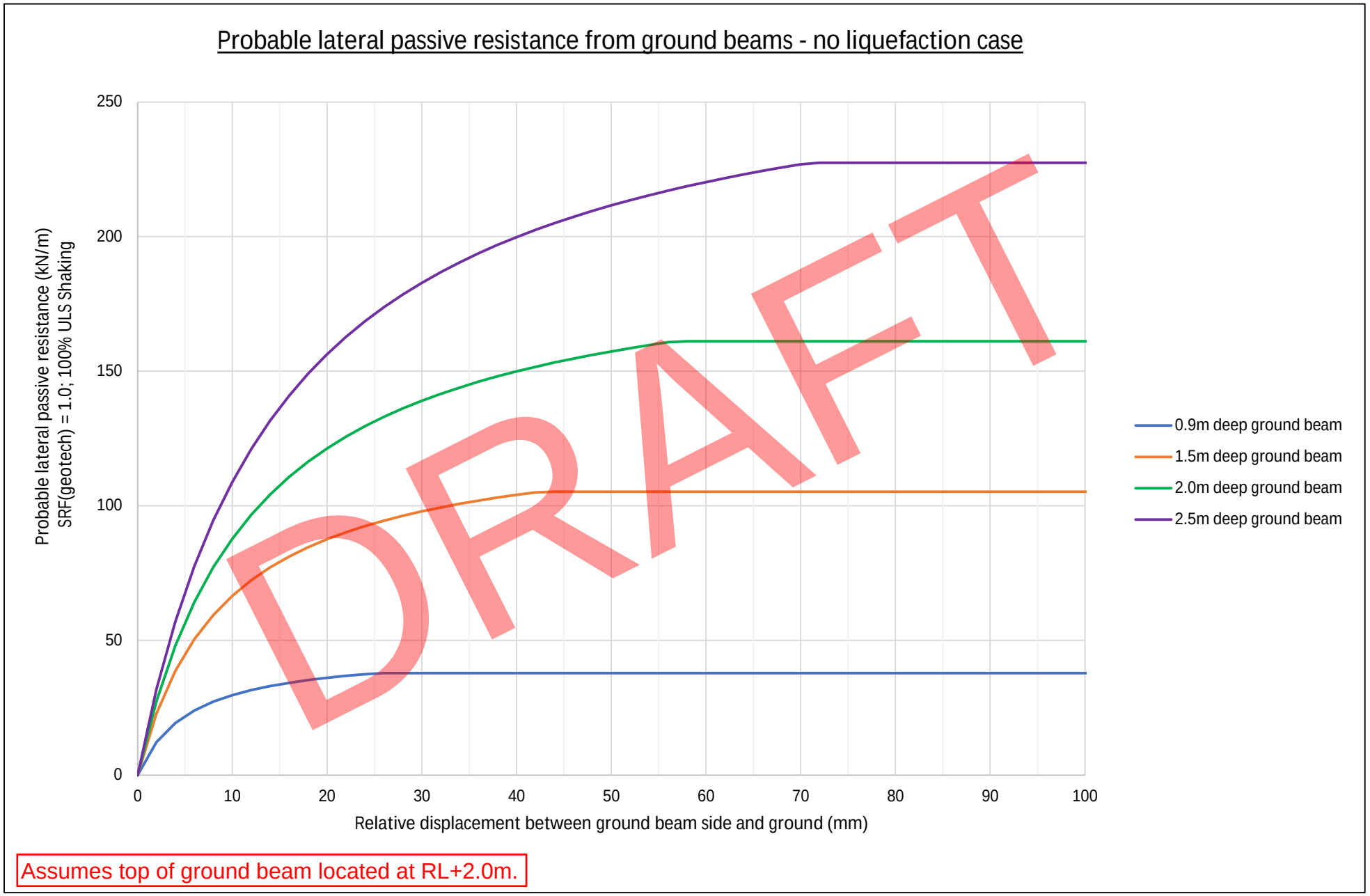
$R_{Sliding}$ (kPa), **without** liquefaction = ((H_1 x 19kN/m^3) + (H_2 x 9kN/m^3)) x 0.65. -> to be applied over the plan area of the "trapped" soil.

$R_{Sliding}$ (kPa), **with** liquefaction = 8.5 kPa -> to be applied over the plan area of the "trapped" soil.

=> Use lowest resisting load estimated from $R_{Sliding}$ and $R_{Passive}$ **(inside)**, for any particular foundation bay.

Michael Fowler Centre - Geotechnical input to DSA
Probable lateral passive resistance from ground beams for 100%ULS(IL3)$_{\text{Module 1 (2016)}}$ - No liquefaction case

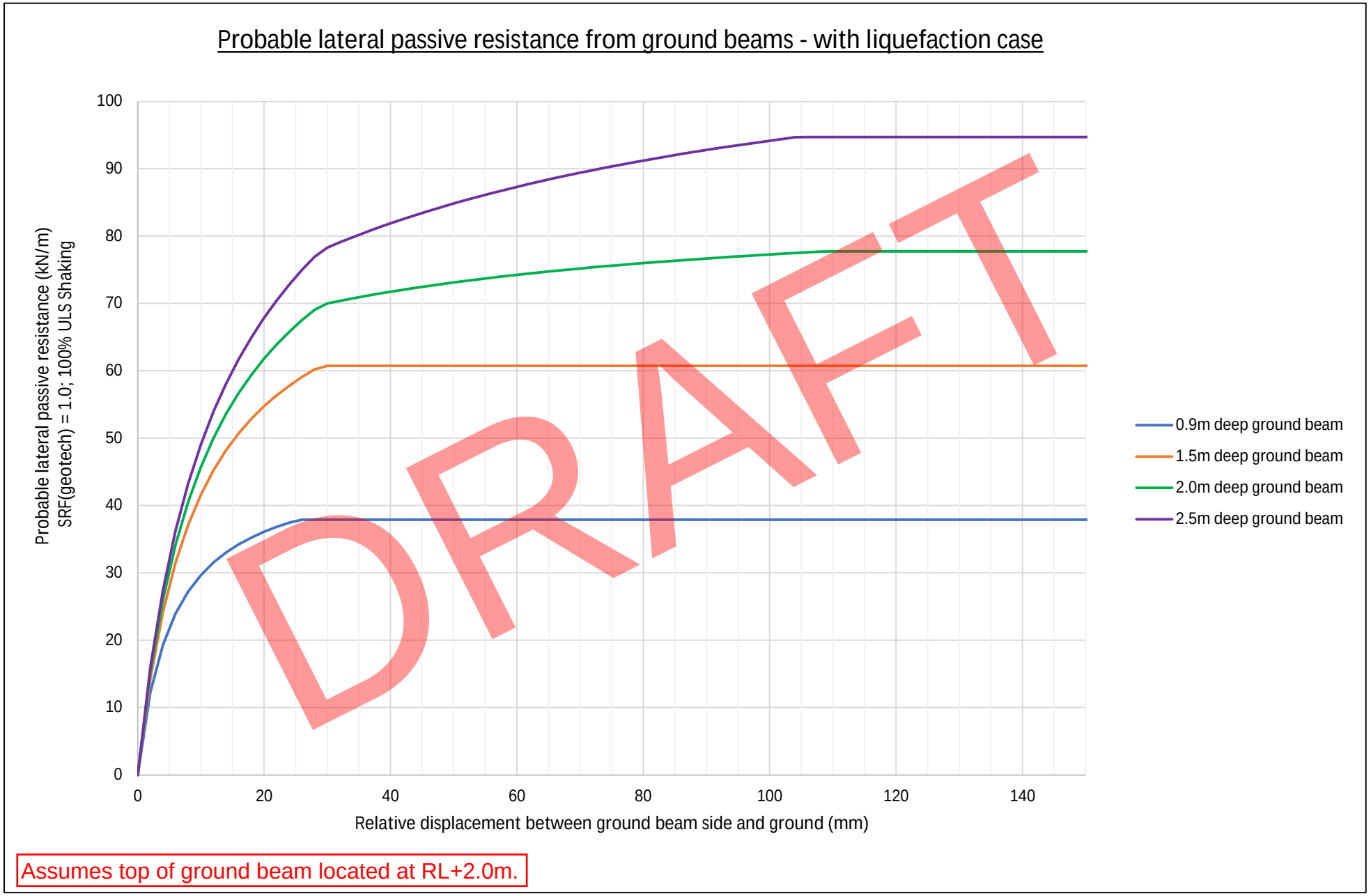
Plot G1



Assumes top of ground beam located at RL+2.0m.

Michael Fowler Centre - Geotechnical input to DSA
Probable lateral passive resistance from ground beams for 100%ULS(IL3)$_{\text{Module 1 (2016)}}$ - With liquefaction case

Plot G2

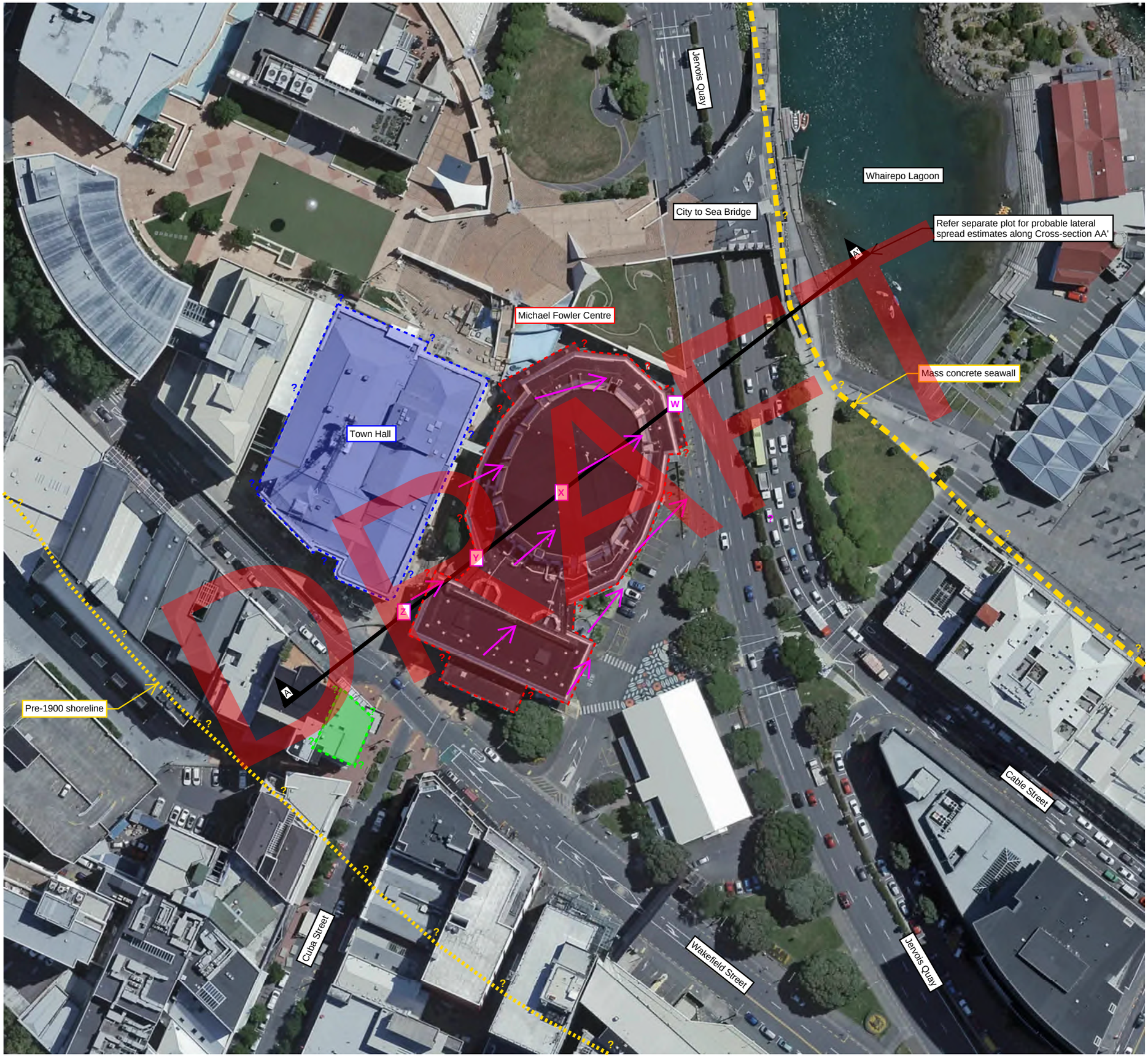


Assumes top of ground beam located at RL+2.0m.

Appendix H Lateral spread assessment

- **Figure H1 – Site plan of lateral spread assessment.**
- **Plot H1 – Probable estimate of lateral spread across the Site.**
- **Plot H2 – Probable estimate of lateral spread at selected distances from seawall.**

DRAFT



LEGEND

Cross-section AA'

Buildings (approximate extent)

- Michael Fowler Centre (the Site)
- Town Hall
- Greenwich

Approximate boundaries

- Pre-1900 shoreline
- Mass concrete seawall & reclamation fill boundary

Lateral spread ground displacement

- Indicative lateral spread vector
- # Point/location along cross-section for lateral spread assessment (refer Figure H2)

Notes:

1. **ULS(IL3) based on MBIE/NZGS Module 1(2016):Method 1.**
2. **Displacements are indications of the risk of lateral spread rather than representing absolute magnitudes. An 'uncertainty factor' of half and double of the reported probable lateral displacements should be considered by Holmes as a sensitivity.**

A3 SCALE: 1:1,000

0 10 20 30 40 50 (m)

N

1. LINZ Aerial Basemap Copyright LINZ CC BY 4.0. www.linz.govt.nz/linz-copyright Imagery Basemap Contributors. https://www.linz.govt.nz/data/linz-data/linz-basemaps/data-attribution

2. Reclamation boundaries are approximate only. Based on Semmens (2010) - An Engineering Geological Investigation of the Central Wellington Area.

Created On:	30/06/2022
Created By:	ARolfe
Approved By:	
TT Proj Ref:	85841.0050
TT Map Ref:	TTMAPREF1433463001.902

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Michael Fowler Centre (MFC)
Geotechnical Seismic Assessment
Lateral Spread Assessment
Site Plan

FIGURE No. H1

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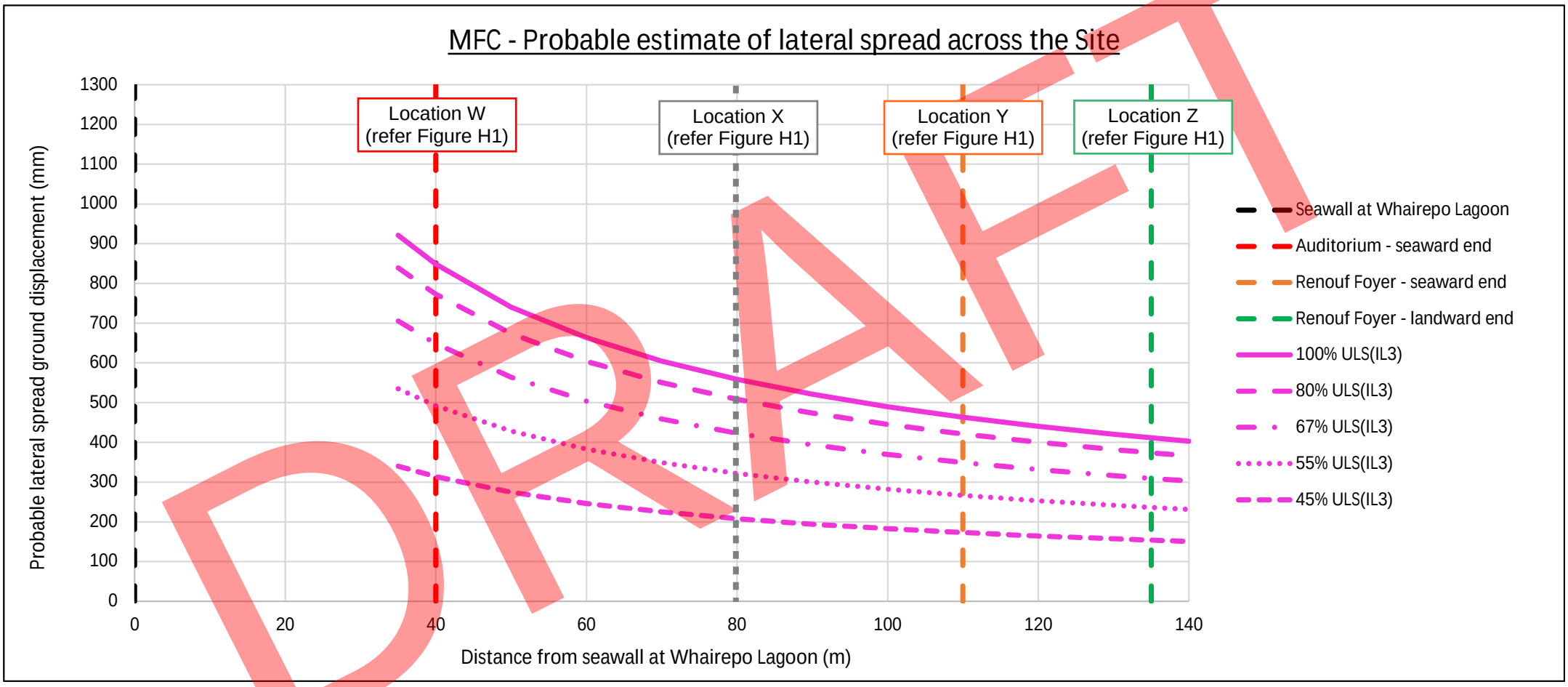
Plot H1



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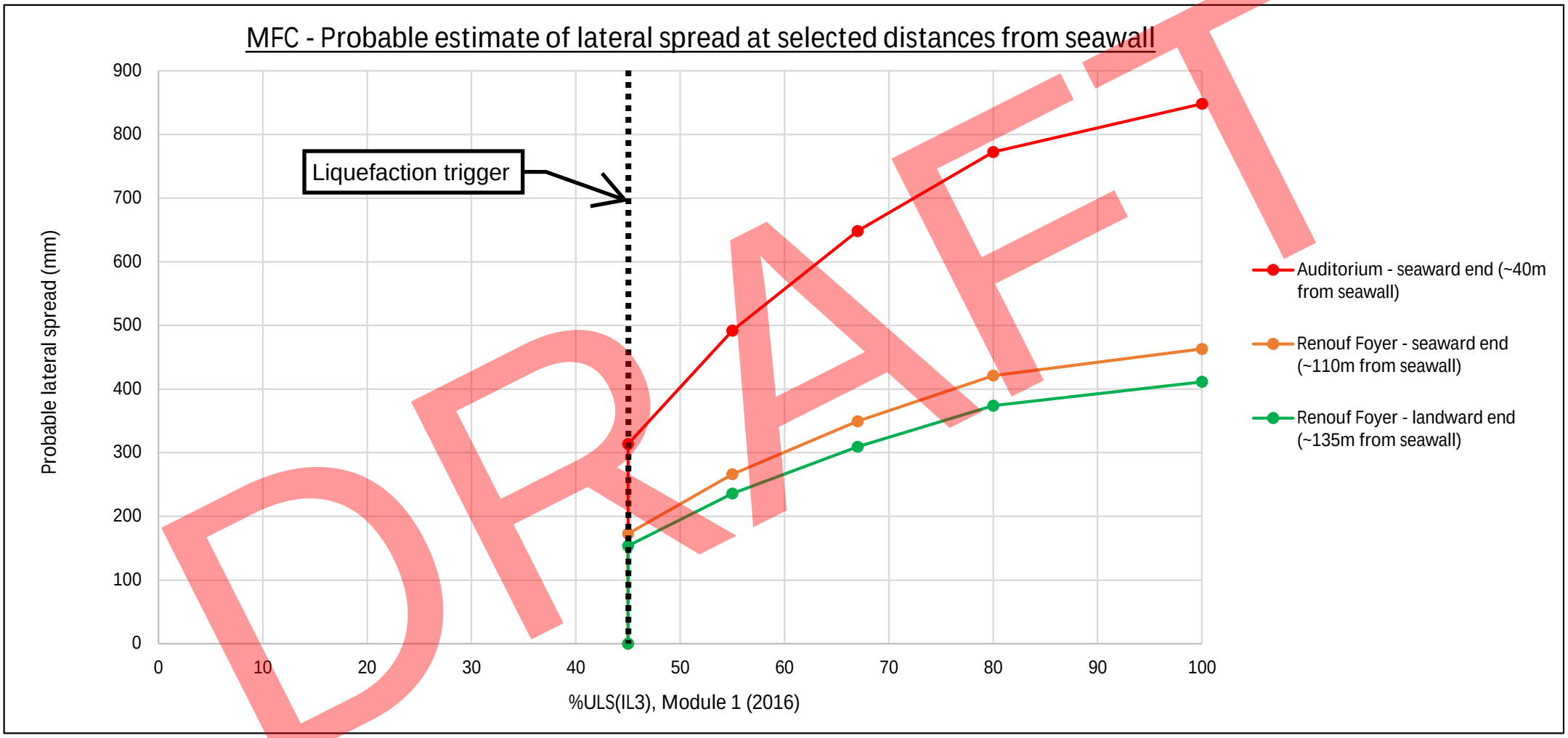
Notes:

1. ULS(IL3) based on MBIE/NZGS Module 1 (2016): Method 1.

2. Displacements are indications of the risk of lateral spread rather than representing absolute magnitudes.
An 'uncertainty factor' factor of half and double of the probable lateral displacements indicated in this plot should be considered by Holmes as a sensitivity.

Plot H2



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Notes:

1. ULS(IL3) based on MBIE/NZGS Module 1 (2016): Method 1.

2. Displacements are indications of the risk of lateral spread rather than representing absolute magnitudes.
An 'uncertainty factor' factor of half and double of the probable lateral displacements indicated in this plot should be considered by Holmes as a sensitivity.

Appendix C

Technical Summary of the Auditorium Analysis and Evaluation

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Appendix C Technical Summary of the Auditorium Analysis and Evaluation

C.1 INTRODUCTION

The Auditorium is analysed using a Modal Response Spectrum Analysis (MRSA) performed in accordance with Section C2.8.2 of the Engineering Assessment Guidelines.

The assessment of the primary structure of the Auditorium is suitable for MRSA as it has low ductility ($\mu \leq 2.0$) and has no significant weak storey present. The Auditorium has some in-plane discontinuities in the Lobby area on the Southern end, located where the bay walls of Bays 6(a) and 7(a) have their web area terminated at the ground floor level. However, through the sloping transfer diaphragm below the Level 2 seating (referred to as Level 1/2), the structure has a rigid load path at this location. The structure is symmetrical along its longitudinal axis.

Inelastic torsion is considered in accordance with Section C2.5.8 of the Engineering Assessment Guidelines and is further described in Section C.2.7 of this Appendix.

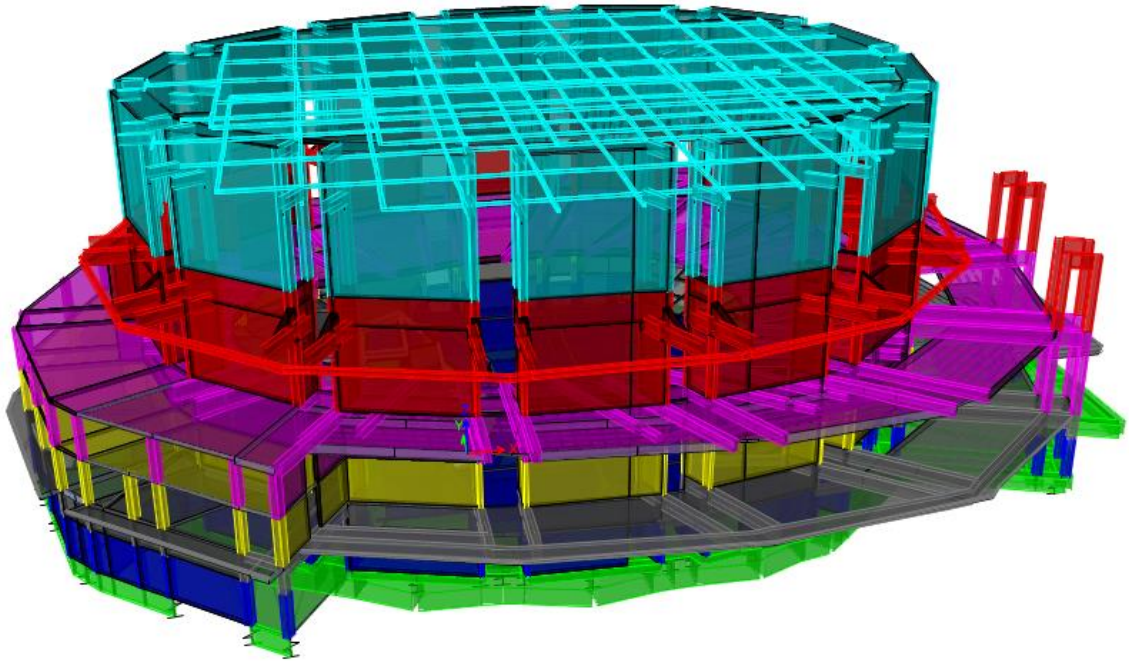

Figure C.1-1 : Snapshot taken of ETABS20 Analysis model, showing the full model

C.1.1 Purpose of MRSA Analysis

Response-spectrum analysis (RSA) is a linear-dynamic analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an elastic structure. Response-spectrum analysis provides insight into dynamic behaviour by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given level of damping.

Primary structural elements have been captured in the analysis model. The element demands output from this model are the drivers for the DSA strength and deformation evaluation. Secondary structural or non-structural elements which are included in the scope of assessment, but which are not directly modelled, use the deformation demands from the MRSA and force demands from NZS 1170.5:2004 Chapter 8, 'Parts and Components'. Refer to Section C.3.2 for the details on the scaling approach.

C.2 ANALYSIS MODEL

The computer analysis has been setup in ETABS 20 using a linear modal response analysis. Extracted results have either been used directly or have been post-processed using spreadsheet software.

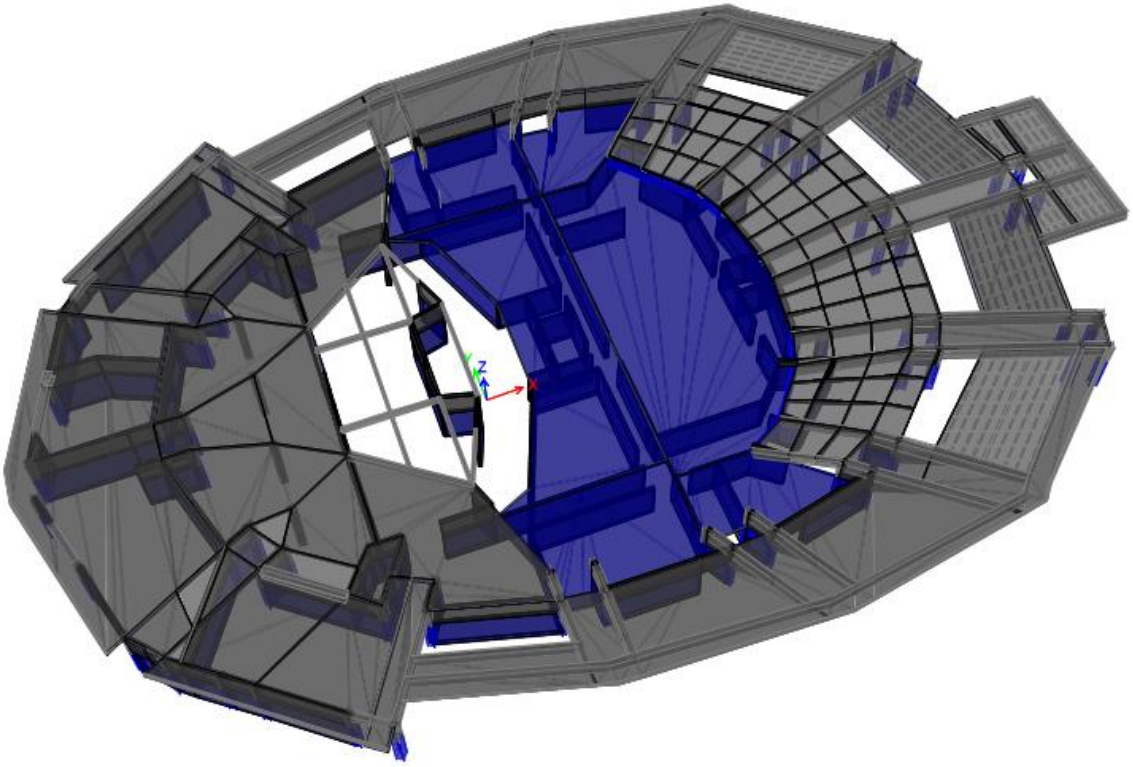

Figure C.2-1: Snapshot taken of ETABS20 analysis model, showing Level ½ and Level 2

C.2.1 General Description and Model Geometry

The analysis model has been constructed using shell elements that represent the wall elements and the floor elements, and with frame elements that represent the beams and columns in the building. The analysis model finds support in point springs that have been assigned the properties of the different pile configurations.

All elements in the model have been meshed to a maximum size of 1 meter.

The floor levels have generally been taken in accordance with the reference levels that are indicated on the original structural drawings, i.e., the ground floor level (Level 1) is at RL+2.300m, the ring beam at Level is at RL+6.620m, etc. Where there are minor floor height differences however, the floor levels are lumped at the same level, this occurs for example at the ground floor level, where the backstage area is at RL+1.900m.

The basement of the building has not been modelled.

The origin has been matched with the location on the structural drawings for convenience of setting out the coordinate system in the model.

Floors have not been modelled as rigid diaphragms due to the flexibility of the narrow diaphragm segments. Rather, floors have been modelled as panels elements.

The model has also explicitly modelled clear spaces to represent the gaps where the floors do not interact with each other This occurs for example in the Level 4 stepped seating floors inside the Auditorium, which comprises several independent segments of floors. The clear spaces have been amplified to >50mm to ensure they are visible in the model.

Columns in the backstage area and the stage area have slab-column joints. For simplicity in the model, they are modelled with pinned boundary conditions to neglect the nominal proportion of lateral load that is taken by them.

Specific modelling issues around the sloping precast concrete wall panels, sloping floor elements and sloping roof have been further described in Section C.2.6.

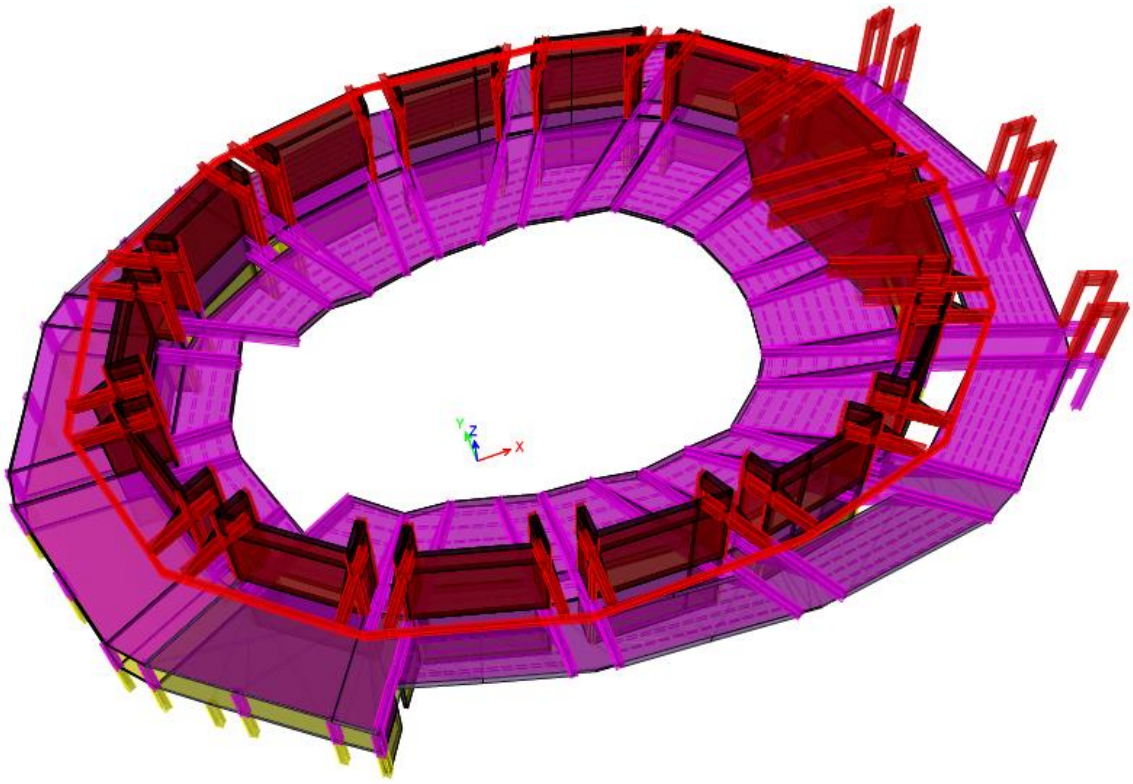

Figure C.2-2 : Snapshot taken of ETABS20 analysis model, showing Level 3 (yellow), Level 4 (purple) and Level 5 (red)

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C.2.2 Material Properties

Material properties used in the analysis and evaluation of the Auditorium are listed in Table C.2-1 below. Material strengths were based on the values in the project Construction Specification.

Table C.2-1 : Material properties used in the ETABS20 analysis model of the Auditorium

Material	Additional info	Specification	Probable strength [MPa]
Concrete	Foundations	20 MPa	30
	Walls below Level 2	35 MPa	52.5
	Level 2 slab (t=250mm)	35 MPa	52.5
	Precast flooring	35 MPa	52.5
	Other precast elements	25 MPa	40
	Other cast insitu elements	25 MPa	40
Reinforcement	Typical	Grade 275	324
	High strength	Grade 380	455
Structural steel	to BS4360:1972 (Grade 40 (A, B & C) *assumed as per Table C6B.2 of the Engineering Assessment Guidelines*		232
Weld material	as per Table C6.3 *of the Engineering Assessment Guidelines*		410
Bolts	as per Table C6.3 *of the Engineering Assessment Guidelines*		400

C.2.3 Element Modelling Parameters

This section describes the section properties and modifiers used in the analysis model. Modifiers have been determined in accordance with Table C6.5 of NZS3101:2006 and references within ASCE41-17.

Bay walls

The lower cast insitu portion of the bay walls (up to Level 5) have been modelled as a set of seven structural elements: the web area, two buttress flange walls and four buttress end columns. The web areas are modelled as 7.4m long shell elements, which is intended to capture length between the central axis of the buttress flange walls. The buttress flange walls themselves have a length of 1.7m between the centroids of the buttress end columns.

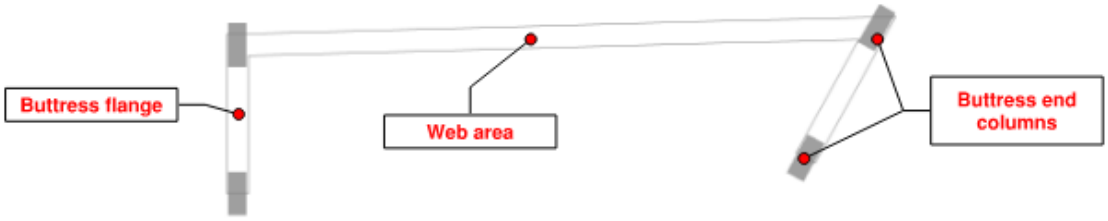

Figure C.2-3 : Illustration of modelling setup for the bay walls (cast insitu portion)

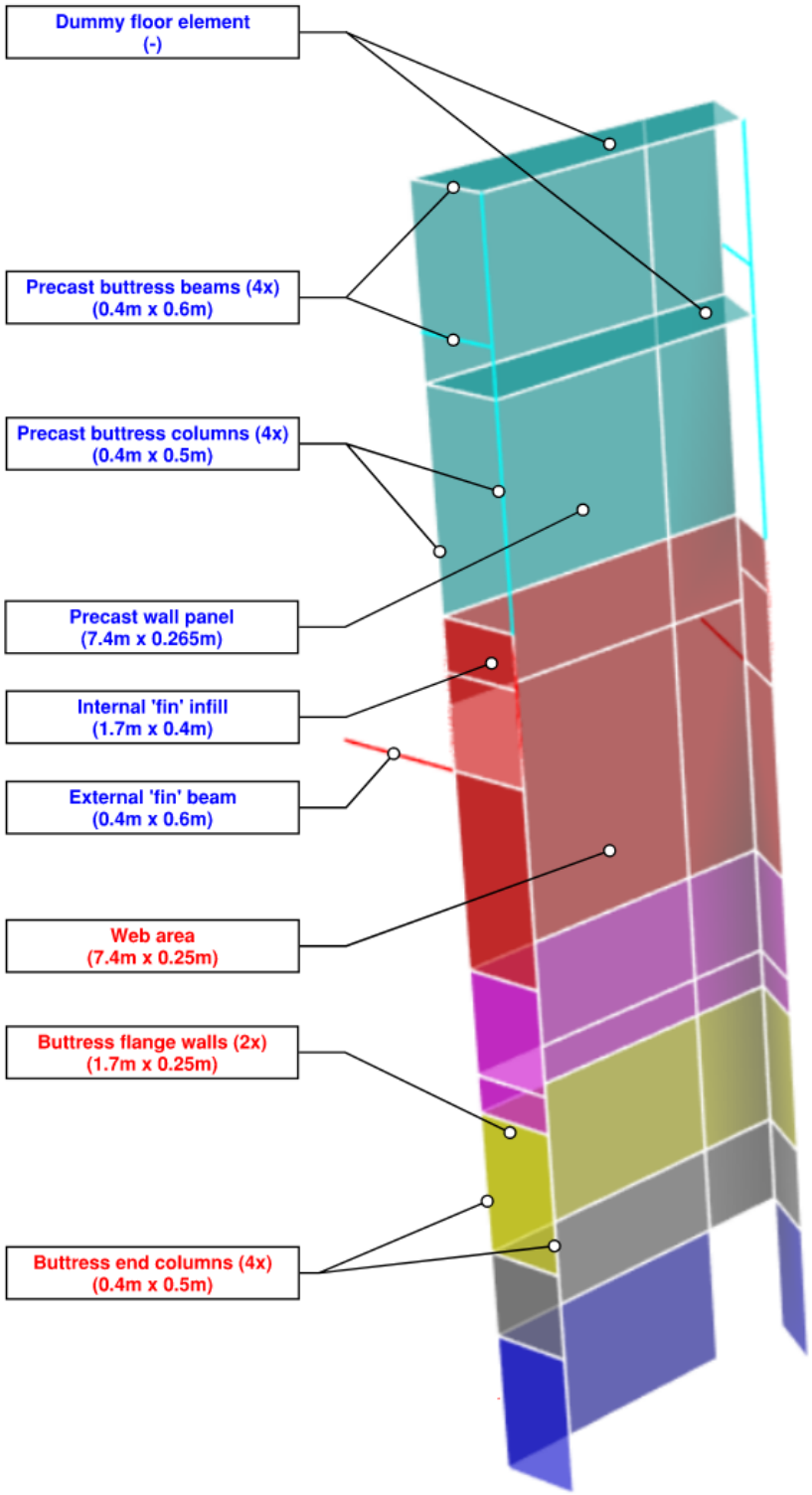

Figure C.2-4: Built-up of bay walls in ETABS20 model

The web area is modelled as connecting into the same node as the buttress end columns, representing a more idealised C-shape. They are actually (physically) slightly offset and connect into the buttress flange walls.

Where the buttress end columns have not constructed integrally with the buttress flange walls, for example where a precast cantilever beam element is connected into the bay walls, the columns have been modelled with reduced section sizes. I.e., 250x500mm instead of 400x500mm.

The upper precast concrete portion of the bay walls (above Level 5) is similarly divided in ten elements: a precast concrete wall panel that forms the web area, the four columns of the precast buttresses, three horizontal beams in the precast buttresses and the two dummy floor elements to capture the offset between the roof and the location of the cast insitu wall portion.

The precast wall panels have been modelled vertically, rather than their actual slope. The dummy floor elements are rigid elements that account for the in-plan offset of the roof relative to the web area of the lower cast insitu portion.

Table C.2-2: Section sizes and property modifiers for the bay walls as used in the ETABS20 analysis model

Material	Element	Thickness [mm]	Depth [mm]	Modifiers			
				f11 / f22	f12	m11 / m22 / m12	Mass
				(axial)	(shear)	(flexure)	
Cast insitu portion	Web area *below Level 2*	250		0.4	0.833 [1]	0.4	1
	Web area *above Level 2*				0.5 [2]		
	Buttress flange walls *below Level 2*	250		0.4	0.833 [1]	0.4	1
	Buttress flange walls *above Level 2*				0.5 [2]		
	Buttress end columns (cast insitu)	400	500	0.4	1	0.4	1
	Buttress end columns (precast)	250	500	0.4	1	0.4	1
Precast portion	Precast concrete wall panel	265		1	1 [3]	1	1
	Precast buttress columns	400	500	0.4	1	0.4	1
	Precast buttress beams	400	600	0.4	1	0.4	1

[1] *The shear stiffness of walls, floors and other deep elements have been determined by 5/6·A_g in accordance with Section C6.9.1.1 of NZS3101:2006 – Amendment 3.*
[2] *The cast insitu portion below Level 2 may see significant cracking in shear, therefore a modifier of 0.50·A_g is adopted in the assessment. This specifically refers to the supporting documentation of PEER/ATC72-1 (Table A-2, Case 1) for wall systems with backstay effects*
[3] *The upper precast portion of the bay walls are expected to remain effectively rigid, with the predominant source of shear degradation coming from their connections.*

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Floor/slab elements

The floor elements have been modelled as shell elements with their relative thickness as indicated on the original structural drawings. They have considered the stiffness modifiers as outlined in Table C.2-3.

The double-tee flooring and the stepped seating at Level 4 have been considered as ribbed elements to achieve a better approximation of their thickness along the primary axis.

Table C.2-3 : Section sizes and property modifiers for the floor systems as used in the ETABS20 analysis model

Description	Location	(Flange) thickness [mm]	Rib depth [mm]	Modifiers			
				f11 / f22	f12	m11 / m22 / m12	Mass
				(axial)	(shear)	(flexure)	
50mm cast insitu topping	Roof	50		1	0.833 [1]	1	1
Stepped seating cast insitu	Level 4	100	500	1	1	1	1
Double-tee (precast)	Level 2 and 4	150	500	1	1	1	1
Double-tee (thick flange) (precast)	Level 2	250	550	1	1	1	1
125mm cast insitu	Level 2 and 3	125		1	0.833 [1]	1	1
150mm cast insitu	Level 5	150		1	0.833 [1]	1	1
200mm cast insitu	Level 1/2 and 2	200		1	0.833 [1]	1	1
215mm cast insitu	Roof	215		1	0.833 [1]	1	1
225mm cast insitu	Level 2, 3 and 4	225		1	0.833 [1]	1	1
250mm cast insitu	Level 2	250		1	0.833 [1]	1	1
300mm cast insitu	Basement	300		1	0.833 [1]	1	1
400mm cast insitu	Level 2, 3 and 4	400		1	0.833 [1]	1	1
100mm S.O.G.	Level 1	100		1	0.833 [1]	1	1
120mm S.O.G.	Level 1	120		1	0.833 [1]	1	1

[1] *The shear stiffness of walls, floors and other deep elements have been determined by $5/6 \cdot A_g$ in accordance with Section C6.9.1.1 of NZS3101:2006 – Amendment 3.*

Squat wall elements

The squat wall elements have been modelled as shell elements with their relative thickness as indicated on the original structural drawings. They have considered the stiffness modifiers as outlined in Table C.2-4.

The squat walls at the ground floor level (Level 1) are considered to be cracked in the global analysis. A section analysis as shown that they crack at load levels of 20-35%ULS.

Table C.2-4 : Section sizes and property modifiers for the squat wall elements as used in the ETABS20 analysis model

Element	Location	Thickness [mm]	Modifiers			
			f11 / f22	f12	m11 / m22 / m12	Mass
			(axial)	(shear)	(flexure)	
Wall 1 Wall 6 Wall 7 Wall 8 Wall 10 Wall 17 Wall 19 Wall 20 Wall 22	Level 1	200	0.4	0.5 [1]	0.4	1
Wall 11 Wall 14 Wall 16 Wall 18	Level 1	250	0.4	0.5 [1]	0.4	1
Wall 2 Wall 5 Wall 6 Wall 7 Wall 11 Wall 12 Wall 13	Basement	300	0.4	0.5 [1]	0.4	1
Wall 21	Level 1 (and basement)	600	0.4	0.5 [1]	0.4	1

[1] *The cast insitu squat walls below Level 2 may see significant cracking in shear, therefore a modifier of 0.50·A_g is adopted in the assessment. This specifically refers to the supporting documentation of PEER/ATC72-1 (Table A-2, Case 1) for wall systems with backstay effects.*

Concrete masonry block walls

The majority of the concrete masonry block walls in the building have been included in the model. However, following preliminary assessment, it was concluded that these walls do not play a significant role in the take-out of lateral (seismic) load. At nominal levels of lateral loading the interface shear capacity between the Level 2 floor and the top of these walls is exceeded. Therefore, the walls have been modelled with a significantly reduced axial (f11 and f22) and shear stiffness (f12) to limit the amount of load taken by them.

However, since they do play an important role in the gravity load take-out of the building, they have not been omitted from the analysis model.

They have been further considered as secondary structural elements in Section G.4 of Appendix G. They have been assessed for displacement compatibility with the primary lateral load resisting system, and for their out-of-plane capacity.

Concrete frame elements – not part of the primary lateral load resisting system

The concrete columns and beam elements in the building that are not part of the primary lateral load resisting system of the building have been modelled with similar parameters as the 'Buttress End Columns' as listed in Table C.2-2.

The columns in the backstage area and the stage area have also been given end releases to neglect the amount of lateral load that is attracted to them.

Foundation beam elements

The analysis model comprises dummy foundation beams under primary squat wall elements, where the piles are not connected to them directly, for example in a door opening.

The large 1500x2000mm ring beam that is under the bay walls has been modelled without any stiffness modifiers, as preliminary assessment results show that the flexural and shear demands on this element were relatively low. Further sensitivity studies surrounding the flexibility of the foundation system have however considered modified properties. This is further described in Section C.3.4.

C.2.4 Foundation/Soil Interface Modelling

The Geotechnical Assessment Report (see Appendix B) advises that the base-shear take-out mechanism of the Auditorium comprises primarily the lateral capacity of the piles and the passive ground pressures that are developed on the sides of the foundation beams. Both these mechanisms have been included in the analysis model to capture the lateral and vertical stiffness of the foundation.

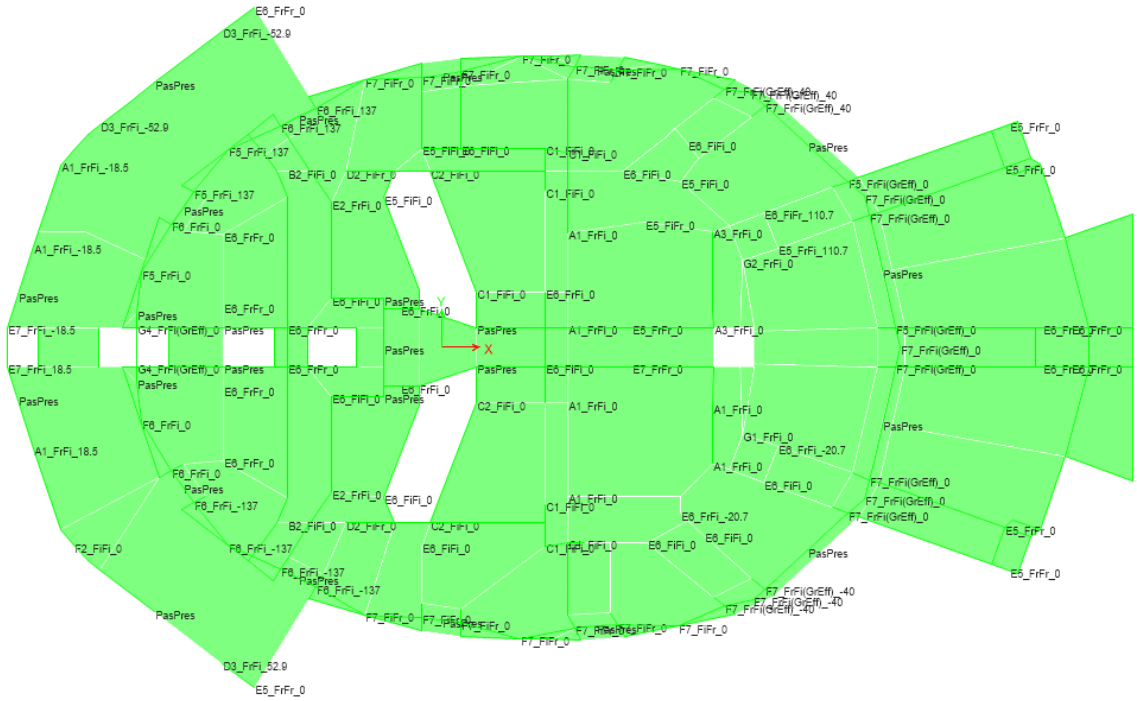

Figure C.2-5 : Snapshot taken of ETABS20 analysis model, showing the point spring assignments at the ground floor level (Level 1)

Piles

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There has been made distinct separation between the seven pile configurations that are listed on drawing S29 of the original 1980' structural drawings. These piles have varying pile base's reference levels, pile head's reference levels and varying reinforcement cage configurations which required them to be all considered individually. Especially, considering some piles are founded almost entirely in the liquefiable ground layers.

In collaboration with Tonkin+Taylor Ltd, a representative selection of upper and lower bound for the seven pile configurations has been determined, based on the probable flexural capacity, the pile head's reference level, the pile base's reference level as well as fixed-head and pinned-head boundary conditions.

The Geotechnical Assessment Report has then outlined the vertical stiffness of the piles as well as the results of the LPile analysis, which considers a push-over of the piles. This has been undertaken for undegraded ground conditions and liquefied ground conditions. The LPile analysis has also been extended to consider the effects of kinematic cyclic displacements and lateral spread on the piles, however these scenarios have not been included in the ETABS analysis model.

The resulting lateral and vertical pile stiffness have been included in the analysis model as non-linear links which specify the exact LPile output data. The non-linear links have then been assigned to point springs which are located at the matching location in the building. However, for the MRSA – which is an elastic analysis – the initial stiffness is used as the effective stiffness of the piles. This is taken between the first data points of the LPile analysis output.

Point springs have considered the LPile results corresponding to fixed-head boundary conditions if they match the in-plane direction of the wall elements or foundation beam that they are supporting. Several pile-wall joints have been evaluated to confirm that the associated pile overstrength demands can be developed in the wall. For the perpendicular direction, where the pile supports the out-of-plane direction of wall elements or foundation beams, the pinned-head boundary condition has been adopted. For piles that support wall elements that are rotated in plan, the point springs considers the same rotation to match the fixed-head and pinned-head boundary conditions accordingly.

Passive pressures against foundation beams

Passive pressures are acting on the sides of the foundation beams and are dependent on the total movement of the structure. The Geotechnical Assessment Report has outlined the amount of passive pressure that can be formed against each different configuration of ground beam. They have been prepared for undegraded and liquefied ground properties.

The total passive pressure resistance for the Auditorium is determined by a summation of the passive pressures acting on the foundation beams. This is done in both principal directions of the building, along the central axis and perpendicular to that axis. This generates a total push-over curve that has been included in the analysis model by 25 equal non-linear links that are evenly across the foundation beams in the model.

Similar to the piles, the non-linear links that represent the passive resistance have been assigned to point springs. The point springs have been added in the model at representative locations at the ground floor level. These locations are primarily around the ring beam, around the foundation beam at Wall 1 and along the location of the walls of the basement structure.

C.2.5 Diaphragm Modelling

Floor diaphragms are modelled as flexible (semi-rigid) concrete floor elements in the analysis with the property modifiers outlined in Section C.2.3. No rigid diaphragms have been assigned in the ETABS20 analysis model.

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The sloping transfer diaphragm below the Level 2 seating of the Auditorium (referred to as Level 1/2) is acting as a large transfer diaphragm and is considered to be the primary element that deals with the vertical irregularities in the primary structure.

C.2.6 Special Modelling Issues

Considering the complex geometry of the Auditorium, not all features have been modelled exactly as they are in the building. The modelling approach focusses on the primary load paths and has made the following assumptions to arrive at the best estimate of the actual structural behaviour.

Roof

The slope of the roof is not specifically captured in the analysis model. Instead, the roof is considered to be flat at the average roof height, at RL+28.032m. This is considered to give an appropriate representation of seismic mass and overturning moment.

The change in roof height was not found to have a significant effect on the results. Whilst the primary lateral load resisting system in the longitudinal direction of the Auditorium is expected to be taller, a relative reduction in period (thereby reduction of lateral load) was found to negate the relative increase in overturning demands.

The slope of the roof is modelled in more detail in the ANSR model, used to determine the floor accelerations at this level, see Appendix D.

Precast concrete wall panels

The sloping plane of the precast concrete wall panels in the bay walls is not specifically captured in the analysis model. The primary reason is that ETABS was found to not provide reliable output on the load transfer through sloping wall panels and that no pier label could be assigned to these walls. Instead, these walls are modelled vertically.

Given the shallow slope of the precast wall panels, this approximation is not considered to have a significant effect on the stiffness or primary load paths.

Dummy elements with a large stiffness and zero mass have been used to capture the offset that has therefore been created. These connect the roof slab elements and the roof trusses to the precast concrete wall panels.

The sloping transfer diaphragm below the Level 2 seating (Level 1/2)

The lower sloping transfer diaphragm at Level 1/2 that connects the bay walls at Bays 6(a) and 7(a) to the squat walls 18 and 21 is modelled as a series of smaller square shell elements to capture the best approximation of the actual sloped and curved floor.

The upper portion of the Level 2 seating floor at this location runs all the way towards Wall 16, and partially comprises precast bleacher seating elements. Due to the number of discontinuities and the reinforcing of this floor it is considered to be relatively flexible compared to the lower sloping diaphragm at Level 1/2. Therefore, this portion is not modelled. Instead, the weight of this floor is added as a superimposed dead load on the sloping transfer diaphragm at Level 1/2.

C.2.7 Building Masses, Weights, Mass Eccentricity and P-Delta

Table C.2-5 lists the structure's levels and total floor masses. The total building seismic mass is 8436 tonnes excluding the ground floor and foundations (2303 tonnes) and comprises modelled element self-mass and applied diaphragm mass. The mass of the sloping diaphragm is lumped to Level 2.

The seismic combination gravity loads are applied independently of the seismic weight, using element self-weight plus applied vertical loads (shell, beam, and node loads). The sum of building gravity weight has been validated through comparison to the NLTHA model build up. See Section D.2.3 for a more detailed comparison of the models.

Table C.2-5 : Building masses for ETABS model

Level	Element	Reference level	Storey height [m]	Floor area [m²]	Storey mass [t]
Roof		RL +28.032	23.844	1 188	1 138
Level 5		RL +18.550	16.250	134	1 313
Level 4	Backstage area	RL +12.060	9.760	311	1 922
	Stepped seating			674	
	Promenade			516	
Level 3	Backstage area	RL+ 9.340	7.040	320	926
Level 2	Backstage area	RL +6.620	3.320	380	2 110
	Stage area			318	
	Promenade			697	
	Sloping floor			281	
Level ½	Lower seating area	RL +4.975	2.675	591	1 214
Level 1	120mm S.O.G.	RL +2.300	0	1 598	2 667
	100mm S.O.G.			321	
	Foundations			n/a	

Due to the low drifts that the building experiences, global P-delta effects have not been investigated in detail.

Superimposed Dead and Live Loading

The superimposed dead load allowance is 0.5kPa for all floor and roof diaphragms.

The live load for the roof is 0.25 kPa, however, this considers a (earthquake) combination factor of $\psi_E = 0$.

The live loading zones are highlighted in Figure C.2-6. This figure is a plan of Level 2; however, the loading regions and intent are the similar for other floors at higher floor levels. Live load area reduction is only used in the "office loading" backstage portion of the Auditorium, a factor of 0.5 was used. The remaining areas (seating, stage, and promenade) did not receive a live load reduction in acknowledgement that these zones are either fully occupied or not occupied, the intent of a live load area reduction is not met. The slab-on-grade over the whole of the Auditorium footprint received a similar (3kPa · 0.5 (area reduction) =) 1.5kPa.

For calculation of seismic mass, the backstage and slab-on-grade use a (earthquake) combination factor of ψ_E of 0.3 and the public crowd loading areas a factor of ψ_E of 0.6.

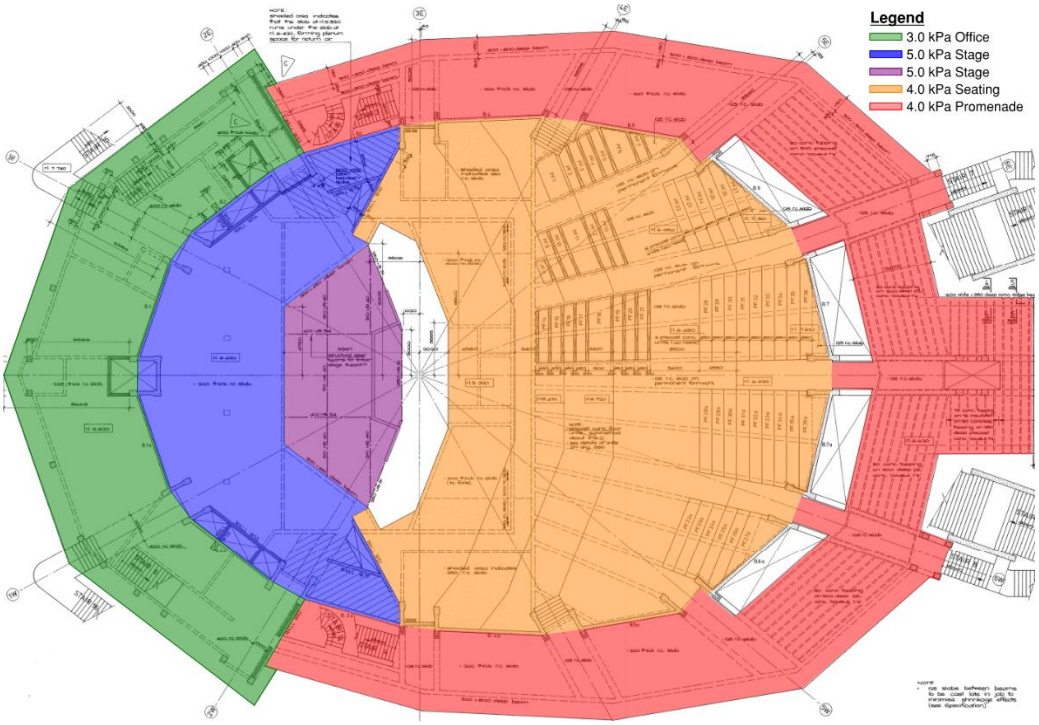

Figure C.2-6: Auditorium live loading areas

Mass eccentricity and torsion

Semi-rigid diaphragms have been assigned to the majority of the floor areas. This includes the roof diaphragm, the floors in the backstage area (at Level 2, Level 3 and Level 4), the Level 2 stage, the Level 2 seating area and the ground floor level (Level 1). These have been considered in the MRSA with a 10% accidental mass eccentricity.

The other floor elements, predominantly the promenade floors with double-tees and a topping reinforced with non-ductile mesh, have been considered to respond as flexible diaphragms. The requirements of NZS1170.5:2004 and the Engineering Assessment Guidelines do not consider an accidental mass eccentricity for flexible diaphragms.

The structure of the Auditorium is very stiff torsionally because it is a tubular structure. The structure was found not to be torsionally sensitive in accordance with Section 4.5.2.3 of NZS1170.5:2004. Therefore, no amplification was considered for the inelastic effects from torsion in accordance with Section C2.5.8 of the Engineering Assessment Guidelines

The elastic effects of torsion are expected to be captured by the MRSA load cases.

C.2.8 Modal Response

The analysis model considers Ritz vectors to determine the fundamental modes. Ritz vectors are efficient and widely used for dynamic analyses involving horizontal motions. Compared to Eigenvector analysis, Ritz vectors provide a better participation factor, which enables the analysis to run faster, with a similar level of accuracy.

It is noted that the modal response includes the (upper bound) flexibility of the foundations, which account for a large portion of the flexibility in the analysis model. However, excluding this from the analysis would likely yield conservative results.

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The fundamental modes built up to >95% mass participation in the fifth and sixth fundamental modes, which combines displacements in the three principal directions. The results are summarised in Table C.2-6. It is normally very unusual for a building of this form to need so few modes to achieve this level of mass participation – this is predominantly due to the rigid nature of the structure, relative to the flexibility of the foundations.

Table C.2-6 : Building fundamental periods and mass participation

Mode	Period [s]	Mass participation [%]					
		U_x	U_y	R_z	Sum U_x	Sum U_y	Sum R_z
1	0.607		0.613	0.226		0.613	0.226
2	0.539	0.851			0.850		
3	0.456		0.107	0.707		0.720	0.933
4	0.289		0.138			0.857	
5	0.280		0.102			**0.988**	**0.955**
6	0.275	0.139			**0.990**		

The first mode shows displacements in the shorter transverse (Z) direction of the Auditorium and responds at T_1 = 0.607s to ~61% mass participation in that direction. There is also a rotational component (R_z) to this mode at ~23% mass participation which forms the largest deformations at the southern portion of the Auditorium at Bays 6(a) and 7(a). Likely because there is relatively more mass concentrated there and because the structure is more flexible at that end as the web area of those bay walls terminate at Level 2.

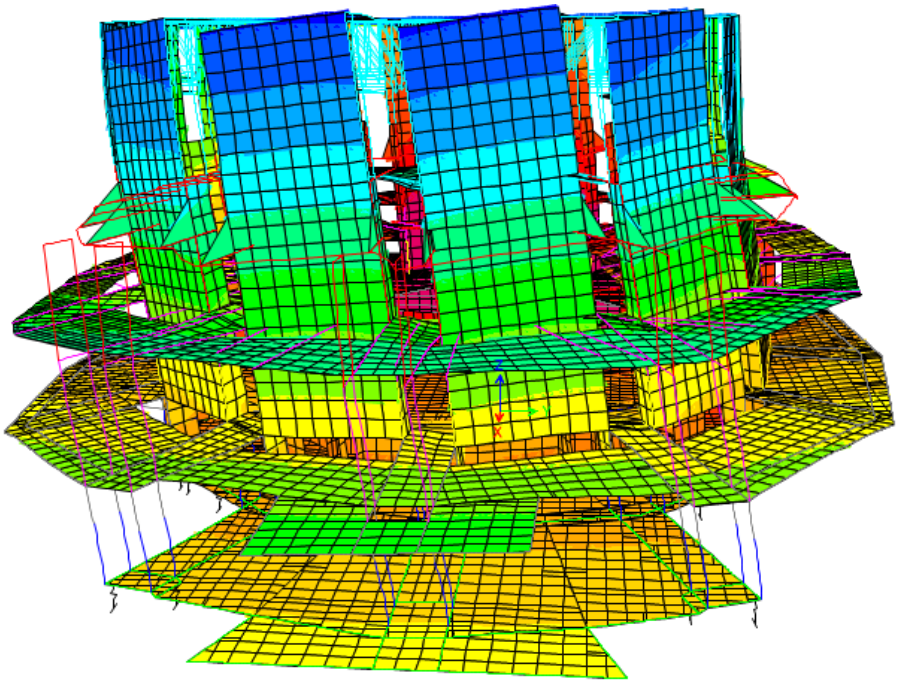

Figure C.2-7 : Snapshot of the ETABS20 analysis model - showing the building response at T_1 = 0.607s, looking from the Southern end

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The second fundamental mode of the building acts predominantly in the longitudinal direction (X) of the building responds at T_2 = 0.539s to ~85% mass participation in that direction.

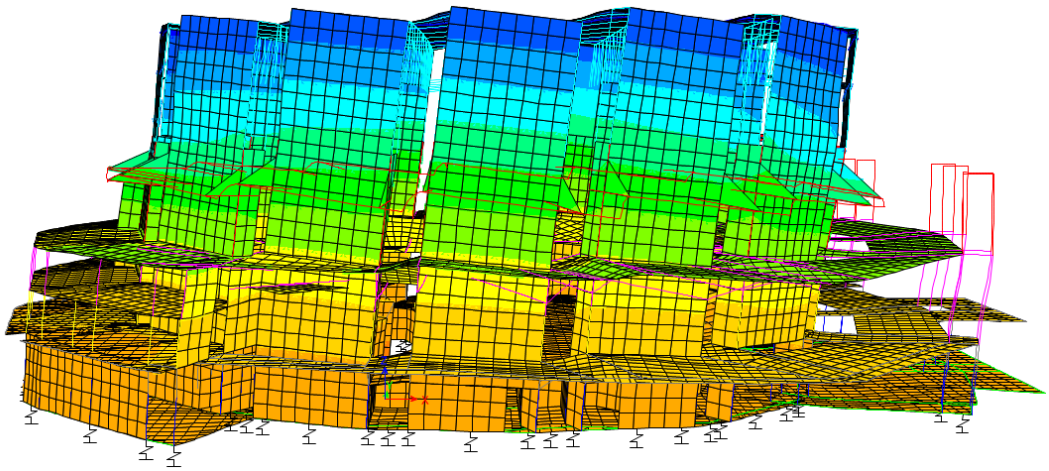

Figure C.2-8 : Snapshot of the ETABS20 analysis model - showing the building response at T_2 = 0.539s, looking from the Western end

The third fundamental mode is a torsional mode (Rz) of the building that responds to T_3 = 0.456s to another ~71% mass participation in that direction. This mode also has relatively large contribution of ~11% mass participation in the shorter transverse (Z) direction, which is predominantly concentrated at the Northern end of the building. Figure C.2-9 shows the inflated displaced shape over the wire-shadow of the building.

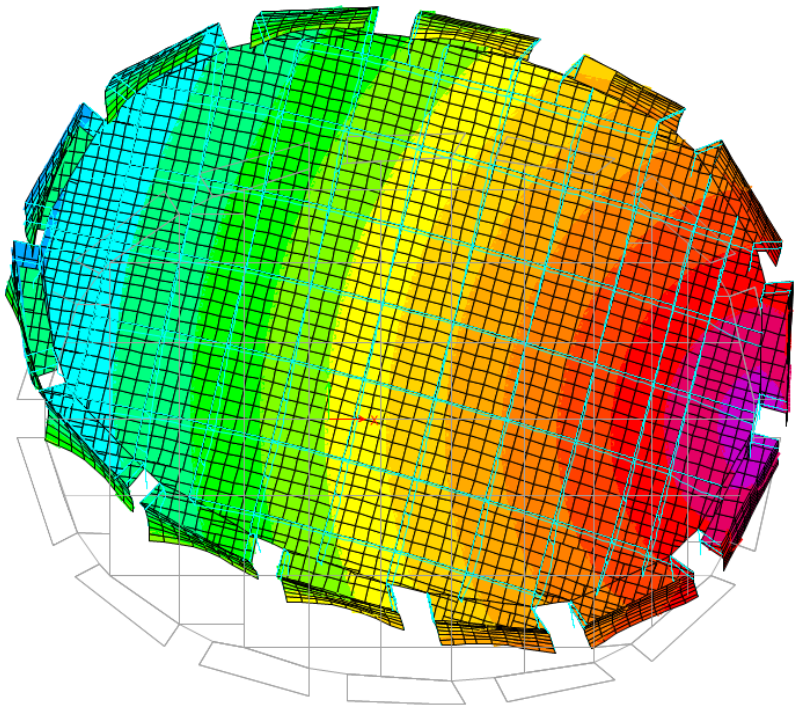

Figure C.2-9 : Snapshot of the ETABS20 analysis model - showing the building response at T_3 = 0.456s, looking onto the roof structure with the north on the left

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C.3 SEISMIC INPUT AND RESPONSE

C.3.1 Seismic Input Parameters and Response Spectra

The parameters in Table C.3-1 are used to generate the spectrum for MRSA. Modal combination using the CQC method is used.

The global system ductility is found to be μ_{sys} = 2.0 for the primary lateral load resisting system. This is based on the formation of inelastic mechanisms in the bay walls above Level 2. The primary lateral load resisting system on the ground floor level is considered to remain generally elastic as the plastic mechanisms form. The total strength hierarchy is further described in Section C.4.1. The associated ductility modification factor (k_μ) and the structural performance factor (S_p) have been determined in accordance with the ductility of the primary lateral load resisting systems.

For elements of nominal ductility (i.e. μ = 1.25), which do not have the capacity to resist the overstrength demands of the primary lateral load resisting systems, the ductility modification factor and structural performance factor have been adjusted accordingly.

Table C.3-1: Parameters used in determining the seismic loading

Parameter	Notation	Value	Commentary
Loading standard		NZS1170.5:2004	Excluding Amendment 1
Hazard factor	Z	0.4	Wellington
Return period factor	R_u	1.3	Importance Level 3
Site Subsoil Classification		D	
Near-fault factor	N(T,D)	1.0	T < 1.5s
Ductility modification factor	$k_{\mu,X}$	1.771	T_2 = 0.539s, μ = 2.0
	$k_{\mu,Z}$	1.867	T_1 = 0.607s, μ = 2.0
Structural performance factor	S_p	0.7	μ = 2.0

C.3.2 MRSA Shears Scaling

For assessment of primary lateral load resisting system of the Auditorium, the MRSA results are scaled to the base shear determined from the elastic site hazard spectra defined in NZS1170.5:2004. This considers a ductility of μ = 1.0 and a structural performance factor of S_p = 1.0. The total mass of the Auditorium is determined as 8623 tonnes.

In the longitudinal (X) direction of the Auditorium, the fundamental period (T_2 = 0.539s) results in a base shear coefficient (C_d) of 1.560g. This corresponds to a base shear of 131.9MN. The ETABS MRSA results in a base shear above the ground floor level of 124.9MN and therefore needs to be scaled by a factor of 1.056.

In the transverse (Z) direction of the Auditorium, the fundamental period (T_1 = 0.607s) is outside of the plateau of the elastic spectrum. However, since this first mode only accounts for 60% mass participation and the foundation flexibility is accounted for, the base shear coefficient is also raised to a coefficient (C_d) of 1.560g, corresponding to the same base shear as in the longitudinal direction. The ETABS MRSA results in a base shear above the ground floor level of 124.3MN and therefore needs to be scaled by a factor of 1.061.

The MRSA output gives a load distribution that is less triangular than an equivalent static analysis (ESA) method in accordance with NZS1170.5:2004. They are compared in Figure C.3-1 and Figure C.3-2, which also shows the floor accelerations that have been derived from the NLTHA, see Appendix D. The MRSA is

then, lastly, scaled to the global system ductility, the associated ductility modification factor and structural performance factor to derive the loads on the primary lateral load resisting system.

Global system ductility

The global system ductility is derived in Section C.4.1 for the primary lateral load resisting structure.

In general, the Auditorium structure is expected to respond in a nominal ductile manner (μ_{sys} <= 1.25) to seismic loading. The assessment identified the critical structural weaknesses to respond in a relatively brittle manner which are associated with the following elements:

- diaphragms with topping over precast flooring (reinforced with cold-drawn non-ductile mesh),
- sloping transfer diaphragm at Level 1/2 near Bays 6(a) and 7(a), and,
- upper precast concrete portion of the bay walls.

In isolation of those structural weaknesses (if the above-mentioned structural weaknesses are suppressed or perform better-than-expected), the global ductility capacity of the Auditorium structure is formed by the flexural mechanism in the bay walls above Level 2. Here, the deformation capacity is limited by bar buckling in the web area of the bay walls and this responds with a system ductility capacity of μ_{sys} = 2.0.

Other elements that potentially respond in a nominal ductile manner (i.e. walls in shear, etc.) have been assessed against the demands from flexural overstrength of the bay walls. Further description of this is presented in Section C.4.1.1.3.

Comparison of load distribution between different analysis methodologies

The total base shear of the Auditorium, including the mass from the ground floor level and the foundations, is 147MN in each principal direction.

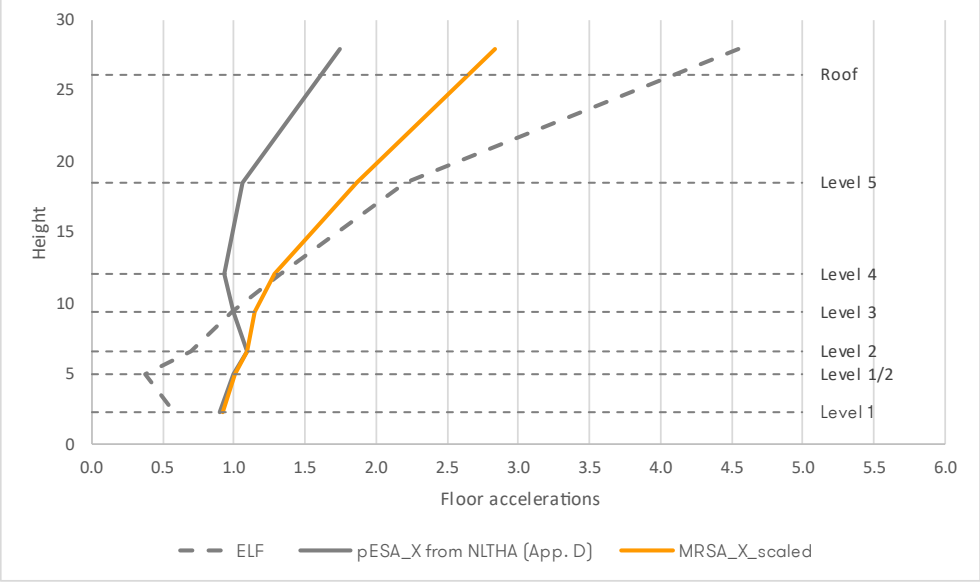

Figure C.3-1 : Comparison of floor accelerations in the longitudinal (X) direction of the Auditorium from different analysis methods

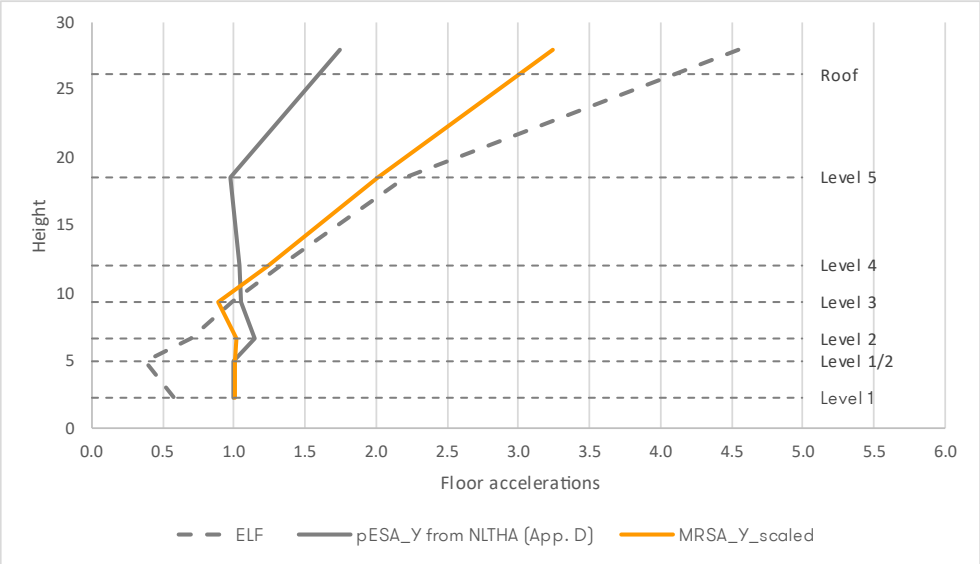

Figure C.3-2 : Comparison of floor accelerations in the shorter transverse (Y/Z) direction of the Auditorium from different analysis methods

Directional consideration and concurrency

For nominally ductile structures the provisions of Section 5.3.1.2 of NZS1170.5:2004 prescribe a set of actions comprising 100% of the specified earthquake action in one direction plus 30% of the specified earthquake action in an orthogonal direction. However, due to the shape of the structure this could potentially not capture all the most unfavourable cases for all the wall elements.

Therefore, the MRSA load cases in the analysis model have been set up in several angles in plan of the structure, namely at 30, 45 and 60 degrees. This is done to create an envelope of the most unfavourable cases for each of the wall elements that are not in an orthogonal direction of the building.

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C.3.3 Building Displacements and Drifts

Displacements of the Auditorium are considered in accordance with Chapter 7 of NZS1170.5:2004. This includes considerations of multiplication of the deflections by a scale factor equal to the global system ductility (μ = 2.0) and, subsequently, the multiplication of the drifts by a drift modification factor (k_{dm}). The results are shown in Table C.3-2 and Table C.3-3.

The drift modification factor (k_{dm}) is determined based on the height of the building in the analysis model, where the roof is modelled at RL+28.032m and the base is modelled at RL+2.300m. This concludes a factor of k_{dm} = 1.41.

The Auditorium responds with probable inelastic mechanisms forming above the Level 2 floor, this is further described in Section C.4.1. As such, the global drift profile is determined on that basis. The primary lateral load resisting system at the ground floor level (Level 1) is expected to remain elastic at the formation of the inelastic mechanisms at Level 2.

The strength hierarchy outlined in Section C.4.1.1.3 identified that there are two other potential mechanisms that could form within the overstrength of the inelastic mechanism at Level 2. However, the associated drift profiles to these mechanisms are expected to be off relatively similar magnitude and shape.

The drifts in the longitudinal direction (X) of the Auditorium have been taken as the maximum displacements of any node in the analysis model associated with that floor level.

In the shorter transverse (Z) direction there is considerably less building drift in the northern part of the building near Bays 1(a) and 2(a). This may be caused by the presence of the cast insitu diaphragms in the backstage area at Level 2, Level 3 and Level 4, as these can distribute the loading evenly from Bays 1(a) across to Bays 2(a) and 3(a).

The southern portion near Bays 6(a) and 7(a) experiences a lot more drift from the eccentric mass in the lobby area, the flexibility between Level 1 and Level 2 and the relatively more flexible foundation system in the Southern end of the Auditorium. Also, the diaphragm is much less able to engage the stiffness of the nearby Bays 5(a) and onwards.

As such, a drift profile is given for both ends of the Auditorium in the transverse direction, taken as the maximum displacements of the nodes at the bay walls of Bays 1(a) and 7(a).

The drifts at the southernmost extent of the Auditorium, at the frame extensions of Bays 7(a) in the lobby area, can be even higher. Between the ground floor level (Level 1) and the promenade floors at Level 2 and Level 4 they can reach up to drifts of 1.27% at Level 2 (at 100%NBS_{IL3}).

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Table C.3-2: Overview of displacements and drifts in the Longitudinal (X) direction

Level	Height	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]
		35%ULS at yield		*100%NBS$_{IL3}$*	
Roof	RL +28.032m	65	0.29%	203	**0.66%**
Level 5	RL +18.550m	45	0.26%	141	0.55%
Level 4	RL +12.060m	34	0.21%	105	0.60%
Level 3	RL +9.340m	30	0.17%	89	0.39%
Level 2	RL +6.620m	26	0.14%	78	0.40%
Level 1/2	RL +4.975m	25	0.12%	71	0.33%
Level 1	RL +2.300m	22	-	62	-

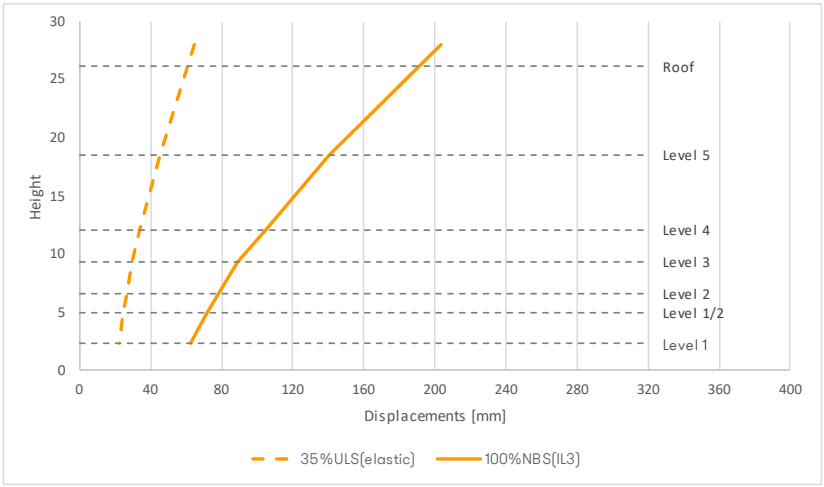

Figure C.3-3: Displacement Profile

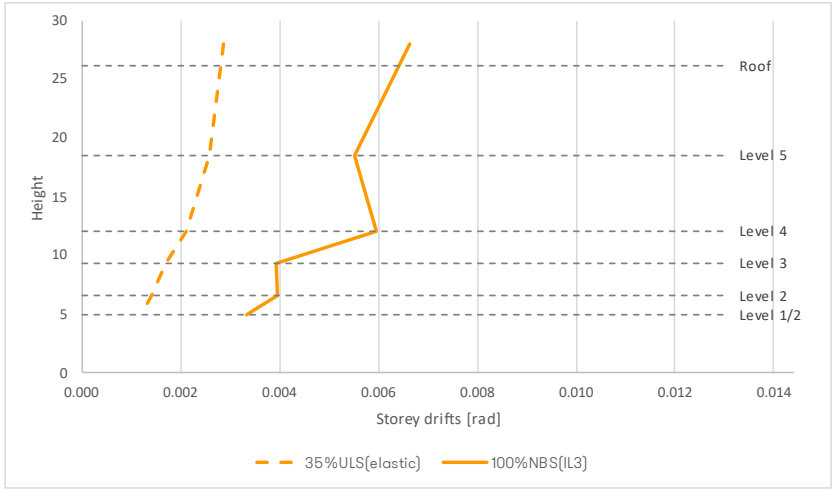

Figure C.3-4: Drifts

Table C.3-3: Overview of displacements and drifts in the transverse (Z) direction

Level	Height	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]
		Bay 1(a)				Bay 7(a)			
		35%ULS at yield		*100%NBS$_{IL3}$*		*35%ULS at yield*		*100%NBS$_{IL3}$*	
Roof	RL +28.032m	56	0.30%	168	0.65%	112	0.43%	342	0.93%
Level 5	RL +18.550m	35	0.30%	107	0.66%	84	0.38%	254	0.84%
Level 4	RL +12.060m	22	0.21%	64	0.44%	66	0.34%	200	0.73%
Level 3	RL +9.340m	18	0.15%	52	0.33%	60	0.27%	180	0.61%
Level 2	RL +6.620m	15	0.08%	43	0.22%	54	0.38%	163	1.04%
Level 1/2	RL +4.975m	14	0.06%	39	0.17%	50	0.45%	146	**1.24%**
Level 1	RL +2.300m	13	-	35	-	41	-	113	-

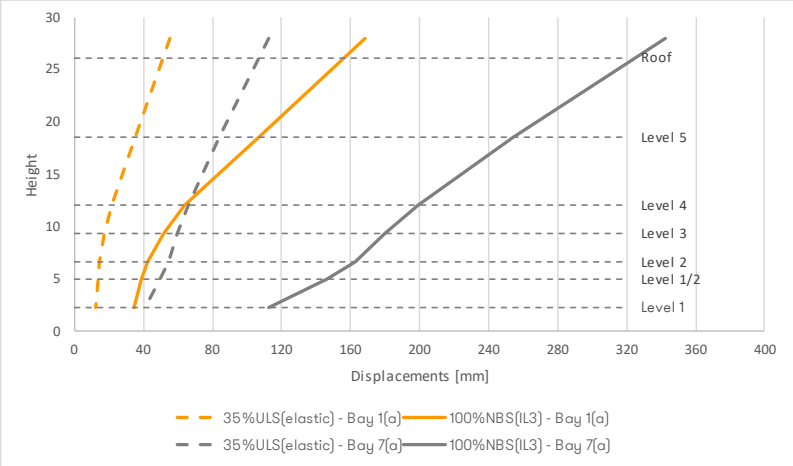

Figure C.3-5: Displacement Profile

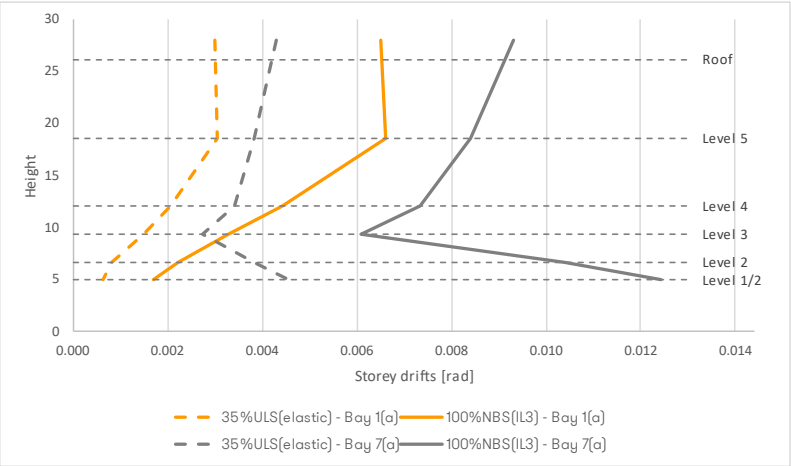

Figure C.3-6: Drifts

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C.3.4 Sensitivity Studies

The preliminary analysis results showed that the primary load paths are clearly defined, but that they are leading to high forces, stresses, and strains in localised areas in the linear elastic analysis model. This occurs at the roof diaphragm and the sloping transfer diaphragm below the Level 2 seating (referred to as Level 1/2). Furthermore, the influence of the foundation stiffness on these elements is specifically investigated.

Roof diaphragm stiffness

The primary element that provides tie capacities in the diaphragm is the reinforcement. However, since this comprises cold-drawn (non-ductile) mesh, this load path is unable to be maintained. In a series sensitivity study, all secondary load paths have been investigated to see how they influence the load paths if elements yield and soften as a consequence. This was undertaken to determine whether the structure (globally) can tolerate more inelastic behaviour and redistribute loads to secondary load paths.

There are secondary load paths that can provide tie capacities through the roof trusses, the chord stiffeners and the saddle bars over the DyCore floor units. However, these are all providing only limited contribution to the overall strength. The load path from the topping to the roof trusses goes through small cleats that will yield and deform at very low levels of lateral loading, and the roof trusses are also not connected to the bay walls. Then, there is no direct connection between the topping and the chord stiffeners, and the bolted connections of the chord stiffeners have minimal shear capacity. And lastly, the saddle bars are only located at 1.2m spacings (one saddle bar per DyCore unit width) and may potentially suffer from localised delamination between the topping and the DyCore units.

Another secondary load path is the 215mm cast insitu concrete slab that surrounds the DyCore units around the non-orthogonal edges of the roof diaphragm. However, this area is minimally reinforced – only at the bottom face, likely for gravity loads. In the top face, there is only the cold-drawn (non-ductile) mesh that continues towards the bay walls. So, it could very well be the case that this portion of flooring will also see significant cracking at relative low levels of lateral loading.

Following the preliminary results, the diaphragm was softened. As a first pass, the 50mm concrete topping was modelled with a softened in-plane stiffness modifiers f11 and f22 at 0.40 and with the shear stiffness modifier f12 at 0.50, to simulate widely distributed cracking across the diaphragm. This resulted in a series of pier shear forces in the bay walls that were exported to a Microstran micro strut-and-tie model. The micro strut-and-tie model was set up to be a free-body diagram intended to be assessed as a strut-and-tie model to Section C5.6 and Appendix C5D of the Engineering Assessment Guidelines. However, this model still showed high tie forces in the diaphragm.

To account for this potential loss of rigidity in the diaphragm the sensitivities study considered for the 50mm concrete topping and the 215mm cast insitu slab to consider less and less in-plane stiffness. In stiffness modifiers f11 and f22 in the ETABS20 model were reduced to limit any tie force taken by the reinforcement. The shear stiffness (f12) was kept at a limit of 0.50, to allow for the diaphragm to keep its ability to form diagonal compression struts. Furthermore, in parallel to the softening of the slab elements, the axial stiffness of the chord stiffeners was also reduced to consider slip in the bolted connections as well as yield from ply bearing. The stiffness of the roof trusses was kept the same, except at their extents where they connect with the bay walls – as their connection does not allow for this force to transfer over.

At the final runs, the diaphragm was effectively behaving as a flexible diaphragm, distributing load on a tributary width basis to all the lateral load resisting systems. Even to the precast buttresses which are relatively flexible.

The sensitivity study showed that with decreasing in-plane stiffness of the concrete elements and chord stiffeners there became a larger reliance on the bay walls to provide a macro tie to hold the diaphragm

together. Particularly in the shorter transverse (Z) direction of the Auditorium, the diaphragm wanted to open up at one end and close at the other end. In the longitudinal (X) direction of the Auditorium, the effect was similar, however smaller, as the roof trusses restrained the opening and closing effect. The study also showed that there was relatively a lot more lateral load transferring to the buttresses of the bay walls.

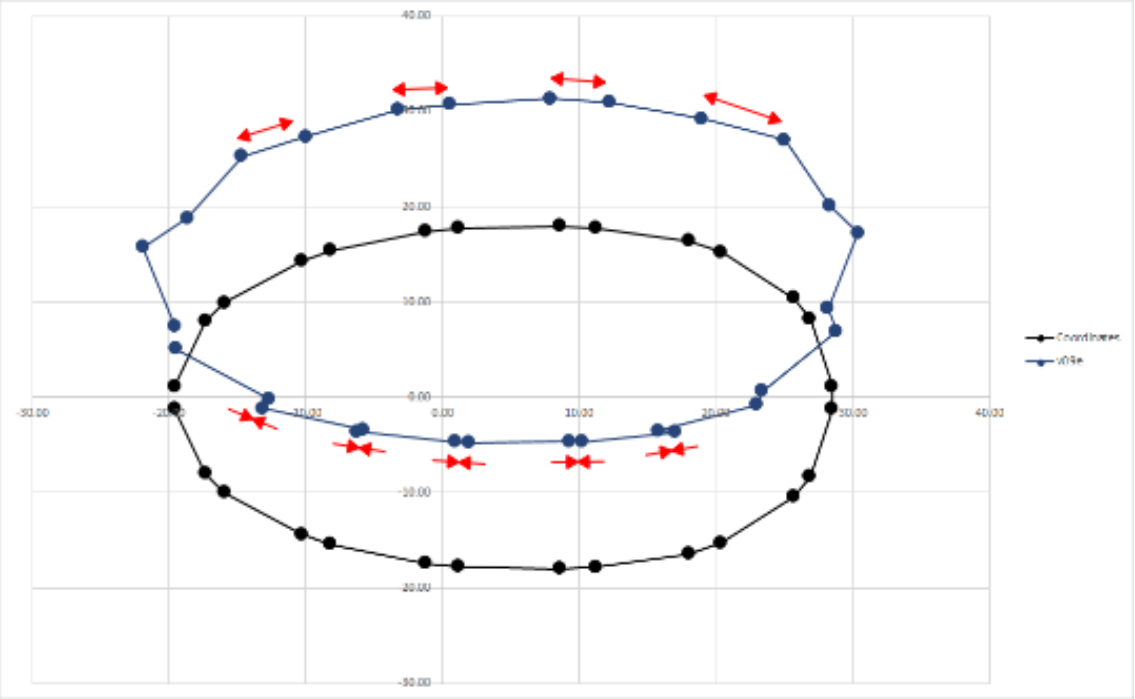

Figure C.3-7: Plot of the displacements of the nodes at the edge of the roof - showing the 'opening' of the diaphragm at the upper end and 'closing' at the lower end

Ultimately, this approach appears to predict that if the nominal capacity of the non-ductile mesh is exceeded and the diaphragm becomes more flexible, there is an alternative load path through the bay walls and the roof trusses. The critical parameter however remained to what extend this diaphragm could be allowed to open, and at which point the loss of integrity of the diaphragm becomes a significant life-safety hazard.

This approach was adopted as the means of assessing the roof diaphragm and is further discussed in Section C.4.2.3.

Stiffness of sloping transfer diaphragm (Level 1/2)

Similar to the roof, the transfer diaphragm at floor Level 2 showed a lot of reliance on the primary load paths down the sloping diaphragm at Level 1/2 to Walls 16, 18 and 21. This transfer diaphragm needed to account for the lateral loading in Bays 6(a) and 7(a), which have their web areas terminated and are very flexible down to Level 1. However, the transfer diaphragm also picks up the lateral loading from the floor extending southwards, in the extended frames of Bays 6(a) and 7(a) towards the Renouf Foyer.

It was investigated in this sensitivity study that when the transfer diaphragm cracks, yields and softens, whether the majority of the lateral loading could find an alternative load path. For example, through compression struts in the promenade flooring and ties through the beams of the extended frames at Bays 6(a) and 7(a). Thereby, reducing the total amount of lateral load that traverses down the sloping diaphragm and increasing the loads that would end up in Bays 4(a) and 5(a) for example.

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For this sensitivity study, the in-plane stiffness modifiers (f11 and f22) as well as the shear stiffness modifier (f12) were considered at different levels of reductions. This was considered for the sloping diaphragm at Level 1/2.

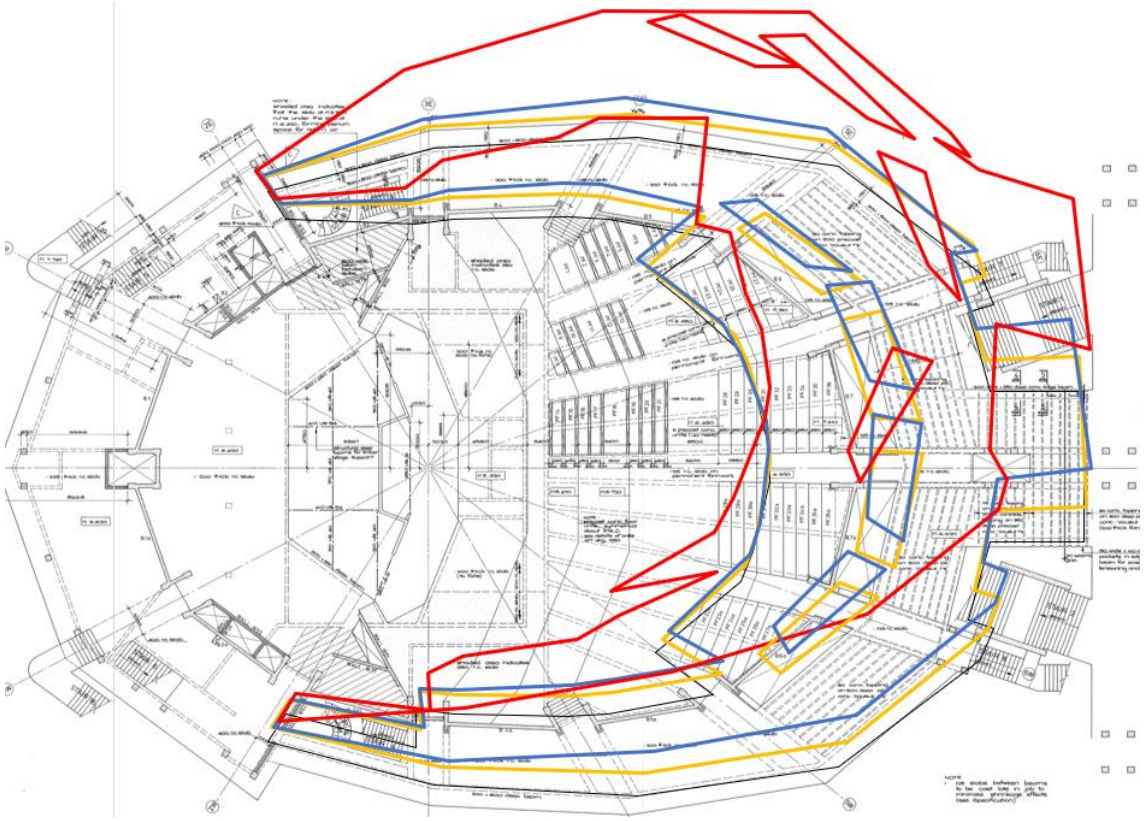

Figure C.3-8: Plot showing the displacement shape at different iterations of the sensitivity study to reduce the stiffness modifiers of the floor diaphragms

The outcome of the study showed very similar behaviour throughout the different iterations and the load distribution did not significantly change at reduced stiffness modifiers. Only when the diaphragm was significantly softened by the modifiers (<0.001) did the model show a notable reduction in the lateral loads in Walls 16, 17 and 18 and an increase in the loading at Bays 4(a) and 5(a). However, this scenario was paired with significantly large displacements at the outer edges of the diaphragm at the extended frames of Bays 6(a) and 7(a). These displacements were so large that they could lead to seating loss of the double-tee flooring in the promenade at Level 2, leading to a significant life-safety hazard, as well as a loss of structural integrity.

In the longitudinal (X) direction of the building, there was also no significant change of behaviour, other than an increased level of displacement.

It is concluded from this sensitivity study that the structure is heavily reliant on the primary load path through the sloping transfer diaphragm at Level 1/2. And that the loss of this load path would lead to significantly larger displacements within the structure.

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Foundation and ground floor diaphragm flexibility

In the preliminary assessment, the Auditorium was considered to have a fixed base. However, early discussions with Tonkin+Taylor Ltd suggested that the building would likely have a higher stiffness than the ground, especially in liquefied ground conditions.

A fixed-based model suggested that the fundamental periods of the building were clearly lower than the corner period of the elastic site spectra for subsoil class D (T_C = 0.56s), and so the building would see the 'plateau level' of accelerations, up to 140MN. However, the combined base-shear take-out capacity of the piles and the passive pressures against the foundation beams is only 50-60MN, depending on the direction of the lateral loading that is considered. Furthermore, that base-shear capacity is only achieved at 50-100mm of displacements as passive pressures around the piles and foundation beams require that movement to build up their resistance.

There was also a disconnect between the lateral load transferred to certain squat walls at Level 1, which were only reinforced with a small number of piles. On the other hand, the ring beam foundation under the bay walls has a relatively large number of Ø560mm 'Franki' piles, which would see minimal lateral load in the out-of-plane direction. At Wall 21 this behaviour was particularly obvious. This wall is founded on two Ø900mm diameter bored piles which can tolerate lateral loading in the range of 1,000-1,500kN per pile and a single Ø450mm 'Franki' pile with relatively negligible capacity. The shear capacity of this wall however was found to be several magnitudes larger, up to 19,000kN. So, for this wall to the be fully loaded in its capacity required significant redistribution through the ground floor level (Level 1) diaphragm.

These considerations are adopted in the assessment of the ground floor level (Level 1) diaphragm and the foundation system, which are further described in Section C.4.4. However, they also could potentially play a significant role in the primary load paths through the structure.

This sensitivity study considered the undegraded and liquefied foundation base-shear take-out capacity combined with the upper and lower bound vertical capacities – a total of four scenarios. These were combined with the results and conclusions from the previous sensitivity study for transfer diaphragms, including those at the ground floor level (Level 1). It was tested whether the structure would be capable of finding alternative load paths, for example through the promenade floor.

The conclusions were similar as for the sensitivity study of the transfer diaphragms. However, throughout the different iterations the load distribution did not significantly change at reduced stiffness modifiers. Only at the extreme case, when the diaphragm(s) considered property modifiers that significantly lowered their stiffness (<0.001) did the analysis model show any notable change in load distribution. And, again, in that scenario the displacements were considerably large that they would likely lead to the loss of integrity of other elements in the Auditorium, such as the seating of the double-tee flooring.

It is concluded from this sensitivity study that the structure is heavily reliant on the primary load paths through the sloping diaphragm at Level 1/2, down through Walls 16, 18 and 21 and back through the ground floor level (Level 1) diaphragm to the pile foundations around the structure.

C.4 EVALUATION

This section describes the elements of the primary structure of the Auditorium that were evaluated as part of the detailed seismic assessment. It describes the primary load paths through the structure, the probable hierarchy within the load paths, the potential structural weaknesses, the method of assessment, the lateral loading used in the assessment and, ultimately, the seismic score of each of the potential structural weaknesses.

C.4.1 Primary Lateral Load Resisting System

The primary lateral load resisting system of the Auditorium is considered to be through the bay walls around the egg-shaped outer ring of the structure. The bay walls are ten continuous cantilever wall systems (Bays 1(a) till 5(a)) as well as four wall systems that are a frame system between Level 1 and Level 2 and are a wall system upwards from there (Bays 6(a) and 7(a). They collect the lateral loading at the roof level and each of the floor levels and take it out to the foundations. The bay walls are supported in their lateral load take-out capacity by a series of long squat walls at Level 1.

The bay walls and the squat walls at Level 1 are tied by concrete diaphragms at Level 1/2 and 2, as well as on the ground floor level. They are supported by a series of foundation beams and pile foundations, that provide vertical support as well as base-shear take-out capacity. The ring beam that supports the bay walls is a significant part of this foundation system.

C.4.1.1 Bay walls

The bay walls consist of a cast insitu portion up to the roof level of the promenade (above Level 4) and a precast portion up to the Auditorium roof level.

The strength hierarchy of the bay walls is presented in Section C.4.1.1.3.

The frame extensions of the bay walls at Bays 6(a) and 7(a) are not considered to be part of the primary lateral load resisting system and are considered in Section C.4.3.3.

C.4.1.1.1 Upper precast portion

The precast portion of the bay walls is described in detail in Section 5.1.1.1 of the main body of this report. The precast concrete wall panels and the internal columns of the precast buttresses collect the lateral loading of the roof diaphragm through starter bars that have been cast into the elements of the roof structure.

From there, it is considered that the shear flows through the precast concrete wall panels as compression struts between a top corner and the bottom corner on the other end. The in-plane flexural behaviour of the precast concrete panels is not considered to be a governing structural weakness.

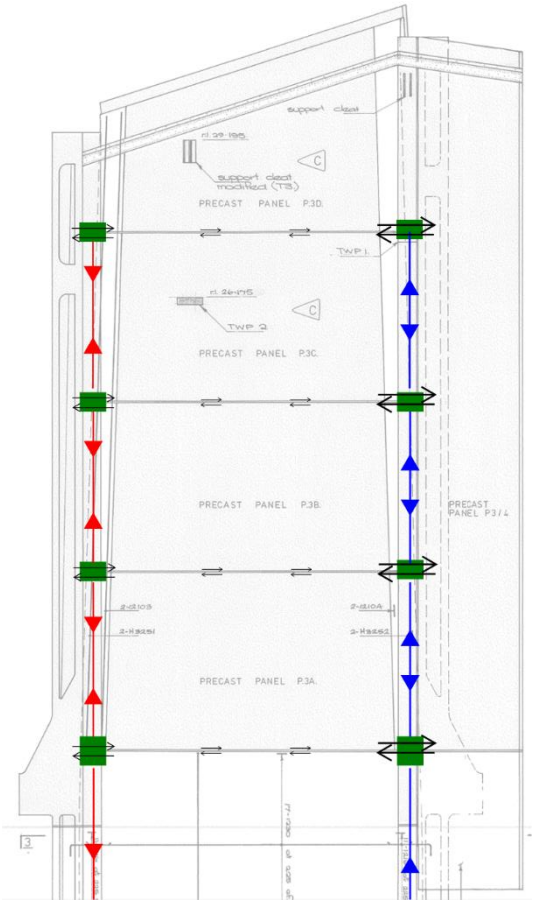

Figure C.4-1: Elevation of the precast portion of the bay wall at Bay 3(a), showing the expected load paths through the system

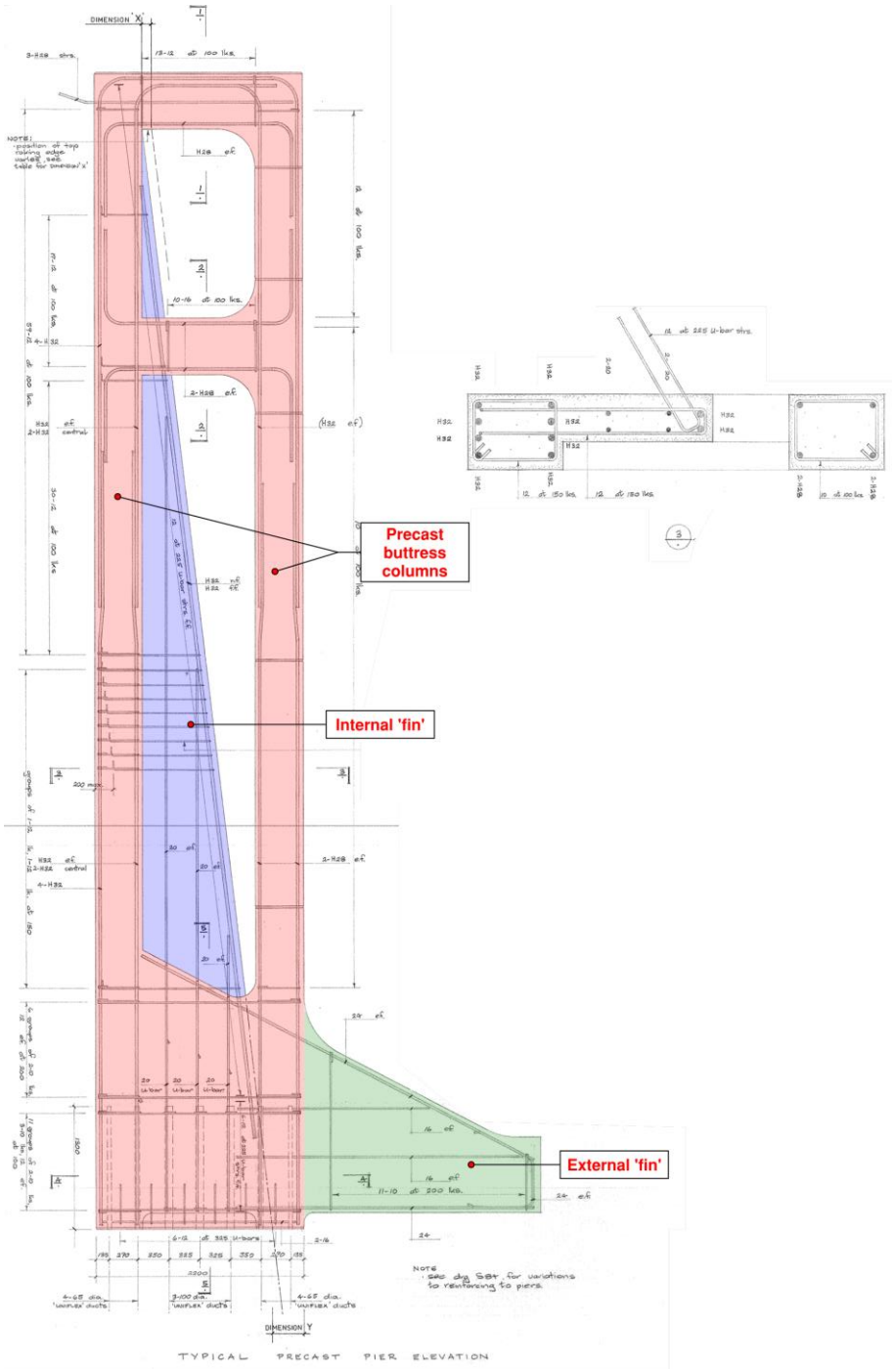

Figure C.4-2: Typical precast buttress

Since, the precast concrete wall panels are not connected integrally to each other, the shear between precast concrete wall panels is considered to be taken out by two mechanisms. Firstly, there are two weld plates that are connected into the edge faces of the precast concrete wall panels. They are located at thirds of the panel widths and their load paths are shown by the smaller arrows on Figure C.4-1. Secondly, there is a stitch joint with the precast buttresses. There will be conventional shear in the wall section here.

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At the bottom precast concrete panel, the stitch joint is not continuous into the cast insitu portion of the wall. Here, the shear connection is required to go through the precast buttress – which only has a thickness of 400mm – and the two weld plate connections.

These locations are considered to be the weakest link of the Bay Walls.

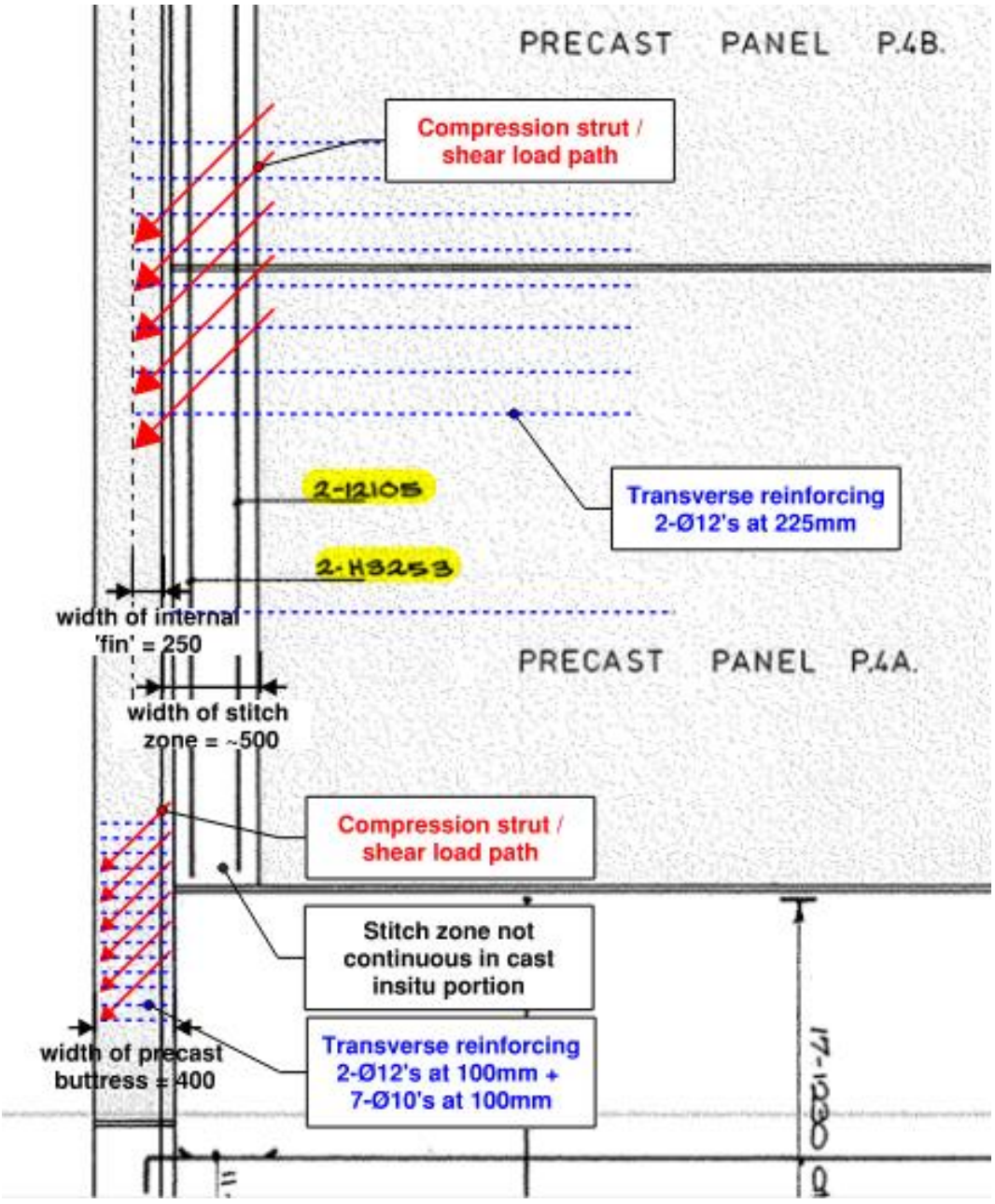

Figure C.4-3: Extract from drawing S35B of the original 1980's structural drawing, highlighting the shear load path through the stitch zone and the precast buttress

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The shear flow is then finally considered to result in a global overturning of the bay wall, forming in a push-pull mechanism between the two precast buttresses. These are connected to the cast insitu portion below them through 'Uniflex' ducts.

In the out-of-plane direction, the precast concrete wall panels would see flexural and demands at a different mode to the in-plane direction. For this situation the precast walls are considered to be subject to accelerations from Chapter 8 of NZS1170.5 for 'Parts and Components'.

The assessment has considered the following elements to be potential structural weaknesses of the precast portion of the bay walls:

- The interface shear connection between the roof and the precast concrete wall panel and inner columns of the precast buttresses.
- The shear flow through the precast concrete wall panels.
- The shear flow between different precast concrete wall panels.
- Plastic rotation limits at the interface area between precast buttresses and the cast insitu buttress flange walls below them (at the 'Uniflex' ducts).
- Out-of-plane shear and flexural capacity of the precast concrete wall panels.

Exceedance of the capacity of each of these potential structural weaknesses could lead to loss of integrity of the mass of the roof diaphragm, potentially resulting in large differential displacements which may not be tolerable by the roof structure.

Interface shear at connection to roof

The 50mm concrete topping of the roof diaphragm is connected to the precast concrete wall panels via a 215mm thick cast insitu section of slab between the last Dycore unit and the precast concrete wall panel. Ø12mm U-bars start from the precast concrete wall panel at 250mm spacing to join these elements.

At Bays 1(a) and 7(a) there is no 215mm concrete slab. Here, the DyCore units are seated directly onto a steel angle which is bolted into the precast concrete wall panels. Therefore, the connection is formed by Ø12mm starters, at 125mm spacing, directly into the 50mm concrete topping.

This connection is assessed as an interface shear check, in accordance with Section C5.5.5.3 of the Engineering Assessment Guidelines that refers to Section 7.7 of NZS3101:2006. This accounts for a strength reduction factor of ϕ=0.85 and a friction coefficient of μ_f = 1.00 to be considered in the capacity. There is no axial load considered to be acting on this interface and so the capacity is only generated by the reinforcement crossing the joint.

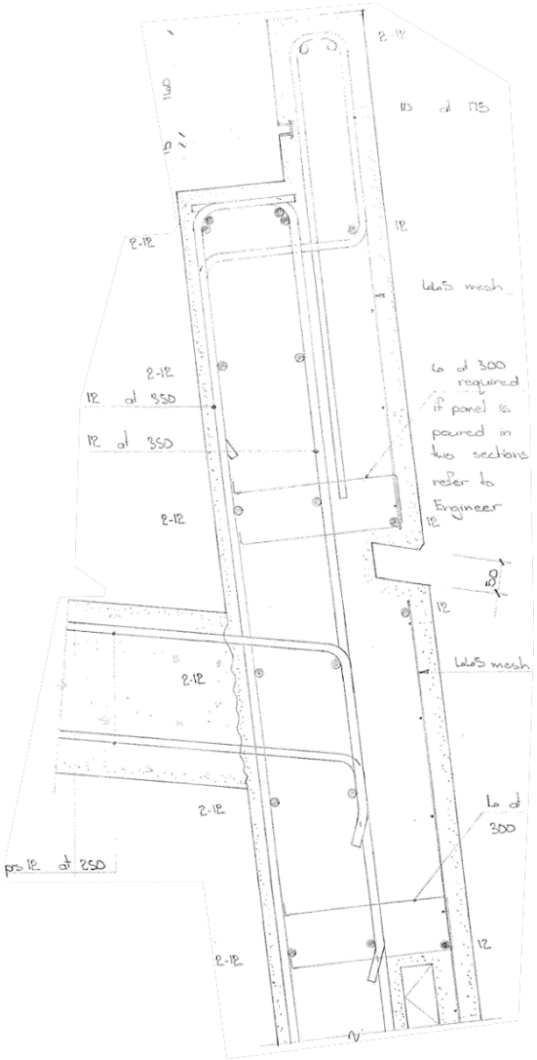

Figure C.4-4: Typical detail of the precast concrete wall panel joint with the roof

The capacity for each precast concrete wall panel is determined to be between 1246kN and 1370kN. This results in a shear stress of ~1.0 MPa.

The demand on the connection is taken from the output of the MRSA at a ductility of μ = 1.25, predicting a reasonably brittle failure mode upon exceedance.

The assessment resulted in a seismic score of **44%NBS$_{IL3}$** for this element.

Table C.4-1 : Assessment of interface shear between

Location	No. of starters	Capacity [kN[	Demand (at 100%NBS$_{IL3}$) [kN]	Rating [%NBS$_{IL3}$]
Bay 1(a)	42	1308	3340	50%
Bay 2(a)	42	1308	2381	78%
Bay 3(a)	42	1308	3061	56%
Bay 4(a)	42	1308	3251	52%
Bay 5(a)	40	1246	2622	65%
Bay 6(a)	44	1370	2836	65%
Bay 7(a)	43	1339	3792	**44%**

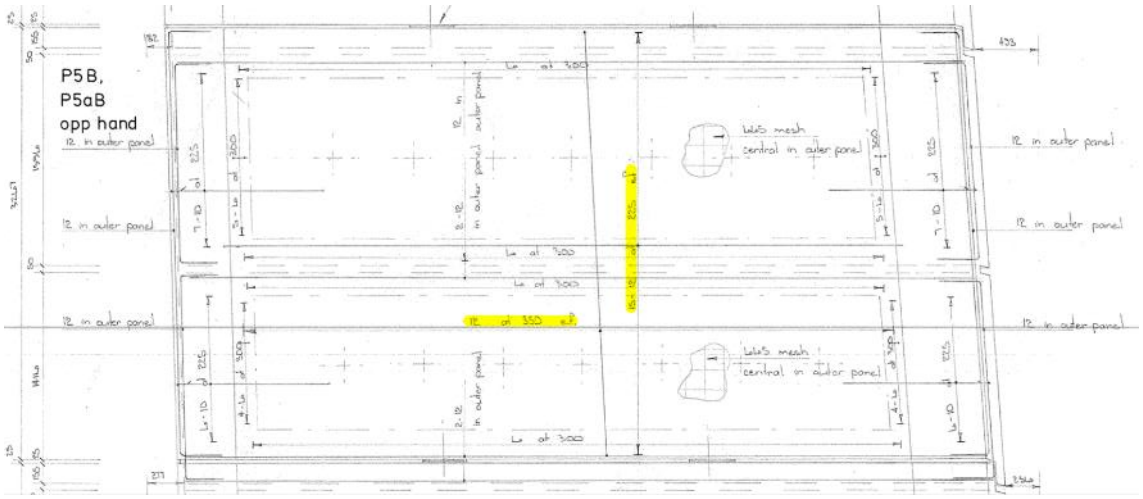

Figure C.4-5: Typical reinforcing of the precast concrete wall panels

Shear flow through each precast concrete wall panel

Once the lateral loading is in the precast concrete wall panel, it will flow down through the panel to the connections with the panel below it. The walls are reinforced with a layer of Ø12mm bars at 350mm spacing at each face of the panel spanning vertically. The horizontal reinforcement is Ø12mm bars at 225mm spacing at each face. This reinforcement lay-out, shown in Figure C.4-5, is the same for all the precast concrete wall panel configurations.

The precast concrete wall panel is considered as a wall element and the shear capacity is determined in accordance with Section C5.5.5.1 of the Engineering Assessment Guidelines. For the purpose of these calculations, it was assumed that the α-factor could be taken as 1.5 and the γ-factor as 0.29, as the wall is unlikely to see major flexural demands and flexural degradation.

The demand on the connection at 100%ULS loading was taken to be similar as for the interface shear calculation with the roof interface.

The analysis has however shown that this transition area can develop the overstrength capacity of any plastic mechanism below it. Therefore, the assessment of this portion of the precast buttresses is considered with the global system ductility (μ_{sys} = 2.0). The capacity is lowest at Bays 3(a) and 4(a) which have a seismic score of **85%NBS$_{IL3}$**.

Shear-flow between precast concrete wall panels

The precast concrete wall panels are connected to the precast buttress columns through a vertical stitch zone and are connected to the precast concrete wall panels below it through weld plates.

The weld plate detail is shown in Figure C.4-6 and the cast-in bracket in Figure C.4-7. In each of the precast concrete wall panels there are two of the cast-in plates at the top edge and bottom edge. They comprise a 100x10mm flat plate which is 300mm long and have 40x10mm hooked plates anchored into to the concrete. At the connection, there is a 350mm long 190x25 flat plate provided between the two panels which provides room for a welded connection.

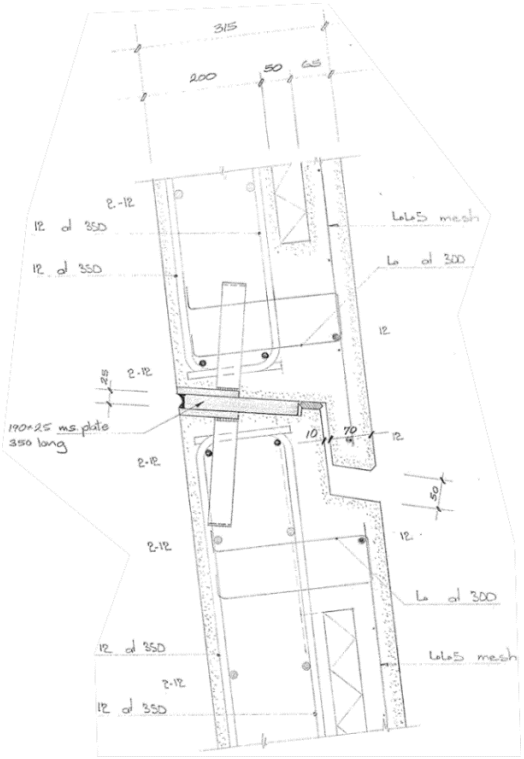

Figure C.4-6: Typical weld plate detail between precast concrete wall panels

The welded connections between the plates, including those of the hooked plates, are not called out on the original drawings nor the original structural specifications. For the purpose of this assessment however, they are considered to not be the weakest link.

There are three potential modes of failure considered for this connection. Firstly, a limit provided by the weld in the connection. Secondly, the yielding of the 40x10 flat hooked bars in tension and secondly, the pry-out of the flat hooked bar due to the shallow embedment into the concrete element. These modes are not specifically described in the Engineering Assessment Guidelines, so reference is made to Chapter 17 of NZS3101:2006, which represents the closest source of knowledge applicable to the assessment of this element. However, in lieu of the strength reduction factor prescribed in NZS3101:2006, a factor of 0.85 is used in the assessment.

It is found that the capacity of this weld plate connection is limited by the pry-out of the concrete, at a capacity of **V_{wp} = 84kN**. And so, the weld plates are likely only providing minimal capacity to the shear-flow between the precast concrete wall panels. It also gives some justification that the welds are unlikely to govern the capacity of the connection, as this corresponds to a 4mm fillet weld of 100mm length. The actual weld is expected to be larger. However, if the weld plates are relied upon to justify a higher seismic score, than it is recommended that they are visually inspected as a minimum.

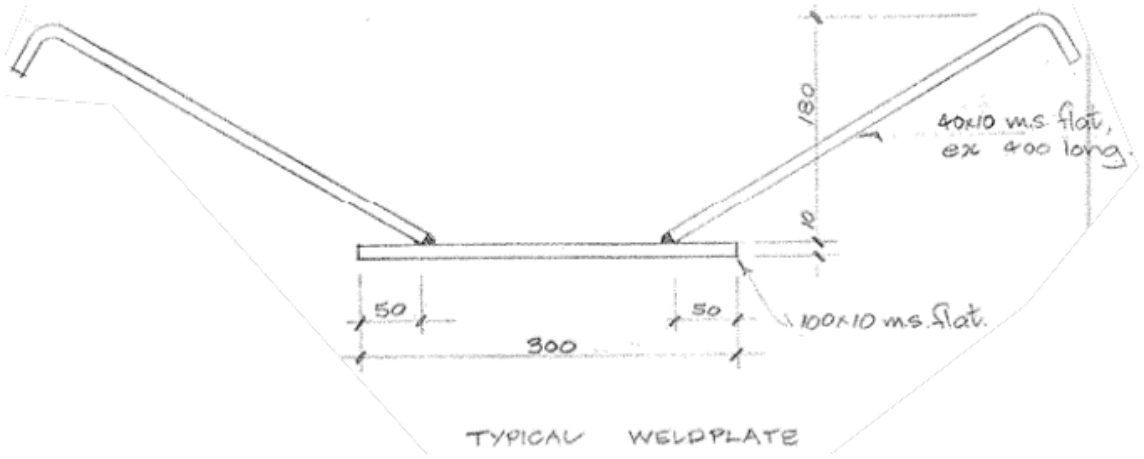

Figure C.4-7: Typical weld plate configuration

In the stitch zone, highlighted in Figure C.4-3, the remainder of the shear needs to be taken out. This area is horizontally reinforced with 2-Ø12mm bars of the lower reinforcing grade, spaced at 225mm. The stitch zone has 4 reinforcing bars: 2-Ø32mm bars of the higher reinforcing grade and 2-Ø12mm bars of the lower reinforcing grade. The shear is considered to included the stitch zone (~500mm wide) and the width of the internal 'fin' of the precast buttress (250mm) wide.

The shear capacity of this area has been determined in accordance with Section C5.5.5.1 for walls. The concrete contribution and shear contribution consider the following parameters:

▪ α	1.5	low flexural demands in section
▪ β	0.87	reinforcement in the stitch zone
▪ γ	0.29	undegraded properties
▪ c	30mm	
▪ c_0	20mm	
▪ θ_{cr}	45°	

No contribution from axial loads has been considered as this is expected to distribute to the precast buttress columns and the two weld plates, resulting in nominal axial load on the stitch zone.

This results in a total shear capacity of $0.85 \cdot (V_c + V_s)$ = **437kN** across each shear interface.

Considering that there two interfaces for each panel and there are two weld plates, this results in a total shear capacity across the panels of **1042kN.**

This shear-flow capacity between precast concrete wall panels was found to be unable to account for the overstrength of any of the more ductile mechanisms in the primary lateral load resisting structure. As such, the demand on the connection is taken from the output of the MRSA at a ductility of μ = 1.25, predicting a reasonably brittle failure mode upon exceedance.

The capacity is lowest at Bay Walls of Bays 1(a) and 7(a) and results in a seismic score of **25%NBS$_{IL3}$**.

Transition of shear load path into the cast in-situ portion

At the base of the precast concrete elements of the Bay Walls the shear flow is only possible through the precast buttress columns as highlighted in Figure C.4-3. The precast buttresses are horizontally reinforced with 2-Ø12mm and 7-Ø10mm bars of the lower reinforcing grade, spaced at 100mm. The precast buttresses

have considerable longitudinal reinforcing: 6-Ø32mm bars and 4-Ø28 of the higher reinforcing grade and 6-Ø20mm bars of the lower reinforcing grade.

The shear capacity of this area has been determined in accordance with Section C5.5.5.2 for columns as the section is relatively slender. It will act more as a column in shear than a wall. The concrete contribution, shear contribution and axial load contribution consider the following parameters:

- k_{nl} 1.0 undegraded properties
- M* / V·d 2.0 low flexural demands in section
- N^*_G 500kN the weight of the roof and the precast elements
- d 328mm
- $\alpha_{v,col}$ 1.0 closely spaced transverse reinforcing

The concrete contribution including the influence of axial load is 1209kN and the shear contribution is 825kN.

Considering there are two precast buttresses providing this shear load path for each Bay Wall, this results in a total shear capacity across the panels of **4067kN.**

The analysis has however shown that this transition area can develop the overstrength capacity of any plastic mechanism below it. Therefore, the assessment of this portion of the precast buttresses is considered with the global system ductility (μ_{sys} = 2.0).

The capacity is lowest at Bay Walls of Bays 1(a) and 7(a), but results in a seismic score of **100%NBS$_{IL3}$**.

Transition of overturning into the cast in-situ portion

At the base of the precast concrete elements of the Bay Walls there is an integral connection between the cast insitu buttresses and the precast buttresses. The joint is formed by starters from the cast insitu buttresses that enter 'Uniflex' Ducts in the precast buttresses.

There is a considerable change in reinforcement quantity between the two elements, as the precast buttresses are reinforced with 6-Ø32mm bars and 4-Ø28mm bars of the higher strength grade reinforcement ($f_{y,prob}$ = 455 MPa), supplemented with 6-Ø20mm bars of normal grade reinforcement ($f_{y,prob}$ = 324 MPa).

On the other hand, the cast insitu portion is only reinforced with 4-Ø24mm bars and 6-Ø20mm bars of lower grade reinforcement – which only represents ~30% of the yield strength of the precast buttress above it. This suggests that plastic behaviour is likely to be expected at the transition area or below it – although only if this section exceeds the capacity.

The transition zones at Bays 6(a) and 7(a) have a larger reinforcement content. This is likely because they support the precast cantilever beams at the Level 5 floor level. The effect of this additional overturning has been considered in the capacity checks of the transition zone.

The flexural capacity at the transition zone is limited by the capacity through the 'Uniflex' ducts, as the web only provides a transition through the weld plates – and, as shown, in the previous subsection, they are unlikely to contribute much to that end. Once the tension load is transferred to the cast insitu portion, then the full cast insitu system below it can be engaged further down the system.

The capacity of this transition area has considered a small increase in demand from the eccentricity in plan between the connection at the roof and the connection to cast insitu portion of the bay walls. This originates from the slope of the precast wall panels. The demand expresses itself as an interface shear demand on the transition zone.

To assess the flexural capacity of the precast system overall (as a C-shaped section) and the probable inelastic rotation capacity of this transition zone, the section is considered to act as a couple between the two precast buttresses at each Bay. It is expected that distributed cracking cannot form considering the nature of the joint and that the inelastic deformations will form in a single crack. In accordance with Section C5.5.3.4 of the Engineering Assessment Guidelines, the associated plastic hinge length for a connection with ducts (such as 'Uniflex' or 'Drossbach') is limited to 1.5x the strain penetration length. Considering then that the reinforcement can reach up to 6% strain at the ULS, there is a plastic rotation capacity of the global precast system of ~0.17%.

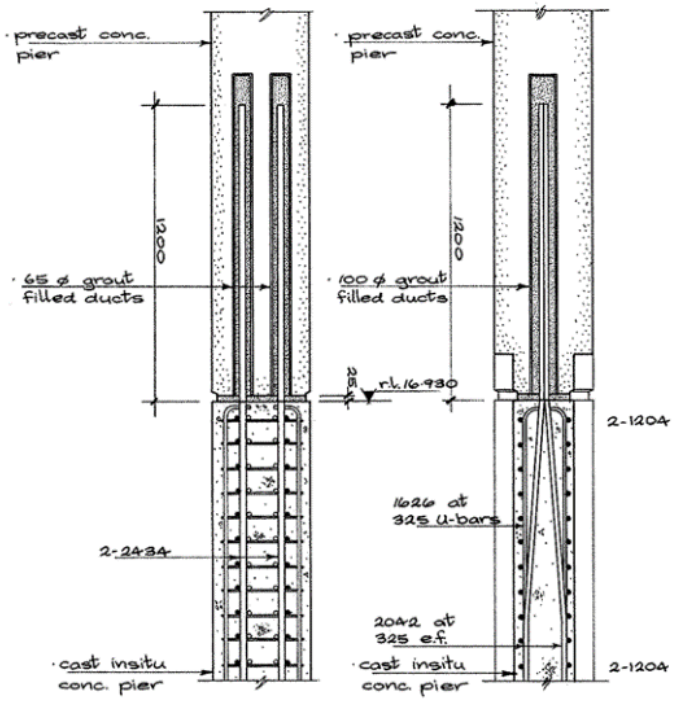

Figure C.4-8: Typical splice joint between the precast buttresses and the cast insitu buttresses

This suggest that any ductility capacity would be small (μ = ~1.25), when considering the upper bound yield rotations of the bay walls (see Section C.4.1.1.2).

The analysis has however shown that this transition area can develop the overstrength capacity of any plastic mechanism below it. Whilst the reinforcement is typically continuous across the cast insitu buttresses, the lower cast insitu portion sees additional overturning in the section, which makes that area critical for any flexural demands.

Therefore, the assessment of this portion of the precast buttresses is considered with the global system ductility (μ_{sys} = 2.0). The capacity is lowest at Bays 3(a) and 4(a) which have a seismic rating of **50%NBS$_{IL3}$**.

Other Bays have similar scores up to 75%NBS$_{IL3}$.

Out-of-plane capacity of the precast concrete wall panels

Seismic shaking in the out-of-plane direction of the precast concrete wall panels leads to out-of-plane flexure and shear demands. They would need to span between the precast buttresses and are considered to have a fixed connection at either end.

Since they could behave independently of the primary lateral load resisting system, they need to consider loading to the 'Parts & Components' accelerations specified in Chapter 8 of NZS1170.5:2004. The parameters that are considered are listed below. The ductility is limited at μ = 1.25, since the precast concrete wall panels are part of the in-plane primary lateral load resisting system and, therefore, cannot accept any inelastic out-of-plane displacement.

Table C.4-2: Parameters used in determining the 'Parts and Components' acceleration of the precast concrete wall panels out-of-plane

Parameter	Notation	Value	Commentary
Site hazard coefficient for T = 0s	C(0)	0.5824	NZS1170.5:2004 – Chapter 8
Part risk factor	R_p	1.0	Category P.1, P.2 and P.4
Floor height coefficient	C_{hi}	3.0	$h_i > 0.2h_n$
Part spectral shape coefficient	$C_i(T_p)$	2.0	T < 0.75 s
Part horizontal response factor	C_{ph}	0.85	μ = 1.25

The capacity of the precast concrete wall panels for out-of-plane loading is **35%NBS$_{IL3}$**.

C.4.1.1.2 Lower cast insitu portion

The lower portion of the bay wall comprises a cast insitu C-shaped wall that cantilevers from the ring beam that runs around the Auditorium. The web area is 250mm thick and approximately 7.4m long between the centrelines of the buttresses. The buttresses are also 250mm thick and are 2.2, long with 400x500mm columns at their ends. The end columns are sometimes part of the cast insitu system, however, on other occasions they are formed by precast concrete elements which have likely been used as permanent formwork.

There are precast cantilever beam elements attached to the buttresses of the bay walls that cantilever inwards or outwards of the Auditorium. At Level 4 these precast cantilever beams act as a balanced cantilever system around the buttress, with the promenade floor on the outside and the stepped seating on the inside of the Auditorium. At Level 2 the precast cantilever beams support the promenade floor on the outside of the Auditorium.

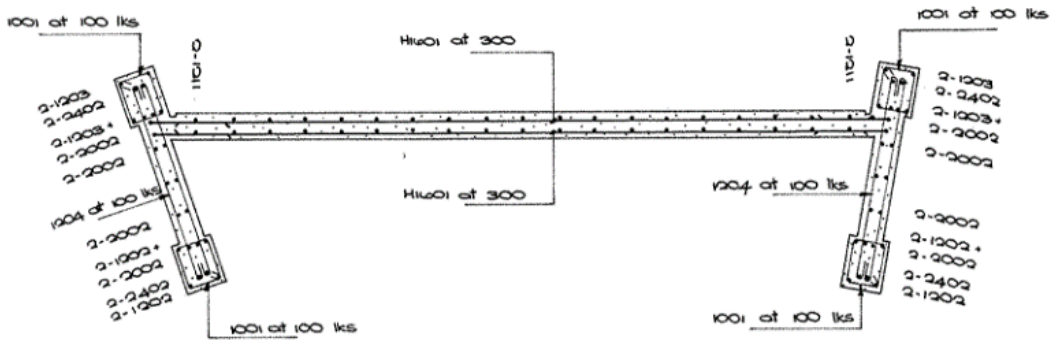

Figure C.4-9: Typical configuration and reinforcing of the cast insitu portion of the bay walls

The web areas are doubly reinforced in two-ways. The vertical reinforcement is Ø12mm bars at 350mm spacing, all throughout the height of the cast insitu wall. The horizontal reinforcement typically has three different configurations. Between the ground floor level (Level 1) and Level 2 the walls have Ø16mm high strength grade reinforcement at 200mm spacing. Up to Level 4 the walls have the same Ø16mm bars, but only at 300mm spacing. The upper portion, till the connection with the precast portion has Ø12mm bars of the lower reinforcement grade at 225mm spacing. There are no links connecting the reinforcement at the different faces of the wall.

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The buttress walls are typically vertically reinforced with four Ø24mm bars and ten Ø20mm reinforcement bars, all the lower reinforcement grade. The horizontal reinforcement comprises Ø12mm stirrups at 100mm spacing. The end columns each add another four Ø12mm bars to the vertical reinforcement which are confined with Ø10mm stirrups at 100mm spacing. The buttresses also don't have any links connecting the reinforcement at either face.

The walls act as a cantilever system from the foundation level upwards. The primary lateral load resisting system acts mainly in the in-plane direction of the C-shaped wall. The consideration of semi-rigid diaphragms at the roof level and the Level 4 promenade distributes the load away from the more flexible out-of-plane direction of the wall – especially because the precast buttresses above the buttresses are even more flexible.

Because of the presence of the squat walls at Level 1, there is a lot more strength in the primary lateral load resisting system of the Auditorium at this level. The semi-rigid diaphragms at Level 2 (including the sloping transfer diaphragm at Level 1/2) drag a lot of lateral loads away from the bay walls and into these squat walls. This results in the general behaviour that the shear load in the walls is typically critical in the web area above Level 2. However, the flexural demands in the section, especially the net compression and tension in the buttresses, are still required to be taken down directly to the foundations. Consideration is therefore given to two scenarios that could limit the capacity of the cast insitu portion of the bay walls:

- Shear of the web area above Level 2 with nominal ductility capacity.
- Flexural behaviour of the total section with a plastic hinge developing either locally at the ground floor level, across the Level 2 floor height or at a larger height covering both those areas.

Behaviour in flexure

The flexural capacity and overstrength are determined through hand calculations and are verified through modelling of the section in Response 2000.

If the bay walls are controlled by a flexural behaviour, then the cast insitu portion of the wall has been found to most likely yield either at the transition at the Level 2 floor or at the ground floor level (Level 1). It is expected that the hierarchy is largely dependent on the amount of restraint that the floor levels can provide – which is difficult to predict. In the analysis model, the floor levels are modelled as thin shell elements which can underestimate this restraining effect.

The assessment has found that the level of ultimate limit state shaking which yields the wall is similar at the wall between Level 1 and Level 2 and at the wall above Level 2. Even the outliers which the assessment has found are within the overstrength capacities of one another. This was also only observed at the walls of Bays 3(a) and 4(a). And it is noted that the floors in this area comprise double-tee flooring elements, which may crack (and soften) relatively early.

This consideration concludes that the inelastic mechanism is most likely to form above Level 2. However, the assessment has accounted for these mechanisms to occur independently as well as simultaneously. If the hinge occurs at the base of the structure than the plastic hinge length can be determined in accordance with equation C5.49 of Engineering Assessment Guidelines – with the shear span parameter ($k \cdot L_c$) being the largest contributor in that equation. If the wall forms a hinge above Level 2, then the plastic hinge length is conservatively considered using the same equation, however, considering the shear span taken from Level 2. Lastly, if the wall yields simultaneously over the both the yielding locations, then the walls are considered to form a plastic hinge that extends from the ground floor level up to the extent of the plastic hinge length above Level 2. The strain penetration length is determined on the basis of the Ø12mm vertical bars in the web area of the wall.

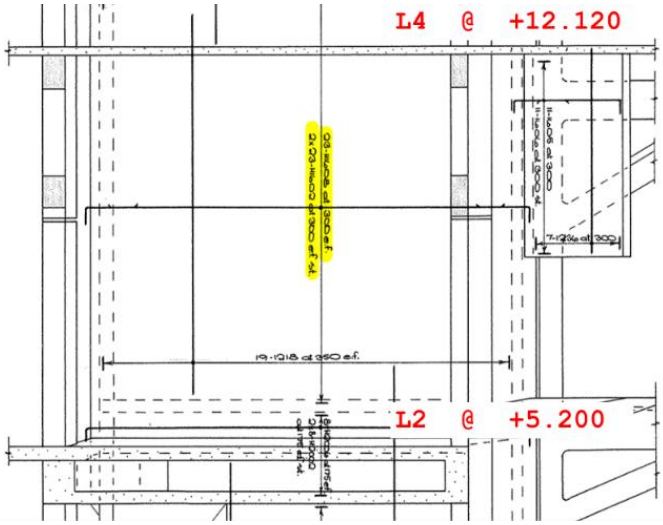

Figure C.4-10: Elevation of a typical buttress wall, highlighting the limited amount of horizontal reinforcement between Level 2 and Level 4

The inelastic rotation capacity is limited by bar buckling, both in the web area and in the buttresses. The walls don't have any reinforcement that appears to be particularly intended for confinement. The wall will likely form large cracks that can penetrate the entire section, resulting in bar buckling. The equations C5.57 and C5.58 of the Engineering Assessment Guidelines are copied below for reference.

$$\varphi_{cap} = \frac{\varepsilon_p^*}{\gamma \cdot L_w} \qquad 0.02 < \varepsilon_p^* = \frac{11 - (s/d_b)}{150} \leq 0.06$$

In the web area, the horizontal reinforcement is spaced at distances much larger than 11x the bar diameter. This requires for the lower bound of 2% buckling strain to be considered in the curvature capacity, resulting in a curvature capacity of the wall of 2.8 rad/km. The buttresses are slightly better confined by the stirrups as the reinforcement is spaced at 5x the bar diameter, resulting in a limit of 4% buckling strain. However, since the web area is the primary load path for shear above Level 2, the inelastic rotation capacity of the wall is considered to be limited to the 2% buckling strain limit.

The yield curvature of the wall can be determined in accordance with the equations by Priestley et al. (2007) for flanged shear walls at 0.33 rad/km, the equation of which is given below.

$$\varphi_y = \frac{1.5 \cdot \varepsilon_y}{L_w}$$

This allows for the inelastic rotation capacity to be calculated for the three scenarios that are considered.

$$\theta_p = (\varphi_{cap} - \varphi_y) \cdot L_p$$

The outcome is presented in Table C.4-3.

Yield rotation and ductility capacity

The elastic behaviour of the bay walls is influenced by two major factors, the yield curvature within the wall itself and the elastic rotations of the foundations. The effect of these combined give the total yield drift of the building.

The flexibility of the foundations is considered in the analysis model, both laterally and vertically. The vertical stiffness of the piles is considered as the compression stiffness as the piles do not see any net tension at the yield point of the bay walls. Both the upper and lower bound stiffness have been considered in accordance with the recommendations from the Geotechnical Assessment Report. The lower bound stiffness scenario also considers the perimeter ring beam with stiffness modifiers to represent a cracked concrete section.

To determine the yield rotations of the foundation, the node displacements are extracted of the bay walls at their connection to the foundation system. This is undertaken for the MRSA load combinations in the X-direction for Bays 3(a), 4(a) and 5(a) and the Z-direction for Bays 1(a), 2(a), 6(a) and 7(a). The load combinations have been considered at 35%ULS loading at a ductility of μ=1.0 and a structural performance factor of S_p=1.0 as this was found to be the approximate yield point of the bay walls. This approach concluded that the yield displacements range between <0.01% and 0.17% yield drift. At the lower bound, the yield rotations from the foundations have been ignored.

The yield rotation due to the internal forces in the wall have been determined using the equations in Section C5.5.3.1 of the Engineering Assessment Guidelines and the references from Priestley et al. (2007). The yield curvature is determined for flange walls as described earlier in this section. The β_v-factor has been determined on the basis of a Table C6.6A of NZS3101:2006 as per the recommendations of the Engineering Assessment Guidelines. For yielding that occurs at the base of the bay walls, the shear-span-to-depth ratio ($\frac{M}{V \cdot L_w}$) was determined to be ranging between 2.0 and 3.0. For the walls yielding above the Level 2 floor this ratio was between 1.0 and 1.5. The length of the shear span (L_c) has been considered as the height of the wall from the location of the potential plastic hinge till the roof.

$$\theta_y = \beta_v \cdot \varphi_y \cdot \frac{L_c}{3}$$

Table C.4-3: Ductility capacity of the bay wall, based on flexural behaviour

Location of plastic hinge		Yield rotation from foundations [%]	Yield rotation from flexure of wall [%]	Plastic hinge length [m]	Inelastic rotation capacity [%]	Ductility capacity [-]
Above level 2	Lower bound	0.00	0.36	2.37	0.58	2.60
	Upper bound	0.17	0.48	2.37	0.58	1.89
At base (Level 1)	Lower bound	0.00	0.36	2.72	0.66	2.87
	Upper bound	0.17	0.38	2.72	0.66	2.21
Large hinge developing over Level 1 and Level 2	Lower bound	0.00	0.36	6.69	1.64	5.61
	Upper bound	0.17	0.38	6.69	1.64	3.97

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The probable strength hierarchy of the system suggests that the bay wall will most likely form a hinge above Level 2. This behaviour suggests that the ductility of the system ranges between μ = 1.90 and μ = 2.60.

For the purpose of this assessment, the probable global system ductility of the bay walls is assumed to be μ_{sys} = 2.00. Whilst the lower bound (see Table C.4-3) is slightly lower than this, this was determined on a wall-by-wall basis, and the global system ductility is expected to be slightly higher.

Behaviour in shear

The shear capacity has been determined in accordance with Section C5.5.5.1 of the Engineering Assessment Guidelines, adopting a shear strength degradation factor of γ = 0.29 to assess the strength hierarchy.

A shear magnification factor is not included in the assessment. The Auditorium effectively responds as a building with a low number of storeys, therefore higher mode effects are expected to be minimal. Also, there is sufficient margin between the overstrength demands from the flexural mechanism in the walls and that shear is not critical.

If the bay walls are capable of developing larger ductilities, then they could become flexure-shear limited at at a ductility which is estimated at μ = 4.0. This is because the bay walls will see degradation of the shear capacity as the plastic hinges are formed. However, this scenario is not considered to be the controlling mechanism of the bay walls.

If the bay walls were to perform in a shear critical manner, this would most likely develop above the Level 2 floor.

A wall that is critical in shear or flexure-shear may is expected to behave with low ductility. In accordance with the Engineering Assessment Guidelines, the potential inelastic displacement capacity is considered as per Equation C5.45, copied below for reference. The bay walls have an axial load ratio which requires the lower bound deformation capacity to be considered.

$$\frac{\Delta_{cap}}{L_c} = 0.0075$$

Buckling of the bay walls

The bay walls have a relatively long unrestrained length between Level 2, the precast cantilever beams at Level 4 and the roof structure. This may lead to buckling of the web or of the flanges of the C-shaped section.

Buckling of the flanges is discussed in Section 11.4.3 of NZS3101:2006 and by Equation C5.25 of the Engineering Assessment Guidelines. However, the geometry of the flanges comply with these requirements and no further reduction on the inelastic rotation capacity is required.

Buckling of the web is not explicitly covered for walls with flanges in NZS3101:2006 nor the Engineering Assessment Guidelines. It is expected that the magnitude of energy required to buckle the web is so enormous that other elements would yield first and relieve the compression strut of the energy.

Irregularity at Bays 6(a) and 7(a)

The bay walls are differently configurated at Bay 6(a) and 7(a) as they have their web areas terminated above Level 2. At Bay 6(a) one of the buttresses extends to the foundations and the ground floor level (Level 1) as it would for the typical bay wall configuration. It also supports a precast cantilever beam at the Level

2 promenade floor. However, the other buttress of Bay 6(a) and the buttresses of Bay 7(a) transition to sets of two independent 400x500mm columns. On the outside of the Auditorium these columns support precast beams that are part of an extended frame system that comprises another set of two 400x500mm columns.

Both the independent buttress at Bay 6(a) and the independent sets of 400x500mm columns are reinforced with higher quantities of vertical and horizontal reinforcing. The buttress at Bay 6(A) that extends to the foundations has another eight Ø28mm vertical bars and there are Ø16mm stirrups at 100mm spacing. The sets of two 400x500mm columns are reinforced with eight Ø32mm vertical bars and with Ø12mm stirrups at 100mm spacing. All providing a significantly larger reinforcement ratio then the typical buttress element above it.

With the higher content of reinforcing the columns and buttress are able to develop the flexural overstrength of the walls above them – in the couple that is generated from the flexural demand.

The columns are however much more flexible in their out-of-plane direction (in-plane of the bay walls) and are expected to be reliant on the sloped diaphragms at Level 1/2 and Level 2 to restrain their movement.

As the bay walls at Bays 6(a) and 7(a) overturn, the couple generates a compression force that now can only be taken by the set of 400x500mm columns. As such, these columns achieve critically high axial load ratio's at the overstrength flexural capacity of the wall above it. The bay walls have been determined to reach the yield capacity of the walls at ~25%ULS loading.

At the flexural overstrength of the bay walls, the columns geometrically distribute the axial load on a 78:22 ratio between the outer column and the inner column. This is because the web area is more aligned with the outer columns. The outer 400x500mm columns could therefore see another 5,000kN on top of their gravity load demands of 1,200kN. This puts their axial load ratio at ~60%.

Considering Equation C5.59 of the Engineering Assessment Guidelines, referenced below, this means that the inelastic rotation capacity of the column should be limited to <0.01% drift. Effectively, the column can only reach its yield capacity.

$$\theta_a = \frac{0.25}{5 + \frac{N^*}{0.8 \cdot A_g \cdot f'_{c,prob}} \cdot \frac{1}{\rho_t} \cdot \frac{f'_{c,prob}}{f_{y,prob}}} - 0.005 \geq \theta_p$$

The yield drift of the columns have been determined in accordance with Section C5.5.3.1 of the Engineering Assessment Guidelines, considering a β_v-factor of 1.0, a yield curvature in accordance with Priestley et al. (2007) and a shear span of half the storey height. This gives a yield drift of 0.59%.

Therefore, the total drift of the Auditorium at this location should be limited to 0.59% drift to prevent the columns losing their axial load carrying capacity.

Considering the drift of this location, the columns at Bays 6(a) and 7(a) are rated at **55%NBS$_{IL3}$**.

C.4.1.1.3 Strength hierarchy

Table C.4-4 summarises the strength hierarchy of the bay walls. In general, it concludes that the primary mechanism of the bay walls is the formation of a flexural mechanism, just above the Level 2 floor. The original 1980's design intend aligns with that. However, this table is used to assess which elements (precast walls, 'Uniflex' ducts location, shear mechanisms, etc.) are protected from the overstrength demands developing in the bay walls.

The analysis shows that yielding of the flexural mechanism in the lower cast insitu portion of the bay walls is triggered between 25-50% of ULS loading. See the 'ULS-to-yield'-rows, corresponding to the 7th column. The large variance is caused by foundation flexibility, location in the building, height in the building, etc. The most likely location of the initiation of a flexural hinge is just above floor level 2 (say, 25%ULS). However, using upper bound properties (including overstrength), the analysis shows that a hinge may be formed at much larger levels of ULS loading (say, up to 50%ULS). Hence, the yield strength is presented as a range for each bay walls.

Then, independently, the capacity of other elements of the bay walls was assessed. For example, the shear capacity of the walls. The 8th column shows that this is expected to yield in shear between 40-60%ULS loading, with the critical location at Bays 7(a) (42%ULS). However, what the table also shows, is that the lower cast insitu portion of the bay walls is likely (most probable) to yield much earlier than that. The 'Ratio' rows highlight this. Therefore, even when accounting for overstrength actions, the bay walls are not expected to form a shear mechanism.

The third row 'Overstrength' highlights whether each of the various mechanisms is capable of dealing with the flexural overstrength of the bay walls forming from the flexural mechanism, just above the Level 2 floor. If the answer is 'yes', then this element is capacity design protected, and it is not a Structural Weakness. If the answer is 'no', then the element is assessed to its own member ductility. For brittle elements, such as shear, this is typically $\mu = 1.25$. The latter may mean that this element could be a Structural Weakness.

An overstrength factor of $\phi_o = 1.25$ has been used in the assessment.

Furthermore, the flexural strength of the bay walls at the ground floor level (Level 1) could also be exceeded due to the overstrength demands of the bay walls above Level 2. This could result more distributed cracking and a larger plastic hinge zone. This would result in a larger global system ductility.

For the lower cast insitu portion it is found that this has a seismic score of **50%NBS$_{IL3}$**, governed by the inelastic rotation capacities in the web area – reduced to account for bar buckling. It is limited at Level 2 of the bay walls of Bay 7(a).

At other locations, such as Bays 3(a) and 4(a), there is yielding which starts at the Level 1 interface. This results in a seismic score of **55%NBS$_{IL3}$** at a system ductility of $\mu_{sys} = 2.0$. However, this area may potentially consider a higher system ductility as demonstrated in Table C.4-3.

Table C.4-4: Parameters used in determining the 'Parts and Components' acceleration of the precast concrete wall panels out-of-plane

Bay		Interface shear at connection with roof	Shear through the precast concrete wall panels	Shear between precast concrete wall panels	Transition of loading from precast into cast insitu (at 'Uniflex' ducts)	Flexure in cast insitu wall between Level 2 and Level 4	Shear in cast insitu wall between Level 2 and Level 4	Flexure in cast insitu wall between Level 1 and Level 2	Tension/Compression in 400x500mm columns
1(a)	%ULS-to-yield [1]	31	35	25	45	25 - 29	50	29	
	Ratio [2]	1.25	1.42	1.00	1.67	1.00	2.05	1.18	
	Overstrength [3]	No	Yes	No	Yes	Global	Yes	No	
2(a)	%ULS-to-yield	54	45	25	65	30 - 49	53	25	
	Ratio	1.76	1.48	0.86	1.64	1.00	1.74	1.00	
	Overstrength	Yes	Yes	No	Yes	Global	Yes	No	
3(a)	%ULS-to-yield	41	35	25	45	26 - 42	51	22	
	Ratio	1.59	1.34	0.98	1.33	1.00	1.97	1.00	
	Overstrength	Yes	Yes	No	Yes	Global	Yes	No	
4(a)	%ULS-to-yield	38	45	25	40	24 - 38	53	23	
	Ratio	1.56	1.86	1.02	1.28	1.00	2.20	1.00	
	Overstrength	Yes	Yes	No	Yes	Global	Yes	No	
5(a)	%ULS-to-yield	49	45	25	55	28 - 52	58	32	
	Ratio	1.76	1.63	0.92	1.38	1.00	2.11	1.16	
	Overstrength	Yes	Yes	No	Yes	Global	Yes	No	
6(a)	%ULS-to-yield	43	35	25	50	25 - 42	48		32
	Ratio	1.71	1.41	1.01	1.50	1.00	1.95		1.30
	Overstrength	Yes	Yes	No	Yes	Global	Yes		Yes
7(a)	%ULS-to-yield	32	35	25	50	23 - 32	42		32
	Ratio	1.40	1.52	1.05	1.83	1.00	1.82		1.40
	Overstrength	Yes	Yes	No	Yes	Global	Yes		Yes

1 The '%ULS-to-yield' rows indicate the level of seismic shaking required to initiate yielding of this element at the loading which considers μ=1 and S_p=1.
2 The 'Ratio' rows indicate the ratio of the elements compared to the weakest link in the overall system (typically plastic hinge formation at Level 2).
3 The 'Overstrength' rows indicate whether the element has the capacity to withstand the overstrength demands of the flexural mechanism at Level 2 where the same global system ductility (μ=2.0) is used. For elements that cannot withstand this, and perform with less ductility, their element ductility is used. This considers nominal ductile behaviour is assumed (μ=1.25) for most elements, and an element ductility of μ=1.70 for the transition zone with the 'Uniflex' ducts.

C.4.1.2 Squat Walls at Level 1

At the ground floor level (Level 1) the primary lateral load resisting system is supplemented by a series of reinforced concrete squat walls. These walls are located surrounding the backstage area and inside the Auditorium, providing several rooms, shown in red on Figure C.4-11. There are also a series of concrete masonry block walls, shown in green, however, these are not considered as the lateral load resisting system of the building. The concrete masonry block walls are considered as secondary structural elements in Appendix G, Section G.4.

The walls that are considered to be part of the primary lateral load resisting system of the Auditorium are considered in this Section. These are all reinforced with the higher reinforcing grade.

For this assessment the squat walls are considered to be governed by their shear capacity and to respond nominally ductile (μ=1.25) to seismic shaking. However, a higher ductility for some of the walls may be feasible when adopting the displacement capacities of Equations C5.44 and C5.45 of the Engineering Assessment Guidelines. In addition, they can redistribute the seismic loading to the adjacent Bay Walls.

Wall 1 and Wall 6 can redistribute loads between the Bay Walls of Bays 1(a) and 2(a), and Walls 14 can redistribute loads to the Bay Walls of Bays 4(a) and 5(a). For the assessment of these walls, the seismic score has been determined on the combined strength and displacement capacity of these wall systems.

The wall system that comprises Wall 16, Wall 18 and Wall 21 cannot redistribute the loads back to the Bay Walls, as the walls of Bays 6(a) and 7(a) are terminated above Level 2. Their capacities are assessed in isolation.

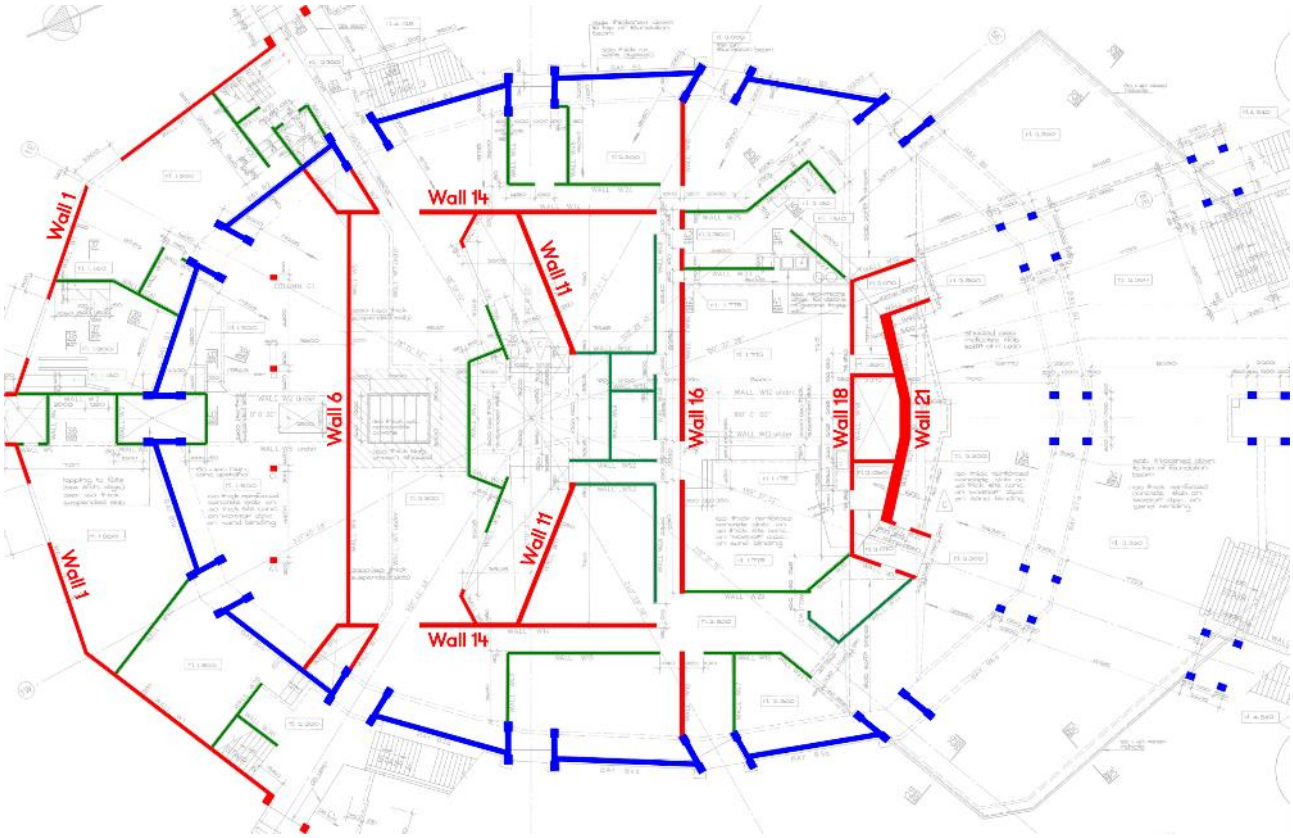

Figure C.4-11: Plan view showing the primary lateral load resisting system at Level 1

Wall 1 goes along the backstage area on the northern extent of the building. It is made up of several portions, separated by large access doors and the basement tunnel. It supports the Level 2 floor directly as well as a series of 400x400mm columns that support the Level 3 and Level 4 floors. The wall is supported on 11 piles along its length, predominantly located on the corner points. The wall is 200mm thick and doubly reinforced Ø16mm bars at 275mm spacing each way, at each face. The analysis showed that the wall helps with the capacity in the transverse (Z) direction of the Auditorium, but that it is offset from the rest of the primary lateral load resisting system and does not attract more load. Wall 1 would be scored at 65%NBS$_{IL3}$ when assessed to a ductility of (μ = 1.25). However, as described above, Wall 1 is capable of redistributing loads with the Bay Walls of Bays 1(a) and 2(a), and even Wall 6. For this assessment, this wall system has been given a seismic score of **70%NBS$_{IL3}$**, when the strength and displacement capacity of this combined wall system is exceeded.

The columns above Wall 1 are supported with an eccentric connection, the details of these are considered in C.4.3.4.

Wall 6 is located inside the Auditorium, under the stage area. It is a long continuous wall between the buttresses of the bay walls at Bays 2(a), where it connects into Wall 14. The wall is part of the basement system, providing the walls of the tunnel that runs across the Auditorium. It supports only the Level 2 floor at the stage area. The wall is supported on 4 piles along its length, located at the connection with Wall 14 and near the basement tunnel. The wall is 200mm thick and doubly reinforced with of Ø16mm bars at 250mm spacing each way at each face. The basement portion of the wall is 300mm thick and similarly reinforced. The analysis shows that the wall provides a lot of restraint to the overall system in the transverse (Z) direction. Particularly for the loads that come from the promenade floor at Level 2. These are tied back to this wall through the 400mm thick slab element locally. Wall 6 would be scored at 35%NBS$_{IL3}$ when assessed to a ductility of (μ = 1.25). However, as described above, Wall 6 is capable of redistributing loads with the Bay Walls of Bays 1(a) and 2(a), and even Wall 1. For this assessment, this wall system has been given a seismic score of **70%NBS$_{IL3}$**, when the strength and displacement capacity of this combined wall system is exceeded.

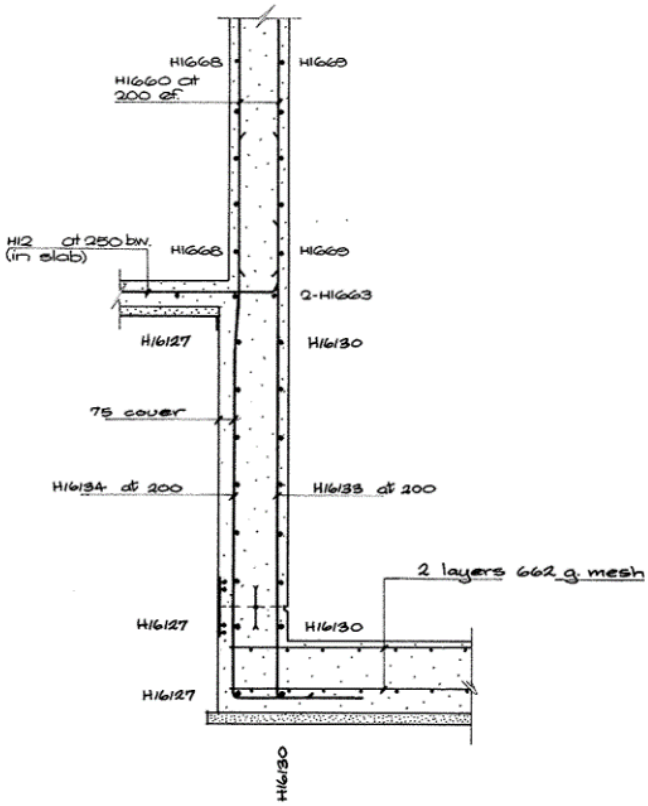

Figure C.4-12: Typical section of Wall 11 at the connection with the foundation. Wall 6 similar

Walls 11 are located on the sides of the movable stage lift. They comprise two symmetrical walls along the central axis of the Auditorium that extend to the basement floor. They connect into Walls 14 at one end and into concrete masonry block walls on the other end. They are supported by two Ø450mm 'Franki' piles at the extents of the wall. The walls are 250mm thick and are doubly reinforced with Ø16mm bars at 200mm spacing each way at each face. The basement portion of the walls is 300mm thick and similarly reinforced. The analysis shows that the wall predominantly helps take-out a large portion of the inertia of the Level 2 floor which can be directly tied back to it. Walls 11 are rated at **75%NBS$_{IL3}$**.

Walls 14 are the only longitudinal squat walls in the Auditorium. They comprise a wall on either side of the Auditorium that run parallel with the bay walls of Bays 3(a), 4(a) and 5(a). The walls connect into Walls 6 and 16 at their extends. They are supported on piles of various sizes, predominantly located at the corner locations. The walls are 250mm thick and are doubly reinforced with Ø16mm at 150mm spacing each way at each face. The walls are an integral part of the primary load paths through the building, as they attract a lot of lateral loading in the longitudinal (X) direction of the building and provide a horizontal tie support for loading in the transverse (Z) direction. Wall 14 would be scored at 60%NBS_{IL3} when assessed to a ductility of (μ = 1.25). However, as described above, Wall 14 is capable of redistributing loads with the Bay Walls of Bays 4(a) and 5(a). For this assessment, this wall system has been given a seismic score of **90%NBS_{IL3}**, when the strength and displacement capacity of this combined wall system is exceeded.

Wall 16 is located in the centre of the Auditorium and supports the Level 2 seating area. It extends all the way between the southern buttresses of Bay 4(a) to which they have an integral connection. The wall has door openings at various locations. The wall is supported by a series of 900x900mm foundation beams which connect to Ø450mm 'Franki' piles. The wall is 200mm thick and doubly reinforced with Ø16mm bars at 200mm spacing each way, at each face. The walls provide lateral support to a large area of the Level 2 floor and also contribute in the large transfer forces that come out of the Level 1/2 sloping diaphragm. Wall 16 is rated at **75%NBS_{IL3}**.

Wall 18 is located in underneath the Level 2 seating of the Auditorium and ties in with both the sloping floors at Level 1/2 and at Level 2. The wall has two small flanges (Walls 17 and 19) and is supported by four Ø450mm 'Franki' piles. The wall is 250mm thick and is doubly reinforced with Ø16mm bars at 200mm spacing each way at each face. Since the wall is connected to the Level 1/2 sloping diaphragm, it plays an important role in taking out the in-plan irregularity caused by the termination of the web area at Bays 6(a) and 7(a). Wall 18 is rated at **55%NBS_{IL3}**.

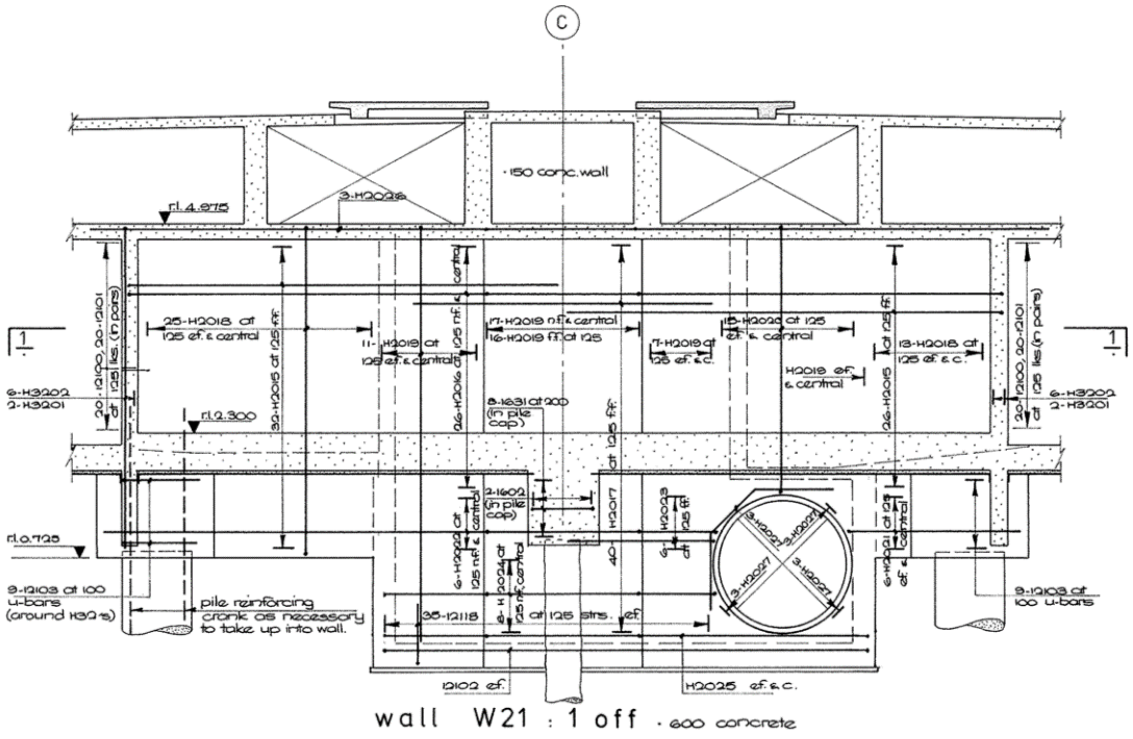

Figure C.4-13: Elevation of Wall 21

Wall 21 is located closest to the location where the Bays 6(a) and 7(a) have their web areas terminated at Level 2. At the ends, the wall has two small flanges (Walls 20 and 22). The wall connects into the sloping Level 1/2 floor and supports some of the beams that run between this floor and the Level 2 sloping floor. The wall is extended down to the basement structure, here there is a large Ø1.6m services tunnel that passes through the wall. The wall is supported by two Ø900mm bored piles and one Ø450mm 'Franki' pile. The wall is 600mm thick and is triply reinforced with Ø20mm bars at 125mm centres each way. The flanges provide additional reinforcing in another eight Ø32mm bars. The wall is the primary mechanism that 'replaces' the web area that is terminated at Bays 6(a) and 7(a), as such, it attracts a large transfer force coming out of the Level 1/2 sloping diaphragm. The wall is rated at **70%NBS$_{IL3}$** for its shear capacity.

Though it is noted that Wall 21 will likely reach its overturning capacity prior to its shear capacity. From all the squat walls, it is the shortest and it only has a single pile at each end to provide a tension capacity. At 45%ULS loading, the overturning overcomes the gravity demands in the analysis model. However, it is expected that Wall 21 will have sufficient capacity to overcome initiation and overstrength of inelastic mechanism in the primary lateral load resisting system of the Auditorium (including that of the sloping transfer diaphragm at Level 1/2).

C.4.2 Diaphragm Evaluation

C.4.2.1 Introduction

This section comprises the evaluation of the diaphragms of the Auditorium. The diaphragms have been evaluated using both macro and micro strut and tie analysis models in accordance with Section C5.6.1 of the Engineering Assessment Guidelines and Appendix A of NZS3101:2006. For a description of the build-up of the diaphragms, refer to section 5.1.1.4. The probable capacity and the performance of the diaphragms at each level will be covered subsequently.

Diaphragm demand

The seismic loading for each concrete diaphragm was determined through the NLTHA, which supersedes the NZS1170.5 pESA distribution. Due to the mass and story height irregularities, a traditional triangular pESA load distribution was considered to produce overly conservative diaphragm accelerations. The NLTHA model was created to produce an upper-bound response for 100% Ultimate Limit State (ULS) average diaphragm accelerations. The lateral soil stiffness for the model is for the undegraded, no liquified condition. Further detailed information on the NLTHA is in Appendix D. The NLTHA derived floor accelerations have been summarised in Table C.4-5.

Table C.4-5 : Overview of floor accelerations

Level	Floor acceleration in longitudinal (Z) direction [g]	Floor acceleration in transverse (X) direction [g]
Roof	1.42 (1.75[1])	1.54 (1.75[1])
Level 5	1.00	1.10
Level 4	1.05	0.95
Level 3	1.05	1.00
Level 2	1.13	1.05
Level 1.5	1.00	1.00

[1] Bracketed roof acceleration not used for assessment. The calculations and rating discussed in Section C.4.2.3 use the unbracketed values. Update to the higher acceleration would maintain the same seismic score for the roof diaphragm.

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'Parts and Components' loading has been used where local mechanisms occur, such as for individual double-tee units. This has been determined using a part spectral shape coefficient of C_i = 2.0 and a ductility factor of μ = 1.25 (C_{ph} = 0.85).

Micro strut & tie analysis

For the micro strut and tie analysis the diaphragms have been discretised into an orthogonal grillage of 1m with diagonal compression-only struts in Microstran V10. Using the ETABS analysis model the total shear at each level has been calculated from the differences in shear above and below the diaphragm. This shear demand has been equally distributed to the strut and tie grillage. The shear resultants at each lateral support point, a column or shear wall, have been calculated from the difference in the major and minor pier and column shears above and below the diaphragm. As a result of the varying orientation of the piers and columns these results have first been resolved to global x- and z-directions.

For model stability and equilibrium, two relatively weak, lateral spring supports have been included in the model. By providing both the demands and resultants in the Microstran model a strut and tie load path matching the ETABS model has been generated. This approach allows for individual diaphragm assessment without having to calculate the effective support stiffness of each individual lateral supporting element.

Only the primary positive combinations of floor accelerations have been extracted from the ETABS analysis models. The negative and concurrent load combinations have been included in the Microstran models.

Equally distributing the total lateral demand at each level to the diaphragm is quite conservative for levels having relatively small diaphragms. The total demand introduced at these levels no longer consists primarily of diaphragm mass, but also wall mass. At these locations the seismic inertia has been calculated by hand and the resultants at the supports extracted from the ETABS model have been scaled to match these demands. Out-of-plane mass of walls has also been accounted for in these models.

It is noted that the main disconnect between the ETABS analysis model and a typical Microstran strut and tie diaphragm model is that an orthogonal distribution of struts and ties is forced upon a circular (non-orthogonal) floor plan. The tension members in the strut and tie model then do not necessarily align with the reinforcement layout in the diaphragms. This can however be accounted for in the capacity calculations using trigonometry.

Macro strut & tie analysis

In the macro strut and tie model the extent of the struts and ties have typically been confined to the span of the different floor sections. This method has primarily been used to study more localised diaphragm behaviour. Demands have been calculated by amplifying the floor accelerations with the factors from 'Parts and Components' of Chapter 8 of NZS1170.5:2004.

Primary parameters and material properties

The primary parameters and material properties for the strut and tie assessment of the diaphragms have been summarized in Table C.4-6. Probable strengths for struts and ties have been allowed for in accordance with the assessment guidelines. The struts are assumed to not be bottle-shaped and for the micro models the strut width in generally limited to 1/3rd of the grid spacing for strength.

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Table C.4-6: Primary parameters and material properties used in determining the diaphragm performance.

Parameter	Notation	Value	Commentary
Probable concrete strength	f_c	37.5 MPa	Probable strength of slab concrete, per original specifications
Probable grade 275 yield strength (mild steel)	$f_{y,275}$	324 MPa	Assessment Guidelines Table C5.3
Probable grade 380 yield strength (high tensile)	$f_{y,380}$	455 MPa	Assessment Guidelines Table C5.3
Strength reduction factor (strut and tie)	φ	1.0	Assessment Guidelines Section C5.6.3.5
Strut shape factor	β_s	1.0	NZS3101 – A5.2 – not bottle shaped
Strut width (micro model)	w_{strut}	333 mm	Taken as 1/3rd of grid spacing

Cold-drawn (non-ductile) mesh

In accordance with requirements by Yellow Chapter C5, the cold-drawn (non-ductile) mesh has typically not been considered to provide reliable tie capacity as this is likely fractured at the transition of different floor interfaces due to cracking. Creep and shrinkage calculations have been undertaken to confirm the strains in the mesh exceed the tensile strains.

However, where the mesh can provide an internal tie while not crossing any precast floor boundaries, we have considered the capacity of the mesh – as the mesh is not considered to be subject to excessive differential shrinkage strains. An example is the mesh in the topping within the bounds of the prestressed double-tee unit floors at the Level 2 and Level 4 promenades. Outside of those bounds, when transitioning to an adjacent prestressed double-tee unit the mesh is not considered.

Inertia of double-tee units

The double-tee precast flooring typically needs to rely on the cold-drawn (non-ductile) mesh to resist its own inertia and transfer these into the supporting members. If the mesh has no capacity left, then the inertia of the double-tees needs to be transferred by other (less conventional) means.

The assessment has found that the double-tees are partially keyed into the cast insitu block-out area, as shown in Figure C.4-14. This has been dimensioned on the original 1980's structural drawings to have a dimension of (110-85=) 25mm. However, this localised key may be subject to construction tolerances and is likely in the concrete cover area. When subject to lateral loading, the key may therefore be prone to premature spalling and provide an unreliable magnitude of resistance.

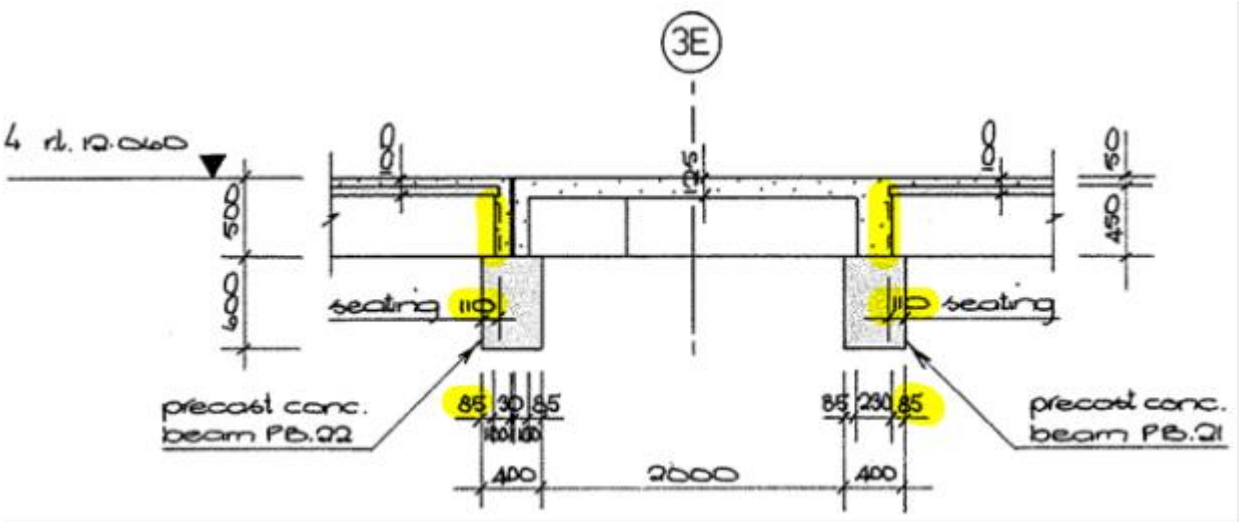

Figure C.4-14: Example of keyed in situation of double-tee units at floor Level 4

The only quantifiable manner of resistance, is a friction mechanism between the seating underneath the webs of the double-tees and the supporting beams. This mechanism has formed the basis of the assessment of these load paths. However, this is likely to be a conservative lower bound, as there are other non-quantifiable mechanisms that could provide additional capacity. Also, in the axis of the beams, the double-tee units may need to displace 250mm before the webs lose their seating. The actual life-safety risk of these elements may therefore be overpredicted.

C.4.2.2 Summary

The performance of the diaphragms has been summarized in Table C.4-7 below. This table is organized by floor level then by rating score.

Table C.4-7: Summary of the diaphragm assessment.

Level	Area description	Structural weakness	%NBS$_{IL3}$
Roof		Various interactions, refer C.4.2.3.	20 – 30
Level 5	Director's area	Ties tying the precast walls to diaphragm	85
Level 4	Promenade floor segments	Insufficient continuity/capacity of non-ductile mesh reinforcing to provide adequate tie capacity	25
Level 4	Precast cantilever beams	Combined actions on beam reinforcing: gravity, lateral ties, vertical acceleration	75
Level 4	Backstage area	Multiple tie locations: around shaft opening and connectivity to block wall, ties transferring loads to buttress and main walls	80
Level 3	Backstage area	Multiple tie locations: around shaft opening and connectivity to block wall, ties transferring loads to buttress and main walls	80
Level 2	Promenade slab on east and west sides	Chord tie capacity and continuity	20
Level 2	Lobby area	Insufficient continuity/capacity of non-ductile mesh reinforcing to provide adequate tie capacity	25
Level 2	Bridging slab between the lobby area and the sloping diaphragm	Tie capacity of the slabs connecting inner and outer ring adjacent to the sloping diaphragm	55
Level 2	Precast cantilever beams adjacent to double-tee flooring	Combined actions on beam reinforcing: gravity, lateral ties, vertical acceleration	55-70
Level 2	Backstage area	Gravity ties to bay walls also transferring load to the squat walls	75
Level 1.5	Connectivity to squat walls	Tie capacity to internal squat walls and bay walls	20
Level 1.5	Sloping transfer diaphragm and beams	Tie capacity of the sloping diaphragm, tie capacity to buttress piers, tie strength and connectivity to non-sloping slab	20-35

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C.4.2.3 Roof diaphragm

The roof of the Auditorium comprises Dycore flooring units spanning between steel trusses and topped with a 65mm concrete layer reinforced with 661-gauge and 663-gauge mesh. The Dycore floor units span in the longitudinal (north-south) direction of the building and the roof trusses in the transverse (east-west) direction of the building. Reference is made to Section 5.1.2.1 for a more detailed description of the roof build-up.

As the cold-drawn (non-ductile) mesh will not be able to provide any reliable tie capacity the typical micro strut and tie approach is not directly suitable. Aside from the cold-drawn (non-ductile) mesh, secondary load paths are found through the roof trusses, the chord stiffeners, saddle bars over the Dycore units, the cast insitu slab around the perimeter and splayed drag ties (3-H28 bars) from the precast buttresses.

Different variations of micro strut and tie models have therefore been assessed to study the bounds of the behaviour, assess the performance, and verify the sensitivity of the roof diaphragm. Reference is made to section C.3.4 for a more detailed description of the sensitivity studies. The strut and tie models, expected behaviour, structural weaknesses, relation to previous assessment, and expected performance will be discussed subsequently.

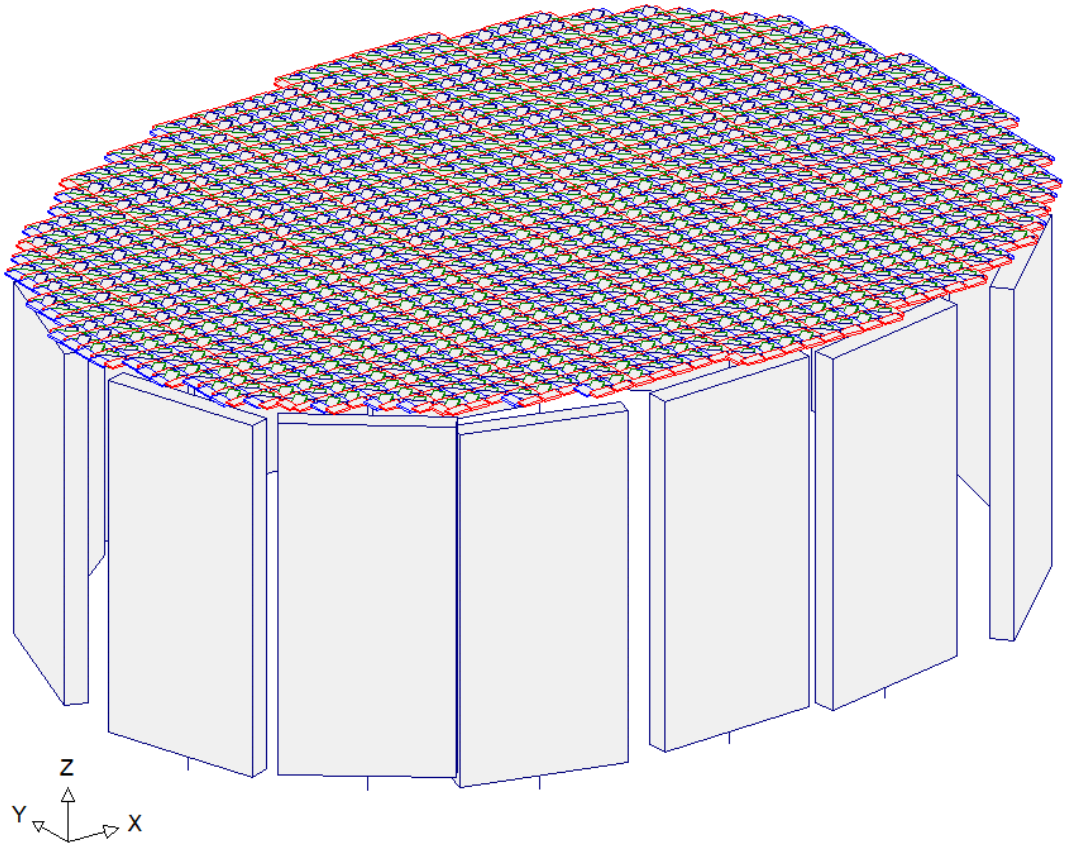

Figure C.4-15 : Strut and tie model of roof with primary buttress walls

Strut and tie results

As discussed in section C.3.4 the typical free body model with demands and support resultants extracted from the ETABS analysis resulted in a load path with high tie forces in the diaphragm, whilst there is no direct load path for these tie forces as the cold-drawn (non-ductile) mesh is considered to be fractured. To

further investigate the behaviour the Microstran strut and tie model has been upgraded to include the perimeter buttress walls as illustrated in Figure C.4-15.

The perimeter walls have been modelled as rectangular concrete sections matching the in- and out-of-plane bending stiffness of the primary buttress walls. In addition, the orthogonal members have had their stiffness reduced by a factor of 1000 to negate the contribution of ties reliant on the non-ductile mesh. Effectively this forces a strut mechanism while allowing for resolving of small residual forces.

Various alterations of these models have been investigated from which two will be addressed in this section. These models will be referred to as the roof model without trusses and the roof model with trusses. The roof model without trusses has the same 1000x reduced stiffness for all orthogonal members. The roof model with trusses has an effective stiffness of 1.41 times the diagonal strut stiffness for the orthogonal members representing the roof trusses and the same 1000x reduced stiffness for all other orthogonal members. It is important to note that the ratio between the stiffness of the roof struts and the stiffness of the steel trusses has been estimated and bounded. The purpose of these models is to investigate the roof behaviour with and without the roof trusses and provide a bounded.

A limitation of the above-mentioned approach is that the reduction of the orthogonal stiffness also negates formation of orthogonal struts whereas, in reality, these could form. However, it is noted that the roof is expected to have a tendency to strut outwards to transfer the lateral demands towards the stiffer walls aligning with the direction of loading and using the stiff direction of the other walls to resist the tie forces.

The strut and tie demands for the roof model without the trusses have been illustrated in Figure C.4-16. The struts, running towards the bay walls, for both x and y direction can be clearly identified. The tie forces are comparably small as they are negligible on the inside of the diaphragm and increase in demand towards the outer edges for equilibrium and model stability purposes. Due to the absence of ties the bay walls have to bend in-plane and out-of-plane to resist the resultant tie forces. This leads to large out-of-plane deformations of the bay walls.

The strut and tie demands for the roof models with truss stiffness included have been illustrated in Figure C.4-17. The struts forming in the x-direction are almost identical to struts forming in the model without the trusses. The main difference is the vertical compression struts forming on the left side of the diaphragm resisting the 'closing' of the egg-shaped roof. The struts in the z-direction differ significantly. The struts are primarily running towards the trusses. From there, they are transferred by compression and tension to respectively the top and bottom of the trusses. They are either transferred directly into the bay walls on top or are transferred down through tension in the trusses and back into the roof where struts are formed running towards the relatively stiff bay walls aligning with the direction of loading (Bay1/1a and Bay 7/7a). As the connection of the roof trusses only has a nominal

The tension results are similar to the model without the trusses with negligible tension forces on the inside and increasing demands on the outside. Naturally, the main difference is the formation of vertical ties on the right side for the x-direction and the trusses acting as tension ties in the z-direction to transfer the lateral directly into the walls located on the bottom (Bay 3a -5a) or back into the roof to strut into the walls on the side as discussed earlier.

The model without trusses sees strut demands of about 500kN. For the model with trusses this increases to about 1,000kN in the diagonal struts forming at the bottom. The trusses in the model with trusses see tension demands in the order of magnitude of 2,000kN.

Expected Behaviour

The true behaviour is expected to lie between the models presented above. The roof diaphragm is expected to be able to rely largely on a compression load path with struts forming towards the bay walls. However,

the roof cannot freely crack to develop this mechanism as it is restrained by the roof trusses acting as ties and connecting the walls at the top and bottom. Given the connections between the roof trusses and the diaphragm are rather limited in number (and expected strength and stiffness) it is expected the diaphragm will crack at the roof trusses (at the ends of the Dycore units), effectively the 'egg'-shape of the roof opens up along the trusses and subsequently pushing the behaviour more towards the model without the trusses (i.e. a flexible diaphragm).

The primary risk associated with this behaviour is sufficient damage to or displacement of the Dycore flooring resulting in the dropping of (parts) of the Dycore flooring. Importantly the Dycore units will have some resilience against the life safety hazard of falling due to the increased seating to the floor units as has been improved during the seismic strengthening works described in Section 5.2.5.

Structural Weaknesses

To following structural weaknesses have been identified and will be discussed in this section. These weaknesses are considered to limit the acceptable outcome of the diaphragm performance.

- Exceedance of the axial load capacity of the roof trusses,
- Excessive stretching of the diaphragm, potentially leading to loss of seating of the Dycore units
- Excessive out-of-plane loads on the bay walls, resulting in large deformation throughout the building, also on other floor levels.

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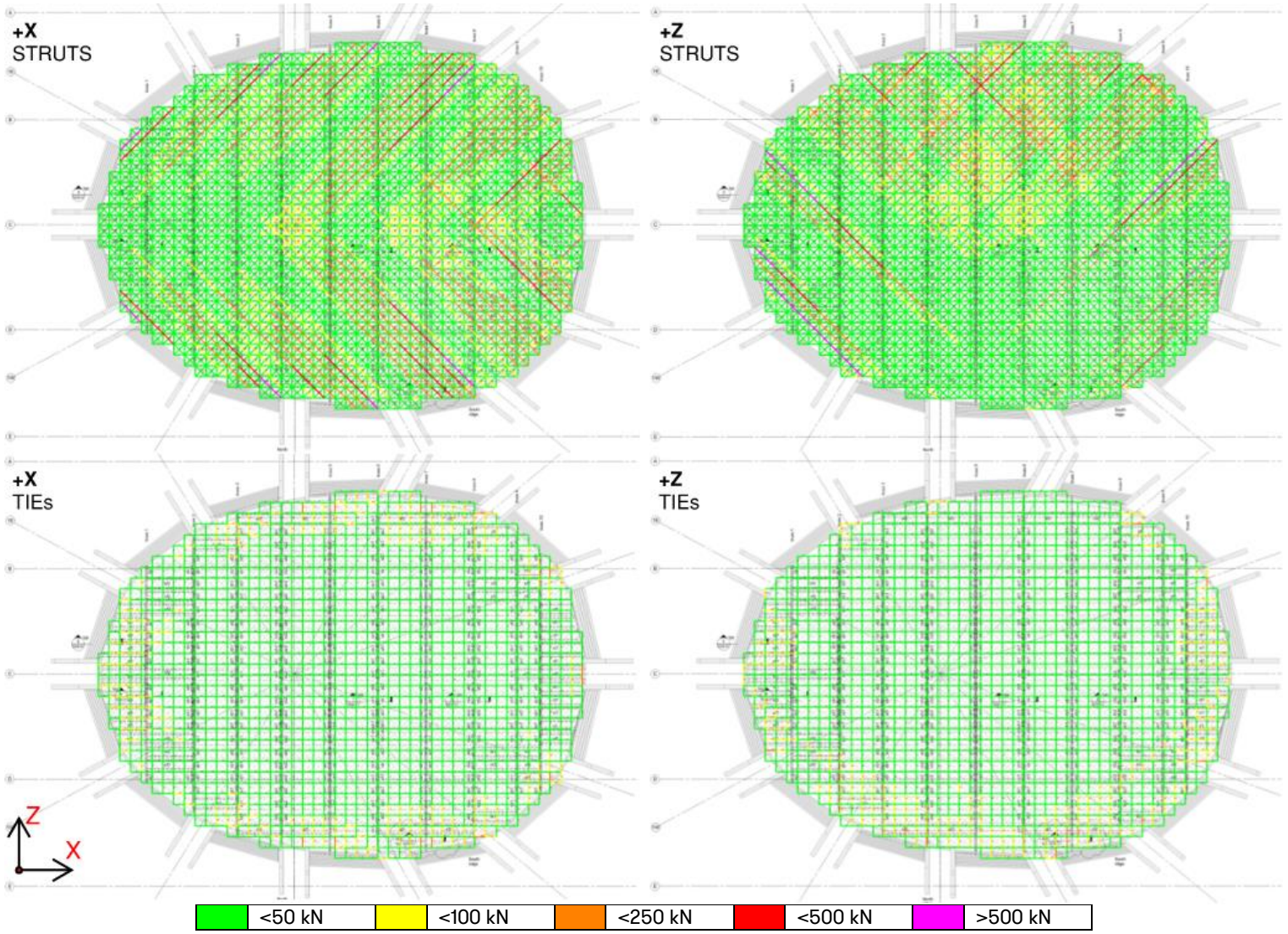

Figure C.4-16: Strut and tie demands for roof model without trusses

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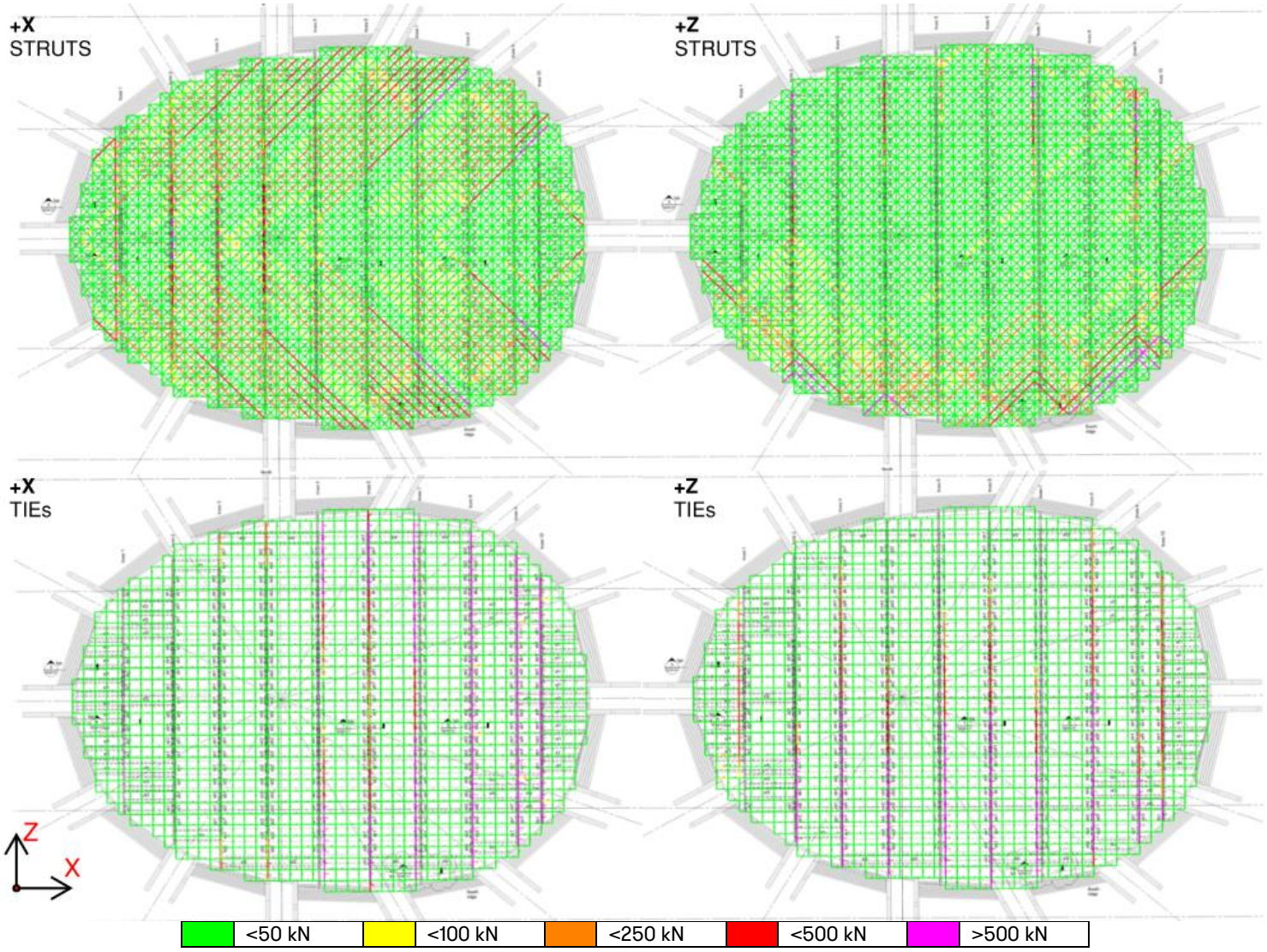

Figure C.4-17: Strut and tie demands for roof model with trusses

Relation to previous assessment

Section 6.4 describes some of the recent high-level assessments that were undertaken in 2018 and early 2022 on the capacity of the roof diaphragm. Through micro and macro strut and tie analysis of the roof diaphragm, the high-level assessments have attempted to demonstrate that the roof diaphragm could be rated above 34%NBS_{IL3}. However, these analyses have been undertaken in isolation, only considered the inertia of the roof itself and did not consider the global behaviour of the structure. These have now been modelled to establish the compatibility forces. Also, the previous analyses relied on assumptions associated with the capacity of the cold-drawn (non-ductile) mesh, the roof trusses or the rigidity of the perimeter buttress walls from the 2014 DSA to justify the simplifications made in the analysis.

The assessment of the roof diaphragm for this DSA used the output of the global behaviour as a starting point and supplemented this with more detailed understanding of the roof trusses (and their connections) and the perimeter buttress wall systems. In general, the assessment of the roof diaphragm identified three potential structural weaknesses:

- Exceedance of axial loads in the top chords of the roof trusses, especially at the connections with the precast concrete wall panels. The roof trusses could be relied upon to provide a tie capacity, however, there is a limit to that capacity, especially if the system relies on the capability of the roof trusses to transfer any loads in the wall panels.

- Excessive 'stretch' of the diaphragm as it tries to 'open up' from the flexural behaviour – relying on the perimeter buttress walls' in-plane strength to form a tie. There is some tolerance to allow strains in the topping, however this should be limited to ensure the Dycore floor units don't lose their seating and the chord stiffeners do not exceed the total displacement capacity within the bolted connection (ply bearing and bolt slip).

- Excessive out-of-plane loads on the bay walls (in-plane for the buttresses) as this would likely lead to other load path related issues down the walls. As the diaphragm would crack and soften, there would be more reliance on the precast buttresses to provide restraint. However, this is paired with large displacements which would result in additional demands imposed on the Level 4 promenade floor. Also, there would be excessive tie forces in the Level 2 and Level 1/2 diaphragms to try and hold the system together.

Exceedance of the axial loads in the roof trusses

As discussed in this section the roof trusses could act as struts and ties. There are three risks associated with this interface. The highest risk is associated with failure of the connection between the truss and the bay wall as this is a brittle failure mode and would result in local collapse of the roof. Given the lateral mechanism of the roof relies on the roof remaining intact this would likely result in the progressive collapse of the roof. Secondly, the axial load in the top chord could be exceeded. And lastly, the connection between the Dycore and the roof can lose capacity. This connection consists of welded cleats with a 20mm saddle bar at 1200mm spacings. Given the roof is not a flat diaphragm this connection is also affected by a strut passing over these changes in angle resulting in a local uplift. This is somewhat offset by the gravity demand but an uplift up to 30% is expected based on roof angle.

Excessive stretching of the diaphragm

The diaphragm will exhibit excessive displacement and stretching as it tries to 'open up' from the flexural behaviour. There is some capacity to allow some strains in the topping, however this should be limited to ensure the Dycore floor units don't lose their seating and the chord stiffeners do not exceed their total displacement capacity from slip and ply bearing in the bolted connections.

Excessive out-of-plane loads on the bay walls

Due to the lack of tie capacity the strut resultants could induce excessive out-of-plane loads on the bay walls, in-plane loading for the buttresses. This in turn could lead to other load path related issues down the building. Additional demands would be imposed on the floors below inducing excessive tie forces in the lower-level diaphragms trying to hold the system together.

Expected Performance

The assessment has considered various sensitivity studies in the analysis of the roof diaphragm. However, we have been unable to demonstrate a probable scenario, which accounts for all these three potential structural weaknesses achieving a seismic score above 34%NBS_{IL3} without engaging a tie capacity in the cold-drawn (non-ductile) mesh. On the other hand, there are load paths available, meaning that a sudden loss of capacity is unlikely.

Based on the expected behaviour and the structural weaknesses discussed above the diaphragm is expected to have a rating of **20-30%NBS_{IL3}**. There is potential for an increased rating due to a better-than-expected performance which cannot accurately be captured with the adopted approach. However, a rating of >34%NBS_{IL3} could not be justified.

C.4.2.4 Level 5 – Director's area

The diaphragm at Level 5 is a 150 thick RC slab supported on precast beams cantilevering from the bay walls of Bays 7(a). The slab is reinforced with mild steel bars of varying size, spacing and orientation. Typical bar diameters vary from 12 – 20mm and spacings from 175 – 450mm.

This floor was assessed using a micro strut and tie model. The orthogonal members of the model aligning with the beams in the floor had their stiffness increased by a factor of ~3, based on the members' relative axial stiffness to the slab. Stress in the reinforcement from gravity and vertical acceleration loading was included in the assessment. Where appropriate, a 1.5 collector load factor was included in the member diaphragm demand.

The diaphragm at level 5 is rated at **85% NBS_{IL3}**, where gravity load paths to the stair landing may be compromised. An overview of the potential structural weaknesses and their ratings are shown in Figure C.4-18 below. Figure C.4-20 shows the enveloped strut and tie demands. These enveloped results include both positive and negative directions and all X and Z concurrency combinations.

The strut and tie demands are illustrated in Figure C.4-21 through Figure C.4-22 for the primary floor accelerations in the positive direction. For the building longitudinal (X) direction, the struts are formed around the stairs towards the beams and the buttresses of the bay walls. The largest tie tries to form between the stair openings as a chord. For the building transverse (Z) direction, a typical cantilever diaphragm forms with diagonal struts forming to the bay walls and equilibrium ties within the beams.

For loading in the X-direction, a secondary load path was relied upon. The chord force is unable to be restrained by the existing reinforcing and therefore Bay 7 and Bay 7a will act more independently in this direction. See Figure C.4-19 for a diagram of this secondary load path and the ties checked. The negated, primary ties are shown as grey in Figure C.4-18.

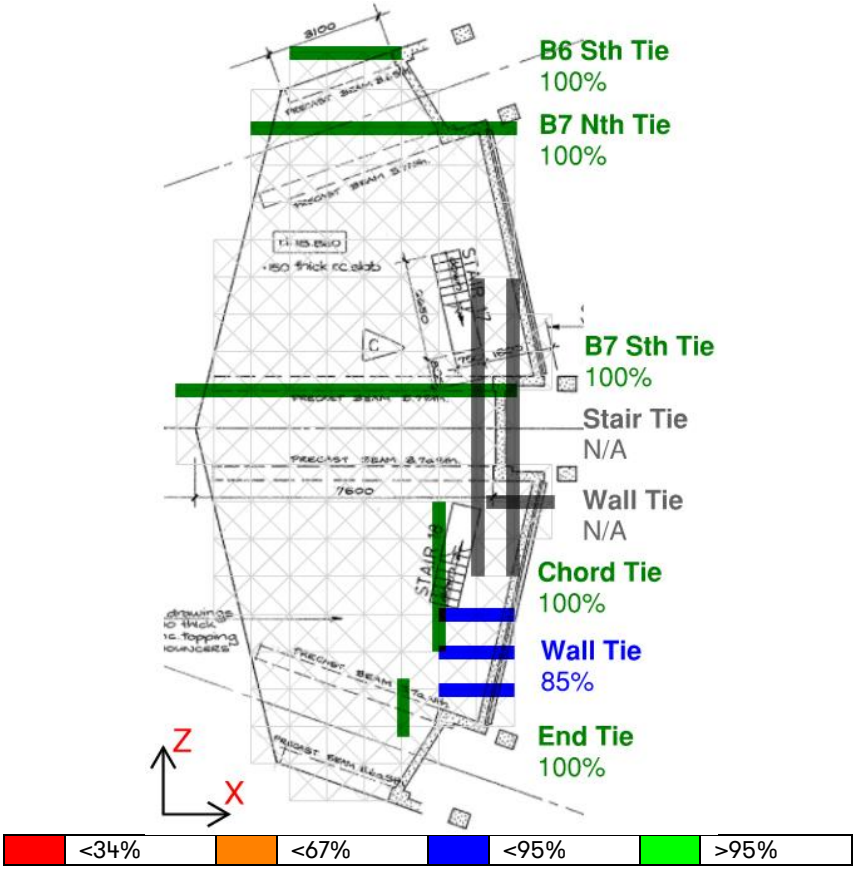

Figure C.4-18: Summary of Level 5 diaphragm score

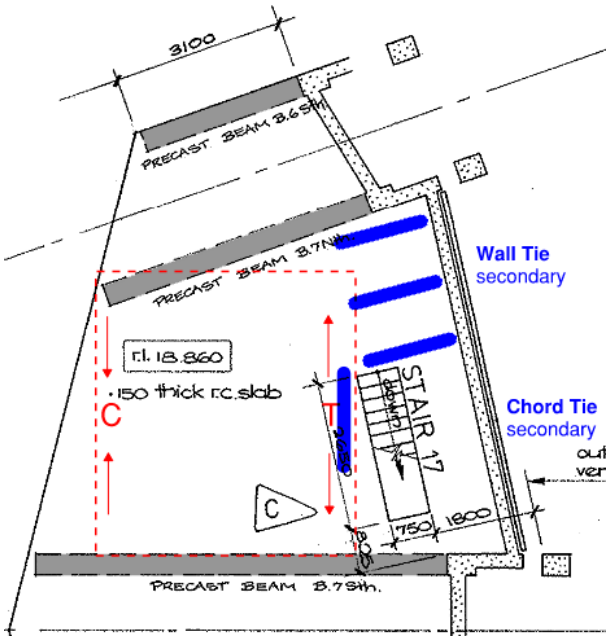

Figure C.4-19: X-direction secondary diaphragm load path and checked ties

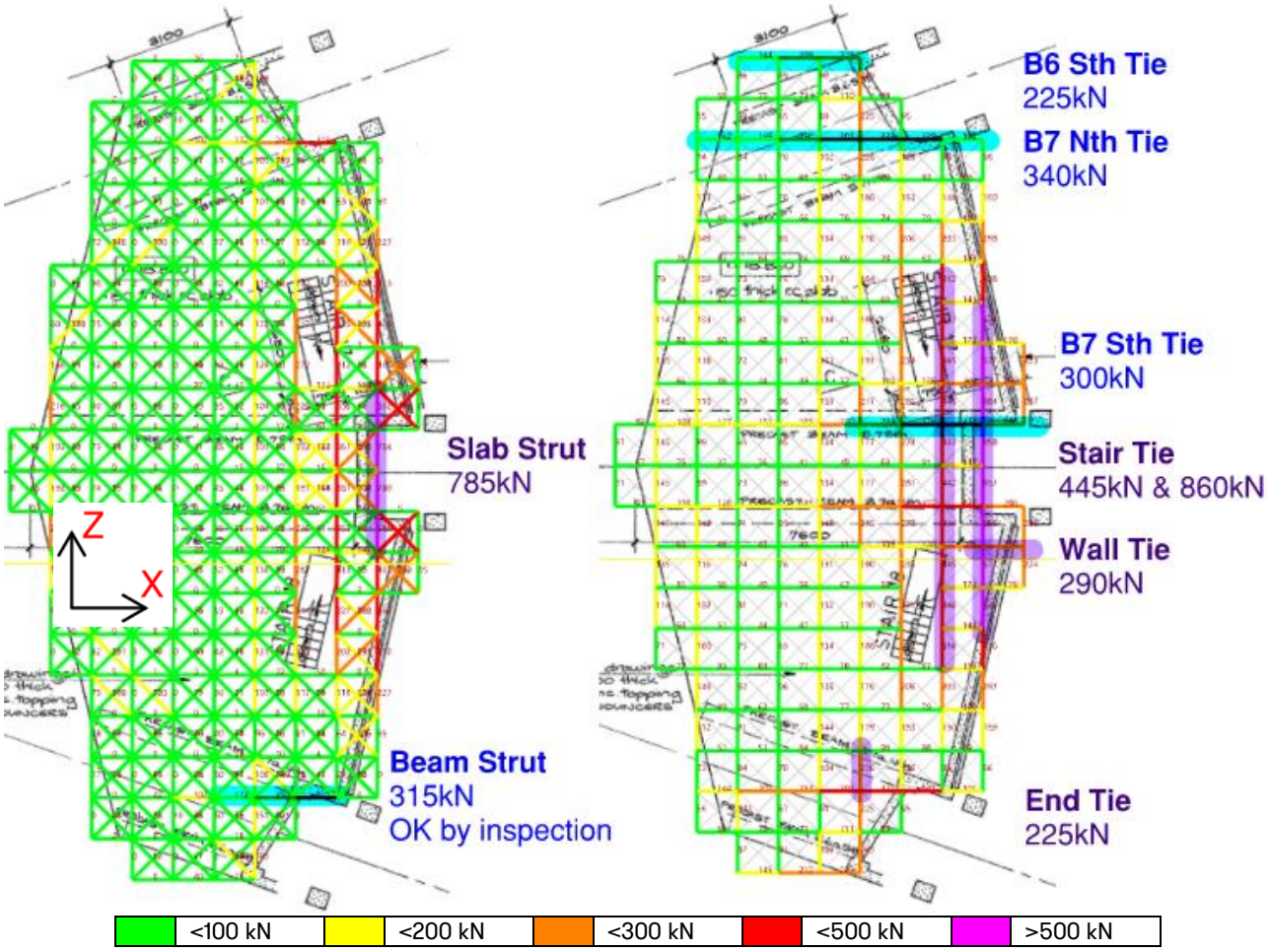

Figure C.4-20: Enveloped primary strut (left) and tie demands for the Level 5 diaphragm

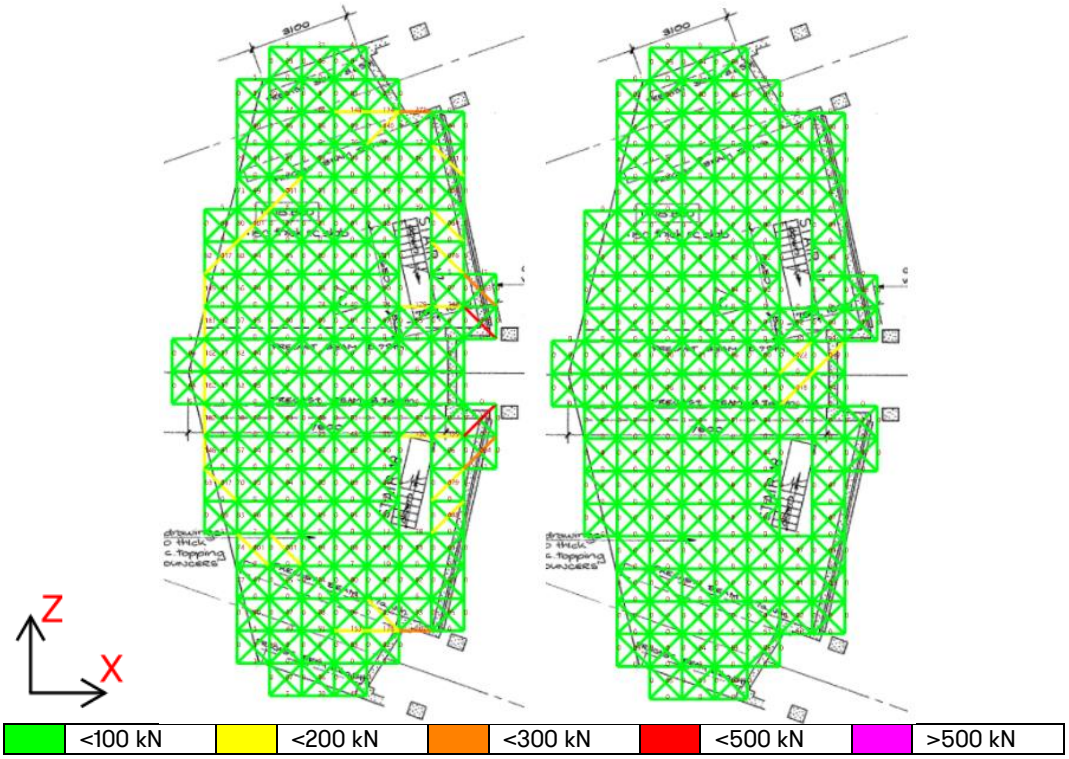

Figure C.4-21: Primary strut magnitudes for the L5 +X (left) and +Z (right) load case

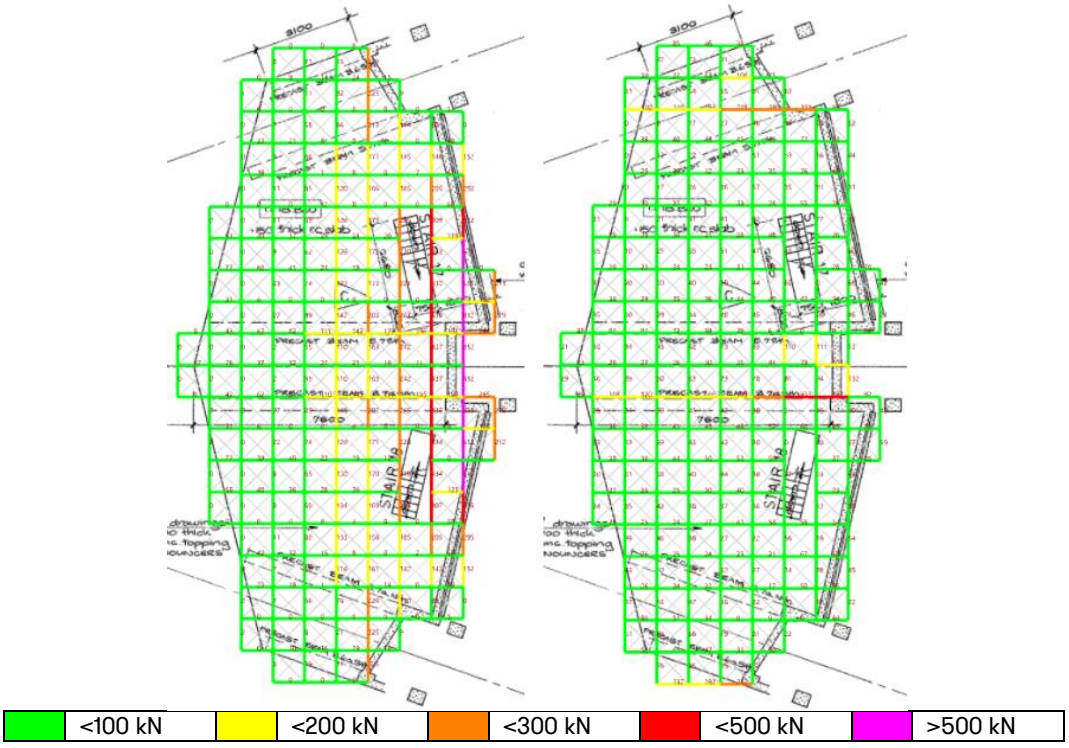

Figure C.4-22: Primary tie demands for the L5 +X (left) and +Z (right) load case

C.4.2.5 Level 4

The assessment for Level 4 is split into two parts. A micro strut and tie assessment for the backstage area and a macro strut and tie assessment for the individual bleacher wedge segments. The assessment of the backstage area is the same conclusion as in Level 3; the two slabs have the same reinforcing and a higher acceleration present at Level 3. The assessment of the promenade is presented in the following subsection.

C.4.2.5.1 Level 4 – Promenade Segments

At Level 4 there is a ring of seating on the inside of the Auditorium. The seating consists of a stepped seating floor with a thickness ranging between 100mm and 200mm and mild steel bar reinforcing. The outside of the ring consists of 450mm deep double-tee units with a 50mm concrete topping – this forms the promenade at this floor level.

The topping of the double-tees is reinforced with cold-drawn (non-ductile) mesh reinforcing, which is considered to be fractured. The stepped seating floor is reinforced with a combination of deformed bars and mesh reinforcing. All slabs are supported by and connected to the bay walls by the Precast Cantilever Beams. For additional information on the built-up refer to Section 5.1.1.4.

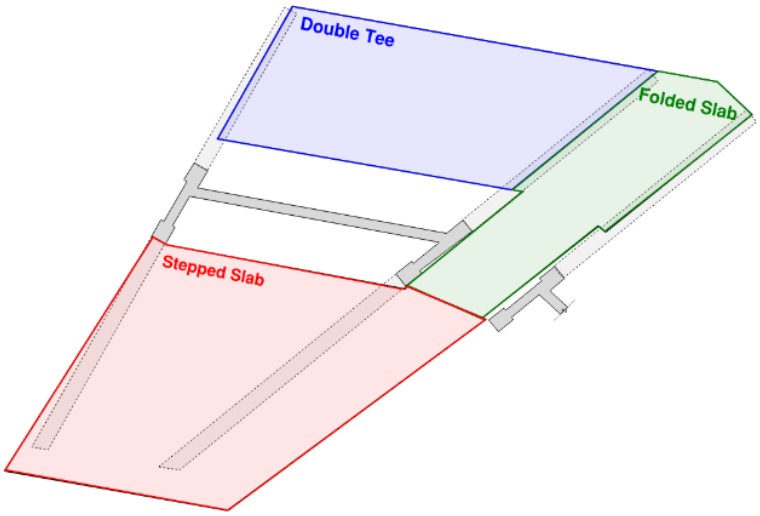

Figure C.4-23: Overview of different floor sections for a single segment.

The segments at Level 4 are scored at **25%NBS$_{IL3}$** and are limited by the double-tee floors which are not adequately tied back to the support beams. The primary tie mechanism is formed in the cold-drawn (non-ductile) mesh reinforcing in the topping, which is considered to be fractured in the assessment. Therefore, the system needs to rely on less conventional ways to transfer the load into the supporting members – such as friction underneath the web areas of the double-tees.

An overview of the potential structural weaknesses and their ratings has been included in Figure C.4-24 below. These will be discussed in more detail later in this section. Typical results presented are for the floor accelerations from the NLTHA (Appendix D), whilst the friction check has been based on loading from 'Parts and Components' of Chapter 8 of NZS1170.5:2004.

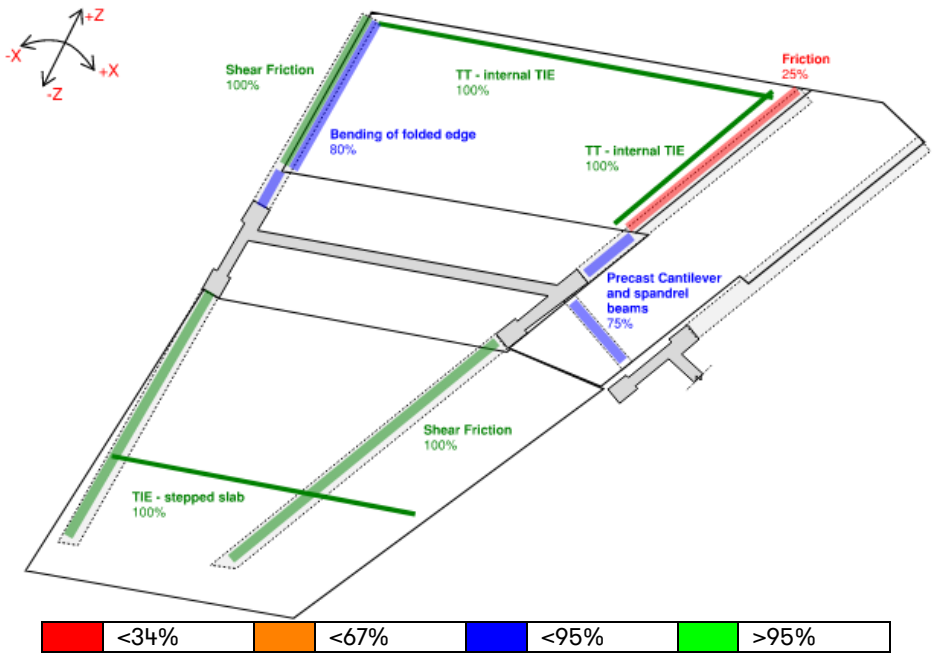

Figure C.4-24: Summary of the Level 4 segment ratings

Macro Strut and Tie

The floor segments have been assessed using a macro strut and tie analysis. Given the Level 4 segments rotate around the building they have been assessed for their radial and circumferential direction. The macro strut and tie for the positive radial and positive circumferential direction have been illustrated in Figure C.4-25 and Figure C.4-26 below. The negative directions loadings are similar.

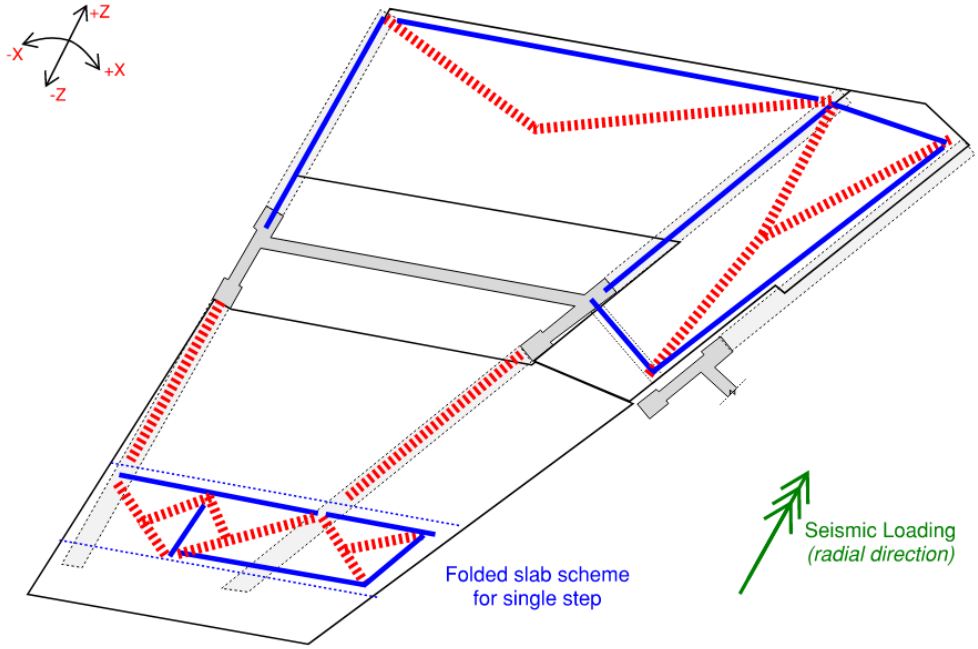

Figure C.4-25: Macro strut and tie for positive radial direction

For the double-tee units in the positive radial direction, struts are formed towards the corner edges of the double-tee unit floors. Which are tied back into the support buttress wall by the precast cantilever beams acting as ties. An internal tie is formed between the struts in the concrete topping and the strands in the web of the double-tee units.

The stepped seating floor cannot form a strut directly to the supporting bay walls as these struts would need to pass through the risers. However, each step can span to the supporting beams as indicatively indicated in Figure C.4-25. From here the beams can strut into the supporting wall. The right side of the slab shows a cantilever-like mechanism, however both support beams will be loaded in compression.

The stepped seating floor is only laterally supported on one of its long sides. In addition, it is supported by the spandrel connected on the right side of the buttress walls. The slab will form a typical cantilever strut and tie scheme in the positive direction. The horizontal resultant in the top left corner will be resisted through a compression strut in the double-tee unit. Rotation in the negative direction will be resisted through a strut in the stepped seating floor as the top left corner is not able to form a tension tie as the non-ductile mesh cannot be relied on through the different floor interfaces.

For the double-tee units in the positive circumferential direction a typical cantilever strut and tie scheme is formed, refer Figure C.4-26. The interior stepped slab can form a similar strut and tie scheme at each horizontal and vertical plane. The horizontal resultants of the panel struts are primarily resisted by out-of-plane bending of the Precast Cantilever Beams and supplemented by the tie in the spandrel beam. The assessment includes gravity and vertical acceleration demands on the cantilever beams, along with 1.5 factor on collector demands where appropriate.

This lateral mechanism transfers the demands to the support beams and buttress walls, loading them in compression and tension to resist the moment induced by the cantilever action.

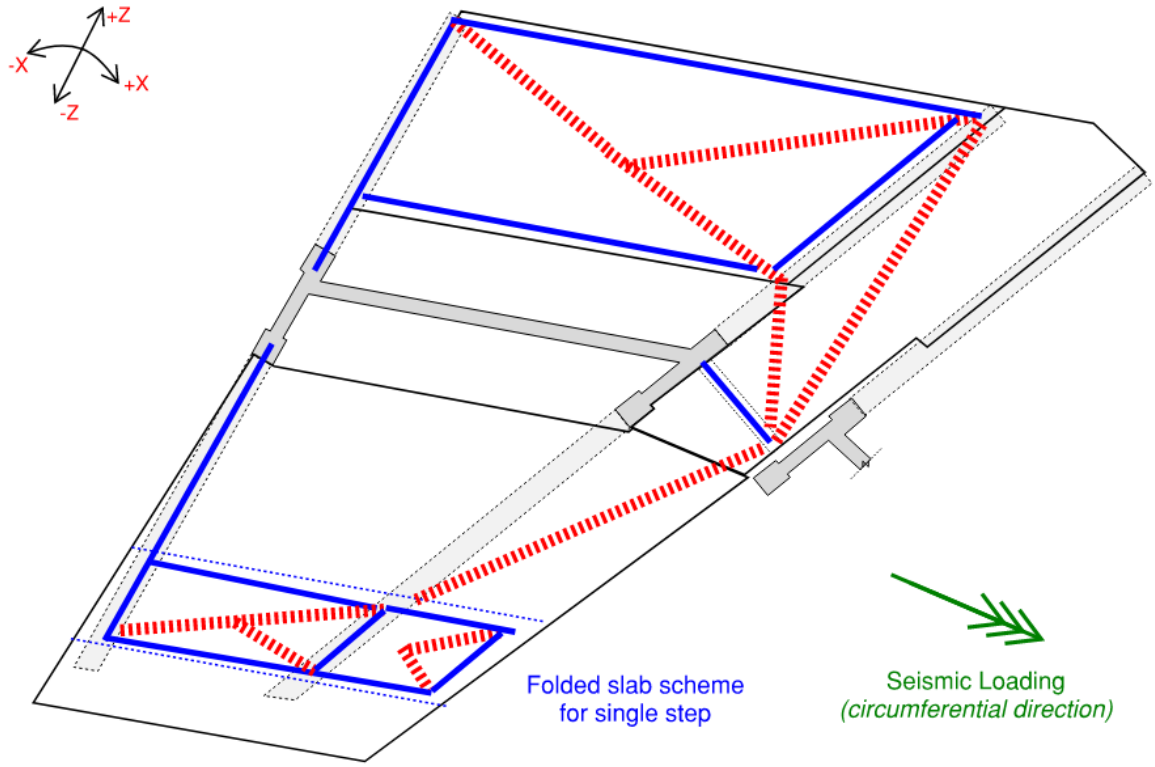

Figure C.4-26: Macro strut and tie for positive circumferential direction

Potential structural weaknesses

Struts have not been considered as potential structural weaknesses as there are no large transfer forces associated with the strut and tie mechanisms presented.

The main structural weakness for the double-tee unit occurs in the negative circumferential direction as there is no valid load path (apart from friction) to resist the tie force at the middle beam. The only continuous reinforcement at the double-tee units to the stepped seating is the non-ductile mesh which cannot be relied on. The following potential structural weaknesses can be identified for the diaphragm over the double-tee unit floor.

- Bending failure of the folded edge of the double-tee unit
- Internal tie demands of the double-tee unit floor system
- Tie capacity to resist rotation in negative circumferential direction (friction only)
- Combined actions on the Precast Cantilever Beams

The primary structural weakness for the stepped slab is the connection between the slab and the Precast Cantilever beams. Secondly, the longitudinal bars running parallel to the span of the stepped slab are activated by both the horizontal seismic demand and vertical gravity demand and could limit the performance of part of the floor.

- Connection between stepped slab and Precast Cantilever beams
- Horizontal and vertical bending of the steps slab

The stepped seating floor is considered to not have structural weaknesses that govern over the weaknesses of the other floor types. The slab is both relatively small and has, comparably strong, compression supported load paths through the other floors.

C.4.2.6 Level 3 – Backstage area

The diaphragm at Level 3 is a 225mm thick floor with local areas having a thickness of 400mm. The floor is reinforced with mild steel bars of varying size, spacing and orientation. Typical bar diameters vary from 12 – 16mm and spacings from 150– 450mm.

This floor has been assessed using a micro strut and tie model. Stresses in the reinforcement from gravity loading have been included in the assessment.

The diaphragm at Level 3 (and Level 4) is rated at **80%NBS$_{IL3}$**, when multiple gravity and lateral load paths become compromised. Local redistribution of forces was utilized to determine the life safety hazard, the ratings presented include this approach while the strut and tie images illustrate the initial load paths. An overview of the potential structural weaknesses and their ratings is included in Figure C.4-27 below. Figure C.4-28 shows the enveloped strut and tie demands. These enveloped results include both positive and negative directions and all X and Z concurrency combinations.

The strut and tie demands have been illustrated in Figure C.4-29 and Figure C.4-30 for the primary floor accelerations in positive direction. These demands illustrate the strut and tie load paths. For the X-direction struts and their associated ties are formed diagonally towards the bay walls of Bays 2(a). This is less clear from the snip due to most struts being <100 kN. The other primary strut and tie pair occurs as chord forces develop at the opening between Bays 1(a) because of the tendency of the floor to fold around this point. For the Z-direction the floor is pushed clockwise around the building. Struts are formed towards the bay walls, primarily at Bay 1(a) due to their alignment with the load direction. Ties are primarily formed at the bottom end of the diaphragm, resisting the diaphragm tearing away from the walls as a result of the clockwise rotation.

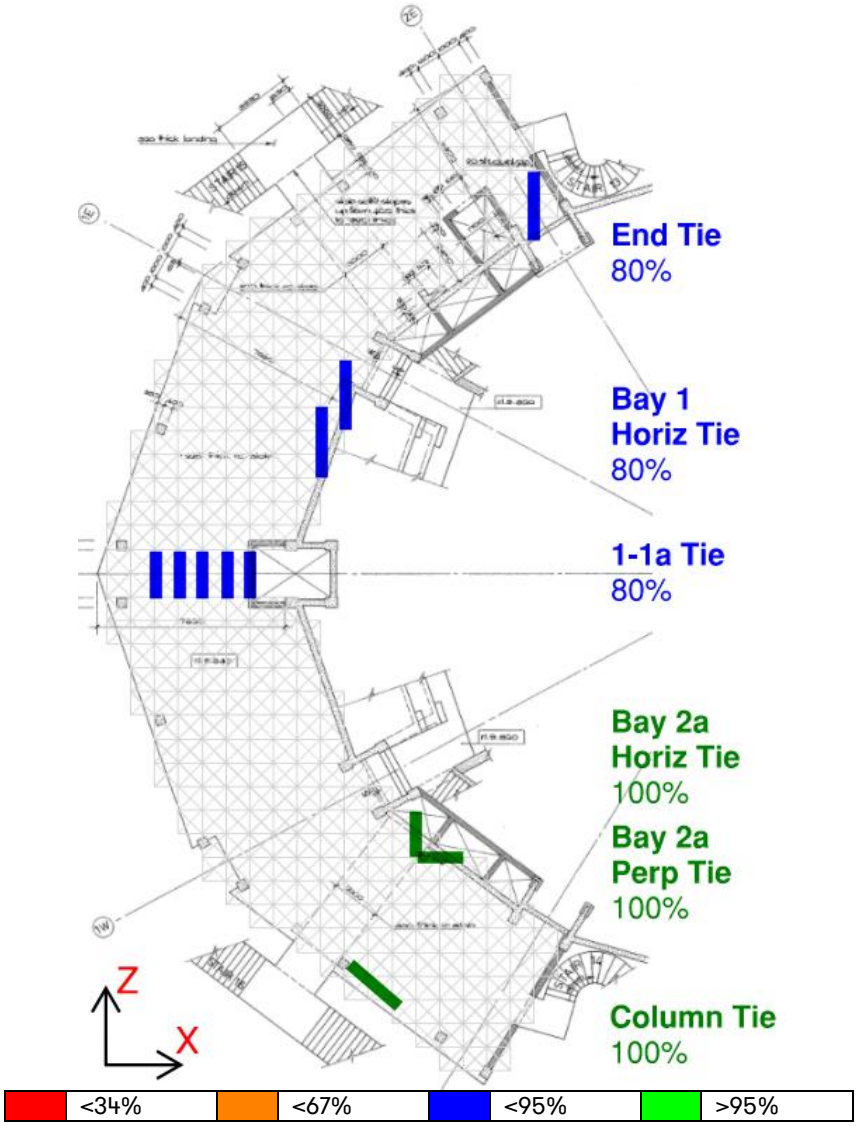

Figure C.4-27: Summary of Level 3 diaphragm rating

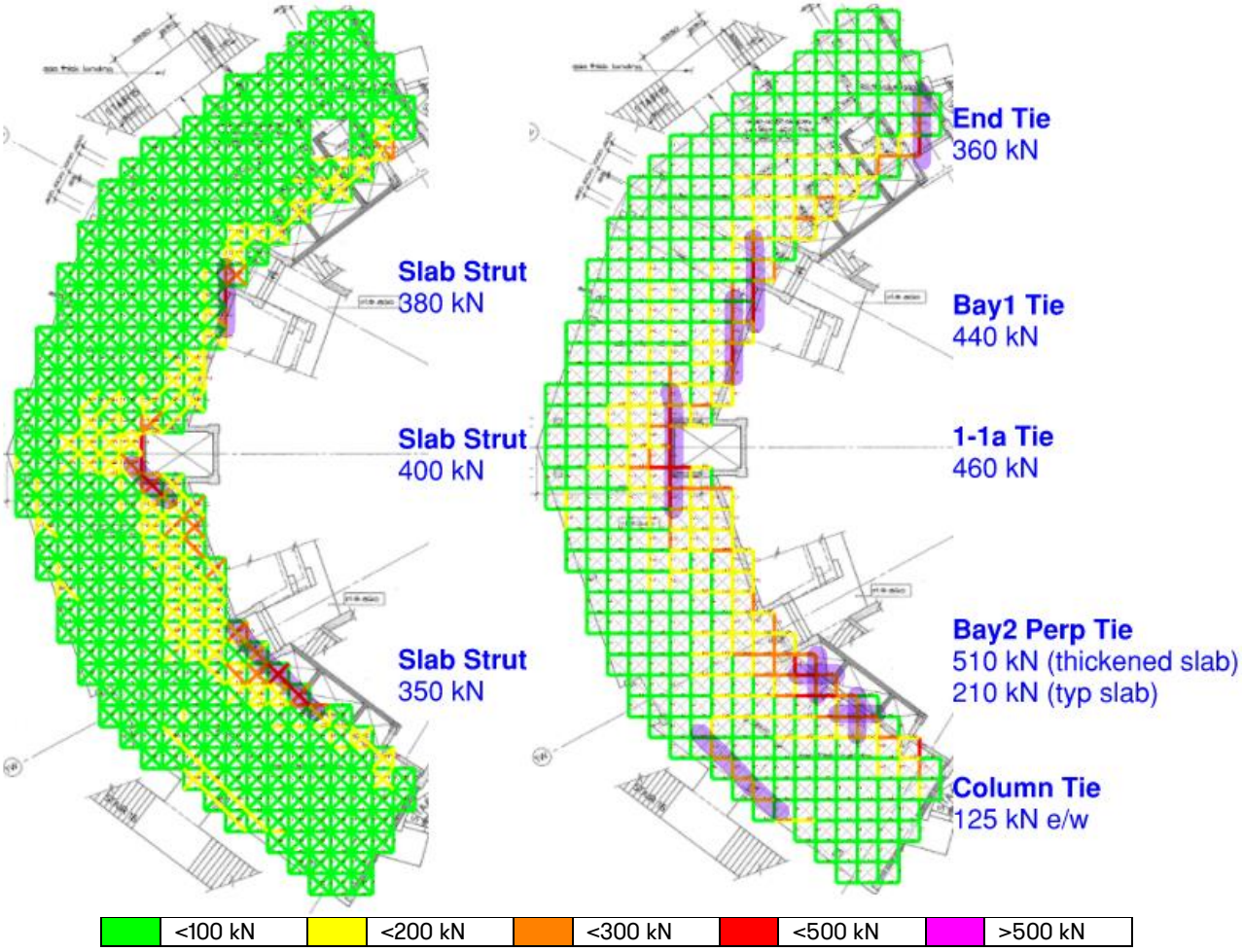

Figure C.4-28: Enveloped strut (left) and tie (right) demands for the Level 3 diaphragm

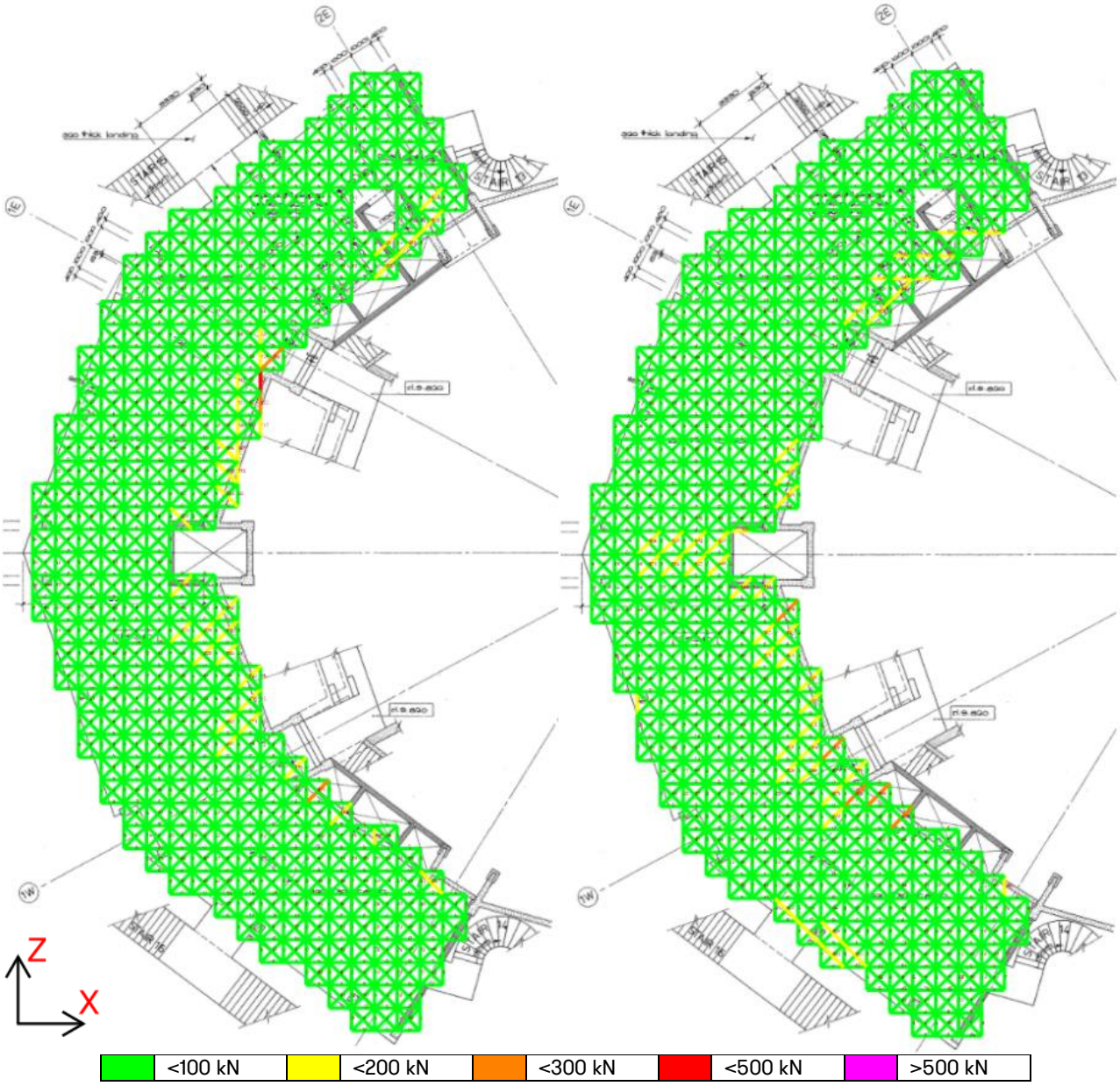

Figure C.4-29: Strut demands for the Level 3 -X (left) and +Z (right) load case

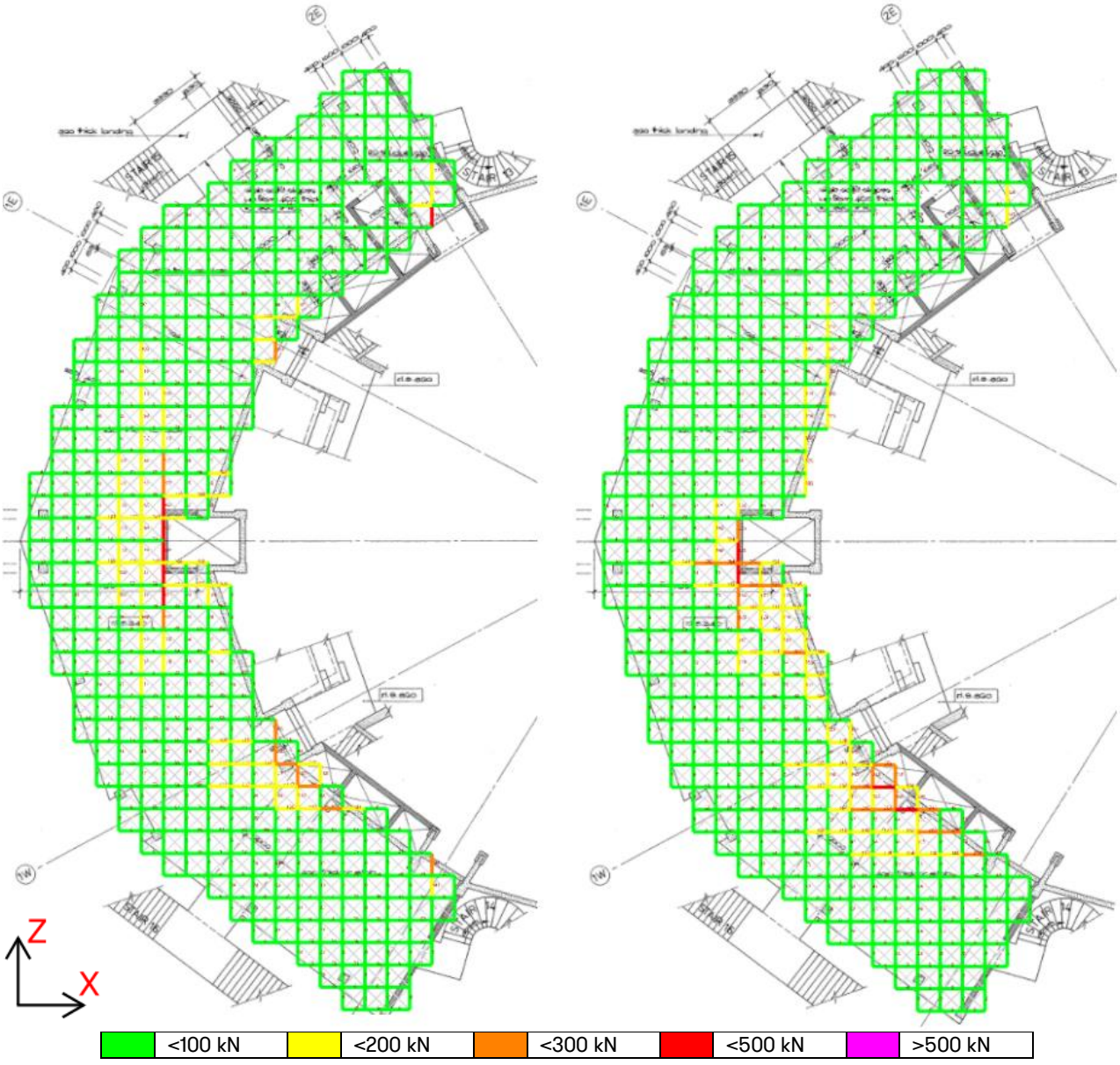

Figure C.4-30: Tie demands for the Level 3 -X (left) and +Z (right) load case

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C.4.2.7 Level 2

The primary element and function of the diaphragm within the ring of the bay walls at Level 2 is to act as a transfer diaphragm. The four bay walls at Bays 6(a) and 7(a) are supported on columns below Level 2. The lateral force present in these walls is transferred down through the sloping diaphragm into the walls at Level 1/2. The outer ring of the diaphragm at Level 2 consists primarily of a 200mm to 225mm thick floor. This is reinforced with mild steel bars of varying size, spacing and orientation. A few segments of the outer ring on the right side consist of double-tee unit floors with non-ductile mesh reinforcing.

The assessment for Level 2 is split into two parts. A micro strut and tie assessment of the full floor plate including the sloping diaphragm and a macro strut and tie assessment of the promenade on the southern end. These have been presented subsequently in the following paragraphs.

C.4.2.7.1 Level 2 – Global

The Level 2 diaphragm was assessed using a micro strut and tie model. The model was cut horizontally at the sloping diaphragm. The internal shear forces at the cut were applied to the model in addition to the floor demands and support resultants. The internal shear forces have been extracted from the ETABS analysis using a section cut.

The enveloped strut and tie demands are included in Figure C.4-32 and Figure C.4-33 below. These enveloped results include both positive and negative directions and all X and Z concurrency combinations. The strut and tie demands are illustrated in Figure C.4-34 for the individual primary orthogonal load cases in positive direction.

The sloping diaphragm between Level 2 and Level 1/2 is scored at **20%NBS$_{IL3}$** and is limited by the tie capacity of the sloping transfer diaphragm. The sloping transfer diaphragm is discussed in additional detail in paragraph C.4.2.7.2 covering the Level 1/2 assessment.

Promenade floor

The diaphragm at Level 2 is limited to **20%NBS$_{IL3}$** by the chord tie capacity at the east and west faces of the promenade. A detailed overview of the potential structural weaknesses and their ratings is included in Figure C.4-31. The critical ties are labelled as 'Edge Horizontal'.

In the transverse (Z) direction, the analysis model shows that the Auditorium wants to act as a deep beam, bending between the walls at Bays 1(a) and the squat walls 16, 18 and 21. The system relies on the cast insitu slab around the promenade at Level 2 to restrain that as a large tie. This is evident in the image on the bottom right of Figure C.4-34. Due to the plan eccentricity (the discontinuity of the bay walls at Bays 6(a) and 7(a), there is an additional tie demand on this area.

The promenade has significant tie capacity across Bays 3(a) and 4(a) where they are a series of at least 14-H32 diameter bars of the higher reinforcing grade, which provide the tie capacity. Further to the south, across Bays 5(a), the reinforcing reduces significantly to a series of Ø24 bars of the lower reinforcing grade. These provide a much lower tie capacity and this transition area is considered to be the governing structural weakness of the Level 2 diaphragm.

Backstage area

Some ties in the Backstage area are coloured as purple. These demands occur from shear transferring from the Bay Walls to Wall 1 and 6 through rigid diaphragm behaviour. This load path will begin to soften at the percentages noted but do not indicate a Life Safety hazard, and therefore do not control the seismic rating. The bay walls have sufficient strength to maintain all of the shear demand without diaphragm distribution.

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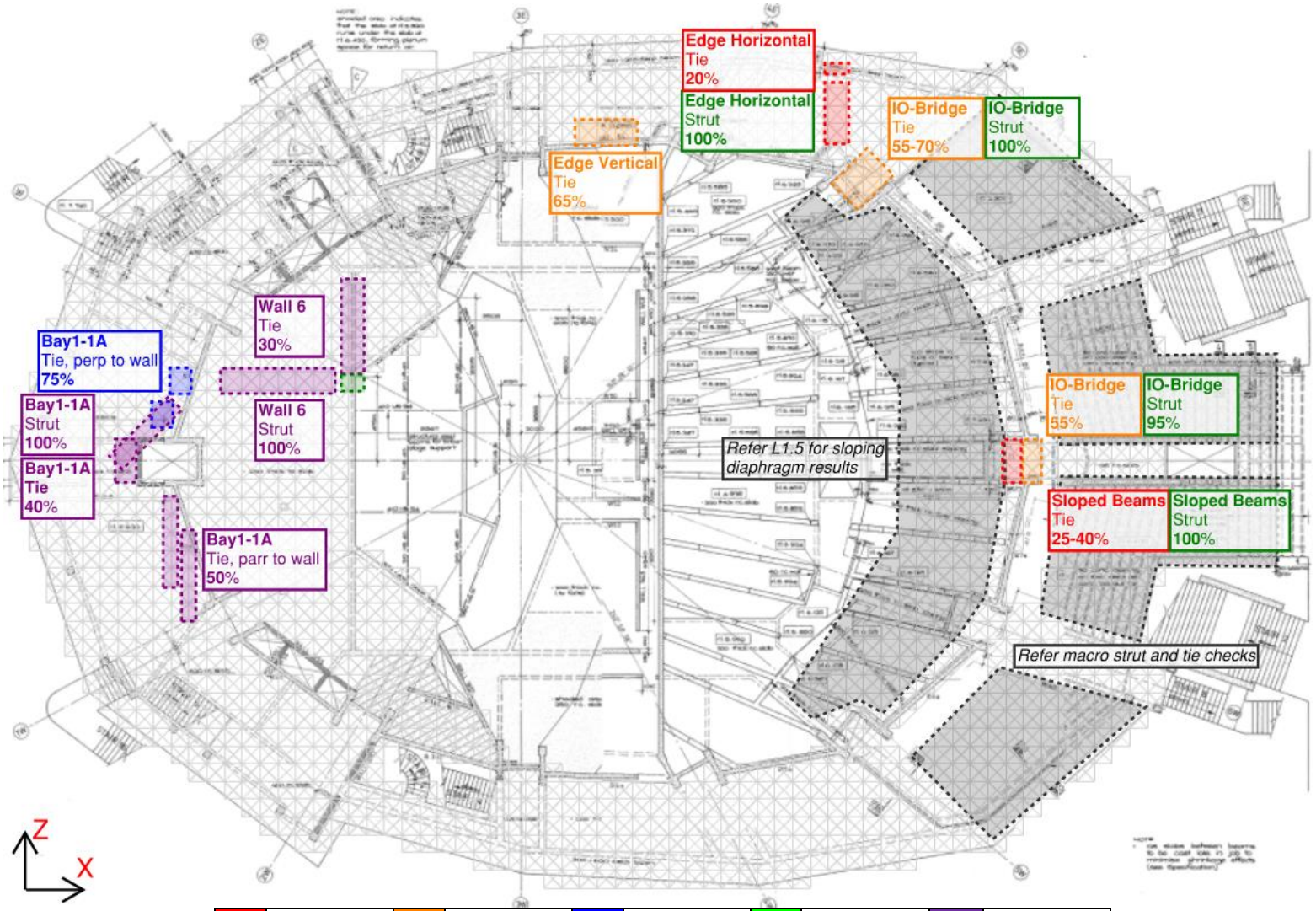

Figure C.4-31: Summary of Level 2 diaphragm rating

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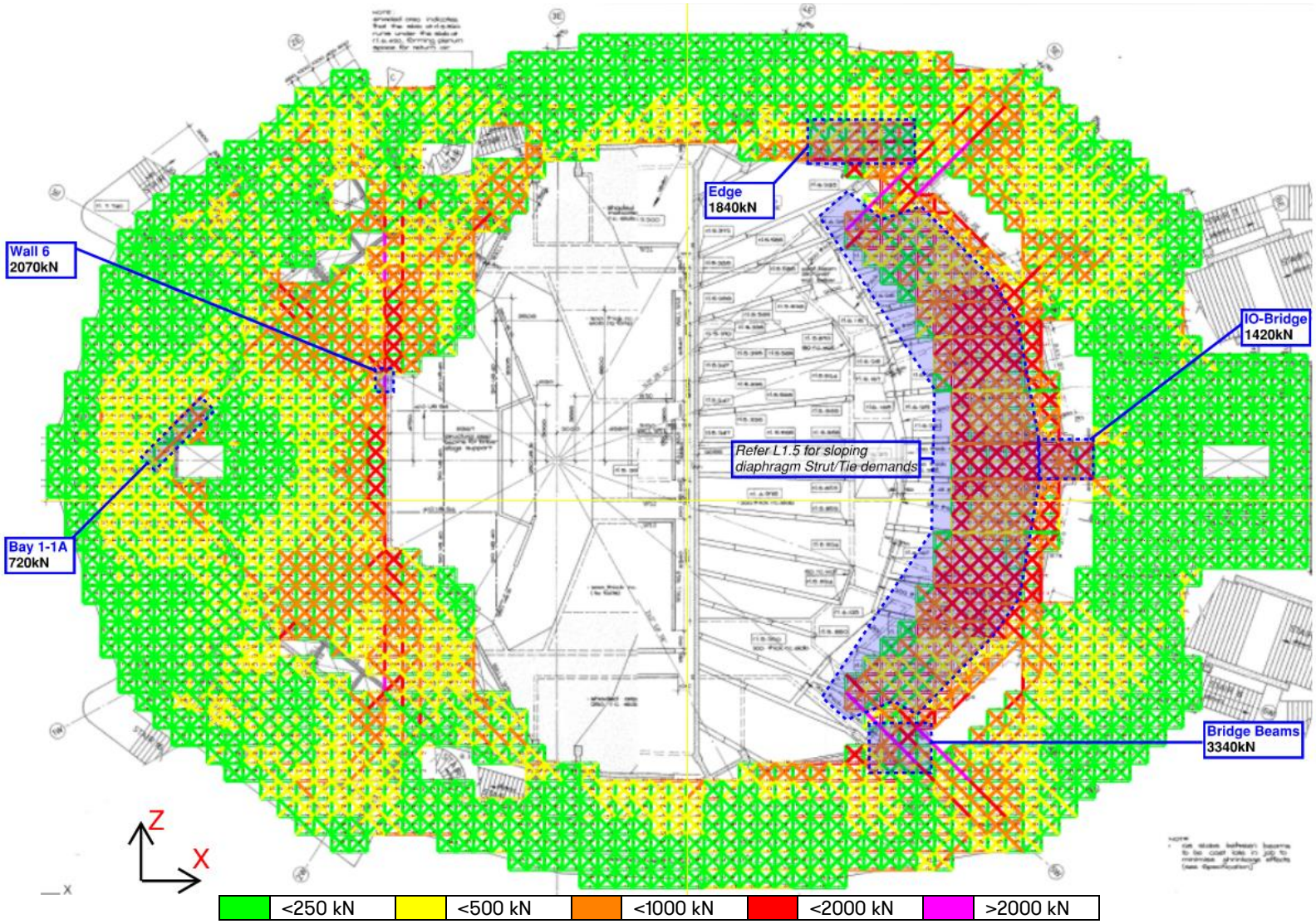

Figure C.4-32: Enveloped strut demands for the Level 2 diaphragm

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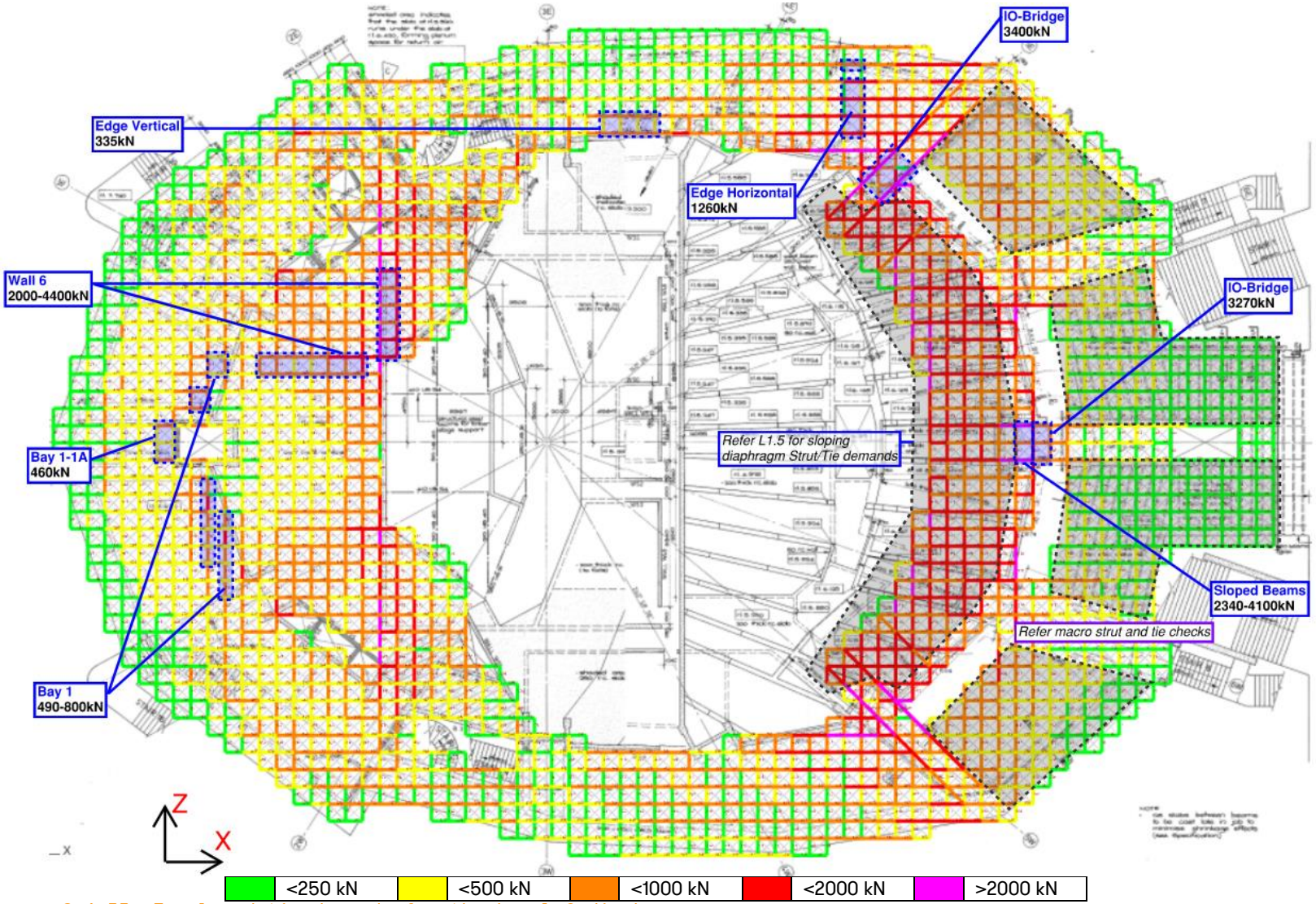

Figure C.4-33: Enveloped tie demands for the Level 2 diaphragm

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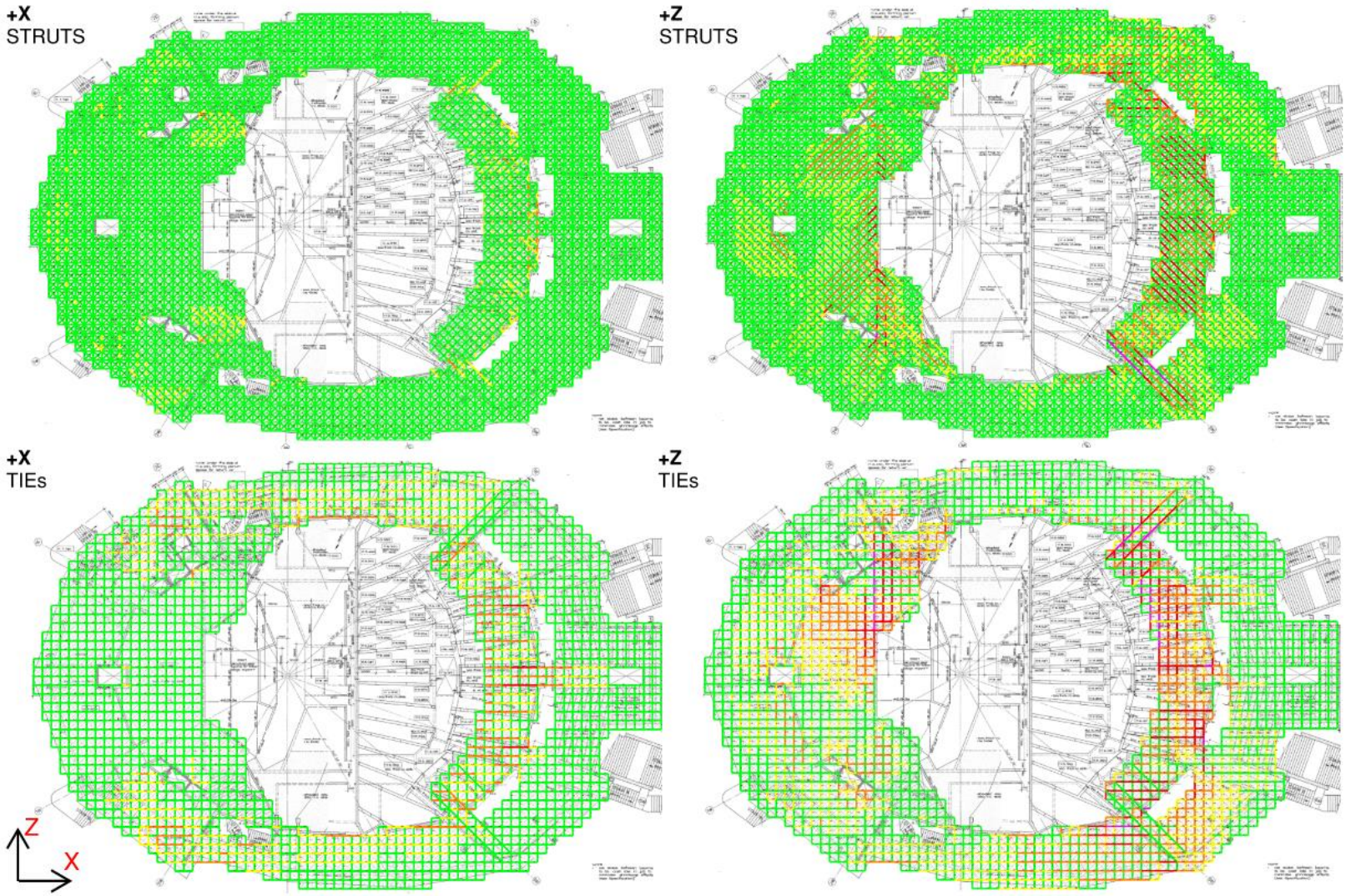

Figure C.4-34: Strut and tie demands for the Level 2 +X (left) and +Z (right) load case

C.4.2.7.2 Level 2 – Lobby area promenade

The Lobby area of the Level 2 promenade consists of double-tee unit floors with cold-drawn (non-ductile) mesh reinforcing and cast insitu slabs with bar reinforcing extending out from the sloping diaphragm on the inside ring. The cold-drawn (non-ductile) mesh reinforcement limits the formation of strut and tie mechanism as the mesh cannot be relied on over the different slab interfaces. These localised discontinuities in the formation of strut and ties have been assessed using macro strut and tie models. Two models can be identified namely for the Bay 6(a) and the Bays 7a(a) model as illustrated in Figure C.4-36. The Bays 7a(a) model is the governing model as this includes the floor extension towards the Renouf Foyer. This section will therefore be dedicated to the Bays 7a(a) model.

The cold-drawn (non-ductile) mesh reinforcing cannot be relied on at the floor interfaces and therefore these interfaces need to rely on the friction between the units and the supporting beams. The edges of the double-tee unit floors are only minimally adequately tied back to surrounding floors and supporting structure. The double-tee units at Level 2 are rated at **25%NBS$_{IL3}$**, limited by the point at which they overcome friction. An overview of the potential structural weaknesses and their ratings have been included in Figure C.4-36 below. These will be discussed in more detail later in this section.

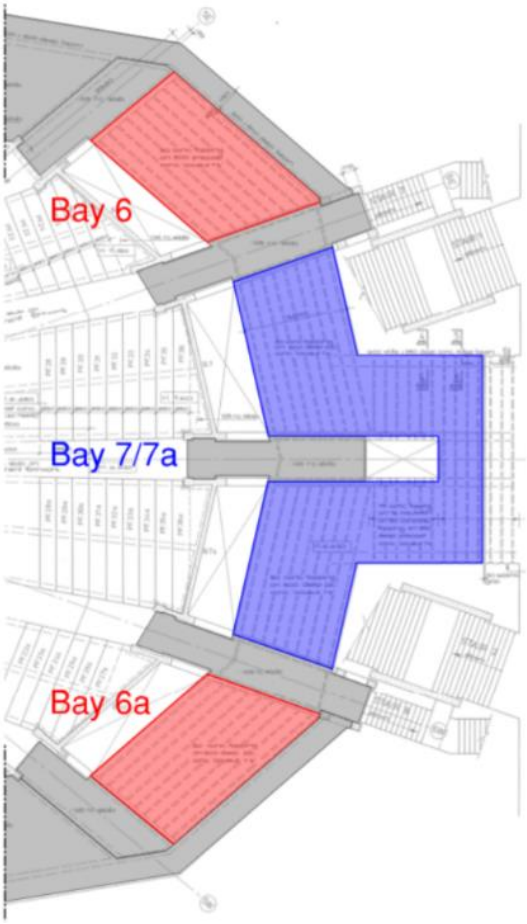

Figure C.4-35: Overview of Level 2 promenade double-tee unit macro models

Macro strut and tie

The Bays 7(a) macro strut and tie model for the positive x and positive z-direction have been illustrated in Figure C.4-37 below. The schemes for the negative directions look similar all be it mirrored.

For the global x-direction the double-tee unit floor sections can strut towards the beams which are tied back into the perimeter walls and transfer diaphragm. However, the floors are not restrained in the positive direction. There is no continuity of reinforcing and therefore the double-tee unit inertia need to be transferred into the beams through friction as indicated in orange in Figure C.4-37.

For the global z-direction the inertia of the floor wants to rotate around the Auditorium's bay walls and therefore effectively this set of floors is cantilevering from bay walls and transfer diaphragms. The strut and tie largely analysis relies on bearing struts and ties within the confines of the double-tee units However the equilibrium tie resisting rotation of double-tee unit adjacent to Bays 7(a) can only be transferred into the beam through friction as there is a discontinuity in reinforcing.

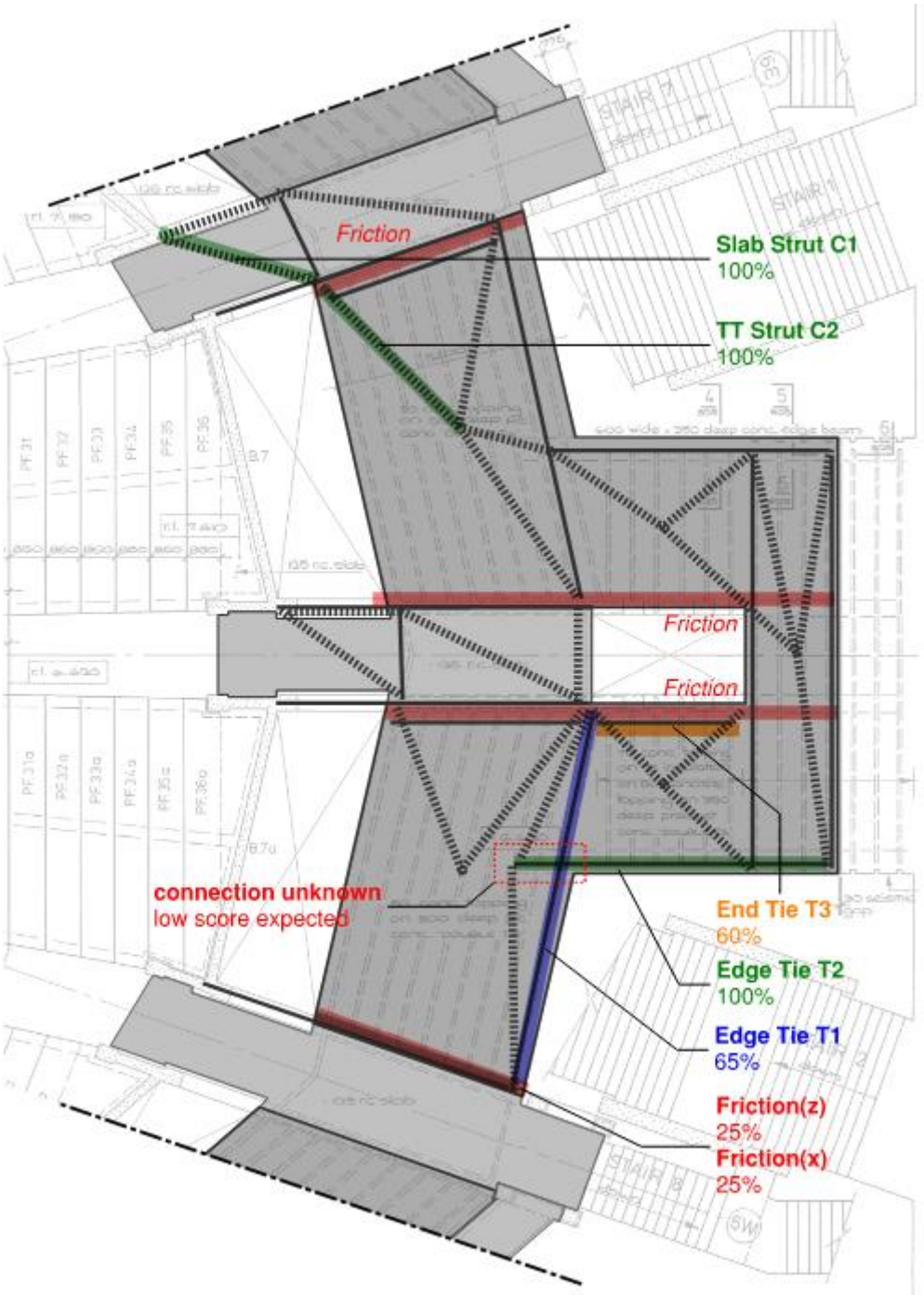

Figure C.4-36: Summary of the Level 2 double-tee unit ratings (overlayed on strut and tie scheme for z (up-down) loading)

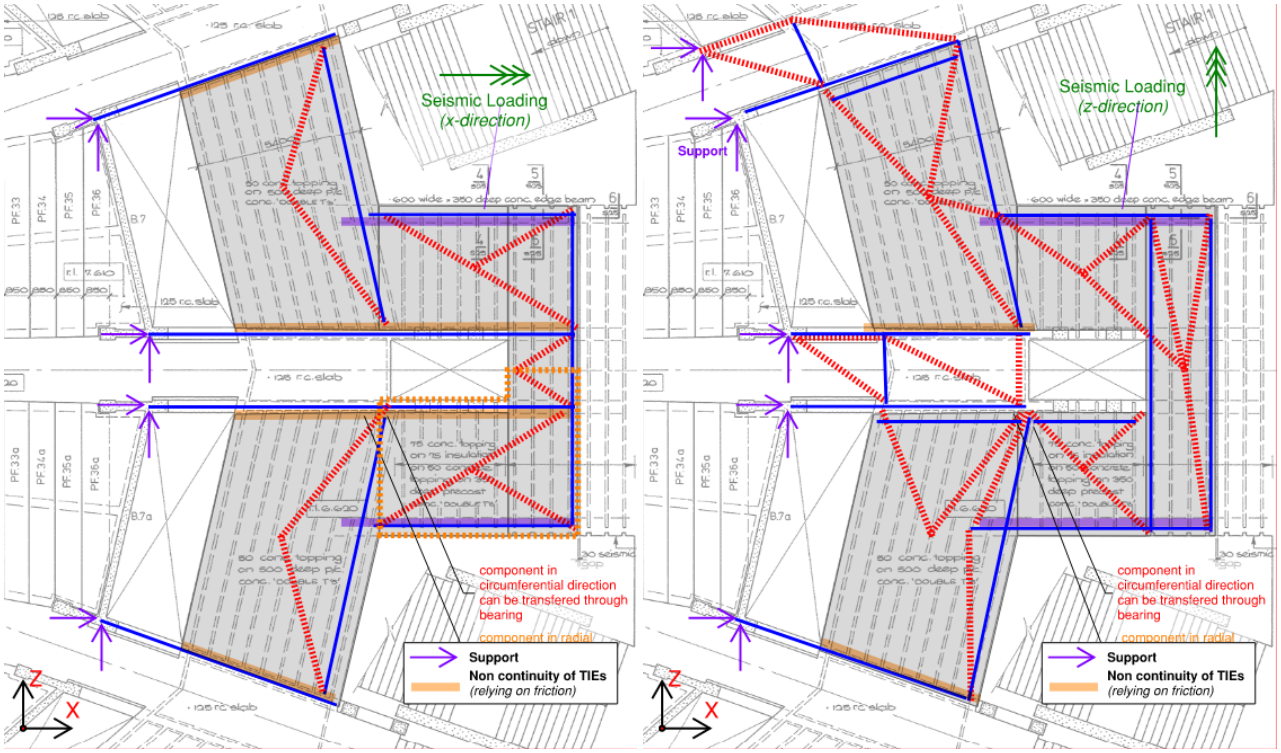

Figure C.4-37: Macro strut and tie model for Bay7/7a

Potential structural weaknesses

The main structural weakness for the double-tee unit floors in Bays 7(a) occurs there where tie forces need to be transferred where the double-tee unit and the cast insitu slab interfaces. The only continuous reinforcement at these interfaces is the non-ductile mesh which cannot be relied on and therefore the only potential mechanism for lateral resistance in these areas is friction from the double-tee units.

Given the double-tee units have been pretensioned the non-ductile mesh is assumed to contribute to tie capacity within the confines of a double-tee unit as cracking occurs around the perimeter of the double-tee unit floors. The following potential structural weaknesses have been identified for the Bays 7(a) model.

- Tie capacity to at double-tee unit to the cast insitu slab interfaces (friction mechanism)
- Internal tie demands of the double-tee unit floor system
- Tie capacity of the edge beam

The friction type of resistance is not a reliable method used in the design of new diaphragms over precast flooring. It can be associated with unpredictable damage, such as spalling and out-of-plane bending of the webs of the double-tees. This could lead to other associated life safety hazards, such as loss of seating. As such, the exceedance of the friction capacity is considered as the governing mechanism.

The site observations from the 2016 post-Kaikoura Earthquake inspections and the site observations undertaken for this DSA highlight that there is cracking in the web areas of some of the double-tee units in the lobby area – in particular, at those areas which rely on friction according to this assessment. Aside from some localised cracking in other parts of the building, this was the primary damaged observed in the building during these inspections. Whilst it cannot be confirmed with certainty that this damage is related to the 2016 Kaikoura earthquake; it is an area where damage may be expected early from the mode of behaviour determined in this assessment.

Figure C.4-38: Photograph, showing the onset of cracking in the web area of the double-tee units. This cracking is only present at those locations which are highlighted as needing to rely on the friction interface.

The site observations also highlighted that there may be a bit more concrete infill than shown on the drawings, particularly around the lift as shown on Figure C.4-39. This may result in a better-than-expected performance as the double-tee units could bear directly on the columns to provide a load path. However, the reinforcing of these concrete infill areas is expected to be minimal. Also, they are only located at the lift, which is not expected to be the critical area relying on friction.

Figure C.4-39: Photograph of the concrete infill between the columns of the extended frame a Bays 7(a)

Lastly, it is noted that the structural weaknesses could be progressively worse when subject to the effects from pounding with the Renouf Foyer, as described in Section C.4.5.1.

C.4.2.8 Level 1/2

The Level 1/2 diaphragm is fully situated on the inside of the perimeter walls. Effectively an extension of the Level 2 diaphragm as these are connected through the sloping diaphragm. The sloping transfer diaphragm is a 200mm thick concrete diaphragm with Ø20mm bars of the higher reinforcing grade spaced at 275mm in both directions. The sloping transfer diaphragm acts as a transfer element between Level 2 and Level 1/2 transferring the lateral demands from the four bay walls at Bays 6(a) and 7(a) down through the sloping transfer diaphragm and flat diaphragm at Level 1/2 into the walls supporting Level 1/2. The flat portion of the floor comprises a 200-250mm thick floor with areas having a thickness of 400mm.

The sloping transfer diaphragm between Level 2 and 1/2 is scored at **20%NBS$_{IL3}$** and is limited by the distributed tie capacity. There is also a structural weakness identified with a score of 20%NBS$_{IL3}$ for the ties resolving the transfer force eccentricity into the support walls below (Walls 14, 15, and Bay 4). A detailed overview of the potential structural weaknesses and their ratings has been included in Figure C.4-40 below. The enveloped strut and tie demands are included in Figure C.4-41 and Figure C.4-42 below. These enveloped results include both positive and negative directions and all X and Z concurrency combinations.

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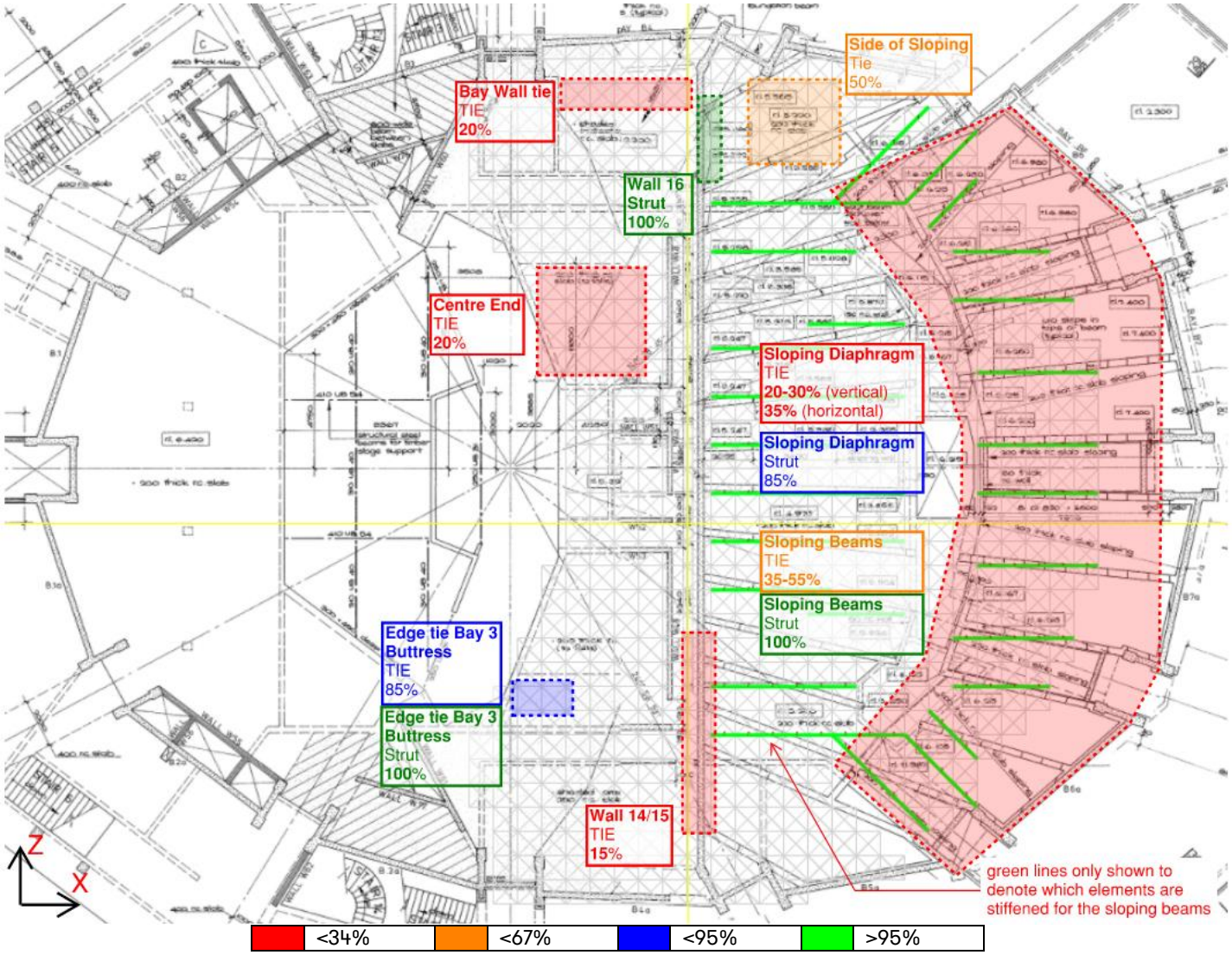

Figure C.4-40: Summary of Level 1/2 diaphragm rating

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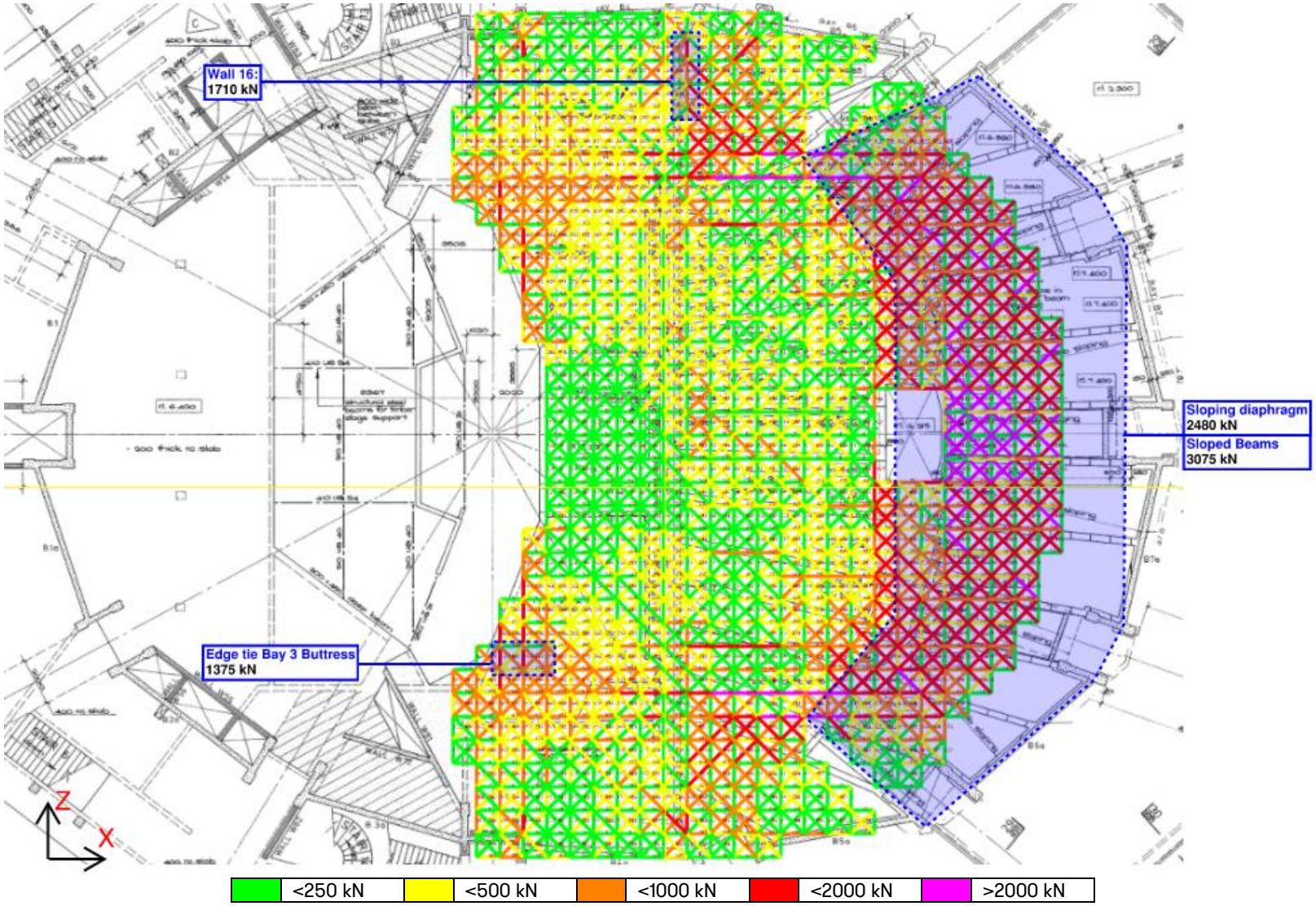

Figure C.4-41: Enveloped strut demands for the Level 1/2 diaphragm

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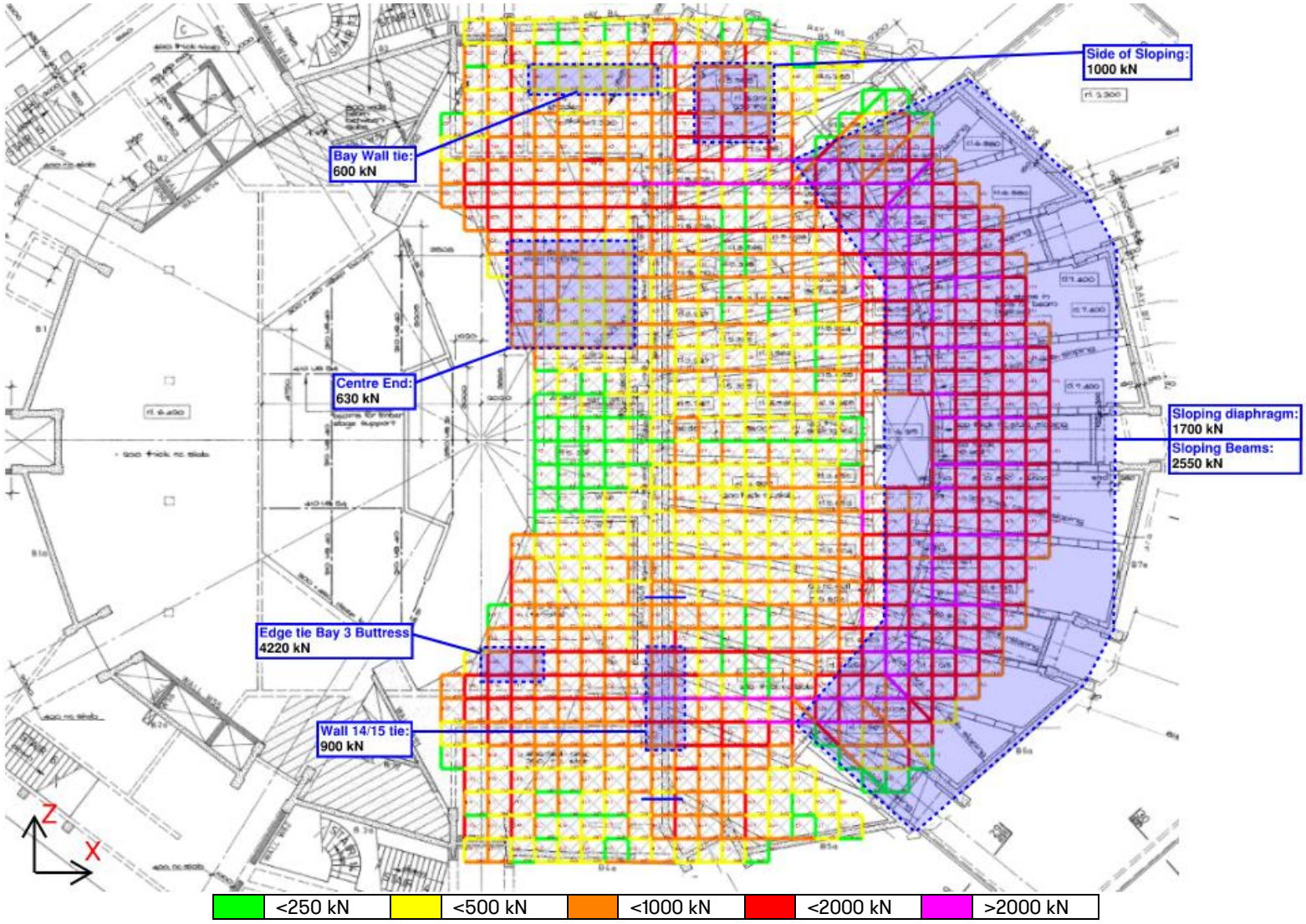

Figure C.4-42: Enveloped tie demands for the Level 1/2 diaphragm

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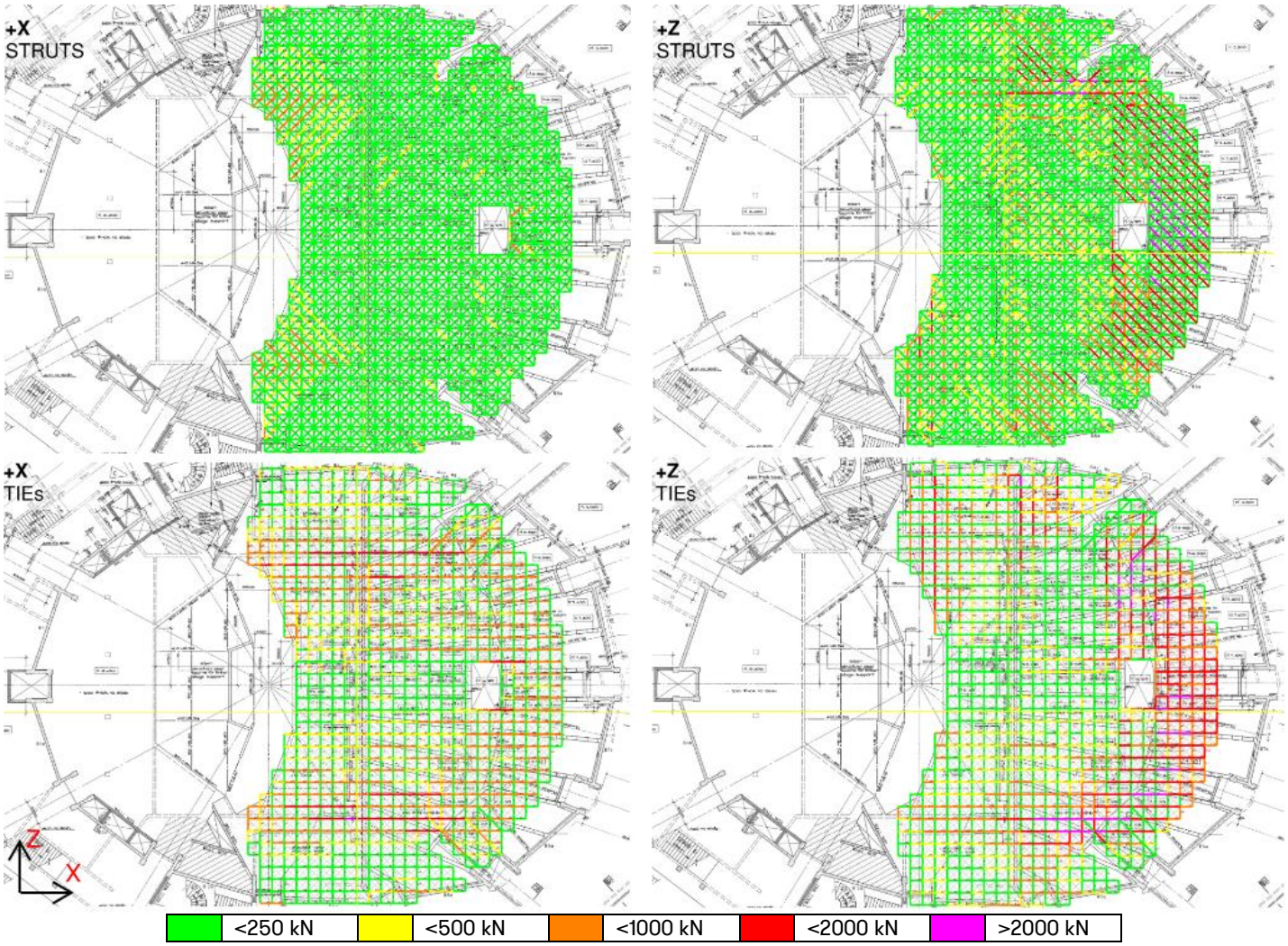

Figure C.4-43: Strut and tie demands for the Level 1/2 +X (left) and +Z (right) load case

C.4.3 Primary Gravity System

The elements that are discussed in this section of the report are not considered to be part of the primary lateral load resisting system, however, since they are part of the primary gravity supporting system, they need to accommodate for the displacements and forces that arise in the structure from seismic shaking.

C.4.3.1 Roof Trusses

The roof of the Auditorium comprises Dycore flooring units spanning between steel trusses. The trusses are 3.5m high and are made from structural steel angles. The trusses span from east to west of the Auditorium and connect to the precast wall panels or the precast buttresses of the bay walls. The trusses are labelled T1-T10 and are shortest near the northern and southern ends of the Auditorium, whilst the central ones T3-T7 are the longest. The top and bottom chords of the trusses comprise 2-152x152 angles that are longitudinally welded together to form a square hollow section. The chord sections have a thickness of 12.5mm for the longer trusses (T3-T7) and a thickness of 9.5mm for the other shorter trusses (T1-T2 and T8-T10). The verticals and diagonals of the trusses also comprise structural steel angles, either single profiles or two sections welded together to form square hollow sections. They have various sizes, ranging from 102x102x9.5mm to 152x152x12.5mm, but in general they are smallest in the middle and increase in section size towards the supports.

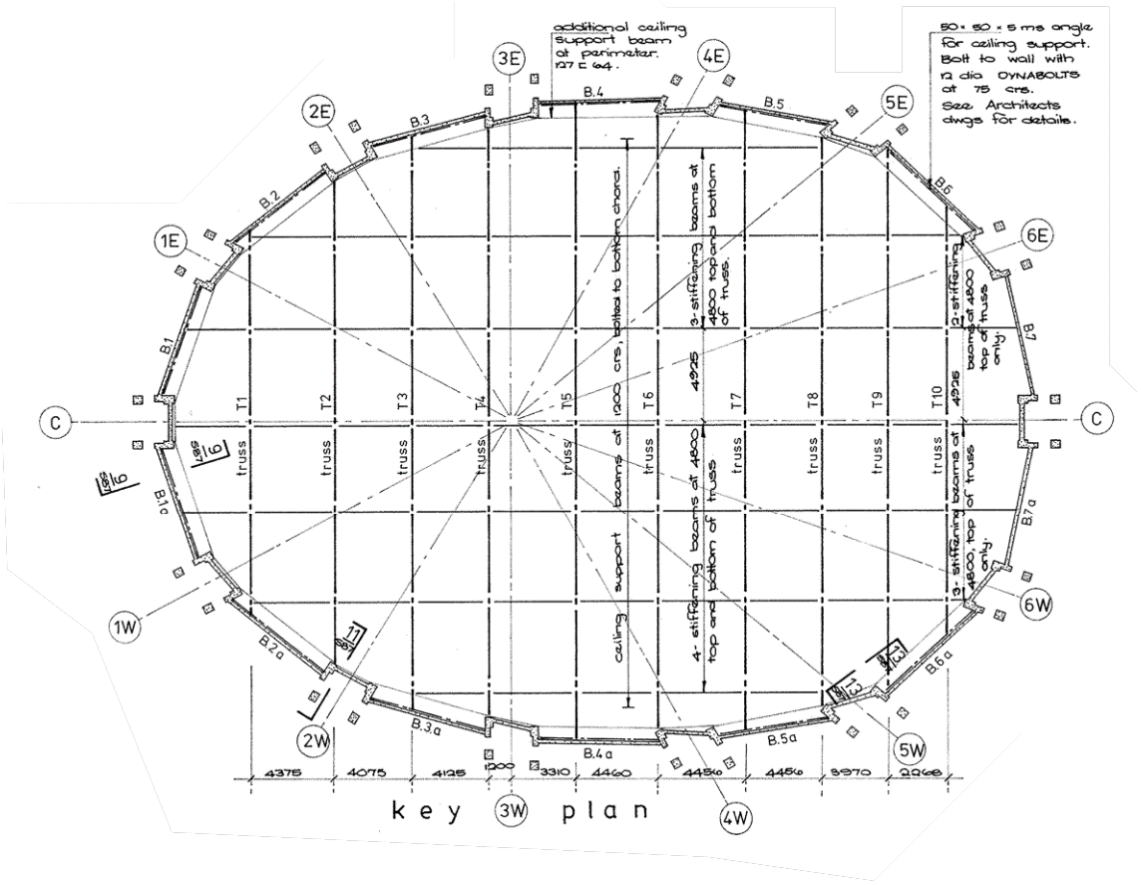

Figure C.4-44: Snapshot from drawing S86A of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing the set-out of the trusses in plan

The bottom chords of the roof trusses support an acoustic ceiling made of timber panels that span between ceiling beams. These elements are considered in Section G.8.

The chords have been constructed (likely prefabricated) in two segments. A bolted end plate splice at the top chord and a large, pinned connection at the bottom chord connect the two segments together as shown in Figure C.4-45.

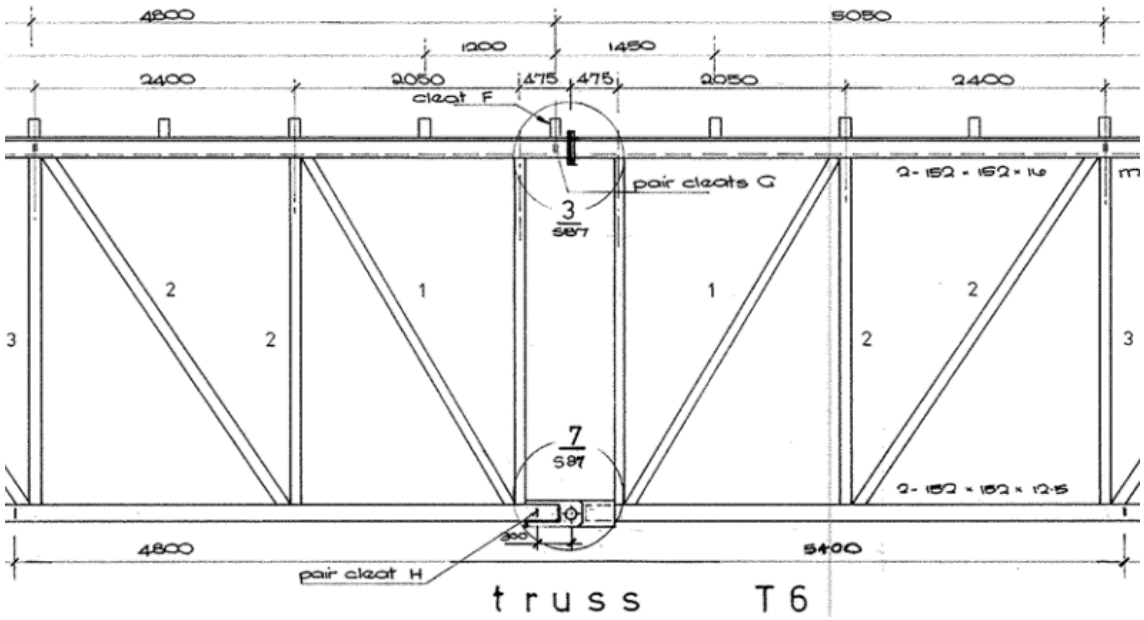

Figure C.4-45: Snapshot from drawing S86A of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing a typical elevation of the truss' splice connection

Along the length of the trusses, there are several elements referred to as 'stiffening beams' on the original drawings. These elements connect into the sides of the top and bottom chord and span in the perpendicular direction at approximately 4.8m spacing. They comprise 152x76 structural steel channels that are bolted to the trusses with single Ø16mm bolts.

On the top chord of the roof trusses there are also 16mm cleats that extend between the Dycore units into the 65mm concrete topping. They are located at 1.2m spacing and allow for Ø20mm saddle bar to pass through.

Potential structural weaknesses

During seismic shaking the primary lateral load resisting system, the bay walls, impose the drifts on the roof trusses. These drifts could result in additional forces in the roof trusses as they try to stay compatible with the deformed shape of the bay walls. When these additional loads start to exceed the capacity of the steel angles, the roof trusses may lose their ability to carry the gravity loads. The gravity loads that the trusses support was found to already absorb a significant portion of the section capacities.

The deformed shape of the bay wall could also force the connections of the roof trusses to lose support, either by force or displacement.

Furthermore, vertical seismic accelerations may excite the roof structure beyond the vertical load capacity of the roof trusses. This could result in excessive displacements of the roof trusses and the potential loss of gravity support.

Connection at precast wall panel

The roof trusses T1, T3, T5, T7 and T10 connect to the sloping precast concrete wall panels of the bay walls. They connect to the sloped precast walls through a series of welded angles and channels as shown in Figure C.4-47 and Figure C.4-48. Two 254x89x35.7mm steel channels span the 3.5m height of the truss and create seating for the top and bottom chords. The steel channels are connected to the precast concrete wall panel with steel angles and brackets that have been cast into the precast concrete wall panels.

At the top chord, the roof trusses are seated in a 20mm thick cradle plate (shown in orange on Figure C.4-47) which provides a vertical support as well as a stiffening of the section. The cradle plate has a 30mm packer plate and sits atop a fabricated box section (shown in red on Figure C.4-47) which spans between the steel channels. Lastly, there are two 10mm plates that give the top chord an in-plane restraint. This connection provides a pinned or semi-rigid connection for the top chord in all principal directions.

Figure C.4-46: Photograph from inside the ceiling space - showing the bottom connection of roof truss

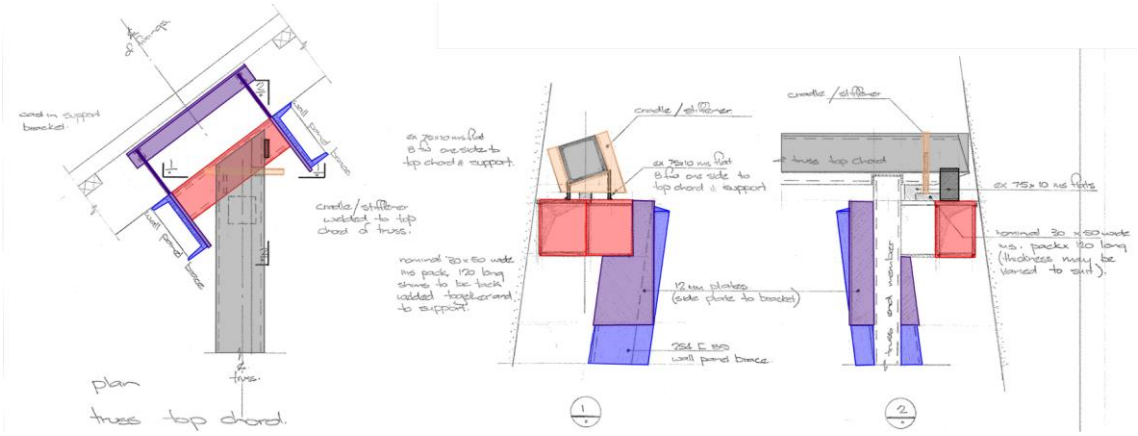

Figure C.4-47: Snapshot from drawing S88/6 of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing a typical top chord connection to the precast concrete wall panels

At the bottom chord, the roof trusses connect via a 12mm steel plate to an 80x80x8.0mm steel angle. This angle spans between the two steel channels. Effectively, this only creates an out-of-plane support to the bottom chord as the 12mm plate is not considered to be able to carry gravity loads of the roof truss. The steel angle is approximately 100mm offset from the precast concrete wall panel and, therefore, displacement or drift of the bay walls can be absorbed by yielding the steel angle and deforming it – the bottom chord is considered to be much stronger in compression than the steel angle is in bending.

For the assessment we have assumed that the bottom chord is connected approximately at the middle of steel angle, as per the 1980's structural drawings. However, the connections have been visually observed to vary considerably, with the worst case shown in Figure C.4-46. This shows the bottom chord is connected right against the steel channels. This would result in nominal displacement capacity and could potentially result in the roof trusses to 'lock up' at relatively low drifts.

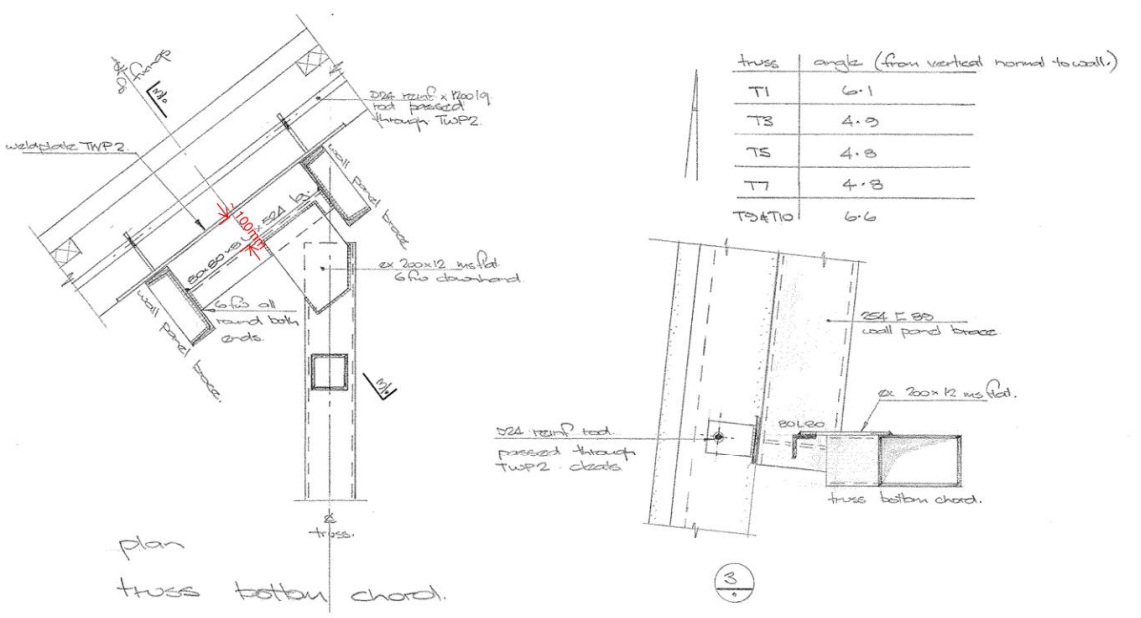

Figure C.4-48: Snapshot from drawing S88/6 of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing a typical bottom chord connection to the precast wall panels.

Connection at precast buttresses

The roof trusses T2, T4, T6, T8 and T9 connect directly to the internal column of the precast buttresses. The connections at both the top and bottom chord are through brackets that are cast into the precast buttresses with Ø24mm u-bars as shown in Figure C.4-49 and Figure C.4-50.

The connection of the top chord is relatively similar to those connecting to the precast concrete wall panels as the top chord lands on a 20mm thick cradle plate. This cradle plate acts as a stiffener and provides vertical support via a 30mm packer plate onto the cast-in bracket. Lastly, there are two 10mm plates that provide an in-plane support to the top chord. Therefore, this connection provides a pinned connection for the top chord in all principal directions.

The bottom chord connects to the cast-in bracket via a 10mm plate that is welded on both ends. The connection appears to have been considered for some tolerance in construction, but effectively butts up against the precast buttresses directly. It provides an out-of-plane and in-plane support to the bottom chord; however, it is considered that the 10mm plate is unable to provide a vertical support for gravity loads.

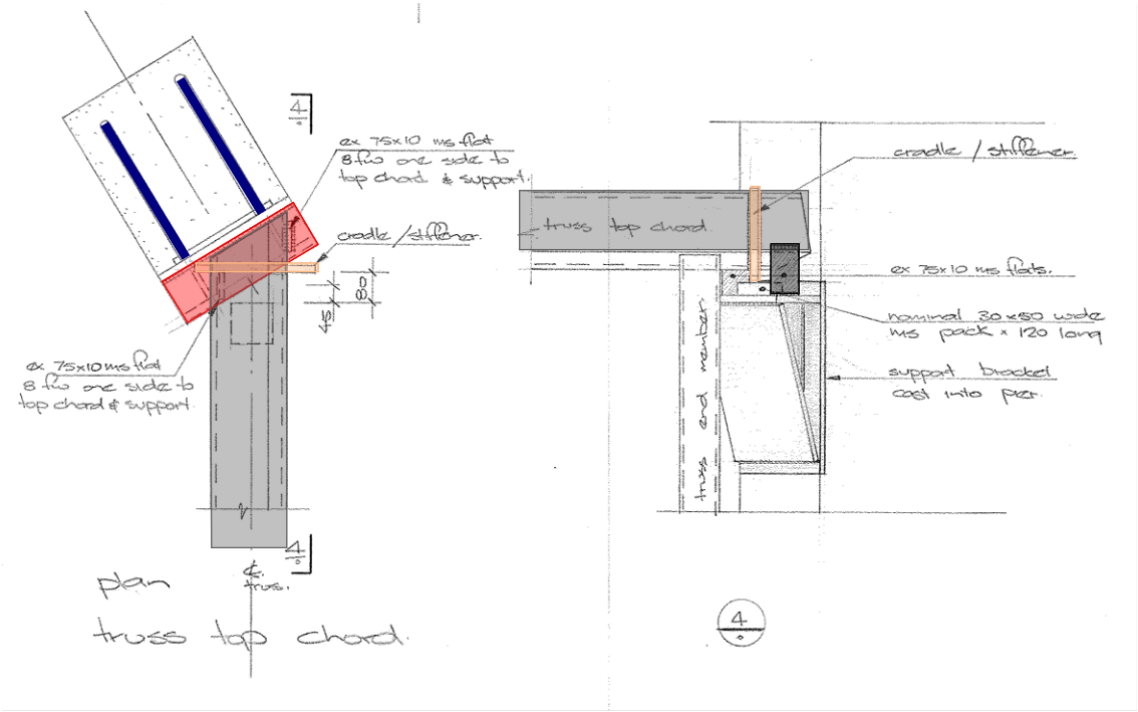

Figure C.4-49: Snapshot from drawing S88/6 of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing a typical top chord connection to the internal column of the precast buttresses

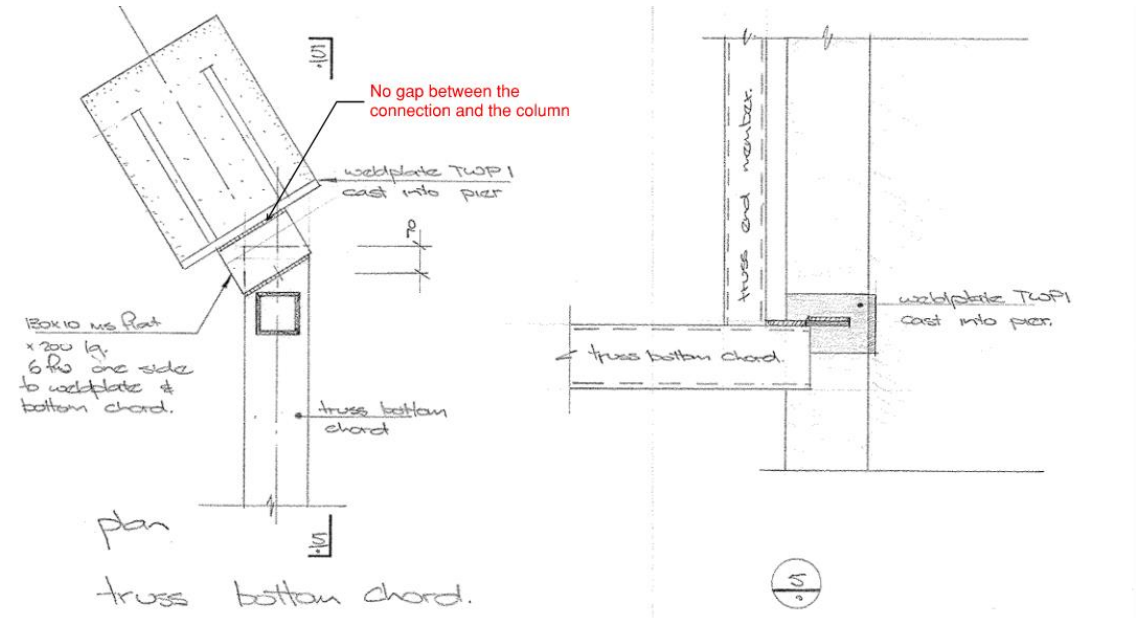

Figure C.4-50: Snapshot from drawing S88/6 of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing a typical bottom chord connection to the internal column of the precast buttresses

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Microstran model

The roof trusses were evaluated using an elastic truss model in Microstran where gravity loads (G + ψ_EQ) were combined with imposed drifts that were applied as loads to the truss model. A live load combination factor of ψ_E = 0 was used per NZS1170.0 for roof live loading. For the assessment, the longest and shortest truss conditions were modelled with vertical support at top connection location (as shown in Figure C.4-51).

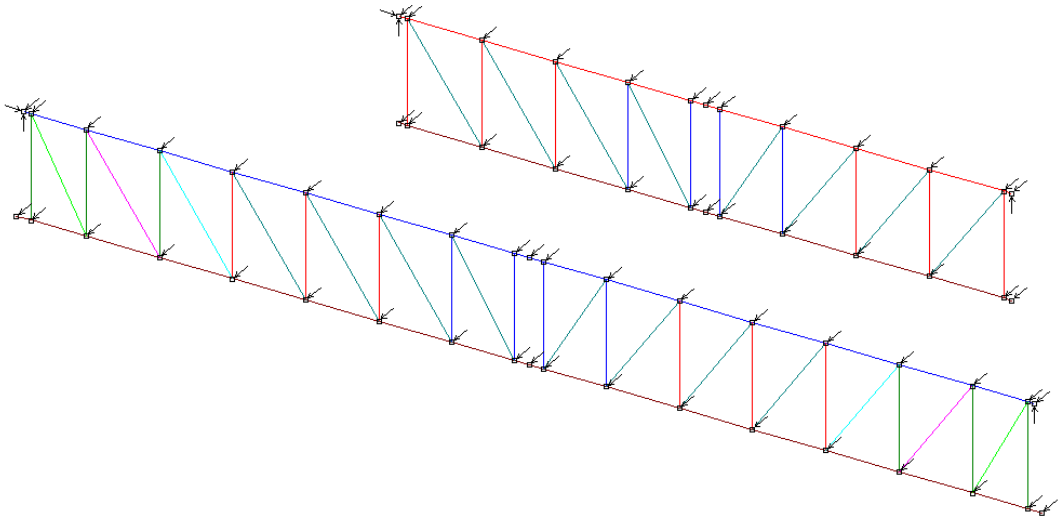

Figure C.4-51: A snapshot from the Microstran model showing the longest and shortest truss with their boundary conditions

In accordance with the configuration of the connections, the top chords have been considered with vertical restraints (Z) providing support for gravity loads. Restraints in the in-plane (X) direction have been assigned to the ends of the top chords. All nodes have been given out-of-plane restraints (Y).

As the structure drifts, the bottom chords roof trusses displace the angle seating in bending until they make contact with the precast concrete wall panel or the precast buttress columns, inducing large forces. This has been modelled by applying lateral forces in the Y-direction at the location of the bottom chords connections to the bay walls. The magnitude of this force is incrementally scaled up until the roof trusses achieves the corresponding drift of the bay walls.

Assessment of internal forces

The assessment considers that there is no displacement capacity before the trusses 'lock up'. The internal forces, combined with the effects of gravity loading, are critical at two locations in the chord members.

The top chord is found to be critical at midspan of the trusses as the compression forces from the gravity loads are already using up a large portion of the member's capacity. The eccentricity due to the spliced connection at midspan then exceeds the element's capacity from compression and flexure. The assessment has considered a buckling length of 1.2m, as restraint is provided by the connection with the roof diaphragm. It was found that the top chords are not critical.

The bottom chord is found to be critical at the ends of the trusses near the connections. Here a buckling length of 4.8m is considered and so the chords have a much lower member capacity. However, these were found to be critical at 0.31% drift for the longer trusses and 0.26% drift for the shorter trusses.

Compared with the structure's expected drifts from Table C.3-3, the associated seismic score of the roof trusses is determined at **25%NBS$_{IL3}$**.

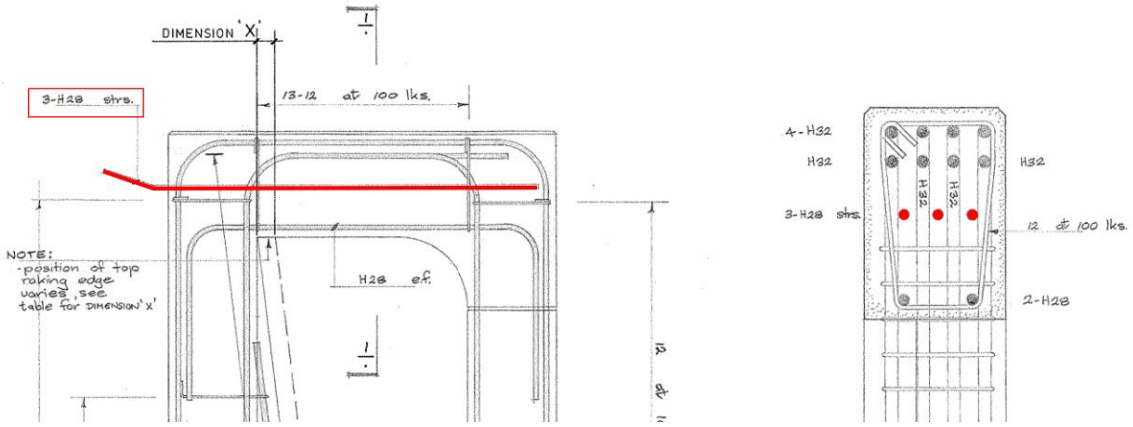

Figure C.4-52: Snapshot from drawing S83A of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing the Ø28mm bars from the internal column of the precast buttresses into the roof diaphragm

Assessment of loss of seating

When the truss 'locks up' with the bay walls, there are prying effects pushing on the top chord.

The 20mm cradle plates and the two 10mm end plates are expected to yield at very small drifts. However, they are found to have welds of sufficient strength to allow a ductile performance through bending or buckling. The roof trusses are tied to the concrete roof diaphragm, which in turn is tied into the bay walls through the three Ø28mm starter bars that extend from the precast buttresses as shown in Figure C.4-52. The rupture strain of these Ø28mm bars (as per Section C5.5.3.4 of the Engineering Assessment Guidelines) is considered to be one of the limiting factors. The bars provide a restraining force beyond yield and until the bars rupture there is a level of integrity remaining in the seating support. This allows for significant drift compatibility as the associated strains in the bars are relatively large (capacity to drift up to 0.9%).

Another limiting factor is the movement of the top chord relative to the supporting cast-in bracket. As the cradle plate and the end plates yield, the trusses can effectively move relative to the supporting cast-in bracket. From the drawings it was found that the sections can rely on approximately 100mm of seating capacity. Considering ~20mm of construction tolerance and considering an element performance factor of Q = 2.0 in accordance with Section C10.4.5 of the Engineering Assessment Guidelines (to capture the potential consequences of the roof trusses losing their seating) this results in ~40mm of available seating. Consideration is then also given then to the fact that the roof trusses can move under an angle of up to 40 degrees relative to the internal columns of the precast buttresses. As such, the associated drift capacity is 0.81% drift.

Therefore, the loss of seating is not considered to be the critical element, performing at 90%NBS_{IL3} when considering a potential loss of seating.

Anchor pull-out of the cast-in brackets was not found to be critical, as there are no major tensile or shear forces developed in them.

Vertical excitation of the roof structure

In accordance with the original 1980's design calculations, the roof structure is designed for an additional live load of 1.5 kPa. The self-weight of the roof structure is 3.22 kPa. Since the live load in seismic coefficient is considered at a factor of 0, the roof structure has an inherent capacity against vertical excitation.

Vertical accelerations of the roof structure are determined in accordance with Sections 3.1.1, 3.2, 5.4.2 and 8.5.2 of NZS1170.5:2004 (excluding Amendment 1). Conservatively assuming the roof trusses result in a period of T<0.4s, they are accelerated vertically by 1.092g. This results in a seismic score of >45%NBS_{IL3}.

The roof trusses and Dycore floor units may have additional capacity that could allow for an increased seismic score, however, since they are already controlled by other mechanisms, this is not further investigated as part of this assessment.

Table C.4-8: Parameters used in determining the vertical 'Parts and Components' acceleration of the roof structure

Parameter	Notation	Value	Commentary
Site hazard coefficient for T = 0.4s	C(0.4)	1.560	
Elastic site hazard spectrum for vertical loading for T = 0.4s	$C_{vd}(0.4)$	1.092	0.7 C(T)
Part risk factor	R_p	1.0	Category P.2 and P.4
Part vertical response factor	C_{pv}	1.0	μ = 1.0

C.4.3.2 Precast Cantilever Beams

For the typical bay wall, there are precast cantilever beams extending on both the inside and outside of the Auditorium. These beams typically consist of a diagonal element and a horizontal element that form a triangular shape that ties back to the buttresses. The connection to the buttresses is through starter bars that have been cast integrally with the buttresses to develop their capacity.

The precast cantilever beams are able to develop their independent structural behaviour, relative to the bay walls. Therefore, they have been assessed for vertical accelerations for 'Parts & Components' in accordance with NZS1170.5:2004 excluding the provisions of Amendment 1. Nominal ductile behaviour (μ_p = 1.25) of the precast cantilever beams is assumed in the assessment. The primary structure is conservatively assumed to have period of less than 0.4s. At 100%ULS seismic loading, this results in a vertical acceleration of 0.93g.

Table C.4-9: Parameters used in determining the vertical 'Parts and Components' acceleration of the precast cantilever beams

Parameter	Notation	Value	Commentary
Site hazard coefficient for T = 0.4s	C(0.4)	1.560	
Elastic site hazard spectrum for vertical loading for T = 0.4s	$C_{vd}(0.4)$	1.092	0.7 C(T)
Part risk factor	R_p	1.0	Category P.1, P.2 and P.4
Part vertical response factor	C_{pv}	0.85	μ = 1.25

Considering that the sections will have typically been designed to accommodate live load of 4.0kPa in load combinations such 1.2·DL+1.5·LL, this has been found to exceed the demands from 1.0·DL+ψ_E.LL+EQ_v. Therefore, the precast cantilever beams are scored at **100%NBS$_{IL3}$**.

Effects on bay wall

The precast cantilever beams are connected to the buttresses in a typical balanced cantilever system where the demands from one side are relatively matched by the other side. Where there are no balancing elements, the structural design has ensured that the bay wall ties in with other floor elements to provide a level of restraint. This design feature ensures there are no major flexural demands occur in the buttresses.

Vertical accelerations are considered to occur simultaneously in the precast cantilever beams on the outside and inside of the structure. The scenario where the precast cantilever beam on one side may unload almost completely from upwards accelerations while the other side sees additional downwards accelerations is deemed to be very unlikely. However, should such an event occur, then the buttresses have sufficient flexural capacity to deal with those effects.

The buttresses are considered to span vertically between the different floor and roof levels, each providing a lateral (spring) support. Therefore, the flexural demands are typically resolved locally through reversing of the flexural demands.

Interface with diaphragm ties

Vertical loads provide in-plane bending demands on the precast cantilever beams. As described in Section C.4.2.5.1, some of the precast cantilever beams also see out-of-plane bending demands and axial demands following the diaphragm effects on the Level 4 diaphragm segments at each Bay Wall. Refer to that Section for a detailed description of the seismic score.

C.4.3.3 Beam-Column System at Bays 6(a) and 7(a)

The extended frames at Bays 6(a) and 7(a) comprise 400x500mm coupled column units and 900x400mm beams that form a moment frame type structure which matches the architectural features of the coupled column units in the Renouf Foyer. They are considered to be relatively flexible in comparison to the lateral load resisting system of the Auditorium.

The detailing of the beam-column joints is similar to those in the Renouf Foyer; however, the frames see relatively less axial load in the columns as they support a smaller area of floor.

Hierarchy

The hierarchy of the beam-column joints in the longitudinal (X) direction is similar to the coupled column units in the Renouf Foyer – a column-sway mechanism. The assessment hierarchy has compared the capacity against those from joint shear and beam flexure but concluded the columns to be the weakest link with the lowest displacement capacity.

In the transverse (Z) direction a plastic hinge at the base of the columns is expected to form at large displacements. There are no return members that make it behave a frame system.

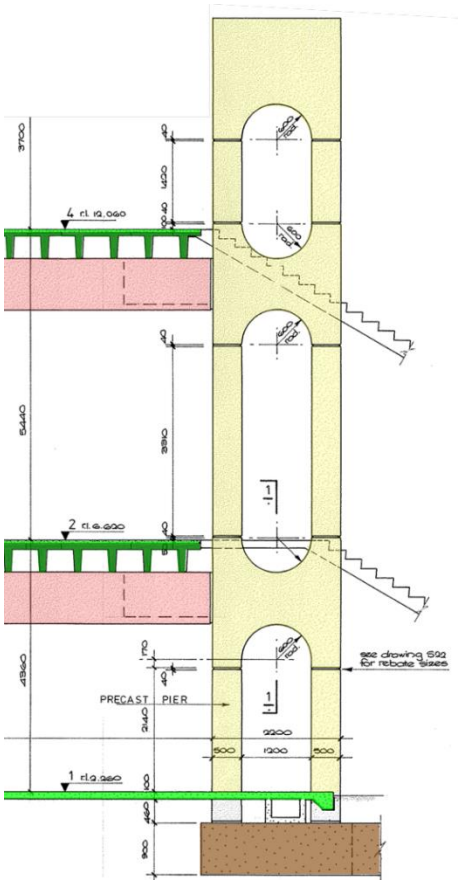

Figure C.4-53: Typical elevation of the coupled column units at the end of the extended frames at Bays 6(a) and 7(a)

Drift capacity and demand

The columns are reinforced with 4-Ø28mm longitudinal bars of the lower reinforcement grade in the corners of the columns. At the foundations, the longitudinal bars have been plugged into 'Uniflex' ducts to create an integral connection. The longitudinal bars are wrapped by Ø10mm links at 200mm spacing.

The column's rotation capacities are determined in accordance with equations C5.33 and C5.59, which are referenced below. It considers the onset of loss of gravity load capacity the potential structural weakness.

$$\theta_p = 0.031 - 0.032 \cdot \frac{N^*}{A_g \cdot f'_c} + 0.47 \cdot \rho_t - 0.017 \cdot \frac{V_y}{V_{p,col0}}$$

and,

$$\theta_a = \frac{0.25}{5 + \frac{N^*}{0.8 \cdot A_g \cdot f'_c} \cdot \frac{1}{\rho_t} \cdot \frac{f'_c}{f_y}} - 0.005$$

The assessment concludes that the displacement capacity is limited at 2.24% drift. When accounting for yield drift, the columns are limited by the 2.50% drift limit.

Assuming the drift is concentrated at the ground floor in the longitudinal (X) direction, this means that the columns can sustain inelastic displacements of 110mm at Level 2 in both principal directions. This is sufficient to achieve 100%NBS_{IL3} with regards to the imposed drifts.

Consequences of lateral stretch of the ground

The consequences of lateral stretch and a potential structural step change are considered in Section C.4.4.4.2.

C.4.3.4 Wall/Column System at Wall 1

Along Wall W1 at the western end of the auditorium building 17 gravity columns are located. The purpose of these columns is to support the reinforced concrete floors which span out from the perimeter buttress wall system (bays 1-2). Most of these columns span from Level 2 to Level 4, although the two northernmost and two southernmost columns reach all the way down to the Level 1 slab.

The gravity columns are reinforced longitudinally with 6x H28 bars at Level 2 and 6x H24 bars at Level 3. Shear reinforcing is present in the form of 10 mm loops at 200 centres. The slab reinforcing runs through the column, there is no dedicated joint reinforcing. The longitudinal reinforcing runs straight through the Level 3 slab and is hooked into the Level 2 and Level 4 slabs. The slabs themselves are conventionally two-way reinforced around the columns. Typically, they are doubly reinforced around the column, though in some specific cases, no top reinforcing is present. It appears that the original design intent was for the concrete slabs to span from the squat wall W1 out to the gravity columns.

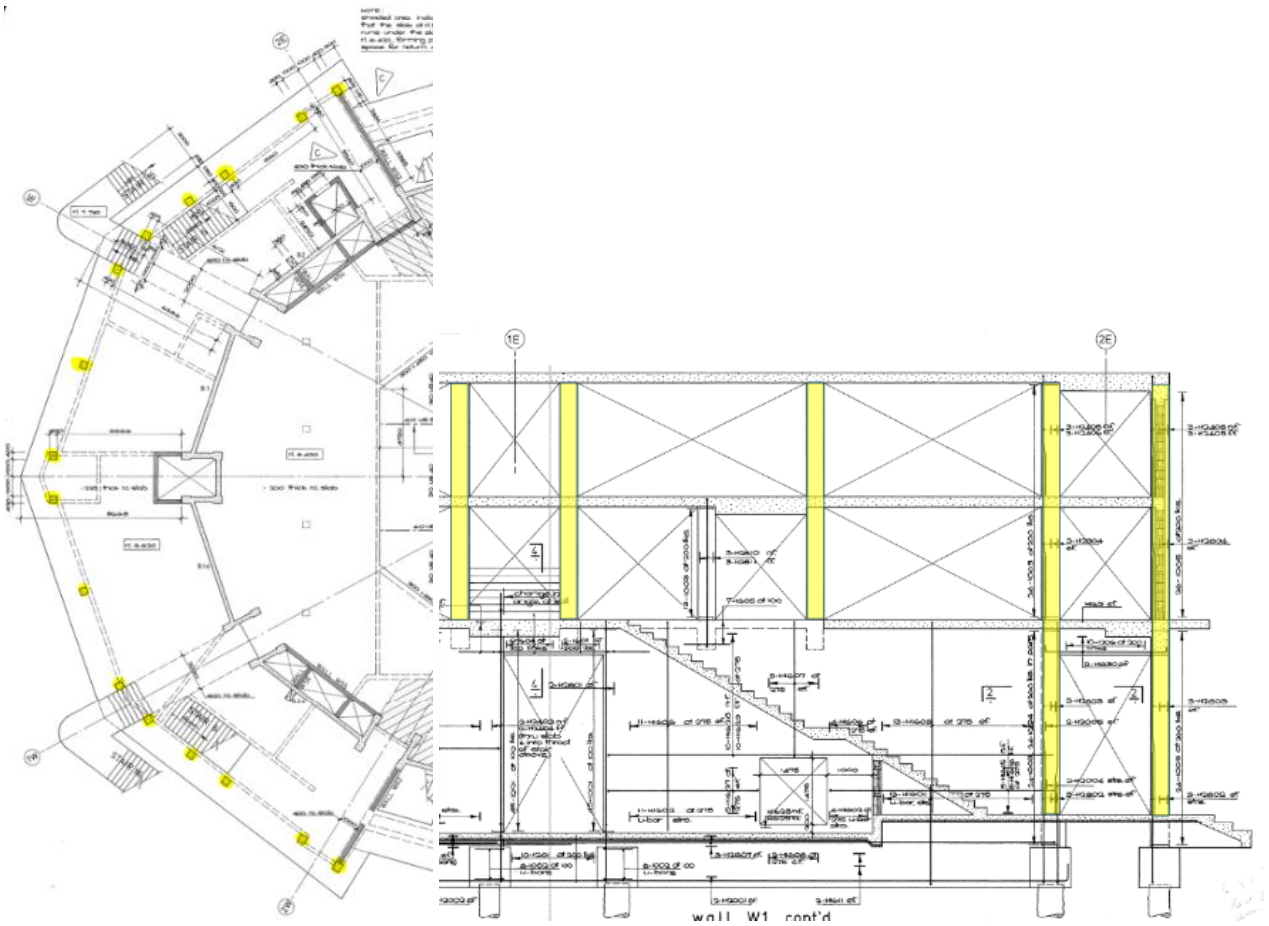

Figure C.4-54 - Plan and elevation of the gravity columns at Wall 1

The gravity columns are only connected to the slab, not tied into any of the other lateral load resisting systems. This means that it is unlikely that they will contribute significantly to the lateral stiffness of the building or change the seismic response considerably. This is in keeping with Yellow Chapter C5 of the Engineering Assessment Guidelines which notes that it is generally appropriate to assume that slab-columns joints are pinned for analysis purposes and do not contribute to the stiffness of the structure.

Slab-column joints

A deformation-based assessment has been used to determine the available inelastic drift capacity in the slab-column joints following Section C5.5 of the Engineering Assessment Guidelines. The issue with slab-column joints is that often these connections involve columns which were designed only for gravity loading with no considerations of the displacement demands that the remaining structure can impose. The critical issue is that the gravity load capacity of these connections is retained even after being subjected to numerous lateral displacement cycles. Slab-column connections can be susceptible to progressive collapse if punching shear failure occurs.

Using a deformation-based assessment of the Level 2 and Level 3 slab-column joints, it has been determined that the inelastic drift capacity of these connections is limited to 0.2% in the worst case. The maximum inter-storey drift demand from Level 2 is 0.40% in the longitudinal (X) direction. Therefore, the gravity column-slab connections are expected to perform at **45%NBS$_{IL3}$**.

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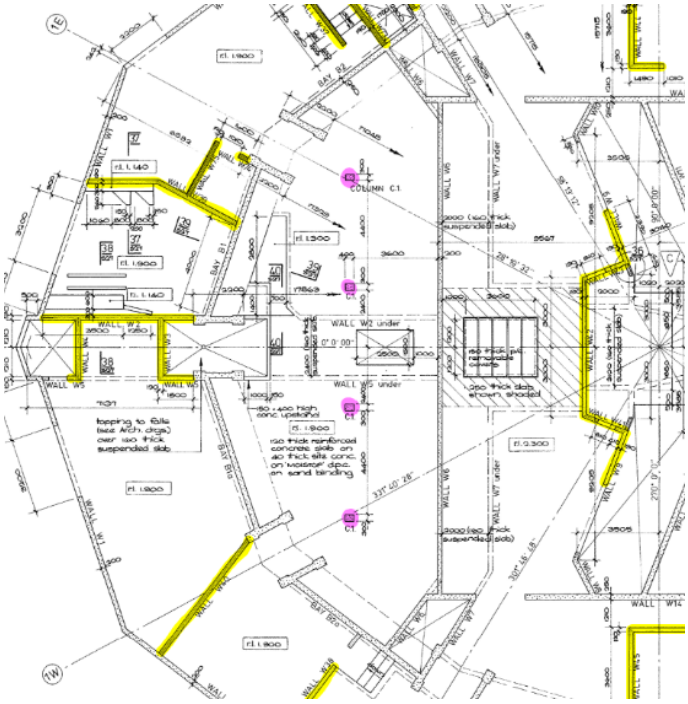
C.4.3.5 Other gravity columns



Another four gravity columns exist under the auditorium stage. These gravity columns provide an intermediate support for the Level 2 floor which otherwise spans from the perimeter buttress walls (bays 1-2) to the internal reinforced concrete walls (W6).

The gravity columns (type C1) are 4.3 m long and have a 400x300 cross-section. They are reinforced longitudinally with eight ø20 mm bars (three each face). These longitudinal bars are hooked into the Level 2 floor slab and into the foundation beam. The transverse shear reinforcing is provided by ø10 mm bars looped around the longitudinal reinforcing at 150 mm centres.

The configuration and reinforcing of these gravity columns are similar to the set of columns described in Section C.4.3.4. However, because these columns sit inside the perimeter of the bay walls at Level 1, the expected displacements are significantly lower. As a result, the gravity columns underneath the stage are not considered as a structural weakness.

C.4.4 Foundation System

The foundation system of the Auditorium comprises a mix of reinforced concrete foundation beams and spread footings. These beams tie together some piles while others remain isolated piles, only connected by the slab-on-grade forming the ground floor level (Level 1) diaphragm. The piles are Ø450mm or Ø560mm 'Franki' piles which are constructed in six different variations. There are some pile configurations that have a double bulb at the bottom, likely to give it tension capacity, and there is further variation in the reinforcement cages. There are also four Ø900mm bored piles with a bell at the base.

The bay walls are founded on a 1500x2000mm deep reinforced concrete ring beam. This ring beam is interrupted at the Northern end where the relatively narrow basement runs through the ring beam. There are three Ø560mm piles under each bay wall, one under the centre of each buttress and one under the web. The bay walls that have their web area terminated at Level 2 (Bays 6(a) and 7(a)) have a group of three piles at the locations where the buttresses/columns of two bay walls meet. At the location where the ring beam is terminated, there are Ø900mm piles at each end instead of the Ø560mm piles.

The concrete squat walls at Level 1 have either 900mm wide foundation beams that run underneath them. The walls are then supported by the varying configurations of Ø450mm and Ø560mm piles. The exception is Wall 21, as the 600mm thick wall is supported on Ø900mm bored piles at either end, as well as a Ø450mm pile at the midspan.

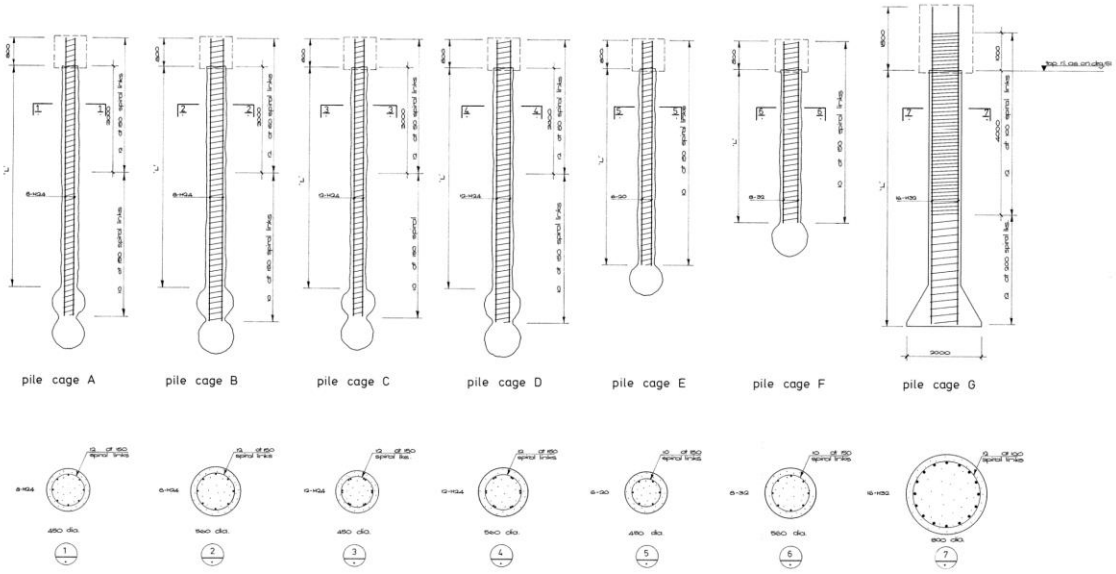


Figure C.4-55: Snapshot taken from drawing S29 of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing seven configurations of piles

The concrete masonry block walls are typically founded on slab thickenings and Ø450mm piles at their ends.

There are seven different pile configurations used in the Auditorium, comprising 69-Ø450mm and 49-Ø560 'Franki' piles and the 4-Ø900mm bored piles. Some piles have a double 'bulb' at the base, to give it more tensile capacity. However, since the longitudinal reinforcement is not shown on the drawings to continue in the second 'bulb', the Geotechnical Assessment Report considered the tension capacity only on the basis of the first 'bulb'. Typically, their transverse reinforcement comprises Ø12mm spirals at 150mm spacing in the first 3 meters of the pile. Further down the pile these transition to Ø10mm spirals at 150mm spacing. The unique features of each pile configuration are described below:

- Pile cage A is a Ø450mm 'Franki' pile and is primarily located under the concrete squat walls 1, 16 and 18. During seismic loading they are therefore directly part of the base-shear take-out mechanism system through these walls. They are reinforced with 8-Ø24mm longitudinal reinforcing bars of the higher reinforcement grade and have a second 'bulb' at the base. There are eleven of these piles in total in the Auditorium, but they have also been used in the independent staircase structure (see Section Appendix F).
- Pile cage B is a Ø560mm 'Franki' pile and is primarily located under the intersection of concrete squat walls 6 and 14 – at their corners. During seismic loading they are therefore directly part of the base-shear take-out system through these walls. They are reinforced with 8-Ø24mm longitudinal reinforcing bars of the higher reinforcement grade and have a second 'bulb' at the base. There are only two of these piles in total in the Auditorium, but they have more readily been used in the Renouf Foyer (see Section Appendix E).
- Pile cage C is a Ø450mm 'Franki' pile and is primarily located under the concrete squat walls 11, 14 and 16. During seismic loading they are therefore directly part of the base-shear take-out system through these walls. They are reinforced with 12-Ø24mm longitudinal reinforcing bars of the higher reinforcement grade and have a second 'bulb' at the base. There are ten of these piles in total in the Auditorium.
- Pile cage D is a Ø560mm 'Franki' pile and is primarily located under the concrete squat walls 1 and 14 – at their corners. During seismic loading they are therefore directly part of the base-shear take-

out system through these walls. They are reinforced with 12-Ø24mm longitudinal reinforcing bars of the higher reinforcement grade and have a second 'bulb' at the base. There are five of these piles in total in the Auditorium.

- Pile cage E is a Ø450mm 'Franki' pile and is primarily located under the basement walls and the ends of the concrete masonry block walls. They are considered to be primarily used as a gravity load path but during seismic loading they may indirectly be a part of the global base-shear take-out system. They are reinforced with 6-Ø20mm longitudinal reinforcing bars of the lower reinforcement grade. There are forty-eight of these piles in total in the Auditorium.
- Pile cage F is a Ø560mm 'Franki' pile and is primarily located under the perimeter ring beam that supports the bay walls. There is also one of this pile configuration under Wall 1. During seismic loading they are therefore directly part of the base-shear take-out system of the bay wall. They are reinforced with 8-Ø32mm longitudinal reinforcing bars of the lower reinforcement grade. There are forty-two of these piles in total in the Auditorium.
- Pile cage G is a Ø900mm bored pile and is located under wall 21 and under the perimeter ring beam where it is terminated by the basement structure. During seismic loading they are therefore directly part of the base-shear take-out system of Wall 21 and the bay wall. They are reinforced with 16-Ø32mm longitudinal reinforcing bars of the higher reinforcement grade. There are four of these piles in total in the Auditorium.

Base shear is taken out through a combination of pile lateral capacity and passive resistance against the foundation beams and basement structure. Friction capacity under the foundation beams and ground floor is considered to provide negligible capacity, particularly in liquefied ground conditions.

Pile strength and capacity

The piles have been analysed by Tonkin+Taylor Limited using the LPile software. This determines the behaviour and internal forces of the pile when forces or displacements are applied on the pile from the ground or external forces from the superstructure. Ultimately, this evaluates the geotechnical lateral capacity of the piles. The analysis has considered four bounding cases for each of the seven pile configurations, including those for fixed-head and pinned-head boundary conditions at the top of the pile. The analysis was subject to varying levels of displacements and forces at the head of the pile during undegraded and liquefied ground conditions. These were also combined with the effects of kinematic cyclic displacements of the ground and lateral spread of the ground.

The LPile analysis considered each pile individually which resulted in them showing 'P-delta'-type failure behaviour at large displacements, resulting in degrading capacities and non-convergence in the analysis. However, the passive pressures that act on the foundation beam and basement structure can provide a lateral restraint and therefore stability against P-delta effects. As such, the piled foundations of the global system are more likely to show a more linear behaviour after reaching their ultimate capacity.

The piles' structural capacities were determined in accordance with Chapters C4 and C5 of the Engineering Assessment Guidelines, which used the axial load from gravity load cases in the ETABS20 analysis model. The flexural capacity of the section was analysed in Response2000. For the shear capacity of the piles, the reference of Section 4.7.3 of Priestley et al. (2007) was adopted. It has considered a shear strength reduction factor of 0.85 to align with the requirements Section C5.5.5.1 of the Engineering Assessment Guidelines.

In the ETABS analysis model, the piles are considered with a fixed-head boundary condition if they act in the plane of the foundation beam or the wall system that they are supporting. If they act out of the plane of the foundation beam or the wall, they are considered as a pinned-head.

Pile inelastic displacement capacity

For the inelastic rotation capacity, the Geotechnical Assessment Report gave no geotechnical limit. As such, guidance was sought in the Engineering Assessment Guidelines which refer to Module 4: Earthquake Resistant Foundation Design of the geotechnical guidance documents by NZGS and MBIE. In Section 6.3 for 'Deep Foundation Design', this document suggests plastic hinge strain limits for piles in Table 6.1. However, these are considered to not be directly applicable for the assessment of the probable behaviour of the foundations, as they are intended for design. The footnote in Module 4 specifically states that they are "… conservative for the ULS case to permit re-use after both SLS and ULS earthquake.", which doesn't align with the Life-Safety objectives of the Engineering Assessment Guidelines. Also, they do not specifically reference concrete piles, only prestressed concrete piles. The recommendations in Module 4 are based on the 2010 version of the 'Port of Los Angeles (POLA) Code for Seismic Design, Upgrade and Repair of Container Wharves' and as such this document has been further considered for determining the inelastic rotation capacities of the piles.

This general approach is considered applicable to the Michael Fowler Centre, as the soil-structure interaction predicts the movement of a relatively large mass on a series of flexible piles.

The plastic strain limits that the Engineering Assessment Guidelines suggests for concrete columns, either through the direct-rotation method or the plastic curvature method, only allow up to 0.5% inelastic rotation capacity, limited by the strains in the concrete section, up to 0.006(mm/mm). It is our opinion that, to meet the Life Safety objectives of the Engineering Assessment Guidelines, this is overly conservative for concrete piles. As the piles are embedded in the ground, they experience a lot of restraint against instability. This is reflected by the strain limits provided in POLA.

The POLA document discusses the plastic strain limits for piles in Section 1.3.2 and outlines three design levels. To meet the Life Safety objectives of the Engineering Assessment Guidelines, the 'Design Earthquake (DE)' is considered the most applicable. This design level is described in the US codes for design and assessment as "… the earthquake effects that are two-thirds of the corresponding risk-targeted maximum considered earthquake (MCER) effects".

Table 1-2 of this document, which is copied in Figure C.4-56, gives similar limits for the ultimate strains in the reinforcement (8%), while not considering any strain limits in the head of the piles. However, a strain limit of 6% has been considered as an upper bound. This retains the intent of the Engineering Assessment Guidelines, which specify the 6% strain limit.

For the hinges that may form in the ground, there is a limit on concrete strain of $\varepsilon_c \leq 0.005 + 1.1 \cdot \rho_s \leq 0.008$.

Table 1-2: Seismic Piles Top Plastic Hinge Strain Limits

Pile	Design Level		
	OLE	CLE	DE
Solid Concrete Piles	$\varepsilon_c \leq 0.005$ $\varepsilon_{sd} \leq 0.015$	$\varepsilon_c \leq (0.005 + 1.1\,\rho_s) \leq 0.025$ $\varepsilon_{sd} \leq 0.6\varepsilon_{smd} \leq 0.06$	ε_c [(b)] $\varepsilon_{sd} \leq 0.8\varepsilon_{smd} \leq 0.08$
Round Hollow Concrete Piles [(a)]	$\varepsilon_c \leq 0.004$ $\varepsilon_{sd} \leq 0.015$	$\varepsilon_c \leq 0.006$ $\varepsilon_{sd} \leq 0.4\varepsilon_{smd} \leq 0.04$	$\varepsilon_c \leq 0.008$ $\varepsilon_{sd} \leq 0.6\varepsilon_{smd} \leq 0.06$
Steel Pipe Piles: Concrete Plug with Dowels	$\varepsilon_c \leq 0.010$ $\varepsilon_{sd} \leq 0.015$	$\varepsilon_c \leq 0.025$ $\varepsilon_{sd} \leq 0.6\varepsilon_{smd} \leq 0.06$	ε_c [(b)] $\varepsilon_{sd} \leq 0.8\varepsilon_{smd} \leq 0.08$

[(a)] If the interior of hollow concrete piles is filled with concrete, all strain limits shall be the same as for solid concrete piles.
[(b)] No limit.

Figure C.4-56: Plastic strain limits according to the 2010 POLA Seismic Code

These displacements limits are considered to correspond to the lateral load capacity of the piles. At displacements that exceed this capacity the lateral load carrying capacity is considered to drop significantly. For the purpose of this assessment, the residual capacity is considered to be 20% of the total lateral capacity.

The piles are however considered to be able to be part of the gravity load resisting system up to margin of a factor 2 on this displacement limit, which is considered as the criteria for scenarios involving large ground displacements and the consideration of a geotechnical/structural step change. The background to this is further described in Section C.4.4.4.

Passive pressures on foundation beams

The Geotechnical Assessment Report has recommended a series of passive pressures that could develop against the foundation beams and basement structure. This information is copied in Figure C.4-57 for reference.

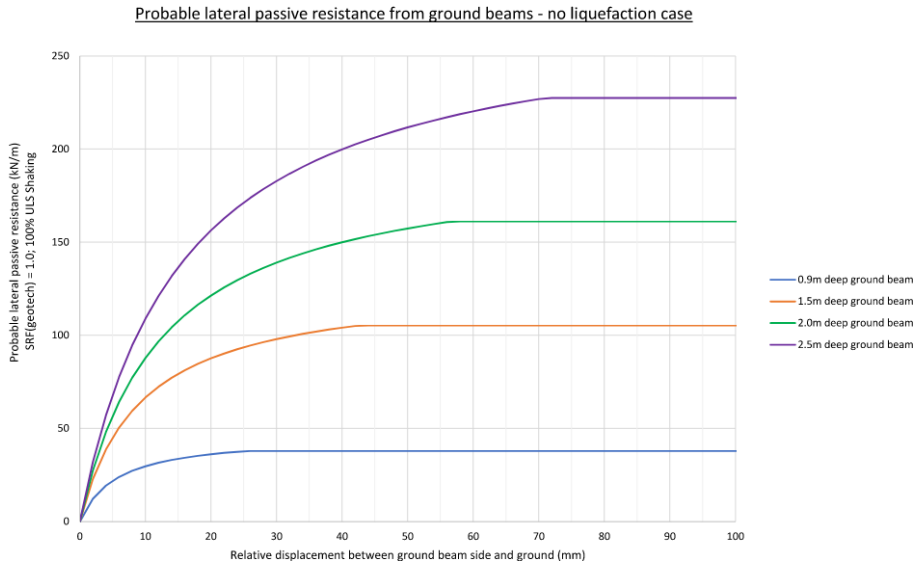


Figure C.4-57: Probable passive resistance against foundation beams and basement structure

The total passive pressure that could develop in between the perimeter ring beam of the Auditorium could potentially have been limited by a shear sliding plane that could develop across the footprint of the structure. However, this scenario was considered and was determined not to govern the passive pressures.

Figure C.4-58 shows an example on how the total passive resistance is accounted for in the transverse (Z) direction of the Auditorium. Noting that the colours in figure match those of Figure C.4-57.

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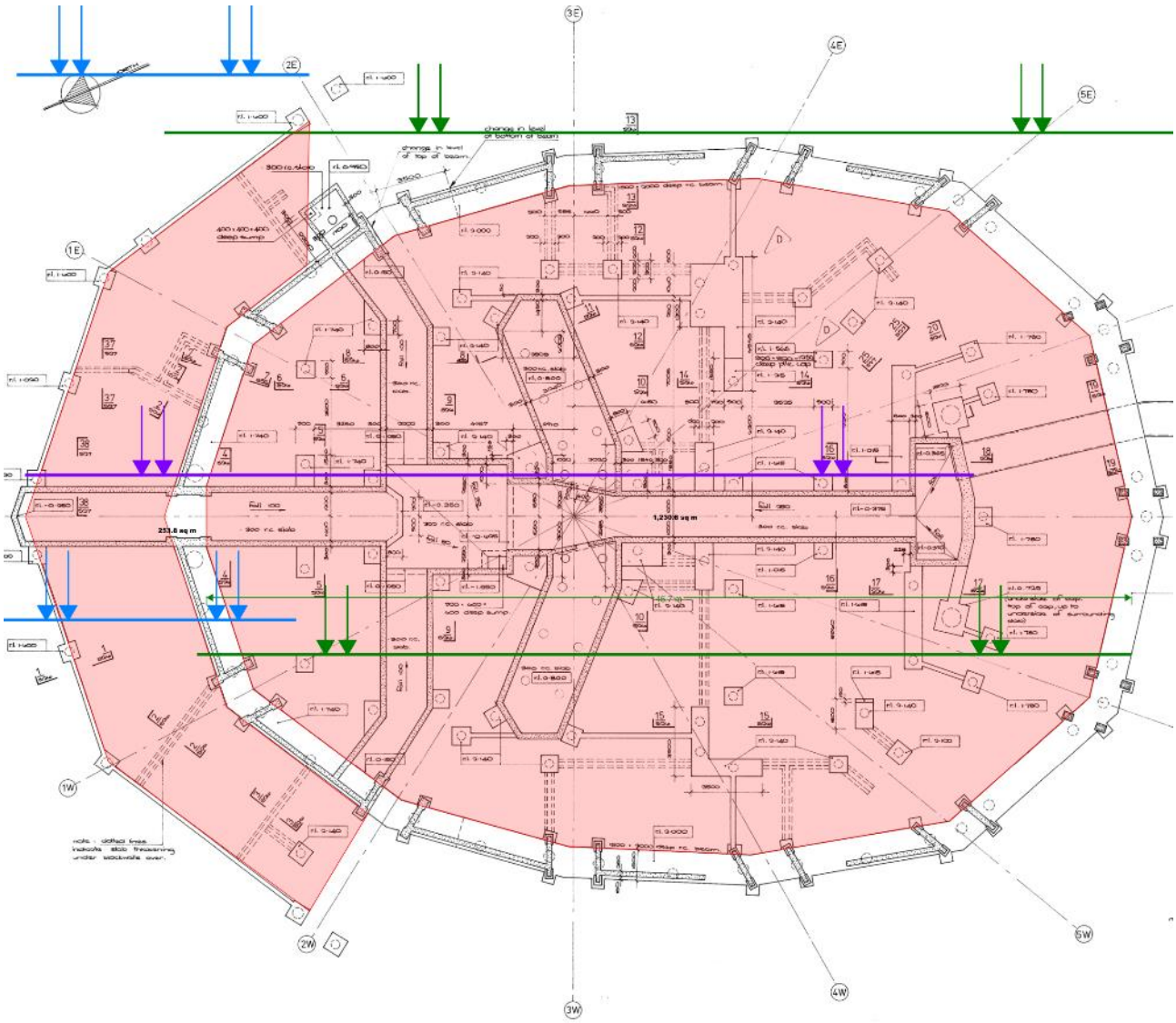

Figure C.4-58: Areas of foundation beams considered to provide passive resistance in the transverse (Z) direction

Passive pressures against basement structure

For the basement structure it could be considered that the walls may deform out-of-plane due to the passive pressures developing against the walls. However, the basement structure is considered to be well tied together with a reinforced basement floor that can redistribute the lateral loading to the return walls and to the pile system. Whilst inelastic deformations may occur in the global system during large displacements, and therefore also in the basement structure, it is not considered to reduce the passive pressures against it.

Displacement-based assessment approach

The foundation system and base-shear take-out capacity have been assessed using a displacement-based approach. The pile system and the passive pressures require 50-100mm of lateral displacements to be fully engaged. The level of ductility and damping is therefore best accounted for by comparing it against the acceleration-displacement response spectrum (ADRS).

For the damping of the ADRS, reference is made to the NZTA Bridge Manual 3rd Edition and Priestley et al. (2007).

The NZTA Bridge Manual gives the equivalent viscous damping ratio's for various component actions. For 'piled foundations where hinges develop in piles' an equivalent viscous damping ratio is given in accordance with the equation below. It is related to the member (pile) ductility.

$$\xi_{i,piles} = 0.10 + 0.565 * (\frac{\mu_m - 1}{\mu_m \cdot \pi})$$

Passive pressures against the foundation beams may have a much smaller damping contribution. The building displaces the ground on the first cycle of seismic shaking and on the returning cycle the passive pressures take that initial displacement to engage. Therefore, it is our opinion that they can be expected to act with a damping of no more than the system inherent damping of ξ_o = 5%.

To combine the equivalent viscous damping contribution of the piles and the passive pressures, reference is made to equation (5-23) of the NZTA Bridge Manual. Copied below for reference.

$$\xi_e = \frac{\sum_{i=1}^{n}(V_i \cdot \delta_i \cdot \xi_i)}{\sum_{i=1}^{n}(V_i \cdot \delta_i)}$$

Scenarios recommended by Geotechnical Assessment Report

The Geotechnical Assessment Report has outlined four scenarios that should be considered in assessing the foundation system of the building. These are:

- **Start of earthquake, with no liquefaction** and therefore undegraded ground conditions. This scenario needs to consider 100% of the base shear of the structure.
- **Zones of liquefaction triggered across the site, but no lateral ground movement.** This scenario captures the onset of liquefaction being formed across the site. The building may still experience the inertia associated with undegraded ground properties, but locally (~20% of the site) the piles may be in liquefied conditions, whereas the other piles (~80%) remain in non-liquefied undegraded conditions.
- **Liquefaction triggered in the ground but no lateral ground movement**. This scenario needs to consider 100% of the base shear of the structure. During this scenario however, the structure is expected to experience a more flexible foundation and therefore a lower base shear.
- **Ground lurch including kinematic cyclic displacements**. In this scenario, the ground is expected to move 100mm in cyclic motion. It also needs to be able to resist 80% of the base shear.
- **Lateral spread of the ground**. In this scenario the ground is expected to slump towards the lagoon in the North-East of the structure. During this scenario, the structure should be able to resist 25% of the base shear.

The Geotechnical Assessment Report further notes that if the structure displaces less than the ground movement, than the structure will experience the passive pressures as an additional lateral load on the system rather than being able to use the passive pressures to resist the load.

C.4.4.1 Undegraded ground conditions

This assessment follows the recommendations of Scenario No. 1, as described in Table 9-1 of the Geotechnical Assessment Report (see Appendix B).

This assessment follows the recommendations of Scenario No. 1, as described in the Geotechnical Assessment Report (see Appendix B).

In undegraded ground conditions, all the pile configurations form their first plastic hinge at the pile head at around 40-70mm of lateral displacement. At this point on the graph, the yield curvature is estimated in accordance with equation 3.6a of Priestley et al. (2007) for circular concrete columns. The ductility demand that induces degradation of the plastic hinge zone is then considered as a ratio over this yield curvature. This results in the piles of the Pile Cage G configuration reaching a member curvature ductility of μ_{φ} = 13.0 around a displacement of 200-220mm. The LPile analysis capacities are considered to reasonably consistent beyond the ultimate capacity after the forming of the two plastic hinges.

In the transverse (Z) direction the piles are considered to behave in a flexure-shear manner which is first triggered at Pile Cage G. The piles develop excessive shear demands due to the combined geotechnical and flexural overstrength. This is graphically shown on Figure C.4-59. This graph shows the data for which LPile analysis converged, for various boundary conditions. The red-dashed line shows the shear capacity, degrading as the plastic hinges form.

All other pile configurations can develop their ultimate capacity before their push-over curve interacts with the shear degradation curve. Therefore, for ease of calculation, the ultimate capacity of Pile Cage G is not further reduced in the analysis model. As can be read from Figure C.4-59, the flexure-shear occurs around lateral displacements 140-170mm – as the LPile push-over curve intersects with the shear capacity at these displacements.

In the longitudinal (X) direction of the Auditorium, the piles of the Pile Cage G configuration are considered to be flexure-controlled as they are considered to be acting with pinned-head boundary conditions.

Pile Cage G is providing the primary gravity load path for Wall 21 and is also its most direct means of lateral load resistance. Also, at the extends of the perimeter ring beam, they provide gravity and lateral support to the bay walls of Bays 1(a). Therefore, these piles play a significant role in both the gravity support system and the lateral load resisting system. A flexure-shear failure would be likely to result in a large reduction of the lateral load resistance of Wall 21, which is heavily relied upon in the primary lateral load path. Also, it would reduce its ability to carry the gravity loads from the beams that run between the Level 1/2 and Level 2 floors – as these beams are not continuous across to Wall 18.

As this scenario considers large levels of seismic shaking, the structural integrity of the Auditorium relies on the load path through the sloping transfer diaphragm at Level 1/2 to remain intact. Therefore, the flexure-shear in Pile Cage G under Wall 21 could pose a life-safety hazard if the capacity is exceeded. Consequently, the capacity of the system is limited by the flexure-shear of Pile Cage G.

The total displacements of the pile capacities and the passive pressures are plotted against an ADRS. The piles contribute approximately 2/3^{rds} of the total capacity. The push-over curve considers a ductility of the foundation system of at least μ_{sys} = 2.0. This is based on a bilinearisation of the capacity curve and is considered to be a conservative lower bound estimate. This corresponds to an equivalent viscous damping of the foundation system of 12%.

The structure can achieve more capacity in the transverse (Z) direction of the Auditorium as there is a larger area of passive pressure that can be engaged. In the undegraded ground conditions the foundation system can achieve a base-shear take-out capacity of 61MN in the transverse (Z) direction and 50MN in the longitudinal (X) direction. However, the foundation system has more displacement capacity in the longitudinal (X) direction, giving it a seismic score of **95%NBS$_{IL3}$**. The transverse direction scores **90%NBS$_{IL3}$** respectively.

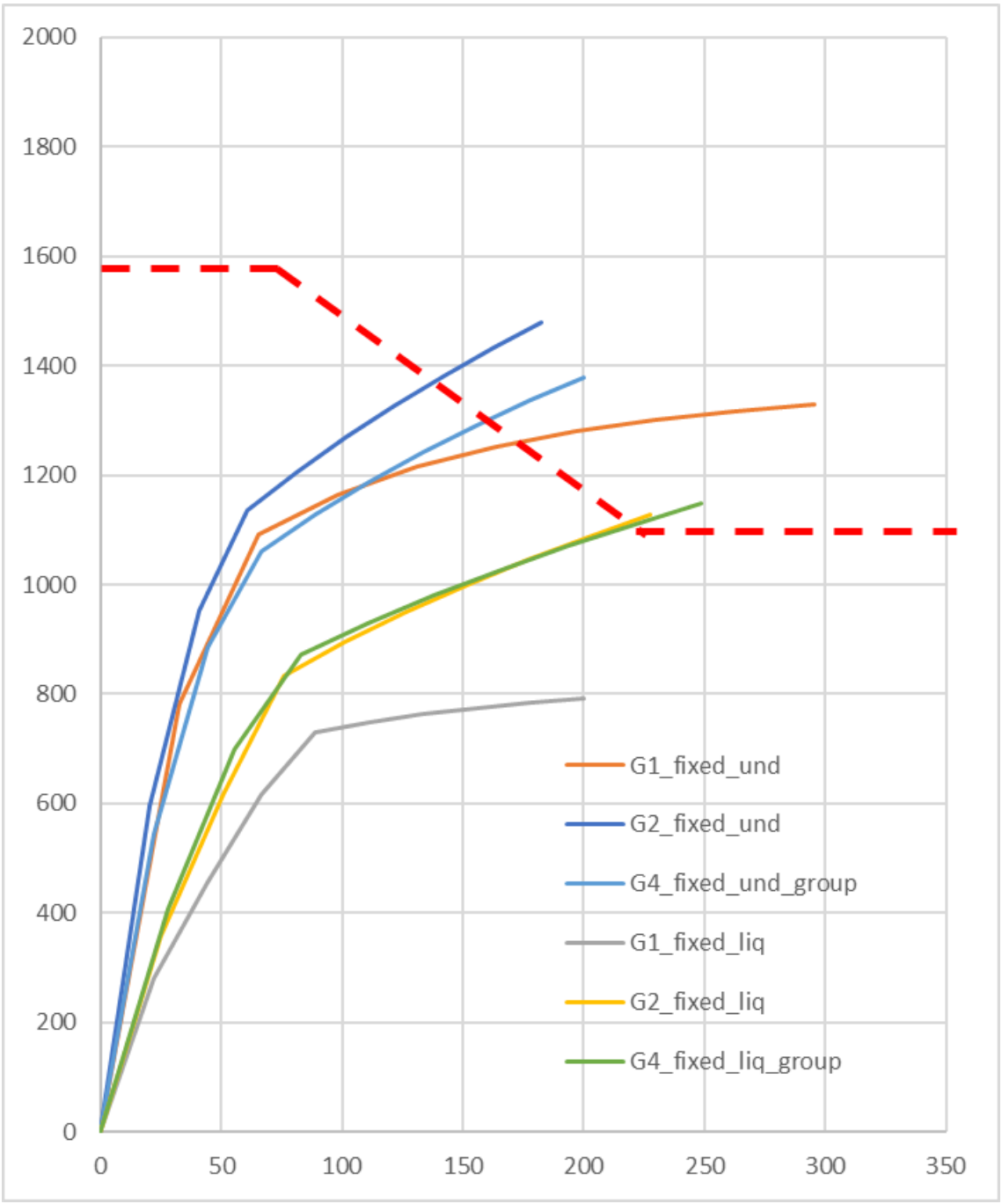

Figure C.4-59: Push-over curve and shear capacity curve of Pile Cage G configuration

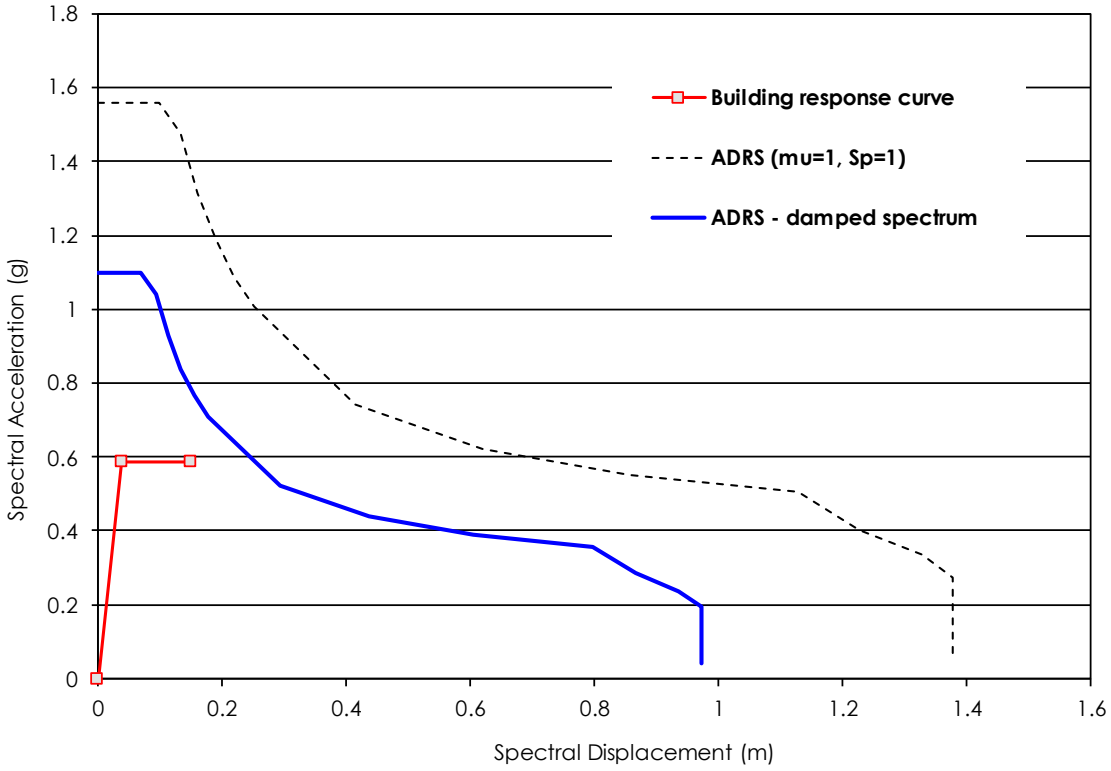

Figure C.4-60: Push-over curve of the foundation system (in undegraded ground conditions) in the transverse (Z) direction plot against the damped ADRS

C.4.4.2 Liquefied ground conditions

The Geotechnical Assessment Report describes that liquefaction is expected to be triggered at seismic shaking that corresponds to 45%NBS_{IL3}. The consequences of the liquefaction are listed below:

- Lateral cyclic ground displacements, in the order of 100mm, dependent on the level of seismic shaking, which will impose lateral kinematic passive pressures onto the foundation beams, basement structure and piles during shaking.
- Lateral spread of the ground, in the order of 100's of mm, dependent on the level of seismic shaking, which will impose lateral kinematic passive pressures onto the foundation beams, basement structure and piles during shaking.
- Sand boils, which form as the pore pressure in the liquefied ground forces the groundwater to rise up to the surface. This will likely lead to local tilting and sinking of the ground in the crust layer.
- Free-field settlements of the ground, in the order of 100mm, which is expected all over the site of the Michael Fowler Centre.
- Buoyancy or uplift pressures on the elements of the structure that are buried below the ground water level.
- Much lower soil strength and softer lateral and vertical ground stiffness around the piles in the liquefied ground layers.
- Lower passive pressures (and resistance) that can be considered during liquefied ground conditions.
- Locally, piles may lose all axial load carrying capacity if founded above a liquefiable pocket of ground (considered unlikely as per the Geotechnical Assessment Report).

It was also confirmed by Tonkin+Taylor Limited that the piles are the only system that can provide a reliable load path for gravity loads in liquefied ground conditions. Whilst there are large foundation beams in the structure that connect the majority of these piles, it was concluded that these are typically founded below the groundwater level and therefore see large (100's of mm's) settlements before they achieve reliable bearing pressures. This level of settlements would likely result in a structural step-change behaviour – although this is difficult to quantify.

C.4.4.2.1 Zones of liquefaction, but no lateral ground movement

This assessment follows the recommendations of Scenario No. 1a, as described in Table 9-1 of the Geotechnical Assessment Report (see Appendix B).

As the earthquake progresses, liquefaction starts to form around the site and locally starts to degrade the soil in pockets. At this point, the building is experiencing 100% of the base shear corresponding to non-liquefied ground properties. However, due to the localised pockets of liquefied ground, there is an estimate of 20% of the piles and foundation beams that respond with their capacity in liquefied ground conditions.

Compared to Scenario No. 1 (start of earthquake, no liquefaction) there is an overall reduced capacity. This was analysed to correspond to giving it a seismic score of **85%NBS$_{IL3}$** in the longitudinal direction and **80%NBS$_{IL3}$** in the transverse direction of the building.

The consequence of sand boils, free-field settlements and buoyancy effects on the buried structures are not considered to affect the base-shear take-out capacity or the gravity load carrying capacity of the foundation system. However, they can provide local damage to the ground floor and the basement structure. Liquefaction triggered, but no lateral ground movement

This assessment follows the recommendations of Scenario No. 2, as described in Table 9-1 of the Geotechnical Assessment Report (see Appendix B). At the onset of liquefaction, the analysis should consider 100% of the base shear. During this scenario, the piles and passive pressures however respond to a much softer ground, which results in a longer effective period of the structure and lower base shear demands.

In the liquefied ground conditions, the piles are typically considered to behave in a flexure-controlled manner. However, as shown in Figure C.4-59, the Pile Cage G configuration acts in a flexure-shear mechanism at the onset of liquefaction. Whilst the lower bound shear capacity and the geotechnical pile capacity provide similar capacities, it is reiterated that the LPile output data does not consider any flexural overstrength developing in the piles. And therefore, full inelastic behaviour from flexure cannot be guaranteed.

In liquefied ground conditions all other pile configurations can develop their ultimate capacity before their push-over curve interacts with the shear degradation curve. Therefore, for ease of calculation, the ultimate capacity of Pile Cage G is not further reduced in the analysis model.

As this scenario considers large levels of seismic shaking, the structural integrity of the Auditorium relies on the load path through the sloping transfer diaphragm at Level 1/2 to remain intact. Therefore, the flexure-shear in Pile Cage G under Wall 21 could pose a life-safety hazard if the capacity is exceeded. Consequently, the capacity of the system is limited by the flexure-shear of Pile Cage G.

The total displacements of the pile capacities and the passive pressures are plot against an ADRS. The piles contribute approximately 2/3^{rds} of the total capacity. The push-over curve considers a ductility of the foundation system of at least μ_{sys} = 2.0. This is based on a bilinearisation of the capacity curve and is considered to be a conservative lower bound estimate. This corresponds to an equivalent viscous damping of the foundation system of 12%.

Similar to the undegraded ground conditions, the structure can achieve more capacity in the transverse (Z) direction of the Auditorium as there is a larger area of passive pressure that can be engaged. In the liquefied ground conditions the foundation system can achieve a maximum base-shear take-out capacity of 33MN in the transverse (Z) direction and 28MN in the longitudinal (X) direction. However, the foundation system has more displacement capacity in the longitudinal (X) direction, giving it a seismic score of **75%NBS$_{IL3}$**. The transverse direction scores **70%NBS$_{IL3}$** respectively.

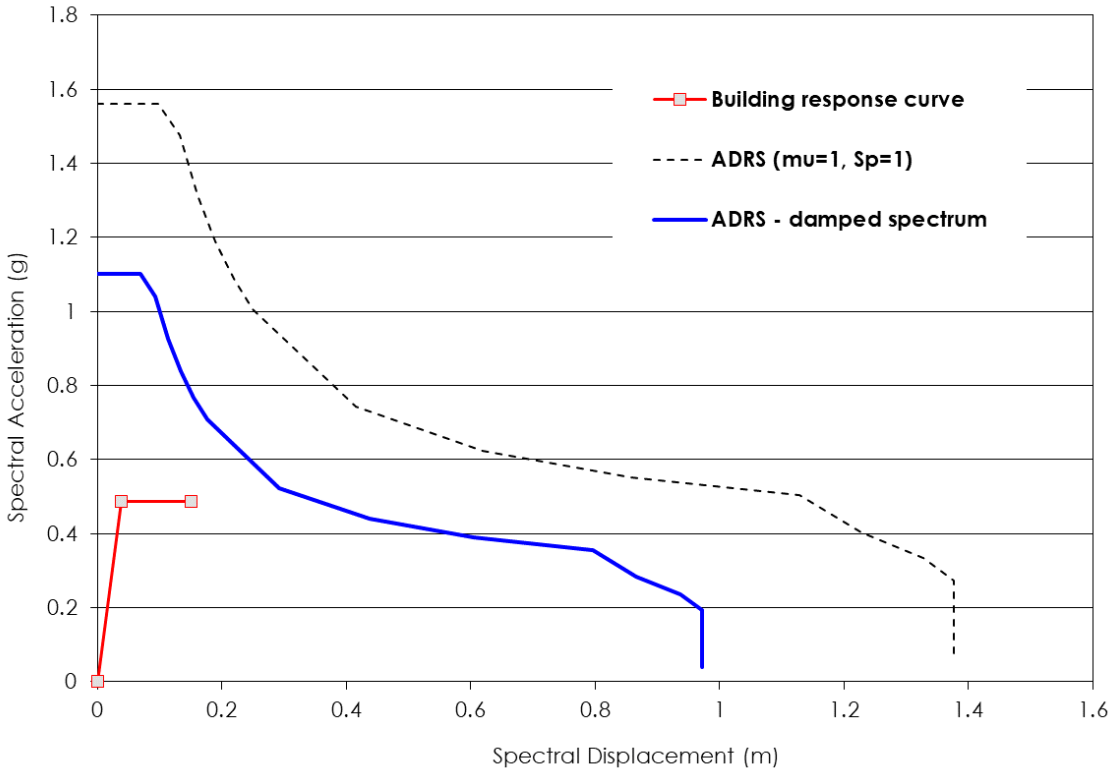

Figure C.4-61: Push-over curve of the foundation system (in liquefied ground condition) in the transverse (Z) direction, plot against the damped ADRS

The consequence of sand boils, free-field settlements and buoyancy effects on the buried structures are not considered to affect the base-shear take-out capacity or the gravity load carrying capacity of the foundation system. However, they can provide local damage to the ground floor and the basement structure.

The potential of a liquefied pocket forming under a single pile, or a small pile group has not been considered to affect the seismic rating of the structure. However, as a sensitivity study, a calculation has been carried out on the shear capacity and potential deformation capacity of the perimeter ring beam under the bay walls of Bays 1(a). If the piles with configuration 'G' lose their axial load carrying capacity, the beam would effectively cantilever 3.5m outwards and this would induce a shear load >6,000kN in the perimeter ring beam. However, the perimeter ring beam is well reinforced and has ample shear capacity (>12,000kN) to deal with that load and the elastic deformations would result in less than 0.1% additional drift.

C.4.4.2.2 Cyclic displacement occurs, during shaking

This assessment follows the recommendations of Scenario No. 3, as described in Table 9-1 of the Geotechnical Assessment Report (see Appendix B).

When the structure is in liquefied ground conditions, it can experience cyclic displacements of 100mm acting on the foundation beams. This scenario is combined with 80% of the base shear of the structure in liquefied ground conditions.

In this scenario, the foundation system is essentially considered to behave in a similar manner as to the 'onset of liquefaction' scenario. However, the main difference is that the cyclic displacements of 100mm don't push the structure the same amount. The LPile analysis that was carried for the Geotechnical Assessment Report advises that the piles may likely only see 10-20mm of movement as they are exposed to the ground movements. At this point the cyclic displacements will impose a passive pressure on the foundation beams, increasing the lateral load on the piles.

The piles are however already close to their ultimate capacity and consequently, the structure has at this point no additional capacity that it can engage. It is therefore expected to take the additional loading from the passive pressures (and the superstructure) and move the residual amount up to the 100mm before it starts to engage passive resistance on the other end. This is shown in Figure C.4-62, which shows the base-shear demand (dashed green line) at 45%NBS_{IL3}, representing the loads from passive pressures until it overcomes the cyclic displacement demands. At that point, shown at 100mm displacement, the passive pressures no longer provide lateral loading on the structure, but instead provide additional resistance (shown by the brown line). The total capacity is shown by the blue line and is considered as the push-over curve.

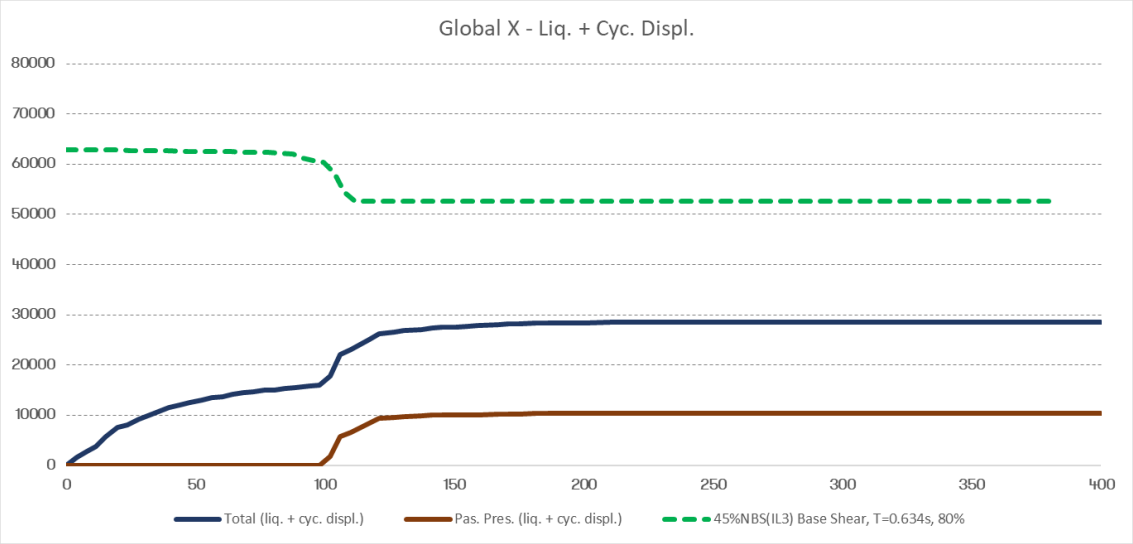

Figure C.4-62: Multilinear plot of the capacity curve of the structure during the cyclic displacements

This is adopted in the displacement-based assessment by moving the acceleration-displacement response spectrum the residual amount of 80mm the right. Effectively, accounting for the loss of displacement capacity that the structure needs to overcome.

As this scenario considers large levels of seismic shaking, the structural integrity of the Auditorium relies on the load path through the sloping transfer diaphragm at Level 1/2 to remain intact. Therefore, the flexure-

shear in Pile Cage G under Wall 21 could pose a life-safety hazard if the capacity is exceeded. Consequently, the capacity of the system is limited by the flexure-shear of Pile Cage G.

The total displacements of the pile capacities and the passive pressures are plot against an ADRS. The piles contribute approximately 2/3^{rds} of the total capacity. This assessment considers a global system ductility of μ_{sys} = 2.0, similar to the 'onset of liquefaction' scenario, however, the damping was considered to have less effect. Whilst it is difficult to predict the actual level of damping, this is considered to be a conservative estimate. This corresponds to an equivalent viscous damping of the foundation system of 10%.

The structure can achieve more capacity in the transverse (Z) direction of the Auditorium as there is a larger area of passive pressure that can be engaged. However, in the longitudinal (X) direction, the structure has more displacement capacity as the piles of configuration 'G' are not flexure-shear controlled. In the liquefied ground conditions the foundation system can achieve a maximum base-shear take-out capacity of 33MN in the transverse (Z) direction and 28MN in the longitudinal (X) direction. However, the foundation system has more displacement capacity in the longitudinal (X) direction, giving it a seismic score of **55%NBS$_{IL3}$**. The transverse direction scores **50%NBS$_{IL3}$** respectively.

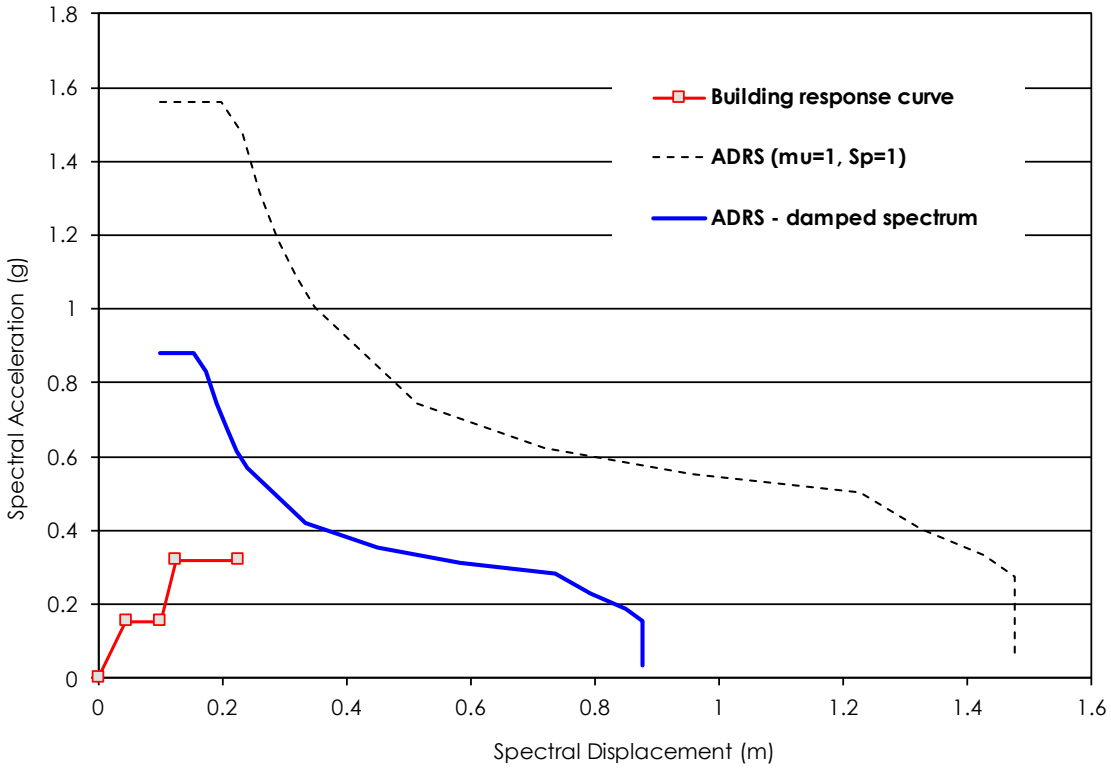

Figure C.4-63: Push-over curve of the foundation system (in liquefied ground conditions, during kinematic cyclic displacements) in the transverse (Z) direction, plot against the damped ADRS (at 80%NBS$_{IL3}$)

C.4.4.2.3 Lateral spreading occurs, towards end of shaking, possibly post-shaking

This assessment follows the recommendations of Scenario No. 3, as described in Table 9-1 of the Geotechnical Assessment Report (see Appendix B).The Geotechnical Assessment Report has provided contour plots which present the probable scenario of lateral spread that can occur at the site following the liquefaction of the ground, shown in Figure C.4-64 for reference. This shows that at the north-eastern extent

of the site, the shaking at 45%NBS_{IL3} could lead to a lateral ground displacement of ~315mm, whilst on the southern side of the Michael Fowler Centre, these lateral displacements could be ~155mm.

The quantification of lateral spread is difficult to predict and therefore Tonkin+Taylor Limited have indicated that these could range between 1/2x and 2x the amount predicted in their report. This is referred to in the Geotechnical Assessment Report as an 'uncertainty factor'. For this assessment, the probable estimates of lateral displacements of the ground are considered to determine the seismic score of the foundation system. The adoption of the 'uncertainty factor' is included as a margin to collapse in assessing for geotechnical and structural step change. This approach is further described in Section C.4.4.4.

The lateral spread of the ground is estimated from the contours plot to occur at an approximate 45 degrees angle to the central axis of the Auditorium. For the purpose of the calculations in this section, the lateral ground displacements in the longitudinal (X) direction and transverse (Z) direction are determined at the bounds of 30 degrees and 60 degrees – giving a factor of cos(30) = 0.866 on the displacements in those principal directions.

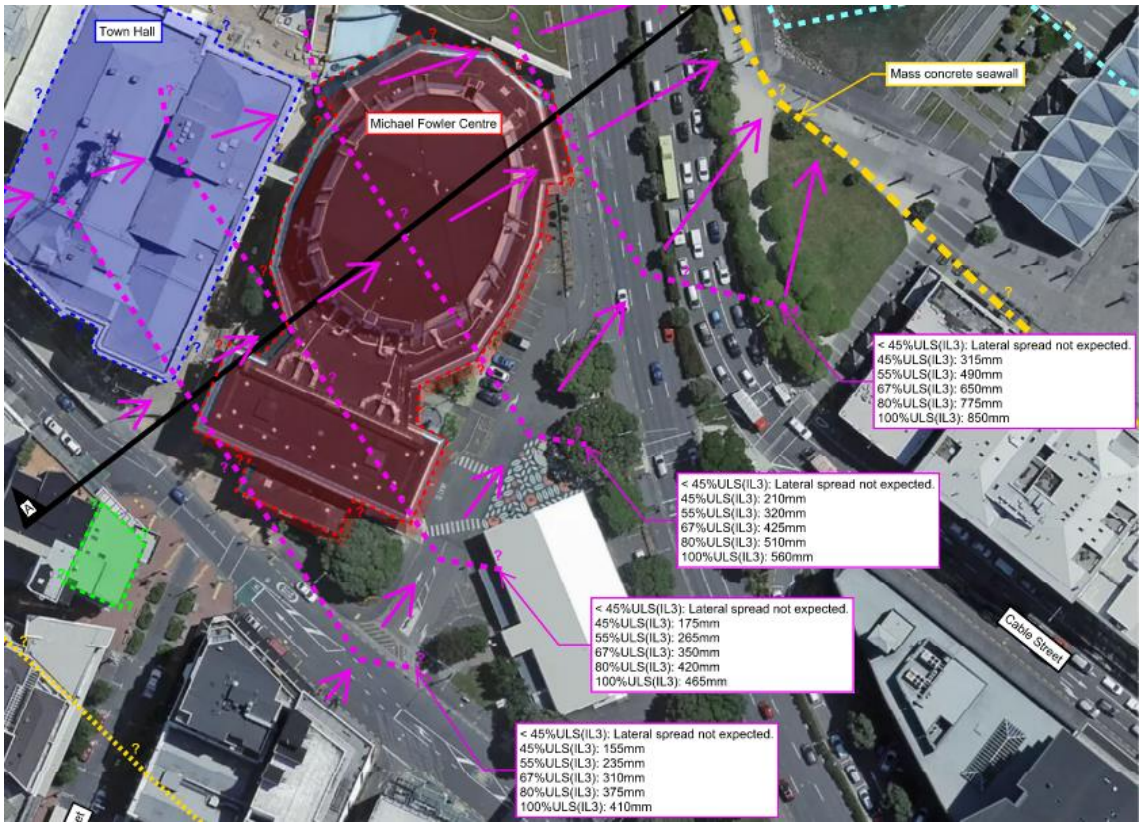

Figure C.4-64: Contours plot of the probable lateral spread - taken from the Geotechnical Assessment Report

The assessment of the foundation system in this fourth scenario considers liquefied ground properties in addition to the lateral ground displacement acting on the piles and the foundation beams. Similar to the scenario with the cyclic displacements, the structure is expected to initially move less than the total amount of lateral spread. This is the expectation, as the piles, the foundation beams and the basement structure initially provide a level of restraint against the movement and the pressures at the front of the structure (in the backstage area) may slump away underneath the foundation beams. It is expected that this area will

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form a crack in the surface of the ground, somewhere between the foundation beams of Wall 1 and the City-to-Sea bridge, which is just North of the site.

At the onset of lateral spread of the ground, the structure is expected to move along with the average line across the Auditorium (shown as 210mm displacements at 45%NBS_{IL3} on the contours plot). No LPile analysis has been undertaken to review whether the piles could restrain the displacement of the Auditorium, as they are likely to well into inelastic displacements. Therefore, it is assumed that the structure equally moves 210mm.

Though, at this point, the piles will have reached their ultimate capacity and are unable to provide additional lateral load resistance. Since this scenario needs to be combined with seismic shaking at 25% of the base shear demand of the superstructure, the structure will likely see this in additional displacement of the foundations until they are able to engage passive pressure again to resist the 25% of the base shear demand. The ground in front of the building, closer to lagoon is expected to have slumped and displaced 315mm laterally. Therefore, the passive pressures on that face of the foundation beam are expected to have reduced significantly. However, the foundation system is still able to build up passive pressures against the internal sides of the ring beam, the foundation beams and the basement structure. It is calculated that the structure would need to move an additional 40mm is liquefied ground properties, to develop sufficient passive pressures to stabilise the 25% base shear associated with this lateral spread scenario. The building can therefore be expected to displace an amount of 250mm.

The piles of configuration 'G' only have a displacement capacity of 225mm in the transverse (Z) direction from flexure shear. Therefore, the lateral spread of the ground combined with the 25% of the base shear demand from the superstructure will exceed that capacity. However, the 25% base shear of the structure considering liquefied ground properties does not suggest that the superstructure is subject to large levels of shaking. It would only correspond to 12%ULS seismic loading in undegraded ground properties. At this point the primary lateral load resisting system would be elastic.

Should Pile Cage G under Wall 21 exceed the limit for flexure-shear, then there is an adjacent Ø450 'Franki' pile of configuration 'E' that could provide the necessary gravity support to the structure above. For the loads in the transfer diaphragms at Level 1/2 and Level 2, there is a load path to the other squat walls 11, 14, 16 and 18, which provides resilience to the primary lateral load resisting system. The gravity support at the bay walls of Bays 1(a) is guaranteed by the perimeter ring beam, which is strong enough to redistribute the additional gravity load to the other piles along the beam, without overly deflecting. Concluding that for the lateral spread scenario, the exceedance of the flexure-shear limits of the Pile Cage G configuration is not limiting the seismic score of the foundation system. However, the structure is unlikely to tolerate larger displacements.

The other pile configurations have displacement capacities of 300-450mm based on the strain limits (6%) in the longitudinal reinforcing. Since it will require some displacement before the full passive pressures are engaged that will allow it to take-out the 25% base shear, the displacement capacities are not considered to be exceeded. Since the probable displacement capacity of the majority of the piles exceeds the probable demand for lateral spread, the piles are expected to be able to provide gravity support to the structure.

In the push-over plot that is compared against the ADRS, there is no capacity of the pile system included as the probable capacity is expected to be exceeded. Based on the passive pressures alone, the system can develop capacities of 10MN in the longitudinal (X) direction and 13MN in the transverse (Z) direction. The push-over plot follows the capacity curve of the passive pressures. This assessment shows that at a system damping of 5% (passive pressures only), the structure only needs to move an additional 40-45mm to activate sufficient passive pressures to achieve a rating of 45%NBS_{IL3} – not even engaging the maximum achievable passive pressures that can develop on the inside of the structure.

On this basis, the piles are likely to experience flexure-shear or strain limits that exceed their ultimate capacities once the lateral spread is induced. However, the structure can still take-out the base shear through the passive pressures and maintain gravity support through the piles.

However, it is deemed unlikely that the structure is capable of dealing with the effects of seismic shaking that is much larger than 45%NBS_{IL3}.

Consequently, the foundation system score of the foundation system is limited at **45%NBS_{IL3}**.

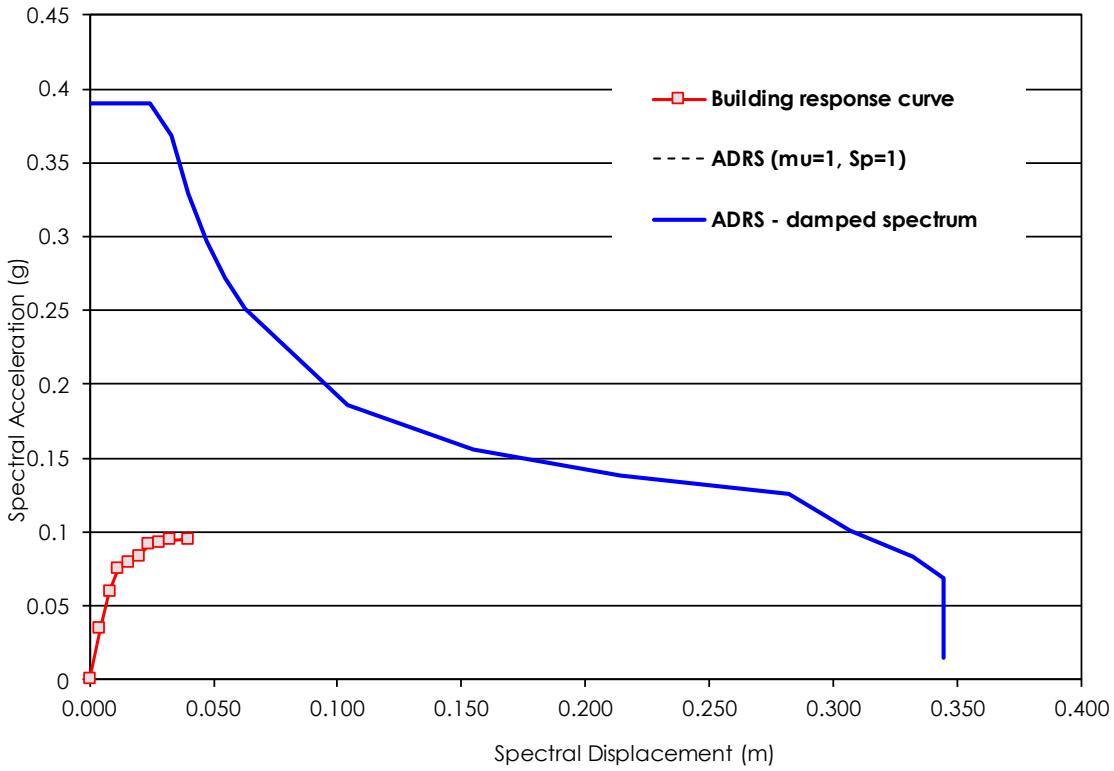

Figure C.4-65: Push-over curve of the foundation system in the transverse (Z) direction, plot against the damped ADRS (at 25%NBS_{IL3}) in the lateral spread scenario

C.4.4.2.4 Lateral Stretch Imposed on the Ground Floor Diaphragm

As described in the previous subsection, it is expected that the structure of the Auditorium will generally move as an 'island' along with the lateral spreading ground. The most probable scenario is that the structure would want to follow the ground pressures that act on the Southern end and the Eastern and Western sides of the structure, but that the ground near the North of the structure, closest to the free face (the seawall near the lagoon), will want to stretch further. As this happens, there are passive pressures developing against the foundation beams in the Northern end of the structure that will want to stretch the foundation system and the ground floor.

If the structure is not capable of dealing with the consequences of this lateral stretch, this would likely induce a life-safety hazard, as the structure would start to experience displacements that would exceed the seating limits of floor and roof units or exceed the displacement capacity of the precast infill panels. Also, there may other substantial consequences that would lead to the loss of integrity of the primary lateral load resisting system.

There are five distinct locations that are considered in the assessment, which are shown on Figure C.4-66. These consider the differential stretch and associated passive pressures in the:

- The ground floor of the backstage area
- The perimeter ring beam in the longitudinal (X) direction
- The ground floor of the lobby area
- The ground floor in the transverse (Z) direction of the structure where the perimeter ring beam is not continuous
- The perimeter ring beam in the transverse (Z) direction.

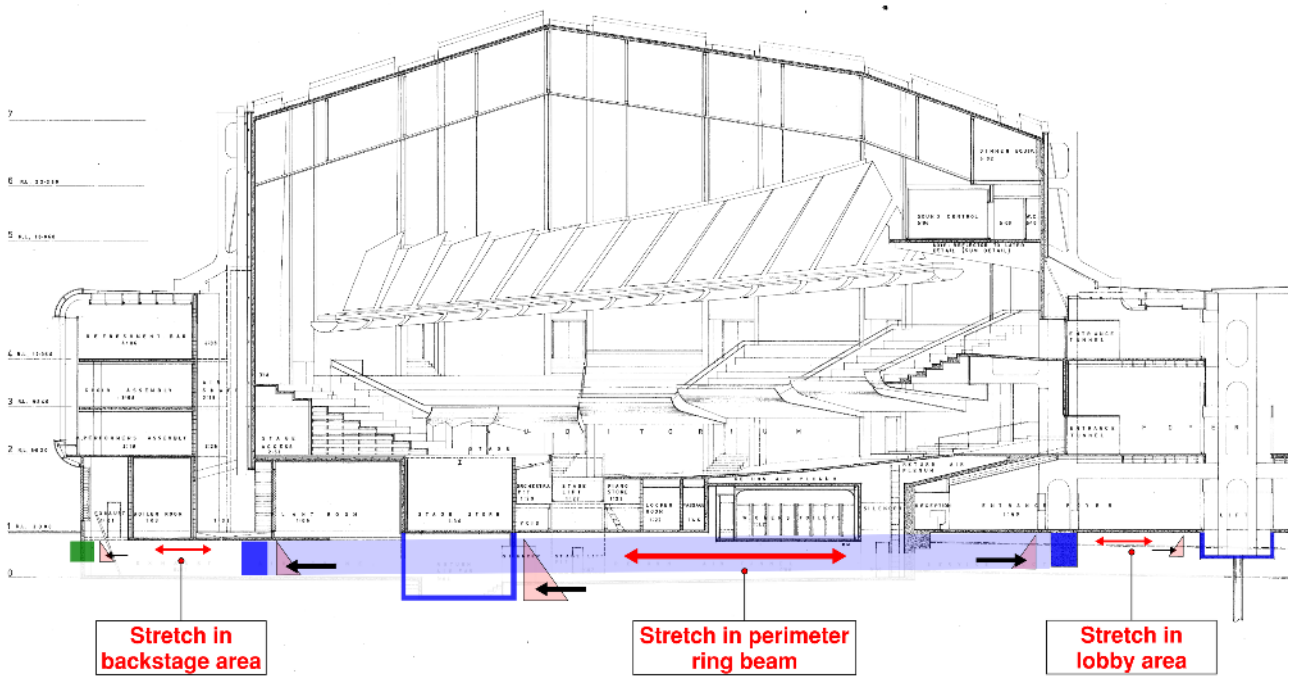

Figure C.4-66: Areas where differential lateral stretch may act on the structure

Backstage area (Wall 1)

The Northern (North-Eastern) face of the structure will see displacements in the ground that are larger than the average across the Auditorium. Assuming that the Auditorium structure can hold itself together, than the ground floor level will need to be able to tie back the passive pressures that develop against the 900x900mm foundation beam that supports Wall 1.

The Geotechnical Assessment Report recommended that the passive pressures against these beams could be up to 40kN/m. These passive pressures are irrespective of the ground conditions that are considered, as they are located above the groundwater level and so will provide pressures corresponding to undegraded ground conditions.

The ground floor level of the backstage area is a 120mm thick slab-on-grade which is reinforced with a single layer of Ø12mm bars of the higher reinforcing grade at 250mm spacing in both directions. The bars tie in with the foundation elements with well-developed hook ends. This reinforcement configuration can generate capacities up to 206kN/m.

As such, the general conclusion is that there a sufficient margin against the effects of lateral stretch in this area of the structure.

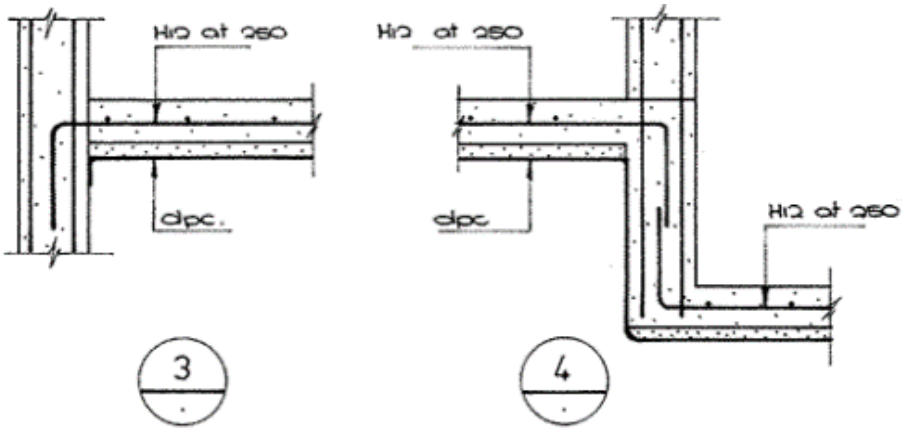

Figure C.4-67: Development of ground floor reinforcing into the wall elements.

Perimeter ring beam in the longitudinal (X) direction

Across the footprint of the Auditorium, the passive pressures on the Northern end can build up against those on the Southern end. Ultimately, the primary means to take out these effects of lateral stretch is the perimeter ring beam.

For this scenario, it is considered that the maximum amount of passive pressure that can build against the inside faces of the foundation beams and basement structure inside the Auditorium are acting 50% towards the North and 50% towards the South. Any other distribution would mean the structure would displace, as there would be an unbalance of loads and reactions. The contribution of the piles is not specifically considered, as these are expected to already have reached their ultimate capacities.

Considering the liquefied ground properties, the maximum passive pressures that the foundation beams were able to generate in the longitudinal direction of the building were 10MN.

The perimeter ring beam is longitudinally reinforced with 32-Ø32mm bars of the higher grade, while also having 8-Ø20mm bars of the lower grade. This gives tension capacity of 12.5MN per beam, and 25MN in total in the longitudinal (X) direction of the structure.

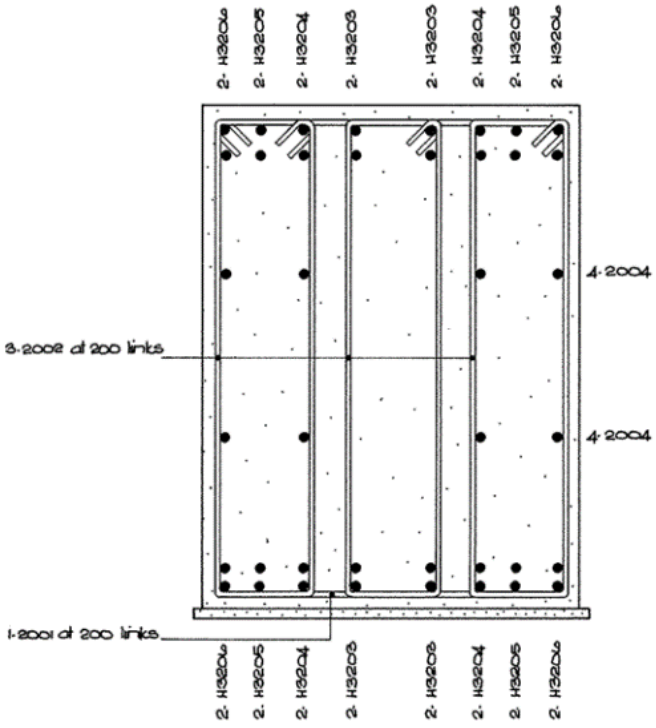

Figure C.4-68: Typical reinforcing of the perimeter ring beam

Lobby area

The ground floor level of the lobby area of the Auditorium in a 100mm slab-on-grade which is reinforced with 665-gauge (non-ductile) mesh. In accordance with the recommendations of the Engineering Assessment Guidelines (and specifically Yellow Chapter C5), the non-ductile mesh cannot be relied upon to transfer tie forces.

This will mean the structure is expected to experience differential displacements between the foundations of the extended frame of the Bays 6(a) and 7(a) bay walls as a consequence of lateral spread.

This is considered as a potential step-change behaviour, as the onset of lateral spread (a geotechnical step change) could cause a life-safety hazard if the differential displacements lead to a structural weakness (structural step change) in this area.

Ground floor across basement between Bays 1(a) and 2(a)

In the transverse (Z) direction of the Auditorium the perimeter ring beam is not continuous. Between Bays 1(a), there is only the ground floor level 120mm slab that continuous across. Furthermore, there are several voids in this slab at this area which limits the total amount of tie force that can be developed across this interface. The basement floor is not considered to be able to contribute to the tie force, as the basement walls can flex out-of-plane.

The capacity includes all the structure at the ground floor area between (and including) Wall 6 up to the basement wall at the Northern end of the basement tunnel (under Wall 1).

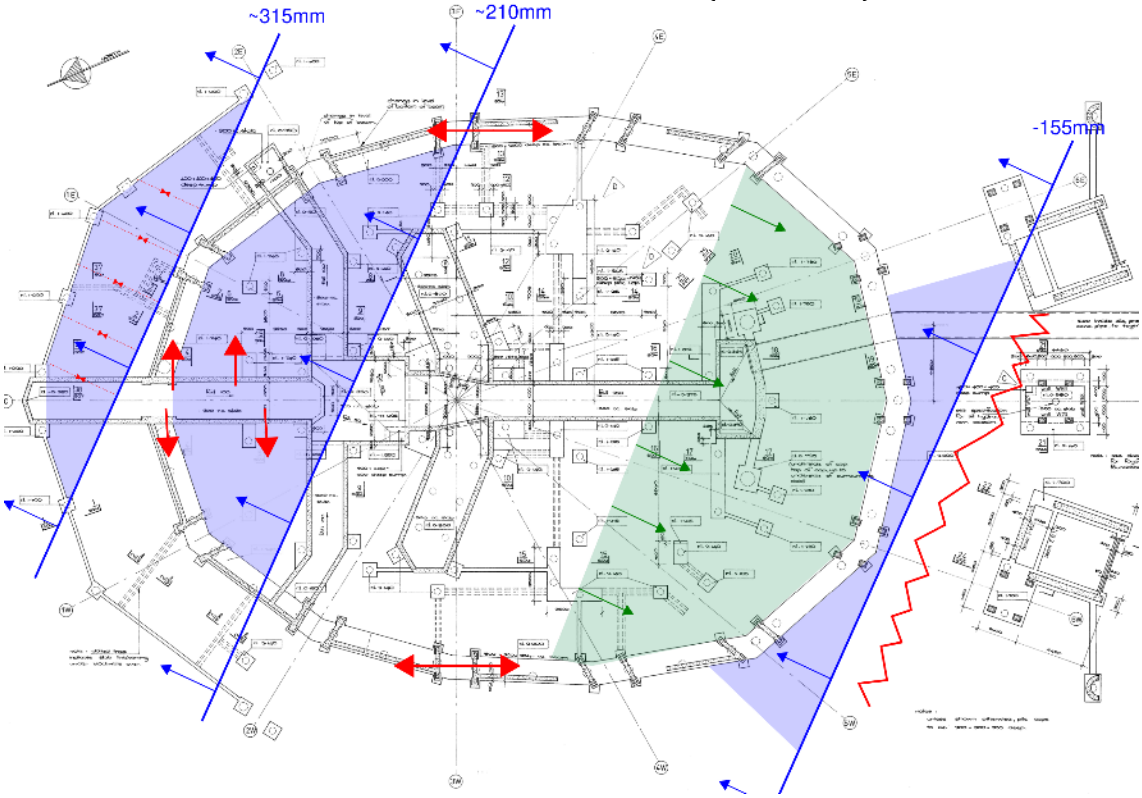

Figure C.4-69: Potential tie forces that could develop as a consequence of the lateral stretch in the Northern end and lobby area of the Auditorium

The end wall of the basement under Wall 1 is reinforced with 10 layers of Ø12mm bars of the lower grade reinforcing at each face. The basement floor is reinforced with 662-gauge mesh and is therefore not considered to provide a tie force. In total this area provides a tie restraint of 733kN.

The 120mm ground floor slab is reinforced with Ø12mm bars of the higher-grade reinforcing at 250mm spacing in both directions, which could develop capacities of up to 206kN/m. Scaled from the drawings there are floor areas with a width of 3.88m, 3.94m and 1.26m that can contribute. Totalling at a capacity of 1,870kN.

Wall 6 can provide addition capacity and is reinforced with Ø16mm bars of the higher-grade reinforcing at 250mm spacing at each face. At the critical section, at centreline of the structure (gridline C), the wall has a height of 4.32m above the ground floor level. There is also a portion of wall in the basement, however this is offset and angled from the plane of Wall 6 and therefore conservative excluded. Wall 6 is expected to contribute a tie capacity of 2740kN.

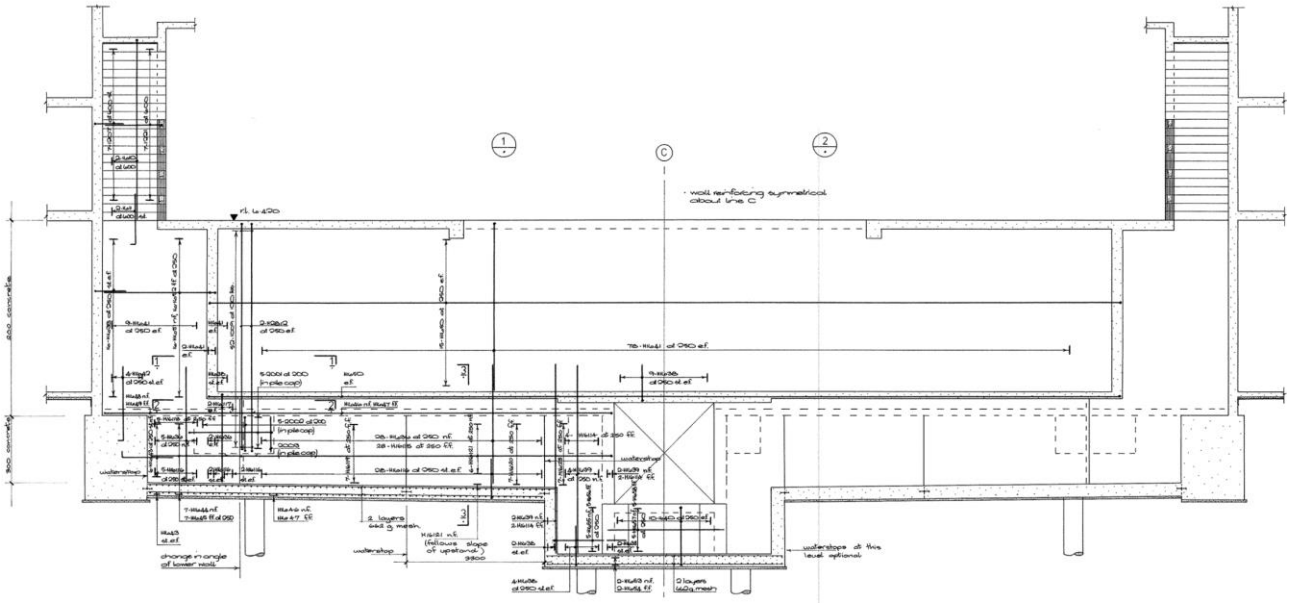

Figure C.4-70: Elevation and reinforcing of Wall 6

The tie at this area experiences the passive pressures on the inside of the foundations on the Eastern end pulling the structure along. Which includes the passive pressures on the 900x900mm foundation beam under Wall 1 and half of those acting against the perimeter ring beam. The passive pressures on the basement tunnel structure are not included in the assessment of this tie force, as those can be tied back to the perimeter ring beam through other load paths.

The total demand under liquefied ground conditions is 550kN against the foundation beam under Wall 1. The total capacity from the basement end wall and 3.88m of ground floor slab provide ample restraint (1,500kN) against this tie force.

On the other side of the perimeter ring beam there is 1780kN from the passive pressures on the internal face of this ring beam. This can be restrained by the 3.94m and 1.26m ground floor area and the reinforcement in Wall 6. This also provides ample restraint (3,800kN) against this tie force.

Concluding there is a considerable margin of safety against failure in this tie.

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Perimeter ring beam in the transverse (Z) direction

The perimeter ring beam in the transverse (Z) direction is assessed in a similar manner to that in the longitudinal (X) direction. As the worst-case scenario, the tie force comprises 50% of the passive pressures pushing in the East direction and the other 50% acting in the opposite direction. The maximum passive pressures that the Auditorium structure can see in the transverse (Z) direction are 13MN.

This force is split across both sides of the structure and the perimeter ring beam can generate capacity of 12.5MN on each side, adding to a total of 25MN capacity.

C.4.4.3 Assessment Category

The Engineering Assessment Guidelines also require the project to be categorised as either structurally dominated, interactive or geotechnically dominated. This project is considered to be **interactive**, since the assessment of the structure is influenced by a geohazard (onset of liquefaction).

C.4.4.4 Considerations for Geotechnical and Structural 'Step Change'

The Engineering Assessment Guidelines provide additional requirements if a geotechnical step-change, such as a trigger of liquefaction, leads to a structural step-change. There is particular emphasis on structural step change that could lead to a significant life-safety hazard – requiring the seismic score of the element that causes this significant life-safety hazard to be half of the level of shaking (or displacement) that causes the step-change behaviour.

A significant life-safety hazard is defined in Section A3.1.1 of the Engineering Assessment Guidelines as "an unavoidable danger that a number of people are exposed to. … It is distinct from a life-safety hazard and is intended to preclude from consideration building elements that are of insufficient size to constitute a life-safety hazard of reasonable extent. … Failure of a building or building section as a whole (leading to collapse) is considered to be a significant life safety hazard but failure of individual members/elements in the primary structure will only constitute a significant life safety hazard, when considered individually, if their failure causes them to fall. … A significant life safety hazard can also result from the loss of gravity load support of a member/element of the secondary structure, or of the supporting ground, or of non-structural items that would reasonably affect a number of people."

To assess whether an element is triggering a step-change behaviour in the structure, reference is made to the recommendations by Whitehurst, Palmer and Jury (2020). In this paper they discuss the intent behind the guidelines for liquefaction assessment and a simplified representation to assess the margin between a life-safety hazard risk and a collapse risk for a lateral spread scenario. This approach has been adopted for this DSA to demonstrate which structural elements (primary and secondary) are required to consider the halving of the seismic score.

It is implicit (and in some instances explicit) in the Engineering Assessment Guidelines that there is a margin between the life-safety risk and collapse risk. This paper discusses that the intent of the step-change reduction (halving of the seismic score) is to account for elements that lose that inherent margin at a geotechnical step-change.

For the purpose of this DSA, this margin between a life-safety risk and the collapse risk is considered as a factor of 2 consistently across all structural elements. Following discussions with Tonkin+Taylor, this approach was considered to align with the 'uncertainty factor described in their Geotechnical Assessment Report. It is our opinion that this an acceptable approach. However, it is noted that the Engineering Assessment Guidelines do not specifically cover this in detail for each mechanism that each structural element could form.

The Geotechnical Assessment Report has plotted the amount of lateral spread against various levels of seismic shaking, shown in Figure C.4-71. It gives the probable amount of lateral spread demand for the end of the structure closest to the free face (at the seawall near the lagoon), at the Northern end of the Renouf Foyer and at the Southern end of the Renouf Foyer. There is also a similar graph that can be generated approximately halfway across the Auditorium using the data from the contours plot (see Figure C.4-64).

Tonkin+Taylor Limited have further recommended that the probable amount of lateral spread that is presented in their Geotechnical Assessment Report will have a range between ½x and 2x that amount as there is uncertainty in their assessment. However, considering this is a seismic assessment and not a new design, all the parameters that could influence the seismic score are considered at the probable range.

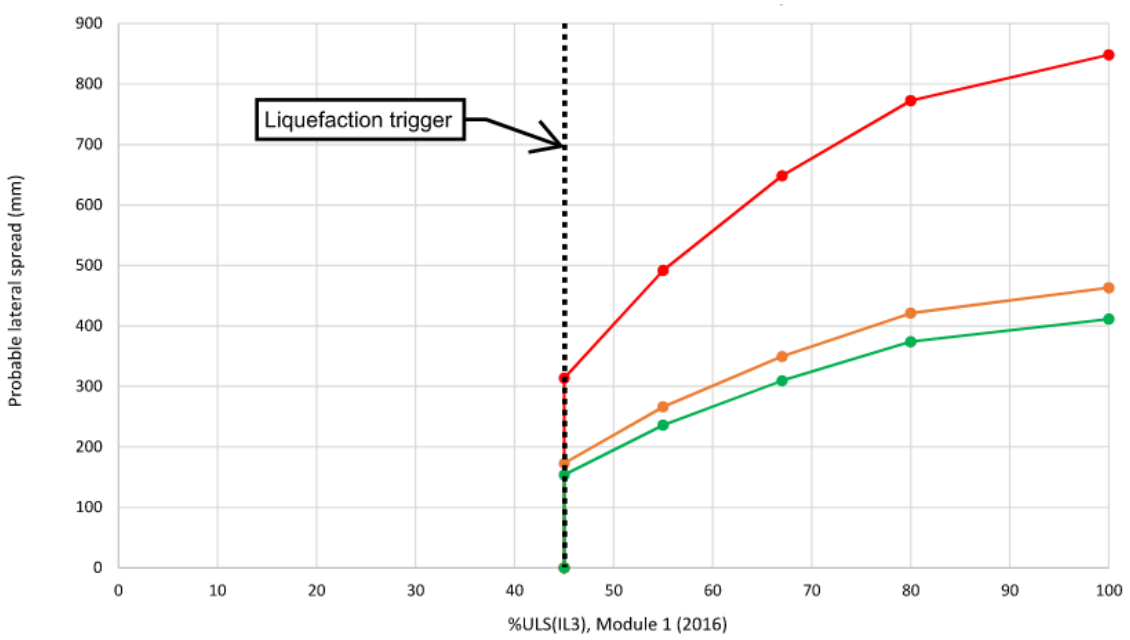

Figure C.4-71: Probable estimate of lateral spread against %NBS$_{IL3}$ seismic shaking, shown for the northern end of the Auditorium (red), the southern end of the Auditorium (orange) and the southern end of the Renouf Foyer (green)

To assess each structural element, the displacement capacity that causes a life-safety hazard can be plotted on the chart of Figure C.4-71 (or the appropriate location), and at the intersection point with the probable lateral spread line it can be determined at which level of seismic shaking the lateral spread causes this structural element to be a significant life-safety hazard. The same can be done for a displacement capacity at a factor of 2.0, which represents the approximate margin to collapse. This presentation can then result in three possible conclusions:

- Both the displacement capacity of the life-safety criteria and the collapse criteria intersect the graph of the lateral spread past the initial vertical portion. This indicates the geotechnical step-change does not affect this structural element. The seismic score of the structural element does not need to be halved.
- The displacement capacity of the life-safety criteria meets the graph of the lateral spread amount in the vertical portion, but the displacement capacity of the collapse risk intersects in the diagonal portion. This indicates that there is a residual margin between the life-safety risk and the collapse risk. As such, there is a geotechnical step-change, but this does not result in a structural step-change. The seismic score of the structural element does not need to be halved.

- Both the displacement capacity of the life-safety criteria and the collapse criteria intersect the graph of the lateral spread amount in the vertical portion. This indicates that there is no nominal margin between life-safety risk and the collapse risk. This indicates a structural step-change which could cause a significant life-safety hazard following a geotechnical step-change, and therefore the seismic score of the structural element must be halved.

For elements that are affected by differential lateral spread across the footprint of the Michael Fowler Centre, the difference between the graphs is used in the assessment.

Figure C.4-72 in the next Section shows an example of this assessment approach for the assessment of the piles. It shows the probable amount of lateral spread in the solid grey line. The two dashed grey lines show the ½x and 2x bounds of the lateral spread, however these are not used for the assessment. The orange lines show the displacement capacity at the life-safety criteria and the collapse criteria (as a margin of 2.0 on the life-safety criteria).

C.4.4.4.1 Piles

The piles were assessed to have a displacement limit of 300-450mm, governed by the strain limits in the reinforcement. The lower bound capacity (~300mm) applies to Pile Cage G, which have been assessed in Section C.4.4.1 and C.4.4.2. The remaining piles, and in particular Pile Cage F around the perimeter ring beam, have displacement capacities of 350mm and higher. Exceedance of the displacement limit can see the piles show significant degradation of their axial load carrying capacity which could ultimately lead to (partial) collapse.

The piles are assessed against the effects of the geotechnical step-change of lateral spread by assessing them against the maximum lateral spread that the structure can undergo, at the Northern edge, closest to the free face (the lagoon). Although, this is probably conservative as the structure is expected

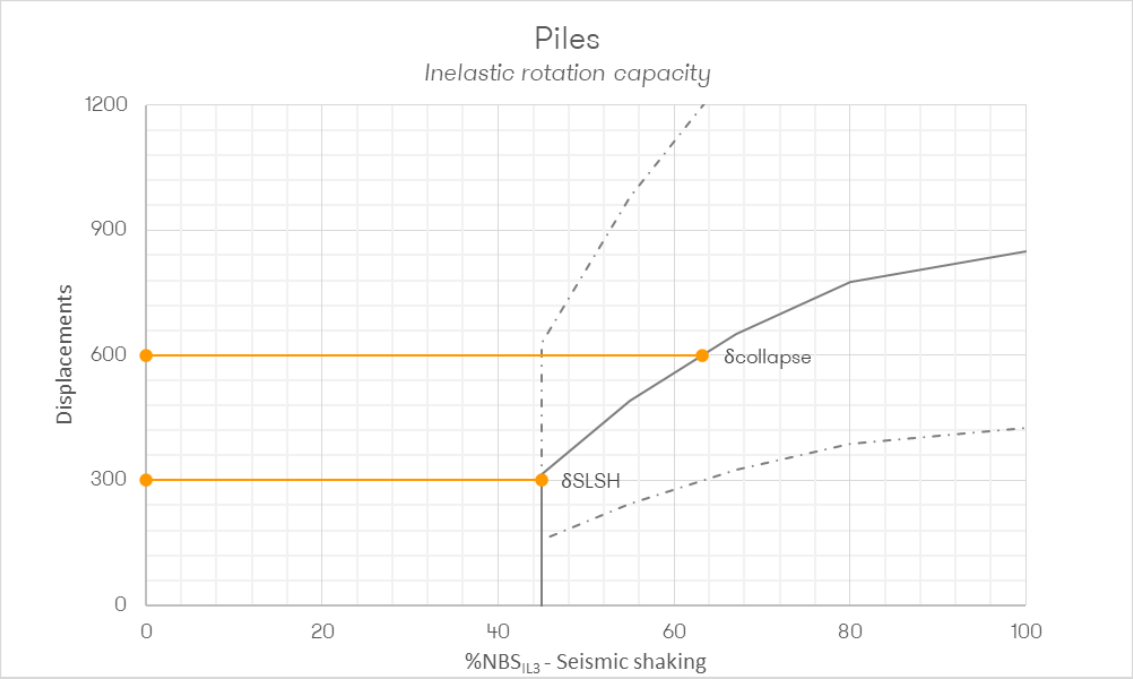

Figure C.4-72: Assessment of the piles for the step-change reduction from lateral spread

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As presented in Figure C.4-72, there is a sufficient residual margin between the life-safety risk and the collapse risk in terms of the levels of seismic shaking. The seismic score of the piles is not required to be halved in this scenario.

C.4.4.4.2 Lateral Stretch in Lobby Area

The columns of the extended frames of Bays 6(a) and 7(a) are founded on the same foundations as the independent staircase structures, however, they are only tied back to the Auditorium through the 100mm slab-on-grade which is reinforced with 665-grade mesh. This reinforcing cannot be relied upon to provide a tie force in accordance with the Engineering Assessment Guidelines. The Auditorium is expected to displace as an 'island', held together by the perimeter ring beam and the ground floor; however, that connection ends at the ground floor where it is reinforced with non-ductile mesh. The 'islands' underneath the staircases and the columns of the extended frames will displace independently of the Auditorium, likely at a much lesser magnitude.

The differential lateral spread between the Auditorium and the independent staircase structures can therefore cause the effects of lateral stretch to have the two foundations elements displace independently relative to each other. If the Auditorium is considered to laterally displace as an 'island' to the maximum amount near the free-face (250mm at 45%NBS_{IL3}, see section C.4.4.2.4) and the other columns of the frame extensions are laterally displacing along with the Independent Staircase Structures (175mm at 45%NBS_{IL3}), then the differential displacement is 250-175 = 75mm at 45%NBS_{IL3}.

This displacement is considered to be taken out by flexure of the extended frames. The structural hierarchy of this frame was determined to be controlled by a column-sway mechanism, similar to the transverse frames in the Renouf Foyer (see Section C.4.3.3). The columns are assessed to have a displacement capacity of 68mm in accordance with the Equations C5.33 and C5.59 of the Engineering Assessment Guidelines. Which considers the deformation at onset of loss of gravity load capacity.

As presented on Figure C.4-74, the columns are not expected to experience a step-change in behaviour at the onset of lateral spread. There is a margin to collapse within the inelastic rotation capacity of these columns and as such, these are scored at **45%NBS_{IL3}**.

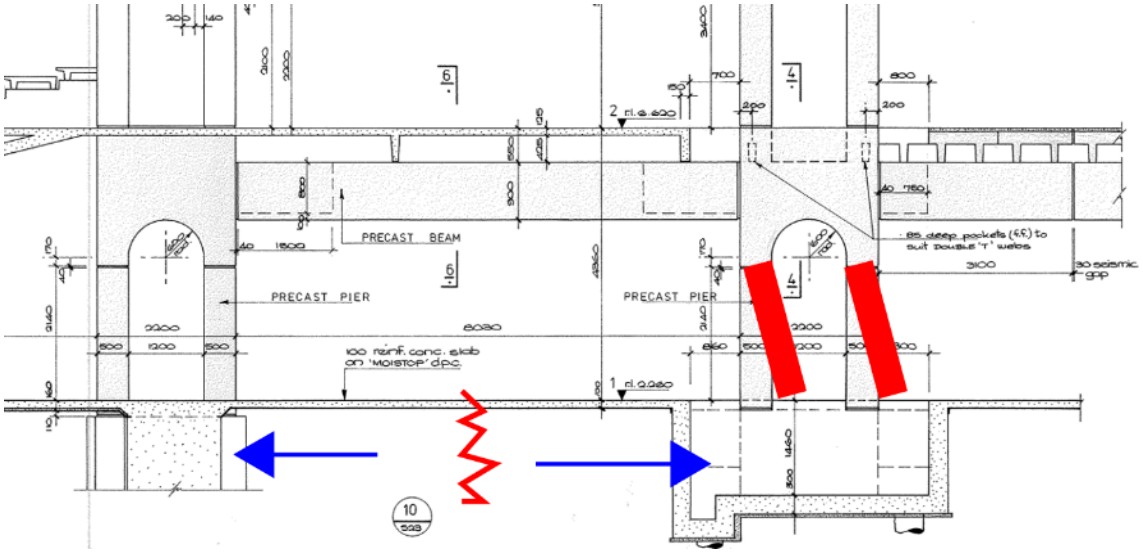

Figure C.4-73: Differential displacements (lateral stretch) leading to the formation of a mechanism (column-sway) in the columns of the extended frame at Bays 6(a) and 7(a)

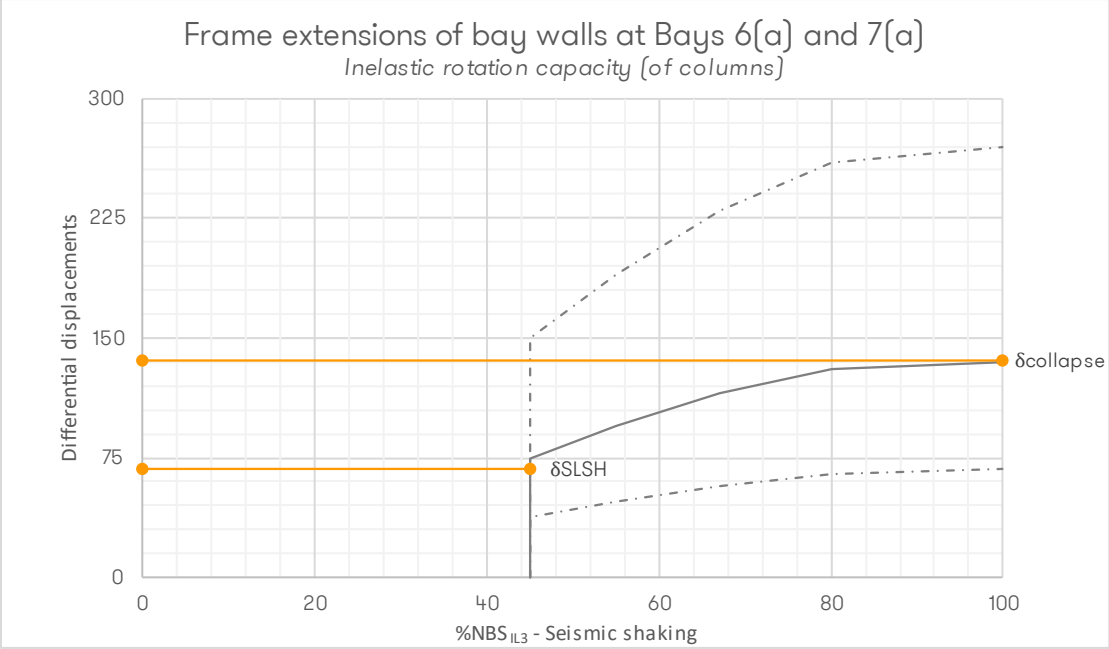

Figure C.4-74: Assessment of extended frames at Bays 6(a) and 7(a) against the effects of lateral spread

C.4.5 Evaluation of Seismic Gaps

Considering that the different structures of the Auditorium, Renouf Foyer and the independent staircase structures have been built in the close vicinity of each other, they have the potential to pound into each other. The consequences of those have been considered in accordance with Section C2B of the Engineering Assessment Guidelines.

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C.4.5.1 Seismic Separations between Auditorium and Renouf Foyer

The Auditorium and the Renouf Foyer provide access to each other via the Level 2 floor. On the Auditorium side there is a system of double-tee flooring that acts as a balanced cantilever on the two beams of the extended frame at Bays 6(a) and 7(a). On the side of the Renouf Foyer, the configuration is relatively similar, but mirrored, and the beams extend from the intermediate transverse frames. Both sides have the double-tee elements finished with a concrete edge beam which ties back to the primary structure.

There is a 30mm seismic gap provided in the original design which is still present and has been observed on site. Within the concrete edge beam there is a shear connector detail which allows shear transfer in the vertical plane, but initially not in either of the lateral planes until the either structure has moved to close the gap. The shear connector is assumed to be in its centralised position at the start of an earthquake event and provides 30mm of movement in both lateral directions before engaging in shear.

The shear connector is intended to limit the amount of differential vertical displacement between the two floor elements in the different buildings. It is not intended as a gravity load path, and therefore the direction where the buildings are pulling away from each other is not considered to be critical.

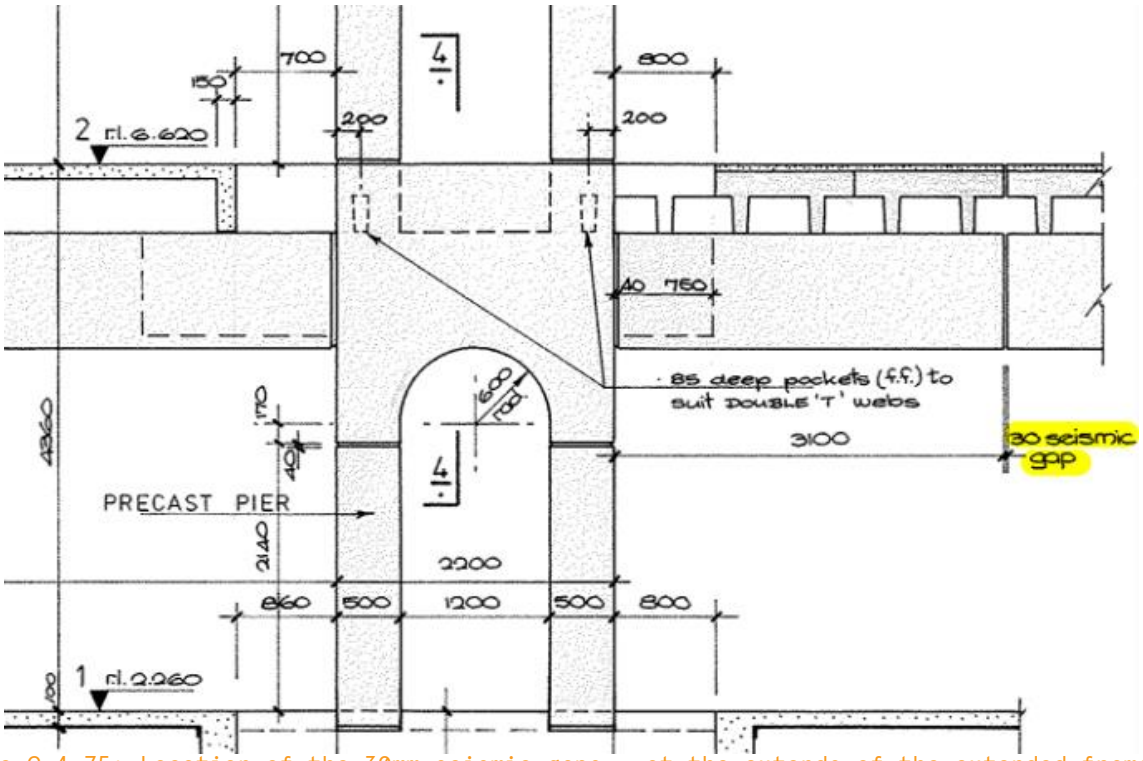

Figure C.4-75: Location of the 30mm seismic gaps - at the extends of the extended frame at Bays 6(a) and 7(a)

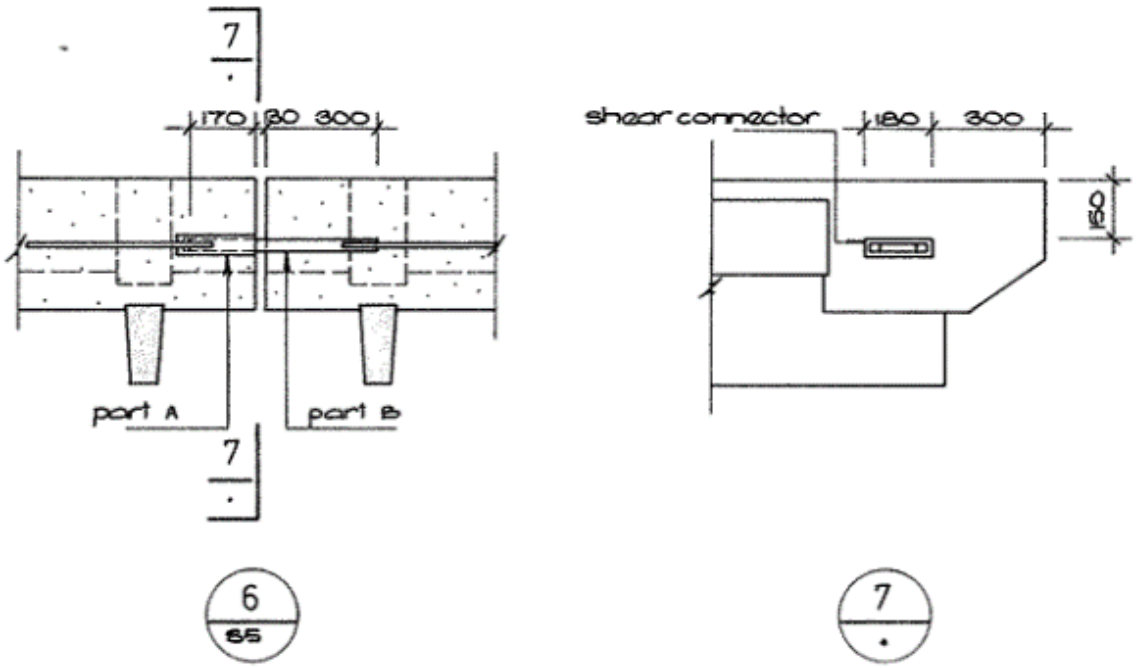

Figure C.4-76: Shear connector detail at the concrete edge beam

The displacements at Level 2 of the Auditorium and the Renouf Foyer are presented in Table C.4-10 for both the longitudinal (X) direction and the transverse (Z) direction. They are combined using the SRSS method. Concluding that the structures are expected to pound into each other at a level of 15%ULS seismic shaking in the longitudinal (X) direction of the Auditorium and 10%ULS seismic shaking in the transverse (Z) direction.

Table C.4-10: Drifts (in mm) of the Auditorium and Renouf Foyer at Level 2

Level of shaking [%NBS$_{IL3}$]	Auditorium		Renouf Foyer		SRSS	
	[X]	[Y] *	[X]	[Y]	[X]	[Y]
5%	4	8	10	14	11	16
10%	8	16	20	28	21	**32**
15%	12	24	30	42	**32**	**48**
20%	16	32	39	55	**43**	**64**
…						
35%	28	56	69	97	**74**	**112**
…						
100%	83	165	261	381	**274**	**415**

**Taken at the extended frames of Bays 7(a)*

In accordance with Section C2B.5.2 for the 'Simplified Checks' of buildings with 'aligned floors but with mass difference', the stiffer building is expected to receive 20% of the storey shear of the adjacent building. The Auditorium is considered to be the stiffer building and will therefore likely see 900kN as a force applied at that interface.

The Renouf Foyer is considered in accordance with the requirements for buildings with 'aligned floors but with a height difference'. It will need to consider to be truncated by the height at which the pounding occurs, this is further described in Appendix E.

As described in Section C.4.2.7.2, the 900kN is applied to an area that is already prone to the lateral displacements of the floor units, as the double-tee flooring has no resilient load paths to the primary lateral load resisting system. The floor diaphragm relies on the friction component between the double-tee units and the supporting beams to take out its own inertia.

Pounding along the central axis of the Auditorium and Renouf Foyer

Two scenarios are highlighted below and shown on Figure C.4-77, which shows the compression load paths in red arrows, the required tension ties with blue arrows, the load paths through friction in orange arrows and the reactions in the black arrows.

- The pounding occurs centralised, and the additional lateral load is transferred directly into the beams of the extended frames towards the buttresses of the bay walls at Bay 7(a). From there the load can be considered to enter the sloping diaphragm of Level 1/2 and be transferred to the parallel Walls 14 and the bay walls at Bays 3(a), 4(a) and 5(a).
- The pounding occurs on the corner of the floor area and the additional lateral load needs to develop compression struts across the floor to engage the beams of the extended frames in a couple. However, along this path the structure needs to rely on tension ties that can only be formed by the friction component between the double-tees and the supporting beams.

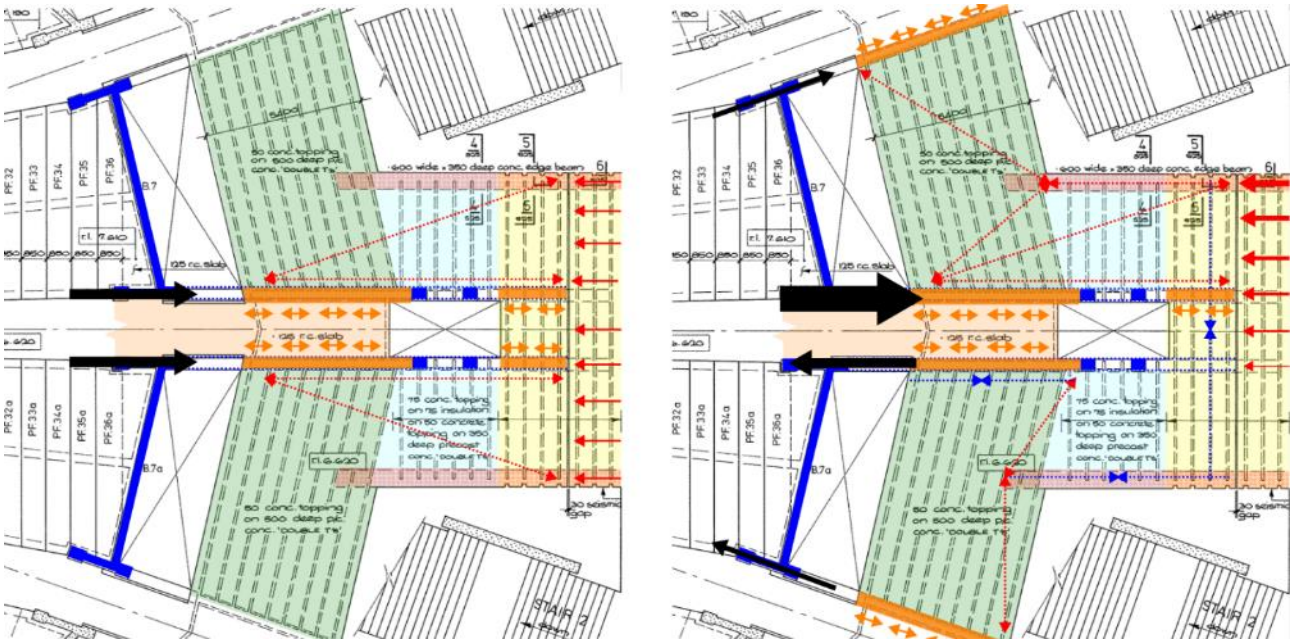

Figure C.4-77: Internal strut-and-tie for pounding in the global (X) direction

Whilst it is expected that the majority of the loading is introduced directly into the primary lateral load resisting system, from direct pounding, it cannot be excluded that this pounding exposes the structural weakness of this diaphragm even further.

Pounding perpendicular to the central axis of the Auditorium and Renouf Foyer

In the other direction, transverse for the Auditorium, there is a potential load transfer through the shear connectors in the concrete edge beam. The overstrength shear capacity of the shear connectors exceeds the 20% storey shear that is required to be transferred. Therefore, the pounding in this direction follows the similar sections of the Engineering Assessment Guidelines (Appendix C2B) for buildings with 'aligned floors but a mass difference' or with 'aligned floors but a height difference'.

Figure C.4-78 shows an indication of the potential load paths through the structure but highlights that there is still a significant reliance on load paths through the friction mechanism under the webs of the double-tee units. The system is symmetrical along the central axis of the Auditorium, therefore only the eastwards direction is shown.

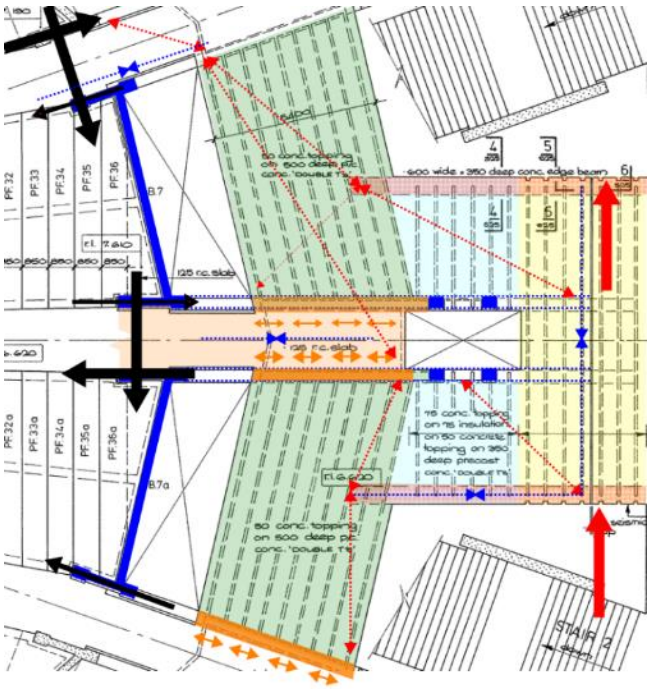

Figure C.4-78: Internal strut-and-tie from pounding in the global (Z) direction

As loading is applied to the concrete edge beam this will put a torsional demand on the system of double-tee units. It can be resolved through the internal strut-and-tie mechanisms through the double-tee units; however, as mentioned before, it ultimately relies on areas of friction to transfer tie forces into the supporting beams. The bottom half of the strut-and-tie diagram on Figure C.4-78 is similar to the own inertia of the diaphragm itself as shown in Section C.4.2.7.2. Considering that the demand from pounding quickly will exceed the demands from the inertia of the diaphragm it is considered that this pounding would result in large displacements of the double-tee floor elements.

The double-tee units that are positioned as balanced cantilevers (yellow in Figure C.4-78) are not considered the weakest link. However, the tie of the concrete edge beam (red) to the double-tee units of the promenade (green) is of unknown configuration and may also be a structural weakness.

Similar to the pounding parallel to the central axis of the Auditorium, there is expected to be a level of internal resilience before the inertia exceeds the friction capacity of the double-tee units. Therefore, the seismic score from pounding with the Renouf Foyer, perpendicular to the central axis of the Auditorium, is determined at **20%NBS$_{IL3}$**.

However, a better-than-expected performance against life-safety criteria is also possible as the double-tees are keyed into the supporting beams and will need to displace a considerable amount before they lose their seating.

C.4.5.2 Seismic Separations between Auditorium and Independent Staircase Structures

At the Level 4 promenade floor of the Auditorium, there is an access passage toward the toilet facilities which are located in the independent staircase structures. A short span, 1760mm wide, is constructed from cast insitu concrete, which extends from the topping of the double-tee units and is simply-supported across the gap between the two structures. The short span is supported on the independent staircase towers with 100mm seating and a 25mm seismic gap, as shown in Figure C.4-79.

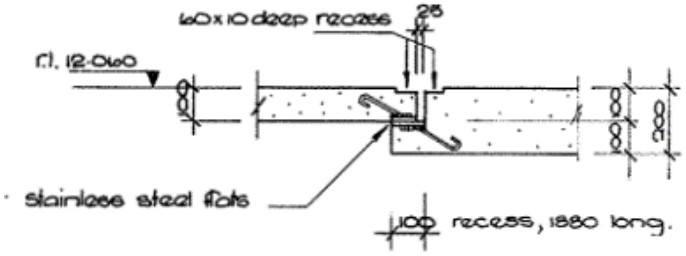

Figure C.4-79: Support detail of cantilever slab onto the independent staircase structures

The slab is reinforced with deformed bars in the bottom portion and with 665-gauge mesh in the top reinforcing. In accordance with the recommendations of Yellow Chapter C5, the non-ductile mesh cannot be relied upon to develop a tie capacity and so the floor is expected to have minimal hogging capacity. It therefore relies on its seating and cannot cantilever by itself.

The recess in the floor of the independent staircase structures is 1880mm wide, thereby providing a gap of 60mm in the transverse (Z) direction.

The displacements at Level 2 of the Auditorium and the independent staircase structures are presented in Table C.4-11 for both the longitudinal (X) direction and the transverse (Z) direction. They are combined using the SRSS method. The displacements of the Auditorium are considered under an angle of 20 degrees, however, when combined with concurrency in the perpendicular direction, the results are of a similar nature to the principal directions. Concluding that the structures are expected to pound into each other at a level of 10%ULS seismic shaking in the longitudinal (X) direction of the Auditorium, a loss of seating is expected at 35%ULS seismic shaking. In the transverse (Z) direction, pounding is expected at 15%ULS seismic shaking in the transverse (Z) direction.

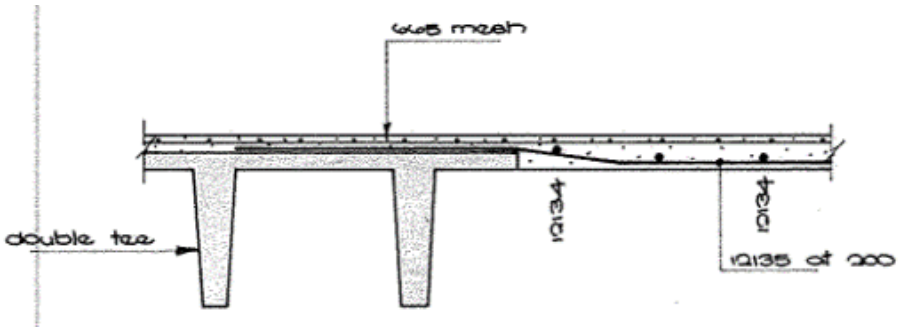

Figure C.4-80: Reinforcement from the double-tee unit to the cantilever slab

Table C.4-11: Drifts (in mm) of the Auditorium and Independent Staircase Structures at floor Level 4

Level of shaking [%NBS$_{IL3}$]	Auditorium		Independent Staircase Structures		SRSS	
	[X]	[Y] *	[X]	[Y]	[X]	[Y]
5%	5	9	13	16	14	18
10%	10	19	26	32	**28**	37

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Level of shaking [%NBS$_{IL3}$]	Auditorium		Independent Staircase Structures		SRSS	
	[X]	[Y] *	[X]	[Y]	[X]	[Y]
15%	15	28	39	48	**42**	**55**
20%	20	37	52	64	**56**	**74**
…						
35%	35	65	92	111	**98**	**129**
…						
100%	104	191	262	318	**282**	**371**

**Taken at the extended frames of Bays 7(a)*

Pounding against the double-tee floor element

In accordance with Section C2B.5.2 for the 'Simplified Checks' of buildings with 'aligned floors but with mass difference', the stiffer building is expected to receive 20% of the storey mass of the adjacent building. The Auditorium is considered to be the stiffer building and will therefore likely see 60kN as a force applied at that interface. There are no specific requirements for the lighter building, the independent staircase structure.

The pounding against the double-tee element at the Level 4 promenade is resolved by internal strut-and-ties towards the 125mm cast insitu slabs at the buttresses of the bay walls. It ultimately relies on the 14-Ø16mm bars of the lower reinforcing grade, spaced at 350mm centres, to transfer the load into the supporting beams. This is taken out by shear-friction which results in a capacity of ~780kN on either side. And is unlikely to pose the governing structural weakness.

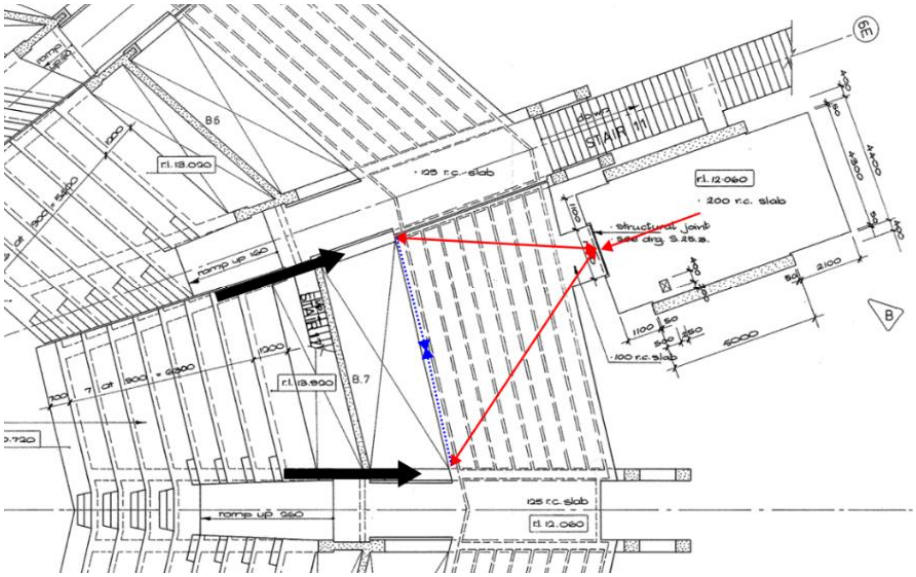

Figure C.4-81: Internal strut-and-tie to resolve the pounding force from the independent staircase structures parallel to the structure

Independent staircase structure pulling away from Auditorium

As the structures move in opposite directions, the seating capacity of the short span can be exceeded. As this happens the slab will show excessive displacements, which will result in additional pounding forces

when the structures cycle back to close the seismic gap again. This could lead to a life-safety hazard and, in accordance with the general intent of the Engineering Assessment Guidelines, should include a factor of Q = 2 on the displacement demand.

The unseating will likely occur at displacements associated with 32%NBS_{IL3} and therefore the loss of seating is found to correspond to a (rounded) seismic score of **20%NBS_{IL3}**.

Displacements perpendicular to the seating

Perpendicular to the short span of floor that bridges the gap between the two structures, there is a 60mm gap between the slab and the edges of the recess – assuming the floor is centralised within the recess.

If the seismic gap is exceeded, then pounding will occur of similar magnitude to that in the parallel direction (60kN). The system could resolve that through internal strut and ties; however, the double-tee units were shown in Section C.4.2.5.1 to rely on a component of friction.

On this basis, the pounding will likely result in a progressively worse performance of this friction interface and therefore the pounding in this direction is also scored at **20%NBS_{IL3}**.

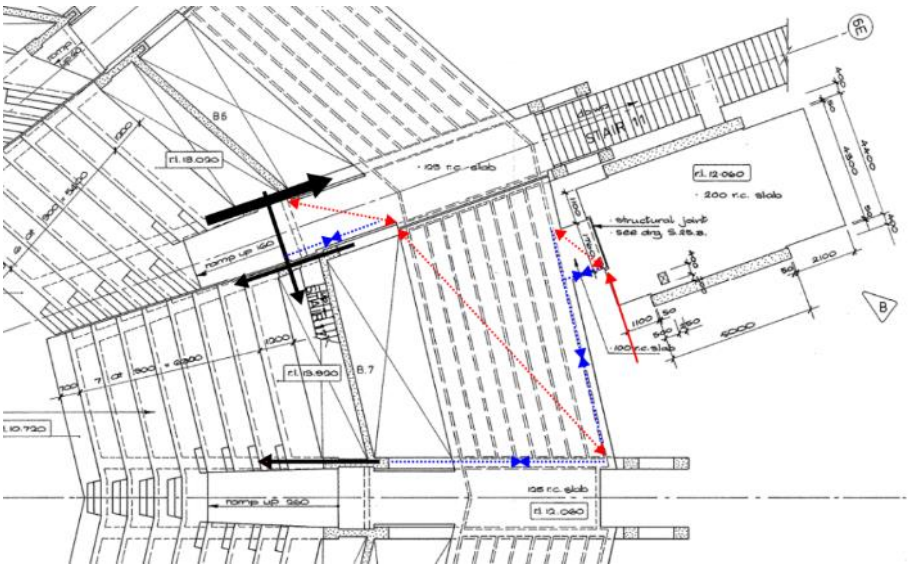

Figure C.4-82: Internal strut-and-tie to resolve the pounding force from the independent staircase structures perpendicular to the structure

C.5 SUMMARY OF IDENTIFIED STRUCTURAL WEAKNESSES

Structural Weaknesses are those deficiencies identified which have an assessed seismic rating of less than 100%NBSIL3. These are listed in Table C.5-1.

This table is limited to the structural elements of the primary lateral load resisting system and primary gravity supporting system of the Auditorium. Other secondary structural and non-structural elements are discussed in Appendix G.

Table C.5-1: Structural Weaknesses of the primary structure of the Auditorium

Building Element	Structural Weakness	%NBS$_{IL3}$
Transfer diaphragm at Level 1/2	Exceedance of tie capacities in the diaphragm (provided by deformed bars in the diaphragm) and ties to the flat diaphragm	20
Level 2 promenade diaphragm	Exceedance of chord tie capacities in the diaphragm (provided by deformed bars in the diaphragm).	20
Double-tee flooring at Level 2 in the lobby area	▪ Exceedance of friction capacity due to the lack of tie capacity in the diaphragm (665-gauge non-ductile mesh). ▪ Loss of seating due to excessive differential displacements and pounding between the Auditorium and the Renouf Foyer	25
Roof diaphragm	Lack of tie capacity in the diaphragm (661-gauge non-ductile mesh) leading to subsequent structural weaknesses in other elements, such as: ▪ Excessive axial loads in the top chords of the roof trusses ▪ Excessive loads in the connection between the roof trusses and the precast portion of the perimeter buttress wall system ▪ Excessive size of cracks in the concrete diaphragm across the supports of the DyCore roof elements ▪ Redistribution of seismic lateral loading to the perimeter buttress walls leading to excessive out-of-plane loading	20 – 30
Level 4 wedge-shaped diaphragm of promenade floor and stepped seating	▪ Exceedance of friction capacity due to the lack of tie capacity in the diaphragm (665-gauge non-ductile mesh). ▪ Loss of seating due to excessive differential displacements between the Auditorium and Independent Staircase Structures	25
Bay walls (upper precast wall panels)	Exceedance of shear capacity at the upper precast portion of the bay walls, at the horizontal interface between different precast wall panels	25
Bay walls (upper precast wall panels)	Exceedance of the out-of-plane flexural capacity of the precast concrete wall panels	35
Gravity columns at the extended frames of Bays 6(a) and Bays 7(a)	Lack of compatibility with the displacement demands from differential lateral spread between foundations of the Auditorium and the independent staircase structures ***(does not require a reduction for the consideration of a geotechnical step change)***	45

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Building Element	Structural Weakness	%NBS$_{IL3}$
Bay walls (connection of upper precast wall panels to the roof)	Exceedance of shear capacity at the interface between the precast wall panels and the roof	45
Foundation system	Exceedance of inelastic rotation capacity of the piles during the lateral spread scenario ***(does not require a reduction for the consideration of a geotechnical step change)***	45
Gravity columns in the backstage area	Exceedance of rotation limits for slab-column joints.	45
Bay walls (cast insitu portion above floor level 2)	Exceedance of inelastic rotation capacity, limited by bar buckling in the web area at Level 2 of Bays 7(a).	50
Bay walls (isolated columns in lobby area at Bays 6(a) and 7(a))	Exceedance of inelastic rotation capacity, which is significantly reduced due to the large axial load ratio. Because of the discontinuation of the Bay Wall system in the lobby area the flexural demands in the Bay Walls result in significant axial loads on the buttress columns at the ground floor level.	55
Level 2 Precast Cantilever Beams adjacent to double T slabs	Combined actions on beam reinforcing: gravity, lateral ties, vertical acceleration	55-70
Squat walls at Level 1	Exceedance of shear capacity in the walls. (the lowest score is assessed for Wall 18, other squat walls are achieving higher scores, see Section C4.1.2).	55-75
Level 4 Precast Cantilever Beams	Combined actions on beam reinforcing: gravity, lateral ties, vertical acceleration	75
Floor diaphragms at backstage area (Level 2, Level 3 and Level 4)	Exceedance of tie capacities in the diaphragm (provided by deformed bars in the diaphragm).	75-80
Floor diaphragm at director's area (Level 5)	Exceedance of tie capacities in the diaphragm (provided by deformed bars in the diaphragm).	85

Appendix D

Technical Summary of the Auditorium's Supplementary Nonlinear Time-History Analysis (NLTHA)

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Appendix D Technical Summary of the Auditorium's Supplementary Nonlinear Time-History Analysis (NLTHA)

D.1 INTRODUCTION

The Nonlinear Time-History Analysis (NLTHA) for the Auditorium has been used for supplementary inputs into the Modal Response Spectrum Analysis (MRSA) described in Appendix C.

The element demands and capacities are not assessed in the NLTHA for this DSA. This analysis procedure is only selected to verify diaphragm demands in the superstructure over the more conventional pseudo-Equivalent Static Analysis (pESA) procedure, which is described in Section C2E.5 of the Engineering Assessment Guidelines. Parts and Components loading was calculated per Chapter 8 of NZS1170.5:2004.

D.1.1 Purpose of NLTHA Analysis

Primary structural elements and a selection of contributing secondary structural elements are modelled to capture their mass and modal participation in the structure. The NLTHA model was created to produce an upper-bound response for Ultimate Limit State (ULS) diaphragm floor accelerations, which supersedes the pESA distribution for the seismic assessment of the concrete diaphragms. The model was built in general accordance with the Engineering Assessment Guidelines with limited supplementation by the ASCE41 Tier 3 procedures. A suite of 11 ULS hazard level ground motions was used to determine a mean response for diaphragm accelerations.

The seismic score for the primary lateral load resisting system elements, and secondary and non-structural elements (SSNS) is assessed per the element demands and drifts from the MRSA. Element capacities are determined outside of the analysis and checked in post processing, refer Appendix C.

D.2 ANALYSIS MODEL

The analysis model is built using ANSR II, Holmes' in-house nonlinear modelling software. The software comprises the non-linear analysis engine ANSR II from (Mondkar & Powell, 1979), further developed with additional functionality and input and output processing. The program is based on a concentrated or lumped plasticity model refer Figure D.2-1.

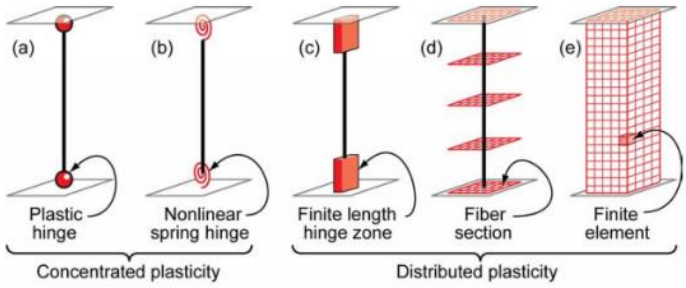

Figure D.2-1: ANSR II uses concentrated plasticity image (a), from (NEHRP, 2010)

The program can perform and process linear modal analysis, non-linear gravity analysis, non-linear static analysis (NSP or pushover) and non-linear dynamic analysis (NLTHA or NDP). For the NSP or NLTHA analyses, the self-weight and imposed floor live loads are applied in a gravity analysis prior to the NSP or NLTHA itself.

Nonlinear displacement pushovers can also be performed. These involve applying a user defined displacement history to a selected node (usually a diaphragm node). These are generally only used for model verification and testing.

The most current model version is v10a, with supplementary results from v20.

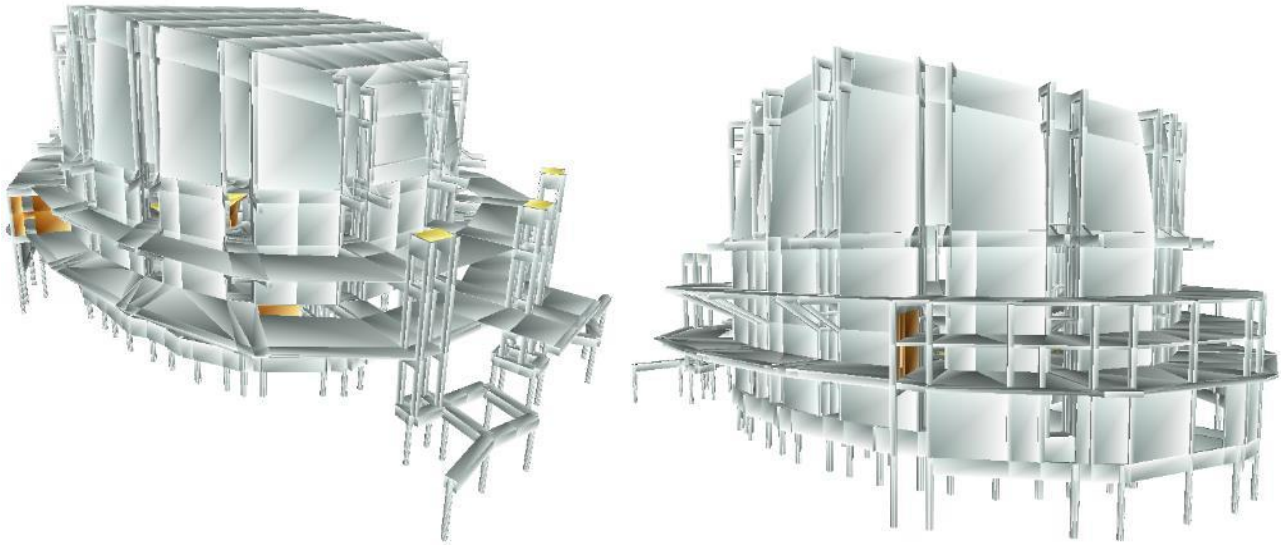

Figure D.2-2: Analysis model 3D renderings

D.2.1 Model Geometry

The ANSR model includes all the primary lateral and vertical superstructure above the ground. Below grade, a majority of the ground beams are modelled along with pile element locations captured with massless column elements. The model is restrained vertically at the base of the piles ends and horizontally at the ground floor. More detailed information on the modelled foundation approach is found in Section D.2.4. Secondary elements such as hanging ceiling structure, stairs, etc. are not modelled, however their mass is accounted for.

The geometry is created by a series of column lines to identify plan locations and levels for sections in the horizontal plane. The model has been defined, in detail, per the original structure drawings including the irregular pitching of the roof. To accomplish this, hundreds of column lines are required. Figure D.2-3 shows the primary plans up the height of the structure; only the foundation plan shows the column line numbers for clarity. The various colours of the plotted elements designate different types of structural elements, as noted. The plots in Figure D.2-4 are the primary longitudinal and transverse building elevations along gridlines C and 3.

Section 5.1 of the DSA provides a detailed description of the building geometry and should be read in conjunction with this model description.

Model axis X is referred to as the longitudinal direction and is positive pointing south along gridline C. The Z axis is the transverse direction, orientated East-West along gridline 3. These axes and the model origin (at the intersection of gridline C and 3) are shown in Figure D.2-3.

In pursuit of upper-bound superstructure accelerations, a number of simplifications are made in the model which may not be sufficient for a full Tier 3 evaluation. Further model refinement is required in potential future design phases. A brief summary of these simplifications is as follows:

- Limited nonlinearity in vertical lateral load resisting elements (cantilever Bay Wall systems and isolated columns).
- The diaphragms, precast concrete elements, and elements which are not part of the lateral load resisting system, are input as elastic to simplify analysis run time.
- Strength and ductility assessments of these elements are checked in detail on the basis of the MRSA described on Appendix C Section C.4.
- Sensitivity to degradation of these elements is assessed in Appendix C Section C.3.4.
- Degradation of these elements is not expected to significantly impact the lateral response characteristics nor diaphragm accelerations.
- Diaphragm discontinuities (seismic gaps) have not specifically been implemented.
- Vertical and horizontal soil stiffnesses kept at upper bound values.

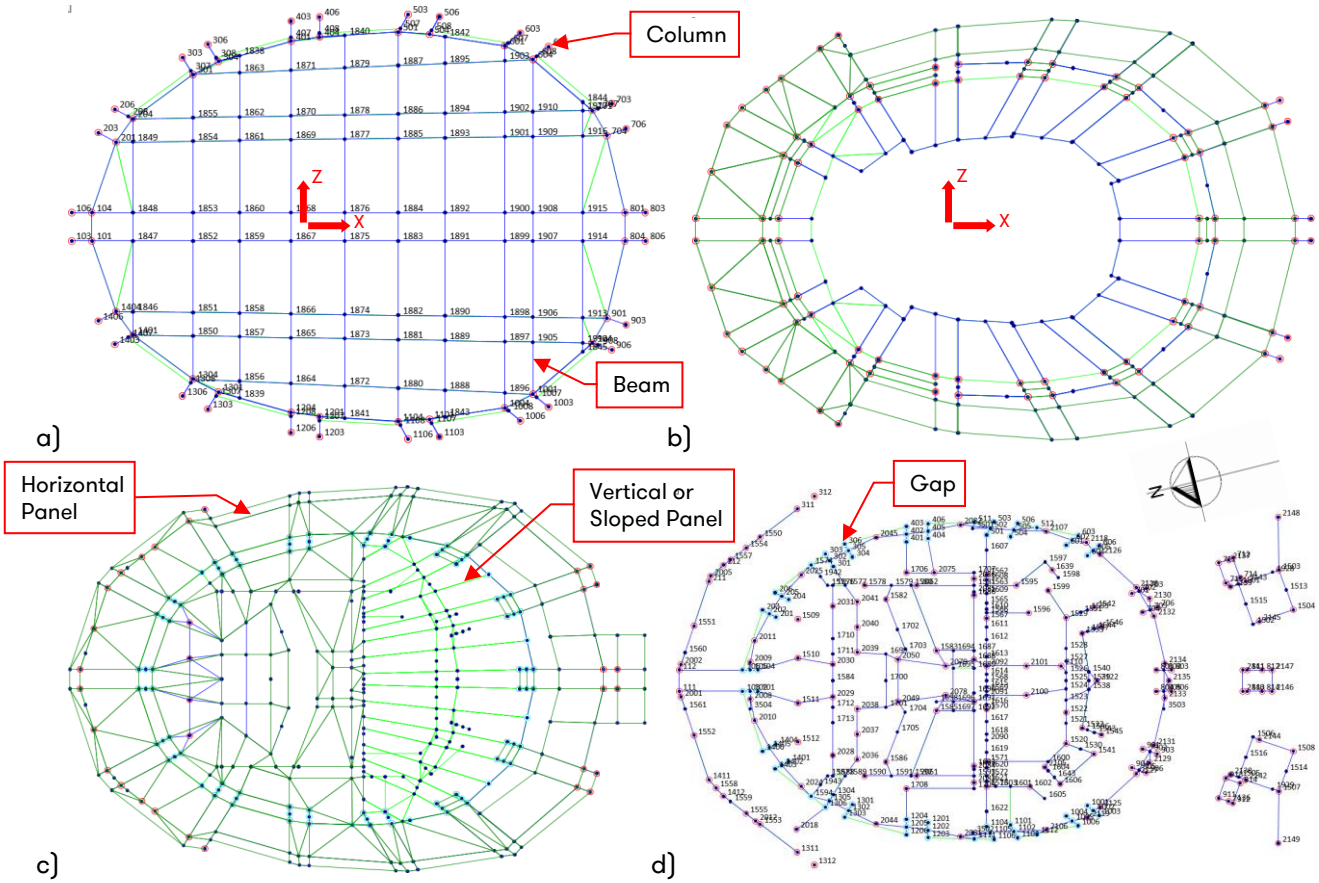

Figure D.2-3: ANSR plans of the Auditorium a) Roof b) Level 3 to 4 c) Level 1/2 to 2 d) Level 1 and foundation plan

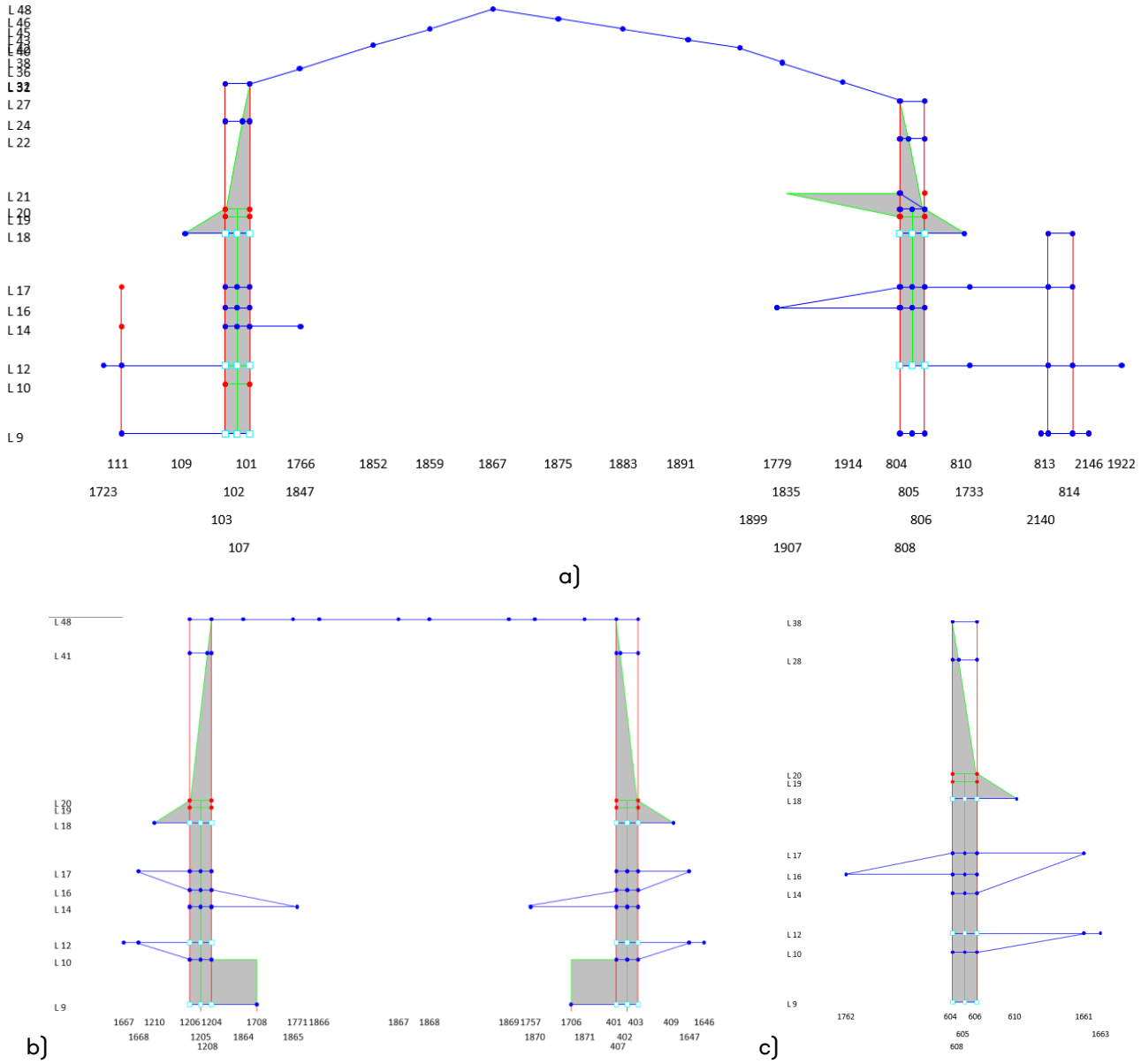

Figure D.2-4: ANSR elevations of Auditorium:
a) longitudinal
b) transverse
c) typical buttress elevation

Walls

Wall elements are modelled as 2D membrane elements that have no out-of-plane stiffness. Therefore, any mid-height or mid-bay corner nodes must be braced for analytical out-of-plane stability. This is accomplished for the buttresses of the cantilever Bay Walls by providing column elements at the extents, as boundary elements. These boundary elements are referred to as capped and un-capped, as reflected in Figure D.2-5. The buttresses are capped when they receive a Precast Cantilever Beam and have a different reinforcing ratio and smaller compressive area to the un-capped condition. The modelled column elements also serve to pick up the cantilevered gravity load. The precast beam and brace elements back-span across the buttress wall panels to create rotational fixity for the cantilevered truss. As seen in Figure D.2-4, most buttress walls support three Precast Cantilever Beams: for the Level 2 and 4 promenades and the Inner Auditorium upper seating. The Precast Cantilever Beams are modelled as elastic, with no strength and stiffness degradation, and are evaluated as discussed in Appendix C Section C.4.3.2.

The Bay Walls have vertical gap elements at structural Levels 1 and 2. The panel elements only reflect the shear behaviour of the wall and vertical gap elements are added at key elevations to reflect wall flexural hinge behaviour. Flexural rotations are expected to be concentrated at the interface with the foundation system (Level 1) and where there is a significant change in concrete strength and lateral load redistribution to the inner squat concrete walls (Level 2). Additional gap elements exist at ANSR level 18 to capture the flexural discontinuity between the precast buttresses and the insitu buttress walls. Above this level, all precast frame elements and precast panels are modelled without strength and stiffness degradation and are evaluated in Appendix C Section C.4.1.1.1.

Various horizontal cantilever wall segments are present in the building, appearing as spandrels (refer Figure D.2-5). The elements have a seismic separation to the adjacent Bay and do not contribute the lateral resistance of the system. They have been omitted from the model for simplicity, with a negligible amount of mass missing.

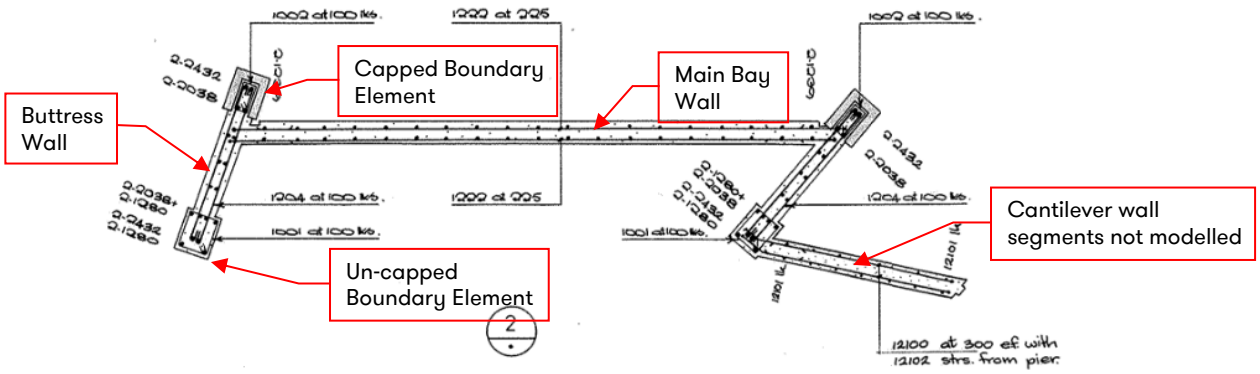

Figure D.2-5: Typical section of cantilever Bay wall and buttresses

Diaphragms

Diaphragms have typically been modelled as membrane panel elements; the only rigid diaphragm designation is at the ground floor slab-on-grade. The membrane panel diaphragms are discretised into three and four node elements to pick up all associated column and wall nodes connecting to the diaphragm, see Figure D.2-3 for the complexity of the Level 2 slabs. A key transfer mechanism exists at the wall discontinuity along Bays 6 through 6a to the squat Wall 21 through the sloping diaphragm (refer DSA Figure 5-6). This sloping diaphragm has a thin upper panel (largely modelled to capture the mass of the non-structural precast seating panels) and a thicker 200mm lower diaphragm. This stacked configuration can be seen in the 3D section cut of Figure D.2-6. Between these two horizontal sets of panels are vertical panel elements used to represent the tapered beams, which allows connectivity to the top and bottom side of the element (refer Figure D.2-3).

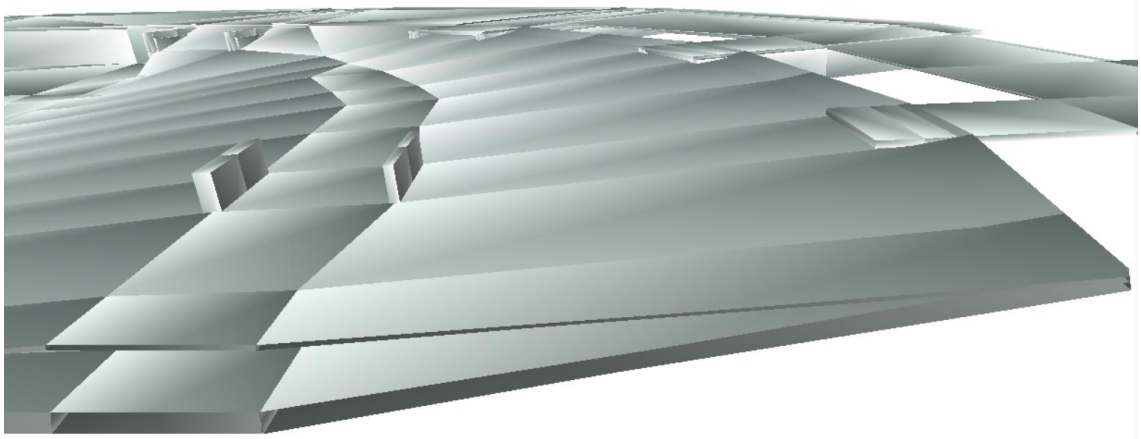

Figure D.2-6: 3D ANSR section of Auditorium Level 1/2 sloped diaphragm

The second complex diaphragm occurs between Level 3 to 4, refer Figure D.2-7. The seismic gapping noted in the diaphragm analysis of Appendix C Section C.4.2 has not been implemented, as one of the modelling simplifications deemed appropriate in determining an upper-bound ULS diaphragm accelerations. The interior seating has maintained more of its true geometry, as compared to the ETABS model. The tiered seating effect spreads the mass distribution between Level 3 and 4, seen in Figure 5-3 of the DSA main body, where the seating elevation is higher in the south and low in the north. There is no diaphragm connectivity between the sloped floor seating and the backstage area at Level 3.

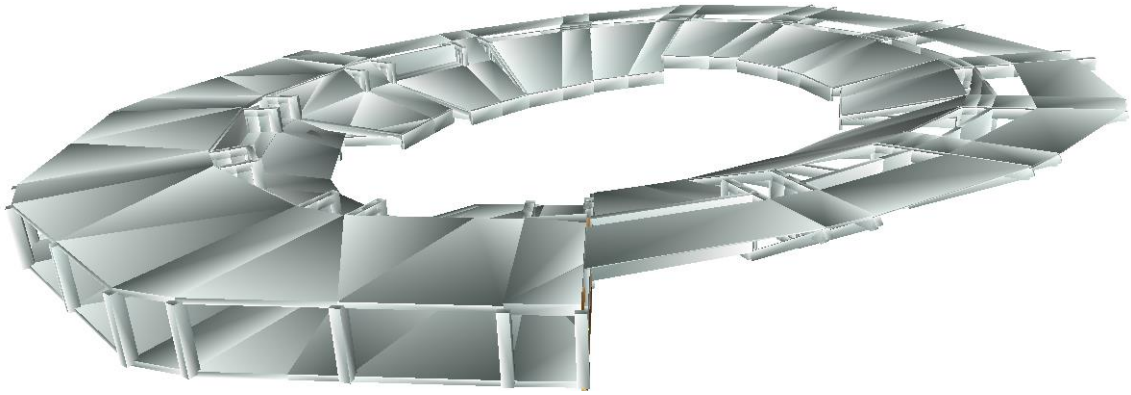

Figure D.2-7: 3D ANSR section of Auditorium Level 3 to 4

The Dycore roof has been modelled at its true slope and is meshed to its gravity supporting truss elements, see Figure D.2-8. The trusses and their connections to the supporting precast walls and precast buttresses frames have been idealised. The evaluation of these elements can be found in Appendix C Section C.4.3.1. For the purpose of this model, the roof trusses are simplified to an effective I-section with the same approximate stiffness and mass. Dummy gravity and lateral ledger elements existing to reconcile the truss plan locations back to wall and frame nodes required for support (not shown in Figure D.2-8).

The timber roof, glulam framing, and façade mullions evaluated in Appendix G Section G.3 are not modelled. They represent a small amount of the Auditorium mass and do not contribute to primary lateral load resisting system. The one location where this omission is impactful is in the entry frames, discussed in the following subsection for the isolated column elements.

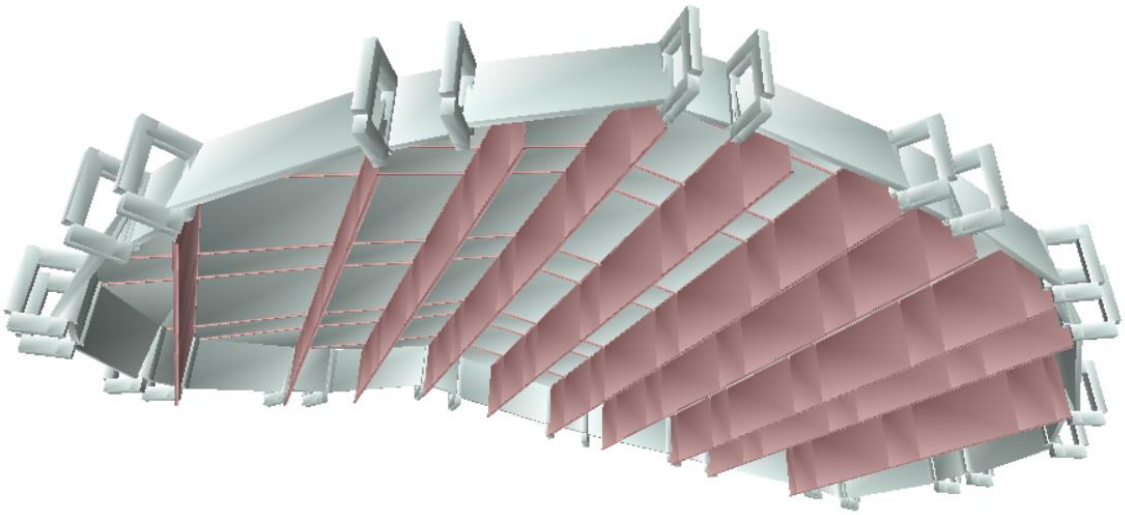

Figure D.2-8: 3D ANSR section of Auditorium Roof

Isolated column elements

A limited number of isolated frame extension columns exist in the lobby, which are not under a wall system. These are located outside Bays 6 through 6a where the Bay Wall web area is terminated. Figure D.2-9 shows these frames and the three groups of four isolated columns. A lightweight timber roof occurs between these columns which loosely ties them to the Independent Staircase Structures. Omission of this framing and diaphragm may under restrain the top story of columns, those above Level 4. See Section D.2.7.2 for mitigation of this effect.

Other isolated columns exist in Wall 1 and four columns under the stage at the north of the structure.

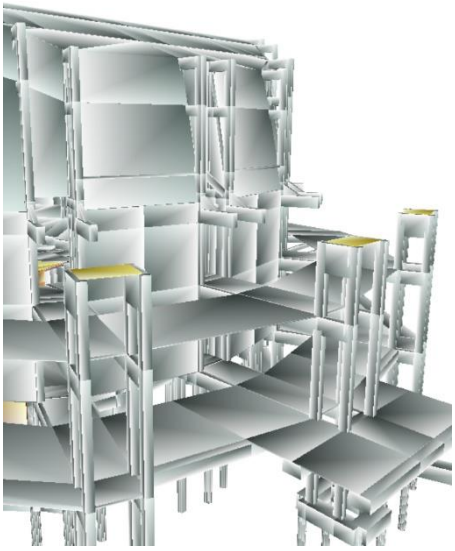

Figure D.2-9: Auditorium Lobby frame extensions and isolated columns

D.2.2 Material Properties

Materials used for analysis are listed in Table D.2-1. Material properties are probable strengths based on the properties specified in the original structural specifications and supplemented by the Engineering Assessment Guidelines. Of note the mortar strength, f'_m, for the concrete masonry unit (CMU) walls at Level 1 is 12MPa per the specifications. A factor of 1.3 to calculate probable compressive strength is used per ASCE 41-17 Table 11-1. A massless concrete Material 5 is defined for analytical elements required for stability but are not present in the structure eg. ledgers and flexural element back spans. Dummy timber Material 12 is used for diaphragm continuity at the stage and local stability at the top of the isolated frame columns (refer Figure D.2-9).

Table D.2-1 : Material Properties for NLTHA

ID	Material	Strength (f_b, f_c, f_y) [MPa]	E [kPa]	G [kPa]	ρ [-]	Weight (γ_w) [kN/m³]	Mass (γ_m) [t/m³]
1	20 MPa Concrete (foundation)	30000	25742960	10726233	0.20	25.0	2.55

ID	Material	Strength (f_b, f_c, f_y) [MPa]	E [kPa]	G [kPa]	ρ [-]	Weight (γ_w) [kN/m³]	Mass (γ_m) [t/m³]
2	275 MPa reinforcing	324000	200000000	76923077	0.30	78.0	7.95
3	BS1460: 1972 (Structural steel)	232000	200000000	76923077	0.30	78.0	7.95
4	Concrete masonry block walls	15600	15000000	6000000	0.25	18.0	1.83
5	25 MPa Concrete (massless)	37500	28781504	11992294	0.20	0.0	0.00
7	380 MPa reinforcing	455000	200000000	76923077	0.30	78.0	7.95
10	25 MPa Concrete (Typical)	37500	28781504	11992294	0.20	25.0	2.55
11	35 MPa Concrete (Special)	52500	34054735	14189473	0.20	25.0	2.55
12	Dummy timber	12000	6500000	2241379	0.45	8.0	0.82
13	Reduced strength gap Material	3500	8792895	3663706	0.20	25.0	2.55

D.2.3 Building Masses, Weights, Mass Eccentricity and P-Delta

The mass and weight of the building is captured by the self-weight of the modelled elements. The superimposed dead and seismic live loads have been implemented by applying an effective uniform area weight. ANSR allows two definitions of thickness, one for stiffness/strength and one for mass/weight. The thickness for mass/weight was increased to include mass related to superimposed dead (SDL) and live loading (LL) requirements. These loadings are noted in Appendix C Section C.2.7.

Table D.2-2 summarizes the total seismic mass and weight by floor level for D + ψ_EQ as defined in NSZ1170.5. As per typical analyses, seismic weight and mass are often introduced to models separately. The difference between the global ANSR seismic mass and weight in the model is less than 1% indicating appropriate implementation. This is in addition to the comparison to the total mass as compiled by ETABS. When comparing to ANSR mass, there is a 1% difference in total mass. There is variance between the two models when looking at story mass and the percent difference calculation has been largely carried out two levels at a time. This is done to acknowledge the following known discontinuities between the models:

- **Roof and Level 5**: The ETABS model does not incorporate the sloping roof elevation and therefore will have less mass associated to it. ETABS automatically meshed all structural elements at Level 5 even though the diaphragm is only located on the southern end, artificially lumping mass the level. This same limitation is not present in ANSR, allowing mass to be attributed only above and below Level 5, as appropriate. Therefore, the total mass difference of the Roof and Level 5 combined should be similar.

- **Level 4 and Level 3**: The ETABS model does not incorporate the change in vertical elevation of the interior seating, as ANSR does. Additionally, all the bottom braces of the Precast Cantilever Beams and walls are auto meshed to Level 3 artificially increasing the associated mass here. Therefore, the combined mass between the two floors should be similar.
- **Level 2 and Level 1/2**: Intermediary levels are defined differently due to separate modelling approaches for the sloped diaphragm portion of the Auditorium. Similar ETABS auto-meshing issues exist at Level 1/2 for the model and so a lumped comparison for the floors is considerate appropriate.
- **Level 1**: the highest mass discontinuity exists at this level due to more complete modelling of the ground beam system in ANSR. This was required for model stability with implementation of the horizontal soil stiffness elements.

Table D.2-2: ANSR Building Masses and Weights with ETABS Comparison

Level	ANSR Level	ANSR Elevation [m]	Mass [kN]	Weight [kN]	Sum Mass [kN]	Sum Weight [kN]	ETABS Mass [kN]	Difference[1] [%]
Roof	>=22	22.335	14027	14080	14027	14080	10104	
Level 5	21	18.550	7217	7262	21244	21342	11598	-2.2%
Level 4	17	12.060	19782	19963	41025	41305	18314	
Level 3	14	9.340	6969	7035	47994	48340	8612	-0.7%
Level 2	12	6.620	18522	18536	66516	66876	19940	
Level 1/2	10	5.320	11760	12161	78276	79037	11906	-5.2%
Level 1	9	1.900	28482	28230	**106758**	**107267**	24906	12.6%
B.O. Pile [2]	4	-3.000	0	0	106758	107267	**105379**	1.3%

[1] Intermediary levels are used to connect gravity structure. Mass and weight occurring at these elevations are manually lumped to the adjacent level for reporting purposes.
[2] Bottom of pile is not true founding depth, see Section D.2.4.

A 10% mass eccentricity is not investigated in the ANSR model. For the purpose of estimating the average acceleration of a floor, it was not considered impactful for a well distributed ring-shaped lateral system.

P-Delta is modelled by including geometric stiffness of elements. Generally, P-Delta effects are produced by the vertical self-weight of modelled elements and applied vertical imposed loads, unless specifically overridden.

D.2.4 Foundation and Soil Interface Modelling

Nonlinear soil-structure interaction (SSI) is not included in the analysis. Nor are foundation beams and piled foundations directly evaluated; they are assessed through investigation of the ETABS model (see Appendix C Section C.2.4 and C.4.4). The ETABS model implements the nonlinear lateral foundation flexibility provided by piles and passive pressure on the ring beam at individual pile locations. The change in lateral stiffness and behaviour through liquefaction and in the undegraded soil conditions is investigated there

through sensitivity studies. ANSR foundation vertical and horizontal stiffnesses were informed by the Geotechnical Assessment Report and the global initial lateral stiffness determined in the ETABS model.

Lateral soil stiffness is implemented in a simplified manner into ANSR. The softening behaviour caused by lateral foundation flexibility is influential in predicting the superstructure response. Therefore, its inclusion is required but still with the aim of predicting an upper bound for the diaphragm accelerations. The sum of all nonlinear link elements in ETABS produces a global foundation stiffness backbone curve. Only the elastic initial stiffness, for the undegraded soil condition, is input into ANSR; this reflects the stiffest soil condition. Dummy flexural elements are modelled at the four extents of the ring beam, where the majority of the soil resistance occurs (refer Figure D.2-10). The axial stiffness of the dummy elements is adjusted to reflect half of the soil stiffness in each direction. This approach captures the effective translational stiffness and likely over-predicts the rotational stiffness of the system. This is considered appropriate for the model requirements. The vertical pile elements are pinned at each end to not contribute to the lateral stiffness.

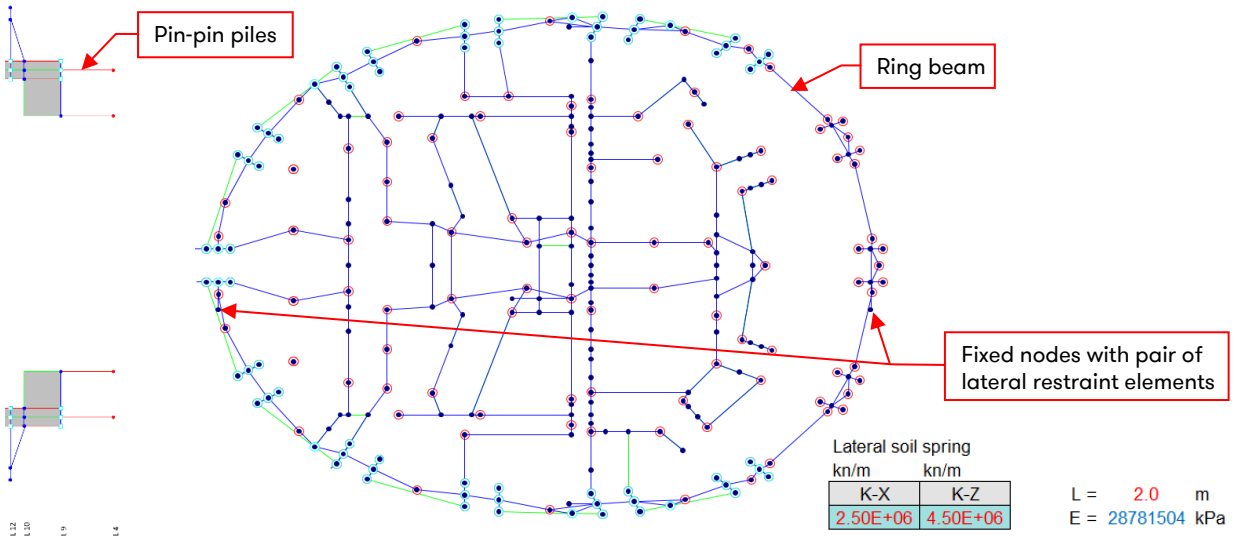

Figure D.2-10: Auditorium ANSR foundation modelling approach; section (left), plan (right)

Vertical soil stiffness values provided by Tonkin+Taylor (T&T) have an upper and lower bound for both compression and tension for each pile type. The Pile Cages A to G are associated to each pile location as noted on the original structural drawings and refer to DSA Section 5.1.3.2. See Table D.2-3 for modelling properties used for the foundation stiffness elements. The pinned vertical elements have their axial stiffness properties adjusted to achieve the recommended upper bound compression spring stiffness properties and have P-delta effects removed. These elements are linear elastic and do not mimic the difference in compressive and tension stiffness. Use of the stiffer compressive stiffness is considered appropriate in predicting a maximum superstructure acceleration.

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Table D.2-3: ANSR modelled foundation stiffness elements

Frame Element	Section ID	Material ID	Section Type	X Dimension (Beam Depth Below)	Z Dimension (Beam Width)	Beam Depth Above	Flange Thk. T_{fl}	Web Thk. T_w	Component Type	Strain Hardening Ratio	Area Overide[1] [m²]
Pile Cage A	102	5	CIRCLE	0.450					132	0.010	0.03807
Pile Cage B	103	5	CIRCLE	0.560					132	0.010	0.05330
Pile Cage C	104	5	CIRCLE	0.450					132	0.010	0.03807
Pile Cage D	105	5	CIRCLE	0.560					132	0.010	0.05330
Pile Cage E	106	5	CIRCLE	0.450					132	0.010	0.03807
Pile Cage F	107	5	CIRCLE	0.560					132	0.010	0.05330
Pile Cage G	108	5	CIRCLE	0.900					132	0.010	
Massless Ground Restraint X	653	5	CIRCLE	0.500					101		0.08686
Massless Ground Restraint Z	654	5	CIRCLE	0.500					101		0.15635

[1] Area overrides used to reduce element elastic stiffness to the required soil stiffness value, either vertical or horizontal.

D.2.5 Modal Response

An eigen analysis of the model, including the foundation initial horizontal stiffness was performed in ANSR to determine the fundamental periods and mode shapes. These properties are summarized in Table D.2-4; the two primary translational modes and rotational mode are shown visually in Figure D.2-11. The period in the longitidinal direction, model direction X, is 0.47s and the transverse direction, model direction Z, is 0.41s. The number of modes needed to achieve 90% mass participation for both directions is small due to the inclusion of the lateral soil springs; which means the Auditorium largely behaves as a rigid box on a flexible base. The mass partication ratios are used to calculate the damping model as discussed in Section D.2.6.

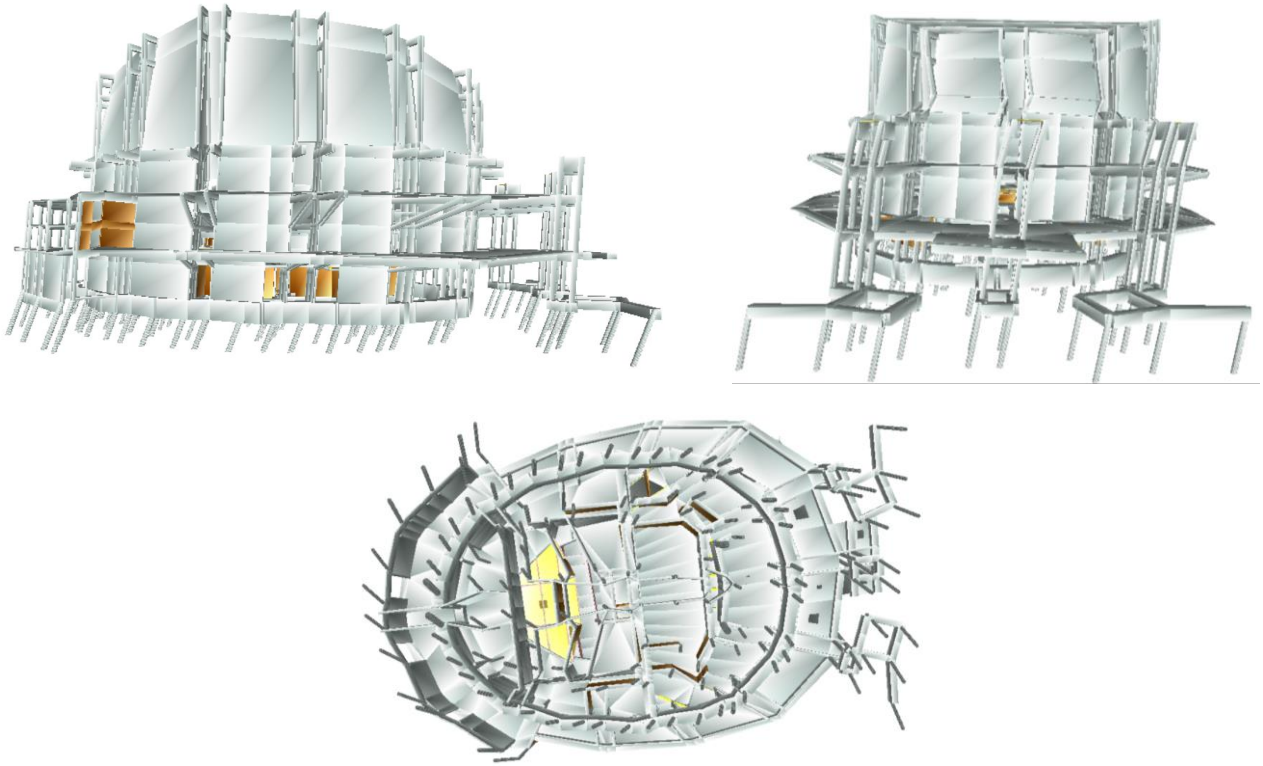


Figure D.2-11: ANSR fundamental modes 0.47s X longitudinal translation (left), 0.41s Z transverse translation (right), 0.29s torsion (centre)

Table D.2-4: Auditorium ANSR modal properties with horizontal stiffness

Mode	Period [Seconds]	Effective Mass			Cumulative Mass		
		EM X	EM Z	EM YY	SEM X	SEM Z	SEM YY
1	0.467	96.41%	0.16%	0.00%	96.41%	0.16%	0.00%
2	0.406	0.21%	82.50%	0.06%	96.62%	82.66%	0.06%
3	0.290	0.00%	4.15%	6.14%	96.62%	86.81%	6.20%
4	0.220	0.00%	0.01%	0.39%	96.62%	86.82%	6.59%
5	0.218	0.00%	0.00%	0.00%	96.62%	86.82%	6.60%
6	0.212	0.00%	0.00%	0.05%	96.62%	86.82%	6.64%
7	0.185	0.04%	12.08%	0.08%	96.66%	98.91%	6.73%
8	0.180	2.55%	0.07%	0.00%	99.20%	98.98%	6.73%
9	0.171	0.54%	0.00%	0.01%	99.75%	98.98%	6.74%
10	0.157	0.00%	0.05%	0.29%	99.75%	99.03%	7.03%
11	0.148	0.00%	0.00%	0.37%	99.75%	99.04%	7.40%
12	0.141	0.00%	0.00%	0.01%	99.75%	99.04%	7.41%
13	0.135	0.00%	0.00%	0.00%	99.75%	99.04%	7.41%
14	0.135	0.00%	0.01%	0.00%	99.75%	99.05%	7.41%
15	0.130	0.00%	0.00%	0.00%	99.75%	99.05%	7.42%
16	0.127	0.00%	0.00%	0.00%	99.76%	99.05%	7.42%
17	0.127	0.00%	0.01%	0.06%	99.76%	99.06%	7.48%
18	0.123	0.00%	0.03%	0.36%	99.76%	99.08%	7.85%
19	0.122	0.00%	0.00%	0.07%	99.76%	99.09%	7.91%
20	0.117	0.00%	0.00%	0.89%	99.76%	99.09%	8.80%
21	0.117	0.00%	0.01%	0.63%	99.76%	99.10%	9.43%
22	0.117	0.00%	0.04%	0.82%	99.76%	99.14%	10.25%
23	0.116	0.02%	0.00%	0.15%	99.78%	99.14%	10.39%
24	0.113	0.01%	0.02%	0.27%	99.79%	99.16%	10.67%
25	0.112	0.02%	0.00%	0.06%	99.81%	99.16%	10.73%

D.2.6 Damping for NLTHA

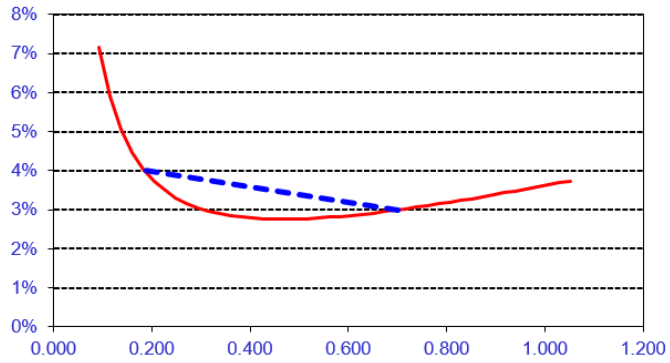
The Rayleigh damping model used in the NLTHA is adopted as described in ASCE 41-17 Section 7.4.4.4. A maximum target damping ratio of 3% at the first translational mode (including expected period lengthening) is used in accordance with ASCE 41-17 Section 7.2.3.6. Expected period lengthening is taken to be $1.5*T_1$.

The above damping definition differs from, and is more conservative than, that recommended in the Assessment Guidelines. However, ACSE 41-17 Section 7.4.4.4 introduces further manipulability to the damping model:

- The weighted average of the equivalent damping ratio, over the period range of damping, shall not exceed the target damping ratio, and
- No more than eight times the damping ratio provided at the higher period is provided at T_{90}.

These bounds allow for increasing the damping provided at T_{90} which has the effect of increasing the minimum damping provided at any period. This is considered the most contemporary approach to damping and reduces the potential for under damping the primary superstructure mode, where a large number of higher modes or high period elongation exists.

Mass and stiffness proportional damping methods are included with initial stiffness damping generally used for the stiffness proportional part of the matrix.



Mode	T	Damping
1	0.70	0.03
2	0.18	0.04
Alpha	0.374665	
Beta	0.002031	

	X	Z	
Weighted average:	2.76%	2.94%	OK

Use 1.5*T_1= **0.70** s
Use T_90= **0.18** s

Figure D.2-12: Rayleigh damping coefficients of NLTHA

D.2.7 Element Modelling Parameters

There is a mix of nonlinearly and elastically responding elements. To rationalize the model complexity for determining diaphragm accelerations, nonlinearity is limited to vertical lateral load resisting elements - generally the primary cantilever wall systems and isolated columns. The diaphragms, precast concrete elements, and elements which are not part of the lateral load resisting system, are input as elastic to minimise analysis run time and improve the numerical stability of the model. Degradation of these elements is not expected to significantly impact the lateral response characteristics.

Strength properties are determined by Section C5 of the Engineering Assessment Guidelines. Where required, supplementary information is adopted from ASCE41-17.

D.2.7.1 Concrete Wall Elements

D.2.7.1.1 Concrete Wall Element Shear Model

The shear stiffness of the concrete shear wall elements is modelled using plane stress elements with a thickness corresponding to the values specified on the drawings. These panels are generally modelled a full storey height. Although they are meshed between diaphragms if they are required to connect Precast Cantilever Beams, refer Figure D.2-4.

The modelled behaviour includes degradation in strength and stiffness, depending on the level of shear stress. The wall panel yield function is shear controlled. A strength envelope as shown in Figure D.2-13 is developed. Regions of this backbone curve are defined, respectively, as the strength provided by the concrete, the nominal strength of the shear reinforcing (strain of 0.004), and onset of strength degradation at a strain which is a function of the axial load level (typically a strain of 0.010). The axial load for all cantilever walls is below 0.05 Ag*f'c; consequently, no walls are classified as force controlled.

Under cyclic loading, non-recoverable stiffness degradation is modelled; however, for a pushover, the load-deformation response follows the strength backbone, see Figure D.2-14.

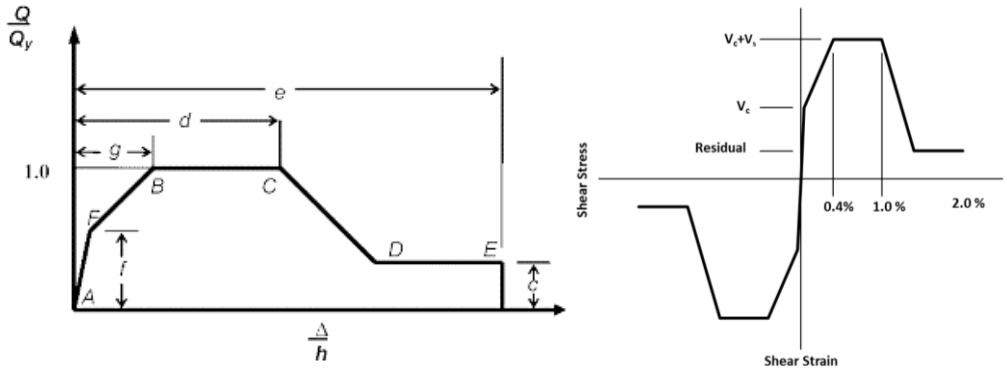


Figure D.2-13: Concrete wall element shear strength backbone definitions (left); strength backbone for a wall with high axial load (right)

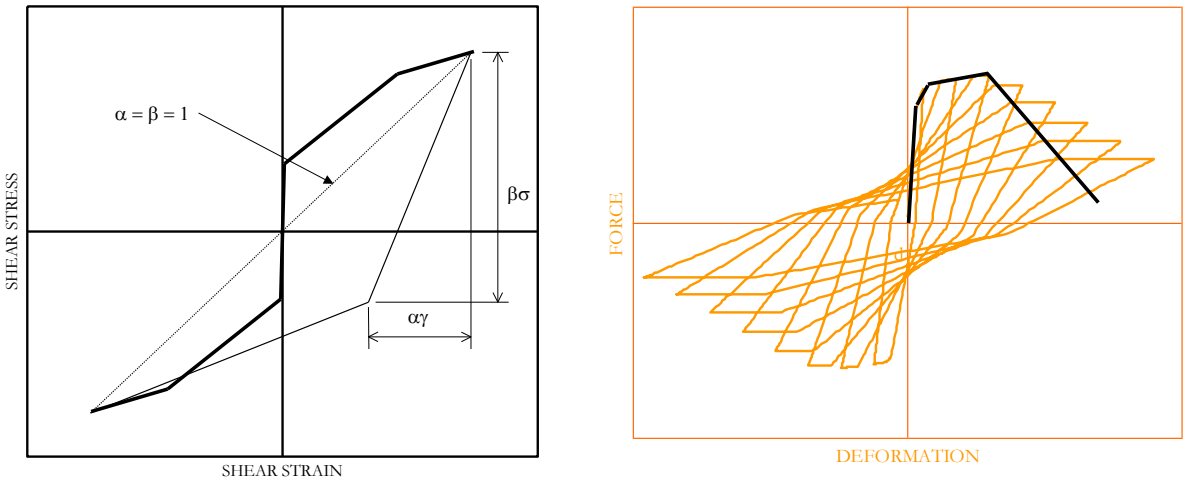


Figure D.2-14: Definition of stiffness degradation (left); in-cycle degradation used for static force pushover shown in black, and cyclic degradation used for NLTHA shown orange (right). Figures indicate a strain hardening slope, but this is set to zero per Figure D.2-13

D.2.7.1.2 Concrete Wall Element Flexure Model

The panel element models the shear degradation of walls; however, higher aspect ratio walls may be controlled by flexure. To model flexural yielding, gap elements are placed at each node across the length of the wall at the elevation at which yielding is expected to occur. Each gap element contains two elements in parallel, a concrete element which is elastic in compression but with zero tensile strength and a reinforcing bar element which is bi-linear, yielding in both tension and compression. The inelastic rotation can be determined by the width of the gap opening divided by the wall length. The concrete area and steel area at each gap is taken as the tributary areas of all panels incident to each gap.

Columns, which pass through the walls as boundary elements, cross the gaps and therefore also carry tensions and compressions. Plastic deformations in column elements include both axial extensions and chord rotation in proportions determined by the vector through the yield surface. Where the boundary element columns are split mid-floor height, to accommodate connectivity of the Precast Cantilever Beams, this creates a point of potential model instability. The nonlinear and degrading flexural column element can degrade on either side of the laterally unrestrained mid-height column node and cause convergence

instability. Consequently, the ends of the boundary element at the mid-height node have been assigned an artificially high strength to maintain the out-of-plane stability of the wall boundary element, and therefore the panel.

Flexural strength and stiffness degradation can be included, however we do not expect significant degradation to occur in this model for the range of seismic loads considered. This additional superstructure softening and damping would also likely predict lower diaphragm accelerations. Therefore, the hinge elements are bilinear with no strength or stiffness degradation modelled.

D.2.7.1.3 Modelled Concrete Wall Elements

Table D.2-5 outlines the shear panel elements used in the Auditorium model.

Table D.2-5: ANSR modelled wall panel sections

Location	Plate ID	Material ID	Structural Thickness (m)	Thickness for Weight/Mass (m)	Reinforced Sections			Component Type	Shear Strength vc	Reinf. Strength vs
					Material ID	Vert. ρv	Horiz. ρh			
W16, W18	10	11	0.250	0.250	7	0.0080	0.0080	50	1203	3658
W17,18,19,20,21,22	11	1	0.300	0.300	2	0.0067	0.0067	50	910	2171
W8, 10	12	11	0.200	0.200	2	0.0032	0.0025	50	1203	814
W7	14	11	0.200	0.200	2	0.0045	0.0045	50	1203	1464
W21 L8-10	16	11	0.600	0.600	2	0.0126	0.0126	50	1203	4069
W21 L10-12	17	11	0.150	0.150	2	0.0026	0.0038	50	1203	1220
Bays to L12	18	11	0.250	0.250	7	0.0026	0.0080	50	1203	3658
Bays to L17	19	10	0.250	0.250	7	0.0026	0.0054	50	1017	2439
Bays to L20	20	10	0.250	0.250	2	0.0026	0.0040	50	1017	1302
Bays above L20	21	10	0.250	0.250	2	2.0000	0.0040	50	910	838
W17 & W60	22	11	0.200	0.200	2	0.0057	0.0057	50	1203	1831
W1	23	10	0.200	0.200	7	0.0073	0.0073	50	1017	3326
W6	24	11	0.200	0.200	7	0.0080	0.0080	50	1203	3658
W11	25	11	0.200	0.200	7	0.0101	0.0101	50	1203	4573
W14	26	11	0.200	0.200	7	0.0134	0.0134	50	1203	6097
Bays 4&5 to L12	27	11	0.250	0.250	7	0.0026	0.0144	50	1203	6531
Buttress to L12	28	11	0.250	0.250	2	0.0190	0.0161	50	1203	5210
Buttress to L18	29	10	0.250	0.250	2	0.0074	0.0090	50	1017	2929
Buttress Precast Joint	30	10	0.400	0.400	2	0.0048	0.0057	50	1017	1831
Block Walls	31	4	0.190	0.190	2	0.0020	0.0020	58	700	642

D.2.7.2 Frame Element Model

Frame elements use concentrated plasticity models. For beam and column frame elements the strength is modelled as a bilinear yield function (up to a certain point beyond which strength is degraded). The elastic stiffness is based on effective properties up to the calculated probable yield moment. Properties are defined by axial area, shear area, and moment of inertia about each axis.

Beams have a separate yield moment specified for positive and negative bending at each end of the beam. Once the yield moment is attained, the flexural stiffness is reduced to the initial stiffness times the specified strain hardening ratio.

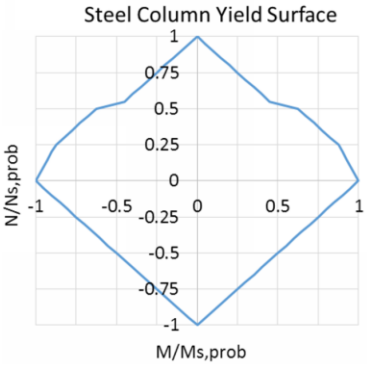
Columns are represented by a flexural element similar to beams. However, the yield moments about each axis and the axial load are coupled. An interaction diagram is calculated based on probable material strengths. For concrete columns, the interaction between bending moments and axial load is parabolic and is defined by:

$$\sqrt{\left(\frac{M_y}{M_{yu}}\right)^2+\left(\frac{M_z}{M_{Zu}}\right)^2}+\left(\frac{F-F_o}{F_u}\right)^2=1.0 \qquad F_o=\frac{1}{2}(F_{ut}-F_{uc}) \qquad F_u=\frac{1}{2}(F_{ut}+F_{uc})$$

M_y, M_z, and F denote bending moments about the element local y- and z-axes and the axial force respectively (F is negative for compression). Subscript 'u' denotes ultimate. F_{ut} and F_{uc} are axial ultimate strengths in tension and compression (taken as absolute value). M_{yu} and M_{zu} are the maximum balanced moments. As for the beams, a bilinear strain hardening yield function is generally used in the model.

Another yield surface is included for modelling steel columns and is defined as shown in Figure D.2-15.

Frame elements are elastic by default unless a Strength ID is associated to the member. A Strength ID provides the probable strength values at which yielding occurs, as calculated per the Engineering Assessment Guidelines. Per the analysis objective discussed, only the entry frame and boundary element members have this yielding designation.



For columns in tension: $\frac{M_y}{M_{yu}}+\frac{M_x}{M_{xu}}+\frac{F}{F_{ut}}=1.0$

For columns in compression where F/Fuc < 0.20: $\frac{M_y}{M_{yu}}+\frac{M_x}{M_{xu}}+\frac{F}{2F_{uc}}=1.0$

For columns in compression where 0.20 ≤ F/Fuc ≤ 0.5: $\frac{8}{9}\left[\frac{M_y}{M_{yu}}+\frac{M_x}{M_{xu}}\right]+\frac{F}{F_{uc}}=1.0$

For columns in compression where F/Fuc > 0.50: $\frac{M_y}{M_{yu}}+\frac{M_x}{M_{xu}}+\frac{F}{F_{uc}}=1.0$

Figure D.2-15: Yield Surfaces for steel columns

The flexural element used for beams and columns permits degrading strength and/or stiffness characteristics to be specified. The implementation of strength backbones and stiffness degradation is shown in Figure D.2-16. Cyclic degradation is used for dynamic analyses. For static pushover analysis, the degradation follows the strength backbone curve.

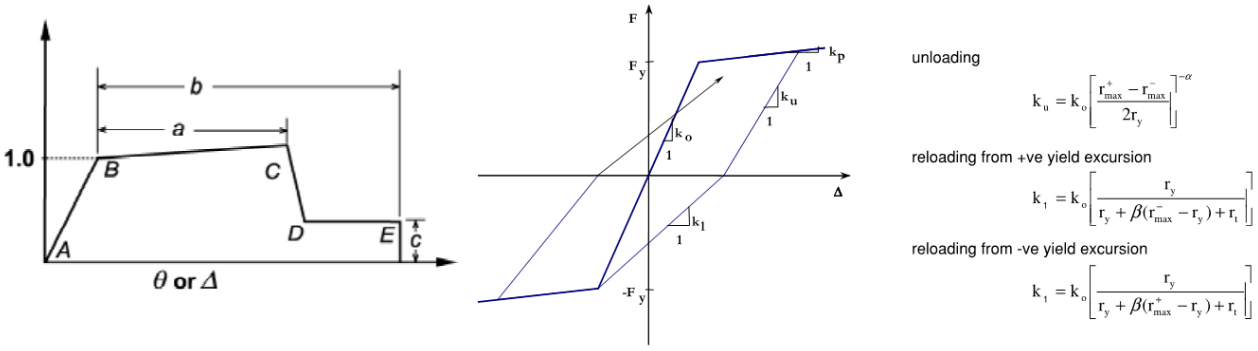


Figure D.2-16: Definition of Modelling Parameters for Reinforced Concrete Frame Elements

Due to the timber roof not modelled between the Auditorium and the Independent Stair Structures (refer Section D.2.1 for isolated column elements), this lack of lateral restraint creates artificial modes and degradation in the top story. These have been classified as elastic for model stability.

D.2.7.2.1 Modelled Concrete Column Elements

For yielding elements, flexural strengths and transverse reinforcing are defined at each end of the column. The column behaviour is a function of axial load and shear demand. The program automatically calculates the appropriate allowable plastic rotation. No confinement or shear checks are required in post-processing, as this is already incorporated in the assignment of modelling parameters and the setting of plastic rotation limits.

Table D.2-6 presents the concrete column and beam elements in the model. Only the Lobby frame extension elements with strength designations behave nonlinearly, as noted in Table D.2-7.

Table D.2-8 and Table D.2-9 show the boundary element sections and strengths used at the ends of buttress walls.

D.2.7.2.2 Modelled Concrete Beam Elements

Concrete beam elements are generally modelled as flexure-controlled elements with conforming tie elements. Since they are typically gravity only beams (i.e the Precast Cantilever Beams) they are maintained as elastic and the element hysteresis is not used.

Nonlinear behaviour sections and strength properties for the entry frame are included in the section above and in Table D.2-6 and Table D.2-7 below. These beam elements are generally modelled as flexure-controlled elements with conforming or non-conforming ties as appropriate to their detailing shown in the original structural drawings. Inadequate development or anchorage into the beam-column joint is also taken into account. The only shear-controlled beam elements are the arch shaped linked beams in the precast column pairs of the Lobby frame extensions.

Table D.2-6: ANSR modelled concrete frame elements

Frame Element	Section ID	Material ID	Section Type	X Dimension (Beam Depth Below)	Z Dimension (Beam Width)	Beam Depth Above	Flange Thk. T_f	Web Thk. T_w	Component Type	Strain Hardening Ratio
Pile cap	110	1	RECT	0.900	0.900				131	0.010
Fin Col & base cols	202	10	RECT	0.500	0.400				131	0.010
Fin Dummy Web	203	10	RECT	2.200	0.400				131	0.010
Type C1	204	10	RECT	0.400	0.300				131	0.010
Wall 1 columns	205	10	RECT	0.400	0.400				131	0.010
Fin beam top and Bot	301	10	RECT	0.600	0.400				124	0.010
Fin base beams	302	10	RECT	0.000	0.400	0.800			124	0.010
Entry pier mid beam	303	10	RECT	0.900	0.400				126	0.010
Entry top of pier	304	10	RECT	1.480	0.400				124	0.010
Precast entry beams	305	10	RECT	0.900	0.400				125	0.010
Fdn beam	306	1	RECT	1.200	0.200				124	0.010
Ring Beam	307	1	RECT	2.000	1.500				124	0.010
Interior grade beams	308	1	RECT	2.000	0.300				124	0.010
Pile cap	309	1	RECT	0.900	0.900				124	0.010
Fdn beam	313	1	RECT	2.000	0.900				124	0.010
L2 C edge beam	314	5	RECT	0.650	1.000				124	0.010
Stage Beam	410	10	RECT	0.450	0.300				124	0.010
Stage Beam	411	3	OTHER	0.403	0.178		0.011	0.008	155	0.030
Stage Beam	412	3	OTHER	0.304	0.165		0.010	0.006	155	0.030
Stage Beam	413	3	OTHER	0.252	0.146		0.009	0.006	155	0.030
Roof Glulam Support	414	3	OTHER	0.308	0.305		0.015	0.010	155	0.030
Precast Beams PB1-PB6 Top	420	10	RECT	0.800	0.400	-0.200			124	0.010
Precast Beams PB1-PB6 Brace	421	10	RECT	0.400	0.400				124	0.010

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Frame Element	Section ID	Material ID	Section Type	X Dimension (Beam Depth Below)	Z Dimension (Beam Width)	Beam Depth Above	Flange Thk. T_f	Web Thk. T_w	Component Type	Strain Hardening Ratio
L2 Slab Beam	422	5	RECT	0.200	0.800				124	0.010
L2 Ring Beam	423	10	RECT	0.800	0.300				124	0.010
W62-W63 Conc Bm	424	10	RECT	0.800	0.400				124	0.010
W62-W63 CMU Beam	425	4	RECT	0.800	0.180				124	0.010
Precast Beam Level 4	426	10	RECT	0.600	0.400				124	0.010
Precast Brace Level 4	427	10	RECT	0.400	0.400				124	0.010
Roof Truss	502	3	I-SECT	3.500	0.150		0.038	0.003	155	0.030
Truss wall beam	503	5	RECT	3.500	0.300				124	0.010
Truss chord support	504	3	OTHER	0.150	0.075		0.010	0.006	155	0.030
Roof Wall Ring Beam	505	5	RECT	1.000	0.250				124	0.010
Stair Slab Beam	506	5	RECT	1.000	0.250				124	0.010
CMU Grade Beam	600	1	RECT	0.300	0.500				124	0.010
Conc Wall Grade Beam	601	1	RECT	0.900	0.250				124	0.010
Full BE L1-2	610	11	RECT	0.500	0.400				131	0.010
Full BE L2-5	611	10	RECT	0.500	0.400				131	0.010
Capped BE L1-2	612	11	RECT	0.400	0.200				131	0.010
Capped BE L2-5	613	10	RECT	0.400	0.200				131	0.010
Massless Buttress Outrigger	620	5	RECT	0.500	0.500				124	0.010
Massless Ring Beam Outrigger	621	5	RECT	2.000	1.500				101	
Massless Restraint Beam	650	5	RIGID_LINK	0.500	0.500				101	
Massless Dummy OOP Column	651	5	RECT	1.000	0.001				101	
Massless Dummy OOP Ledger	652	5	RECT	0.250	1.000				101	

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Table D.2-7: ANSR modelled concrete frame strengths

Strength Description	Strength ID	Compression Strength (kN)	Tension Strength (kN)	+ve Moment Strength (kN-m) [1]	-ve Moment Strength (kN-m) [1]	Tie Size (mm)	Tie Spacing (mm)	No. Legs Parrallel to X	No. Legs Parrallel to Z	Ties Adequately Tied into Core	Inadequate Development or Splice	Reinforcing Flexural Mat'l	Reinforcing Shear Mat'l
Entry Col - Internal	202	8000	2085	565	430	12	100	2	2	Yes	No	2	2
Entry Col - External	212	8000	798	203	160	10	200	2	2	Yes	No	2	2
Entry pier mid beam - Internal	303			480	480	12	300	2	2	Yes	No	2	2
Entry pier top beam	304			480	480	10	250	2	2	Yes	No	2	2
Entry Beams - wall side	305			480	480	12	200	2	2	No	Yes	2	2
Entry pier mid beam - External	313			480	480	12	300	2	2	No	Yes	2	2
Entry Beams	315			480	480	12	200	2	2	Yes	Yes	2	2

[1] For column elements the moment strength are those imposed by X-direction and Z-direction shear, respectively

Table D.2-8: ANSR wall concrete boundary elements

Frame Element	Section ID	Material ID	Section Type	X Dimension (Beam Depth Below)	Z Dimension (Beam Width)	Beam Depth Above	Flange Thk. T_f	Web Thk. T_w	Component Type	Strain Hardening Ratio
Full BE L1-2	610	11	RECT	0.500	0.400				131	0.010
Full BE L2-5	611	10	RECT	0.500	0.400				131	0.010
Capped BE L1-2	612	11	RECT	0.400	0.200				131	0.010
Capped BE L2-5	613	10	RECT	0.400	0.200				131	0.010

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Table D.2-9: ANSR wall concrete boundary element strengths

Strength Description	Strength ID	Compression Strength (kN)	Tension Strength (kN)	+ve Moment Strength (kN-m) [1]	-ve Moment Strength (kN-m) [1]	Tie Size (mm)	Tie Spacing (mm)	No. Legs Parrallel to X	No. Legs Parrallel to Z	Ties Adequately Tied into Core	Inadequate Development or Splice	Reinforcing Flexural Mat'l	Reinforcing Shear Mat'l
Full BE L1-2	610	7700	150	525	415	10	100	2	2	Yes	No	2	2
Full BE L2-5	611	5500	150	415	330	10	100	2	2	Yes	No	2	2
Capped BE L1-2	612	3400	500	200	100	10	100	2	2	Yes	No	2	2
Capped BE L2-5	613	2550	500	175	80	10	100	2	2	Yes	No	2	2
Dummy BE mid node	1000	1000000	1000000	1000000	1000000	10	100	2	2	Yes	No	2	2

[1] For column elements the moment strength are those imposed by X-direction and Z-direction shear, respectively

Table D.2-10: ANSR modelled steel beam elements

Frame Element	Section ID	Material ID	Section Type	X Dimension (Beam Depth Below)	Z Dimension (Beam Width)	Beam Depth Above	Flange Thk. T_f	Web Thk. T_w	Component Type	Strain Hardening Ratio
Stage Beam	411	3	OTHER	0.403	0.178		0.011	0.008	155	0.030
Stage Beam	412	3	OTHER	0.304	0.165		0.010	0.006	155	0.030
Stage Beam	413	3	OTHER	0.252	0.146		0.009	0.006	155	0.030
Roof Glulam Support	414	3	OTHER	0.308	0.305		0.015	0.010	155	0.030
Roof Truss	502	3	I-SECT	3.500	0.150		0.038	0.003	155	0.030
Truss chord support	504	3	OTHER	0.150	0.075		0.010	0.006	155	0.030

D.2.7.2.3 Modelled Steel Beam Elements

All steel beams are modelled as elastic frame elements in the ANSR model, this includes the steel beams supporting the stage and the roof trusses. Refer to Table D.2-10 for the details of the modelling parameters used for these members. The assessment of these of these elements is based on the outputs from the MRSA model. They are included in this model to ensure their mass and the supported structure has been adequately incorporated. The roof truss element is captured as an effective I-section with approximately the same weight and stiffness.

D.2.7.3 Beam-Column Joints

As a model default condition, the rigid end zones are set to 0.5 for the beam and column elements. This is over ridden is at the frame extensions where the beam element becomes a deep spandrel between column pairs and the column rigid end zone is set to 1.0. Figure D.2-17 shows typical rigid end zone configurations.

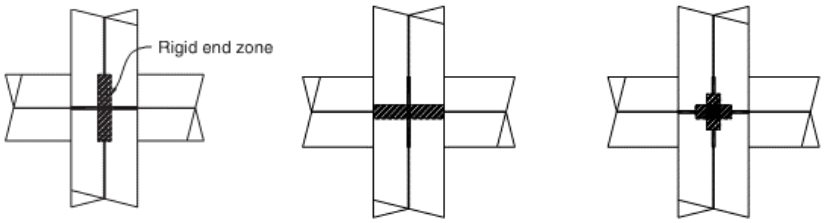

Figure D.2-17: Rigid end zones as presented in ASCE41

D.2.7.4 Diaphragm Modelling

Diaphragms in the superstructure are modelled as panel elements, not with rigid connectivity. The geometry and complexity of each diaphragm is discussed in Section D.2.1.

One rigid diaphragm exists at the Level 1 slab-on-grade to enforce uniform displacement between the tops of piles and the analytical dummy elements which represents the foundation lateral stiffness. Figure D.2-18 illustrated the connectivity of the Level 1 diaphragm. The nodes restraining the dummy lateral soil stiffness elements (refer Section D.2.4) are released from the diaphragm connectivity, as highlighted in the figure.

Table D.2-11 shows the panel properties for the superstructure diaphragms. Also shown in the table are the panel elements used to model the tapered beam elements at Levels 1/2 and 5. The thickness for mass and weight is used to apply superimposed floor loads, as outlined in Appendix C Section C.2.7. Where precast flooring is used, only the topping is modelled as the structural thickness and the precast weight is added as mass.

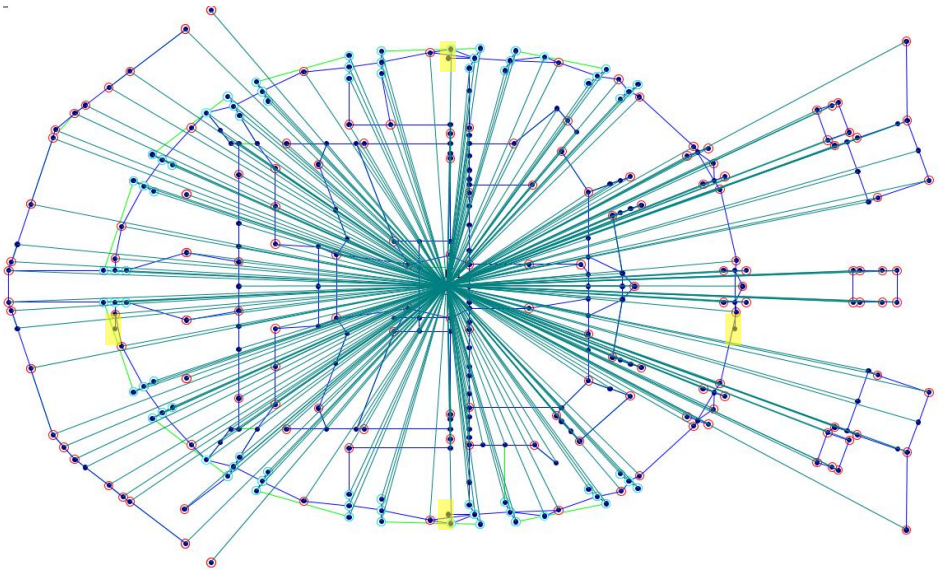

Figure D.2-18: Level 1 rigid diaphragm connectivity

Table D.2-11: ANSR modelled diaphragm panel elements

Location	Plate ID	Material ID	Structural Thickness (m)	Thickness for Weight/Mass (m)	Reinforced Sections			Component Type	Shear Strength vc	Reinf. Strength vs
					Material ID	Vert. ρv	Horiz. ρh			
Level 1/2 Lower	41	10	0.200	0.200	2	0.0057	0.0057	50	1017	1850
L1/2 Beams	51	10	0.150	0.150	2	0.0075	0.0075	50	1017	2441
L1/2 Beams	52	10	0.300	0.300	2	0.0021	0.0121	50	1017	3909
L1/2 Upper Slab	53	10	0.050	0.241	2	0.0036	0.0036	50	1017	1172
L2 200 Slab (stage)	60	10	0.200	0.340	2	0.0023	0.0023	50	1017	732
L2 125 Slab	61	10	0.125	0.241	2	0.0030	0.0030	50	1017	976
L2 200 Slab	62	10	0.200	0.316	2	0.0023	0.0023	50	1017	732
L2 225 Slab (office)	63	10	0.225	0.263	2	0.0051	0.0051	50	1017	1654
L2 L4 400 Slab (office)	64	10	0.400	0.438	2	0.0045	0.0045	50	1017	1453
L2 tees	65	11	0.050	0.325	2	0.0050	0.0050	50	1203	1620
L3 L4 225 Slab (office)	66	10	0.225	0.263	2	0.0060	0.0060	50	1017	1930
L4 tees	67	11	0.050	0.275	2	0.0050	0.0050	50	1203	1620
L4 Seating	68	10	0.100	0.216	2	0.0057	0.0057	50	1017	1831
L5 Slab	69	10	0.150	0.188	2	0.0054	0.0054	50	1017	1737
L5Beams	70	10	0.400	0.400	2	0.0019	0.0079	50	1017	2543
Roof	71	10	0.050	0.174	2	0.0050	0.0050	50	1017	1620
Ground	81	1	0.100	0.100	0	0.0000	0.0000	50	910	0
Dummy Timber Diaphragm	90	12	0.020	0.458	0	0.0000	0.0000	1	100	0
Dummy Concrete Diaphragm	91	5	0.100	0.100	2	0.0050	0.0050	50	1017	1620

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D.3 SEISMIC INPUT AND RESPONSE

D.3.1 Target Spectra, Ground Motion Selection and Scaling

Generally, the ground motion selection and scaling procedure is per ASCE41-17 which modifies ASCE7-16. The structural performance factor (S_p) equal to 1.0 has been adopted for the NLTHA in accordance with Engineering Assessment Guidelines Section C2C.6 for a mean response.

Target spectrum

The NZS 1170.5:2004 IL3 hazard spectrum is adopted for record scaling. This reflects a 1000-year return period for 100% ULS target spectrum. If required, a linear scale factor is applied to produce a lower %NBS magnitude run.

Selection of ground motions

Ground motions have been taken from (Bradley & Tarbali, 2017), a technical report prepared for Holmes which provides ground motions for NLTHA in New Zealand. The resulting motions for the 1000-year return period are provided in Table D.3-2.

Vertical accelerations are generally not included in NLTHA and are not required for the purpose of this analysis. Parts sensitive to vertical excitation are evaluated in Appendix G.

Scaling Procedure

Amplitude scaling is done in accordance with the procedure in ASCE 7-16 (ASCE/SEI, 2017), as modified by ASCE 41-17 (ASCE/SEI, 2017) which affects the period range for scaling.

11 records pairs are amplitude scaled so that the average spectra is above 90% of the target spectra across the period range of interest. This period range is defined as 0.2 T_{min} to 1.5 T_{max}, except that the upper bound period must be taken as at least 1 second. The periods T_{min} and T_{max} are the smallest and largest first-mode periods for the two principal horizontal directions of response, respectively. Table D.3-2 and Figure D.3-1 summarise the scaling of the suite.

Table D.3-1: Summary of Ground Motion Scaling Ranges

Summary of Ground Motion Scaling Ranges		
Limit State	T_{min}	T_{max}
ULS	0.08s[1] Use 0.10	1.00s

[1] Lower bound period below the corner period can cause conservative scaling results; this has been revised to the corner period (0.10s).

The scaled ground motion ensembles are provided in Table D.3-2 and Figure D.3-1. The 1.08 amplification factor required for RotD100 was applied after the scaling procedure and is included in the scale factor presented in the final column.

Table D.3-2: NLTHA Ground Motion Suite and Scaling Factors

				Averages:	Mw =7.66	139.8 km	257 m/s	32.6 sec	1.98		2.14
Record	PEER RSN	Earthquake Name	Station Name	Year	Magnitude, M_w	Distance, R_{rup} (km)	Vs30 (m/s)	Signification Duration $D_{5-95\%}$	Scale Factor	Primary Component	RotD Scale Factor
1	180	Imperial Valley-06	El Centro Array #5	1979	6.53	1.8	206	9.6	1.76	1	1.90
2	3754	Landers	Indio - Jackson Road	1992	7.28	48.8	292	36	2.18	1	2.36
3	4853	Chuetsu-oki	Joetsu City	2007	6.8	25.7	295	28.2	1.96	1	2.11
4	721	Superstition Hills-02	El Centro Imp. Co. Cent	1987	6.54	18.2	192	35.7	2.69	1	2.91
5	723	Superstition Hills-02	Parachute Test Site	1987	6.54	1.0	349	11	1.50	1	1.62
6	179	Imperial Valley-06	El Centro Array #4	1979	6.53	4.9	209	10.3	1.93	1	2.09
7	s_6	Chile	LACH	2010	8.8	339.1	-	38	2.00	1	2.16
8	s_4	Chile	ANTU	2010	8.8	323.0	-	39	2.70	1	2.91
9	s_8	Chile	SJCH	2010	8.8	332.7	-	39	1.38	1	1.49
10	s_9	Chile	STL	2010	8.8	334.1	-	38	2.46	1	2.65
11	s_5	Chile	CCSP	2010	8.8	109.1	-	74	1.22	1	1.32

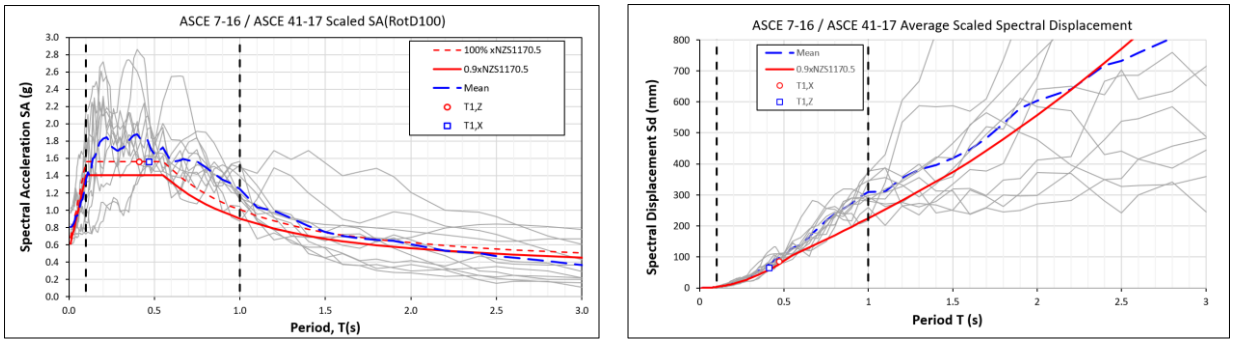

Figure D.3-1: Scaled records for ULS 1000-year return period

Application of Records

The Michael Fowler Centre is a near-fault site, located less than 2km from the Wellington Fault. Therefore, all records, except for subduction zone records, are orientated with their fault normal/fault parallel (FN/FP) components rotated to be in alignment with the Wellington Fault. Subduction records are applied in a random orientation in accordance with general guidance and interpretation of ASCE7-16 by Bradley Seismic. See Figure D.3-2 for a diagram on orientating the site in relation to the Wellington Faultline. A total 75-degree counter-clockwise rotation is applied to non-subduction records.

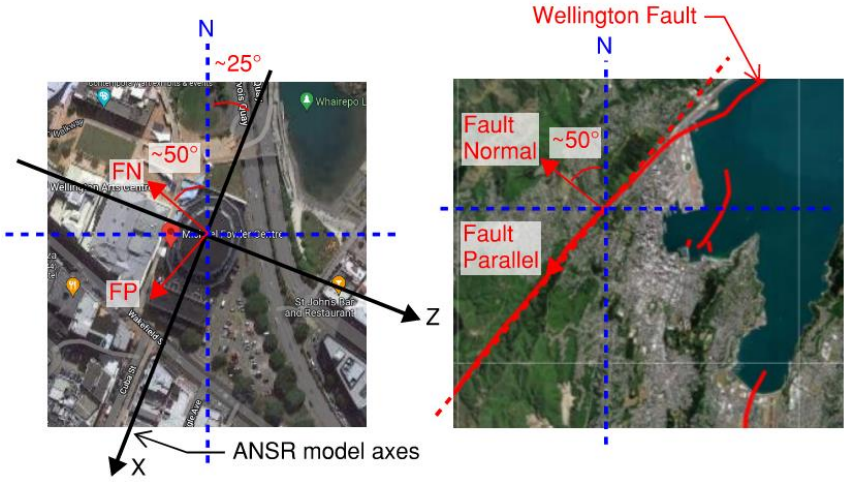

Figure D.3-2: Record orientation to FN/FP

D.3.2 NLTHA Summary of Performance

A summary of key performance metrics is given below in Table D.3-3. A comparison to the MRSA value is presented. Further detail and information is provided in Section D.3.3.

Table D.3-3: Summary of Key Global Performance Metrics

Hazard Level		100% ULS_UB	MRSA Value
Base-shear coefficient [g]:	(Z, X)	0.81, 0.90	1.56
Base-shear [MN]:	(Z, X)	86.7, 95.8	146, 146
Max inter-storey drift [%]:	(Z, X)	1.01%, 0.81%	0.81%, 0.31%
Max Lvl 4 - Roof Drift [%]:	(Z, X)	0.28%, 0.26%	0.56%, 0.35%
Max Floor Acceleration [g]:	(Z, X)	1.44, 1.15	2.11, 1.92
Max Roof Acceleration [g]:	(Z, X)	1.47, 1.44	2.90, 2.40

D.3.3 Building Global Displacements, Drifts and Accelerations

11 out of 11 records converged in the 100% ULS suite. The centre of mass suite, with no accidental mass eccentricity, was run to determine diaphragm accelerations.

Graphs are provided illustrating the maximum storey drift around the ring of the cantilever wall system and the maximum floor accelerations, refer Figure D.3-3. See Figure D.3-4 which highlights the five column lines used for reporting on the primary lateral load resisting system and Figure D.4-1 for all recorded floor nodes. The average response is computed by taking the maximum and minimum of each converged record and averaging all records. The values plotted are the absolute maximum between the positive and negative directions.

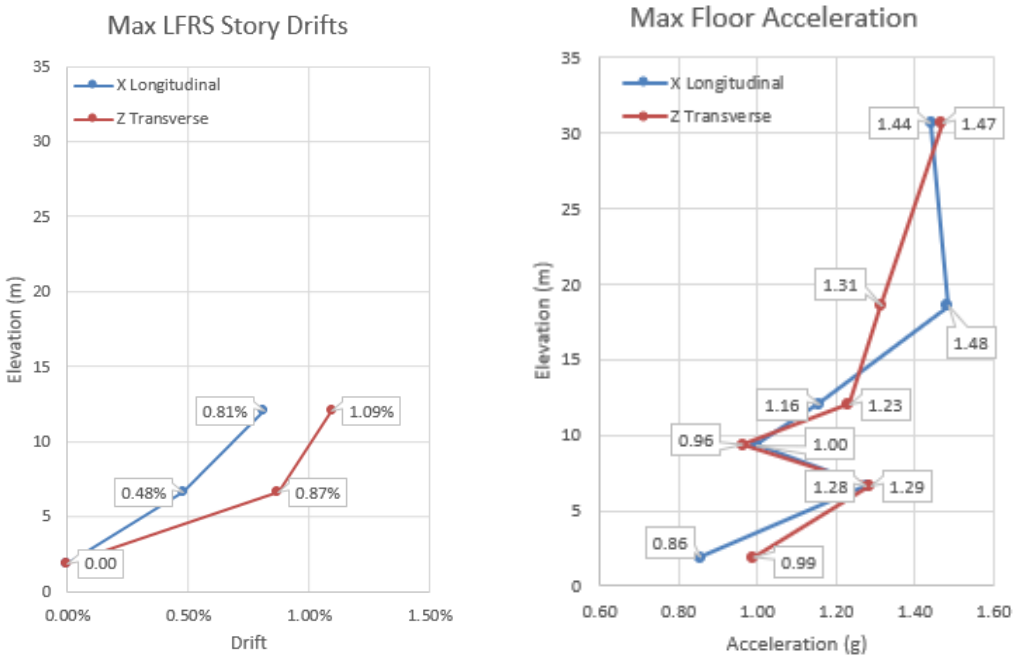

Figure D.3-3: Auditorium ANSR LLRS storey drifts (left) and floor accelerations (right)

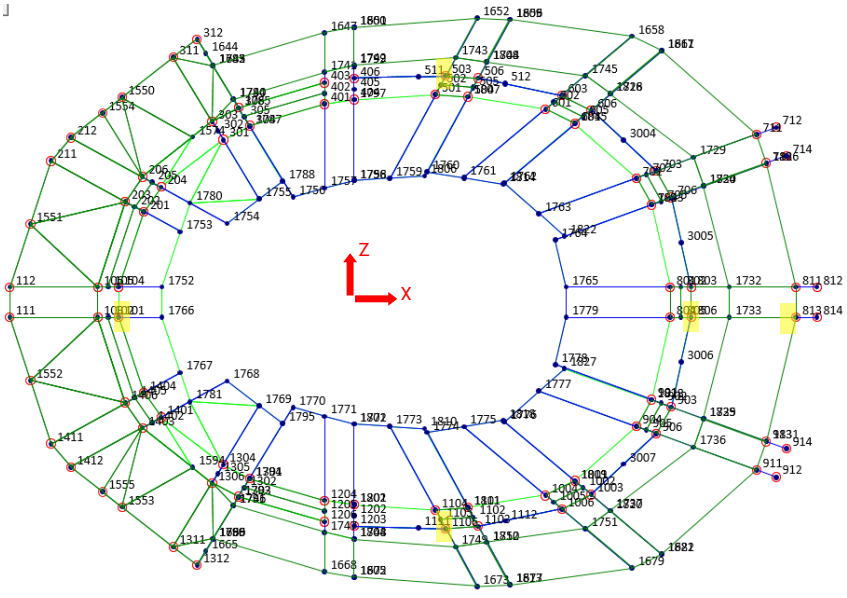

Figure D.3-4: ANSR lateral system global response recording locations

D.4 EVALUATION

D.4.1 Average Diaphragm Acceleration

Average diaphragm accelerations are computed for each diaphragm and are summarised in Table D.4-1. This acceleration then informs the diaphragm analyses of Appendix C Section C.4.2. Acceleration information is recorded for only the requested node locations highlighted in Figure D.4-1.

The modelled connectivity of the upper precast infill panels (those that infill the smaller dimension between the primary perimeter wall buttresses) was found to overly constrain the behaviour of the primary lateral system, relative to what the panels and their connections would be able to contribute. A separate model removing those panels and revising other node connectivity issues generally produced higher average floor accelerations. After making adjustments for a small number of non-converged records, the second set of accelerations noted in Table D.4-1 was determined. The larger acceleration values are used for the inertial load distribution that is used to evaluate the diaphragms.

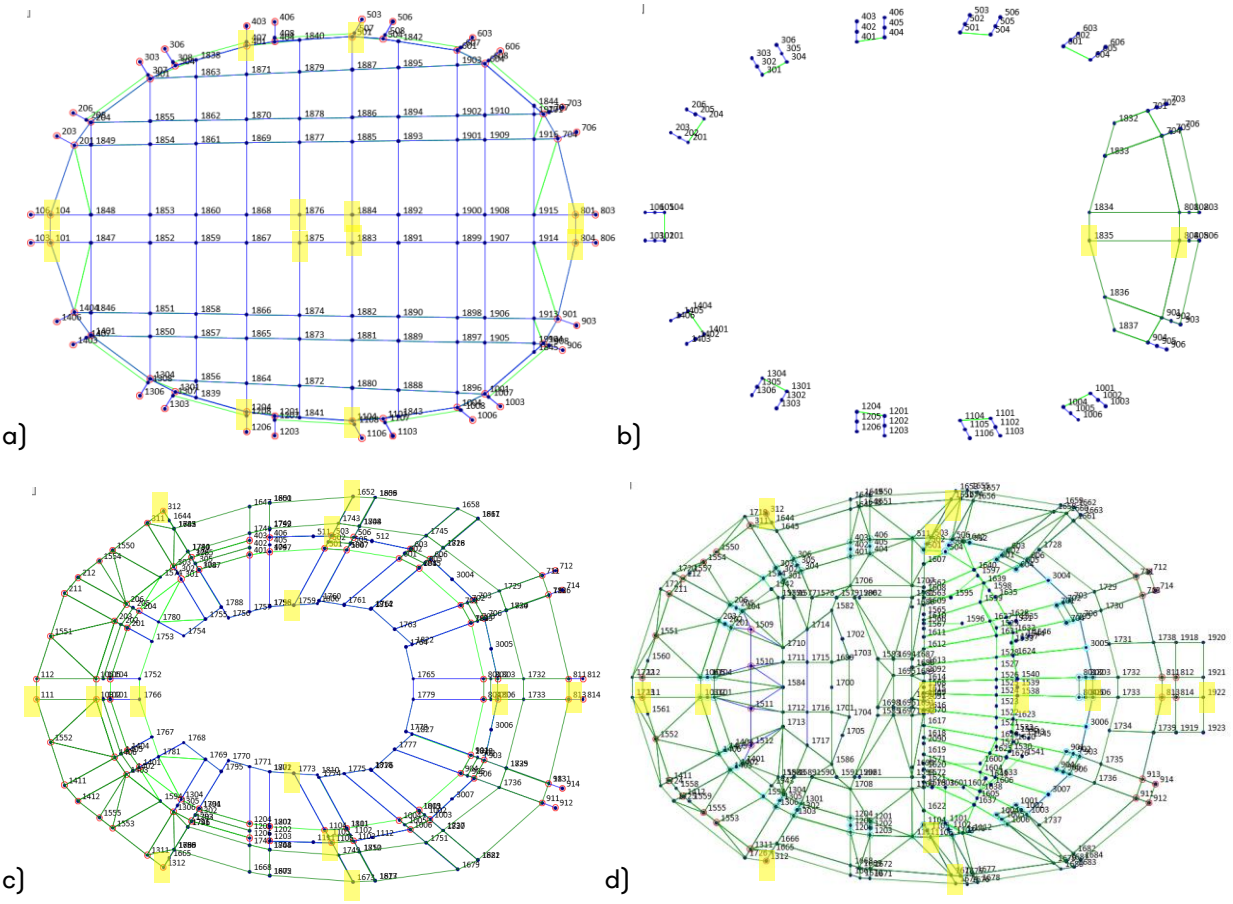

Figure D.4-1: ANSR diaphragm acceleration recording points a) roof, b) Level 5, c) Level 3 and 4, and d) Level 2 and 1/2

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Table D.4-1: ANSR Average Diaphragm Accelerations

Diaphragm Level	With Precast Infill Panels		Without Precast Infill Panels	
	Average Z Acceleration (g)	Average X Acceleration (g)	Average Z Acceleration (g)	Average X Acceleration (g)
Roof	1.44	1.42	1.75	1.75
Level 5	1.30	1.48	1.00	1.10
Level 4 (including sloped seating)	1.02	1.07	1.05	0.95
Level 3 (backstage area only)	0.95	0.98	1.05	1.00
Level 2	1.01	1.00	1.13	1.05
Level 1/2	0.91	1.00	1.00	1.00

D.4.2 Lateral Load Resisting System (LLRS) Behaviour

Performance of the primary lateral load resisting system is evaluated in the MRSA, refer to Appendix C for details. For comparison purposes, the maximum element behaviour is summarized for key elements.

Maximum ultimate shear strain for all elements of the primary cantilever wall systems is 0.006 which occurs at Level 2 at Bay 4 and 4a. This is below the 0.0075 shear limit determined in Appendix C Section C.4.1.1.2 and with the Engineering Assessment Guideline Equation C5.45.

Maximum gap opening in the Bay Walls also occurs at Bay 4 and 4a Level 1. A 0.016m opening over a 7m wall reflects a 0.0023 rad wall rotation; this is below the lower bound yield rotation of 0.0036 determined in Appendix C Section C.4.1.1.2. The Figures C.3-1 and C.3-2. in Section C.3.2 show a reduction in roof force per the ANSR "pESA" distribution. This demonstrates the reduction in wall flexural overturning demand relative to shear.

The columns of the frame extensions of Bays 6 through 6a show a high amount of combined axial and flexural demands in the time-history analysis, both under the discontinued Bay Walls and the isolated columns in the Lobby (see Figure D.4-2). However, the maximum drift in the longitudinal direction is 0.81% and in the transverse direction is 1.09% for the Lobby columns. In Appendix C Section C4.1.1.2 the calculated yield rotation of 0.49% and a total allowable inelastic drift, to maintain gravity support, is 2.24% for transverse motion of the frame. These isolated columns therefore do not control the seismic rating.

For the columns under the discontinued Bay Walls, these are rated at 60%NBS_{IL3} under the MRSA analysis. The high level of axial demand in these columns significantly limits the inelastic rotation capacity. This reduced level of performance is supported in the NLTHA by their exceedance of the Life Safety criteria under 100% ULS demands.

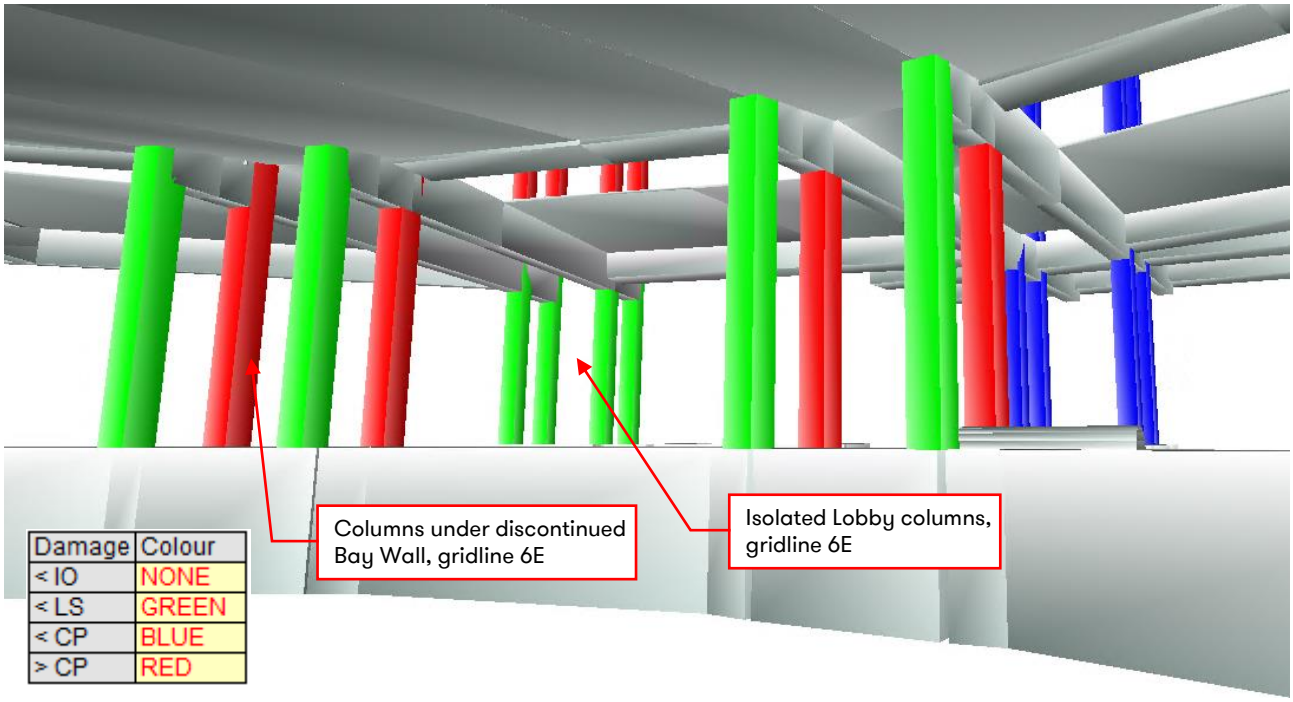

Figure D.4-2: Damage plots for 100% ULS runs showing columns in Lobby, view from Wall 21

Appendix E

Technical Summary of the Renouf Foyer Analysis and Evaluation

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Appendix E Technical Summary of the Renouf Foyer Analysis and Evaluation

E.1 INTRODUCTION

The Simple Lateral Mechanism Analysis (SLaMA) method was used for the complete assessment of the primary structure of the Renouf Foyer to determine the seismic score. This method involves identifying the likely elastic and inelastic deformation mechanisms, and the lateral strength and displacement capacity of these mechanisms.

SLaMA is an appropriate form of assessment for the Renouf Foyer, as the structure consists of only two suspended levels and has several similar frames of regular layout. This indicates the overall seismic response is dominated by the fundamental vibration modes in each principal direction and higher modes are not critical. The simple layout of the frames meant that any possible inelastic mechanisms could be readily identified and assessed in accordance with the Engineering Assessment Guidelines.

The SLaMA method was supplemented by a linear elastic analysis of a 3D model in Microstran V10. This was used to better capture effects not easily determined using the SLaMA method such as:

- Foundation flexibility.
- Displacement profile & capacity at first yield.
- Estimate of the initial period of the structure for force-based assessments of the Level 2 diaphragm and secondary structural elements.

Once the SLaMA has been completed, possible inelastic mechanisms of the Renouf Foyer structure were identified considering possible element overstrength, which could influence the behaviour on a local and global level.

The 3D linear elastic model was then used to assess both the probable base shear capacity and displacement profile at yield for each respective frame type. Seismic loads were applied considering an equivalent static force distribution, per Section 6.2, NZS1170.5:2004 with element demands compared to the probable capacities determined during the SLaMA process. The elastic behaviour of the system up to yield could then be combined with the inelastic displacements associated with each possible mechanism to form bilinear pushover (force-displacement) curves.

The behaviour of the structure can be represented in terms of the spectral acceleration and spectral displacement of an equivalent single degree of freedom (SDoF) system. The equivalent SDoF force-displacement response, in both principal directions of the structure, is plotted on a damped horizontal acceleration-displacement-response spectrum (ADRS). This spectrum gives a representation of the ULS seismic demands in the ordinates of spectral acceleration and spectral displacement; As such, the force-displacement response can be directly compared to the damped ADRS to determine the seismic score.

E.1.1 Simplified Lateral Mechanism Analysis (SLaMA)

Primary structural elements and most secondary structural elements are considered in the SLaMA analysis. This includes all reinforced concrete beams, columns and joints that form the primary frames of the Renouf Foyer.

The SLaMA method was used to develop bi-linear force-displacement curves for the lateral load resisting system in each principal direction. This was used to assess the capacity of the primary superstructure following the displacement-based assessment procedure, as described in Section C2.4 of the Engineering Assessment Guidelines.

Secondary structural elements which are included in the scope of assessment, but whose mechanisms are not directly included in the SLaMA, use the deformation demands from the SLaMA. Acceleration sensitive

secondary structural elements are assessed separately, using the provisions for 'Parts and Components' from Chapter 8 of NZS1170.5:2004. Evaluation of secondary structural elements is summarised in Section E.8.5 of this appendix, which includes confirmation/clarification of the above approach for each element type. Evaluation of all non-structural elements is summarised in Appendix G.

For the Renouf Foyer, the SLaMA has considered the following step-by-step approach in both principal directions.

1. Beam/Column Elements

    - Assess detailing of beam/column elements and identify the use of plain bars, insufficient anchorage, etc. if applicable.
    - Determine the probable flexural capacity
    - Determine the probable shear demand, and the shear demand associated with the development of the flexural capacity of the member.
    - Determine the probable shear capacity
    - Determine if flexure or shear governs the members strength hierarchy.

2. ESA to establish storey shear demand.

    - Apply seismic loading using an equivalent static load distribution considering elastic (μ=1) ULS demands, to assess the displacement and base shear capacity at yield for each sub-system.
    - Compare relative storey shear demands to probable element capacities to identify levels at which an inelastic mechanism might form.

3. Evaluate the Strength Hierarchy of sub-systems

    - Evaluate the possible mechanisms for each sub-assembly considering element overstrength.
    - Evaluate the possibility of beam-column joint-shear in both directions due to overstrength actions from the critical yielding elements.
        - This check depended on the anticipated joint behaviour, with joint shear checks per C5.6.2.2, punching shear checks (per Section 12.7 of NZS3101:2006) and strut & tie analysis of the 'macro' coupled column system used to identify possible beam-column joint-shear before or during formation of plastic hinges surrounding the joints. The probable base shear could then be reduced if required.

4. Inelastic Displacement Capacity of Possible Mechanisms

    - Evaluate possible inelastic mechanisms identified during 2.)-3.) in accordance with C5.5.3 of Engineering Assessment Guidelines.
    - In combination with yield displacement and base shear capacity determined in 2.), determine the bilinear force-displacement response and effective SDoF properties associated with each possible mechanism.

5. Allowance for P-Delta and Torsion

    - Determine the centre of mass relative to the centre of strength and make allowance for torsion in accordance with Appendix C2F of the Engineering Assessment Guidelines.
    - Make allowances for P-Delta effects in accordance with Section C2.9.4 of the Engineering Assessment Guidelines.

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6. Assess Possible Seismic Pounding

   - Assess possible seismic pounding with any adjacent structures in accordance with Appendix C2B of the Engineering Assessment Guidelines.

7. Assess the seismic score of the primary structure in both principal directions

   - Assess the primary structure in accordance with the displacement-based assessment (DBA) procedure, as outlined in the Section C2.4.2 of the Engineering Assessment Guidelines.

Not included within SLaMA:

- Foundation beam capacity – Preliminary calculations indicated that the foundation beams would likely remain elastic, so they were not included within the superstructure strength hierarchy. Elastic foundation beam deflections were still considered as part of the analysis.
- The basement structure was not explicitly modelled, a conservative estimate of the restraint provided to the columns framing into the basement structure was instead used in the analysis.
- The lightweight steel roof structure over the Kitchen Extension was not included within the SLaMA analysis as it was not considered to impact the overall performance of the primary superstructure.

E.2 ANALYSIS INPUT

E.2.1 Building Description

The Renouf Foyer is a two-storey, rectangular structure located on the Southern side of the Michael Fowler Centre. The structure primarily consists of an open plan, double-height space suspended above ground floor level (Level 1). The structure was designed and constructed in the 1980's along with the Auditorium. The Renouf Foyer was extended approximately 10 years later, adding an additional suspended floor area over the existing basement structure on the Southern side, known as the 'kitchen extension'. The Renouf Foyer has floor dimensions of approximately 45.0x18.0m, with the kitchen extension having floor dimensions of approximately 10.0x21.0 m.

The primary supporting structure of the Renouf Foyer consists of a series of precast coupled column units which form a frame with beams in the North-South direction at Level 2. Coupled column units are grouped into sets of four around the perimeter of the Renouf Foyer with beams spanning in the North-South direction between groups of coupled column units to support the pre-cast concrete flooring system. The primary beams are precast, using cast in-situ joints within a shell beam section at either end of the span. The kitchen extension is of similar construction, using pre-cast columns and extension beams with shell beam sections to emulate a cast-in-place system.

Gravity Structure

The suspended floor uses a double-tee precast flooring system in the Renouf Foyer that spans in the East-West direction between primary beams. The kitchen extension uses a precast concrete rib flooring system with timber infills (Rib & In-Fill). The precast flooring units are connected through a concrete topping and cast in-situ edge beams around the perimeter.

The double-height frames span above the suspended floor to support a series of steel roof trusses.

The structure has a floor-to-floor height of 4.62 m between Levels 1 & 2, with the double-height frames spanning another 8.1m to the centreline of the uppermost coupling beams which are aligned with the roof truss system.

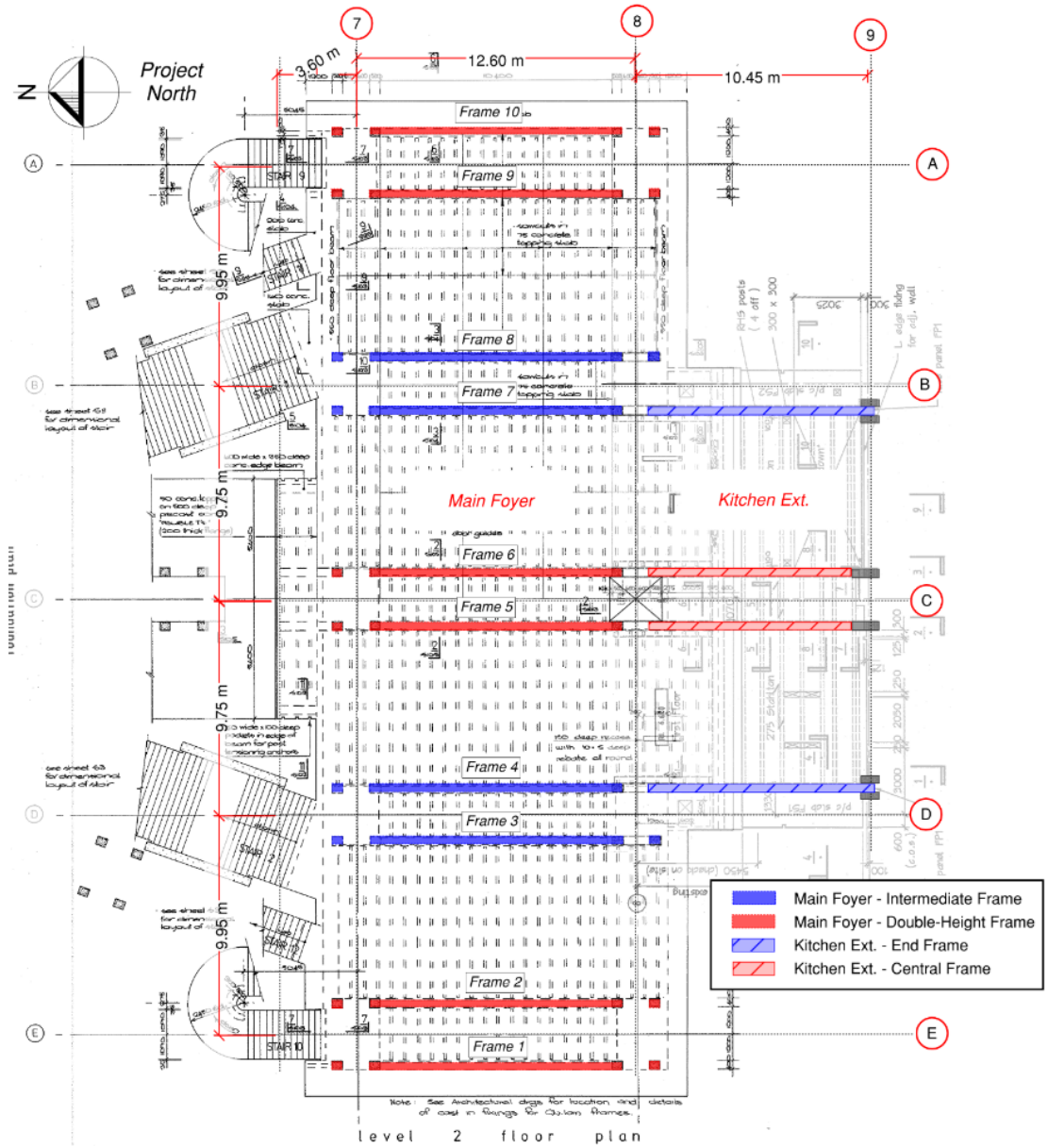

Figure E.2-1: Snapshot taken from drawing S102 of the 1980's Holmes, Wood, Poole and Johnstone drawing set - showing the plan layout of transverse frames in the Renouf Foyer

Lateral Structure

The lateral load-resisting system of the Renouf Foyer consists of a two-way concrete moment frame system. The precast coupled column units form single-bay moment frames spanning ~10.0m in the transverse direction. Either through:

- Double-height frames which support both the suspended concrete floor at Level 2 and the steel roof trusses. These frames consist of coupled columns at either end with a primary transverse gravity beam

spanning the width of the structure. There are six double-height frames in total – two frames at each end of the structure and two along the central gridline C.

- Intermediate frames which support only the suspended floor at level 2 and are located between the double-height frames. There are four intermediate frames in total, located at gridlines B and D.

In the longitudinal direction, adjacent sets of coupled columns are connected via concrete coupling beams to provide frame action within each precast coupled column unit. They are not connected to other precast concrete column units.

The kitchen extension frame consisted of two typical frame types.

- Central frames (at gridline C only), which consist of a fully fixed beam-column connection to the central kitchen extension columns. The central frame columns were connected onto the wall of the existing basement structure.
- End frames (at gridline B and D) which use a pinned beam-column connection. The end frame columns are supported by a single pile and tied back to the basement structure through foundation beams in the East-West direction.

The kitchen extension frames are doweled into the existing beams of the original Renouf Foyer with drilled and epoxied bars. A splint beam was also added to one side of the existing cantilever beams to resist the additional hogging moment demand from the kitchen extension frames.

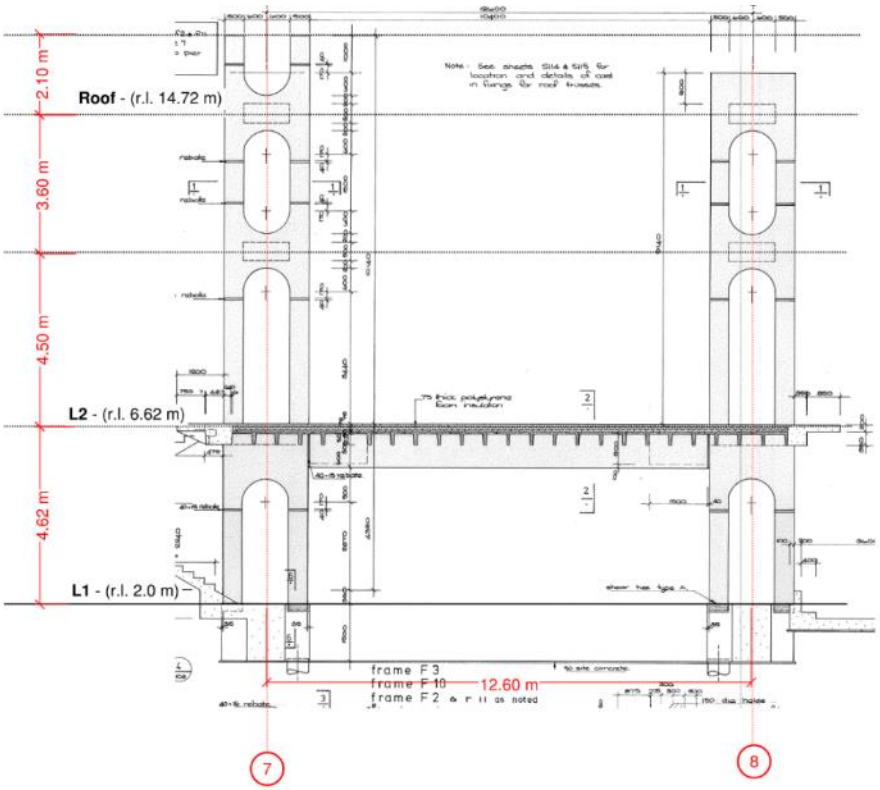

Figure E.2-2: Snapshot taken from drawing S104 of the 1980's Holmes, Wood, Poole and Johnstone drawing set - showing typical double-height transverse frames.

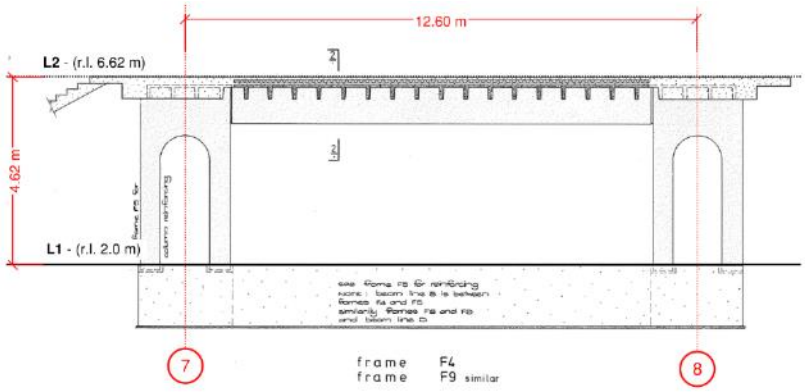

Figure E.2-3: Snapshot taken from drawing S104 of the 1980's Holmes, Wood, Poole and Johnstone drawing set - showing typical intermediate transverse frames.

E.2.2 Identification of potential Structural Weaknesses (SWs)

The following systems and elements were assessed as potential Structural Weaknesses, as exceedance of their capacity could lead to a life safety hazard.

- Renouf Foyer superstructure (ground floor level to Roof)
  - Flexural, shear or deformation capacity of beams or columns
  - Beam-column joint shears
- Kitchen Extension frames (ground floor level to Level 2)
  - Flexural, shear or deformation capacity of beams or columns
  - Beam-column joint shears
  - Connection between original & kitchen extension (see Section E.8.5).

No Severe Structural Weaknesses (SSWs) were identified.

Secondary structure considered:

- Renouf Foyer roof trusses
- Kitchen Extension frames
- Connection between new and existing structure

The impact of the adjacent structures was also considered in accordance with Appendix C2B of the Engineering Assessment Guidelines (see Section E.8.3).

E.2.3 Material Properties

Generally, strength/capacity reduction factors equal to 1.0 have been used. Material strengths used in the assessment are shown in Table E.2-1 and were based on values from the 1980's structural specification from Holmes, Wood, Poole and Johnstone. The concrete modulus of elasticity was determined in accordance with the Engineering Assessment Guidelines as, E_c = 30 GPa.

Reinforcing overstrength factors were also used in accordance with Table C5.4 of the Engineering Assessment Guidelines.

Table E.2-1: Material strengths used in assessment

Material	Strength on drawings	Probable strength used in assessment	Overstrength Factor, ϕ_0
Foyer columns and associated linking beams (excluding transverse gravity beams).	35 MPa	52.5 MPa	-
All other concrete	25 MPa	40 MPa	-
Reinforcing bars marked "H" on drawings (Grade 380 bars to NZS 3402 P:1973)	380 MPa	455 MPa	1.5
All other reinforcing bars (Grade 275 bars to NZS 3402 P:1973)	275 MPa	324 MPa	1.25
Hollow-sections used for foyer roof trusses (per BS 4360:1972)[1]	230 – 270 MPa	267 – 320 MPa	

[1] Range of steel grades considered based on referenced material standard.

E.3 ELEMENT CAPACITIES

The probable and overstrength capacity of elements were determined in accordance with Section C5.5 of the Engineering Assessment Guidelines. These values were used to establish the range of possible inelastic mechanisms that could form within each beam-column sub-assembly considering the range of possible element strengths.

For lateral systems involving frame action, the effects of axial load on columns must be considered to capture the true behaviour of the system. Steps taken to consider axial load in assessment are outlined below.

- Maximum axial loads were conservatively determined based on elastic ULS demands to assess the possible shear mechanism of the columns.
- Gravity axial loads were used to assess the strength hierarchy at each storey level. This was done assuming that the influence of axial load on the flexural strength remained essentially linear, such that there would be negligible impact on the sway index due to seismic beam shears.
- After assessing the average yield point of each frame type, the maximum axial load at flexural yield was extracted from the analysis model and used to calculate the critical inelastic deformation capacity of the columns.

The probable element mechanism was found considering the ratio of the shear demand from flexural overstrength and the probable shear capacity. The shear demand from flexural overstrength was calculated using the equation below as,

$$V_{M_p} = \frac{\phi_o\left(M_{prob,1} + M_{prob,2}\right)}{L_{clear}}$$

Where; ϕ_o = the element overstrength factor, per C5.4.3, $M_{prob,i}$ = the probable moment capacity at end 'i' of the clear span, and L_{clear} = the clear span length of the element. A flexural mechanism was said to develop when the ratio of shear demand from flexural overstrength to the probable shear capacity was < 1.0. Possible flexure-shear mechanisms were generally evaluated using the direct-rotation method.

Transverse direction

In the transverse direction, the analysis of beam elements was carried out considering the dimensions, reinforcement and element capacities as shown in Table E.3-1. Flexural capacities were taken within the

potential plastic hinges zones (PPHZ's) at either end of the span. The shear demand from flexural overstrength was used as part of the direct-rotation method to account for possible shear strength degradation due to inelastic deformations in the beams. It was found that the undegraded shear strength of all beam elements was sufficient such as to avoid brittle shear before flexural hinging would occur.

Table E.3-1: Properties of beam elements in the transverse direction.

Element	Reinforcement [%]			Dimensions [mm]			M_{prob} [kNm]		V_{prob} [kN]	V_{Mp} [kN]	$\frac{V_{M_p}}{V_{prob}}$
	ρ_{top}	ρ_{bot}	ρ_v	L_{clear}	h_b	b_w	+ve	-ve	-	-	
Foyer Gravity Beam	0.64	0.27	0.28	10400	900	400	339	780	611	161	**0.26**
Kitchen Ext. Central Beam (Gridline 8)[2]	0.58 (1.71)[1]	0.14 (1.28)[1]	0.28	8800	1100	400	757	1245	937	229	**0.24**
Kitchen Ext. Central Beam (Gridline 9)[2]	0.46	0.46	0.28	8800	1100	400	521	584	937	301	**0.32**
Link slab	0.41	0.41	1.17	1200	900	400	508	508	1615	1270	**0.79**

[1]*Values in () include additional 8-D28 bars in the splint beam.*
[2]*Element capacities shown for beam sections adjacent to each B/C joint with the dowel connection assessed separately (see Section E.8.5.1).*

Column elements have been considered in a similar way to the beams. The columns are all precast with no lap splices in the PPHZ's. There is no curtailment of reinforcing in the couples column units up the height of the structure. The central kitchen extension frames are considered part of the lateral load-resisting system and have asymmetric reinforcing and curtailment of bars. The moment capacity for each end and load direction are included in Table E.3-2.

The flexural capacity was calculated for two load cases:

1. Columns under seismic weight only, $N_{G + \psi_E Q}$.
2. Maximum axial load from the analysis model under 100%ULS demands, $N_{100\%}$ ($G + \psi_E Q + E_u$ load combination, per NZS1170.0:2002).

The latter value was used to calculate the shear demand from flexural overstrength to check the possibility of a shear mechanism (shown as V_{Mp}).

Table E.3-2: Properties of column elements in the transverse direction.

Element		Reinforcement [%]		Dimensions [mm]			Axial Load [kN]		M_{prob} [kNm]	V_{prob} [kN]	V_{Mp}[1] [kN]	$\frac{V_{M_p}}{V_{prob}}$
-	Frame Type	ρ_l	ρ_v	h_{clear}	h_c	b_c	N_G	$N_{100\%}$	$M(N_G)$	-	-	-
External column at base	Double-Height	2.43	1.07	2900	500	400	135	1400	405	809	605	**0.75**
	Intermediate						-20		377			
	Double-Height	1.74					426	1330	386	837	535	**0.64**

Element		Reinforcement [%]		Dimensions [mm]			Axial Load [kN]		M_{prob} [kNm]	V_{prob} [kN]	V_{Mp} [1] [kN]	$\frac{V_{M_p}}{V_{prob}}$
Internal column at base	Intermediate						460		392			
External column		2.43		3700			117	810	402	809	418	**0.52**
Internal column		1.74					118	1330	330	837	419	**0.5**
Kitchen Ext. Central Columns at Level 2 (S)		1.14	0.81	3500	1150	400	150	660	1500	2070	1124	**0.56**
Kitchen Ext. Central Columns at Level 2 (N)		2.56			600	400	150	660	880	2070	1124	**0.56**
Kitchen Ext. Central Column at base		2.56			600	400	200	700	475	2070	1124	**0.56**

[1]*Shear demand from flexural overstrength based on maximum compressive axial load under 100%ULS elastic loads.*

Longitudinal direction

Beam elements considered in the longitudinal direction consist only of the link slab elements between the coupled columns between Level 2 and roof level. Due to the possible inelastic mechanisms later identified, the capacity of the link slab elements was considered in two possible conditions:

- The initial condition of the link slab elements, where the full concrete section could contribute to the strength hierarchy and possible mechanism of the elements themselves.
- The residual condition of the link elements after undergoing inelastic rotations, where it was expected the unconfined core sections would fail before the edge elements, which were confined with transverse reinforcing.

It was assumed that exceedance of the capacity of the unconfined core section would not pose a life safety risk because:

- The link slabs above Level 2 resist no gravity loads other than self-weight.
- The confined edge elements should restrain the unconfined core such that they do not pose a falling hazard.

This allowed for the capacity of the whole structure to not be limited by the unconfined core sections of the link elements, whilst still considering the initial condition of the structure when identifying the strength hierarchy. The analysis of the longitudinal beam elements was carried out considering the dimensions, reinforcement and element capacities as shown in Table E.3-3. It was found that only the upper link slabs within the central frame (reduced clear span) could possibly fail in shear considering the shear demand associated with flexural overstrength. Possible shear-flexure was considered using the direct-rotation method when determining inelastic rotation capacities.

Table E.3-3: Properties of beam elements in the longitudinal direction.

Element	Reinforcement [%]			Dimensions [mm]			M_{prob} [kNm]		V_{prob} [kN]	V_{Mp} [kN]	$\frac{V_{M_p}}{V_{prob}}$
-	ρ_{top}	ρ_{bot}	ρ_v	L_{clear}	h_b	-	+ve	-ve	-	-	-
Link Slab at Level 2 (Initial)	0.41	0.41	0.27	2400	550	2000	859	859	2057	1289	**0.63**

Element	Reinforcement [%]			Dimensions [mm]			M_{prob} [kNm]		V_{prob} [kN]	V_{Mp} [kN]	$\frac{V_{M_p}}{V_{prob}}$
Link Slab at Level 2 (Confined Edge)	0.93	0.93	0.89	2400		300	285	285	616	428	**0.69**
Upper Link Slab (Initial)	0.75	0.75	0.17	2400/ 2000	500	1200	753	753	965	941/1130	**0.98/1.18**
Upper Link Slab (Confined Edge)	1.03	1.03	0.34	2400/ 2000		300	254	254	324	318/381	**0.98/1.18**
Slab Edge Beams at Level 2[1]	0.57	0.57	0.25	6950	550	450	194	194	394	84	**0.21**

[1]*Average reinforcing details were assumed for in-situ edge beams at Level 2.*

The analysis of column elements was carried out as per the transverse direction, considering the dimensions, reinforcement and element capacities as shown in Table E.3-4 below.

Table E.3-4: Properties of column elements in the longitudinal direction.

Element		Reinforcement [%]		Dimensions [mm]			Axial Load [kN]		M_{prob} [kNm]	V_{prob} [kN]	V_{Mp} (1) [kN]	$\frac{V_{M_p}}{V_{prob}}$
-	Frame	ρ_l	ρ_v	h_{clear}	h_c	b_c	N_G	$N_{100\%}$	M (N_G)	-	-	-
External column below Level 2	Double-Height	2.43	0.86	4070	400	500	135	2500	311	615	381	**0.62**
	Intermediate						-20		290			
Internal column below Level 2	Double-Height	1.74					426	2500	282	618	351	**0.57**
	Intermediate						460		287			
External column		2.43		3700			117	2500	309	615	419	**0.68**
Internal column		1.74					118		240	618	386	**0.63**
Kitchen extension central columns at base		2.76	0.54	3500	400	600	150	200	309	517	265	**0.51**
Kitchen extension end columns at base		2.06	0.94	3500	300	800	150	200	177	711	152	**0.21**

E.4 3D ANALYSIS MODEL

E.4.1 General Description & Geometry

To better capture the three-dimensional behaviour of the structure, it was modelled in Microstran V10. This was used to better capture effects not easily determined using the SLaMA method such as:

- Substructure Behaviour
- Displacement profile & capacity at first yield.
- Estimate of the initial period of the structure for force-based assessments of Level 2 diaphragm and secondary structural elements.

Due to the unequal clear span lengths of the frames and the varying substructure layout, the force distribution within the frames at yield is irregular. Because of this, the moment-curvature method was not used to predict the yield profile of the structure. The 3D elastic model was therefore used in conjunction with an equivalent static force distribution to predict the yield displacement and base shear capacity of each sub-system. The 3D model for the Renouf Foyer is shown in Figure E.4-1.

The stitch joint between the existing and kitchen extension frames was estimated to yield under only nominal seismic demands so was therefore modelled as a pin connection when considering the performance of the primary frames. The displacement capacity of the stitch joint was assessed separately as discussed in Section E.8.5.1.

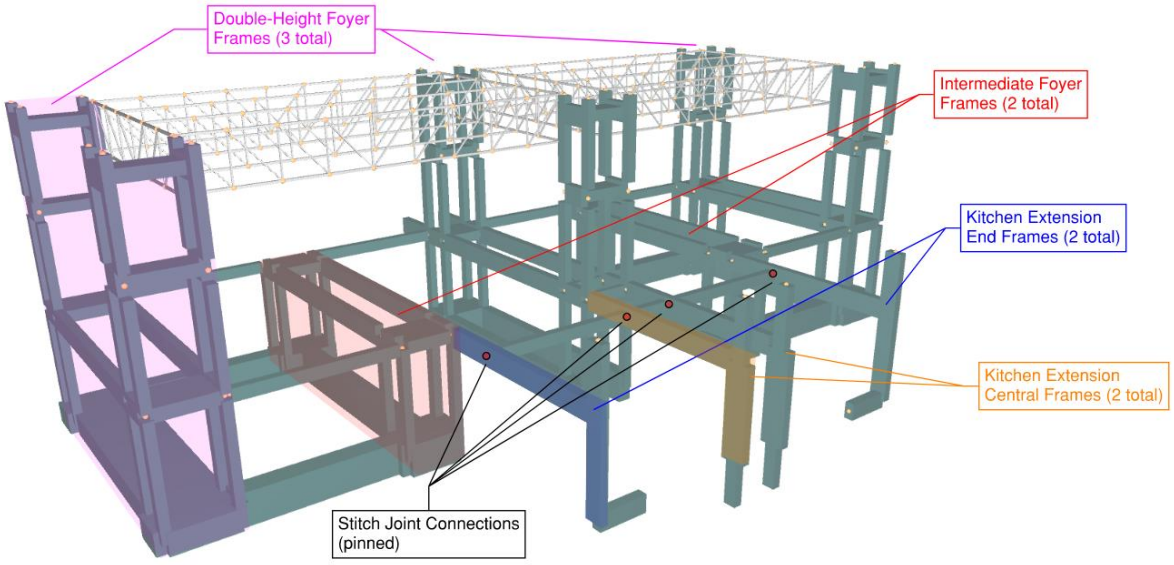

Figure E.4-1: Rendered view of Microstran V10 model of the Renouf Foyer.

The primary superstructure was explicitly modelled using frame elements with the material properties as listed in Section E.2.3. Effective section properties were used in accordance with NZS3101: 2006 Table C6.6, considering Grade 380 bars ($f_{y,prob}$ = 455 MPa) that were typically used for longitudinal reinforcement. The effective section properties are shown in Table E.4-1.

Table E.4-1: Effective Section Properties used in assessment

Element	Effective Properties
Concrete Frame Beams	$0.32I_g$
Concrete Columns	$0.35\ I_g$
Concrete Foundation Pad Grillage	$0.25I_g$

E.4.2 Diaphragm Modelling

Diaphragms are stiff compared to the vertical elements of the lateral load resisting system and are therefore modelled as rigid in analysis. The diaphragm is assessed separately using a 'macro' strut and tie approach with the results summarised in Section E.8.6.

E.4.3 Foundation/Soil Interface Modelling

Vertical pile stiffnesses were not included in the superstructure assessment as the seismic response of the Renouf Foyer is expected to be structurally dominated. This was because:

- The large spans (~10.0 m) between coupled columns indicate vertical pile deformation will have an insignificant impact on the global deformations of the superstructure.
- Axial loads in the piles only occur due to beam shears from the foundation beams and the primary beams at Level 2. The seismic response in both directions is dominated by; 1.) The overturning moment (OTM) of the coupled column units, and 2.) the frame action between coupled column units. Neither of these mechanisms will be impacted by vertical pile stiffness or impart significant axial load into the piles.

This resulted in the 3D analysis model being laterally and vertically restrained (pinned) at the top of the piles beneath each column group pad. The foundation beam system was modelled to capture the additional elastic displacements due to pad rotations beneath each column group. The foundation beam system was not considered within the scope of the SLaMA and assumed to remain elastic for the purposes of this assessment.

The capacity of the Renouf Foyer foundation system is assessed considering the geotechnical report by Tonkin+Taylor Ltd. This is summarised in Section E.8.6.

Basement Substructure

The basement substructure under Southern end of the central double-height frames was not modelled explicitly due to the presence of panel elements that would complicate the analysis. A simplified approach was taken to assess the behaviour of the column group located above the basement where:

- Lateral and vertical restraint was assumed to be provided at the ground floor level by the basement roof slab and lift pit pile, respectively.
- This was modelled as a pin beneath each column, assuming no rotational base fixity, to provide a conservative estimate of the probable base shear capacity of the central double height frame.

E.4.4 Special Modelling Issues

Rigid Offsets

Rigid offsets were incorporated into the 3D model as per the recommendations of Section C5.5.1.7 of the Engineering Assessment Guidelines. Based on the element capacities above, the rigid offsets applied could be summarised in Table E.4-2 below. The Renouf Foyer acted as a two-way moment frame with beam elements of varying depth in each principal direction. Because of this, the column offsets were dependent on the direction of seismic response. This resulted in two variations of the model being developed to capture the stiffness of the structure in each principal direction.

Table E.4-2: Typical rigid offsets applied to the 3D analysis model of the Renouf Foyer.

Frame Type	Beam-column joint level	Joint Offsets	
		Transverse Direction	Longitudinal Direction
Double-Height Frame	Level 2	Both (Mixed)	Both (Mixed)
	Foyer Mid-Height	Column	Column
	Roof	Beam	Beam
Intermediate	Level 2	Both (Mixed)	Beam
Kitchen Extension	Level 2	Both (Mixed)	-

***Note**: The rigid offsets do not indicate a definitive strength hierarchy as typical element overstrength factors were greater than the modelling requirements as set out in C5.5.1.7.*

Roof Structure

The roof structure was modelled using elastic truss elements to provide a better estimate of the rotational fixity and gravity load path at roof level. Preliminary analyses showed that the concrete link slab elements at roof level provided most of the rotational fixity so potential inelastic behaviour within the roof trusses was not considered to impact the overall behaviour of the superstructure. The roof trusses were analysed as part of the secondary structure with the results summarised in Section E.8.5.

E.4.5 Seismic Floor Weight and Building Mass

The roof structure consists of plywood sheathing supported on timber roof battens and cold-formed steel purlins spanning between the longitudinal trusses. The roof trusses support both a suspended timber ceiling along their bottom chord and a plant room between the central longitudinal trusses.

The suspended floor of the Renouf Foyer was found to consist of:

- Precast double-tee flooring units between the primary transverse gravity beams with 125 mm of in-situ concrete (75mm topping, 50 mm screed).
- Cast in-situ cantilever slab portions around the perimeter (typically 200 mm thick).
- Precast concrete ribs with timber infill panels (rib & in-fill) spanning between primary beams in the kitchen extension.

Floor seismic weights were calculated by summing the self-weight of the floor slabs, superimposed dead load, and seismic live load. the floor weights are summarised in Table E.4-3 below.

Table E.4-3: Floor Seismic Weight Summary (kPa)

Level	Gravity Load	Superimposed Dead Load	Seismic Live Load	ψ_E [-]
Roof (typ.)	0.5	0.25	0.25	0
Suspended foyer ceiling	0.5	0.25	0.25	0.3
Foyer plant room	1.0	0.5	5.0	0.3
Foyer – TT flooring	5.6	0.5	5.0	0.3
Foyer – perimeter slab	5.0	0.5	5.0	0.3
Kitchen Extension –rib & in-fill	3.0	0.5	5.0	0.3

Table E.4-4 lists the total seismic weight at each level (including the self-weight of the superstructure).

Table E.4-4 Seismic Weight (kN)

Level	Elevation [m]	Weight [kN]
Roof	12.72	2445
Ground Floor	4.62	11991
TOTAL		**14436**

Modal analysis of the elastic 3D model was carried out to assess the dynamic behaviour of the structure. The first two vibration modes were translational in each principal direction with the corresponding mode shapes shown in Figure E.4-3 and Figure E.4-3. The estimated initial period of vibration in each direction was found as T_1 = 0.8s in the longitudinal direction, and T_2 = 0.64s in the transverse direction.

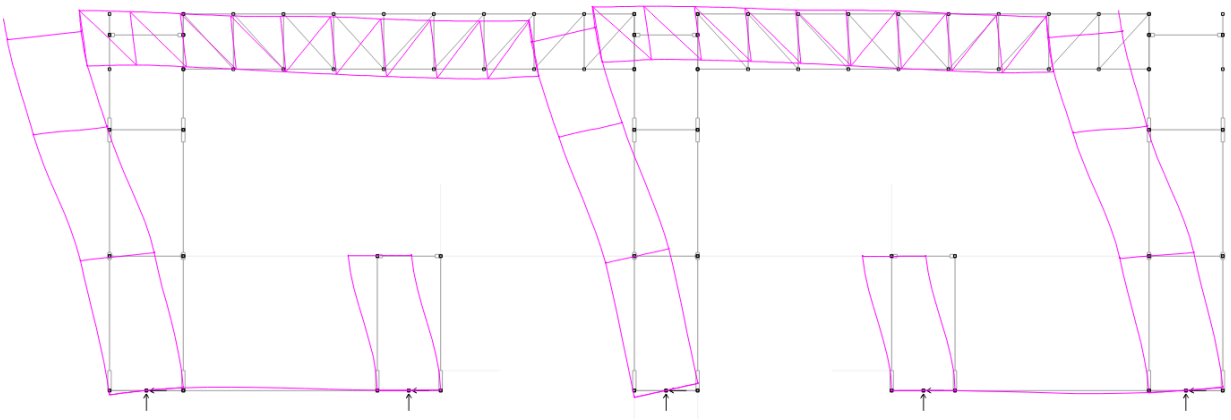

Figure E.4-2: Response of the longitudinal frame line on gridline 7 at mode 1, T_1 = 0.8 s.

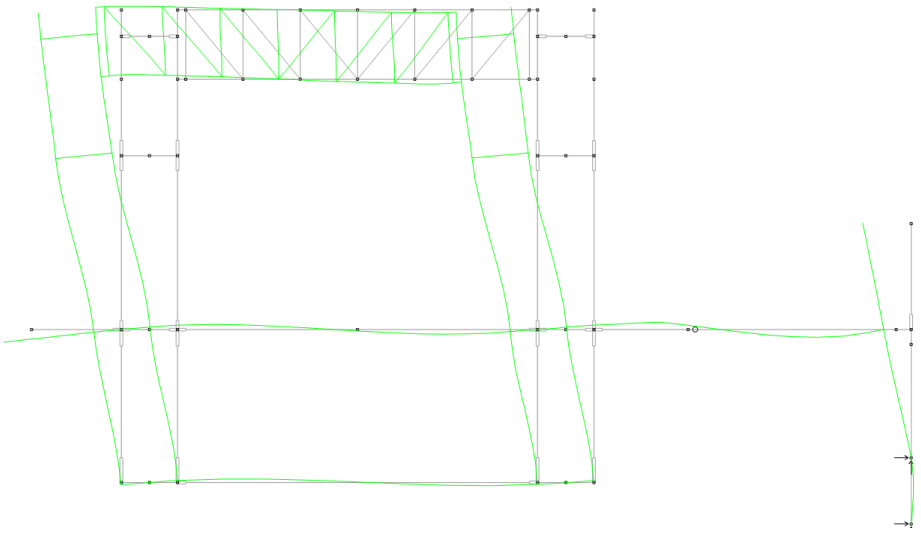

Figure E.4-3: Response of the central transverse Renouf Foyer frames (gridline C) under mode 2, T_2 = 0.64 s (bottom).

E.5 SEISMIC INPUT FOR ADRS AND RESPONSE

E.5.1 Seismic Input Parameters and Response Spectra

The seismic loading parameters used in the assessment are similar for the Auditorium and are shown in Table C-7. The seismic loading parameters allowed for the development of an Acceleration-Displacement Response Spectrum (ADRS), in accordance with Section C3.5 of the Engineering Assessment Guidelines that represented the ULS seismic demand at the site (per NZS 1170.5:2004). The ADRS curve could then be adjusted considering both the level of damping and structural performance (S_p) factor associated with each possible mechanism.

E.5.2 Structural Performance (S_p) Factor

The S_p factor allows for a reduction in the overall seismic demand to account for several factors that are not easily quantified and are not directly accounted for in the design process.

The factor is related to the ductility of the system. Where a larger displacement ductility results in an overall larger reduction of the seismic demands. The S_p-factor is determined in accordance with Section 4.4 of NZS1170.5:2004. For structures that are assessed using the direct displacement-based assessment (DDBA) methodology, the S_p factor is actually applied in two-fold (Marriott 2018). Therefore, the 'DDBD corrected' S_p factor can be determined using the equation proposed by Marriott (2018) below.

$$S_{p,DDBA} = (1 + S_p)/2$$

In the assessment of the capacity curve, an S_p factor of 1.0 was used, as the ductility of the system was not known until the completion of the SLaMA analysis. Instead, the S_p-factor has then been used to reduce the damped spectrum which will result in a similar level of scaling of the results, and as such, in a similar level of scaling of the seismic score.

E.5.3 Damping

Appropriate damping for the Renouf Foyer's primary lateral load resisting system has been determined in in accordance with Section C2D.3.2 of the Engineering Assessment Guidelines. The hysteretic damping appropriate was considered dependent on the possible mechanism:

- For global mechanisms such as beam-sway, column-sway, or a mix of the two, a high level of damping has been assumed, per Section C2D.3.2 for concrete frames with deformed bars.
- For localised beam-column joint-shear at the coupling zone, a medium-high level of damping has been assumed to represent the likely lower dissipation capacity associated with this mechanism.

The system ductility demand, assumed level of damping, and system equivalent viscous damping ratio used to assess the primary structure is summarised in Table E.5-1 below.

Table E.5-1: Assumed level of hysteretic damping associated with each possible inelastic mechanism.

Mechanism	System Ductility Demand, μ_{sys}	Level of Hysteretic Damping	Hysteretic Damping, ζ_{sys}
Transverse Direction			
L1 Column-Sway	2.0	High	10%
L2 Joint Failure	2.4	Medium-to-High	8%
Longitudinal Direction			
L1 Column-Sway	1.76	High	8%
Mixed beam-sway and column-sway	2.1	High	10%

E.5.4 Allowance for P-Delta Effects

An allowance for P-Delta effects should be made for buildings with large drifts. To account for P-Delta effects, the base shear capacity of the system is reduced using the equation below,

$$V_{b,eff} = V_b - C\frac{m_e g \cdot \Delta_{cap}}{H_e}$$

Where; m_e, H_e and Δ_{cap} are the effective SDoF properties associated with each mechanism and C = 0.5 for concrete structures. This resulted in a reduction of 1% and 4% in the transverse and longitudinal directions, respectively.

E.5.5 Allowance for Torsion

Torsion is accounted for in accordance with Appendix C2F of the Engineering Assessment Guidelines. The simplified methods are used to account for torsion due to both strength eccentricity and accidental mass eccentricity as described below. The overall torsion resistance is provided by all lateral load systems in all directions, relative to the effective stiffness of each system.

As the transverse frames at the Eastern and Western ends of the Renouf Foyer have the largest lever arm relative to the centre of mass, it is expected that they will provide the highest contribution to torsion resistance.

Transverse Direction

Based on the layout of lateral load resisting systems as shown in Figure E.5-1, torsion effects in the transverse (Z) direction are only from accidental mass eccentricity. 'Method C' in accordance with C2F.4 of the Engineering Assessment Guidelines could therefore be used as there was limited torsional redundancy expected to be provided by the orthogonal frames. This method allows for the reduction of available base shear capacity to eliminate the strength eccentricity and mitigates any inelastic torsional response. The reduction in base shear capacity was found to be ~20%.

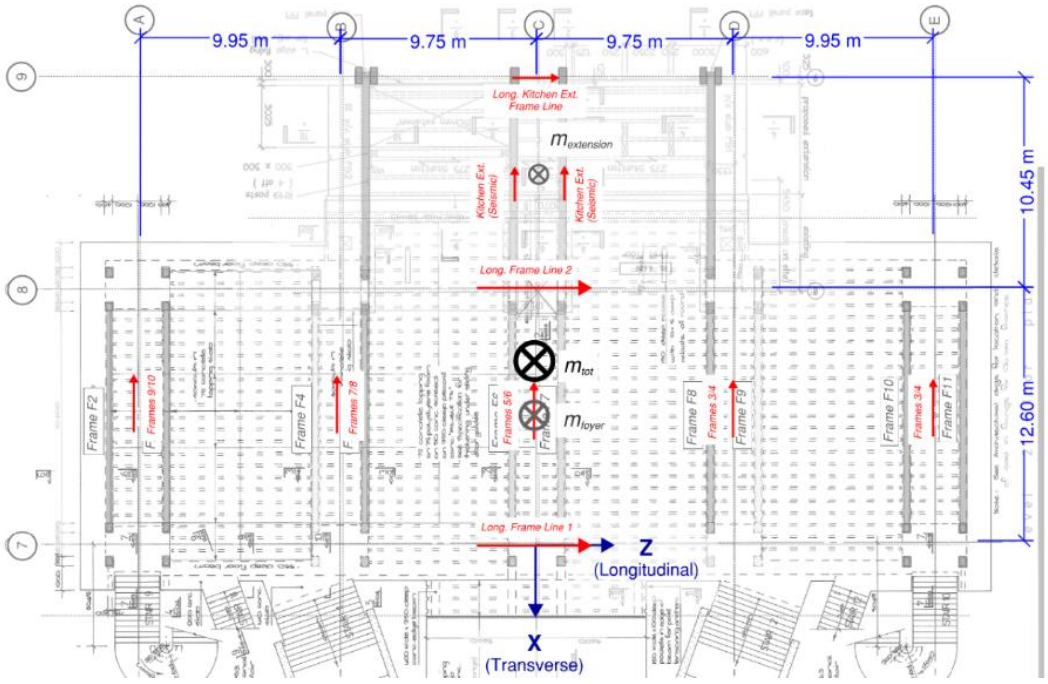

Figure E.5-1: Layout of lateral load-resisting elements and the Renouf Foyer.

Longitudinal Direction

It is expected that for seismic demands in the longitudinal (Z) direction, the torsion demands are assumed to be taken by the double-height frames on gridlines A and E, and the intermediate frames on gridlines B and D. This means that the coupled column units are potentially subject to bi-directional flexure demands. To account for this, the capacity (assumed proportional to the base shear capacity) in the longitudinal (Z) direction was downrated such that there was sufficient residual capacity in the transverse direction to resist the torsion demands due to accidental mass eccentricity. This resulted in ~25% reduction of the total base shear capacity in the longitudinal (Z) direction.

E.6 DESCRIPTION OF PRIMARY FRAME STRENGTH HIERARCHY

The coupled column system creates a frame with unequal spans on a single gridline. Therefore, the non-uniform force distribution and corresponding displacement profile at yield was predicted with the 3D Elastic Microstran V10 model. The model was used to establish which elements would yield first for each frame type.

The strength hierarchy of the critical sub-assemblies was then evaluated following the SLaMA procedure. Overstrength demands from potentially critical elements were used to identify other possible mechanisms that could occur as a full inelastic mechanism develops. Each beam-column joint sub-assembly was generally assessed as follows:

- For typical beam-column joints:
  - Calculate the sway index and identify possible mechanisms.
  - Assess the joint against overstrength demands from the critical elements.

- For atypical beam-column joints:
  - Extract the force distribution within the sub-assembly at first yield from the elastic model.
  - Assess whether the joint or other connecting elements could fail before or during the formation of a full inelastic mechanism, including overstrength.

The findings detailed in the Section E.5 are summarised below.

- All transverse frames were generally seen to first yield between levels 1 and 2 due to ~80% of the seismic weight being located at Level 2. Either the Level 1 columns or primary foyer gravity beams were estimated to yield at comparable levels of seismic demand.
- The critical Level 2 beam-column joint subassemblies generally have inadequate flexural strength margins to develop any one dominant mechanism in the primary foyer frames. The large overstrength factor for Grade 380 reinforcing bars (ϕ_o = 1.5) meant that multiple mechanisms could occur in the primary foyer frames after yielding occurs.
- The Renouf Foyer columns were found to generally limit the inelastic displacement capacity of the system. The plastic rotation capacity of the foyer columns was found to be limited by required margin to onset of loss of gravity load-carrying capacity, in accordance with Section C5.5 of the Engineering Assessment Guidelines.

E.6.1 Yield Hierarchy of Primary Frames

Seismic loads were applied to the Renouf Foyer in accordance with the Equivalent Static Method (ESA), as described in Section 6.2 of NZS1170.5:2004. For the purposes of this assessment, the maximum base shear applied to the structure in both directions was the ULS elastic (μ=1, S_p = 1.0) seismic demand based on the initial period in both directions. This resulted in a horizontal design action coefficient of C_d = 1.19 and C_d = 1.41 in the longitudinal and transverse directions, respectively. The yield capacity of the system is therefore expressed with reference to the elastic ULS demands.

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The equivalent static loads were applied to the model and based on the probable capacity of the beam-column elements within each frame type. The %ULS load at first yield could then be determined through scaling of the equivalent static forces on the structure. The results of the equivalent static analysis of the Renouf Foyer are summarised in Table E.6-1 and Table E.6-2.

It should be noted that the results of the Equivalent Static Analysis were not used in isolation as part of a force-based elastic assessment because:

- The possible inelastic mechanisms had yet to be identified, in order to determine the ductility capacity of the system.
- An elastic model becomes less precise as more elements and mechanisms reach and exceed their yield capacity, because nonlinear behaviour is not captured. The %ULS terminology is used here to communicate the relative yield hierarchy of each element and does not relate to a seismic rating.

Table E.6-1: Yield Hierarchy - Longitudinal Direction.

Frame Type	Est. %ULS at Yield	Critical Frame Element
Intermediate Foyer Frame	25%	Columns hinging (at top & bottom)
End Double-Height Foyer Frames	35%	Level 2 link slab and columns yielding at same demand (±5%ULS)
Central Double-Height Foyer Frame	>50%	Low rotational fixity means central frame yields much later
Kitchen Extension Columns	20%	Cantilever of extension columns results in yielding at the base

It was noted that despite the ground floor level columns typically yielding first in the longitudinal direction, the demands were greater above Level 2 for the central frame and within 15%ULS for the end frames. Therefore, overstrength demands could force column hinging above Level 2. This is further discussed in Section E.8.3.

Table E.6-2: Yield Hierarchy - Transverse Direction.

Frame Type	Est. %ULS at Yield	Critical Frame Element
Double-Height Foyer Frames	35%	Primary gravity beams due to foundation flexibility.
Intermediate Frames	45%	Columns hinging (at top & bottom).
Kitchen Extension (Central Frames)	15% (S) - 40% (N)*	Kitchen extension beam (Southern end), depends on direction.

** Depends on direction of seismic load (N = North, or S = South).*

E.6.2 Transverse Direction

Coupled column units

Based on the ESA described in Section E.6.1, the critical yielding elements were all located between the ground floor level and Level 2 which was deemed appropriate given the weight distribution of the structure. Based on the initial SLaMA procedure the following mechanisms were initially considered possible:

- Column-sway,
- Mixed mechanism with possible yielding of the central gravity beam with column yielding,
- Beam-column joint-shear mechanism between coupled column units.

To assess the likely hierarchy of the beam-column joints at Level 2, the sway index was not considered effective due to the following factors:

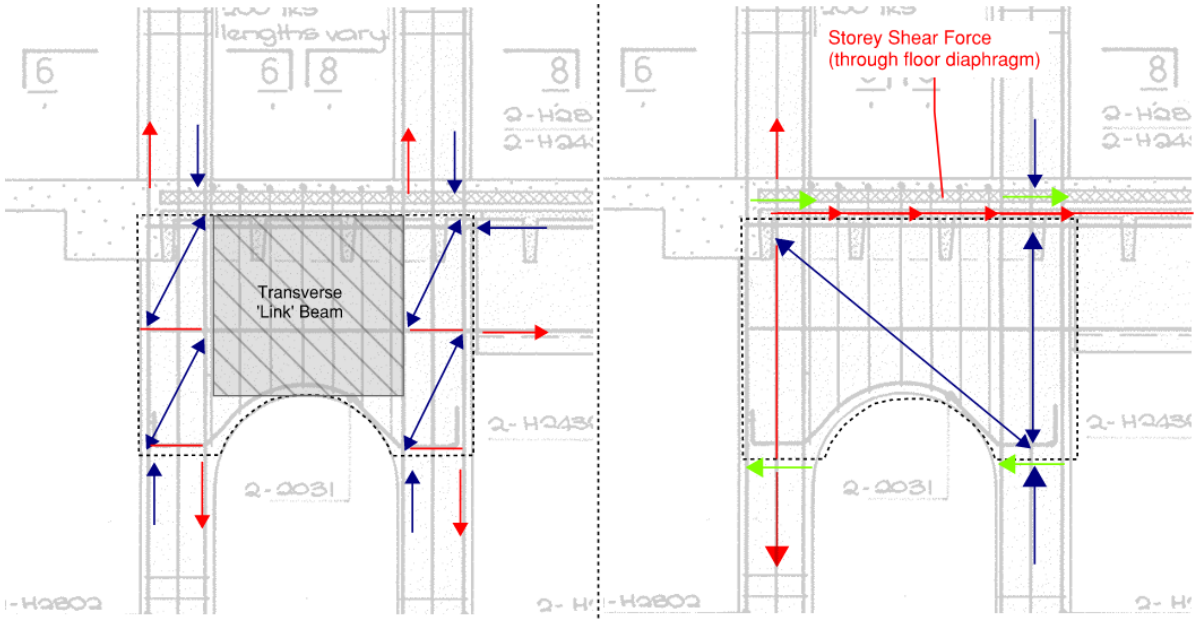
- The joint regions at either end of the main gravity beam were not clearly defined, such that a strength hierarchy could not be readily established.
- The vastly unequal span lengths between coupled column units compared to the main foyer span meant that averaging the behaviour across the whole storey level was assumed to provide limited insight into the strength hierarchy at Level 2.

As shown in Figure E.6-1, the behaviour in the Level 2 beam-column joint could be idealised into the 'micro' frame actions or 'macro' frame actions. The 'micro' frame arises from the coupled column units themselves acting as a frame. The 'macro' frame actions occur where the coupled column unit acts as a large joint zone, with coupled columns providing the moment resistance through a tension/compression force couple.

This joint link beam region was considered a potential deformed region due to the low shear span ratio, where standard flexural theory may not be applicable; as such, the methodology described below was used to assess the strength hierarchy.

- Axial, flexure, and shear demands surrounding the macro joint element were extracted from the elastic model at first yield for each joint type.
- The yield demands were applied to a strut & tie grillage model, which was assessed in accordance with section C5.6.1 of the Engineering Assessment Guidelines.
- The utilization of individual joint components was assessed to identify whether overstrength actions, based on the critical connecting element could be developed without another mechanism occurring.

A typical grillage model used to assess the joint is shown in Figure E.6-2.



"Micro" Frame Model - Column/Beam Moments

"Macro" Frame Model - Seismic Storey Shears & Axial Loads

Figure E.6-1: Indicative strut and tie of Level 2 joint link beam of coupled column units.

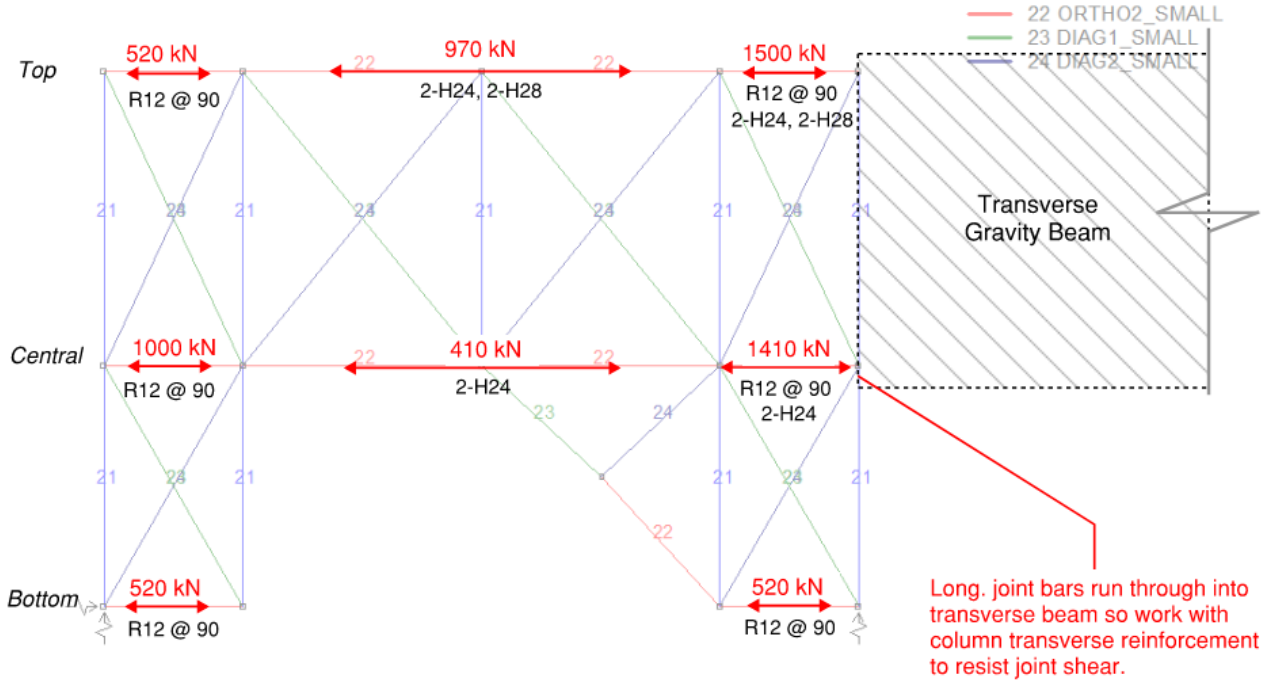


Figure E.6-2: Grillage model layout with horizontal tie capacities for typical beam-column joint (North Side).

Possible mechanisms identified from this analysis are summarised below:

1. Level 1 column-sway
   - Intermediate frames – Level 1 columns identified as first element to yield with the flexural capacity exceeded at ~45%ULS.
   - Double-Height frames – Level 1 columns do not yield first but are unable to resist flexural overstrength demands from connecting foyer gravity beam. Therefore a possible column-sway mechanism was considered.

2. Beam hinging of primary foyer gravity beam.
   - Intermediate frames – foyer gravity beam does not yield first but is unable to resist flexural overstrength demands from connecting foyer columns. Therefore a possible beam-sway mechanism was considered.
   - Double-Height frames – foyer gravity beams identified as first element to yield with flexural capacity exceeded at ~35%ULS.

3. Localised exceedance of the capacity of the horizontal joint reinforcement.
   - In both intermediate & double-height frames, the curved trimmer bars along the joint soffit were found to yield early, without loss of strength of the section. Therefore, these were not considered to be the limiting element. However, they may result in localised spalling between the trimmer bars and the horizontal reinforcement from the beam.
   - The horizontal reinforcement in the joint was found to yield before development of the overstrength actions in either frame type. Typically, the bottom horizontal reinforcement was found to yield due to the imposed shears associated with a flexural mechanism in the external column, as shown in Figure E.6-3.

- As there are no other transverse bars directly restraining the joint, possible localised single crack formation and bar buckling was found to be a potential structural weakness of the section. This is assessed in accordance with section C5.5.3.4 of the Engineering Assessment Guidelines.

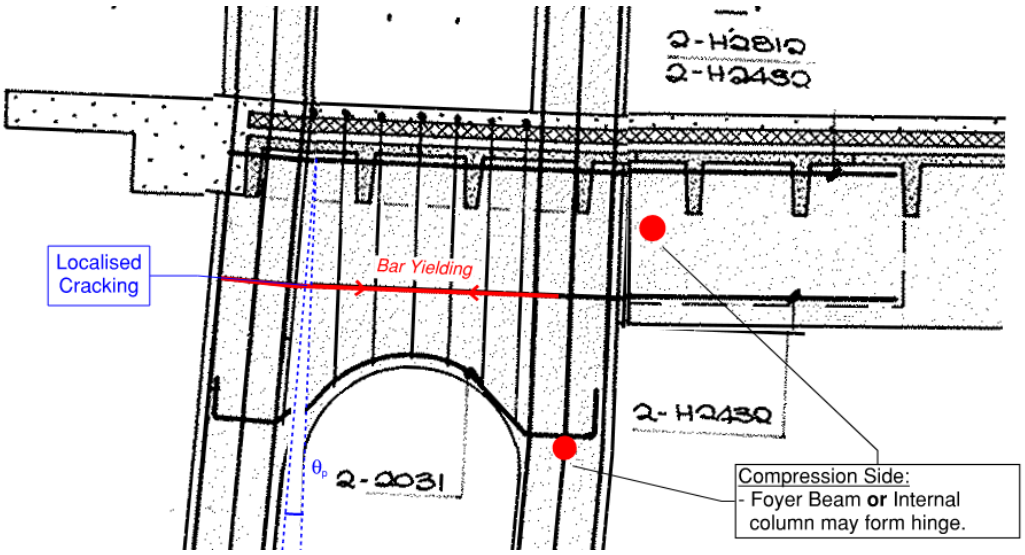


Figure E.6-3: Critical joint mechanism at Level 2 - transverse direction.

Kitchen extension frames

The kitchen extension was evaluated considering two typical frames:

- Central frames which have fixed beam-column connections at the Southern end of the frame bay and where the columns were connected into the existing basement structure.
- End frames which have a pinned beam connection at the Southern end of the frame bay and where the columns have limited base fixity, supported by single 'Franki' type piles.

When assessing the lateral capacity in the transverse direction, only the central frames are considered to be part of the lateral load resisting system. The end frames are considered as a gravity supporting structure and are assessed as part of the secondary structure. The Northern end of the kitchen extension is analysed as part of the primary structure of the Renouf Foyer and found to remain elastic. The summary below therefore refers primarily to the response at the Southern end of the kitchen extension frames.

The potential mechanism of the seismic frames was identified using the sway index shown in Eq. C5.99 from the Engineering Assessment Guidelines below. Where; $\Sigma M_{o,beam}$ = sum of the overstrength capacities of beams framing into the joint, and $\Sigma M_{p,col}$ = sum of the probable capacities of columns framing into the joint.

$$S_i = \frac{\Sigma M_{o,beam}}{\Sigma M_{p,col}}$$

It is generally expected that column hinges form when $S_i > 0.85$, with a mixed mechanism expected for $0.85 < S_i < 1.0$. Because of the configuration and weight distribution of the structure, a column sway mechanism is not necessarily the critical mechanism. A beam-sway at Level 2 was therefore only excluded where the sway index was greater than the beam overstrength factor (typically when $S_i > \phi_0 = 1.5$ for Grade 380 reinforcing). The results are summarised in Table E.6-3 below. The sway index values shown for the ground floor level are indicative with hinges expected to form at the top of the basement wall.

Table E.6-3: Sway index results for central kitchen extension frames - transverse direction.

	South Direction		North Direction	
Level	**Sway Index, Si**	**Possible Mechanism**	**Sway Index, Si**	**Possible Mechanism**
2	0.47	Beam-Sway	0.94	Mixed-Mechanism
1 (Ground)	>1.5	Column-Sway	> 1.5	Column-Sway

Joint shear was checked against the overstrength actions from the critical yielding element. The joint is well reinforced, with the capacity checked in accordance with section C5.6.2.3. The capacity of the joint was directionally dependent on the corbel detail as shown in Figure E.6-4 below. It was found that joint shear could occur in the Northern direction only, when the adjacent plastic hinge undergoes significant inelastic deformations (curvature ductility > 4).

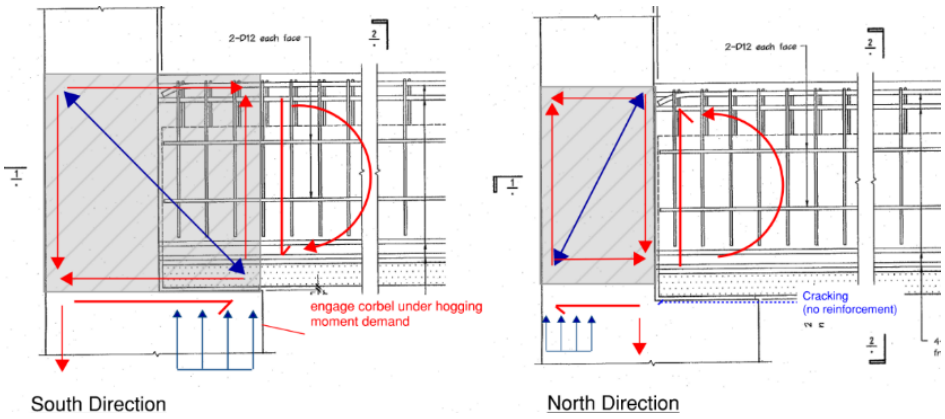


Figure E.6-4: Directional joint shear load path for central frames in kitchen extension.

The basement system was also assessed to consider whether the full shears from flexural overstrength from the central columns could be transferred into the substructure. It was found that when displacing away from the Renouf Foyer (to the south), there is likely insufficient capacity to transfer shears into the basement walls considering:

1. The nominal reinforcing in the basement roof slab required to act in tension.
2. The flexibility of the out-of-plane resistance of the basement wall along gridline 9 to distribute load out to the in-plane walls and the basement floor slab.

Based on this, the columns may need to rely on passive soil pressures to develop the full flexural overstrength in the central frames. This could increase the base flexibility of the central frames compared to that assumed in the elastic model.

E.6.3 Longitudinal Direction

Coupled column units

Similar to the transverse direction, the critical yielding elements are generally located between the ground floor level and Level 2. As the coupled column units are connected in groups of four, each column group was treated as a sub-assembly frame system, where the possible inelastic mechanisms at each level could be determined using the sway indices as described in Section E.6.2. The sway indices for the coupled column units in the longitudinal direction are summarised in Table E.6-4.

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Table E.6-4: Sway index results for the coupled column units - longitudinal direction

	Double-Height Frames		Intermediate Frames	
Level	**Sway Index, Si**	**Possible Mechanism**	**Sway Index, Si**	**Possible Mechanism**
Roof	2.06	Column-Sway	-	-
Mid-Height of Foyer	1.03	Mixed	-	-
Level 2	1.3	Mixed	1.53	Column-Sway
Level 1	> 1.5	Column-Sway	> 1.5	Column-Sway

The following possible mechanisms are identified based on the sway index results in Table E.6-4.

- Intermediate frames: Column-sway
- Double-height frames: Column-sway or Beam-sway with column hinging at the ground floor level and roof level

The longitudinal frame system consists of multiple unconventional beam-column joint connections with the link slab elements. These are assessed considering possible overstrength from the critical connecting elements to identify potential joint that could reduce the capacity of the system. The joint capacity checks are summarised below.

Level 2 link slab element

The longitudinal link slab element at Level 2 is a one-way reinforced slab element between the coupled column units which generates frame action in the longitudinal direction. The link slab elements have confined edge elements, reinforced with stirrups that can undergo inelastic deformations and an unconfined core section that is assumed to lose its capacity shortly after yield. The link slab element was checked for beam-column joint-shear against overstrength actions considering:

- Punching shear of the whole link slab, checked in accordance with Section 12.7 of NZS3101:2006.
- Beam-column joint-shear of the confined edge elements after significant inelastic deformations (link slab section has yielded). The joints were assessed in accordance with Section C5.6.2.2 of the Engineering Assessment Guidelines, considering $\mu_\phi > 9$, per Figure C5.27.

In both cases above there is sufficient capacity to resist overstrength actions from the critical connecting element. This indicates the storey shear capacity at the ground floor level is not limited by joint shear and hinging could develop in either the link slab element or coupled column units.

Upper link slab element

The upper link slab elements above Level 2 consist of the same confined edge elements as the link slab, however, they do not connect directly into the coupled column units as shown Figure E.6-5.

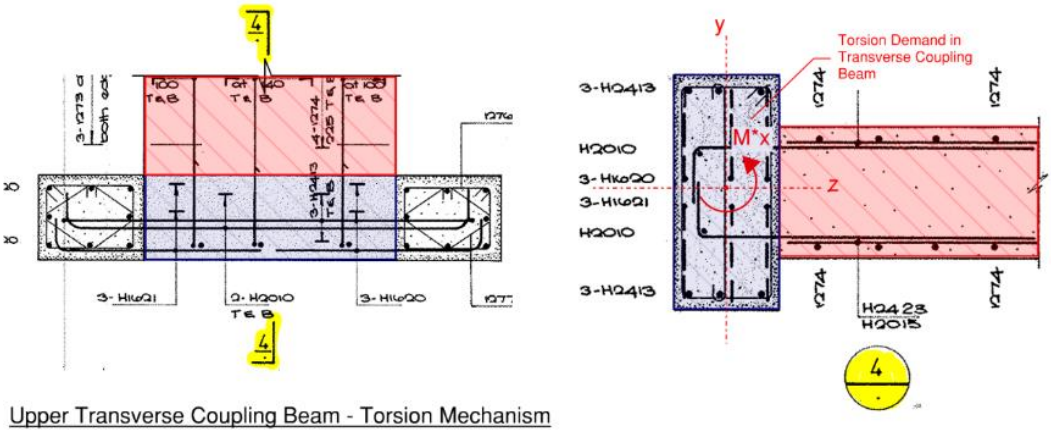


Figure E.6-5: Upper link slab elements connection with the coupled column units

These lead to an unconventional joint mechanism where the transverse link beam zones are required to act in torsion to transfer moment from the link slab out to the columns. The torsion capacity of the transverse link beam was checked considering:

- Local punching due to eccentric shear demand as the link slab transfers moment into the transverse link beam (checked in accordance with NZS3101:2006, Section 12.7).
- Torsion capacity of the transverse beam, assuming the section acted as a thin-walled element, in accordance with NZS3101:2006, Section 7.6.

In both cases, the torsion mechanism was insufficient to resist overstrength demands associated with a flexural mechanism in the link slab. This means that beam-column joint-shear can occur as the link slab reaches its yield capacity. The torsion mechanism was not considered to pose a life-safety risk but would result in the loss of rotational restraint provided to the columns by any link slabs that reach yield.

Kitchen extension

The kitchen extension has no moment frame action in the longitudinal direction. Instead, the only lateral capacity was provided through cantilever action of the coupled column units on gridline 9. At the base, the central columns are restraint by the basement wall acting in-plane and the end frame columns are rotationally restrained by foundation beams connected to the basement structure. Based on the restraint conditions, it was clear that a cantilever mechanism would occur with a single plastic hinge at the base of the kitchen extension columns.

The rotation capacity of this area is concluded not to be critical in the assessment and the kitchen extension will effectively drift along with the primary structure of the Renouf Foyer.

E.7 PRIMARY FRAME INELASTIC DISPLACEMENT CAPACITY

E.7.1 Transverse Direction

In the transverse (X) direction, any base shear capacity of the central kitchen extension frames was conservatively ignored. There is likely to be a possible joint shear mechanism and the exceedance of the basement wall capacity. The base shear contribution from the kitchen frames is small (< 5%) and either mechanism was not considered to pose a significant life safety risk. The beam-column joint is supported by the large concrete corbel, so loss of gravity support of the kitchen extension beam is unlikely.

In the transverse (X) direction, both the mass and displacements are concentrated at Level 2 such that the effective SDoF properties could be approximated by the behaviour at Level 2. The effective SDoF properties assumed in the transverse direction are shown below:

Effective Mass, m_e : 1470 tonnes (100% mass participation)
Effective Height, H_e : 4.62 m (Level 2 floor height)

Based on the equivalent static analysis, the yield displacement and base shear capacity of each frame could be extracted from the model as shown in Table E.7-1 below.

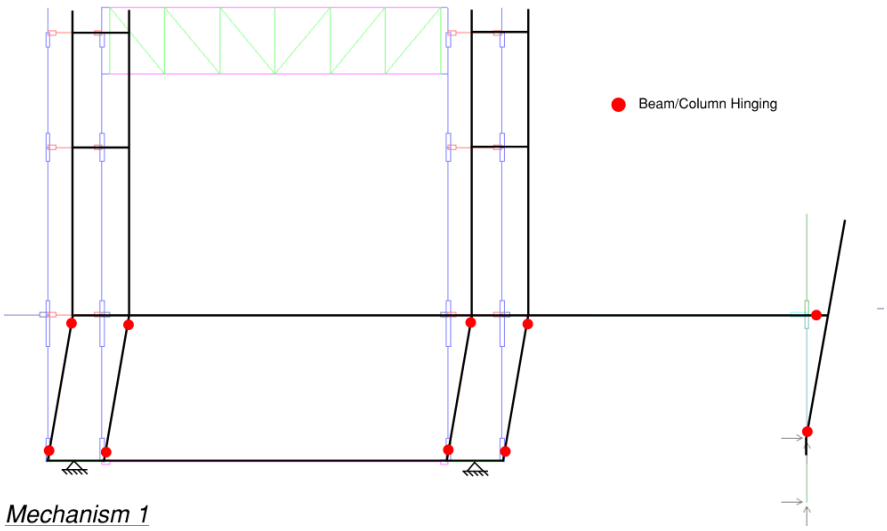
Table E.7-1: Foyer frame capacities at yield - transverse direction.

Frame Type	No. of Frames	%ULS at Yield	Yield Displacement, Δy [mm]	Yield Base Shear (per frame) [kN]
Typical Double-Height frame	4	35	30	505
Typical Intermediate	4	45	38	1060
Central Double-Height	2	35	30	726

Because inelastic deformations are concentrated between the ground floor level and Level 2, the critical mechanism was determined based on the inelastic rotation capacity of each. No clear dominant mechanism could be identified; therefore, the global displacement capacity was found for each of the following:

1. Plastic rotation capacity of the Level 1 foyer columns during a Level 1 column-sway mechanism.
2. Plastic rotation capacity of the primary Level 2 foyer gravity beams during a mixed mechanism.
3. Inelastic displacement capacity associated with localised failure of the Level 2 foyer gridline B and C joint during a mixed mechanism.

It was found that the plastic rotation capacity of the foyer columns during a Level 1 column-sway mechanism limited the global displacement capacity of the system. The assumed inelastic displacement profile of the Level 1 column-sway mechanism is shown in Figure E.7-1 below.



Mechanism 1

Figure E.7-1: Limiting column-sway mechanism in the transverse (X) direction

The plastic rotation capacity of the L2 foyer columns and corresponding inelastic displacement at Level 2 was determined using the Direct-Rotation Method, in accordance with Sections 5.5.3 and 5.5.4 of the Engineering Assessment Guidelines. The double-height frames along gridline C have additional axial load from the kitichen extension frames which limits the rotation capacity of the system due to the higher gravity and seismic actions. The parameters used for the Direct-Rotation Method are shown in Table E.7-2 .

Table E.7-2: Element parameters for inelastic rotation capacity of the foyer columns in the transverse direction.

Parameter	Value
Max axial load at yield, N* [kN]	1900
Shear Demand at yield, V_y [kN]	436
Probable Shear Capacity, $V_{p,col0}$ [kN]	808
Shear Demand Ratio, $V_y/V_{p,col0}$ [kN]	0.54
Transverse reinforcement ratio, ρ_t	1.07%
Plastic rotation capacity θ_p[1]	2.11%
Rotation capacity at onset of axial failure, θ_a[2]	2.47%
Clear span height, h_{clear} [m]	2.90

[1]Calculated using Eq. C5.33 in the Engineering Assessment Guidelines.
[2]Calculated using Eq. C5.59 in the Engineering Asessment Guidelines.

The total displacement capacity of the Renouf Foyer frames was limited by ensuring sufficient margin to onset of gravity support failure as shown in the calculations below.

$$\Delta_p = \theta_p \cdot h_{clear} = 2.11\% \cdot 2900\ mm = 61\ mm$$

$$\Delta_a = \theta_a \cdot h_{clear} = 2.47\% \cdot 2900\ mm = 72\ mm$$

$$\min\{\Delta_y + \Delta_p, \frac{2}{3}(\Delta_a + \Delta_y)\} = \min\left\{30mm + 61mm, \frac{2}{3} \cdot (30\ m + 72mm)\right\} = \mathbf{68\ mm}$$

Considering a Level 2 yield displacement (Δ_y) of 30 mm for the central double-height frames, the inelastic displacement capacity of the system was therefore limited to $\Delta_{p,max}$ = 38 mm to ensure sufficient margin to onset of gravity support failure.

The inelastic displacement capacity at Level 2 associated with flexural hinging of the primary foyer gravity beams or localised joint failure at Level 2 were also evaluated but found not to be controlling. Figure E.7-2 and Figure E.7-3 show the assumed inelastic displacement profiles associated with these mechanisms with the plastic displacement capacity of each possible mechanism summarised in Table E.7-3 below.

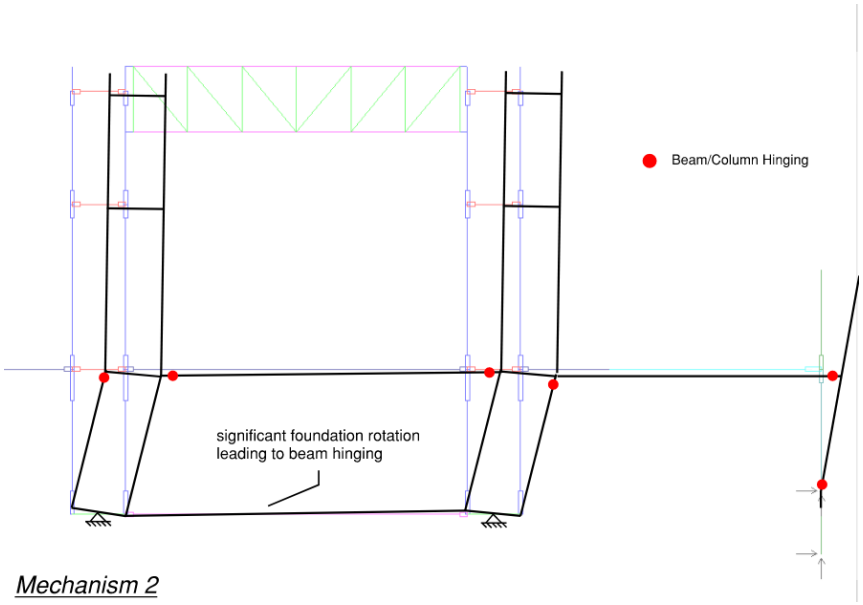


Figure E.7-2: Possible mixed mechanism with foyer beam hinging in the transverse (X) direction

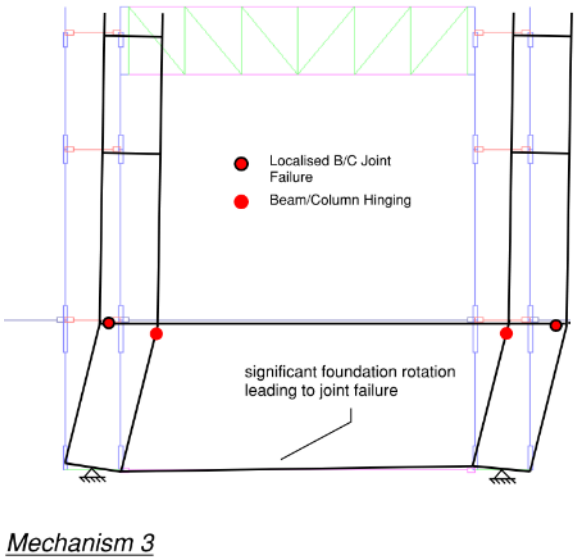


Figure E.7-3: Possible mixed mechanism with localised joint failure at Level 2 in the transverse (X) direction.

Table E.7-3: Summary of inelastic displacement capacity associated with each mechanism - transverse direction.

Mechanism	Method	Inelastic displacement capacity, Δ_p [mm]	Notes:
Column-sway	Direct-rotation method (Eq. C5.33 & Eq. 5.59]	38	▪ Capacity limited by ensuring margin until onset of loss of gravity support (Eq. C5.8). ▪ Typical inelastic rotation capacities ranged from 0.021 to 0.027 rad.
Beam-sway	Direct-rotation method [Table C5.7]	62	▪ Inelastic rotation capacity ranged from 0.017 to 0.02 rad.
Beam-column joint-shear	Moment-Curvature Method [Eq. C5.52]	52	▪ Considering single crack across bottom joint reinforcing. ▪ Limit ultimate strain to consider possible bar buckling (Eq C5.58) ▪ 10 mm elongation of bottom reinforcing.

As shown in the Table E.7-3, the inelastic displacement capacity of the column-sway mechanism is lowest, followed by the localised joint shear at Level 2. It was noted that either the joint shear or column-sway could be critical as each mechanism has a different expected level of hysteretic damping (see Section E.5.3). The critical bi-linear pushover properties in the transverse (X) direction are summarised in Table E.7-4. An effective yield point based on relative base shear contribution of each frame type could also be found as **$\Delta_{y,eff}$ = 34 mm**.

Table E.7-4: Summary of Bi-linear pushover curves - transverse direction.

	Yield Displacement, Δy [mm]	Limiting Mechanism	Ultimate Displacement, Δ_{cap}[1] [mm]	Yield Base Shear (per frame) [kN]
Typical Double-Height Frames	30	Column-Sway	72 (82)	2020
Typical Intermediate Frames	38	Column-Sway	90 (90)	4200
Central Double-Height Frames	30	Column-Sway	**68 (82)**	1452
			Σ	**7672**

[1] *Displacements in () relate to localised beam-column joint-shear at Level 2.*

The critical superstructure response for comparison with the ULS acceleration-displacement response spectrum (ADRS), is shown in Table E.7-5 below.

Table E.7-5: Effective SDoF pushover properties - transverse direction.

Δ (mm)	Probable Base Shear, V_b [kN][1]	Spectral Acceleration, S_A [g]
0	0	0
34	5975	0.414
68	5975	0.414
82	5975	0.414

[1]*Includes reduction for torsion and for P-Δ effects see Sections E.5.4 and E.5.5.*

E.7.2 Longitudinal Direction

A similar method was used in the longitudinal (Z) direction. However, as the response was less dominated by inelastic deformations between the ground floor level and Level 2, the full displacement profile was used to estimate the effective SDoF properties associated with each possible mechanism. The effective SDoF properties were calculated in accordance with Section C2.4.2, Steps D1-3 of the Engineering Assessment Guidelines.

Based on the ESA, the yield displacement and base shear capacity for each frame type could be extracted from the model as shown in Table E.7-6 below. It was noted that the central double height frames did not contribute to the base shear capacity due to the flexibility in the foundation system. Based on the strength hierarchy as summarised in Section E.6.3, the base shear capacity in the double-height frames was also calculated considering the loss in lateral load capacity that may occur due to:

- Loss of the unconfined core of the Level 2 link slab elements as inelastic deformations occur as part of a possible beam-sway mechanism.
- Beam-column joint-shear due to torsion mechanism in upper link slab that could occur if the link slab yields as part of a beam-sway mechanism.

A second elastic model was used to capture the residual yield capacity after these mechanisms occur after the frames yield. The key differences in the updated model (MS Model B) are shown in the right of Figure E.7-4 below.

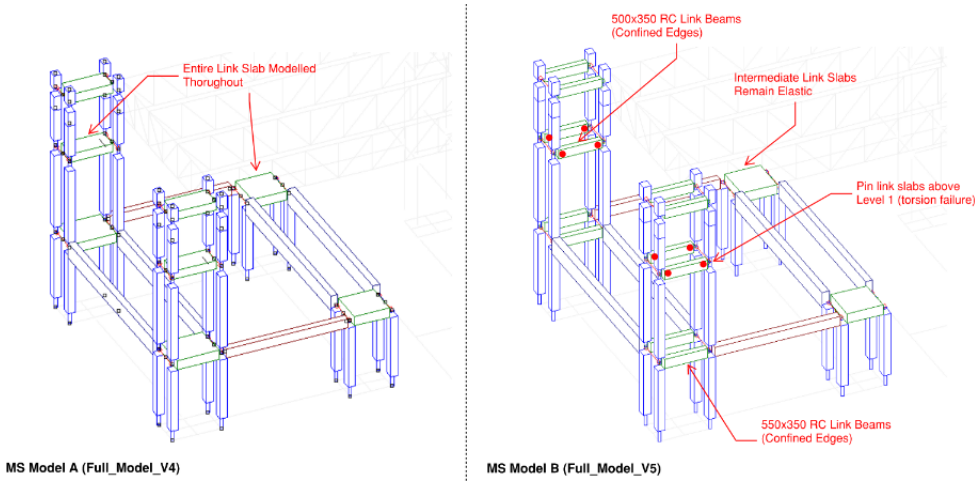


Figure E.7-4: End section of Microstran V10 Model A (left) & updated Model B (right).

Table E.7-6: Foyer frame capacities at yield - longitudinal direction

Frame Type	%ULS at Yield	Yield Base Shear (per frame) [kN]	
		Initial (Model A)	Residual (Model B)
End Double-Height Frames	33%	1905	988
Intermediate Frames	24%	2446	2446
Kitchen Extension Columns	27%	295	295

Based on the strength hierarchy of the frames in the longitudinal direction, three possible inelastic mechanisms were identified.

1. Level 1 column-sway in all frames.
2. Level 1 column-sway in intermediate frames and a beam-sway mechanism in the double-height frames.
3. Level 2 column-sway mechanism in the double-height frames.

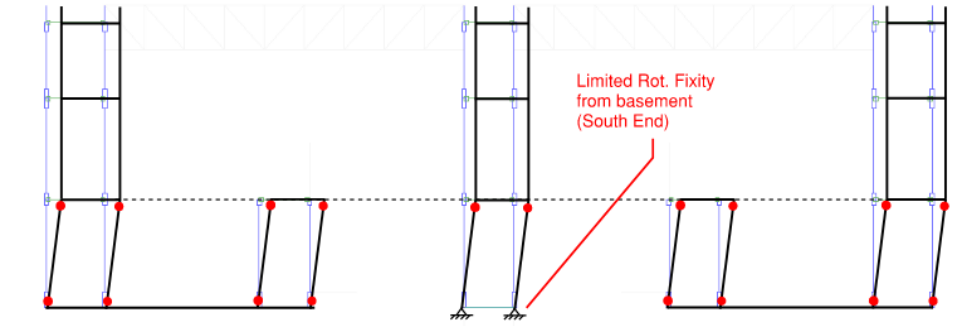
It was found that all three mechanisms were limited by the 2.5% drift limit due to the flexibility of the foyer frames in the longitudinal direction. Each mechanism is qualitatively shown below alongside the corresponding displacement profile for the double-height frames. The allowable plastic inter-storey drifts associated with each mechanism are shown in Table E.7-7 below.

Table E.7-7: Summary of inelastic displacement capacity associated with each mechanism - transverse direction.

Mechanism	Method	Allowable inelastic story drift, θ_p[1] [%]	Notes:
Column-sway (soft-storey)	Direct-rotation method [Eq. C5.33 & Eq. 5.59]	1.0-1.5%	▪ Limited by 2.5% drift limit at L1. ▪ Allowable foyer column inelastic rotations of 0.014-0.017. ▪ Kitchen column rotation capacity not critical.
Mixed beam-sway and column-sway	Beams – Direct-rotation method [Table C5.7] Columns – [Eq. C5.33 & Eq. 5.59]	1.0-1.4%	▪ Limited by 2.5% drift limit at L1. ▪ Inelastic rotation capacity of confined edges of link slabs of 0.014 rad.
Column-sway above Level 2	Moment-Curvature Method [Eq. C5.52]	1.0%	▪ Limited by 2.5% drift above L2.

[1]Intermediate frames yield earlier than double-height frames leading to larger allowable plastic drifts.

1. Column-sway in all frames



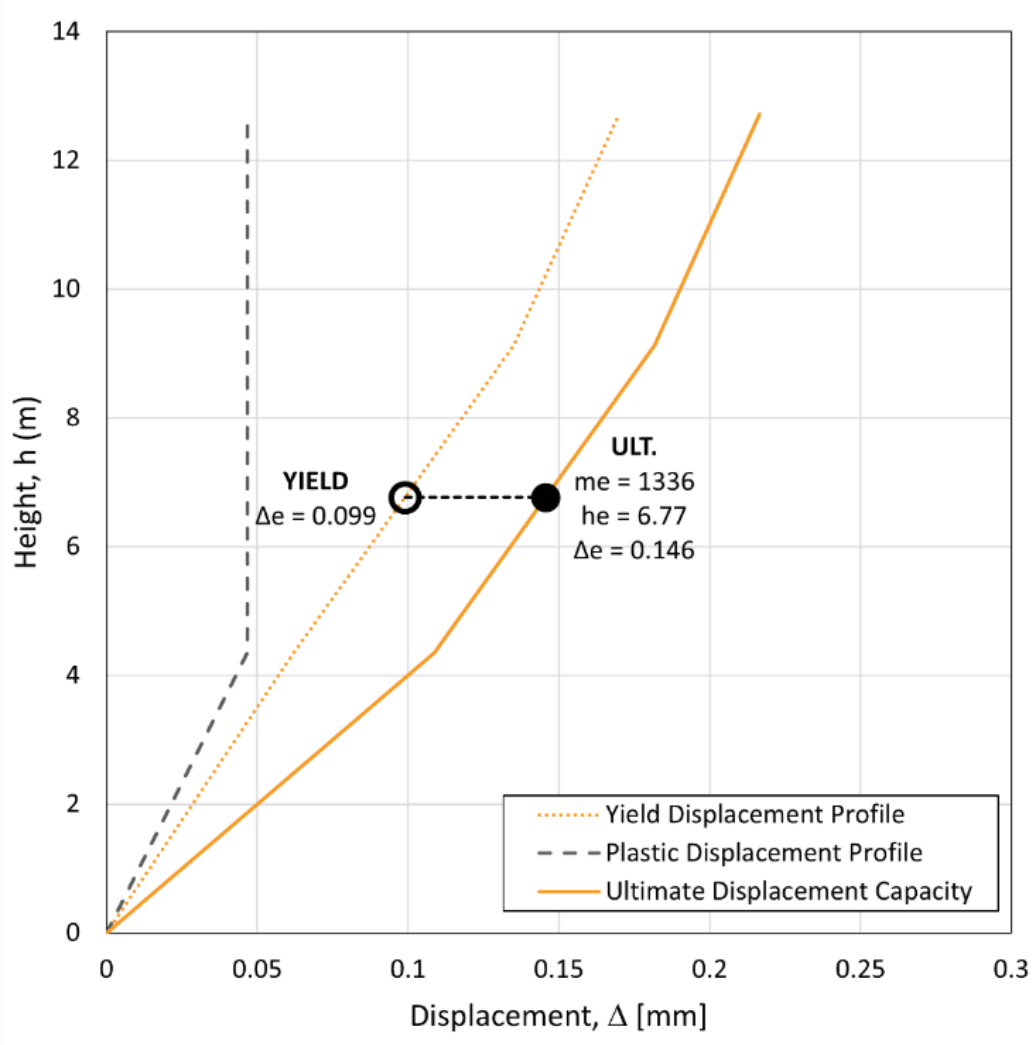


Figure E.7-5: Mechanism and displacement profiles associated with a column-sway in the longitudinal direction

2. Mixed beam-sway (end double-height frames) + column-sway (intermediate frames)

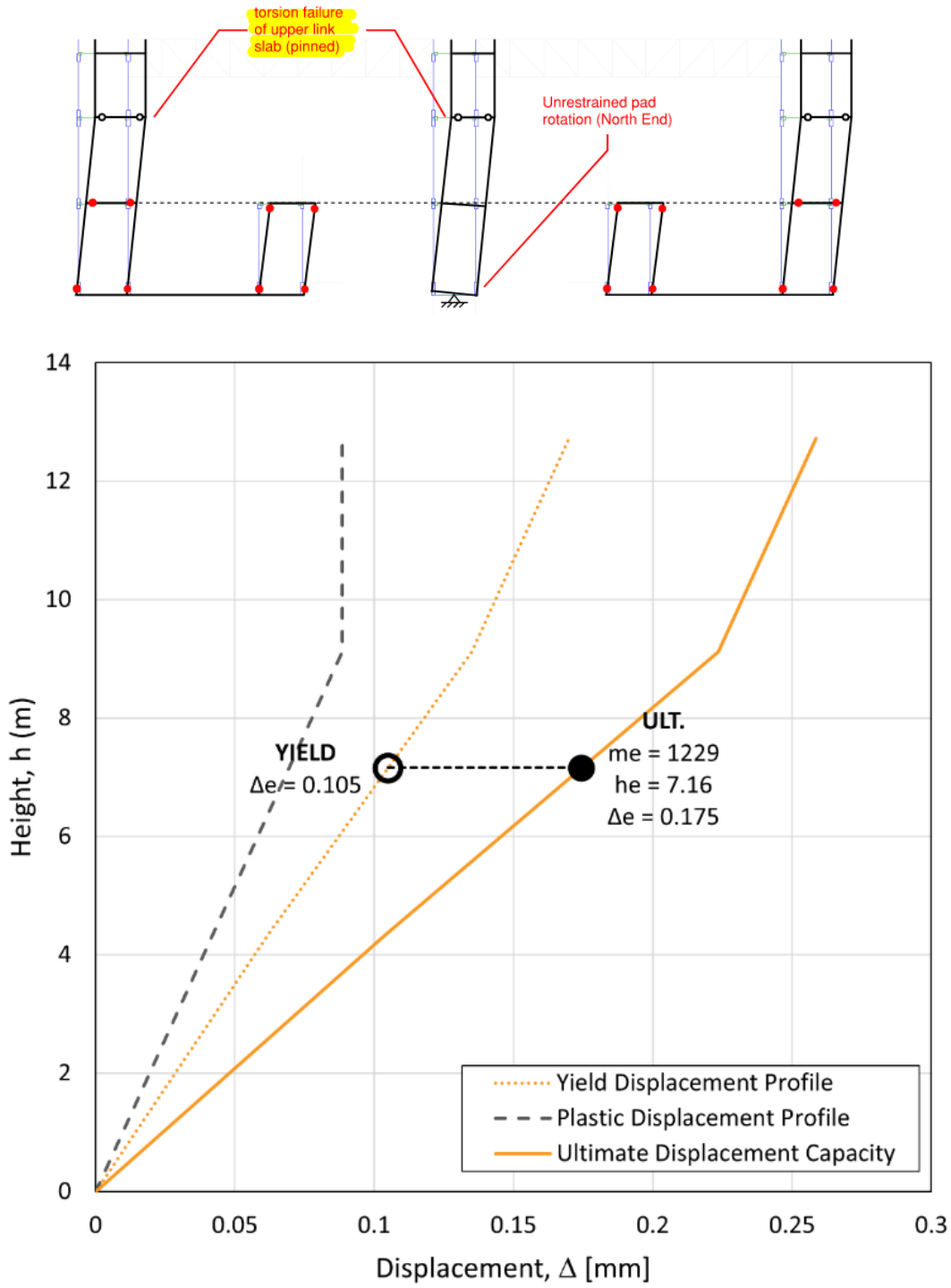


Figure E.7-6: Mechanism and displacement profiles associated with a mixed beam-sway (double-height) and column-sway (intermediate) in the longitudinal direction

3. Column-sway above Level 2

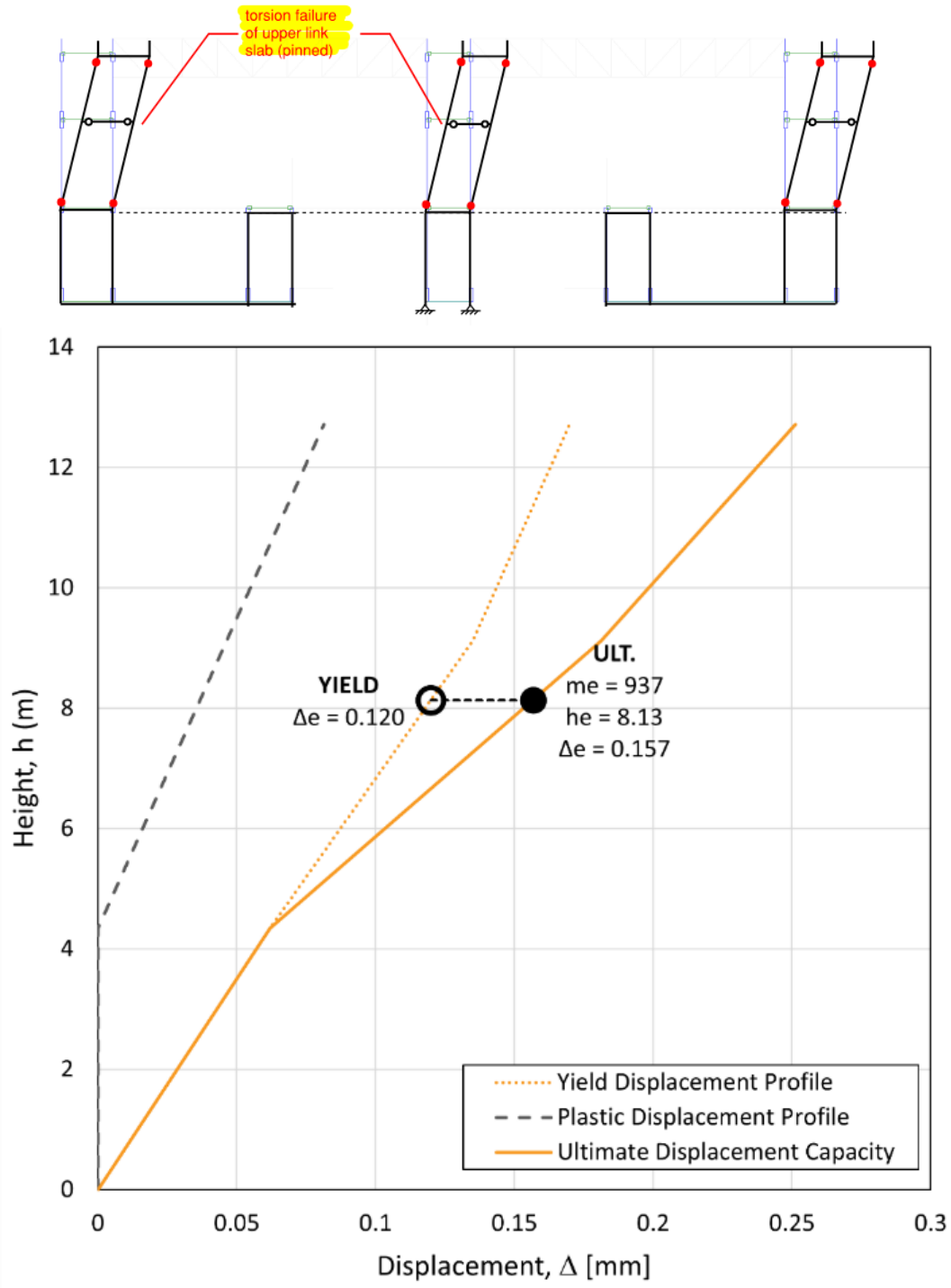


Figure E.7-7: Mechanism and displacement profiles associated with Level 2 column-sway (double-height) in the longitudinal direction.

Based on yield and ultimate displacement profiles associated with each mechanism, bilinear push-over curves were developed for the structure associated with each mechanism, summarised in Table E.7-8.

Table E.7-8: Summary of bi-linear pushover curves associated with each possible mechanism - longitudinal direction.

Mechanism	V_b[1] [kN]	Effective SDoF Properties				
		m_e [tonnes]	H_e [m]	$\Delta_{y,eff}$ [mm]	Δ_{cap} [mm]	s_A [g]
L1 column-sway (soft-storey)	4502	1336(91%)	6.8 (53%)	83	146	0.258
Mixed beam-sway and column-sway	3582	1229(84%)	7.2 (56%)	83	175	0.223
Column-sway above Level 2	3640	937(64%)	8.1 (64%)	95	157	0.297

[1]*Base shear capacity includes reduction for torsion and for P-Δ effects see Sections E.5.4 and E.5.5.*

By inspection, the column-sway above Level 2 will not be critical based on the SLaMA procedure used. However this mechanism was also evaluated considering Parts & Components loading, per Section 8 of NZS1170.5:2004, as a secondary check. This was because the roof weight made up less than 20% of the total seismic mass of the superstructure therefore possible higher mode effects could impact the seismic performance. The force-based assessment of a possible mechanism above Level 2 is discussed in Section E.8.3.

E.8 ELEMENT EVALUATION

This section contains the evaluation of the primary lateral system using the SLaMA method, and for the secondary structural elements not considered part of the primary lateral system included in the scope of this assessment.

E.8.1 Building Global Displacement & Drifts

Displacements of the Renouf Foyer were determined based on the displacement-based assessment procedure carried out in accordance with Section C2.4.2 of the Engineering Assessment Guidelines. The estimated displacement profiles and inter-storey drifts are shown for the column-sway soft-storey mechanism in both directions as this was found to give the lowest seismic score for the superstructure (see Section E.8.1).

Displacements profiles are shown at yield (32%ULS) when a significant life safety hazard is expected to occur (64%ULS), and lastly for an estimated 100%NBS_{IL3} demands.

The displacement profile at 100%NBS_{IL3} is estimated considering the spectral demand for the critical mechanism. The inelastic displacements associated with the critical mechanism were then further extended beyond life safety criteria to match the spectral displacement demand at 100%NBS_{IL3}.

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Table E.8-1: Overview of displacements and drifts of the Renouf Foyer in the Transverse (X) direction.

Level	Height	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]
		32%ULS (at Yield)		64%ULS		100%NBS_{IL3}	
Level 1	RL+2.00m	0	0.73%	0	1.47%	0	2.29%
Level 2	RL+6.62m	34	0.87%	68	0.87%	106	0.87%
	RL+11.12m	73	0.61%	107	0.61%	145	0.61%
Roof	RL+14.72m	95	0.61%	129	0.61%	167	0.61%

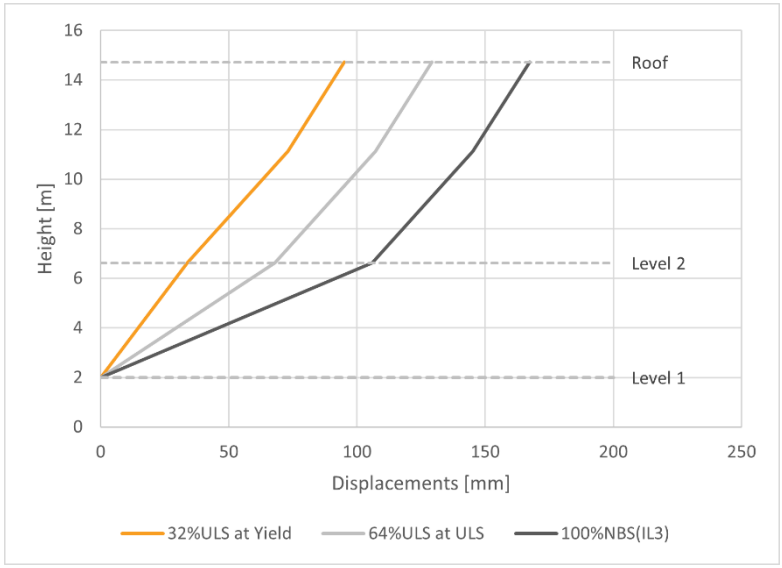

Figure E.8-3: Displacement profile

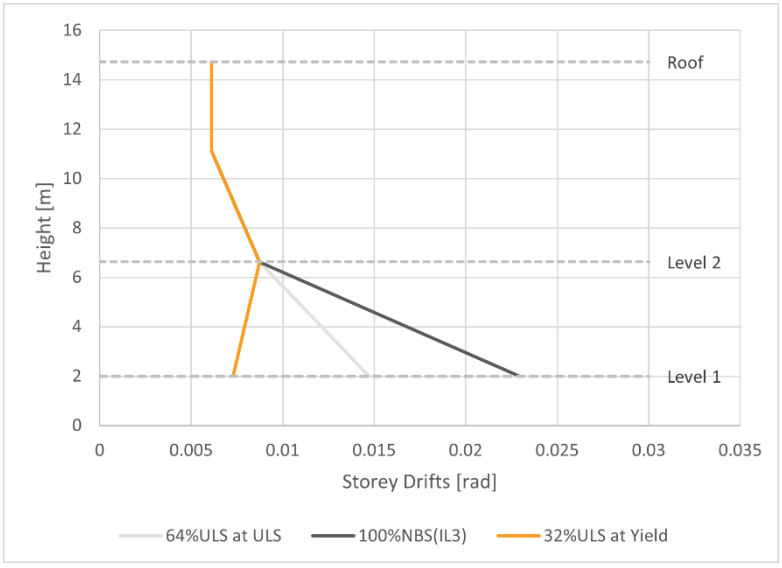

Figure E.8-2: Inter-storey drifts

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Table E.8-2: Overview of displacements and drifts of the Renouf Foyer in the Longitudinal (Z) direction.

Level	Height	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]
		28%ULS (at Yield)		57%ULS		100%NBS(II3)	
Level 1	RL+2.00m	0	1.34%	0	2.36%	0	4.88%
Level 2	RL+6.62m	62	1.62%	109	1.62%	226	1.62%
	RL+11.12m	135	0.97%	182	0.97%	299	0.97%
Roof	RL+14.72m	170	0.97%	217	0.97%	333	0.97%

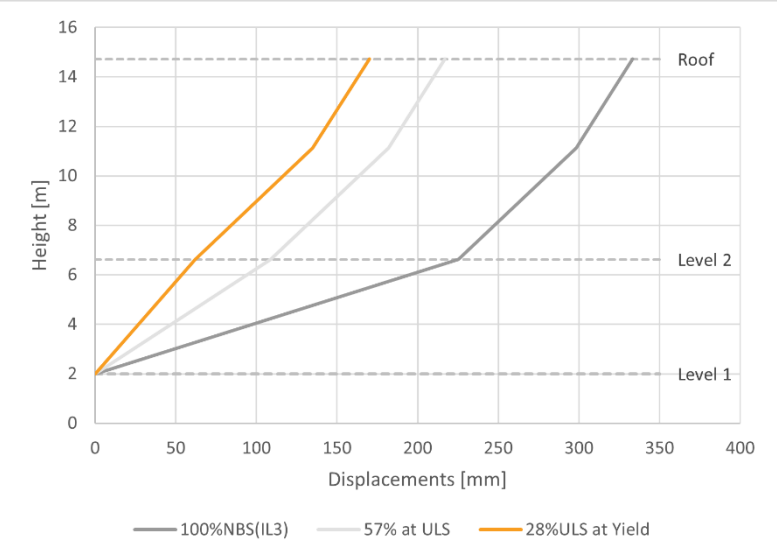

Figure E.8-4: Displacement profile

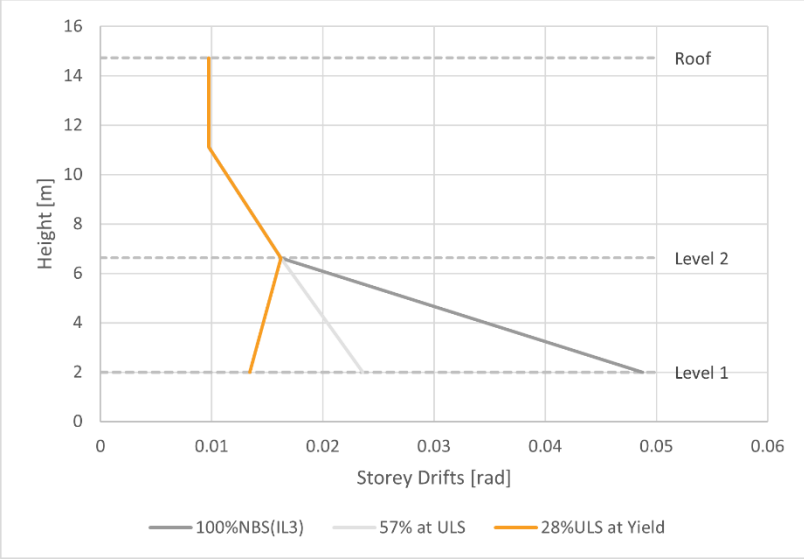

Figure E.8-5: Inter-storey drifts

E.8.2 Summary Primary Frame Seismic Performance

The seismic performance of the primary lateral systems can be evaluated with respect to the seismic demand as discussed in Section E.7.1, considering the possible force-displacement response in both directions as discussed in Section E.7.

Push-over plots

The structure's capacity is determined using the SLaMA technique in accordance with Step 6 of Appendix C2A of the Engineering Assessment Guidelines. This involves plotting the push-over plots for the structure on the Accelerations Displacement Response Spectra (ADRS). A line is extended from the origin through the point of ultimate displacement and spectral acceleration to intersect with the ADRS curve. The %NBS is the ratio of the displacement at the intersection with the ADRS curve and the ultimate displacement.

Figure E.8-6 and Figure E.8-7 show the response of the structure considering the possibly critical inelastic mechanisms in each direction. The corresponding seismic scores are summarised in Table E.8-3.

Table E.8-3: Capacity of the Renouf Foyer two-way moment frame system from SLaMA

	Ultimate Displacement Capacity [mm]	Ultimate Displacement Demand [mm]	%NBS$_{IL3}$
Transverse Direction			
Minimum of Column-Sway (governs) & Beam Sway	68	106	64%
Beam-column joint-shear	82	125	65%
Longitudinal Direction			
Column-Sway (All)	146	255	57%
Mixed beam-sway (end double-height frames) + column-sway (intermediate frames)	175	286	61%
Column-Sway above Level 2[1]	157	258	61%

[1]*See section Section E.8.3 for FBA of mechanism considering P&C loading.*

Table E.8-3 shows the performance of Renouf Foyer frames following the DBA procedure is governed by the column-sway (soft-storey) in both directions, resulting in an seismic score of **57%NBS$_{IL3}$**.

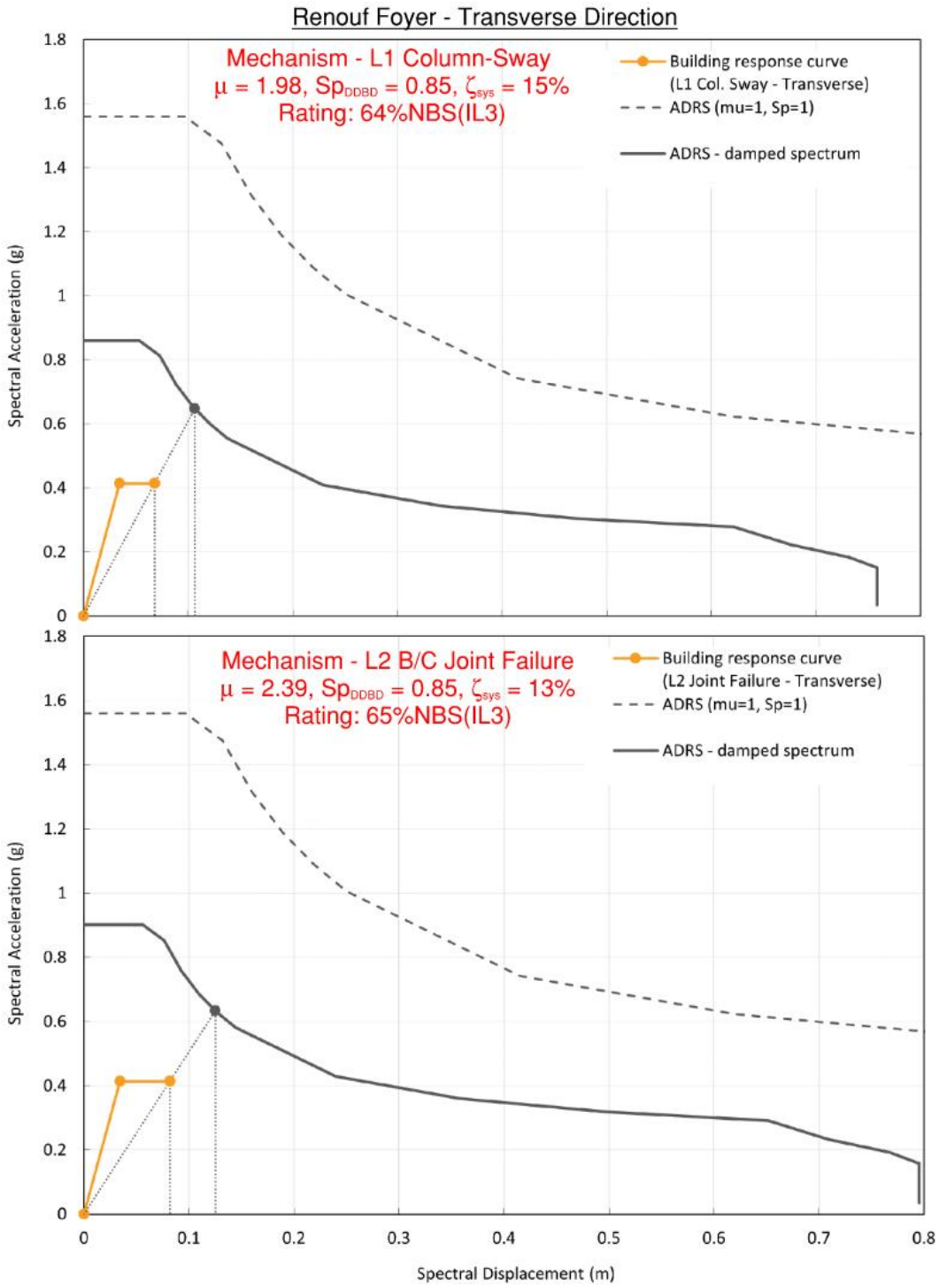

Figure E.8-6: Plot of ADRS and capacity curve associated with potentially critical mechanisms in the transverse direction.

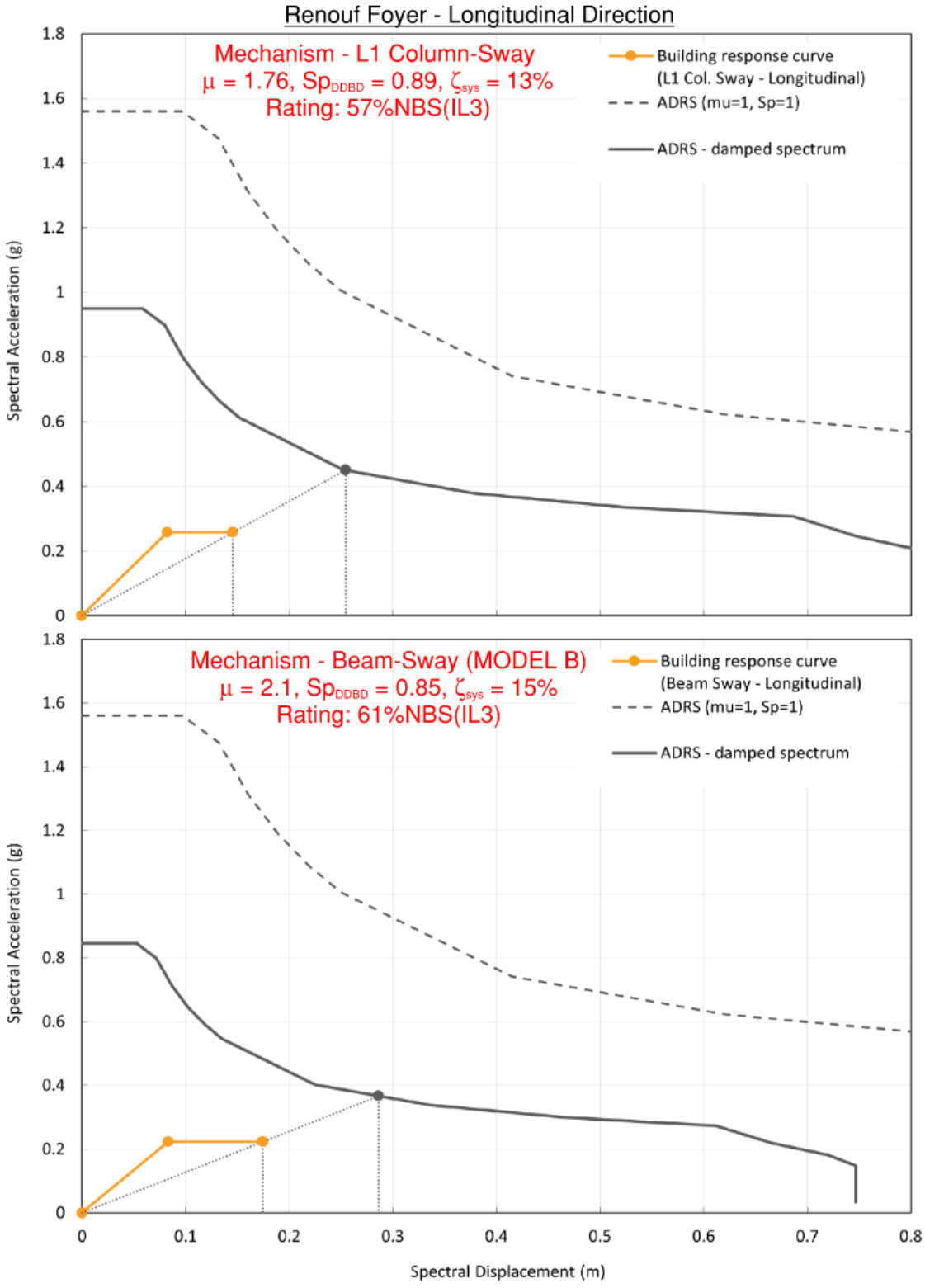

Figure E.8-7: Plot of ADRS and capacity curve associated with potentially critical mechanisms in the longitudinal direction.

E.8.3 Transverse Direction Mechanism Due to Seismic Pounding

Seismic pounding is assessed in accordance with Appendix C2B of the Engineering Assessment Guidelines. Inspection of the structures on site indicates possible pounding vulnerability along gridline 7 with the Auditorium. This is considered for the Renouf Foyer Structure in accordance with 'Scenario 3' of Table C2B.1 of the Engineering Assessment Guidelines, 'aligned floors but with building height difference'.

The floors are separated at Level 2 with a 30 mm seismic gap, however, initial investigations showed that this is likely to be exceeded considering elastic displacement of the Renouf Foyer alone. The following steps were taken to consider seismic pounding:

- Truncate the response of the Renouf Foyer at Level 2 assuming the Auditorium restrains displacements. Due to the relatively low seismic weight at the roof level (<20% of total seismic weight) a force-based assessment was carried out using 'Parts & Components' loading, as per Chapter 8 of NZS1170.5:2004 to also capture possible higher mode effects.
- Consider additional storey shear transferred in the Auditorium from the Renouf Foyer (20% of the storey shear).

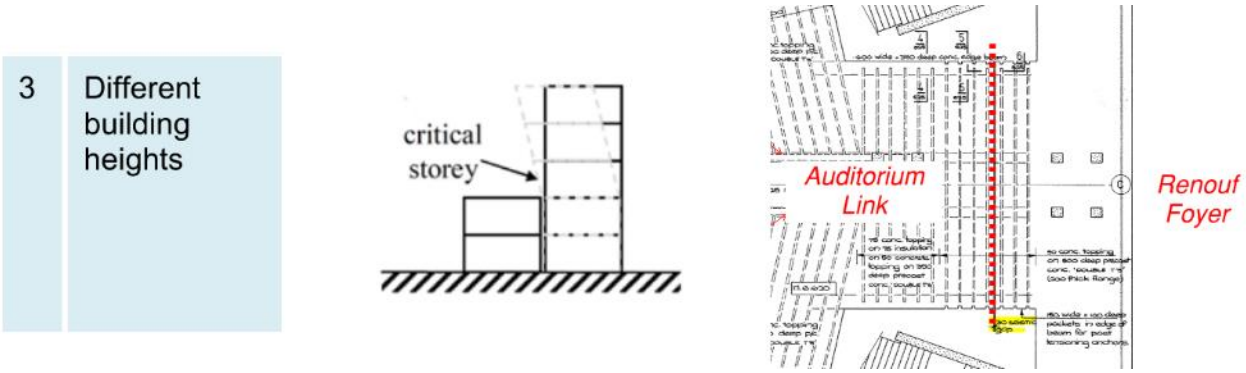

Figure E.8-8: Location of potential pounding (right) and seismic pounding scenario from Table 2B.1 (left).

The force-based approach (FBA) procedure used is summarised below:

- Elastic Microstran model laterally restrained at Level 2 to 'truncate' the structure with storey forces applied at roof level to determine the base shear capacity & roof displacement at yield (on average).
- The displacement capacity associated with each mechanism assessed using the same methods as discussed in Section E.7.
- System ductility determined and used to compare base shear capacity above Level 2 to 'Parts Components' loading.

The seismic score was determined using the equation below as:

$$\%NBS = \frac{V_{y,L2}}{F_{ph}}$$

Where; $V_{y,L2}$ =The base shear capacity associated with a mechanism above Level 2, and F_{ph} = the horizontal design earthquake actions on a part (Eq. 8.5(1)). The acceleration of the 'part' is determined in accordance with NZS1170.5:2004 is summarised below:

Site Hazard Coefficient: C(0) = 0.58

Height of Attachment: h_i = 4.62 m

Height of Structure: h_n = 4.62 m

Spectral Shape Coefficient: $C_i(T_p)$ = 2.0 (Assumed T_p < 0.75 s)

Part Risk Factor: R_p = 1.0

Roof Seismic Weight: W_{roof} = 2400 kN

The possible mechanisms in identified from the Microstran model are show in Figure E.8-9, with the results summarised in Table E.8-4 below. It was found a mechanism above Level 2 assessed following the FBA procedure resulted in an seismic score of **83%NBS$_{IL3}$.**

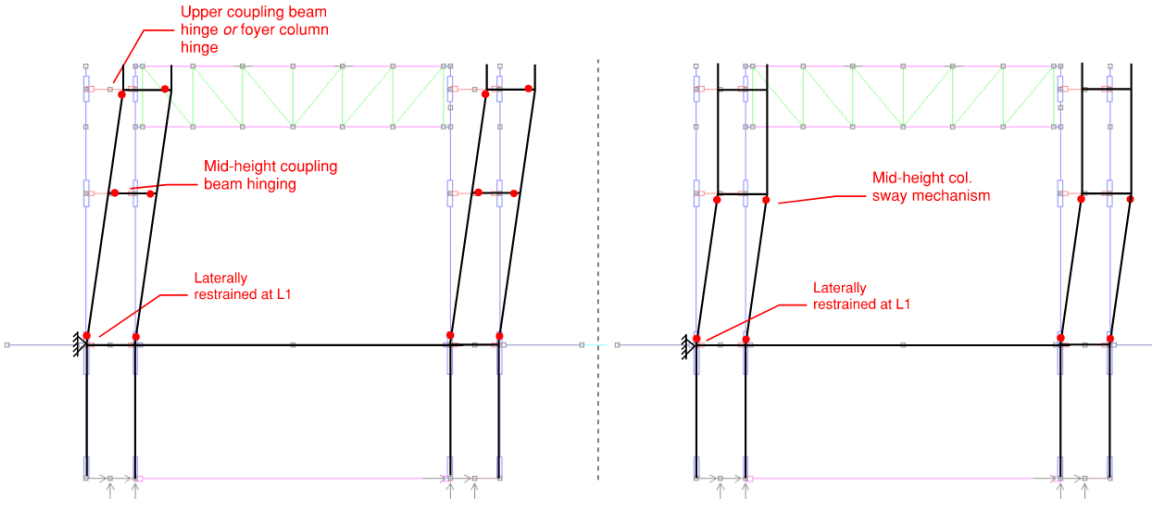

Figure E.8-9: Possible mechanisms above Level 2 in the transverse direction.

Table E.8-4: Summary of FBA procedure considering possible mechanism above Level 2 in the transverse direction.

Property	Transverse Direction[1]
Yield Base Shear Capacity, $V_{y,L2}$ [kN]	1130
Yield Roof Displacement, Δ_y [mm]	36
Ultimate Roof Displacement, Δ_{cap} [mm]	88.2 (Mid-Height Col. Sway Critical)
System Ductility Capacity, μ_{sys}	2.5
Part Response Factor, C_{ph}	0.55
Horz. Design Coefficient, $C_p(T_p)$	2.05
ULS Design Action, F_{ph} [kN]	1355
%NBS$_{IL\#}$	**83%**

[1]*Consider critical central frames which support half total weight of roof (W_p = 1200 kN).*

E.8.4 Secondary Mechanism Above Level 2 in Longitudinal Direction

A possible mechanism above Level 2 in the longitudinal direction was identified when assessing the yield hierarchy as discussed in E.6.1. The mechanism above Level 2 was assessed considering Parts & Components loading following the same method discussed above for the transverse direction. The assumed inelastic mechanism is shown in Figure E.8-10 below with the results summarised in Figure E.8-10 below. This resulted in an seismic score of **64%NBS$_{IL3}$**.

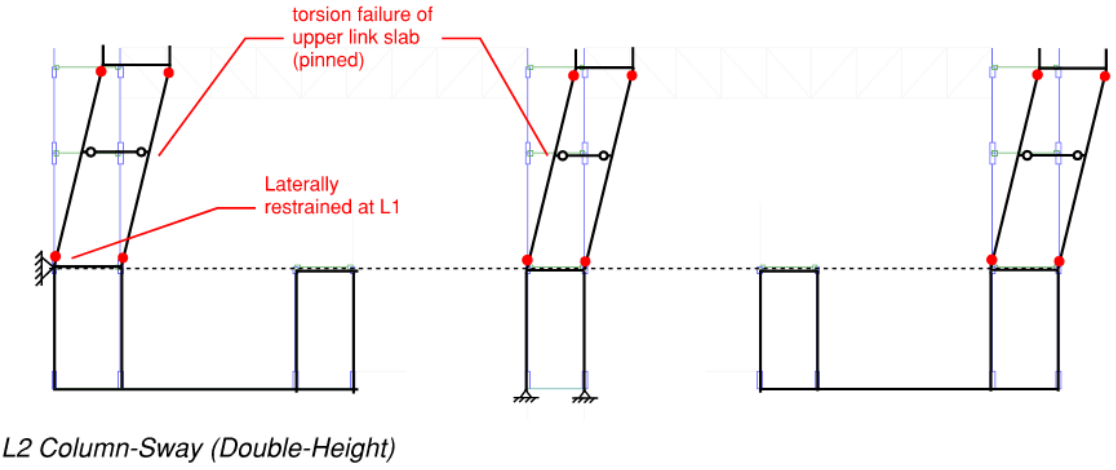

Figure E.8-10: Possible mechanism above Level 2 in the longitudinal direction.

Table E.8-5: Summary of FBA procedure considering possible mechanism above Level 2 in the longitudinal direction.

Property	Longitudinal Direction
Yield Base Shear Capacity, $V_{y,L2}$ [kN]	1720
Yield Roof Displacement, Δ_y [mm]	98.6
Ultimate Roof Displacement, Δ_{cap} [mm]	192
System Ductility Capacity, μ_{sys}	1.95
Part Response Factor, C_{ph}	0.55
Horz. Design Coefficient, $C_p(T_p)$	2.05
ULS Design Action, F_{ph} [kN]	2710
%NBS$_{IL\#}$	**64%**

E.8.5 Secondary Structural Elements

E.8.5.1 Kitchen Extension – Stitch Joint

The kitchen extension frames were not considered as part of the primary lateral system in the transverse (X) direction, as discussed in Section E.7. The frames are however still checked for a possible structural weakness at the stitch joint which connects the existing cantilever beams of the Renouf Foyer to the kitchen extension beams. The stitch joint is shown in Figure E.8-11, where 2-D20 epoxied bars (top and bottom) provide flexural capacity.

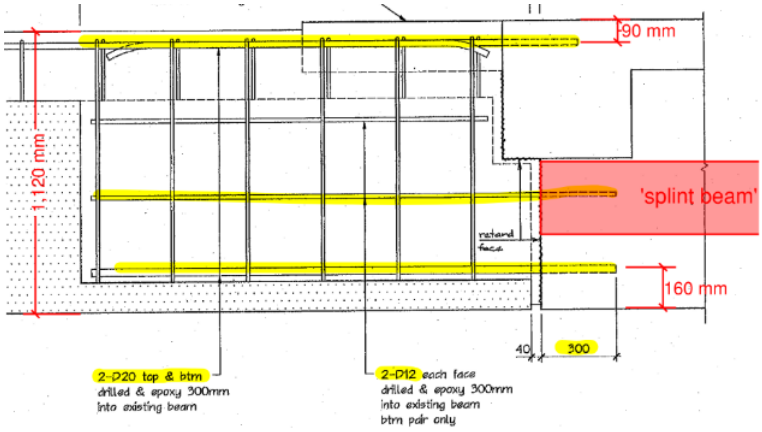

Figure E.8-11: Stitch joint detail connecting new kitchen extension beams to existing foyer.

The stitch joint was assessed as follows:

1. Determine the lateral displacement at which the capacity of the stitch joint is reached under combined seismic + gravity loads (critical in central frames).
2. Determine the inelastic displacement capacity associated with localised exceedance of the capacities which may cause a potential loss of gravity support.
3. Assess the seismic score.

The capacity of either the top or bottom 2-D20 bars was found to be sufficient to transfer gravity shears through dowel action. The capacity was therefore assumed to be limited by the inelastic rotation capacity associated with a single crack across the stitch joint. This was to consider potential load-reversal where the top or bottom bars previously in tension could act in shear through dowel action after reaching their strain capacity. It was noted that pull-out of the epoxied bars could reduce the fixity of the connection, increasing the displacement capacity. This was initially investigated but due to the unknown capacity of the fixing, it was considered conservative to assume the bars yield and the connection is limited by their strain capacity.

The capacity was critical when displacing southwards, as the seismic demands added to the existing gravity demands. It was found that due to the high gravity demands (> 50% of probable capacity), the flexural capacity of the stitch joint is reached at a Level 2 displacement of only 5 mm. The inelastic rotation capacity was determined assuming deformations localise at a single crack, in accordance with Section C5.5.3.4 of the Engineering Assessment Guidelines. The Level 2 displacement capacity was determined considering the inelastic mechanism as shown in Figure E.8-12. This resulted in an ultimate displacement capacity at Level 2 of 64 mm.

The displacement capacity associated with the exceedance of the capacity of the stitch joint could be input into the bi-linear pushover curves in the transverse direction to determine an seismic score. The displacement capacity associated with the stich joint could be incorporated into the capacity curve associated with the Level 2 beam-column joint-shear mechanism (critical as has lowest hysteretic damping). The ADRS and capacity curves considering the exceedance of the capacity at the stich joint are shown in Figure E.8-13 below. The stitch joint has an ultimate displacement demand of 64 mm, resulting in an seismic score of **54%NBS$_{IL3}$**.

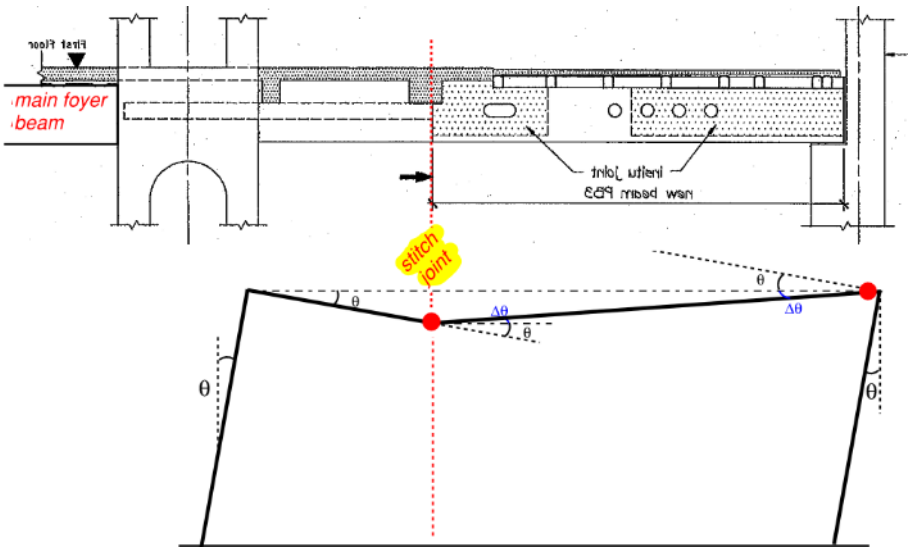

Figure E.8-12: Assumed mechanism associated with exceedance of the capacity of the stitch joint in the kitchen extension.

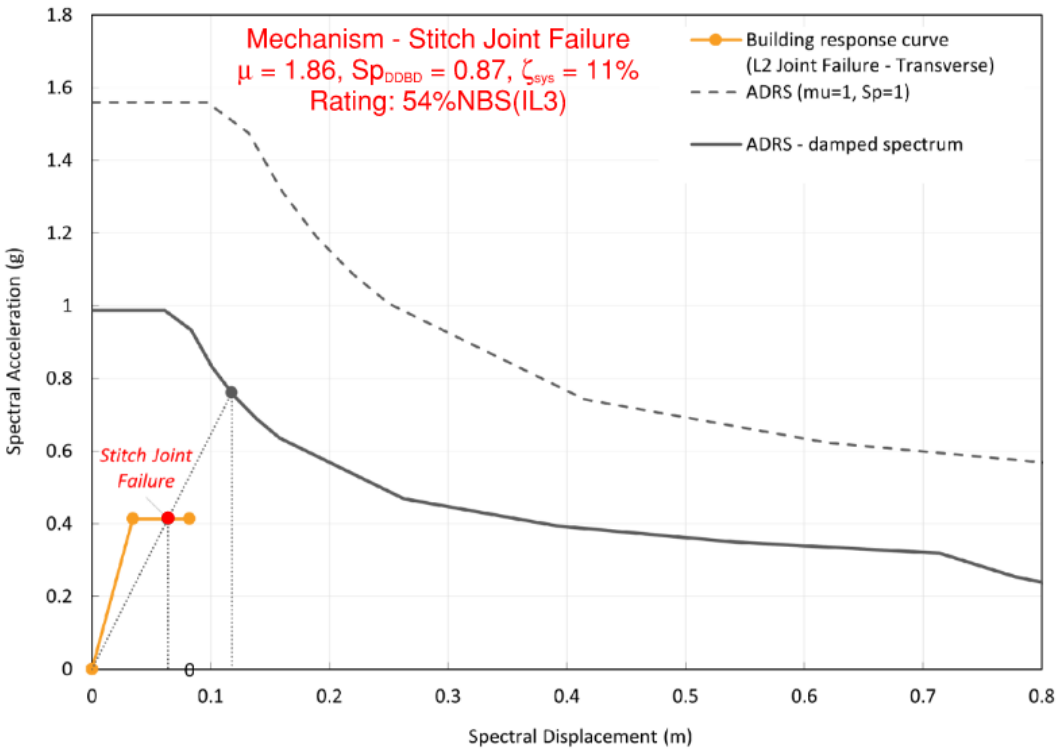

Figure E.8-13: Plot of ADRS and capacity curve associated exceedance of the capacity of the stitch joint in the kitchen extension (considering a Level 2 beam-column joint mechanism in coupled column units)- transverse direction.

E.8.5.2 Kitchen Extension – Hanger Connection

The end frames of the kitchen extension use hanger connections to reduce the rotational fixity in the beam-column joint as shown in Figure E.8-14. This resulted in the decision to exclude the end frames as part of the primary lateral system in the transverse direction. The pinned beam connection was, however, identified as a potential structural weakness due to possible displacement compatibility issues. The hanger connection was assessed considering:

- Transverse displacement capacity – specifically considering the case where hanger bars yield (under hogging moment), leading to inelastic deformations localising at a single crack at the end of the beam.
- Longitudinal displacement capacity – check whether restraint from the Level 2 floor slab as the structure displaces longitudinally will result in high compatibility forces developing in hanger bars under torsion loads.

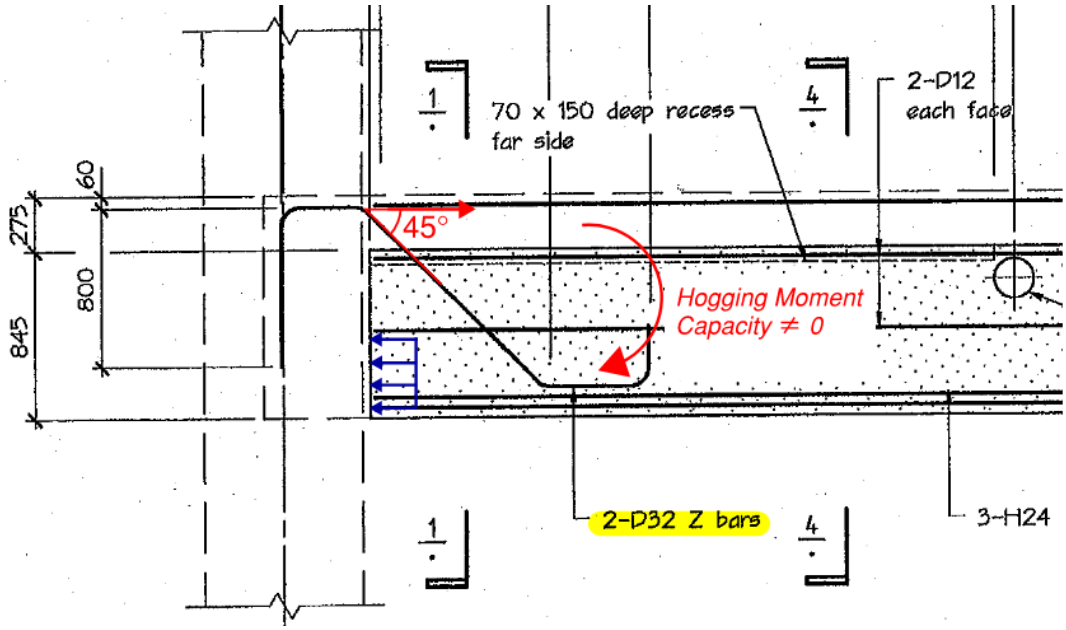

Figure E.8-14:Slotted beam connection used in kitchen extension end frames.

Transverse Direction

The hanger beam connection should be free to rotate as the structure displaces north, but hogging moments will be induced as there appears to be no slot between the beam and column. The moment capacity and subsequent yield displacement at Level 2 of the hanger connection could be determined from the elastic Microstran model. It was found that the hanger connection had a probable hogging moment capacity of 350 kNm and reached its capacity at a displacement of 14 mm.

The inelastic rotation capacity of the hanger connection after yield was assessed assuming deformations localised at a single crack, in accordance with Section C5.5.3.4 of the Engineering Assessment Guidelines. This resulted in an allowable bar elongation of 23 mm (horizontal component), corresponding to a total displacement capacity at Level 2 of 113 mm. The ultimate displacement capacity was input into the global force-displacement response (ignoring the displacement limit of the primary structure) as shown for the stitch joint in Section E.8.5.1. This resulted in an seismic score of 79%NBS_{IL3}.

Longitudinal Direction

The hanger beam connection has limited torsional capacity due to the small moment arm between hanger bars. This could result in yielding of the bars due to compatibility forces from the slab as the structure displaces longitudinally. The capacity of the hanger bars was calculated using the following equation:

$$N^*_{hanger} = \left[\frac{V^*}{x} + \frac{T^*}{jd_{hanger}}\right] \cdot \frac{1}{\sin(\theta)}$$

Where; N^*_{hanger} = axial demand on the hanger bar, T^* =torsion demand at column face, jd_{hanger} =distance between outer hanger bars, V^* =shear demand at column face, x = no. of hanger bars, and θ = angle of hanger bars relative to horizontal.

By estimating the rotational stiffness of the slab spanning between kitchen extension frames, the torsion demands at a given drift (i.e. twist of the kitchen extension beam) could be estimated. The case with the maximum torsion demand is shown in Figure E.8-15, where full restraint was assumed between the kitchen extension frames.

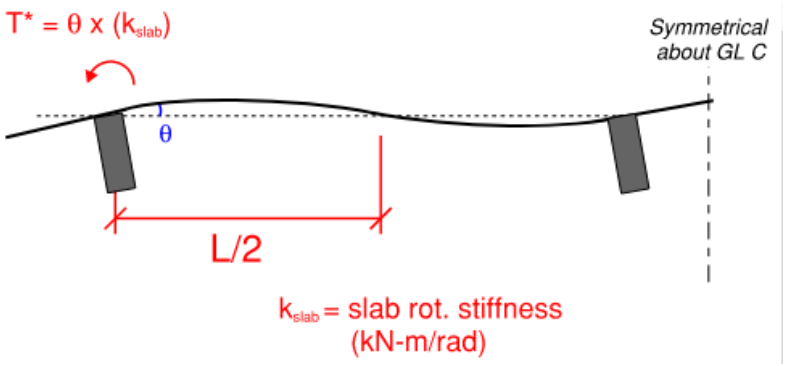

Figure E.8-15: Torsion demand mechanism on end frames of kitchen extension due to restraint from Level 2 slab.

It was estimated that the hanger bars will yield at a storey drift of 0.15-0.25%. The inelastic rotation capacity of the hanger connection was calculated considering deformations localised at a single crack in accordance with C5.5.3.4 of the Engineering assessment guidelines. This inelastic bar elongation capacity was conservatively estimated as 9.7 mm. Considering a distance between hanger bars of, $j_{d,hanger}$ = 140 mm, this corresponds to an inelastic drift capacity of over 5.0%. This was considered reasonable as the small distance between hanger bars means that the strain demands due to torsion in the hanger bars will not be critical as the structure displaces longitudinally. This resulted in an seismic score of >100%$NBS_{IL3.}$

E.8.5.3 Roof Truss Assessment

The roof trusses in both directions consist of SHS sections for all chords, verticals and diagonals with sections ranging from 50x4.0SHS to 100x4.0SHS. Transverse roof trusses are connected to the double-height frames. Longitudinal trusses are connected to the transverse trusses via slotted bolted connections. The roof truss system was assessed using two methods:

1. Force-Based Assessment: The capacity of the top connection to the coupled column units was assessed against inertia demands in both directions, determined considering 'Parts & Components' loading in accordance with Chapter 8 of NZS1170.5:2004.
2. Displacement-Based Assessment: The displacement capacity of the transverse trusses was determined and compared with the global force-displacement response to assess the roof truss system following the DDBD procedure.

Key assumptions made as part of this assessment are as follows:

- The displacement compatibility of the longitudinal trusses will not be critical as; (1) structural details show slotted hole connections, (2) the trusses are connected to the transverse trusses which will be flexible out-of-plane , and (3) the truss spans are much longer compared to the transverse frames.
- The capacity of the bottom connection of the transverse trusses to the coupled column units could not be readily assessed based on the structural drawings or from the visual observations from the site inspections. Based on the structural drawings, there appears to be little or no lateral fixity, allowing for the bottom chord of the transverse trusses to move away from the coupled column units.

When considering inertial demands, it was conservatively assumed all demands were resisted through the more robust top connections (in both directions).

Force-Based Assessment

- The parts loading used is equivalent to that shown in Section E.8.3 ($C_p(T_p)$ = 2.05); However, the design actions consider the reduced part weight of W_p = 1460 kN as the self-weight of the RC superstructure does not contribute to the inertial demand on the roof truss connections. The force-based assessment in both directions is summarised in Table E.8-6, resulting in an seismic score of **60%NBS$_{IL3}$**.

Table E.8-6: Summary of FBA of Renouf Foyer roof trusses.

Property	Transverse Direction	Longitudinal Direction
Horizontal Design Coefficient, $C_p(T_p)$	2.05	2.05
Mechanism	Lack of anchor embedment	Double curvature bending of plates
Top Connection Capacity [kN]	283	191
No. Connections	8	8
Assumed connection ductility, μ_p	1.25	1.25
Part Response Factor, C_{ph}	0.85	0.85
Connection Demand, F_{ph} [kN]	2544	2544
Connection Capacity (Total) [kN]	2264	1488
%NBS$_{IL3}$	**89%**	**60%**

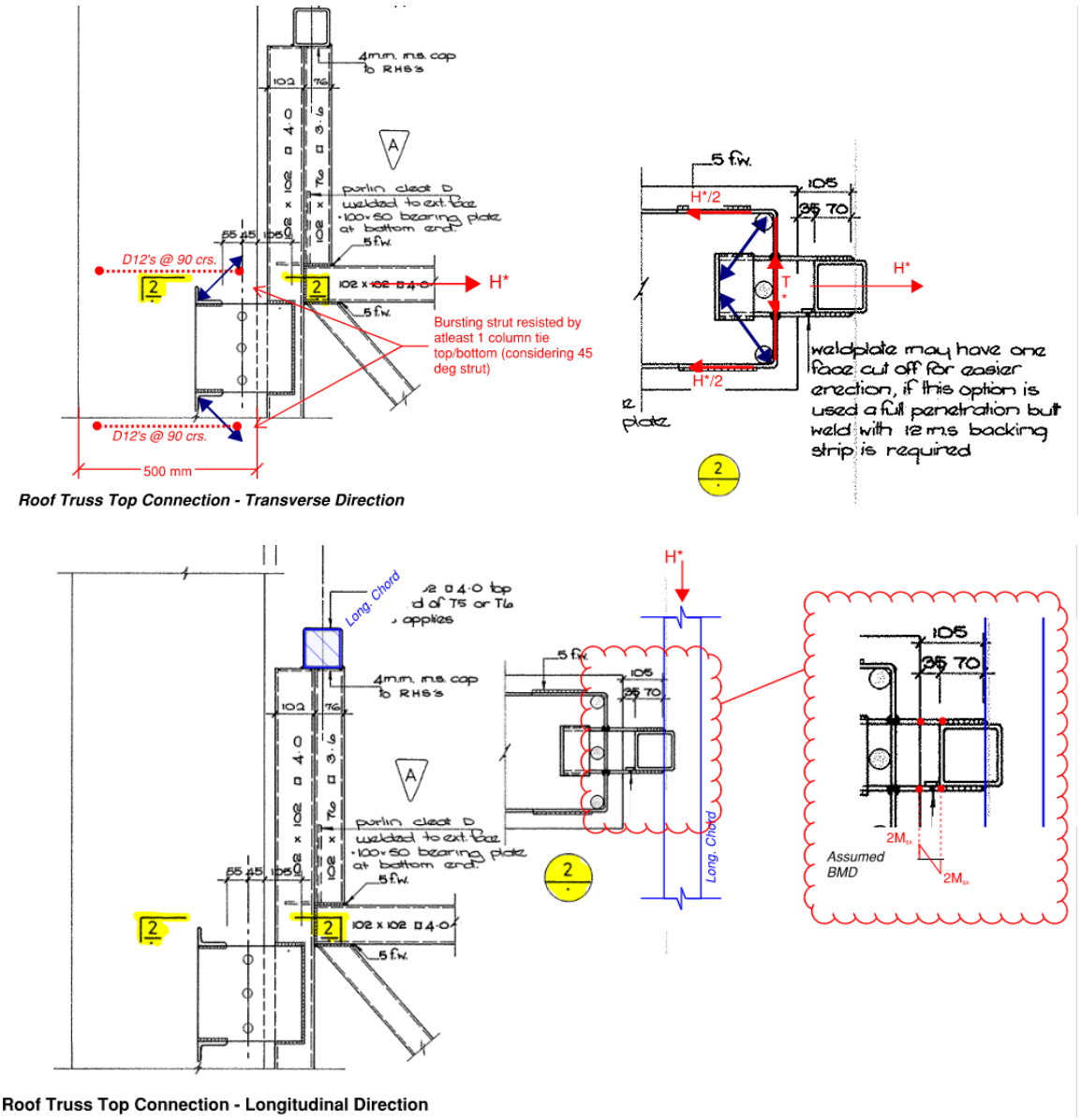

Figure E.8-16: Assumed load path of top roof truss connection resisting inertial demands under transverse (top) and longitudinal (bottom) seismic loading.

Displacement-Based Assessment

The displacement capacity of the transverse roof trusses was evaluated using an elastic truss model in Microstran V10. Gravity loads (G + ψ_EQ) in combination with imposed drifts were applied to the elastic truss model to assess the possible inelastic mechanisms that may occur within the roof trusses as the coupled column units displace laterally. In the transverse direction, drifts above Level 2 were initially ignored in the assessment of the primary structure. The Microstran model was subsequently used to find that yielding of the primary structure occurred at a roof drift of approximately 1.0%, this was used to assess the displacement capacity of the roof trusses in relation to the global force-displacement response in the transverse direction.

It was found that at a drift of approximately 0.25%, the central diagonal compression brace elastically buckles due to the imposed seismic shears. Considering potential overstrength of the central diagonal brace, the bottom chord in compression may also buckle at approximately the same time. Based on this, two potential inelastic mechanisms were identified.

1. **Truss chord hinging** - by applying a plastic fuse at the buckled diagonal compression brace in the analysis model, the post-buckling behaviour could be evaluated in accordance with Section C6.5.5 of the Engineering Assessment Guidelines. Buckling of the diagonal compression brace leads to complete loss of capacity of the central diagonal brace. This results in the top & bottom chords forming flexural hinges in the buckled bay at a drift of approximately 0.5%, shown in Figure E.8-17 below. The inelastic rotation capacity of the top and bottom chords in the buckled bay was assessed in accordance with Section C6.5.2 of the Engineering Assessment Guidelines. The truss chords were found to have a inelastic rotation capacity of 0.037 rad, corresponding to an ultimate roof drift of 1.25% before loss of capacity in the roof chords.

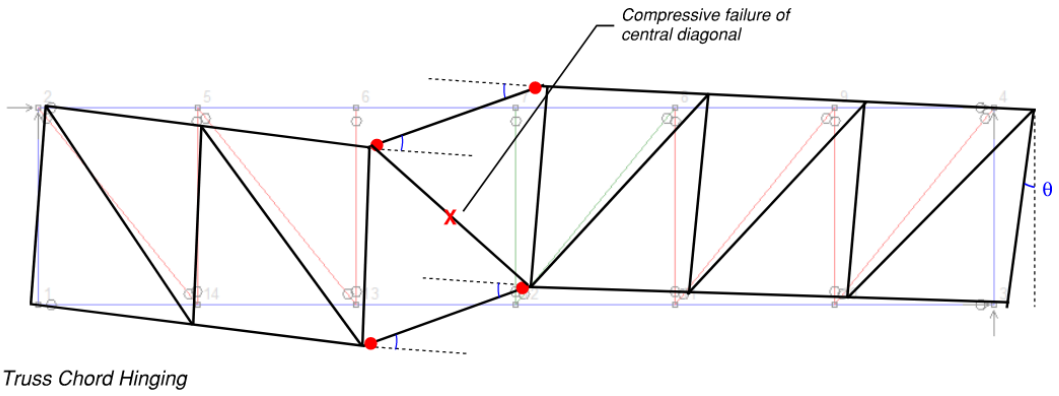

Figure E.8-17: Truss chord hinging mechanism of transverse roof trusses (occurring at 0.5% roof drift).

2. **Bottom chord buckling** - considering overstrength forces from the central diagonal compression brace, the bottom chord could buckle adjacent to the main foyer columns between longitudinal trusses connections. The bottom chord was assessed in accordance with C6.5.2 and found to have a total displacement capacity of 22.5 mm before loss of load capacity. This corresponded to a roof drift of 1.1%.

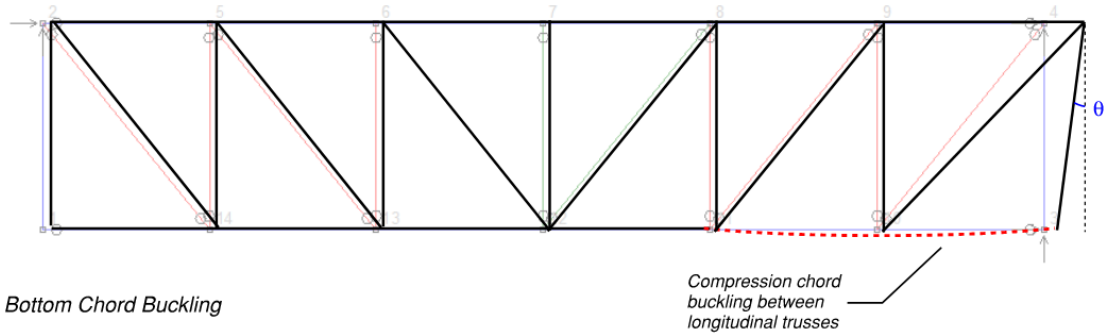

Figure E.8-18: Bottom chord hinging mechanism of transverse roof trusses.

The effective SDoF properties associated with the potential mechanisms in the roof trusses could be determined considering the critical roof drift limit of 1.1%. The displacement profile at yield of the double-height frames was extracted from the Microstran model, with a beam-sway mechanism assumed to increase drifts linearly until the limit of 1.0%. It was assumed that the roof drift limit would be critical for this assessment case. The yield, inelastic and ultimate displacement profiles corresponding to excessive transverse drifts are shown in Figure E.8-19. The effective SDoF properties shown in Figure E.8-19 and the yield base shear capacity as calculated in Section E.7 (Vb = 5975 kN), were used to extract the capacity curve shown in Figure E.8-20. The global system ductility at a roof drift of 1.1% was determined as μ=1.14, corresponding to $S_{p,DDBD}$=0.98 and ζ_{sys} = 7% which were used to define the seismic hazard. This resulted in an seismic score of **55%NBS$_{IL3}$**.

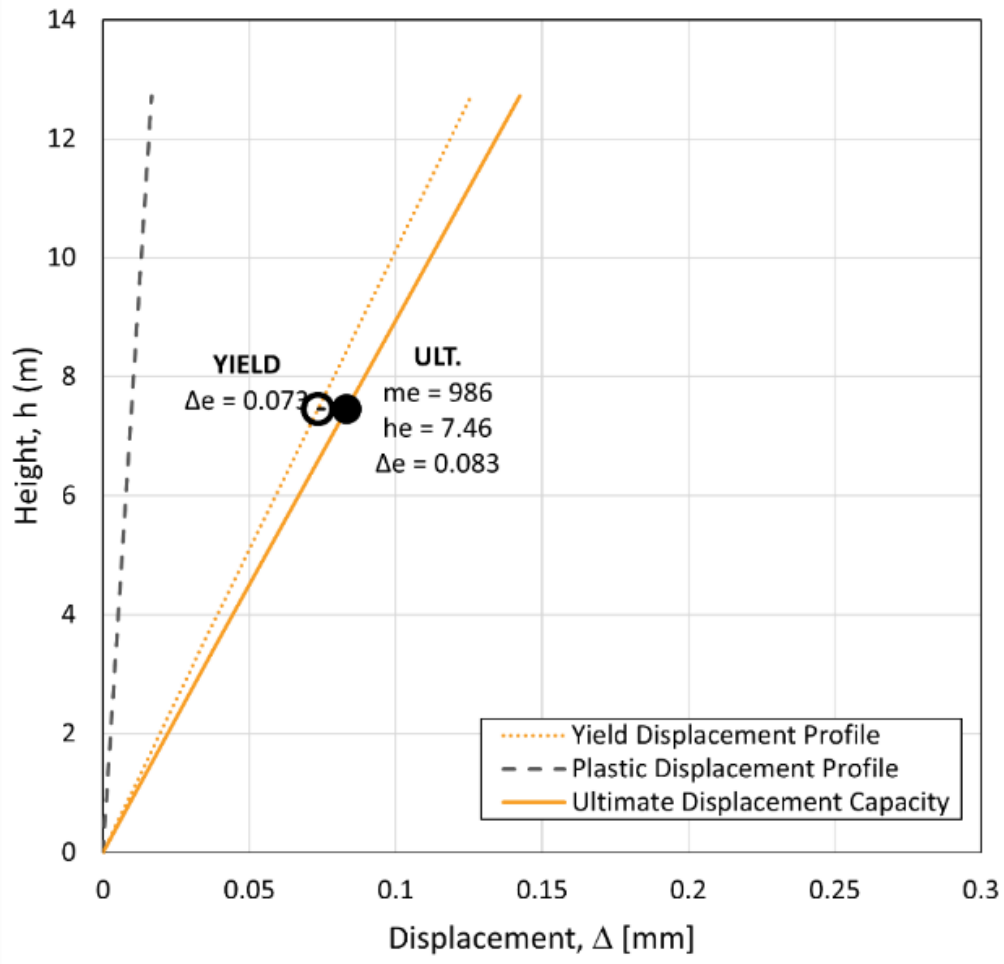

Figure E.8-19: Displacement profiles including potential mechanisms in the roof trusses

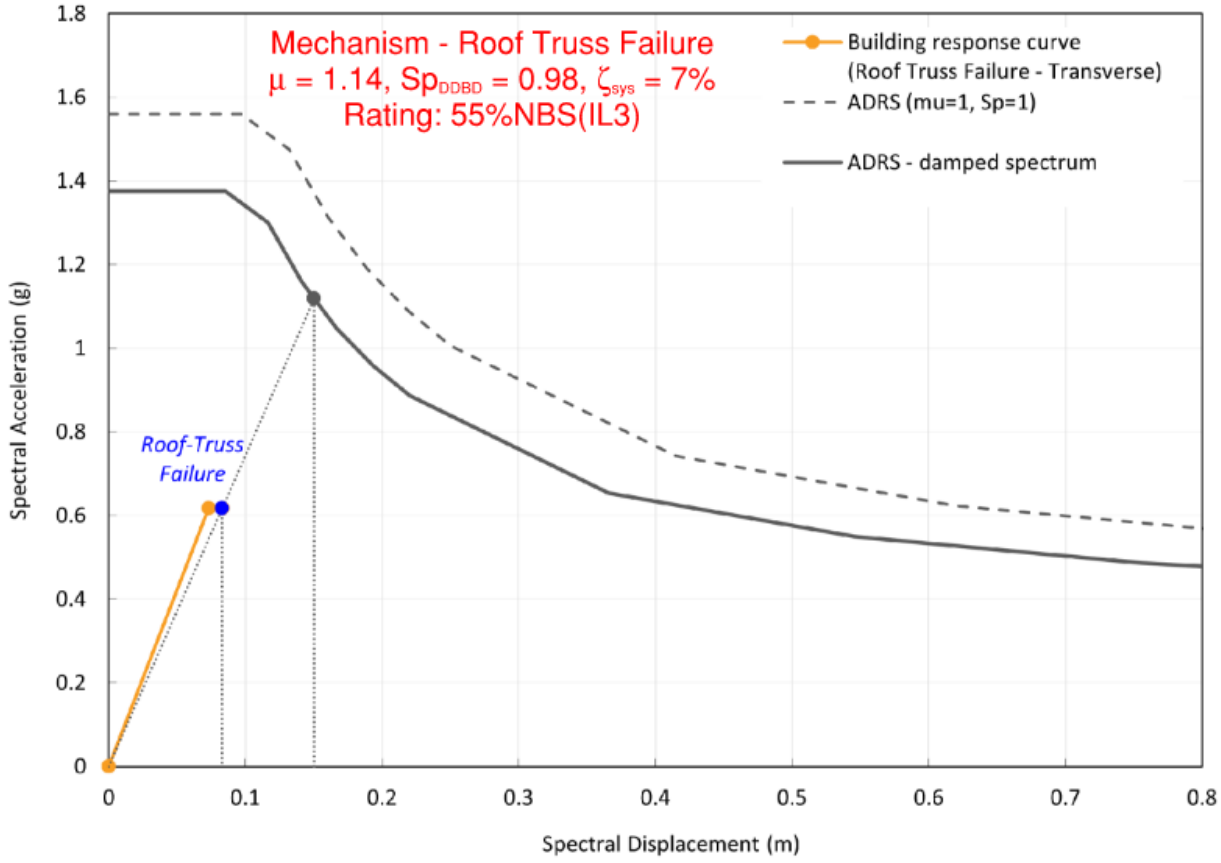

Figure E.8-20: ADRS and capacity curve corresponding to mechanisms in the roof truss

E.8.6 Diaphragm Evaluation

All the reinforcing of the diaphragm consists of 665-gauge cold-drawn (non-ductile) mesh within a 75mm concrete topping. In accordance with Section C5 of the Engineering Assessment Guidelines, cold drawn mesh was not considered to provide reliable tie capacity, therefore, only the longitudinal reinforcing in elements spanning between columns (or coupled column units) was considered to provide tie capacity to the diaphragm.

In the original Renouf Foyer design the tie forces were provided through in-situ slab edge beams running longitudinally, with the primary gravity beams running in the transverse direction between coupled column units. In the kitchen extension, tie forces were provided in the transverse direction through the primary extension beams, however, there are limited ties spanning between the frames themselves. Therefore, alternative tension load paths in the kitchen extension were being considered as discussed below.

Diaphragm demand

The seismic loading on the diaphragm at Level 2 has been determined using a pseudo-Equivalent Static Analysis (pESA) method in accordance with Section C2E.3 of the Engineering Assessment Guidelines. The ULS demands were calculated using the fundamental periods calculated in Section E.4.5. The outcome of this analysis is summarised below:

- Considering a safety factor, K_{dia} = 1.0, overstrength forces are controlling the diaphragm demands in the longitudinal (Y) direction above 63% ULS Demand.
- Floor accelerations are approximately equal under both overstrength and 100%ULS demands (K_{dia}=1.0) in the transverse (X) direction.
- For the nominally ductile upper limit forces (those which are scaled with %ULS demand), an additional safety factor (K_{dia}) of 1.5 has been incorporated for collector elements and 1.0 for all other ties and struts.
- Transfer forces from the roof structure were calculated by applying the pESA storey demands to the 3D Analysis model.

Stut and tie analysis

The diaphragm was assessed using strut and tie analysis in accordance with Section C5.6.1 of the Engineering Assessment Guidelines and Appendix A of NZS3101:2006. The column shears above and below each column, with coupled column units assessed as a single effective node were input into an elastic truss model in Microstran V10. The inertia forces were applied concurrently at the centre of mass between each frame bay to achieve equilibrium with 35-to-50-degree concrete struts distributing load out to the column groups. The column groups are then connected by orthogonal ties.

The capacity of the tie elements is summarised in Figure E.8-21 below, with the structural weaknesses scoring < 67%NBS_{IL3} discussed in more detail below.

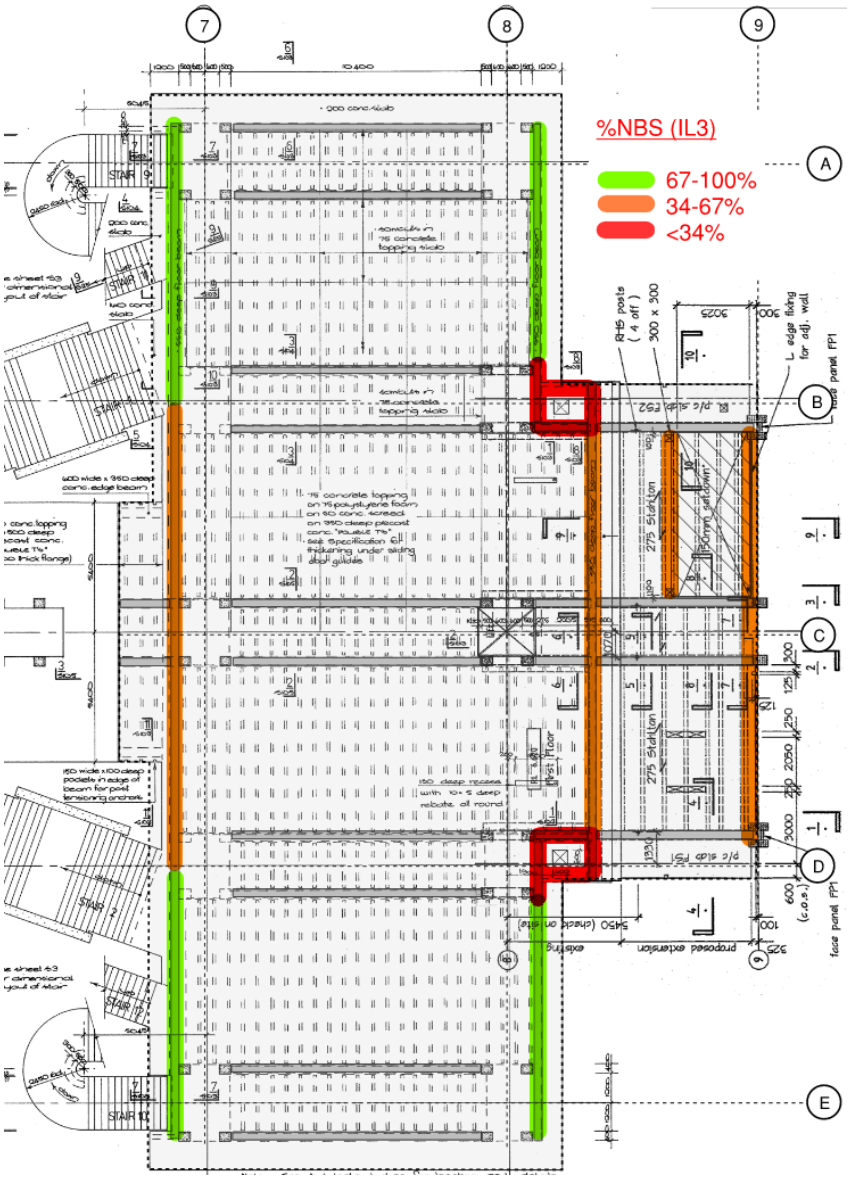

Figure E.8-21: Visual summary of ties identified as structural weaknesses in Level 2 Foyer Diaphragm.

Kitchen extension longitudinal ties

Due to the presence of cold drawn mesh and no tie beams running along gridline 9, the longitudinal tie capacity in the kitchen extension was assumed to be provided through either:

- Prestressing strands in the precast flooring ribs, which formed non-contact lap splices with H12 lap bars located over the central frames and H16 starters bar which provide hogging moment capacity to the precast flat slabs either side of the end frame (gridlines B and D).
- 100x8EA section edge detail which runs on gridline 9 between gridlines B and C along the 150mm slab set down. The edge detail showed 2-D16 starters at 1000mm centres. It was assumed that at least one set of starters at either end of the set down could act in shear-friction to transfer tie forces into the EA section.

- The assumed strut and tie load path in the kitchen extension diaphragm under transverse seismic loading is shown in Figure E.8-22. The capacity-to-demand ratios of the kitchen extension ties under the nominally ductile ULS forces ranged between 35-40% under both transverse & longitudinal demands resulting in an seismic score of 35%NBS_{IL3}.

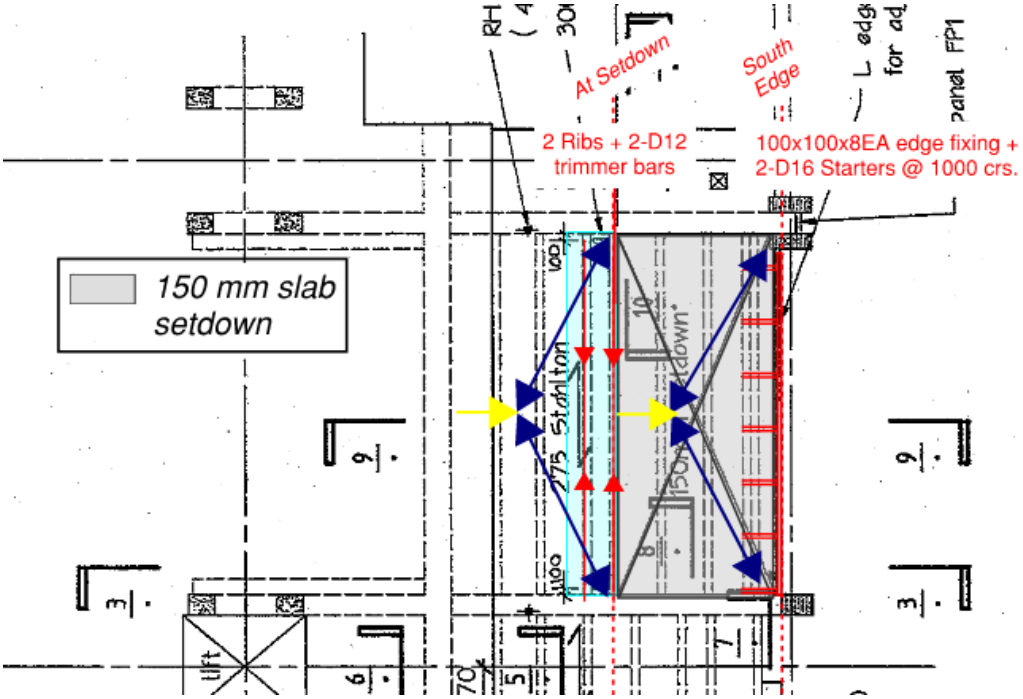

Figure E.8-22: Assumed strut & tie load path through set down in Level 2 kitchen extension diaphragm.

Main foyer longitudinal ties between gridlines B and D

Under transverse seismic demands, there were transfer forces into the Level 2 diaphragm due to the increased tributary width in the roof (double height frames). This means that the inertia load must be taken out instead by the adjacent intermediate frames and increase the tie demands running along gridlines 7 and 8 between gridlines B and D under transverse seismic loading. The capacity-to-demand ratios of the longitudinal ties between gridline B& D under the nominally ductile ULS forces ranged between 35-40% resulting in an seismic score of 35%NBS_{IL3}.

Eccentric load transfer mechanism along gridline 8

The Renouf Foyer originally had a section of floor cantilevered around 2.5m, running between gridlines B and D, adjacent to the kitchen extension on gridline 8. The perimeter slab edge beam was offset to accommodate the cantilevered floor spans. This created an eccentric load path for any tension forces required to be transferred into the adjacent intermediate frames under longitudinal seismic demands. The eccentric load path is shown in Figure E.8-23 below.

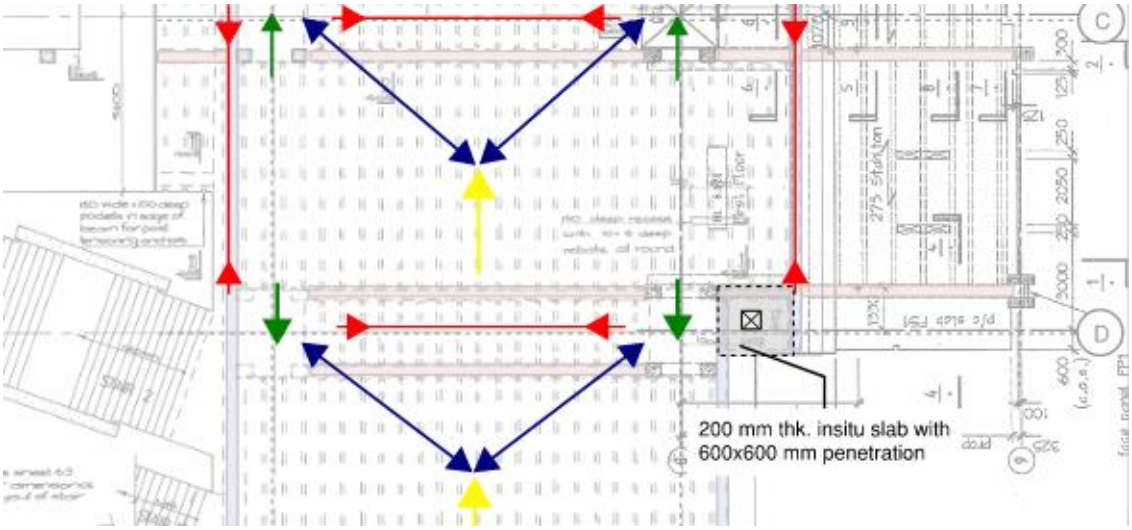

Figure E.8-23: Qualitative strut & tie analysis showing eccentric load path along gridline 8 due to offset slab edge beam.

The central double height frames support approximately half the total weight of the roof structure in addition to the self-weight of the surrounding floor slab. Lateral forces occurring due to the seismic mass under seismic shaking will largely have to be transferred through the Level 2 diaphragm to be resisted by the adjacent intermediate frames due to the following structural features:

- At the northern end of the central double height frames, there is no foundation beam running in the east-west direction. This means that the set of coupled column units are essentially free to rotate on the pile cap – ultimately, reducing the amount of lateral load they will resist.
- At the southern end of the central double height frames, there is a penetration in the floor at Level 2 for the service lift. This means the columns at the southern end see a discontinuation of 550m deep cast insitu slab which is located at the other coupled column units.

The eccentric load path was assessed by considering the shear transfer capacity of the slab around the 600x600 mm penetration through strut & tie analysis. It was found that nominal slab reinforcing (2-D12's top and bottom) either side of the slab penetration limit the tension forces that can be transferred through the cast insitu slab in this region. The exceedance of the tension capacity of the slab reinforcing would result in loss of any load transfer mechanism into the adjacent intermediate foyer columns. The predicted failure mechanism is shown in Figure E.8-24 below.

The exceedance of this tie would result in unequal displacements of the coupled column units at each end of the Renouf Foyer. For the longitudinal loading, the four coupled column units on the eastern end would see more mass than those at the western end. The diaphragm is otherwise reinforced with cold-drawn (non-ductile) mesh, and, therefore, no other ties can form. It would result in a separation (large crack) along gridline D (and gridline B upon load reversal) causing the diaphragm to lose its overall integrity.

The double tee flooring units were previously retrofitted with catch brackets; however, the original seating capacity of the units is expected to be exceeded which would lead to the flooring units dropping onto the catch brackets. This could lead to 'frame elongation' effects, where upon load reversal the flanges of the double tee units will begin to bear against the supporting beam leading to crushing of the precast units.

Due to limited capacity and lack of a redundancy in the diaphragm to accommodate such a mechanism, the diaphragm was given a seismic score of **25%NBS$_{IL3}$**.

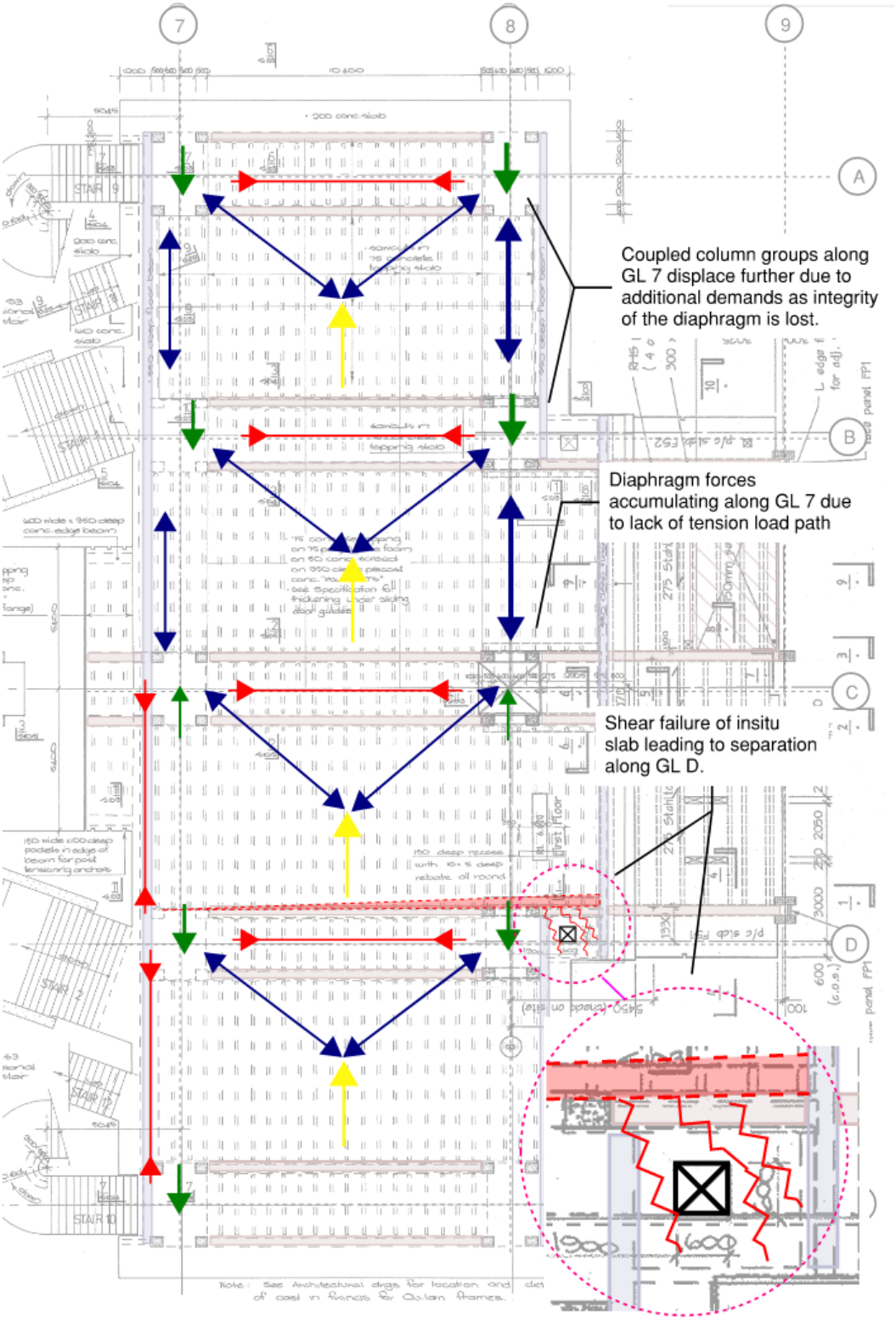

Figure E.8-24: Assumed failure mechanism of the level 2 diaphragm due to shear failure of the cast insitu slab element on gridline 7.

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E.8.7 Foundation System

Based on the geotechnical report by Tonkin+Taylor Ltd and summarised in Appendix B, widespread liquefaction of the reclamation fill and beach sands is possible at >45%ULS(IL3). Tonkin+Taylor Ltd recommended four critical scenarios should be considered to represent the different ground behaviour at various stages of possible seismic shaking, listed below.

1. Start of earthquake – no liquefaction
2. Liquefaction triggered – no lateral ground movement
3. Liquefaction leading to cyclic displacement of liquified ground (100 mm cyclic displacement expected).
4. Liquefaction leading to lateral spreading (towards end of shaking).

For geotechnical scenarios 1 to 3, the performance of the foundation system was evaluated following a displacement-based assessment procedure in accordance with Section C2.4.2 of the Engineering Assessment Guidelines. The probable capacity was compared with the superstructure capacity; however, any foundation flexibility was not included within the assessment of the superstructure. The overstrength capacity of the substructure and the superstructure could not rule out the mechanisms forming in the other.

The assessment also used the following information from Tonkin+Taylor Ltd:

- Lateral capacity of piles
- Lateral passive resistance from outside of embedded ground beams and basement substructure.
- Lesser of (1) passive lateral resistance on inside of embedded ground beams, or (2) sliding resistance due to encapsulated soil.

For geotechnical scenario 4, potentially critical scenarios which could pose a significant life safety hazard were identified and assessed against the risk of lateral spreading. Any potential geotechnical step changes due to liquefaction at >45%ULS were assessed in accordance with the recommendations by Whitehurst, Palmer and Jury (2021) to align with the intent of the Engineering Assessment Guidelines.

The foundation system is shown in Figure E.8-25 and consists of ground beams spanning in both directions with an embedded basement structure on the south side below the kitchen Extension. The foundation beams are supported on Ø560mm 'Franki' piles (9 total). The basement substructure is supported on smaller Ø450mm 'Franki' piles (9 total). In general, the lateral capacity of the foundation system in scenarios 1-3 was found to be critical in the transverse (X) direction (north-south) due to the lower passive lateral resistance from the ground beams and basement structure.

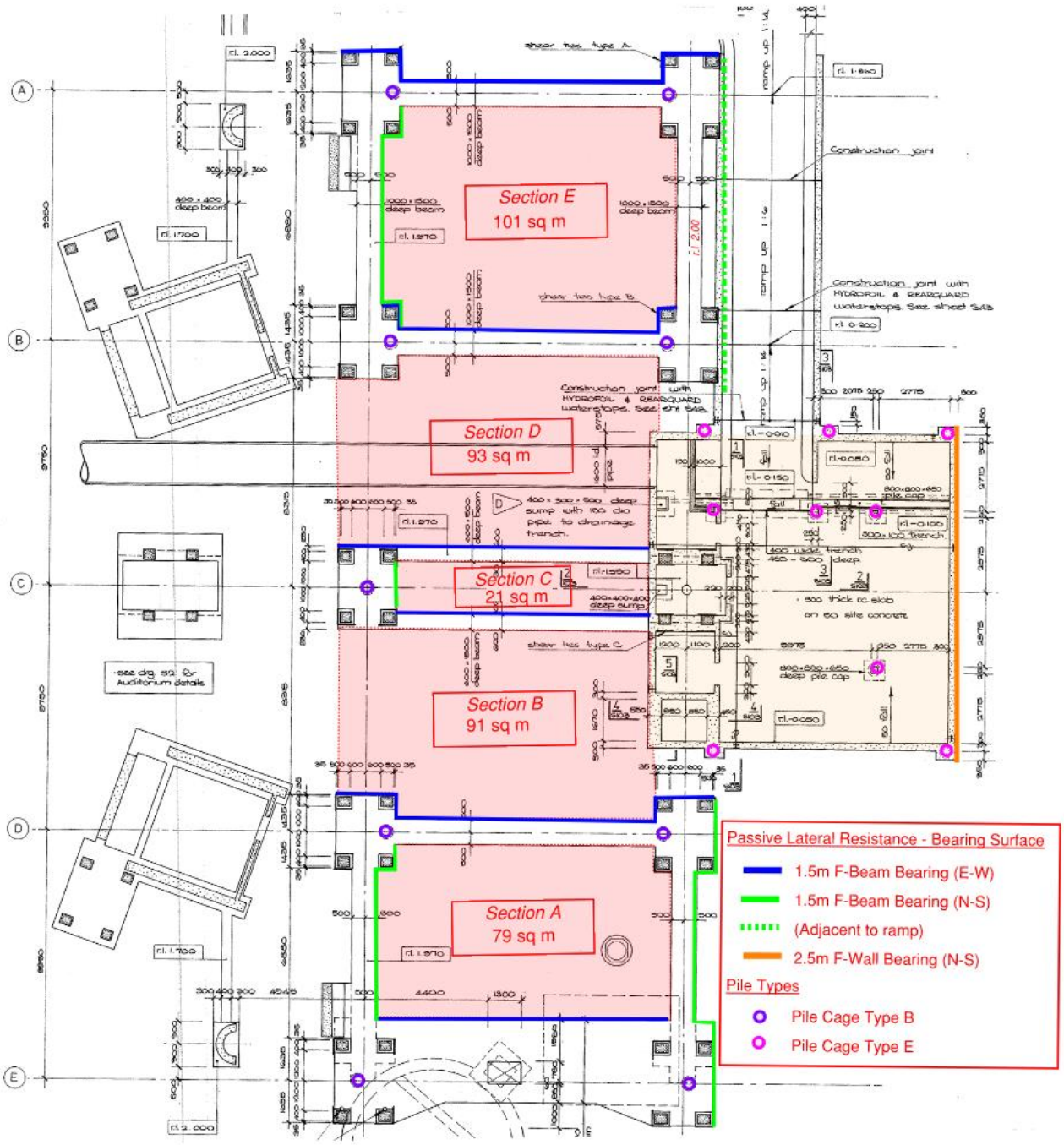

Figure E.8-25: Snapshot taken from drawing S101 of the 1980's Holmes, Wood, Poole and Johnstone drawing set - showing the Renouf Foyer foundation and pile layout.

E.8.7.1 Geotechnical Scenarios 1-3

Geotechnical scenario 1 – No Liquefaction (Start of Earthquake)

The probable resistance of the foundation system for geotechnical scenario 1 in the critical transverse (N-S) direction is shown in Figure E.8-26. The displacement capacity of the foundation system in non-liquified conditions is limited by flexure-shear failure of the piles at a displacement of 100 mm after flexural hinging occurs at the top of the piles at approximately 40 mm displacement. Global flexure-shear failure of the piles was considered to pose a life safety hazard so ULS displacements were limited 100 mm.

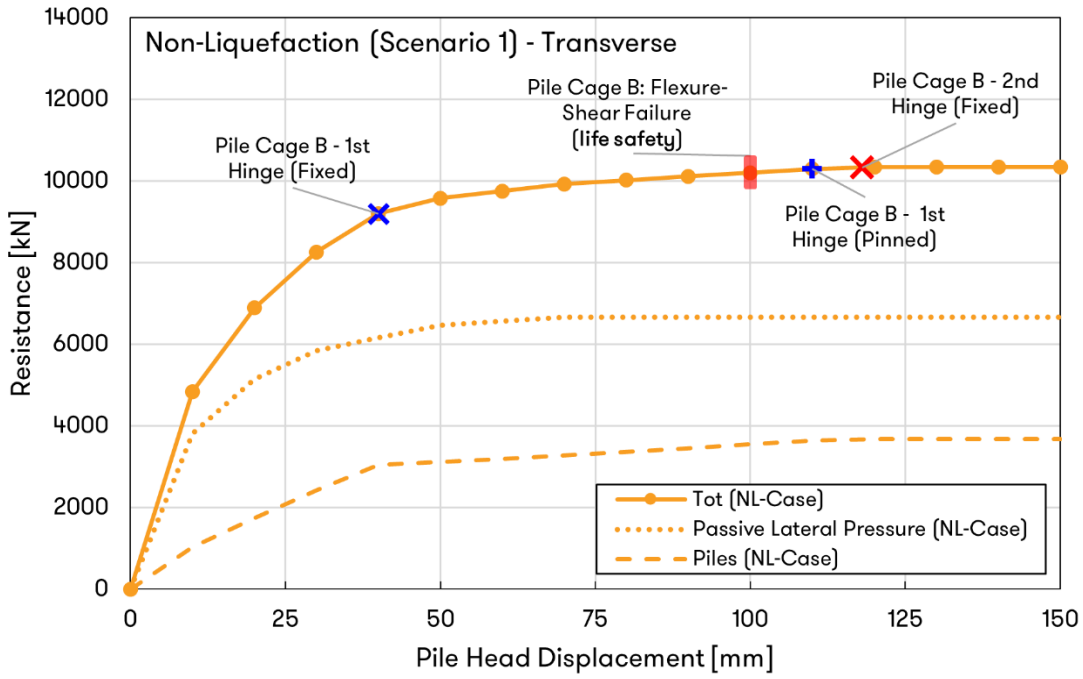

Figure E.8-26: Probable force-displacement response of Renouf Foyer foundation system considering geotechnical scenario 1 (No liquefaction - start of earthquake).

Geotechnical scenario 2 – Liquefaction (No Lateral Ground Movement)

The probable resistance of the foundation system for geotechnical scenario 2 in the critical transverse (N-S) direction is shown in Figure E.8-27. The total resistance is reduced to approximately 50% of its initial capacity in non-liquefied conditions, however, the shear demands in the piles are also reduced such that flexure-shear is not expected in the piles. It is expected the inelastic rotation capacity of the piles will be exceeded at a lateral displacement of 350 mm, such that they could pose a life safety hazard. A ULS displacement limit of 350 mm was therefore considered for geotechnical scenario 2.

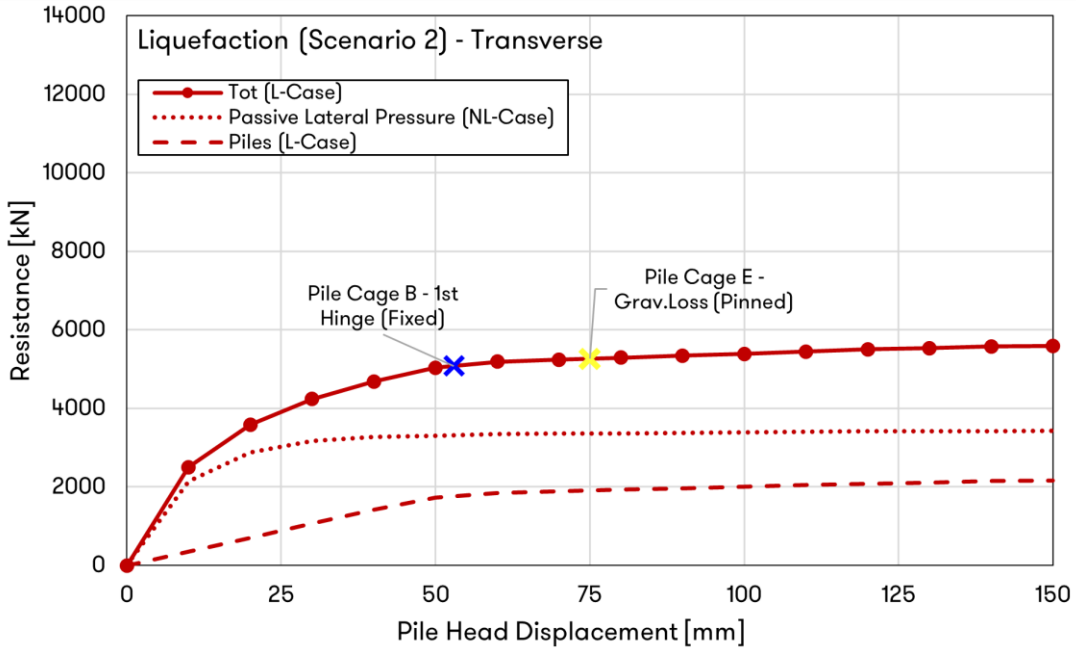

Figure E.8-27: Probable force-displacement response of Renouf Foyer foundation system considering geotechnical scenario 2 (liquefaction - no lateral ground movement).

Geotechnical scenario 3 – Liquefaction + cyclic displacements

The probable resistance of the foundation system for geotechnical scenario 3 in the critical transverse (N-S) direction is shown in Figure E.8-28. An allowance for 100 mm cyclic displacement of the liquified ground was considered following the guidance from Tonkin+Taylor Ltd. As follows:

- If the structure (or pile head) moves less than magnitude of cyclic displacement, the passive lateral resistance and sliding resistance from the foundation beams apply additional soil (kinematic) loads on the structure (in addition to base shear).
- The structure (pile head) must move the magnitude of cyclic displacement to begin to develop the passive lateral and sliding resistance of the foundation system to resist base shear loads.

No flexure-shear of the piles is expected in the liquified ground conditions. Like scenario 2, the total ULS displacement capacity of the foundation system was limited to 350 mm.

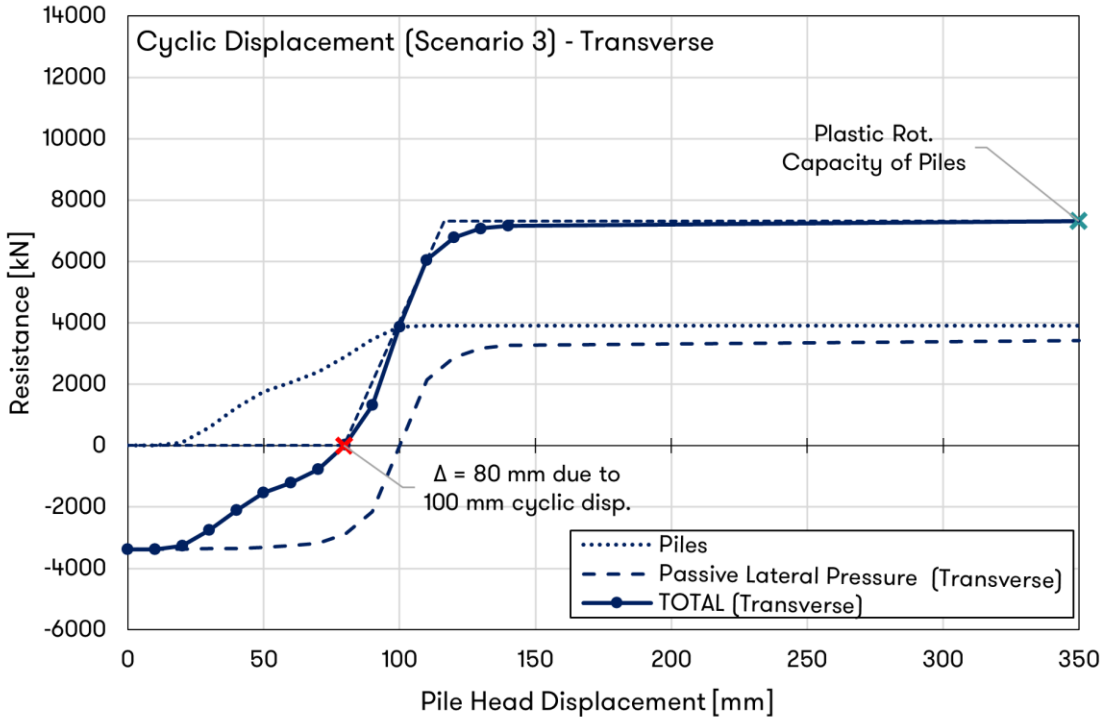

Figure E.8-28: Probable force-displacement response of the Renouf Foyer foundation system considering geotechnical scenario 3 (Liquefaction - cyclic displacement).

Comparison with Superstructure Demands

The base-shear take-out capacity for scenarios 1-3 could be compared with the probable base shear capacity of the superstructure in each principal direction to determine whether the foundation system or superstructure is likely to become inelastic first. If the superstructure was considered to become inelastic before the foundation system, the likely additional foundation displacements are also shown in Table E.8-7. It was found that the superstructure was likely to develop an inelastic mechanism before the foundation system for all cases except for in the transverse direction considering geotechnical scenarios 2 and 3.

Table E.8-7: Comparison of superstructure and foundation system probable base shear capacity for geotechnical scenarios 1-3.

Geotechnical Scenario	Direction	Probable Base Shear Capacity [kN]		Limiting System	Foundation Displacement[1] [mm]
		Superstructure	Foundations		
1	Transverse (X)	7670	10370	*Superstructure*	25
	Longitudinal (Z)	4640	10770	*Superstructure*	10
2	Transverse (X)	7670	5580	*Foundation*	-
	Longitudinal (Z)	4640	6270	*Superstructure*	25
3	Transverse (X)	7670	7320	*Foundation*	-
	Longitudinal (Z)	4640	8020	*Superstructure*	100

[1] Estimated foundation displacements only shown where superstructure expected to become inelastic first.

Foundation Response Curve

- The force-displacement response of the foundation system for each geotechnical scenario was bi-linearised and converted into spectral ordinates through the following assumptions:
- Assume the superstructure was fully rigid such that spectral displacements can be taken as the pile head displacements.
- Assume there was full mass participation such that the effective SDoF mass could be taken as the mass of the superstructure, foundations, and half the mass of the piles.
- The seismic weight of the superstructure, foundations beams and piles are shown in Table E.8-8 below.

Table E.8-8: Seismic weight breakdown of Renouf Foyer (including substructure).

Element	Weight [kN]
Superstructure	14436
Foyer Ground Beams	5800
Basement Structure	1600
Piles (1/2 Total)	210
TOTAL	**22046**

The ultimate displacement capacity of the foundation system was defined as the onset of global loss of gravity support due to excessive pile head displacements. The ultimate displacement limit of the foundation system for both the non-liquefaction and liquefaction cases is summarised in Table E.8-9 below.

Table E.8-9: Ultimate pile head displacement capacity for Renouf Foyer foundation system.

Geotechnical Scenario	Displacement Limit [mm]	Limiting Aspect	Reference
No Liquefaction (Scenario 1)	100	Flexure-shear of Pile Cage B after initial plastic hinge formation	Section C5.3.3 – Engineering Assessment Guidelines
Liquefaction (Scenario 2 & 3)	350	Exceedance of inelastic rotation capacities	2010 PoLA Seismic Code for Upgrade and Repair of Wharves

The capacity curve could be generated for each geotechnical scenario as summarised in Table E.8-10 below.

Table E.8-10: Force-displacement response in transverse direction (critical) for geotechnical scenarios 1-3.

Geotechnical Scenario	V_{prob} [kN]	Effective SdoF Properties		
		m_e [tonnes]	Δ_{cap} [mm]	S_A [g]
1	10337	2247 (100%)	100	0.469
2	5583	2247 (100%)	350	0.253
3*	7323	2247 (100%)	270	0.332

*Initial pile head displacements due to cyclic displacement are subtracted from response.

Damping

For the damping of the ADRS, reference is made to the NZTA Bridge Manual 3rd Edition and Priestley et al. (2007).

The NZTA Bridge Manual gives the equivalent viscous damping ratio's for various component actions. For 'piled foundations where hinges develop in piles' an equivalent viscous damping ratio is given in accordance with the equation below. It is related to the member (pile) ductility.

$$\xi_{i,piles} = 0.10 + 0.565 * (\frac{\mu_m - 1}{\mu_m \cdot \pi})$$

Passive pressures against the foundation beams may have a much smaller damping contribution. The building displaces the ground on the first cycle of seismic shaking and on the returning cycle the passive pressures take that initial displacement to engage. Therefore, it is our opinion that they can be expected to act with a damping of no more than the system inherent damping of ξ_o = 5%.

To combine the equivalent viscous damping contribution of the piles and the passive pressures, reference is made to equation (5-23) of the NZTA Bridge Manual. Copied below for reference.

$$\xi_e = \frac{\sum_{i=1}^{n}(V_i \cdot \delta_i \cdot \xi_i)}{\sum_{i=1}^{n}(V_i \cdot \delta_i)}$$

The ductility demand of the piles and associated system equivalent damping for each geotechnical scenario is summarised in Table E.8-10Table E.8-10 below.

Table E.8-11: System damping associated with each geotechnical scenario.

Geotechnical Scenario	Pile Yield Displacement [mm]	Displacement Limit [mm]	Pile Ductility Demand, μ_{pile}	Pile Damping ζ_{pile}	System Damping ζ_{sys}
No Liquefaction (Scenario 1)	40	100	2.5	20.8	10.6
Liquefaction (Scenario 2)	53	350	6.6	25	12.9
Liquefaction – Cyclic Displacements (Scenario 3)	115	350	3.0	22	13.0

Evaluation

The capacity curves for each geotechnical scenario are shown in Figure E.8-29 and Figure E.8-30. The seismic score could be determined in accordance with step D5 of the Engineering Assessment Guidelines (Section C2.4.2). Geotechnical scenario 1 was critical due to flexure-shear of the piles in non-liquefied soils, resulting in an seismic score of **70%NBS$_{IL3}$**.

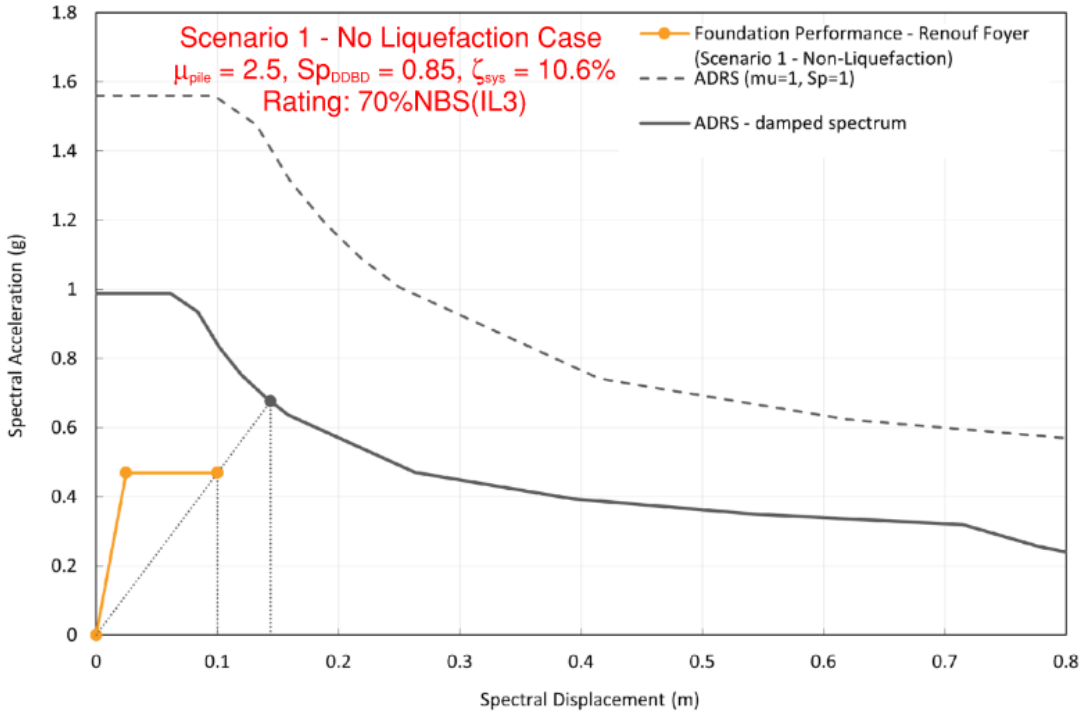

Figure E.8-29: Plot of ADRS and foundation response curve associated with geotechnical scenario 1.

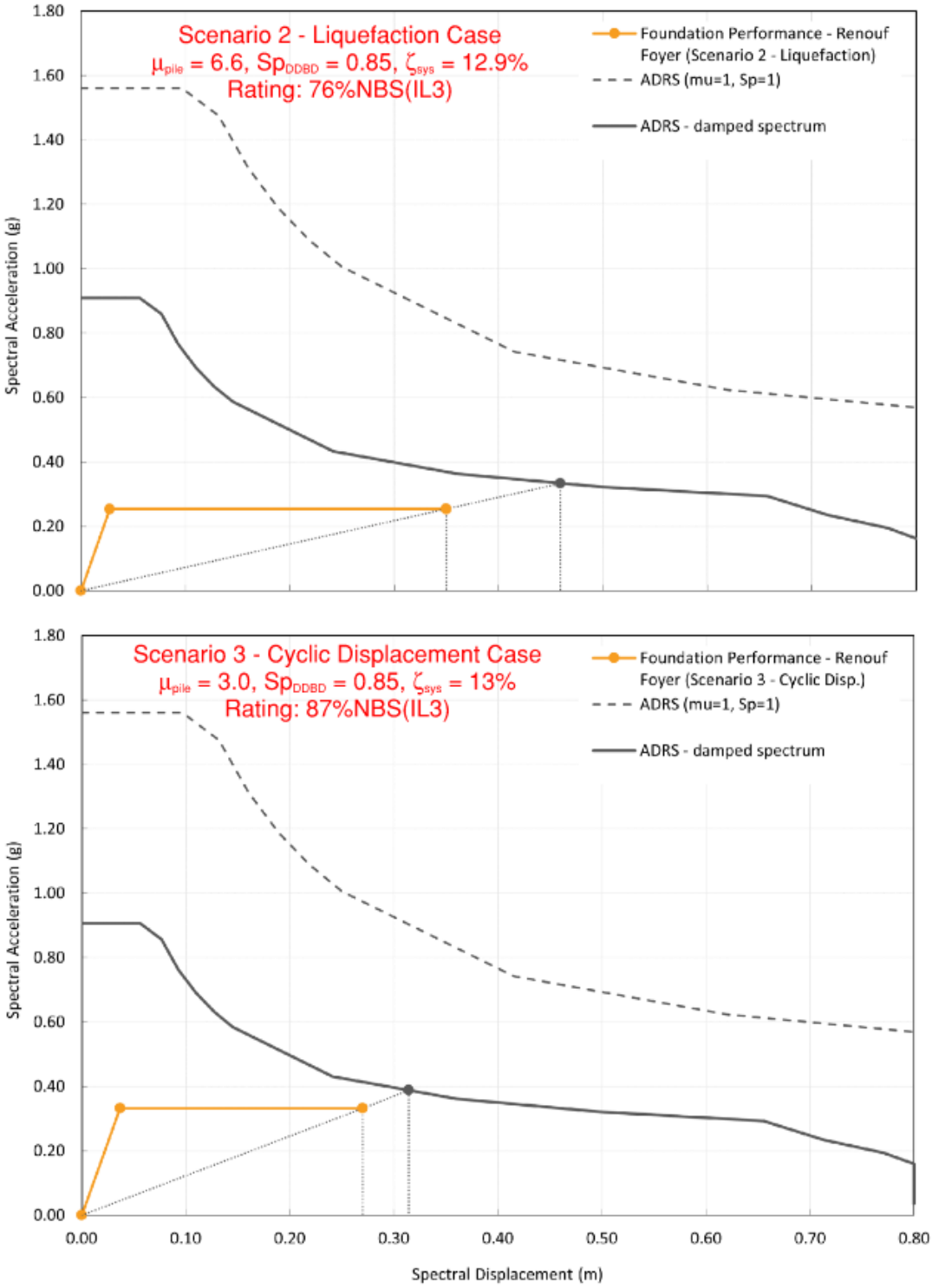

Figure E.8-30: Plot of ADRS and foundation response curve associated with geotechnical scenarios 2 and 3.

E.8.7.2 Geotechnical Scenario 4 – Liquefaction (Lateral Spreading)

For geotechnical scenario 4, liquefaction was assumed to cause lateral spreading throughout the site as the liquified soil displaces towards the Wellington harbour (for more detail, see Appendix B).

Possible ground deformations due to lateral spreading could lead to demands on the structural system. The level of ground deformations could increase rapidly if liquefaction is triggered and as such a possible geotechnical step change behaviour was considered. After discussion with Tonkin+Taylor Ltd., any possible step change behaviour due to ground deformations was assessed following the method proposed by Whitehurst, Palmer and Jury (2021), to meet the intent of Engineering Assessment Guidelines.

Two typical configurations in the Renouf Foyer were identified to be susceptible to ground deformations:

- Stair structures spanning between independent foundation systems, for which lateral spreading could impose a displacement demand on the system.
- Connected primary structure that does not have a robust ground floor diaphragm. Ground deformations may impose localised displacement demands on the structure, resulting in a significant life safety hazard.

Assessment of all stair structures identified as potentially susceptible to lateral spreading is summarised in Appendix G.

Renouf Foyer Column-Sway Mechanism

The foyer frames along gridlines A and B are connected by a series of foundation beams limiting the amount of differential ground movement able to accumulate between them but only a small section of 150 mm thick concrete slab on grade spans between these frames and the central frames on gridline C. Because of this, differential ground movement due to lateral spreading can be expected if liquefaction is triggered which must be accommodated by the superstructure, The differential movement was assumed to fully develop in the frames on gridline A and B as the central frame will be stiffer due to the restraint provided by the additional frames on gridlines D and E. The assumed mechanism forming due to lateral spreading in the coupled column units is shown in Figure E.8-31.

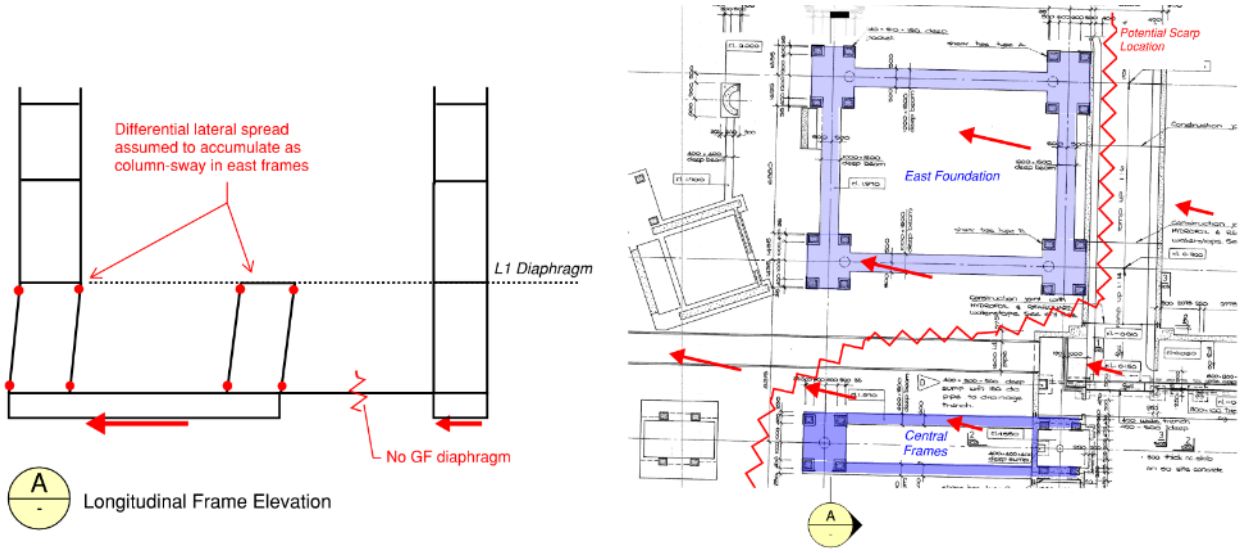

Figure E.8-31: Potential location of surface scarp due to lateral spreading that could impose displacements on the Renouf Foyer superstructure.

It was assumed that all displacements would need to be accommodated between the ground floor level and Level 2 with the diaphragm providing restraint at Level 2. In both frame directions, the displacement capacity under ULS loading was controlled by a potential column-sway mechanism (see Section E.7 for details). The lower displacement capacity under column-sway mechanism occurred in the transverse direction and was conservatively used for this assessment. This corresponded to an acceptable displacement of, δ_{SLSH} = 68 mm.

As shown in Figure E.8-32, the geotechnical step change does not need to be applied to the columns of the Renouf Foyer.

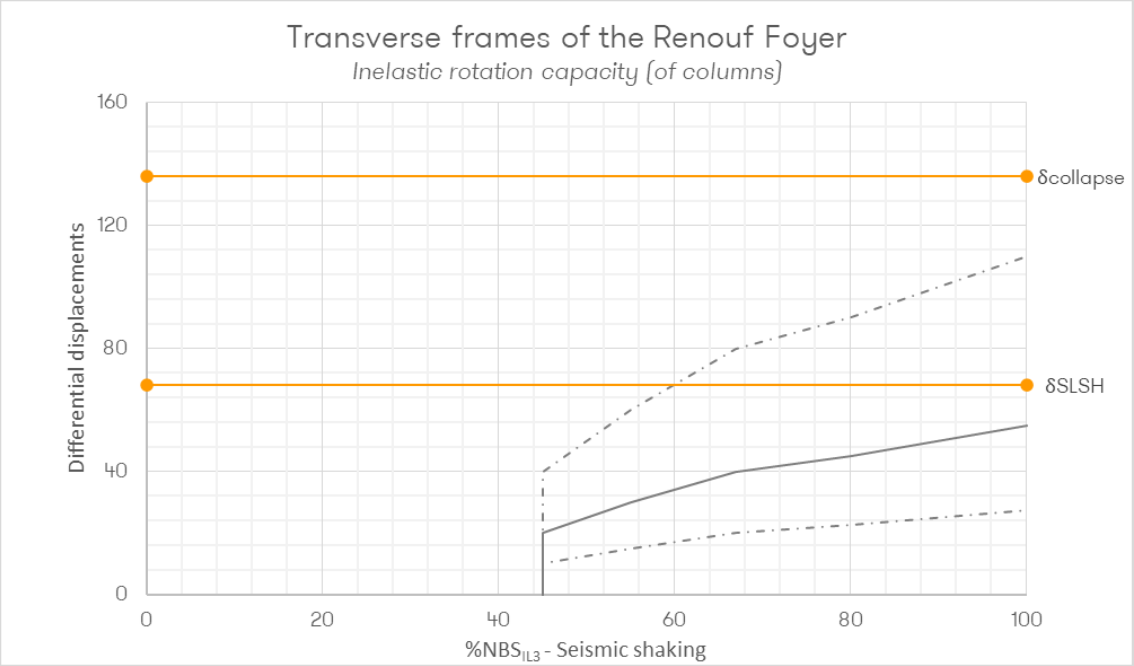

Figure E.8-32: Assessment of Renouf Foyer columns, due to differential lateral spread

E.9 SUMMARY OF IDENTIFIED STRUCTURAL WEAKNESSES

Structural Weaknesses are those deficiencies identified which have an assessed seismic rating of less than 100% NBS. These are listed in Table E.9-1 below.

Whilst there is a complex mixed mechanism of the primary lateral load resisting system, Table E.8-3 showed the performance of Renouf Foyer frames is governed by the column-sway (soft-storey) of the ground floor level columns, in both principal directions. Resulting in an seismic score of 60%NBS_{IL3}.

Table E.9-1: Seismic Rating of potential Structural Weaknesses in the Renouf Foyer.

Building Element	Structural Weakness	%NBS_{IL3}
Level 2 diaphragm	Exceedance of tie capacities in the diaphragm (665-gauge non-ductile mesh)	25
Kitchen extension stitch joint	Insufficient displacement capacity of connection between original Renouf Foyer frames and the 1989 kitchen extension	55
Roof trusses	Both imposed displacements and inertial loads limit the capacity due to: ▪ Deformation/drift compatibility of transverse frames with Renouf Foyer columns ▪ Connections of transverse trusses to coupled column units under longitudinal seismic loading.	55
Coupled column units	Exceedance of inelastic rotation capacity of columns at the base, as a soft-storey forms at the ground floor level.	60
Foundation system	Flexure-shear of piles in undegraded ground conditions	70
Kitchen extension frames	Exceedance of inelastic rotation capacity of hanger beam-column connection when displacing southwards.	80

Appendix F

Technical Summary of the Independent Staircase Structures' Analysis and Evaluation

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Appendix F Technical Summary of the Independent Staircase Structures' Analysis and Evaluation

F.1 INTRODUCTION

The Simple Lateral Mechanism Analysis (SLaMA) method is used for the complete building assessment of the primary structure of the Independent Staircase Structures to determine the seismic score. This method involves identifying the likely elastic and inelastic deformation mechanisms, and the lateral strength and displacement capacity of these mechanisms.

SLaMA is an appropriate form of assessment for the staircase structures as they have a regular lay-out and well-defined lateral systems acting in each direction. The primary walls act in parallel in-plane, and they form a single-bay, four-storey moment frame system with the internal stairs at the ground floor and the floors at Level 2 and Level 4. Based on the lack of significant vertical irregularities in the lateral systems, the overall seismic response is dominated by the fundamental vibration mode in each principal direction and higher modes are not critical. The simple layout of the frames means that any possible inelastic mechanisms could be readily identified and assessed in accordance with the Engineering Assessment Guidelines.

The SLaMA method was supplemented by a linear elastic analysis of a 2D model in Microstran V10. This was used to better capture effects not easily determined using the SLaMA method such as the displacement profile at final yield of the Independent Staircase Structures acting out-of-plane.

Once the SLaMA was completed, possible inelastic mechanisms of the Independent Staircase Structures were identified considering element overstrength, which could influence the behaviour on a local and global level.

The equilibrium method in accordance with Appendix C2A, Section C2A.2 was used to estimate the base shear capacity of the system. It considered the relative contributions from the primary stair walls, floor slabs, and the primary stairs at the ground floor level. The elastic behaviour of the system up to yield can then be combined with the inelastic displacements associated with each possible mechanism to form bi-linear pushover (force-displacement) curves associated with each possible mechanism.

The behaviour of the structures can be represented in terms of the spectral acceleration and spectral displacement of an equivalent single degree of freedom (SDoF) system. The equivalent SDoF force-displacement response, in both principal directions of the structure, is plotted on a damped horizontal acceleration-displacement-response spectrum (ADRS). This spectrum gives a representation of the ULS seismic demands in the ordinates of spectral acceleration and spectral displacement. As such, the force-displacement response can be directly compared to the damped ADRS to determine the seismic score.

F.1.1 Simplified Lateral Mechanism Analysis (SLaMA)

Primary structural elements and most secondary structural elements are considered in the SLaMA analysis. This includes all reinforced concrete walls, floors and stairs that form the primary structure of the Independent Staircase Structures.

The SLaMA method was used to develop bi-linear force-displacement curves for the lateral load resisting system in each principal direction. This was used to assess the capacity of the primary superstructure following the displacement-based assessment (DBA) procedure, as described in Section C2.4 of the Engineering Assessment Guidelines.

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For the Independent Staircase Structures, the SLaMA has considered the following step-by-step approach in both principal directions.

1. Evaluate Wall, Beam, and Column Elements

    - Assess detailing of wall, beam, and column elements and identify the use of plain bars, insufficient anchorage, etc. if applicable.
    - Determine the probable flexural capacity.
    - Determine the probable shear demand, and the shear demand associated with the development of the flexural overstrength capacity of the member.
    - Determine the probable shear capacity.
    - Determine if flexure or shear governs the members mechanism.

2. Evaluate the strength hierarchy of sub-systems

    - Evaluate the possible mechanisms for each sub-assembly considering element overstrength.
    - Evaluate the capacity of the foundation system relative to the superstructure to identify any substructure mechanisms that could impact the global system behaviour (rocking etc.).

3. Evaluate the probable base shear capacity and yield displacement profile of the structure

    - Estimate the probable base shear capacity in accordance with Appendix C2A, Section C2A.2.
    - Estimate the yield displacement of the structure.
        - In-plane: Use the moment-curvature method in accordance with Yellow Section C5.5.3.1.
        - Out-of-plane: Use the moment-curvature method in accordance with Yellow Section C5.5.3.1, supplemented by a 2D elastic Microstran V10 model.

4. Inelastic Displacement Capacity of Possible Mechanisms

    - Evaluate possible inelastic mechanisms identified during steps 1 and 2 in accordance with Yellow C5.5.3 of Engineering Assessment Guidelines.
    - In combination with yield displacement and base shear capacity determined in step 3, determine the bilinear force-displacement response and effective SDoF properties associated with each mechanism.

5. Allowance for P-Delta and Torsion

    - Determine the centre of mass relative to the centre of rigidity and make allowance for torsion in accordance the Appendix C2F of the Engineering Assessment Guidelines.
    - Make allowances for P-Delta effects in accordance with Section C2.9.4 of the Engineering Assessment Guidelines.

6. Assess Possible Seismic Pounding

    - Assess possible seismic pounding with the adjacent superstructure in accordance with Appendix C2B of the Engineering Assessment Guidelines.

7. Assess the earthquake score of the primary structure in both principal directions

    - Assess the primary structure in accordance with the direct displacement-based assessment (DDBA) procedure, as outlined in the section C2.4.2 of the Engineering Assessment Guidelines.

F.2 ANALYSIS INPUT

F.2.1 Building Description

The Independent Staircase Structures, referred to as the 'Staircase Structures' in this technical summary are four-storey reinforced concrete wall structures located along gridlines B and D between the Renouf Foyer and the Auditorium. The Stair Structures support the Stairs 1/2 spanning from the ground floor level to Level 2 into the Renouf Foyer. There are also slabs at Level 3, Level 4, and at their roof level. They have approximate floor dimensions of 5.0x5.2 m and a total height from Level 1 to their roof of 13.2 m. They are constructed from cast in-situ walls and floors. The configuration of the eastern Staircase Structure is shown in Figure F.2-1 below. Both Staircase Structures are identical structural configurations, only mirrored.

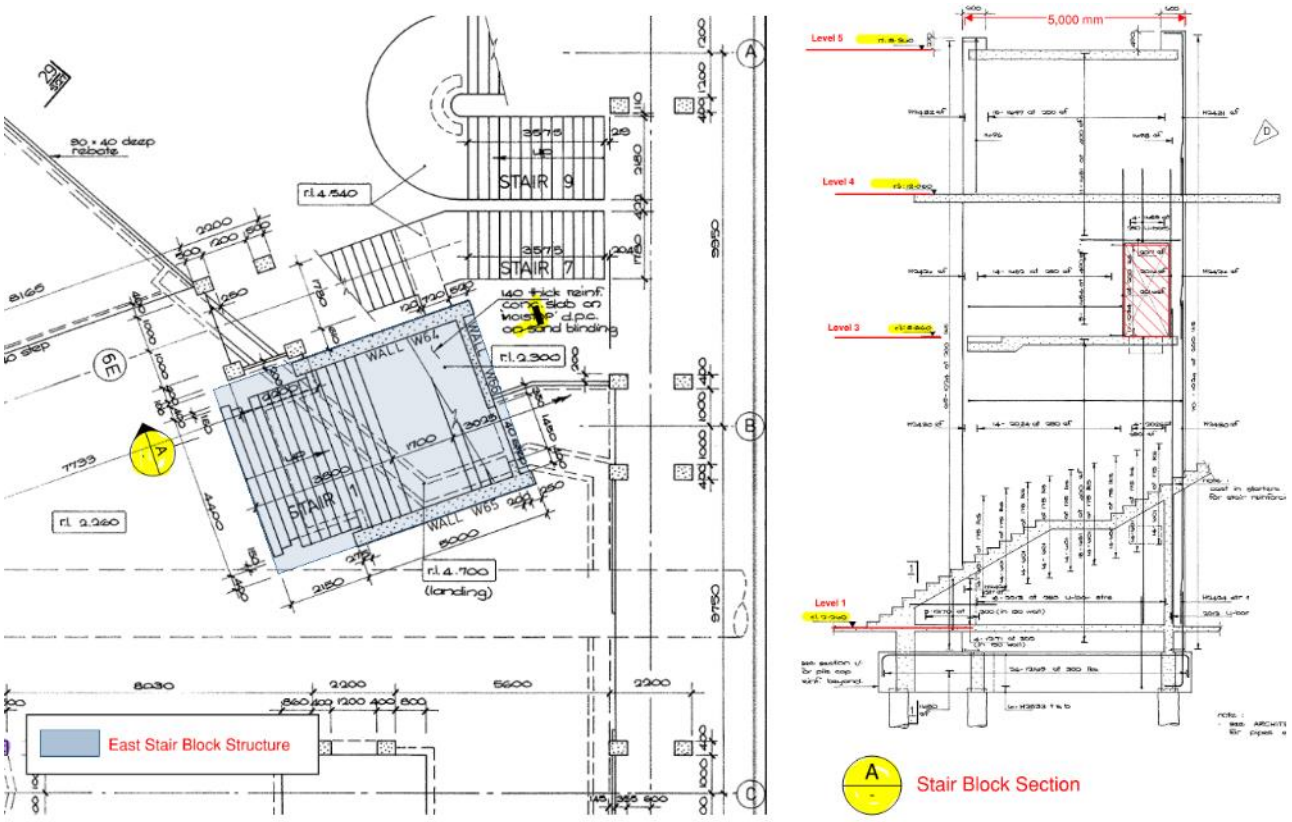

Figure F.2-1: Structural configuration of the eastern Staircase Structure (western as mirrored)

Gravity Load-Resisting System

The Staircase Structure supports slabs at Level 3, Level 4 and their roof level. The structure also supports a reinforced concrete staircase (Stairs 1/2) tied into the primary stair walls that spans from the ground floor level to Level 2 into the Renouf Foyer.

Lateral Load Resisting System

The Staircase Structure resists seismic demand through the walls under in-plane loading and through frame action under out-of-plane loading. Under out-of-plane demands, the floors act as beam elements and the walls act as column elements. A secondary wall (W66) is also located at the southern end of the structure spanning between walls as shown in Figure F.2-2. This secondary wall is considered part of the primary lateral system to assess the different inelastic mechanisms that could develop at each end of the structure.

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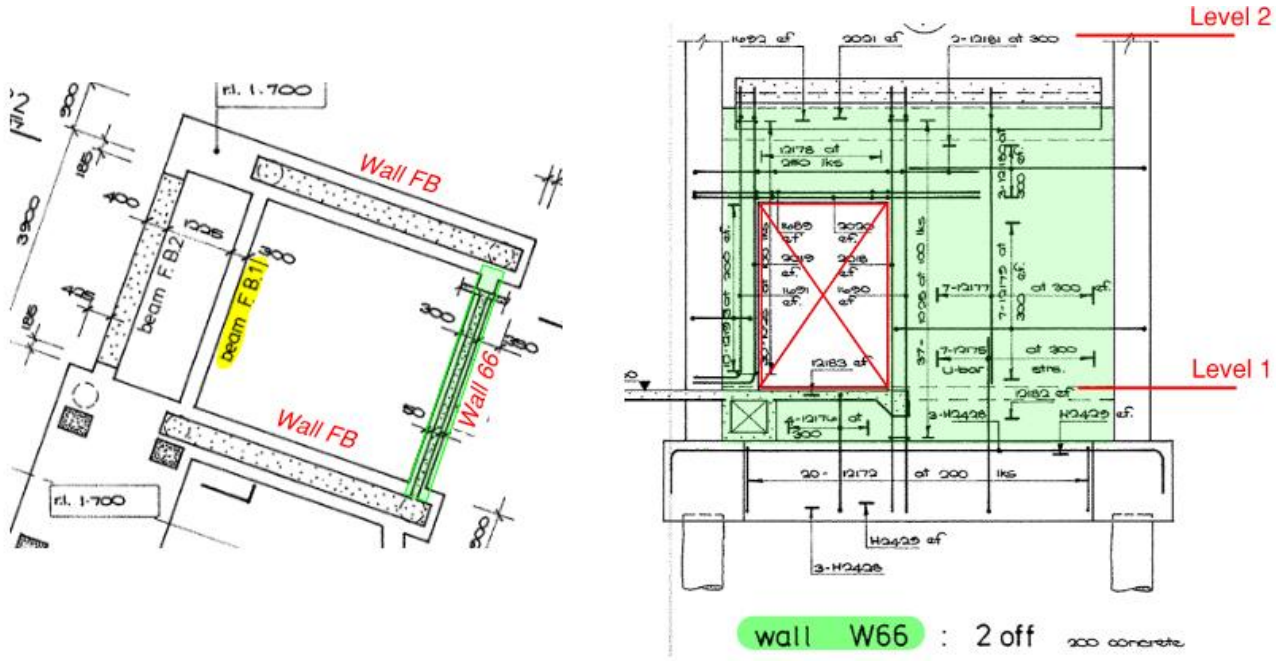


Figure F.2-2: Plan and elevation showing the secondary wall (W66) in the Staircase Structure.

F.2.2 Identification of potential Structural Weaknesses (SWs)

The following systems and elements were assessed as potential structural weaknesses as exceedance of their capacity could lead to a life safety hazard or damage to the adjacent buildings.

- Concrete walls in-plane (Level 1 to Roof)
- Concrete moment frame system out-of-plane (Level 1 to Roof)
  - Flexural, shear or deformation capacity of beams or columns
  - Beam-column joint shear

No Severe Structural Weaknesses (SSWs) were identified.

The impact of the adjacent structures was also considered in accordance with Section C2B of the Engineering Assessment Guidelines, as discussed in Section F.6.2.

F.2.3 Material Properties

Generally, strength reduction factors equal to 1.0 have been used. Material strengths used in the assessment are shown in Table F.2-1 and were based on values from the 1980's structural specification by Holmes, Wood, Poole and Johnstone. The concrete modulus of elasticity could also be determined in accordance with the Engineering Assessment Guidelines as, E_c = 30 GPa.

Reinforcing overstrength factors were also used in accordance with Table C5.4 of the Yellow Engineering Assessment Guidelines.

Table F.2-1: Material strengths used in assessment - Staircase Structure

Material	Strength on drawings	Probable strength used in assessment	Overstrength Factor, ϕ_0
Concrete	25 MPa	40 MPa	-
Reinforcing bars marked "H" on drawings (Grade 380 bars to NZS 3402 P:1973)	380 MPa	455 MPa	1.50
All other reinforcing bars (Grade 275 bars to NZS 3402 P:1973)	275 MPa	324 MPa	1.25

F.2.4 Element Modelling Parameters

Effective section properties were used in accordance with NZS3101:Part 2:2006 Table C6.6. These are shown in Table F.2-2.

Table F.2-2: Effective Section Properties used in assessment

Element	Effective Properties
Concrete floors	0.32 I_g
Concrete walls out-of-plane (Treated as column element)	0.35 I_g

The analysis included the effects of concrete confinement and longitudinal bar buckling based on the transverse steel indicated in the original drawings.

F.2.5 Element Capacities

The probable and overstrength capacity of elements were determined in accordance with Section C5.5 of the Yellow Engineering Assessment Guidelines. These values were used to establish the range of possible inelastic mechanisms that could form within each sub-assembly considering the range of possible element strengths.

For lateral systems involving frame action, the effects of axial load on columns must be considered to capture the true behaviour of the system. Steps taken to consider axial load in the assessment are outlined below.

- Maximum axial loads were determined based on gravity loads in addition to the anticipated beam shears from the floor slabs and primary stair. The maximum axial load was used to calculate the shear demand from flexural overstrength associated with a full flexural mechanism.
- Gravity axial loads were used to assess the strength hierarchy at each storey level. This was done assuming that the influence of axial load on the flexural strength remained essentially linear, such that there would be negligible impact on the sway index due to seismic beam shears.

- The probable element mechanism was found considering the ratio of the shear demand from flexural overstrength and the probable shear capacity. The shear demand from flexural overstrength was calculated using the equation below as,

$$V_{M_p} = \frac{\phi_o\left(M_{prob,1} + M_{prob,2}\right)}{L_{clear}}$$

Where; ϕ_o = the element overstrength factor, per Yellow Section C5.4.3, $M_{prob,x}$ = the probable moment capacity at end 1 or 2 of the clear span, and L_{clear} = the clear span length of the element. A flexural mechanism was said to develop when the ratio of the shear demand from flexural overstrength to the probable shear capacity was < 1.0.

Wall elements

The analysis of wall elements was carried out considering the dimensions, reinforcement and element capacities as shown in Table F.2-3 below. The M-N interaction diagram and shear capacity of the walls was determined considering the curtailment of reinforcement and presence of wall penetrations.

The shear demands associated with the flexural overstrength at the base of the walls was determined as discussed in Section F.3.1. It was found that the walls of the Staircase Structure are flexure controlled when acting in-plane.

The shear demands from flexural overstrength in the out-of-plane direction were conservatively calculated assuming wall hinges form above and below the floors. This is deemed conservative as the floors are significantly weaker than the walls – resulting in shear demands from flexural overstrength much lower than those shown in Table F.2-4. Despite the conservative estimate of shear demands, the walls were still seen to have flexural mechanisms when acting out-of-plane.

Table F.2-3: Properties of wall elements in the Staircase Structure.

Element		Reinforcement [%]		Dimensions [mm]		Axial Load [kN]	
		ρ_l	ρ_v	t_w	l_w	$N_{G+\psi_E Q}$	$N_{100\%}$
Walls	Base	0.72	0.25	400	5,000	1,000	1,350
	Level 3 Penetration	0.47	0.25	400	3,640	425	-
	Upper	0.51	0.25	400	5,000	150	-
Secondary Stair Wall (W66)	-	0.47	0.38	200	300	190	-

Table F.2-4: Capacity of wall elements acting in-plane and out-of-plane

Element		In-plane				Out-of-plane			
		M_{prob} [kNm]	V_{prob} [kN]	V_{Mp} [1] [kN]	$\frac{V_{M_p}}{V_{prob}}$	M_{prob} [kNm]	V_{prob} [kN]	V_{Mp} [kN]	$\frac{V_{M_p}}{V_{prob}}$
Walls	Base	26,000	8,156	1,125	**0.42**	1,040	807	642	**0.8**
	Level 3 Penetration	13,500	5,580	2,377	**0.42**	480	687	360	**0.52**
	Upper	16,500	5,580	3,462	**0.20**	632	886	470	**0.53**
Secondary Stair Wall (W66)	-	1,600	1,458	570	**0.39**				

[1]See Section F.3.1. for determination of inelastic shear demands in IP wall.

Floor elements

The floors slabs are treated as beam elements with the analysis carried out considering the dimensions, reinforcement, and element capacities as shown in Table F.2-5 below. The reinforcement varies slightly between the eastern and western Staircase Structures but was not considered to significantly impact the seismic performance. Stairs 1/2 are also considered to act as a beam element based on the throat thickness of the stair with the capacity expressed per unit width in the Table F.2-5. The shear capacity of the stairs is calculated considering the lower capacity at mid-span and the transverse reinforcement adjacent to the walls is excluded.

Table F.2-5: Properties of slab beam elements in the Staircase Structure

Element	Reinforcement [%]			Dimensions [mm]			M_{prob} [kNm]		V_{prob} [kN]	V_{Mp} [kN]	$\frac{V_{M_p}}{V_{prob}}$
	ρ_{top}	ρ_{bot}	ρ_v	L_{clear}	h_b	b_w	+ve	-ve	-	-	
Roof	0.47	0.26	-	4400	200	5000	119	207	805	93	**0.11**
Level 4	0.62	0.22	-	4400	200	5000	100	275	834	125	**0.15**
Level 3	0.59	0.3	-	4400	200	5000	123	219	845	112	**0.13**
Stairs 1/2	0.4	0.4	-	4400	200	1000	36	36	165	19	**0.11**

F.2.6 Building Masses, Weights, Mass Eccentricity and P-Delta

The Staircase Structures support Stairs 1/2 and the floors at Level 3, Level 4, and their roofs. The assumed floor seismic weights are summarised in Table F.2-6.

Table F.2-6: Floor Seismic Weight Summary (kPa)

Level	Gravity Load	Superimposed Dead Load	Seismic Live Load	ψ_E
Roof	4.8	1.0	0.25	0.0
Level 4	4.8	1.0	4.0	0.3
Level 3	4.8	1.0	4.0	0.3
Stairs 1/2	7.2	0.5	4.0	0.3

Table F.2-7 lists the estimated seismic weight at each level, including the weight of the walls which contributed approximately 60% of the total seismic weight of the superstructure. The Staircase Structures also support the mid-height landings of Stair 7/8 and 11/12. This resulted in an additional seismic weight of 70kN being considered at both level 2 and 3 due to the tributary mass of stair 7/8 and 11/12.

Table F.2-7 Seismic weight distribution of the Staircase Structure.

Level	Elevation (m)	Weight (kN)
Roof	13.10	286
Level 4	9.80	565
Level 3	6.60	579
Stairs 1/2	2.45 [1]	662
TOTAL		**2091**

[1] Average height of stairs used for weight distribution.

P-Delta Effects

An allowance for P-Delta effects should be made for buildings with large drifts. To account for P-Delta effects, the base shear capacity of the system could be reduced using the equation below,

$$V_{b,eff} = V_b - C\frac{m_e g \cdot \Delta_{cap}}{H_e}$$

Where; m_e, H_e and Δ_{cap} are the effective SDoF properties associated with each mechanism and C = 0.5 for concrete structures. Only a nominal reduction for both the in-plane and out-of-plane direction mechanism was determined: $V_{b,\,eff} = 0.97{*}V_b$.

Higher Mode Effects

In accordance with Section C2.5.10 of the Engineering Assessment Guidelines, higher mode effects were considered not to be significant given the configuration of the structure.

Torsion

For 'torsionally unrestrained structures' (where sufficiently stiff lateral structure spaced to form a couple is not present in the perpendicular direction), Method C of C2F.4 of the Engineering Assessment Guidelines may be used provided that the effective period of the system with reduced strength is not greater than the corner period at T=3.0s of the elastic site spectra of NZS 1170.5:2004.

Torsion induced in the system by accidental eccentricity (10% of the width in question) of the seismic mass has been considered as part of this assessment. In the assessment of lateral loading in the out-of-plane direction of the walls, the method described in Section C2F.3 of the Engineering Assessment Guidelines has been used.

In the assessment of lateral loading in the in-plane direction, the lateral capacity (base shear) has been reduced to eliminate the strength eccentricity in accordance with Section C2F.4 of the Engineering Assessment Guidelines. The reduction in base shear capacity was ~20%.

F.2.7 Diaphragm Modelling

Diaphragms are stiff compared to the vertical elements of the lateral load resisting system and are therefore modelled as rigid in the analysis.

F.2.8 Foundation/Soil Interface Modelling

For this assessment the impact of foundation strength and stiffness was significant due to the large height-to-width aspect ratio and the presence of walls which will impart large overturning moments into the foundation system under in-plane loading. The foundation system of the Staircase Structure is shown in Figure F.2-3, consisting of 900 mm deep foundation beams on Ø450mm 'Franki' type piles located at the end of each primary stair wall. The foundation beams spanning between walls (FB1) was included in the SLaMA method when assessing the strength hierarchy in the out-of-plane direction.

A range of probable pile capacities and stiffnesses were supplied by Tonkin+Taylor Ltd, as shown in Table F.2-8. Further description of the Geotechnical Assessment is presented inTable F.2-8. The pile properties were used as part of the assessment to:

1. Assess the possibility of the exceedance of the geotechnical tension or compression capacity of the piles under seismic loading.
2. Consider elastic foundation rotations due to vertical pile displacements based on the range of possible pile stiffnesses.

Generally, the probable pile stiffnesses were used to evaluate the performance of the foundation system, with the upper and lower bounds used to consider the possible variability in superstructure performance. The performance for both the liquefaction and non-liquefied case was also evaluated.

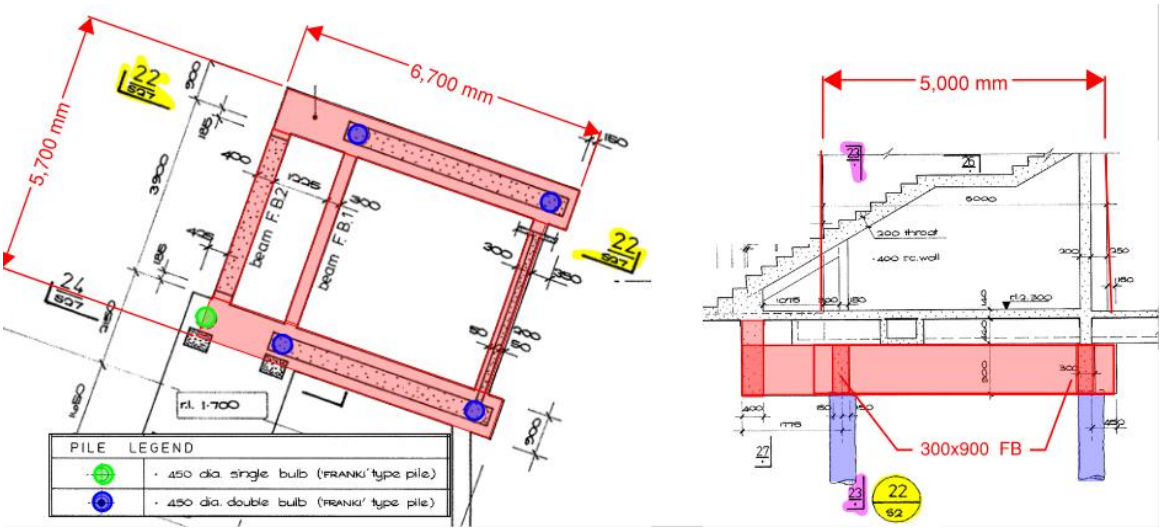


Figure F.2-3: Foundation system of MFC Independent Staircase Structure.

Table F.2-8: Pile properties for 'Franki' type piles used in Staircase Structure Assessment

Properties	Undegraded ground conditions	Liquefaction ground condtitions
Compression		
Probable Limiting Compression Capacity [kN]	1950	1350
Compression Springs [kN/mm]	60 (20-200)[1]	
Pile Tension		
Probable Limiting Tension Capacity [kN]	300	160
Tension Springs [kN/mm]	50 (15-150)[1]	

[1]*Upper and lower bound pile stiffnesses shown in ().*

F.3 DESCRIPTION OF STRENGTH HIERARCHY

F.3.1 In-Plane Direction

Superstructure

The mechanism of the superstructure under in-plane loading was determined considering the seismic demands associated with flexural hinging at the base of the two walls. The seismic demands associated with flexural hinging at the base of the walls were determined using the Equivalent Static Analysis (ESA) method, in accordance with Section 6.2 of NZS 1170.5:2004. The seismic storey forces were scaled until the overstrength flexural capacity was reached at the base of the wall. The shear demands from flexural overstrength were then compared with the probable undegraded shear capacity of the wall up the height of the structure as shown in Figure F.3-1 below. The shear demands from flexural overstrength were also increased considering a dynamic shear amplification factor of, ω_v = 1.3, in accordance with Section CD4.3 of NZS3101:2006. It was found there is sufficient shear capacity in the walls to fully develop a flexural hinge.

A strut mechanism may form through the ground floor stair run and shift the formation of an inelastic mechanism near Level 2 rather than at the base. This added stiffness was not specifically investigated and does not change the primary inelastic mechanism in the foundation.

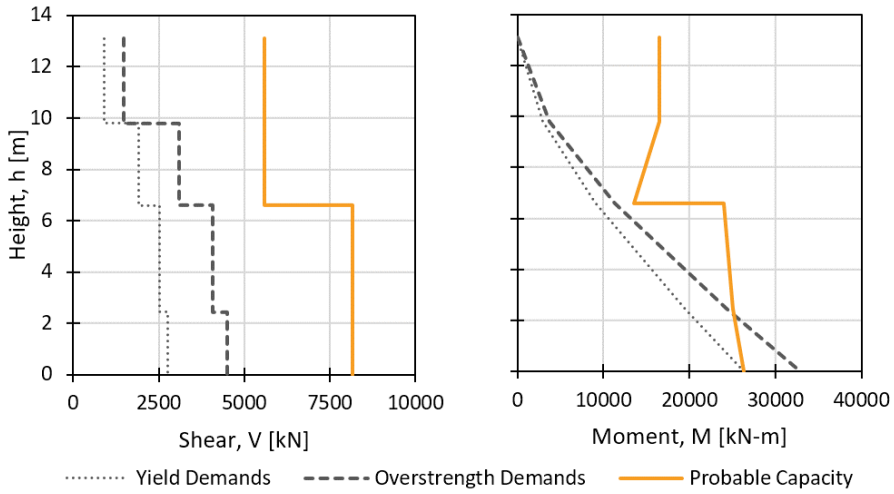


Figure F.3-1: Maximum probable and overstrength demands in primary stairs walls under in-plane loading corresponding to flexural hinging at the base.

Substructure

The foundation system was assessed considering the probable pile capacities shown in Table F.2-8. The overturning resistance of the pile system was determined through equilibrium considering the combined gravity and seismic demands at which the pile capacities were exceeded. The overturning moment and corresponding base shear capacity of the pile system is summarised in Table F.3-1 below. It was found that exceedance of the geotechnical tension capacity will occur between 25-30% of the probable base shear capacity of the superstructure. This pile exceedance will limit the flexural behaviour of the walls above.

In discussion with Tonkin+Taylor Ltd., because the piles are almost at their compressive capacity once the tension capacity is met, residual pile vertical deformations may occur. On repeating cycles, ever increasing permanent settlements would create residual superstructure drifts. However, this permanent uplift and settlement behaviour was not explicitly considered as part of this assessment.

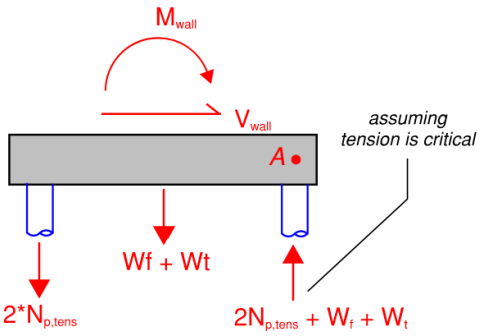


Figure F.3-2: Force equilibrium in pile system at pile tensile capacity

Table F.3-1: Base shear capacity of Staircase Structure foundation system

Foundation Property	No Liquefaction Case	Liquefaction Case
Superstructure Weight, W_t [kN]	1950	
Foundation Weight, W_f [kN]	356	
Probable Limiting Compression Capacity, N_{comp} [kN]	1950	1350
Probable Limiting Tension Capacity, N_{tens} [kN]	300	160
Compression Pile Demand at Tension Capacity (per pile), N_{tens} +1/2(W_f + W_t) [kN]	1453	1312
Mechanism	*Pile in tension*	*Pile in tension*
Overturning Resistance, M_{found} [kNm]	7712	6480
Corresponding Base Shear Capacity, V_{found} [kN]	811	682

Effect on adjacent Stairs 7/8 and Stairs 11/12

As the Independent Staircases Structures are overturning, this has an effect on the adjacent staircases of Stairs 7/8 and Stairs 11/12, which are connected to the side of the walls.

The Independent Staircase Structures are most likely to uplift northwards, as there are less piles that can provide tension capacity in that direction. The contribution of the foundations of Stairs 9/10 is not included in the uplift capacity. Also, the contribution of the 'lock-up' from Stairs 1 and 2 on the floors of the Renouf Foyer has not been included in the uplift capacity.

As the Independent Staircases uplift northwards, this uplifts the flights of stairs 7, 8, 11 and 12. The lower flights of the stairs are lifted off their seating, which will put a hogging flexural demand on the flights at the location of the mid-height landing. For the upper flights, this will put a sagging flexural demand on the flights at the upper most connection. In the opposite direction, as the Independent Staircase Structures

uplift southwards, it is expected that the opposite occurs. Though this will be to a much smaller extend, as the mid-height landing is located further south on the walls of the Independent Staircase Structures.

For the flights of Stairs 7, 8, 11 and 12, it is expected that the uplift of the Independent Staircase Structures will result in a flexural mechanism across the flights, as the displacements will put the sections beyond their yield strength. This will not result in a life safety hazard, as gravity support of the stairs is maintained through dowel action of the reinforcement in the stairs that has not yielded. The stair flights will become simply supported, and do not rely on the fixity at their upper end.

For the mid-height landings, the assessment has considered the combined flexural, shear and torsion demands from flexural overstrength of the stair flights. This found that the mid-height landings can accommodate this uplift and maintain support of the gravity demands from stair flights.

Neither of the mechanisms result in a life safety hazard.

If the uplift/overturning of the Independent Staircase Structures is prevented, by any means, the next element in the strength hierarchy is the capacity of the walls in plane. These have a score of >100%NBS_{IL3}.

F.3.2 Out-of-Plane Direction

Superstructure

The strength hierarchy of the Staircase Structure beam-column elements acting out-of-plane was determined using the sway index, which evaluates whether a beam-sway or column-sway mechanism is likely to occur at each level of the structure. A column-sway should be assumed to form where $S_i > 0.85$. The sway index was calculated at each level using equation C2A.2 from the Engineering Assessment Guidelines shown below,

$$S_i = \frac{\sum(M_{o,bl} + M_{o,br})}{\sum(M_{p,ca} + M_{p,cb})}$$

Where; $M_{o,bl}, M_{o,br}$ = the overstrength flexural strengths of the slab beam at each side of the joint, and $M_{p,ca}, M_{p,cb}$ = the probable flexural strengths of the wall column above and below the joint.

The presence of the secondary stair wall (W66) means that different behaviour will occur at each end of the Staircase Structure. The potential strength hierarchy was evaluated differently for each end of the structure.

- Northern end – Full frame action is expected down to the ground floor level with the sway index checked considering the foundation beam (FB1) resisting actions from at least half the width of the primary stair wall out-of-plane.
- Southern end – The secondary wall (W66) will resist shears at the top of the staircase. The secondary wall was checked considering the overstrength base shear demand associated with half the wall width forming a flexural mechanism at the top of the staircase. This was used to assess whether the secondary stair wall will remain elastic and force a frame mechanism to develop at Level 2.

Sway Index

The sway indices for the Staircase Structure acting out-of-plane are summarised in Table F.3-2. It was found that the primary stair walls acting out-of-plane have sufficient capacity to develop a full-height beam-sway mechanism. It was also found that the foundation beam had sufficient capacity such that the northern end of the primary stair walls will develop at flexural hinge at the ground floor level.

Table F.3-2: Sway Index results for Staircase Structure acting out-of-plane.

Level	Prob Slab Moment $M_{p,b}$ [kNm]	Above Wall Capacity $M_{p,ca}$ [kNm]	Below Wall Capacity $M_{p,cb}$ [kNm]	$\Sigma(M_{p,ca} + M_{p,cb})$ [kNm]	Sway Index, S_i	Mechanism
Roof	207		605	605	**0.43**	Beam-Sway
Level 4	300	632	436	1068	**0.35**	Beam-Sway
Level 3	258	486	938	1424	**0.23**	Beam-Sway
Stair 1/2[1]	33[1]	208[1]	208[1]	417[1]	**0.10**	Beam-Sway
Ground floor level (Northern end)[2]	668[2]	521		521	**1.60**	Column Hinge

[1]Stair 1/2 joint analysed on a per unit width basis.
[2]Ground floor level mechanism checked comparing foundation beam capacity with half the width of the wall.

Secondary Stair Wall (W66)

At the southern end of the Staircase Structure, the secondary wall will resist seismic loads at Level 2 and could potentially force the structure to force a frame mechanism starting just above Level 2. The overstrength base shear capacity associated with half the wall width forming a full beam-sway mechanism above Level 2 was determined using the equilibrium method as, $\phi_o V_{b,south} = 305$ kN. This corresponded to a moment demand in the secondary wall of 1070 kN-m which could be resisted considering the probable capacity of the wall. This meant that at the southern end of the Staircase Structure, a beam-sway mechanism will form starting at Level 2 at the top of the stairs.

Slab-Wall Joints

Considering that a beam-sway mechanism was anticipated in the out-of-plane direction, the joint regions between the floor slabs and primary walls were checked against possible shear as the floors undergo inelastic deformations. A typical section through the Level 3 floor is shown in Figure F.3-3. It shows no joint reinforcement, and the longitudinal reinforcing in the floor slab terminates in the wall with hooks bent away from the joint.

The joints between the walls and the floors were assessed as unreinforced beam-column joints, in accordance with Section C5.6.2.2 of the Yellow Engineering Assessment Guidelines. The total joint shear capacity at each level could be computed in accordance with equations C5.77 to C5.81 of the Engineering Assessment Guidelines. The capacity was influenced by:

- Axial demand on the joint considering gravity and seismic actions
- Curvature ductility of the adjacent floor slab under a beam-sway mechanism.

The reduction in capacity due to curvature ductility adjacent to the joint was determined using Figure C5.27 in the Yellow Engineering Assessment Guidelines (Exterior Joint – Case 2). The curvature ductility in

the floors at beam-column joint-shear, shown in

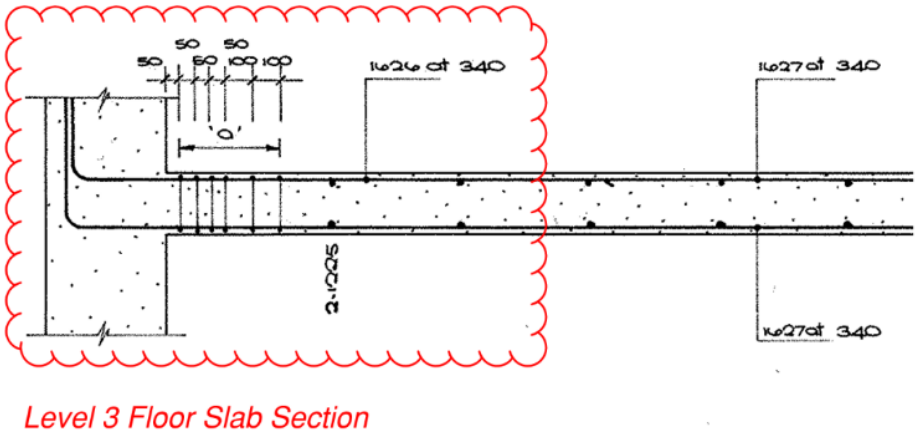


Figure F.3-3: Beam-column joint of Level 3 floor slab in the Staircase Structure.

- Table F.3-3 were iteratively determined using equations C5.77 to C5.81 considering the combined gravity and seismic tension forces (critical) associated with the full-height beam-sway mechanism.

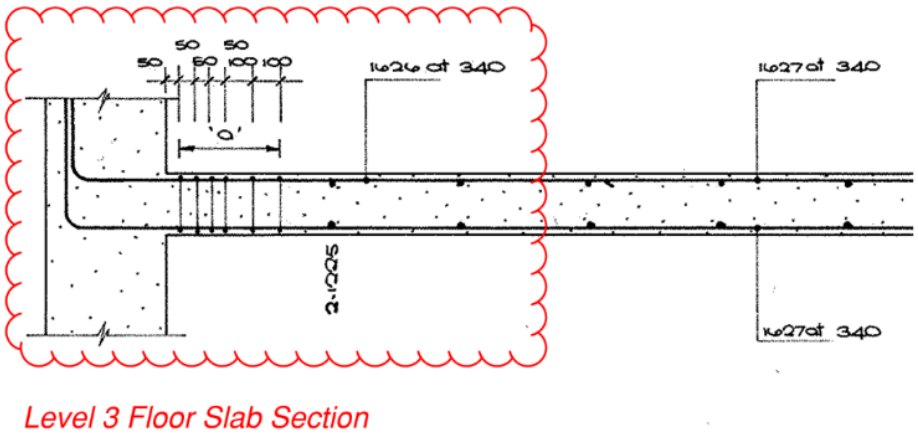


Figure F.3-3: Beam-column joint of Level 3 floor slab in the Staircase Structure.

Table F.3-3: Curvature ductility limits of floor slab-wall joints in the Staircase Structure.

Joint level	Probable Joint Shear Demand V^*_{jh} [kN]	Critical Joint Shear Coefficient, $k_{j,crit}$	Curvature Ductility Limit, $\mu_{\phi,max}$
Roof	1490	0.137	2.45
Level 4	2020	0.180	**1.80**
Level 3	1700	0.144	2.34
Stair	1300	0.100	∞

Substructure

The pile capacities were checked similarly to the in-plane direction to assess whether uplift could occur due to the seismic beam shear from the slabs. It was found that there was sufficient pile capacity to develop a full frame mechanism in the out-of-plane direction, without exceeding the tension capacity of the piles. The incremental permanent pile deformation and settlement was not developed in the out-of-plane actions.

F.4 EVALUATION OF BILINEAR FORCE-DISPLACEMENT RESPONSE

In-plane direction

Based on the strength hierarchy of the system discussed in Section F.3, the capacity of the foundation system will be exceeded before in-plane flexural hinging can occur at the base of the walls or strut action forming through the stair run.

To estimate the displacement profile of the superstructure when the foundation system reaches capacity, the yield profile of the walls was scaled down considering the base shear capacity of the foundations. The yield profile of the walls was determined using the moment-curvature method, in accordance with Section C5.5.3.1 of the Yellow Engineering Assessment Guidelines. The yield curvature was estimated as $6.48(10)^{-4}$ m^{-1} at yield, corresponding to a roof drift of 0.30%. The foundation system reaches capacity at 25-30% of the flexural capacity of the wall. Based on this, an elastic drift of 0.07-0.09% was assumed to occur in the superstructure before reaching the foundation capacity.

The elastic foundation rotations at uplift could be estimated considering the vertical pile stiffnesses given in Table F.2-8, and the maximum pile demands given in Table F.3-1. Elastic foundation rotations were determined using the equation below.

$$\theta_{el,f} = \frac{\Delta_{t,pile} + \Delta_{c,pile}}{L_{pile}}$$

Where; $\theta_{el,f}$ = elastic foundation rotation, $\Delta_{t,pile}, \Delta_{c,pile}$ =the vertical displacement of the tension and compression piles at ultimate capacity, and L_{pile} = the distance between the piles (4.4 m).

The range of possible foundation rotations based on the probable pile properties are summarised in Table F.4-1 below. Possible elastic foundations rotation varied between 0.17-2.1% due to the factor of 10 in the range of probable pile stiffnesses. For the purposes of this assessment, the probable pile stiffnesses were used to evaluate the performance of the structure considering both the undegraded and liquefied ground conditions.

Table F.4-1: Elastic foundation rotation of Staircase Structure under in-plane loading.

Foundation Property	Undegraded ground conditions	Liquefied ground conditions
	Probable (Lower – Upper Bound)	Probable (Lower – Upper Bound)
Tension Pile Capacity	300	160
Tension Pile Stiffness [kN/mm]	50 (15 – 150)	50 (15-150)
Tension Pile Vertical Disp, $\Delta_{t,pile}$ [mm]	6 (2 – 20)	3.2 (1-11)
Compression Pile Demand at Tension Capacity (per pile), N_{tens} + 1/2(W_f + W_t) [kN]	1452	1313
Compression Pile Stiffness [kN/mm]	60 (20 – 200)	60 (20 – 200)
Comp. Pile Vertical Disp, $\Delta_{c,pile}$ [mm]	24 (7.3 – 73)	22 (6.6 – 66)
Elastic Foundation Rotation	**0.7% (0.21 – 2.1%)**	**0.57% (0.17 – 1.7%)**

The structure was found to be controlled by an overturning pull-out mechanism after the capacity of the piles was reached. The pile behaviour was assumed to be elastic perfectly-plastic, such that the base shear capacity will be constant after the pile tensile capacity is reached. This allowed for the displacement profile of the structure to be determined considering; 1) elastic wall deformations, 2) elastic foundation rotations, and 3) overturning rotation at the foundation capacity.

The displacement profile considering the probable pile stiffness for the liquefaction case is shown in Figure F.4-1 below. The effective SDoF properties associated with the overturning mechanism could be determined in accordance with C2.4.2 of the Engineering Assessment Guidelines. The SDoF properties were not determined based on the range of vertical pile stiffness; however with consideration of the upper and lower bound stiffnesses, the sum of superstructure drift and foundation rotation at uplift varies between 0.24-2.2%. The ultimate displacement capacity of the structure was said to have occurred when 2.5% roof was reached, in accordance with limitations in Section 7.5 of NZS1170.5:2004.

The probable strength and stiffness of the system could then be used to form the building response curve considering each geotechnical scenario as summarised in Table F.4-2 below.

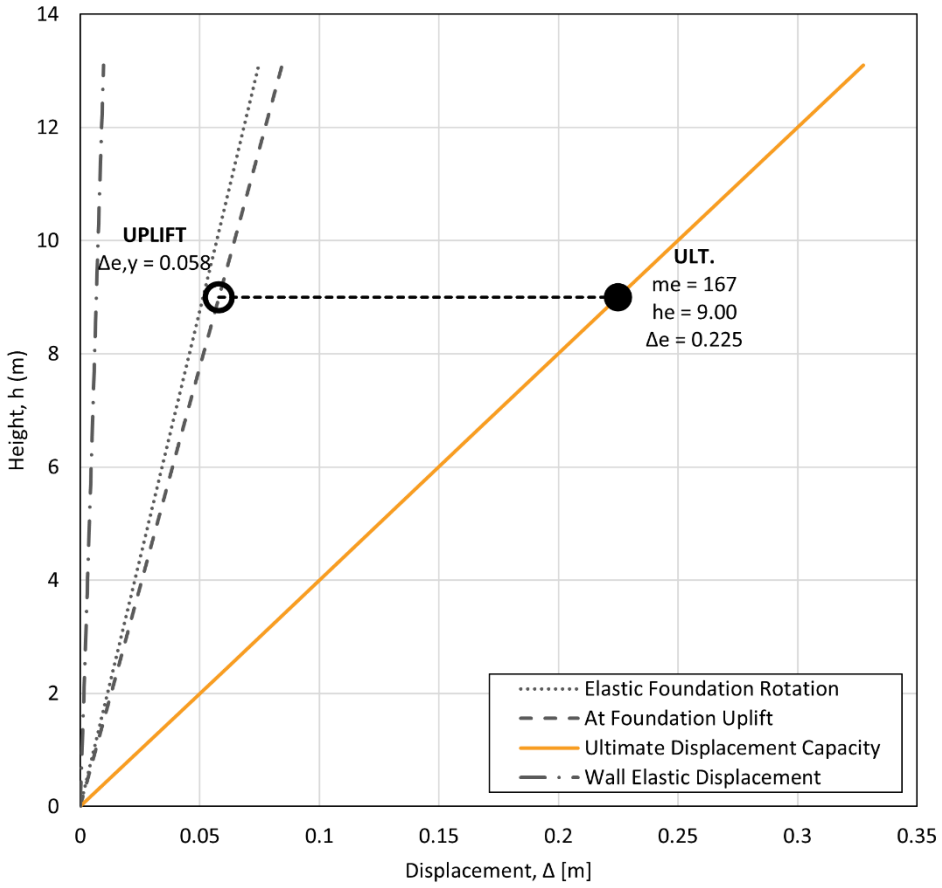


Figure F.4-1: Displacement profile of the Staircase Structures under in-plane loading (probable piles stiffness - liquefaction case).

Table F.4-2: Building response curve of the Staircase Structures under in-plane loading.

Geotechnical Scenario	V_b [1] [kN]	Effective SDoF Properties				
		m_e [tonnes]	H_e [mm]	Δ_{uplift} [2] [mm]	Δ_{cap} [mm]	S_A [g]
Undegraded	660	167 [78%]	9000 [69%]	72 (27 – 200)	225	0.402
Liquefaction	552	167 [78%]	9000 [69%]	58 (22 – 163)	225	0.337

[1] Reduced to consider P-Delta and torsion effects.
[2] Uplift displacements considering upper/lower bound pile stiffnesses shown in ().

Out-of-plane direction

Based on the strength hierarchy of the Staircase Structures under out-of-plane seismic actions, the structures will develop a different mechanism at each end of the structure due to the presence of the secondary stair wall (W66). Likely, the actual mechanism will be varying along the length of the walls. The possible mechanisms at each end of the structure were used to bound the response. To capture both the displacement profile and base shear capacity associated with each mechanism, the probable base shear capacity of the Staircase Structures was calculated on a per unit width basis. The probable base shear capacity associated with a beam-sway mechanism occurring at each end of the wall could be calculated in accordance with Appendix C2A of the Engineering Assessment Guidelines using the equations below,

$$OTM = \sum M_{col,i} + \left(\sum_n V_{end\ beam.n}\right) L$$

and,

$$V_b = \frac{OTM}{H_e}$$

Where; $M_{col,i}$ = the probable moment capacity of the base columns (in this case, walls acting out-of-plane), $V_{end\ beam.n}$ = beam shears associated with flexural hinging of the floor slabs, and L = the distance between wall lines forming the force couple between beam shears.

For the purposes of this assessment, the effective height was iteratively found from the resulting displacement profiles as, H_e = 9.07 m at the North end and, H_e = 10.25 m at the south end. The beam shears at flexural yielding of the floor slabs are shown in Table F.4-3 below.

Table F.4-3: Beam shears from floors in the Staircase Structures in a beam-sway mechanism

Level	Seismic Moments, $M_{p,slab}$ [kNm/m]		$V_{slab,I}$ [kN/m]
	-ve	+ve	
Roof	41	24	14.8
Level 4	63	29	21.0
Level 3	52	27	18.0
Stairs 1/2	36	36	16.4
		Σ	**70.1 (54 [1])**

[1] Stairs 1/2 moment and shear is removed, as it is assumed to not yield at southern end of Staircase Structure

This allowed for the calculation of the base shear capacity (per unit width) at each end of the wall to be determined as shown below.

Northern end – Full-height beam-sway

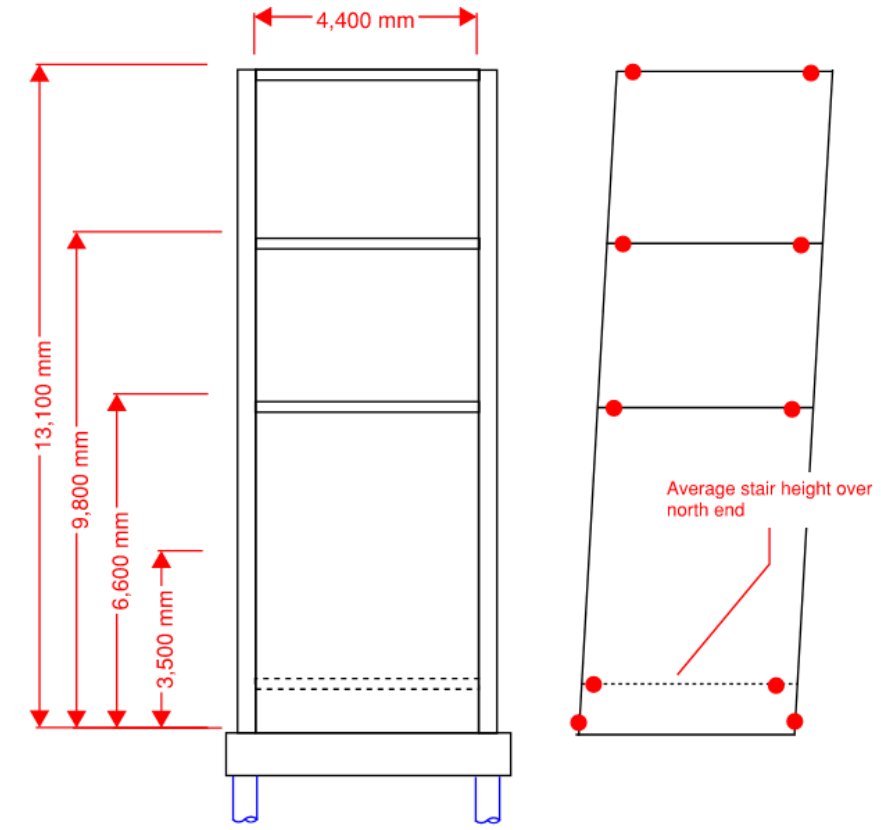


Figure F.4-2: Full-height beam-sway mechanism forming at northern end of the Staircase Structure.

$$\sum V_{slab,i} = 70\ kN/m$$

$$\sum M_{col,i} = 201\ kNm/m$$

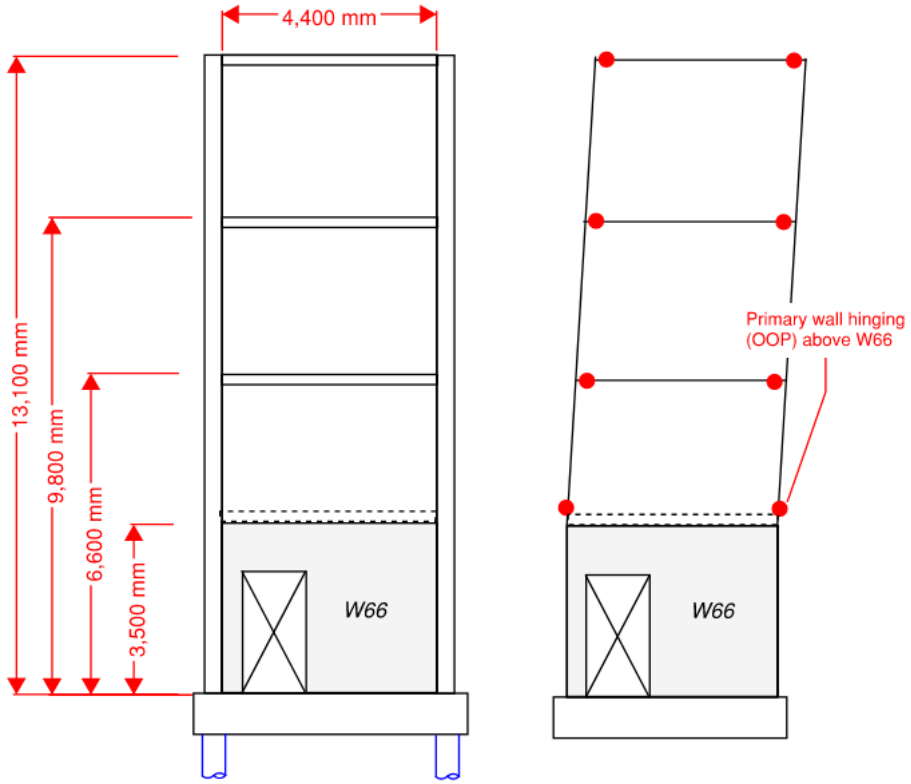
(Calculated based on gravity axial load at base of wall)

$$L = 4.8\,m$$

$$OTM = \sum M_{col,i} + \left(\sum_n V_{end\ beam.n}\right) L = 201\frac{kNm}{m} + \frac{70kN}{m} * 4.8m = 740\ kNm/m$$

$$V_{b,south} = \frac{OTM}{H_{e,south}} = \frac{740kNm/m}{9.1m} = \mathbf{81.5}\frac{\boldsymbol{kN}}{\boldsymbol{m}}$$

Southern end – Beam-sway above W66



South End Mechanism: Beam-Sway above W66

Figure F.4-3: Beam-sway mechanism forming above W66 at the south end of the Staircase Structure.

$$\sum V_{slab,i} = 54\ kN/m$$

(Assuming Stairs 1/2 don't yield due to presence of W66)

$$\sum M_{col,i} = 197\ kNm/m$$

(Calculated based on gravity axial load above W66)

$$L = 4.8\,m$$

$$OTM = \sum M_{col,i} + \left(\sum_{n} V_{end\ beam.n}\right) L = 197\,\frac{kNm}{m} + \frac{54kN}{m} * 4.8m = 661\ kNm/m$$

(Effective height calculated relative to ground level so need to subtract height of W66)

$$V_{b,south} = \frac{OTM}{H_{e,north} - H_{W66}} = \frac{661kNm/m}{10.25m - 3.5m} = \mathbf{98}\,\frac{\boldsymbol{kN}}{\boldsymbol{m}}$$

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Yield displacement

The displacement profile of the structure was estimated using the moment-curvature method, in addition to a 2D elastic Microstran model. Because the walls acting out-of-plane are much stronger than the floor slabs, cantilever action from base hinging of the walls dominated the response and lead to an atypical load distribution in the frame system.

The yield drift of the floors was estimated using the moment-curvature method, in accordance with Yellow Section C5.5.3.1 based on the section properties and element dimensions as, $\theta_{y,slab}$ =1.16%. To capture the yield drift of the primary walls acting out-of-plane, a 2D Microstran model was developed representing the restraint conditions at each end of the wall. The element modelling properties used were in accordance with Section F.2.4. Two models were developed to capture the yield profile at each end of the frame, with each model given an effective width of half the total structure width (2.5 m). To capture the yield displacement at full yield of the Staircase Structures the following procedure was followed:

- Seismic storey forces corresponding to the probable base shear capacity associated with each mechanism was applied to the structure following the ESA method (triangular load distribution), in accordance with NZS1170.5:2004.
- The probable moment capacities in the floors and base column moments were applied to the model through fixed end moments to capture the force distribution at full yield.
- The displacement profiles at each end of the structure from the 2D elastic model are shown in Figure F.4-4 below.

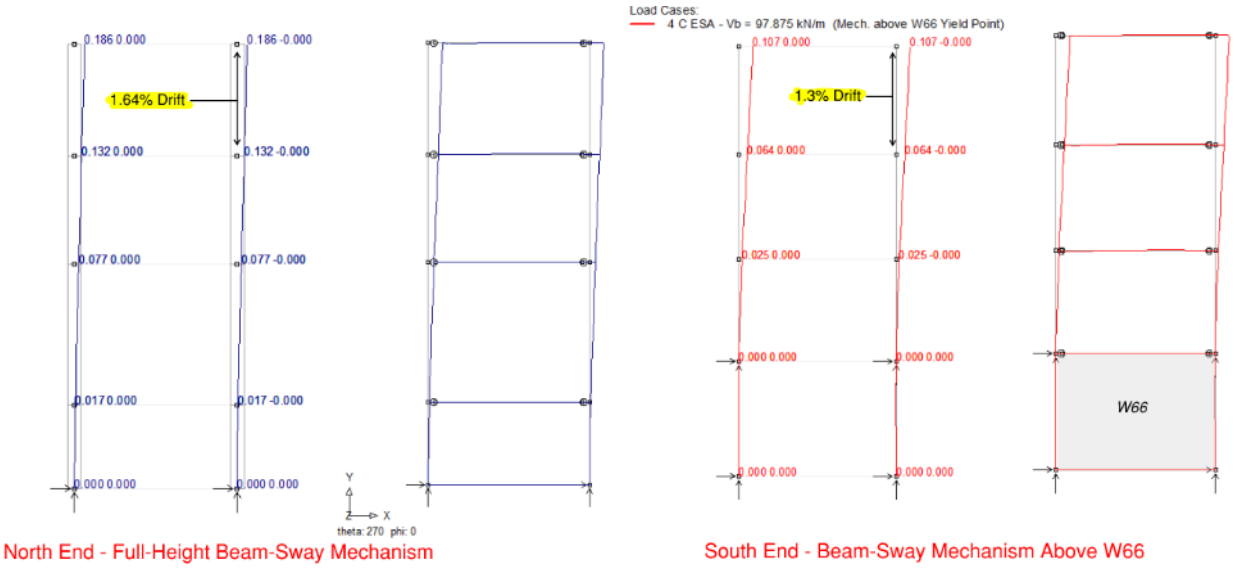

Figure F.4-4: Displacement profile of 2D elastic model at full yield at the northern end (left) and southern end (right) of the Staircase Structures

Using the equations to derive the base shear capacity of the Staircase Structures, the relative base shear contributions from the floor slabs and walls could be determined. Based on the yield drift and relative contribution of each component, an effective yield drift for each mechanism could be estimated. The effective yield drift estimated for each mechanism is shown below.

Southern end – Full-height beam-sway: $\theta_{y,e}$ = 1.43 %

Northern end – Beam-sway above W66: $\theta_{y,e}$ = 1.23 %

The inelastic deformation capacity associated with each mechanism was determined using the direct-rotation method, in accordance with Section C5.5.3.3 of the Yellow Engineering Assessment Guidelines. The inelastic rotation capacity associated with each mechanism was determined based on the section properties as shown in Section F.2.5 and is summarised in Table F.4-4 below.

Table F.4-4: Inelastic displacement capacities of the Staircase Structure beam-column elements

Mechanism	Assessment Method	Allowable inelastic drift θ_p	Ultimate Drift Capacity θ_{cap}	Notes:
Southern end – Full-height beam-sway	Floor Slabs – Table C5.7 Walls (out-of-plane) – Eq. C5.33 & Eq. 5.59	0.86	2.29	- Capacity limited by ensuring margin until onset of loss of gravity support (Eq. C5.8). - Inelastic rotation capacities in wall (out-of-plane) of 0.02 rad. - Inelastic rotation capacity of floors ranged from 0.019-0.02 rad.
Northern end – Beam-sway above W66	Floor Slabs – Table C5.7 Walls (out-of-plane) – Eq. C5.33 & Eq. 5.59	0.98	2.21	- Capacity limited by ensuring margin until onset of loss of gravity support (Eq. C5.8). - Inelastic rotation capacities in wall (out-of-plane) of 0.02 rad. - Inelastic rotation capacity of floor slabs ranged from 0.019-0.02 rad.

The deformation capacity of the wall-floor joints could be determined based on the estimated curvature ductility in the floor at which joint shear would occur. Curvature ductility was converted into an inelastic drift capacity using Eq. C5.46 of the Yellow Engineering Assessment Guidelines. An additional joint shear deformation of 0.0015 radians at ULS was considered based on the reinforcing detailing (hooks bent away from joint) and in accordance with Section C5.6.2.2 of the Yellow Engineering Assessment Guidelines.

Table F.4-5: Deformation capacity of beam-column joint-shear between floors and walls

Level	Curvature ductility limit, $\mu_{\phi,max}$	Yield drift, θ_y [%]	Beam rotation limit, $\theta_{p,j}$ [%]	Joint deformation, γ_j [%]	Ultimate rotation capacity, θ_{cap} [%]
Roof	2.445	1.19	0.75	0.15	2.08
Level 4	1.800		0.41	0.15	**1.75**
Level 3	2.340		0.69	0.15	2.03
Stair	∞		-	-	-

Based on the drift capacities summarised in Table F.4-5 above, the total drift capacity of the structure will be limited by the Level 4 beam-column joints, corresponding to a total drift capacity of $\underline{\theta_{cap} = 1.75\%}$. The assumed displacement profiles at each end of the Staircase Structures could then be determined as shown in Figure F.4-5.

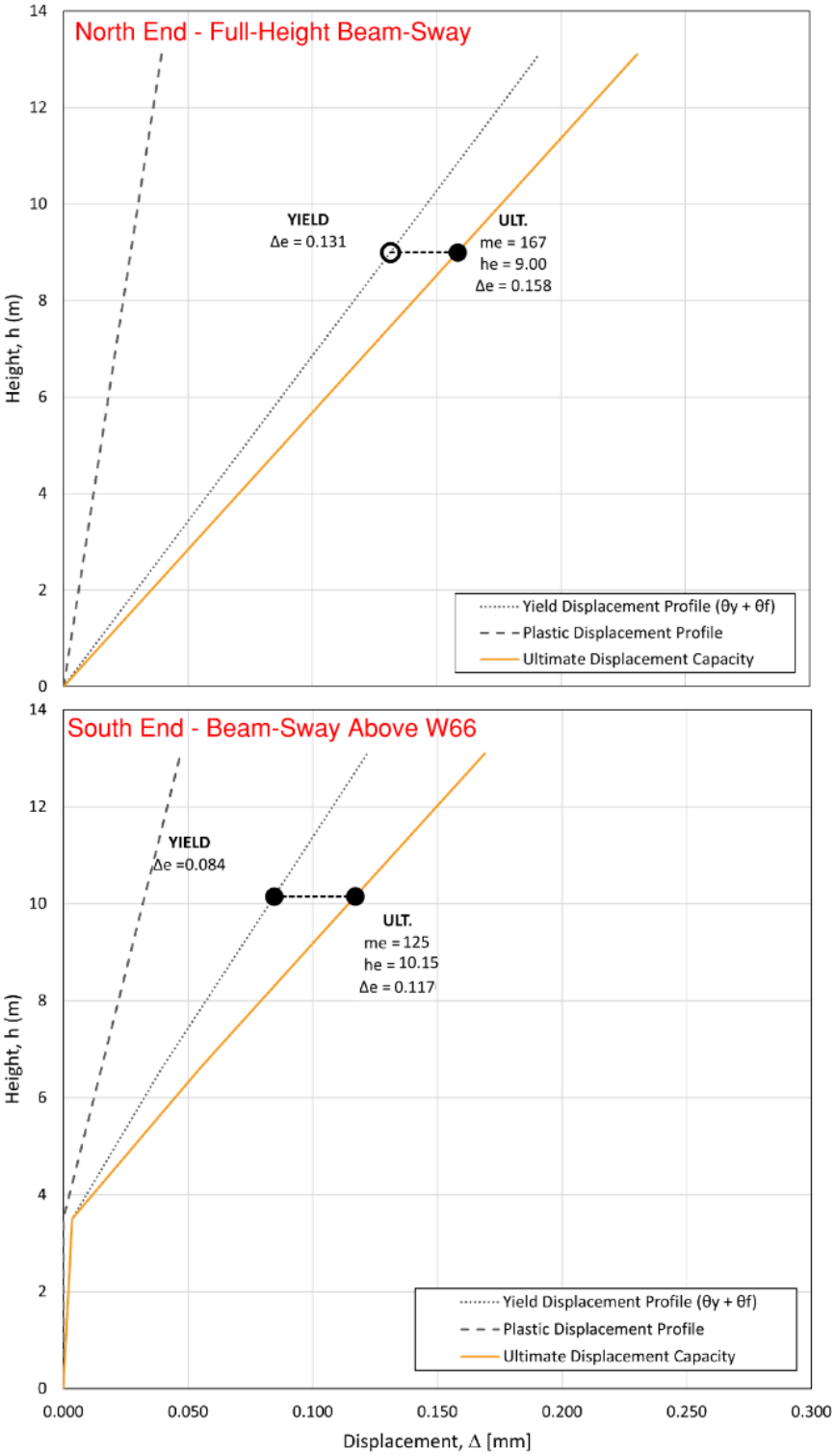

Figure F.4-5: Displacement profiles of the Staircase Structures acting out-of-plane at each end of the structure

The structure's response curve was estimated assuming that:

- The average lateral base shear capacity of the two possible mechanisms would sufficiently represent the behaviour of the Staircase Structures acting out-of-plane.
- The total displacement capacity would be limited by the drifts at the southern end of the structures.
- The final building response curve could be then developed by combining the effective SDoF properties and base shear capacity associated with each mechanism as shown in Table F.4-6 below.

Table F.4-6: Building response curve for the Staircase Structure acting out-of-plane.

	Δ_y [mm]	Δ_{cap} [mm]	V_b [kN]	m_e [tonnes]	s_A [g]
Northern end – Full-height beam sway	131	158	197	167 (78%)	0.120
Southern end – beam-sway above W66	84	117	241	125 (59%)	0.197
Bilinearization for averaged response (shown in Figure F.6-1)	102	117			0.317

F.5 SEISMIC INPUT AND RESPONSE

F.5.1 Seismic Input Parameters and Response Spectra

The seismic loading parameters used in the assessment are similar for the Auditorium and are shown in Table C-7. The seismic loading parameters allowed for the development of an Acceleration-Displacement Response Spectrum (ADRS), in accordance with Section C3.5 of the Engineering Assessment Guidelines that represented the ULS seismic demand at the site (per NZS 1170.5:2004). The ADRS curve could then be adjusted considering both the level of damping and structural performance (S_p) factor associated with each possible mechanism.

F.5.2 Structural Performance (S_p) Factor

The Sp factor allows for a reduction in the overall seismic demand to account for several factors that are not easily quantified and are not directly accounted for in the design process.

The factor is related to the ductility of the system. Where a larger displacement ductility results in an overall larger reduction of the seismic demands. The S_p factor is determined in accordance with Section 4.4 of NZS1170.5:2004. For structures that are assessed using the direct displacement-based assessment (DDBA) methodology, the S_p factor is actually applied in two-fold (Marriott 2018). Therefore, the 'DDBD corrected' S_p factor can be determined using the equation proposed by Marriott (2018) below,

$$S_{p,DDBD} = (1 + S_p)/2$$

In the assessment an S_p factor of unity was used, as the ductility of the system was not known until the completion of the SLaMA analysis. Instead, the S_p factor has been used to reduce the damped spectrum which will result in a similar level of scaling of the results, and as such, in a similar level of scaling of the %NBS.

F.5.3 Damping

Two reduced spectra are shown in the ADRS. These have been reduced by spectral damping reduction factors as per Section C3.3 of the Engineering Assessment Guidelines.

Appropriate damping for the Staircase Structure primary lateral system has been determined in in accordance with Section C2D.3.2 of the Engineering Assessment Guidelines. In the out-of-plane direction, high (H) hysteretic damping appropriate for concrete frames (deformed bars) has been used. In the in-plane direction, Low (L) hysteretic damping appropriate for shallow footing rocking systems has been used.

F.5.4 Global Building Displacements and Drifts

The displacement profile of the staircase structures were determined based on the displacement-based assessment procedure carried out in accordance with Section C2.4.2 of the Engineering Assessment Guidelines. The estimated displacement profiles and inter-storey drifts are shown in both directions. The different possible displacement profiles at each end of the staircase structure under out-of-plane loading are also shown. In reality, the displacement profile will vary between the two displacement profiles along the length of the primary walls.

Displacements profiles are shown (refer Table F.5-1 and Table F.5-2) and related to ULS demand i) at yield, ii) when a significant life safety hazard is expected to occur, and iii) for 100%NBS_{IL3} demands. The profile at 100%NBS_{IL3} was estimated considering the spectral demand for the critical mechanism. The inelastic displacements associated with the critical mechanism were then further extended beyond life safety to match the spectral displacement demand at 100%NBS_{IL3}.

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Table F.5-1: Overview of displacements and drifts of the Independent Staircase Structures in the In-Plane Direction.

Level	Height	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]
		30%ULS(Yield)		72%NBS$_{IL3}$ (Life Safety)		100%NBS$_{IL3}$	
Level 1	RL + 2.26m	0	0.6%	0	2.5%	0	2.7%
Stair 1/2 (mid-landing)	RL + 4.71m	16		61		66	
Level 4	RL +12.06m	63		245		262	
Roof	RL +15.36m	84		328		350	

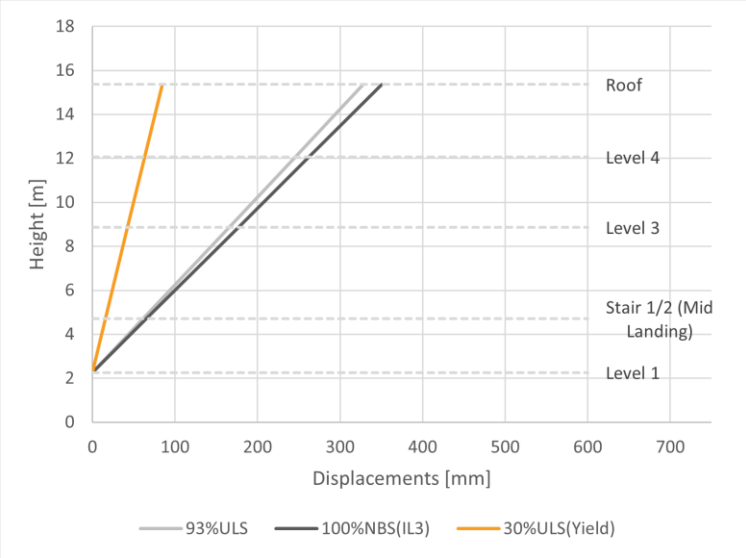

Figure F.5-1: Displacement profile (Staircase Structures - in-plane)

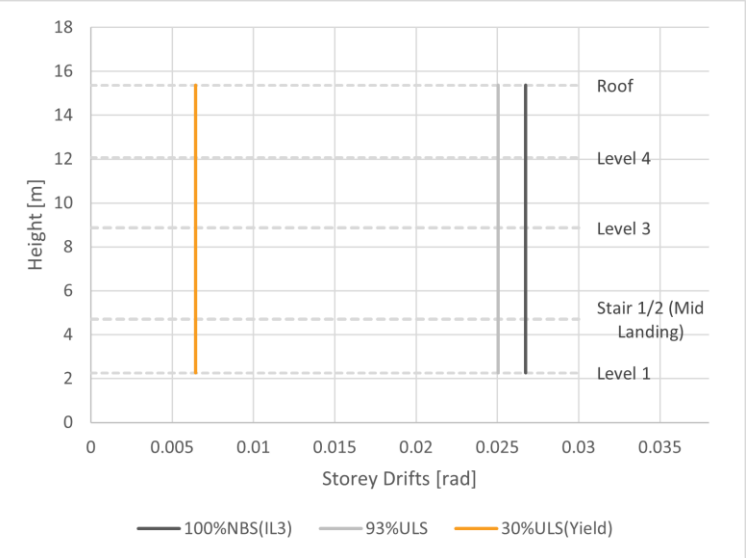

Figure F.5-2: Inter-storey drifts (Staircase Structures - in-plane)

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Table F.5-2: Overview of displacements and drifts of the Independent Stair Structures in the out-of-plane direction - northern End.

Level	Height	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]
		42%ULS (Yield) northern end		42%NBS_{IL3} (Life Safety) northern end		100%NBS_{IL3} northern end	
Level 1	RL + 2.26m	0	1.5%	0	1.5%	0	3.2%
Stair 1/2 (Mid-Landing)	RL + 4.71m	36		36		79	
Level 4	RL +12.06m	143		143		318	
Roof	RL +15.36m	191		191		425	

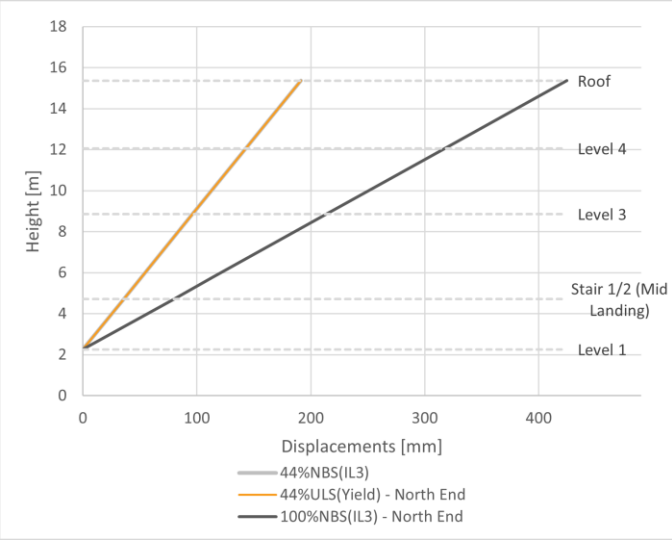

Figure F.5-3: Displacement profile (Staircase Structure - out-of-plane - northern End).

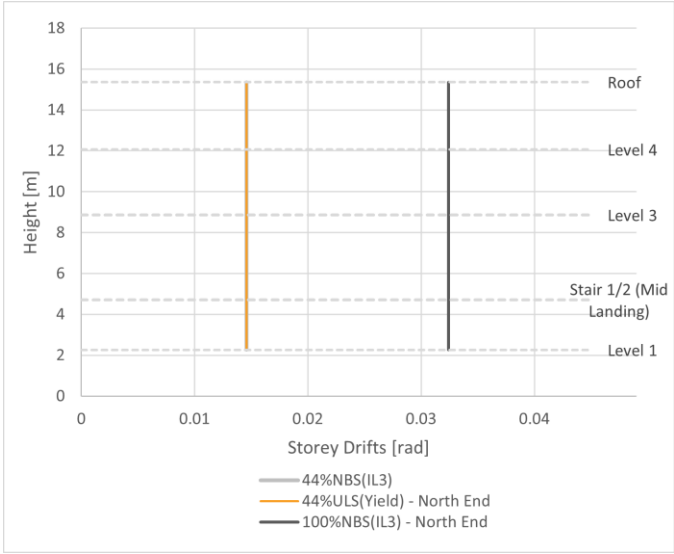

Figure F.5-4: Inter-storey drifts (Staircase Structure - out-of-plane -northern End)

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Table F.5-3: Overview of displacements and drifts of the Independent Stair Structures in the out-of-plane direction - southern end.

Level	Height	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]	Displ. [mm]	Drifts [%]
		40%ULS (Yield) southern end		42%NBS$_{IL3}$ (Life Safety) southern end		100%NBS$_{IL3}$ southern end	
Level 1	RL + 2.26m	0	0.1%	0	0.1%	0	0.1%
Stair 1/2 (Mid-Landing)	RL + 4.71m	2	0.9%	2	1.3%	2	2.9%
Level 2	RL +5.76m	4	1.2%	4	1.7%	4	4.0%
Level 3	RL +8.86m	39	1.3%	55	1.8%	124	
Roof	RL +15.36m	122		170		385	

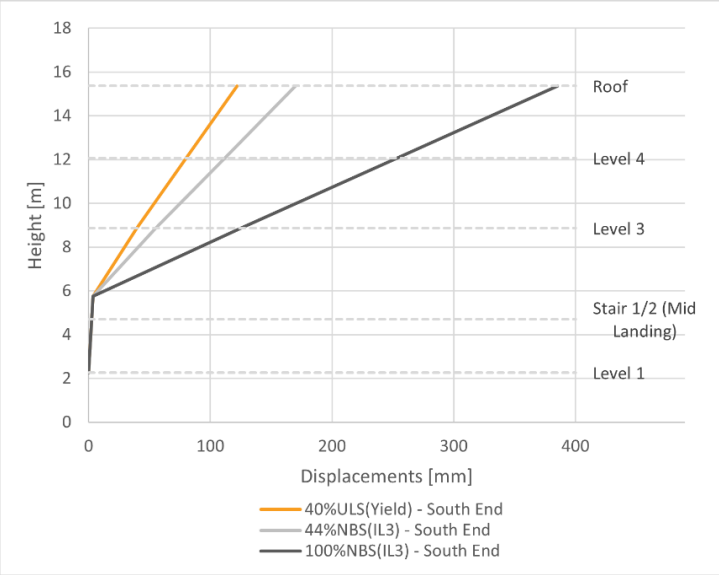

Figure F.5-5: Displacement Profile (Staircase Structure - out-of-plane - Southern end).

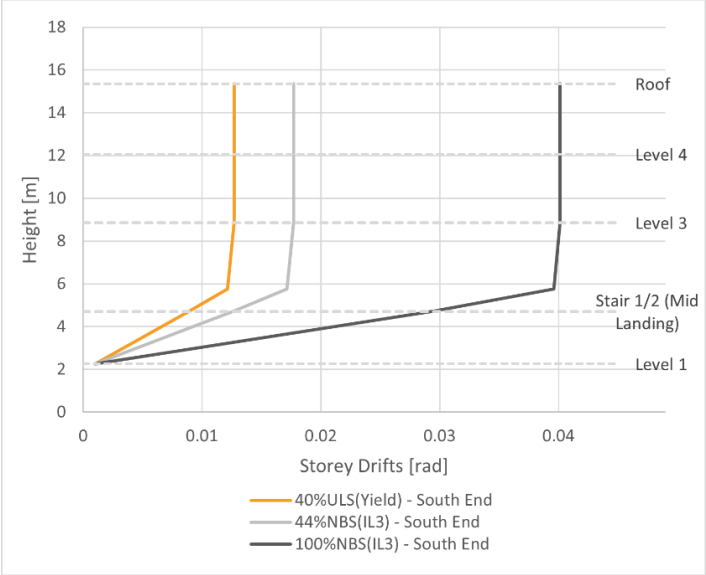

Figure F.5-6: Inter-Storey Drifts (Staircase Structure - out-of-plane - Southern end).

F.6 EVALUATION

This section contains the evaluation of the primary lateral system using the SLaMA method, and secondary structural or non-structural elements not directly modelled but are included in the scope of this assessment.

F.6.1 Primary Lateral System

The seismic performance of the primary lateral systems could be evaluated with respect to the seismic demand as discussed in Section F.5, considering the possible force-displacement response in both directions as discussed in Section 0.

Push-over plots

The structure's capacity is determined using the SLaMA technique in accordance with Step 6 of Appendix C2A of the Engineering Assessment Guidelines. This involves plotting the push-over plots for the structure on the Accelerations Displacement Response Spectra (ADRS). A line is extended from the origin through the point of ultimate displacement and associated spectral acceleration to intersect with the ADRS curve. The seismic score is the ratio of the displacement at the intersection with the ADRS curve and the Ultimate displacement.

Figure F.6-1 and Figure F.6-2 show the response of the building considering the possibly critical inelastic mechanisms in each direction. The corresponding earthquake scores are summarised in Table F.6-1.

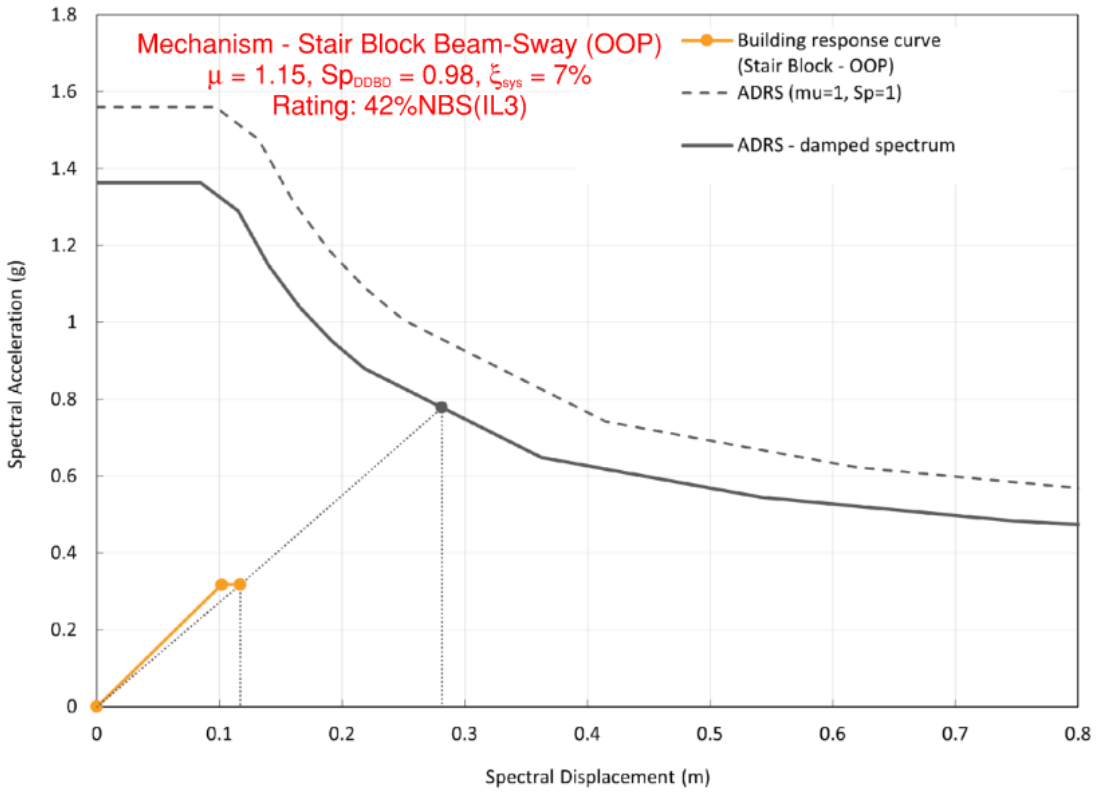

Figure F.6-1: Plot of ADRS and building response curve associated with the inelastic mechanisms of the Staircase Structure in the out-of-plane (E-W) direction.

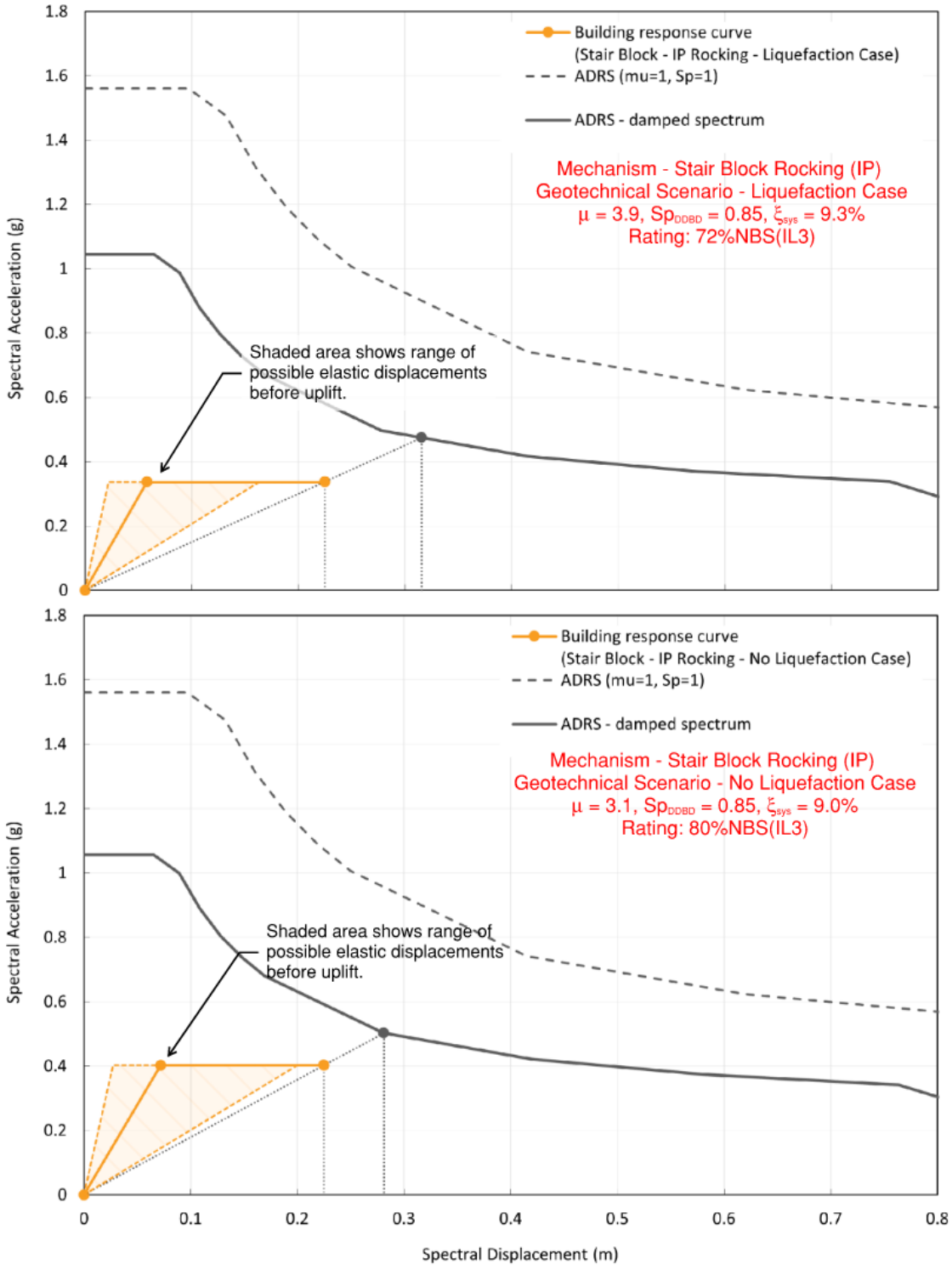

Figure F.6-2: Plot of ADRS and building response curve associated with the inelastic mechanisms of the Staircase Structure in the in-plane (N-S) direction.

Table F.6-1: Capacity of the Staircase Structures from SLaMA assessment

	Ultimate Displacement Capacity [mm]	Ultimate Displacement Demand [mm]	%NBS$_{IL3}$
Out-of-plane direction: - Beam-sway mechanism	117	280	42
In-plane direction – pile tensile exceedance (Liquefied ground properties)	225	315	72
In-plane direction – pile tensile exceedance (Undegraded ground properties)	225	280	80

As shown in Figure F.6-2, the variability in pile stiffnesses may impact both the point of uplift and subsequent drift demands when rocking initiates in the Staircase Structures under in-plane loading. Considering the building response curves developed above, the range of possible structural performance of the Staircase Structures is further discussed below.

The initial stiffness and overturning moment capacity of the foundation system both influence the %ULS demand at which rocking was estimated to occur. It was found that uplift may occur between 25-55%ULS based on upper/lower bound estimates, and between 30-40%ULS based on probable pile properties.

Ultimate drift demands at 100%ULS ranged from 2.3-2.6% drifts based on probable pile properties. It was noted that drift demands will likely increase beyond this if stiffness values tend toward the lower bound estimates. The additional drift demands would be imposed as rigid-body rotations which should have limited effect on the internal actions within the superstructure. The stability of the structure due to P-Delta effects don't appear to be sensitive to possible increased displacement demands. The stability coefficient (as defined in Section 6.5 of NZS1170.5:2004), was found not to exceed 0.1 until drifts of over 4.0%.

Based on the findings above, it considered acceptable to use the probable pile properties when assessing the rocking mechanism in accordance with the Engineering Assessment Guidelines. This resulted in an earthquake score of 40%NBS$_{IL3}$ for the Staircase Structures, which was limited by the performance in the out-of-plane direction.

F.6.2 Evaluation of Seismic Pounding

Seismic pounding was assessed in accordance with Appendix C2B Scenarios in the Engineering Assessment Guidelines. Inspection of the structures on site indicated two possible pounding vulnerabilities as shown in Figure F.6-4.

Pounding at Level 4 with Auditorium (walls in-pane motion)

The link slab elements located at Level 4 have a sliding connection as shown in Figure F.6-3 below. Flat steel plates were used to provide a sliding connection, designed with a 25 mm gap and 75 mm available seating. The link bridge sits approximately 10m above the foundations so at drifts of approximately 0.25% and 0.75%, the designed gap and seating capacity will used up, respectively. Considering that drifts over 2.5% are expected at 100%NBS$_{IL3}$ in the staircase structure alone under in-plane loading, seismic pounding can be expected. Scenario 2 (aligned floors with different floor masses) does not present design checks for the lighter building as it drifts into the stiffer one. As the Auditorium is the stiffer structure, Scenario 3 (aligned floors with building height difference) was considered applicable for checking the Staircase Structure. This means that 20% of the storey shear force of the staircase structure was assumed to be transferred through the link slab element from the Staircase Structure. For detailed assessment of link slab and load transfer

due to seismic pounding into the Auditorium, see Section C4.5. Consideration was also given to the seating capacity of link slab element (and the consequence of seating loss).

Figure F.6-4: Plan showing potential seismic pounding locations of the stair structure (original structural set S7)

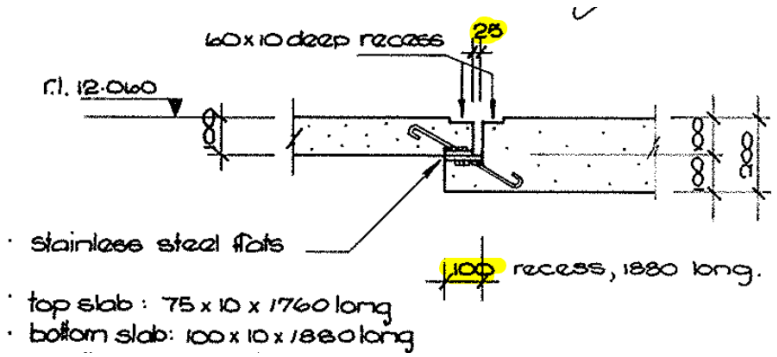

Figure F.6-3: Detail showing the link slab sliding connection (original structural set S25)

Pounding of external face of wall Auditorium (wall out-of-plane motion)

At both Level 2 and 4 there is only a 100 mm gap provided between the end of the external face of the wall and the frame extensions at Bays 6 and 6a of the Auditorium (refer Figure F.6-4). This gap is expected to be used up quickly (especially at Level 4) due to the low stiffness of the structure when acting out-of-plane.

Seismic pounding is expected to transfer load between the wall, through the column of the extended frames and then into the cast insitu stairs which are vertically offset from the floors at both Level 2 and Level 4 of the Auditorium. Noting that the stairs are cast integral with the Auditorium floors at Level 2 and Level 4.

As the stairs are not aligned with the suspended floors in the Staircase Structures, Scenario 1 (column-to-floor pounding) of Appendix C2B of the Engineering Assessment Guidelines has been used. However, only the corner of the 400 mm thick wall will have shear demands imposed due to seismic pounding so complete shear of the wall out-of-plane is unlikely. To assess whether a local exceedance of shear capacity in the

wall could occur due to seismic pounding, the local punching shear capacity of the wall out-of-plane was calculated and compared to the superstructure demands in the Staircase Structure.

The local punching shear capacity of the wall out-of-plane was determined in accordance with Section 12.7.2 of NZS3101:2006. The critical shear section was estimated considering one free edge. It considered a 250x500mm bearing area in the stair, through the 400x500mm column, and then into the 400mm thick wall (see Figure F.6-5). This resulted in a critical perimeter of, b_o = 2.75m and a probable punching shear capacity of V_n = 1500kN.

This capacity is over three times the probable base shear capacity of the staircase structures when acting out-of-plane. This suggests that local punching shear of the wall acting out-of-plane is unlikely.

Another scenario considered, is that the global response may be truncated at the northern end of the structure due to the seismic pounding (Scenario 3). If the response is truncated at the northern end of Level 2, the response will be comparable to that of the southern end mechanism due to the presence of the secondary wall (W66). The southern end mechanism has a higher base shear capacity and is already limiting the displacement capacity of the system so pounding at Level 2 was not considered a structural weakness.

If the structures' response is truncated at Level 4, the staircase structure may begin to behave as a propped cantilever at the northern end. This was not expected to reduce the overall seismic performance, however the short-period ULS elastic accelerations were used to estimate what reaction demands may be transferred through the primary wall into the Auditorium floor. The short-period horizontal design coefficient of, C_d =1.56 could be used to conservatively estimate the reaction force assuming approximately half the total weight of the structure was propped at Level 4, resulting in a shear demand of, $V^* = 1/2 \cdot (1.56 \cdot 1950\ kN) = 1520\ kN$. This was comparable to the local punching shear capacity so was not considered a structural weakness.

Based on the above, seismic pounding was not considered a structural weakness at Level 2 or Level 4.

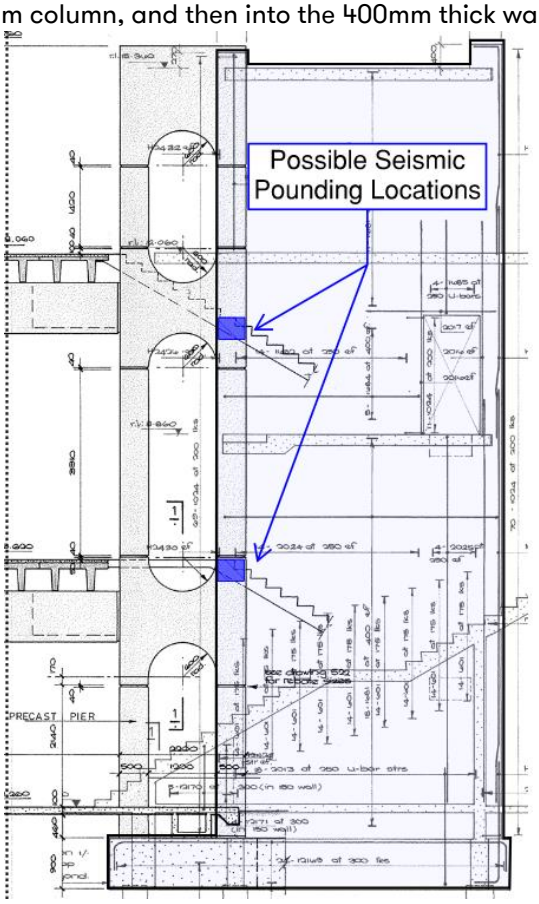

Figure F.6-5: Elevation showing possible seismic pounding locations (original structural set S23 & S51)

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F.7 SUMMARY OF IDENTIFIED STRUCTURAL WEAKNESSES

Structural Weaknesses are those deficiencies identified which have an assessed seismic rating of less than 100%NBS_{IL3}. These are listed in Table F.7-1.

Table F.7-1: Structural Weaknesses identified in the assessment of the primary structure of the Independent Staircase Structures

Building Element	Structural Weakness	%NBS_{IL3}
Walls in out-of-plane direction	Exceedance of joint shear capacity as a beam-sway mechanism degrades the joint shear capacity	40
Foundations in in-plane direction of the walls	Exceedance of tension capacity of piles, resulting in the exceedance of 2.5% drift limit	70

Appendix G

Technical Summary of Analysis and Evaluation of the Secondary and Non-Structural Elements (SSNS)

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Appendix G Technical Summary of the SSNS Analysis and Evaluation

G.1 INTRODUCTION

The secondary structural and non-structural aspects (SSNS) that were considered in this assessment are listed in Table G.1-1. Consideration includes a decision to include or exclude these elements from an overall %NBS Assessed Seismic Rating. The intent of the EPB Methodology is to identify and score those building parts which, should they lose support or collapse, would present an unavoidable danger that a number of people are exposed to.

We have applied Section 2.4.1 of the EPB Methodology, using the guidance in Section A4.3.2 of the Engineering Assessment Guidelines, to determine which parts are included or excluded from the rating.

Building contents are not considered in this assessment.

Table G.1-1: SSNS included or not included in the %NBS Assessed Seismic Rating

Building Part	Included?	Reason for decision to include/exclude
Precast infill cladding panels	Yes	Heavy element above Space Class I
Reinforced concrete masonry block walls	Yes	>25kg/m^2 partition walls bordering Space Class I, II and III
Lightweight façade and glazing suite around promenade and Renouf Foyer	Yes	Support frames that could fall into Space Class I
Lightweight arching skylight roof above Renouf Foyer	No	Light-weight cladding elements
Catwalks	Yes	Considered a heavy (>25kg) horizontal cantilevering element above Space Class I
Reflectors	Yes	Considered a heavy (>25kg) 'sign or billboard' with a fall height greater than 3m onto Space Class I
'Donut'	Yes	Considered a heavy (>25kg) vertical cantilevering element above Space Class I
Rigging points in the Auditorium and Renouf Foyer	Yes	Heavy plant that could lead to the element falling onto the building occupants
Stairs	Yes	Above Space Class I Providing emergency egress paths
Suspended light-weight timber stage	No	Above Space Class V
Suspended light-weight timber ceiling underneath Level 4 cantilever floor segments	No	Lightweight ceiling systems above Space Class III
Movable stage lift	No	Above Space Class V
Acoustic ceiling panels in Auditorium	Yes	Potentially unrestrained ceiling tile (>7.5kg) above Space Class I

Included SSNS elements identified as structural weaknesses (scoring < 100%NBS$_{IL3}$), are listed in Table G.1-2.

Table G.1-2 Seismic Rating of potential Structural Weaknesses in the secondary and non-structural elements

Building Element	Structural Weakness	%NBS$_{IL3}$
Catwalks structures	Exceedance of capacity of in-plan bracing.	20-30
Wall reflectors	Concrete break-out from excessive forces on the post-installed anchors	20
Promenade roof & façade system	Exceedance of deformation capacity of existing glazing suite	25
Precast infill panels between the upper precast portion of the bay walls	Limited displacement compatibility with the primary structure of the Auditorium	25
Stairs 15/16	Loss of integrity of the landings at large inter-storey drifts.	35
Stairs 11/12	▪ Exceedance of flexural capacity from in-plane loading. ▪ Lack of displacement compatibility of lower and upper stair flight sliding connections. ▪ Differential ground movement from lateral spreading of the ground exceeding the seating capacity.	40-45
Stairs 7/8	Differential ground movement from lateral spreading of the ground exceeding the seating capacity.	45
Stairs 13/14	Integral cast insitu connection leading to excessive flexure of stair flight under imposed displacements.	45-60
Concrete masonry block walls	Exceedance of out-of-plane capacity of block walls at Level 4 around services shaft	50
Stairs 9/10	Exceedance of both displacement capacity and structural capacity at comparable seismic demand due to: ▪ Lack of displacement compatibility of level 2 sliding connection. ▪ Exceedance of flexural capacity of lower stair flight due to significant mass eccentricity under inertial loads.	50
Rigging points in the Renouf Foyer	Exceedance of flexural capacity under 'Parts and Components' loading	70
Precast double-tee flooring at Level 2 of the lobby area in the Auditorium	Loss of seating due to excessive drifts of the primary structure	80

Inter-Storey Drift Demand Summary

The inter-storey drift demands used as part of the assessment of SSNS elements is summarised in Table G.1-3 below, with the coordinate system used shown in Figure G.1-1. Displacements were extracted for critical levels of seismic demand to clearly define the different risk thresholds in accordance with section A.2 of the Engineering Assessment Guidelines. Displacements for the Auditorium vary up the height of the structure (and at different ends of the structure), therefore, a range of drifts are shown. The principal directions of the independent staircase structures do not align with the Renouf Foyer, as such, a rotated co-ordinate system is used for the stair structures as shown on Table G.1-3.

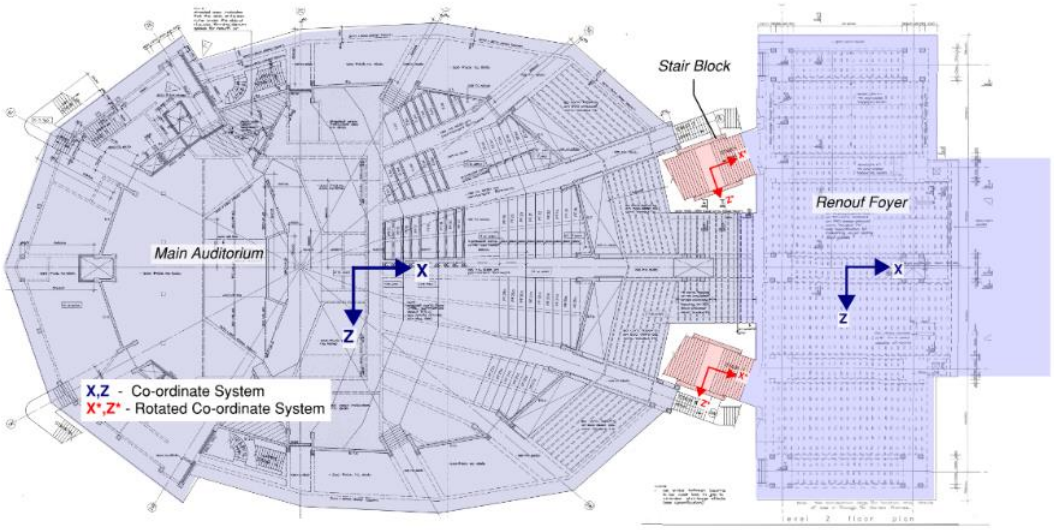

Figure G.1-1: Coordinate system used for each of the seismically separated structures forming the Michael Fowler Centre.

Table G.1-3: Inter-storey drift demands for the seismically separated structures within the MFC at varying levels of earthquake shaking.

	34%NBS$_{IL3}$		67%NBS$_{IL3}$		100%NBS$_{IL3}$	
Auditorium	**S_p**	**Drift [%]**	**S_p**	**Drift [%]**	**S_p**	**Drift [%]**
X-Direction	1.00	0.10 - 0.30	-	-	0.7	0.35 - 0.65
Z-Direction	1.00	0.05- 0.45	-	-	0.7	0.15 – 1.25
Renouf Foyer [1]	**S_p**	**Drift [%]**	**S_p**	**Drift [%]**	**S_p**	**Drift [%]**
X-Direction	0.95	0.9	0.70	1.6	0.7	2.3
Z-Direction	1.00	1.45	0.70	3.1	0.7	4.9
Staircase Structures	**S_p**	**Drift [%]**	**S_p**	**Drift [%]**	**S_p**	**Drift [%]**
X*-Direction	0.97	0.7	0.7	1.34	0.70	2.7

[1] Values are shown for drifts between Level 1 & Level 2 (critical)

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G.2 PRECAST FLOORING ASSESSMENT

In November 2018, Holmes Consulting LP were engaged to undertake a review of the Dycore units and the roof structure of the Auditorium and to provide options on next steps to take to reduce the potential seismic risk associated with the structural response of the roof.

This was followed by calculations and an engineering design in 2019 which involved the seismic strengthening of roof. It involved providing additional seating to the Dycore units and addressing the potential issues with 'alpha' slabs, torsion, and shears from excessive negative moment in the northern and southern extends of the Auditorium. This engineering design was implemented and constructed in 2020.

In October 2019, Holmes Consulting LP were then engaged to also undertake a review of the Auditorium promenade double-tee floors, the precast roof and the precast seating units inside the Auditorium.

This was followed by an engineering design in 2020 to address the most vulnerable precast flooring units in the Michael Fowler Centre, which included the double-tee flooring and rib infill flooring in the Renouf Foyer as well as the flange-hung double-tee units in the Auditorium. There was seismic strengthening focussed on the locations with flange-hung double-tee units which were reinforced with the 'loop bar detail', as well as the precast floor units in the Renouf Foyer, which is considered a high-drift building. This engineering design was implemented and constructed in 2020.

This section reviews the calculations and assumptions that were made at the time of those independent assessments in the broader context of this DSA.

G.2.1 Assessment of the Roof

The 2018 assessment of the roof structure involved a visual inspection of the internal voids and web elements, to observe the potential for web splitting or cracking. Some panels were also inspected for signs of damage at the seating on the roof trusses. None of the units inspected at the time showed any signs of significant cracking in the webs to the internal voids, nor at the seating. Some cracking was observed at other locations, though these were assumed to be resulting from shrinkage cracking and deemed unlikely to affect the behaviour of the structure.

The analysis that was undertaken involved a micro strut-and-tie model of the roof diaphragm to understand the potential differential displacements that the roof would see.

It is noted that in comparison to this Detailed Seismic Assessment, the 2018 analysis model was setup in isolation and did not capture the behaviour of the global structure.

G.2.2 Securing of Dycore Units in Auditorium Roof

The 2019 seismic securing works to the Dycore units involved a series of additional seating angles that were welded onto the roof trusses, and a series of catch-frames under the 'alpha' slabs. The calculations that were undertaken considered the outcome of the 2014 seismic assessment, which predicted a maximum ULS drift of 1.1% at the roof level of the Auditorium.

The calculations were undertaken in accordance with Yellow Chapter C5. It accounted for the following modes:

- Loss of seating: the calculations of the original seating capacity was determined on the basis of the parameters listed in Table G.2-1, which determined the total available seating capacity of 45mm would be exhausted. Conclusively, additional 100x10 EA seating angles were provided to provide additional seating to the units to increase the total seating capacity to 145mm.

Table G.2-1: Parameters considered in assessing the seating capacity of the Dycore units

Parameter	Commentary	Required seating [mm]
Plastic hinge elongation	Not applicable	-
Support beam rotation	Not applicable	-
Plastic strain in starter bars	Considered to be insubstantial, as the 661-gauge mesh across the seating is unlikely to develop notable 'push' on the Dycore unit	-
Imposed deformation of wall building	Taken as the global drift over the height of the roof structure (i.e. 1.1% drift over 215mm = 2.4mm)	3
Construction tolerance	Not observable on site – assumed at 20mm	20
Support and unit spalling	Taken from Figure C5E.11 of the Engineering Assessment Guidelines for unit spalling only	17
Creep, shrinkage and thermal movement	Ignored, as less than the unit spalling	-
Total		**43**

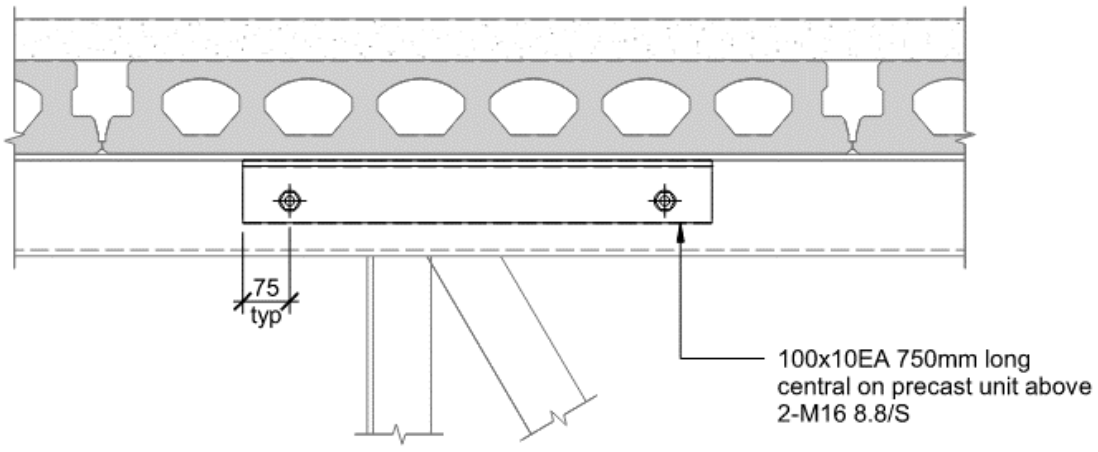

Figure G.2-1: Typical seating angle provided to increase seating capacity of Dycore units

- Shears from excessive positive moments: was not found to be critical, as potential cracks at 1.1% drifts were found to be minimal.
- 'Alpha' slab behaviour: The recommendations of Yellow Chapter C5 stipulate a maximum differential displacement of 8mm, before web splitting could occur. At 1.1% drift, these differential displacements were far exceeded, and, as such, an alternative gravity support was provided by means of a catch-frame.
- Torsion: Elements that connected directly into the bay walls of Bays 1(a) and 7(a) could see those walls drift transversely to units, exposing them to torsion. At 1.1% drift, these acceptable levels of displacements were exceeded, and, as such, an alternative gravity support was provided by means of a catch-frame.
- Shears from excessive negative moments were determined to be possible at units which overlapped with the Ø28mm starter bars that extended from the 'Terriers'. The potential negative moment capacity that those locations would exceed the capacity of the Dycore unit and, therefore, an alternative gravity support was provided by means of a catch-frame. Note that negative moment failure was found to only be potentially critical for Dycore units meeting specific criteria. One of these criteria relates to the length of the Ø20mm or Ø28mm bars over the edge of the Dycore units. If this length is too short, as shown on Figure C5E.14 of the Engineering Assessment Guidelines, then negative moment failure may be an issue. Other criteria when negative moment failure was considered an issue, if there were two Ø28mm bars over the edge of the Dycore unit.

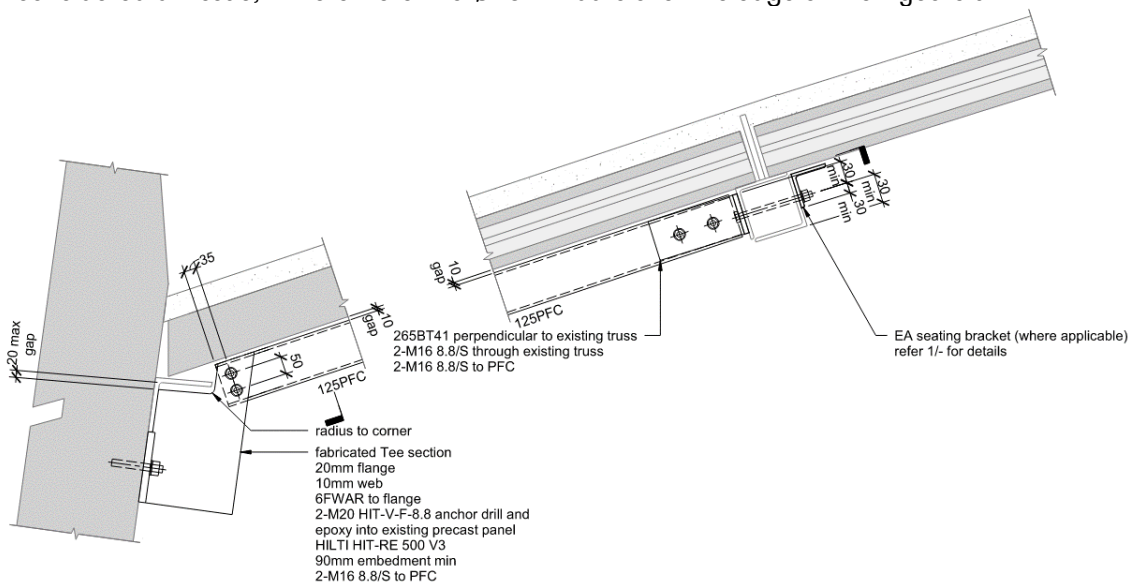

Figure G.2-2: Catch-frame provided under 'alpha' slabs and Dycore units susceptible to torsion or shear from excessive negative moments

G.2.3 Assessment and Securing of Double-Tee Units in the Auditorium

The double-tee units in the Auditorium were split into two categories based on their seating condition, either flange-hung or web supported.

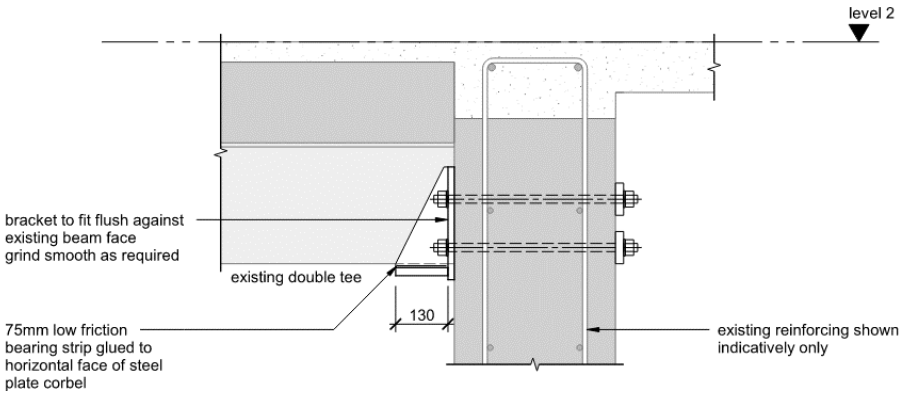

Figure G.2-3: Typical bracket to provide additional seating to the flange-hung double-tee units

The flange-hung double-tees are located at the radial gridlines C, 5E and 5W on Level 2 and at gridlines 2E and 2W on Level 4. Their detailing was considered to include the loop-bar detail shown in Figure C5E.28 of the Engineering Assessment Guidelines. Therefore, this configuration was not specifically assessed and instead strengthened regardless by a seating bracket which provided 130mm of seating.

All seating brackets that were provided were designed for a vertical accelerations to the Parts & Components provisions of Chapter 8 of NZS1170.5:2004 (including Amendment 1).

A representative selection of the web-supported double-tees were assessed against their seating capacity in accordance with Section C5E.6.2 of the Engineering Assessment Guidelines. The parameters that were considered for this assessment are given in Table G.2-2.

The double-tee units are assumed to have shear reinforcement in the webs and are therefore not typically susceptible for shear from excessive negative moments, as such this was excluded from the assessment.

Table G.2-2: Parameters considered in assessing the seating capacity of the web-supported double-tees

Parameter	Commentary	Required seating [mm]
Plastic hinge elongation	Not applicable	-
Support beam rotation	Taken as half the beam height multiplied by the %drift (500mm * %drift)	Var.
Plastic strain in starter bars	Considered to be insubstantial, as the 665-gauge mesh across the seating is unlikely to develop notable ‘push’ on the double-tee unit	-
Imposed deformation of wall building	Taken as the global drift over the height of the floor structure (i.e. for a 500mm deep double-tee with 50mm concrete topping this is 550mm multiplied by the %drift)	Var.

Parameter	Commentary	Required seating [mm]
Construction tolerance	Not observable on site – assumed at 20mm	20
Support and unit spalling	Taken from Figure C5E.11 of the Engineering Assessment Guidelines for both the unit and the support spalling, depends on the %drift	Var.
Creep, shrinkage and thermal movement	Taken as lower bound, as generally less than the unit spalling	6

The calculations were iterated until they exceeded the displacement capacity and the resulting drifts for the various double-tee configurations are shown in Table G.2-3. In this table, the limiting drift is generally based on the assessment from 2019, which was reviewed to be accurate. All numbers from the 2019 assessment are highlighted in a red colour in the table.

For the locations which were the seating brackets were provide at the flange-hung detailing, the limiting seating capacity was determined based on the length of the seating bracket, minus the required seating for acceptable bearing stresses (15mm), divided by a factor of 2.

The seismic score (%NBS_{IL3}) has been determined on the drifts of the buildings that followed from this DSA. It includes a factor of 2 on the actual drifts of the Auditorium.

Table G.2-3: Summary of limiting drifts for a loss of support condition

Location	Support detail configuration	Gridline	Seating capacity [mm]	Limiting drift [%]	Building drift (x2) [%]	Seismic score [%NBS_{IL3}]
Level 2 promenade floor (Unit 1 in the X-direction)	Flange-hung	5E / 5W	130	2.64	0.80	>100
Level 2 promenade floor (Unit 1 in the X-direction)	Web-supported	6E / 6W		1.8	0.80	>100
Level 2 promenade floor (Unit 1a in the Y-direction)	Web-supported	6E/ 6W / C		1.9	2.48	**80**
Level 2 in Auditorium entry foyer (Unit 2 in the Y-direction)	Web-supported	C		2.1	2.48	**85**
Level 4 promenade floor (Unit 3 in the X-direction)	Flange-hung	2E / 2W	130	2.12 *	1.03	>100
Level 4 promenade floor (Unit 4 in the Y-direction)	Web-supported	6E / 6W / C		2.5 *	1.36	>100

** Will experience drift as taken between the storey-height between the Level 2 promenade floor and the Level 4 promenade floor*

G.2.4 Assessment and Securing of Double-Tee Units and Rib Infill flooring in the Renouf Foyer

After the securing works in the Auditorium, the same assessment was undertaken for Renouf Foyer. The typical floor units at Level 2 are double-tee units, whilst the elements in the 1989 Kitchen Extension are rib infill floor elements.

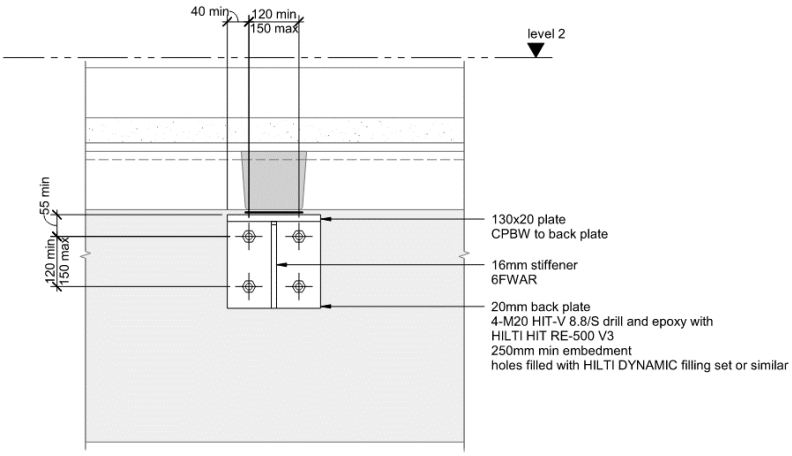

Figure G.2-4: Typical seating bracket under the rib infill flooring

The assessment was undertaken in a similar manner as to the Auditorium and identified that the all the units were potentially prone to loss of seating at a large seismic event. Consequently, a seating bracket was provided for all the units, providing 130mm of seating.

All units span in the longer (east-west) direction of the Renouf Foyer and the 130mm seating allows for a limiting drift of 1.24%, when allowing for a remaining seating for acceptable bearing stresses (15mm) and dividing the drifts of the primary structure by the factor of 2. The assessment of the Renouf Foyer found that all precast flooring has a seismic score of >100%NBS_{IL3}

G.2.5 Assessment of the Precast Seating Units in the Auditorium

The precast seating elements in the Auditorium are double-tee units which are anchored into the beam elements below it with hold-down bolts, which are restrained by a U-shaped reinforcing bar in the beam. The assessment concluded that the bolt was prone to yielding and as such, the loss of support scenario needed to be checked.

The assessment of the seating capacity included allowances for the rotation of the support beams, construction tolerances, spalling, creep and the bearing capacity. However, the conclusion was that there was ample seating capacity in the connection which was sufficiently large to prevent loss of seating.

G.3 PROMENADE ROOF AND FAÇADE SYSTEM

The roof and façade system around the perimeter of the building follow a consistent set of details with minor variations for each of the areas shown in Figure G.3-1.

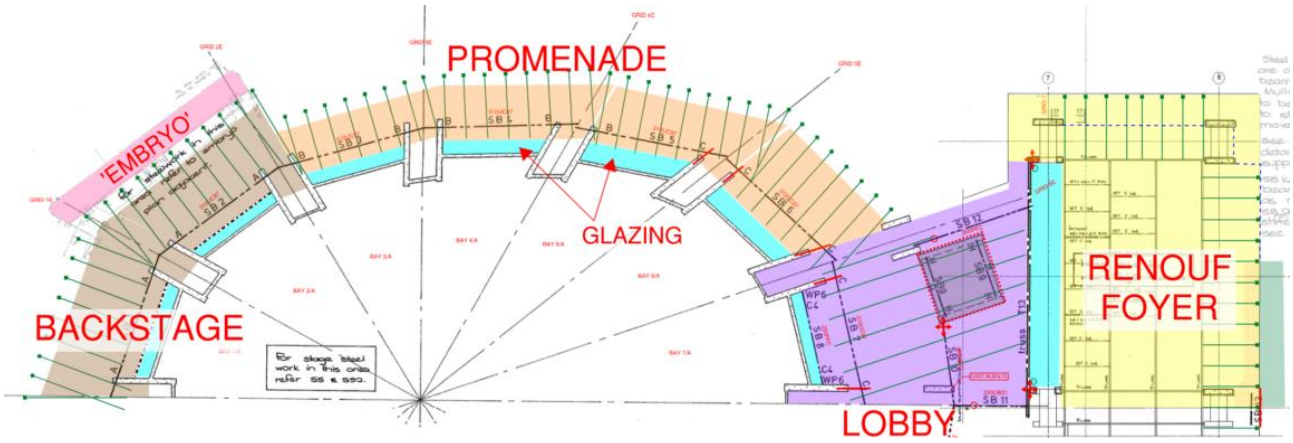

Figure G.3-1: Part plan showing key areas referred to in this section of report.

G.3.1 Description of Load Paths and Potential Structural Weaknesses

As previously noted, the roof and façade system use a consistent set of details. Therefore, the load paths are generally consistent around the building. A schematic sketch of the general load path is provided below in Figure G.3-2. Loads described as out-of-plane are acting orthogonal to the glazing (i.e. radially in plan) and loads described as in-plane are parallel to the glazing (i.e. tangentially or circumferential in plan).

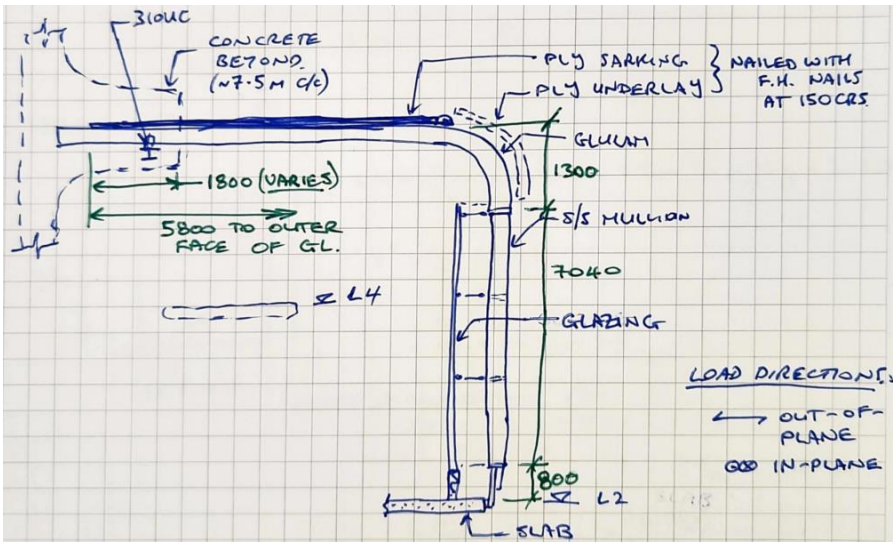

Figure G.3-2: Typical/Indicative cross section through promenade roof and facade.

Out-of-plane load path - general

The main sources of mass are the 12mm glass in the façade glazing and the build-up of timber, ceilings and services in the roof. Inertia loads associated with the glazing are transferred to the stainless-steel mullions by struts in tension and compression. The stainless-steel mullions then span vertically between the Level 2 floor diaphragm and the curved glulam rafters at the roof level of the promenades. These curved glulam rafters drag the load back towards the Auditorium's concrete structure (i.e. the primary structure).

There are two possible load paths for getting the load into the primary structure: the plywood sarking acting as a (flexible) diaphragm; or, the 310UC beam in minor axis bending. Both these elements span horizontally between the precast concrete 'terrier' outriggers. The plywood diaphragm is fixed to a timber ribbon board that is fixed to the precast concrete with M16 anchors. The 310UC is connected to weldplates cast into the precast concrete. In the general case, the drag load from the glulam rafters is applied eccentrically to the 310UC's, loading them in torsion as well as minor axis bending.

In-plane load path - general

The main sources of mass are the same as for the out-of-plane load path. Inertia loads associated with the glazing do not have dedicated structural elements to transfer inertia load up to the plywood roof diaphragm and down to the Level 2 diaphragm. Therefore, the glazing is required to resist its own inertia loads and brace itself up/down to the diaphragms. The glazing will also experience induced stresses due to inter-storey drift between the promenade roof and Level 2.

Plywood sarking on the promenade roof acts as a plywood diaphragm and extends out to an internal gutter. Beyond the gutter a plywood underlay for the stainless-steel cladding has been observed to follow the curve of the glulam rafters and come down to the top of the stainless-steel mullions (Figure G.3-3). In instances where this underlay is well detailed and well-constructed it provides a load path back to the plywood sarking/diaphragm and primary concrete structure. The plywood sarking/diaphragm is 'wedged' between the precast concrete 'Terrier' outriggers and diaphragm loads are transferred to this primary concrete structure by bearing.

Site investigations (November 2022) of a sample area below the internal gutter and curved glulam rafters identified the following key issues:

- Plywood butt joints were not made over the top of support framing
- Nails through the plywood had missed the support framing
- Parallel/paired support framing below the gutter was only lightly nailed together

These issues with the as-built detailing of the plywood and support framing mean that the curved underlay and internal gutter will not provide a continuous load path for in-plane loads. Therefore, the only remaining load path available for in-plane façade loads is for the curved section of the glulam rafters to cantilever downwards and the main span of the rafter to act in torsion. This load path is very flexible.

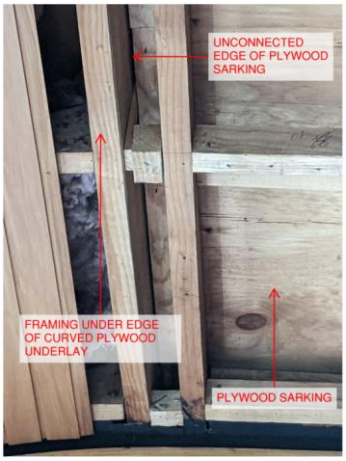

Figure G.3-3: Left: Photograph from 2021 and 2022 roof repair works showing curved plywood underlay. Centre: Photograph from 2022 DSA site investigations showing underside of gutter. Right: Photograph from 2022 DSA site investigations showing outer edge of sarking.

Backstage area

The load paths in the backstage area follow the general load paths described above, but with the following exceptions:

- The drag load in the glulam rafters is applied concentrically to the 310UC beam (see Figure G.3-4).
- There is a minor additional seismic mass associated with the 'embryo' (stair) façade. As part of roof repair works undertaken in 2021 and 2022 the roofing (light gauge steel profile) was screwed together to create a diaphragm and the roof structure was tied into the glulam rafters. These details were based on loads equating to 67%NBS$_{IL3}$ and, given the recent nature of these works, this load path has not been reassessed.

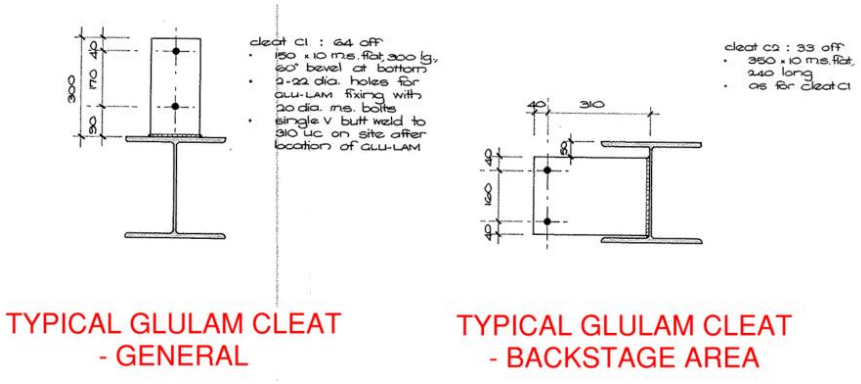

Figure G.3-4: Cleat details on 310UC (taken from 1979 Holmes, Wood, Poole and Johnstone drawings).

Promenades

The load paths in the promenade area follow the general load paths described above.

Lobby area

The main source of mass in the lobby area is the build-up of timber, ceilings and services in the roof. Inertia loads associated with the roof are resisted by the concrete primary structure of the auditorium and the Independent Staircase Structures in the lobby area. These loads are transferred into these elements by the plywood sarking and the 310UC beams – similar to the general load paths described above.

Additional complexity exists in the lobby area because vertical loads from the roof are supported by three separate structures: the Auditorium structure, the Independent Staircase Structures and the Renouf Foyer. For lateral loads the structural steel roof beams/trusses are rigidly connected to the Auditorium structure. The connections to the Independent Staircase Structures and the Renouf Foyer are explicitly detailed with sliding surfaces to accommodate differential lateral displacements between the three structures. However, the glulam rafters are rigidly connected to both the Auditorium structure and the Independent Staircase Structures.

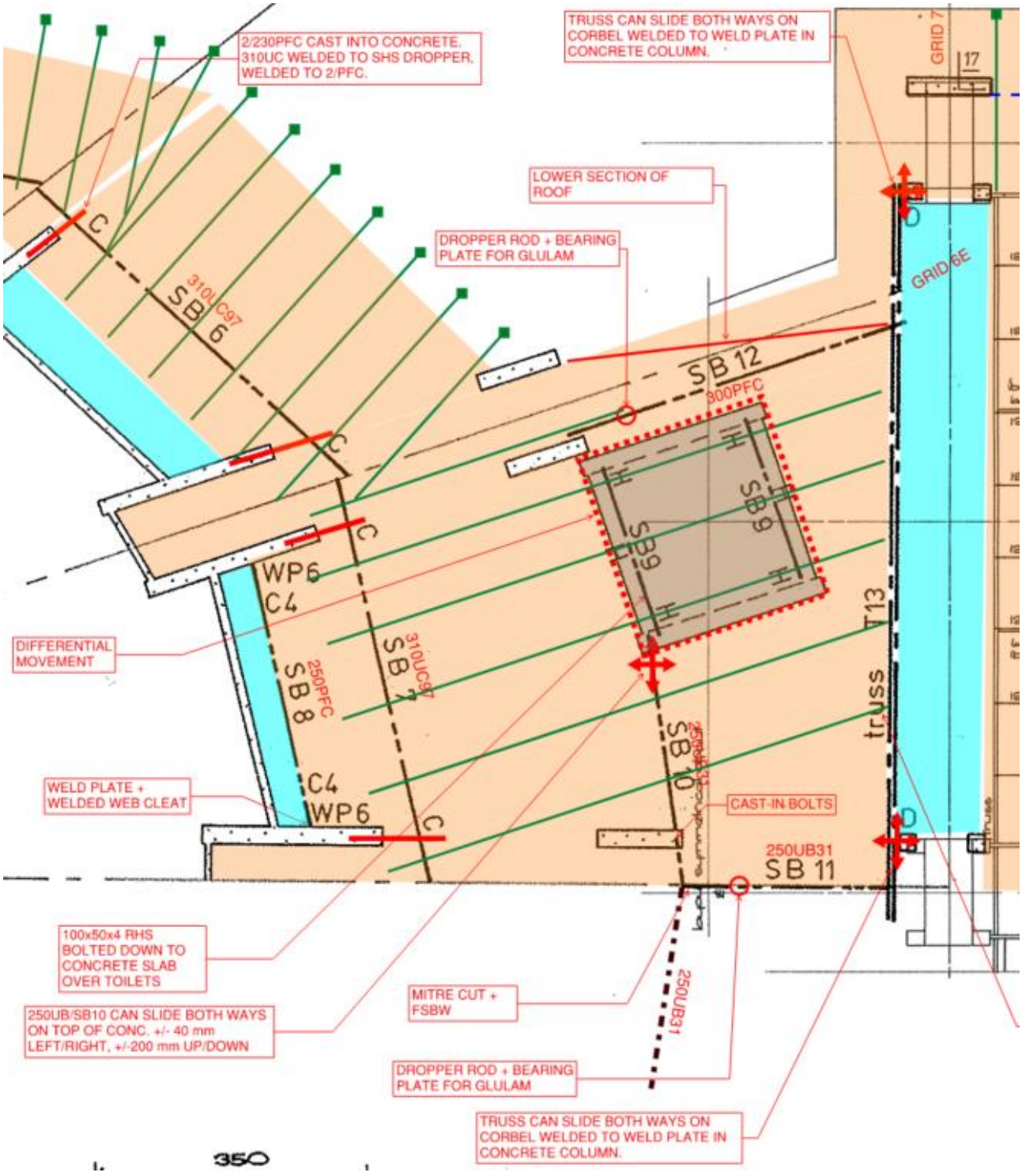

Figure G.3-5: Key plan of lobby area

Renouf Foyer

The main sources of mass are the 12mm glass in the façade glazing and the build-up of timber, ceilings and services in the roof. Inertia loads associated with the glazing are transferred to the stainless-steel mullions by struts in tension/compression. The stainless-steel mullions then span vertically between the Level 2 floor diaphragm and the curved glulam rafters at roof level. These curved glulam rafters drag the load back towards the centre of the Renouf Foyer where the plywood sarking distributes the load out to the primary structure (concrete frames). To create a complete diaphragm, the steel trusses act as drag members and chords (Figure G.3-6). These trusses have been assessed separately and are described in Section E.11.3.3.

As with the general load path described above, the glazing is required to resist its own in-plane inertia loads and brace itself up/down to the diaphragms. The glazing will also experience induced stresses due to inter-storey drift between the foyer roof and Level 2.

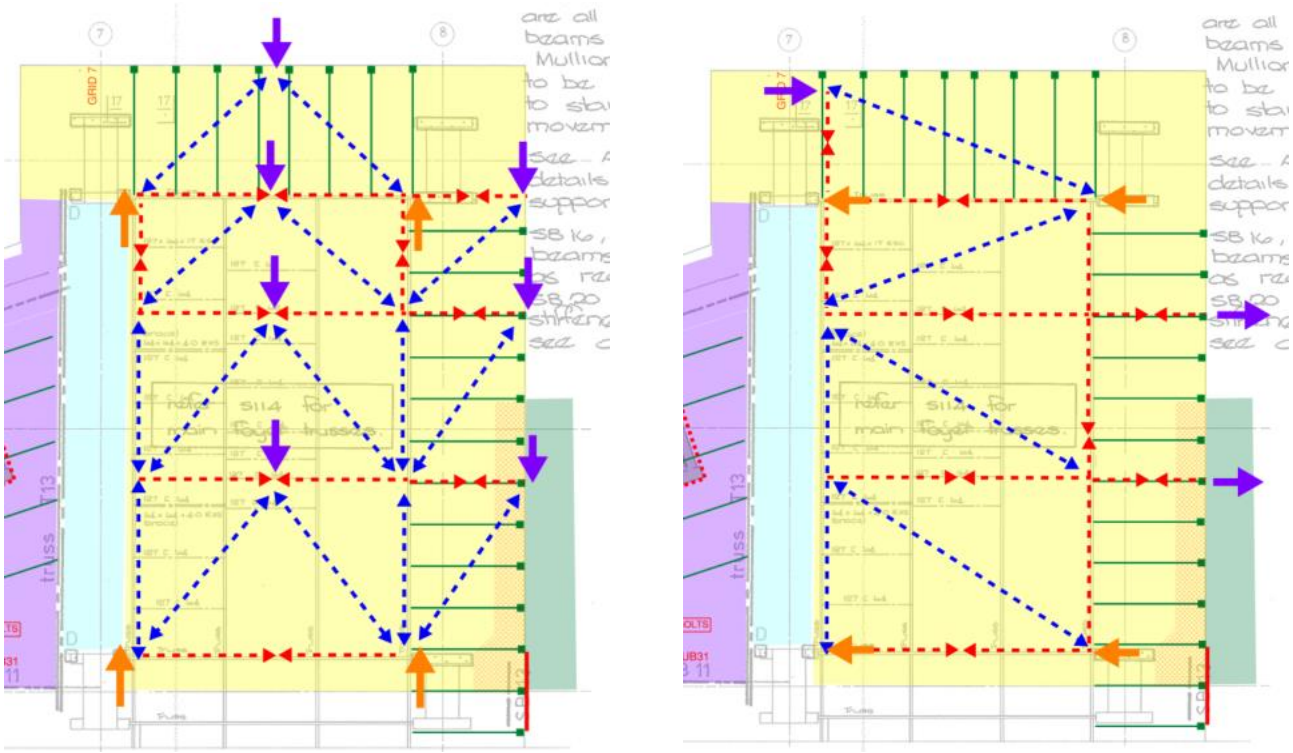

Figure G.3-6: Key part plan of Renouf foyer showing diaphragm load paths for the two principal directions

G.3.2 Method of Assessment and Seismic Rating

The SSNS elements under consideration are not considered to be in the primary load path of the structure and loads have been derived using the 'parts' approach in section 8 of NZS 1170.5.

Force-based assessment

As noted above 'parts' accelerations have been applied to determine seismic loads on the SSNS elements. These have been tracked along the load paths described in the previous section. The probable capacities of elements and their connections have been checked to the following guidelines/standards.

Element	Guideline/Standard
Stainless steel mullion	Section/member properties calculated to NZS3404:1997 methods utilising probable yield stress of 260 MPa. Checked against elastic load determined in accordance with 'Parts and Components' from Chapter 8 of NZS1170.5:2004.
Structural steel members/connections	Section/member properties calculated to NZS3404:1997 methods utilising probable yield stresses from Chapter C6 of Engineering Assessment Guidelines. Checked against nominal ductile load determined in accordance with 'Parts and Components' from Chapter 8 of NZS1170.5:2004.
Weldplates cast into precast	Development length and concrete breakout calculated to NZS3101:2006 methods utilising probable concrete strengths from Chapter C5 of Engineering Assessment Guidelines. Checked against elastic load determined in accordance with 'Parts and Components' from Chapter 8 of NZS1170.5:2004.
Post-installed anchors into concrete	Probable connection strength benchmarked against comparable modern anchors (e.g. Ramset Dynabolts) using probable concrete strengths from Chapter C5 of Engineering Assessment Guidelines. Checked against elastic load determined in accordance with 'Parts and Components' from Chapter 8 of NZS1170.5:2004.
Plywood sarking/diaphragm fixings	Fixing strength/stiffness calculated to NZS 3603 using nail size/spacing observed during 2021 maintenance works. Checked against fully ductile ($\mu \geq 3$) load determined in accordance with 'Parts and Components' from Chapter 8 of NZS1170.5:2004, which is comparable to the recommended load reduction (nominal ductility and S_p=0.5) from Chapter C9 of Engineering Assessment Guidelines.
Glulam/timber members	Section/member strengths calculated to NZS 3603 methods utilising probable material strengths from Chapter C9 of Engineering Assessment Guidelines. Torsion strength calculated using probable material shear strength from Chapter C9 and first principles torsion calculations from published literature. Checked to the recommended load reduction (nominal ductility and S_p=0.5) from Chapter C9 of Engineering Assessment Guidelines.
Glulam/timber connections	Connection strengths calculated to NZS 3603 methods utilising probable material strengths from Chapter C9 of Engineering Assessment Guidelines. Checked against nominal ductile load determined in accordance with 'Parts and Components' from Chapter 8 of NZS1170.5:2004.
Glazing system	Force-based assessment of the system not required because mass of system is less the 70 kg/m^2 (Reference: clause C10.6.8 of Engineering Assessment Guidelines)

Deformation-based assessment

The only element requiring a deformation-based assessment is the glazing. Applying clauses C10.4.5 and C10.6.8.3 of the Engineering Assessment Guidelines, it has been assessed that:

- A typical construction clearance of 10 mm between the glass and frame may be used.
- Deformation capacity of the glazing is 36 mm per panel.
- Total deformation capacity between the level 2 and roof diaphragms (3 panels high) is 107 mm.
- The glazing is pre-1980s, therefore an element performance factor, Q, of 2 may be adopted.

Consequences during lateral spreading of the ground

The consequences of lateral spread have been evaluated by considering a probable differential lateral displacement between the Auditorium and the front of the Renouf Foyer at 45%NBS$_{IL3}$. Two scenarios have been evaluated: a north-south differential and an east-west differential.

For the North-South scenario the connection of the glulam rafters to the top of the stair/toilet structure has been assessed to be the strongest connection and its probable strength exceeds the probable overstrength of the connections to the other structures.

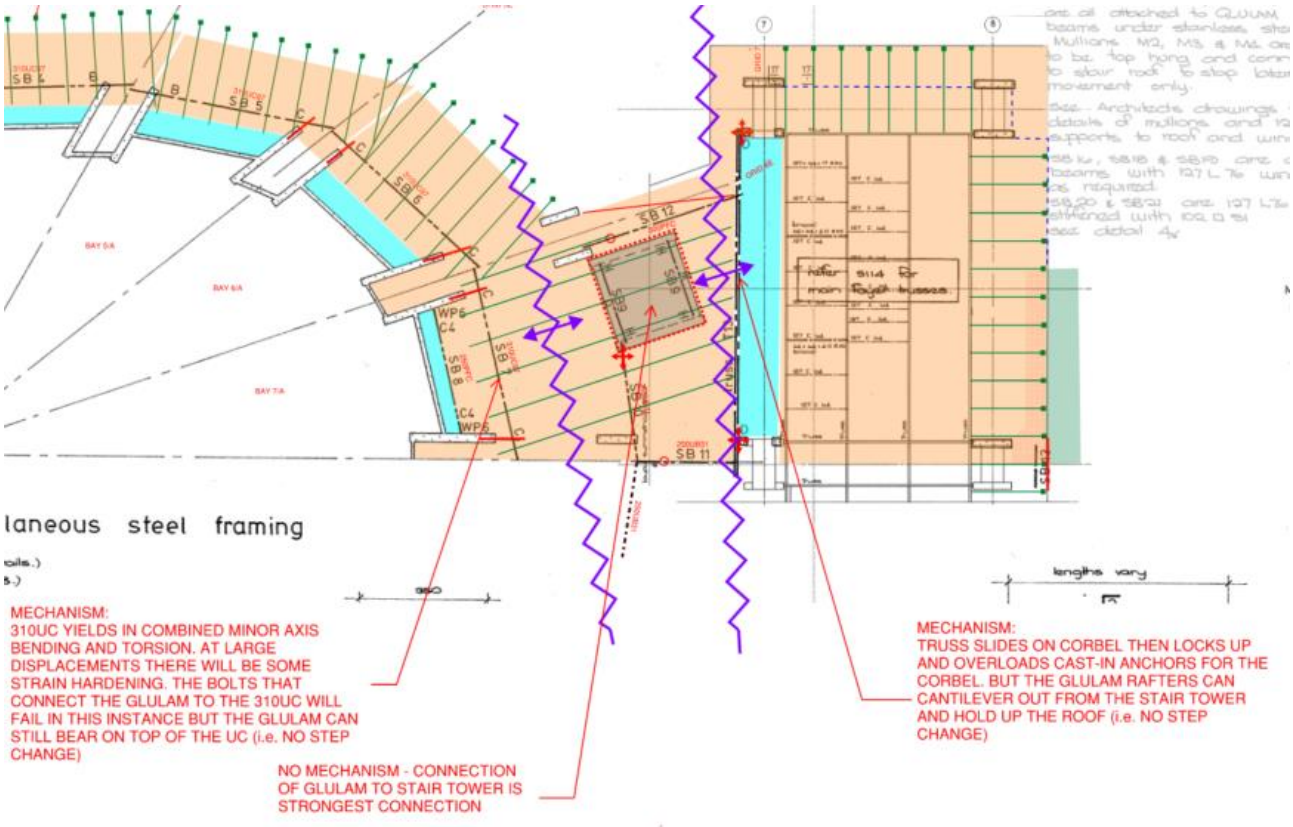

Figure G.3-7: Part plan of Lobby area roof showing mechanisms engaged for north-south differential displacement

The southern edge of the roof is supported by a truss on a sliding corbel at the Renouf Foyer's coupled column units. This corbel allows for 60 mm of sliding movement before 'jamming'. The truss has some ability to rotate and accommodate displacement beyond 60mm, however the weldplate that supports the sliding corbel has limited additional probable strength for increased loads. It is expected that the weldplate will withdraw from the coupled column units as displacements increase, removing the primary load path for the

truss. A redundant load path has been assessed and it has been determined that the glulam rafters can cantilever out from their supporting steel in the lobby area and hold up the dead weight of the truss. The loss of support to the truss is therefore not considered to govern the seismic score and the differential displacement can be accommodated by the roof between the Independent Staircase Structures and the Renouf Foyer, without leading to a significant life-safety hazard.

The northern edge of the roof will pull on the 310UC that spans between the precast concrete 'Terriers'. However, as displacement demands increase the 310UC will strain harden and this will increase demand on the bolted connection between the glulam rafters and the 310UC. The probable capacity of this bolted connection will be exceeded. However, as the glulam rafters are on top of the 310UC their gravity load can still be supported by the 310UC by bearing of the timber on the steel. The exceedance of the capacity of the connection is therefore not considered to govern the seismic score and the differential displacement can be accommodated by the roof between the Auditorium and the Independent Staircase Structures.

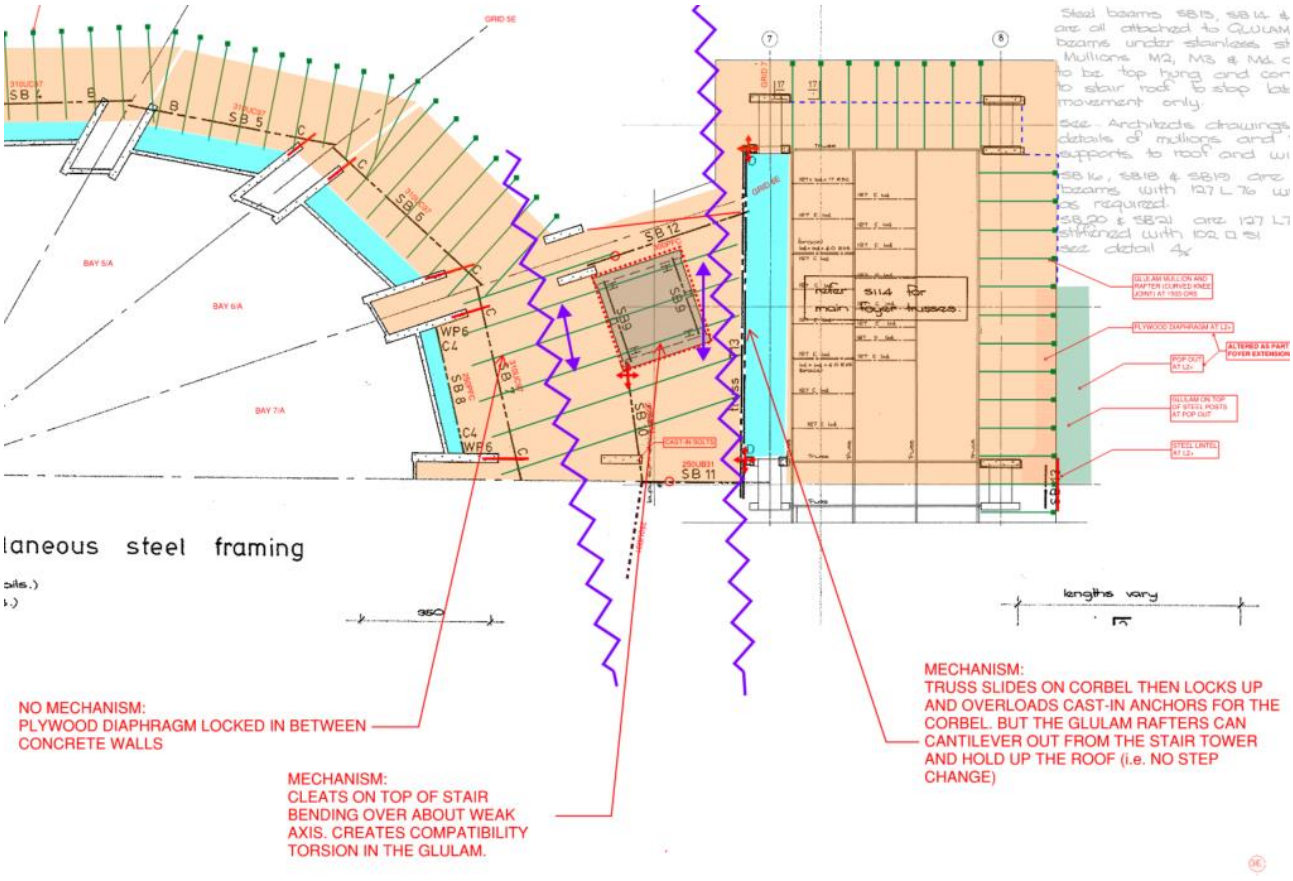

Figure G.3-8: Part plan of Lobby roof showing mechanisms engaged for East-West differential displacement.

For the east-west scenario the connection of the plywood diaphragm to the precast concrete 'Terrier' outriggers has been assessed to be the strongest connection and its probable strength exceeds the probable overstrength of the connections to the other structures.

The southern edge of the roof is supported by a truss on a sliding corbel. This corbel allows for 100mm of sliding movement before 'jamming'. The flexibility of the plywood sarking/diaphragm allows for an additional 30mm of relative movement between the Auditorium and the Renouf Foyer. Therefore, 130mm of relative movement can be accommodated before the corbel supporting the truss is subjected to additional

load. The weldplate that supports the sliding corbel has limited additional probable strength for increased loads. It is expected that the weldplate will withdraw from the precast concrete column as displacements increase beyond 130mm, removing the primary load path for the truss. However, a redundant load path has been assessed and it has been determined that the glulam rafters can cantilever out from their supporting steel in the lobby area and hold up the dead weight of the truss. The loss of support to the truss is therefore not considered to govern the seismic score and the differential displacement demand can be accommodated.

The central area of the roof in the lobby area is supported on top of the Independent Staircase Structures. The connection between the roof diaphragm/sarking and the roof of the Staircase Structures is through the glulam rafters and steel cleats. To transfer load, the steel cleats must act in bending about their minor axis. These cleats have only nominal capacity therefore they offer minimal restraint to the roof diaphragm. However, they do still restrain the bottom of the glulam rafter and since the top of the rafter displaces with the roof, this will twist the rafter (i.e. compatibility torsion).

It has been assessed that a differential displacement capacity of 240mm between the Independent Staircase Structures and the Auditorium can be accommodated before the probable torsional strength of the rafter is reached. Figure G.3-9 shows that at 55%NBS_{IL3} this displacement demand is exhausted.

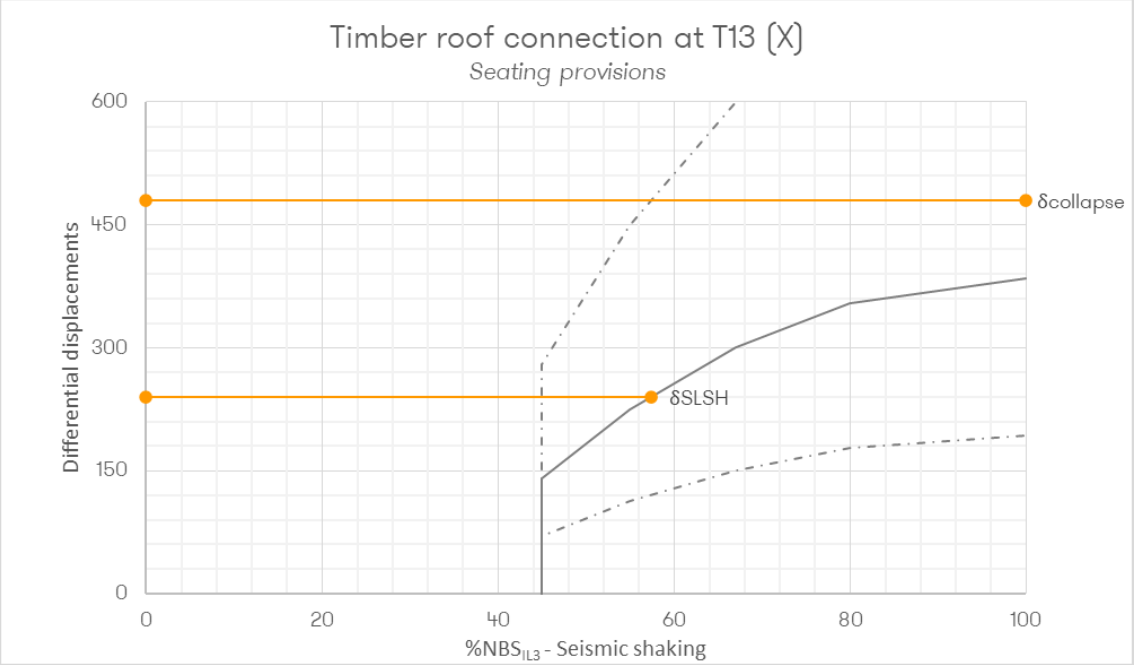

Figure G.3-9: Assessment of glulam roof in the lobby area under differential lateral spread in the east-west direction

Summary of seismic ratings

This section summarises all the potential capacities in the promenade roofs and façade system and determines the governing rating. It is found that the critical structural weakness is that in the in-plane (tangential) direction, the deflection demand on the glazing will exceed its probable displacement at **25%NBS$_{IL3}$**.

Table G.3-1: Seismic Rating of potential Structural Weaknesses in the Perimeter Roof and Facade.

Element	Capacity governed by:	%NBS$_{IL3}$
In-plane direction: Glazing	Deformation capacity of existing glazing suite	25
Out-of-plane direction: 310UC Beams	Combined flexural/torsional strength of 310UC	50
Out-of-plane direction: Type B Weld Plates	Yield capacity of steel flat cast into concrete wall	55
Differential lateral spread between Independent Staircase Structures and Auditorium	Exceedance of capacity in bolted connection and subsequent torsion in the glulam beams. *(noting connection exceeds capacity at 45%)*	55
In-plane direction: Plywood Sarking	Shear strength of nails	55-60
In-plane direction: Glulam rafters	Torsional strength of glulam	70
Out-of-plane direction: Concentric Bolted Connection from Glulam to 310UC	Bolt strength in glulam rafter	>100
Out-of-plane direction: Type A Weld Plates	Concrete breakout strength	>100
Out-of-plane direction: Type C Weld Plates	Pull out (development) of steel round bar cast into concrete wall	>100
Differential lateral spread between Independent Staircase Structures and Renouf Foyer	Roof truss loses support, but alternative load path demonstrated *(noting truss loses support at 45%)*	>100
Out-of-plane direction: Plywood sarking	Strength of nails is exceeded in a localised area, adjacent to the concrete shear walls *(Damage occurs at 20-25% to allow stronger elements to 'engage' and resist loads)*	N/A

G.4 CONCRETE MASONRY BLOCK WALLS

Several reinforced concrete block walls are located throughout the building. Most are concentrated on Level 1, but some reach up to Level 5. A more detailed description of the types of block walls found in the Auditorium is given in Section 5.1.1.5.

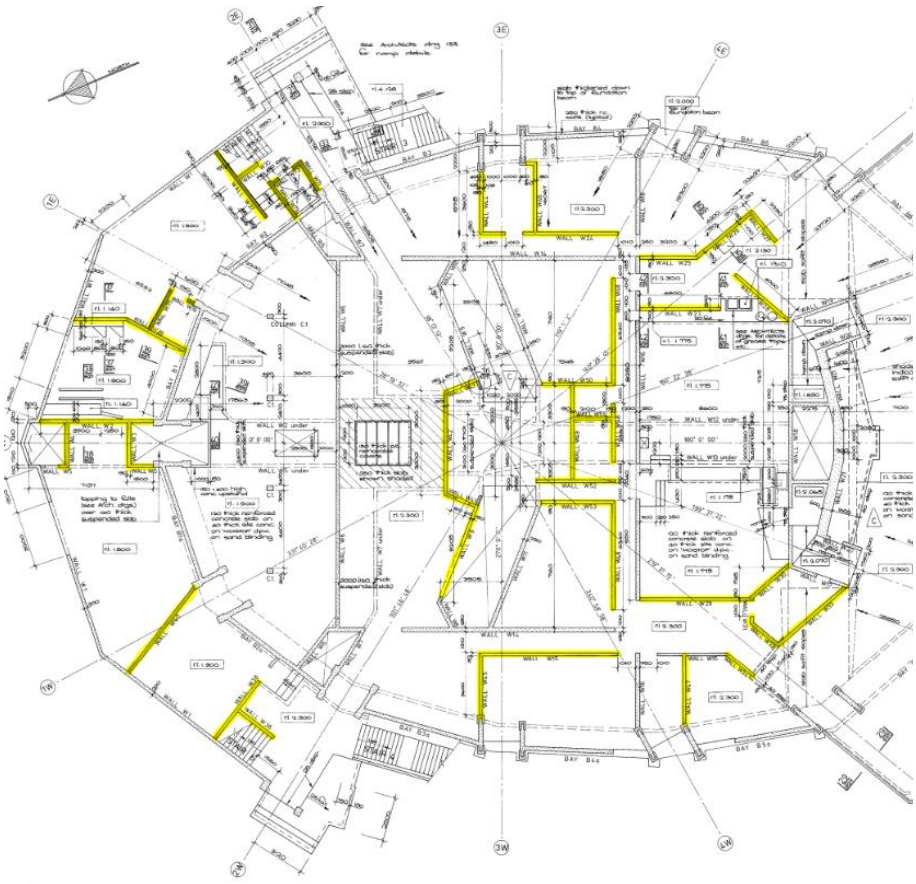

Figure G.4-1: Snapshot taken from drawing S3C of the 1980's Holmes, Wood, Poole & Johnstone drawing set - Level 1 Plan showing the block walls highlighted in yellow.

All of the block walls use 200-series blocks that are fully filled with grout and are reinforced with Ø12 bars of the lower reinforcing grade spaced at 600mm centres in both directions. The bars are consistently hooked into the upper and lower diaphragms connecting to each wall (except around the front of the stage). Because the reinforcing is located at the centroidal axis of the walls, the out-of-plane flexural capacity is relatively small. It has been assumed that the block walls are intended only as part of the gravity load system and do not contribute significantly to the primary lateral load resisting system.

Deformation-based assessment

A deformation-based assessment has been completed for the walls for in-plane loading. Using the provisions in the Yellow C5 section of the Engineering Assessment Guidelines, a drift capacity of 1% is given for the block walls because the axial load ratio on the walls is relatively small (less than 2%). The maximum inter-storey drift demand from the Auditorium between Level 1 and Level 2 is less than 1%. Therefore, the block walls are expected to perform at 100%NBS_{IL3} for their in-plane drift compatibility.

Force-based assessment

A force-based assessment has been completed for the walls for out-of-plane loading. Using the provisions in the Chapter C5 Engineering Assessment Guidelines, the yield and plastic drift capacities for the walls were assessed, giving an expected ductility ratio of >3. Using the provisions from the 'Parts and Components' from Chapter 8 of NZS1170.5:2004, the face loading of the wall was determined and compared to the wall flexural and shear capacity. Based on this, the critical block walls are those at Level 4. These are expected to perform at **50%NBS$_{IL3}$** for out-of-plane loading. P-Δ effects were also considered but are unlikely to be an issue as the stability factor is less than 0.005. NB that several block walls on Level 1-4 reach 100%NBS$_{IL3}$.

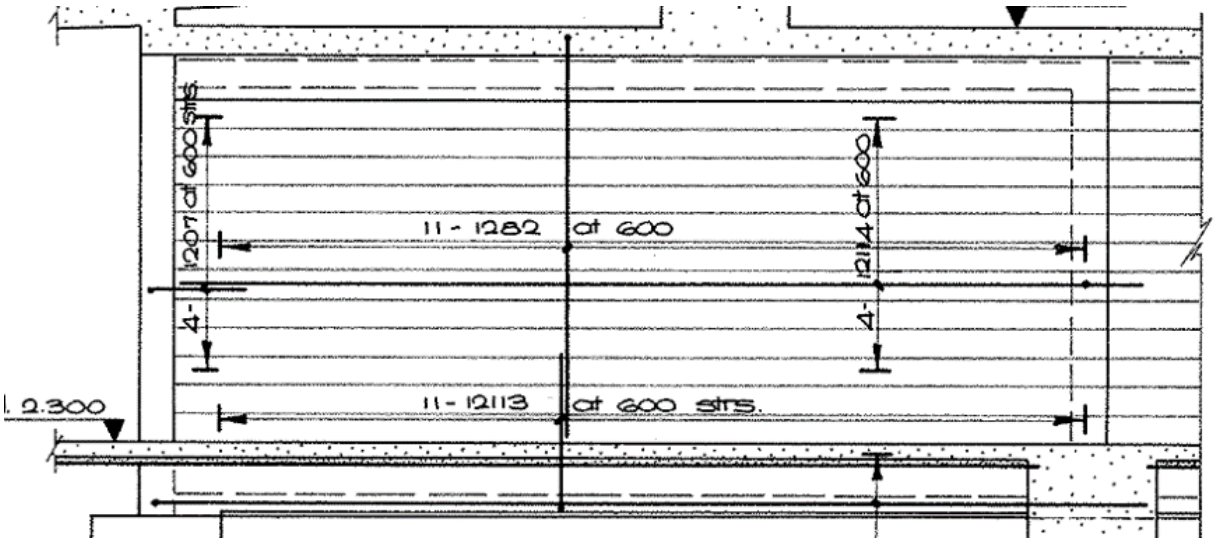

Figure G.4-2: Snapshot taken from drawing S48B of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing the typical block wall layout with reinforcing

G.5 CATWALK STRUCTURE, REFLECTORS AND DOME

G.5.1 Catwalk Structure

At Level 5 of the Auditorium there are acoustic reflectors hanging from the precast buttresses and precast wall panels of the Bay Walls, which have a walkway system running behind them. This system of acoustic reflectors and walkways are referred to as the 'Catwalk structure'. It is a considerable heavy (>>25kg) secondary structure, that could pose a life safety hazard when it loses integrity.

The catwalk structure comprises 9 cantilever frames that connect to the Bay Walls as shown in Figure G.5-1. They support the walkway system through a series of Ø20mm threaded rods and a framework of timber acoustic panels on a diagonal strut. Vertically and in plan, the catwalk structure is braced by a series of diagonal bracing elements in the bays highlighted in blue on Figure G.5-1.

Various changes have been made to the catwalk structure; however, the general structural arrangement still matches the original 1980's structural drawings. Specific changes are covered later in this section.

Due to limited accessibility of some of the elements, no measurements have been taken of the altered elements. They have only been visually inspected and scaled relative to the other members present in the catwalk structures. Also, no testing has been undertaken.

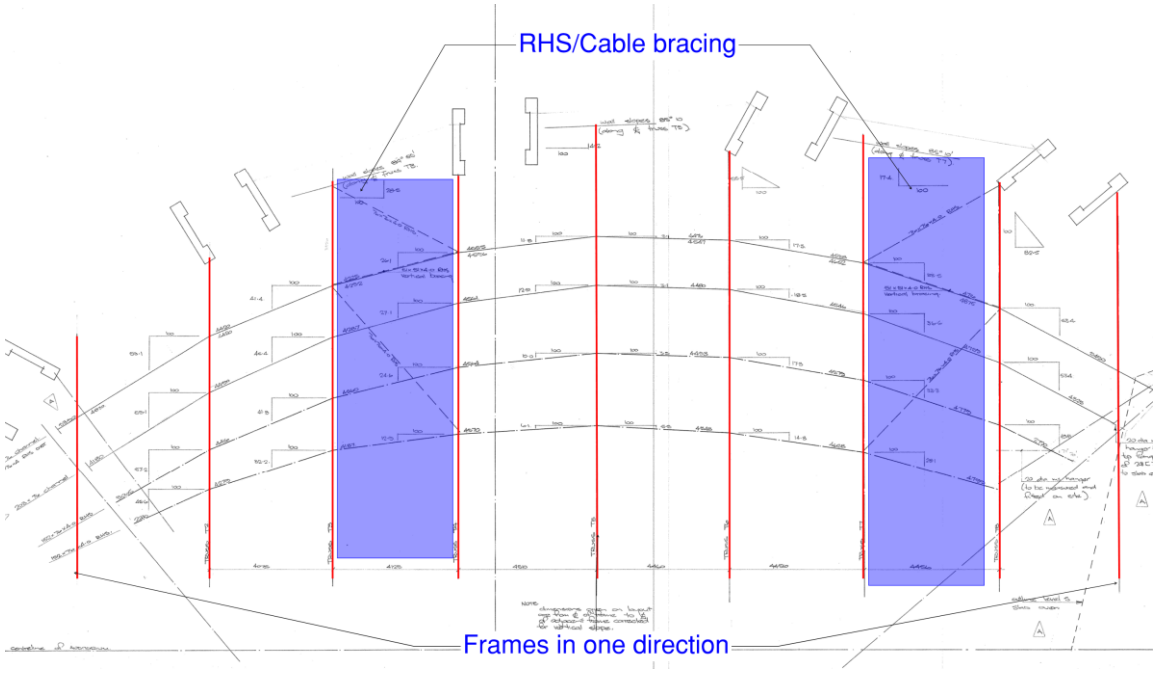

Figure G.5-1: Snapshot from drawing S97/1A of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing a typical plan of the Catwalk structure.

Each cantilever frame of the catwalk structure generally comprises a 250UB31 which is connected to the Bay Walls at one end and is tied back by tie rods at the other end. The tie rods either tie back to the Bay Walls directly or connect to the bottom chord of the roof truss, they are varying of two Ø16mm or two Ø20mm in size, depending on the location. A diagonal 76x4.0mm SHS is attached to the UB to support the timber framework and lighting fixtures. At the bottom end, this diagonal connects to a horizontal 76x4.0 SHS which provides the support for the walkways. Two vertical Ø16mm threaded rods then tie the walkway back to the UB.

In accordance with original 1980's drawings, there are a series of 203x76 channels and 152x76x4.0 RHS' that run in plan (north-south) between the different cantilevering frames. These connect all the bays to make it one system. The catwalk structures are braced in plan by two bays of 76x4.0 SHS members that run diagonally across the UB's. Vertically there are two bracing bays which brace the walkway with the UB. These comprise a diagonal 51x4.0 SHS which runs in the same plane as the outer vertical Ø16mm threaded rod.

All elements of the catwalk structure are typically connected through the threaded rods, Ø20mm bolted connections, fillet welds or Ø20mm 'Dynabolts' into the concrete.

The following pages show some of the key details and alterations that have been made to the catwalk structure.

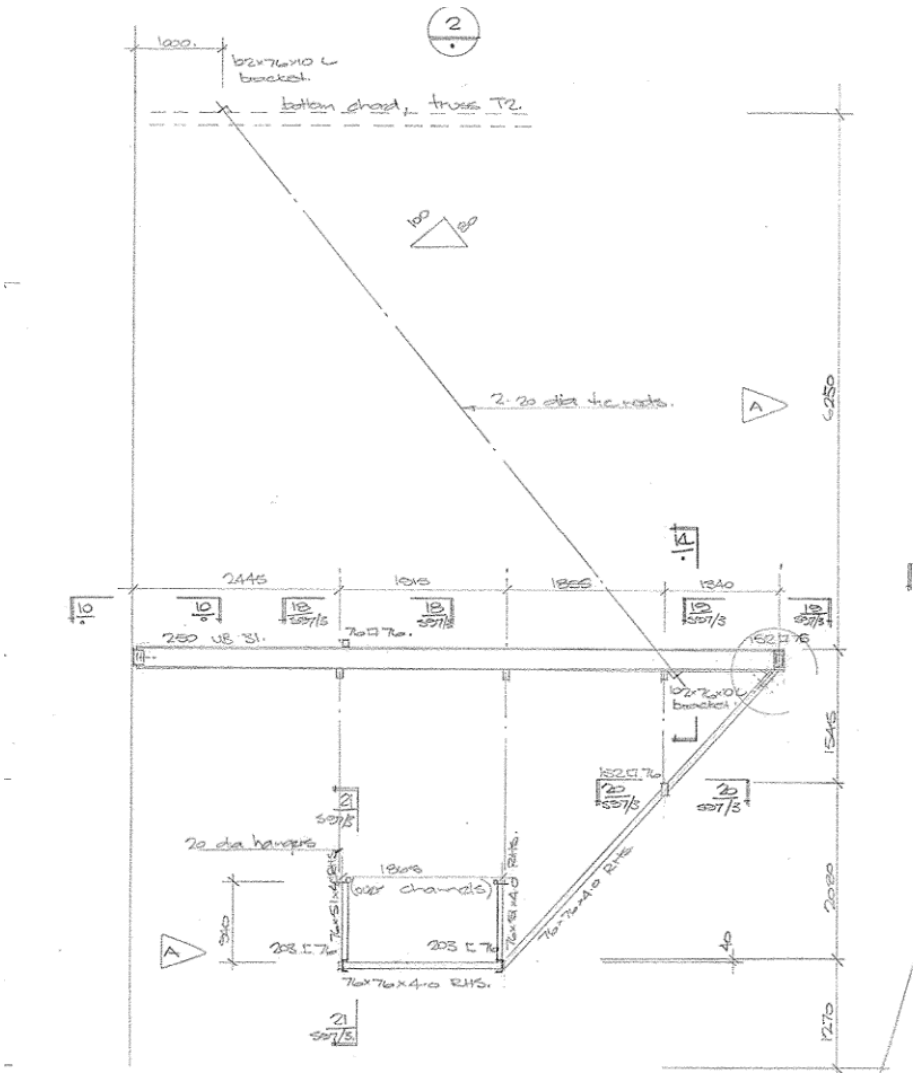

Figure G.5-2: Snapshot from drawing S97/2A of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing a typical elevation of the catwalk structure.

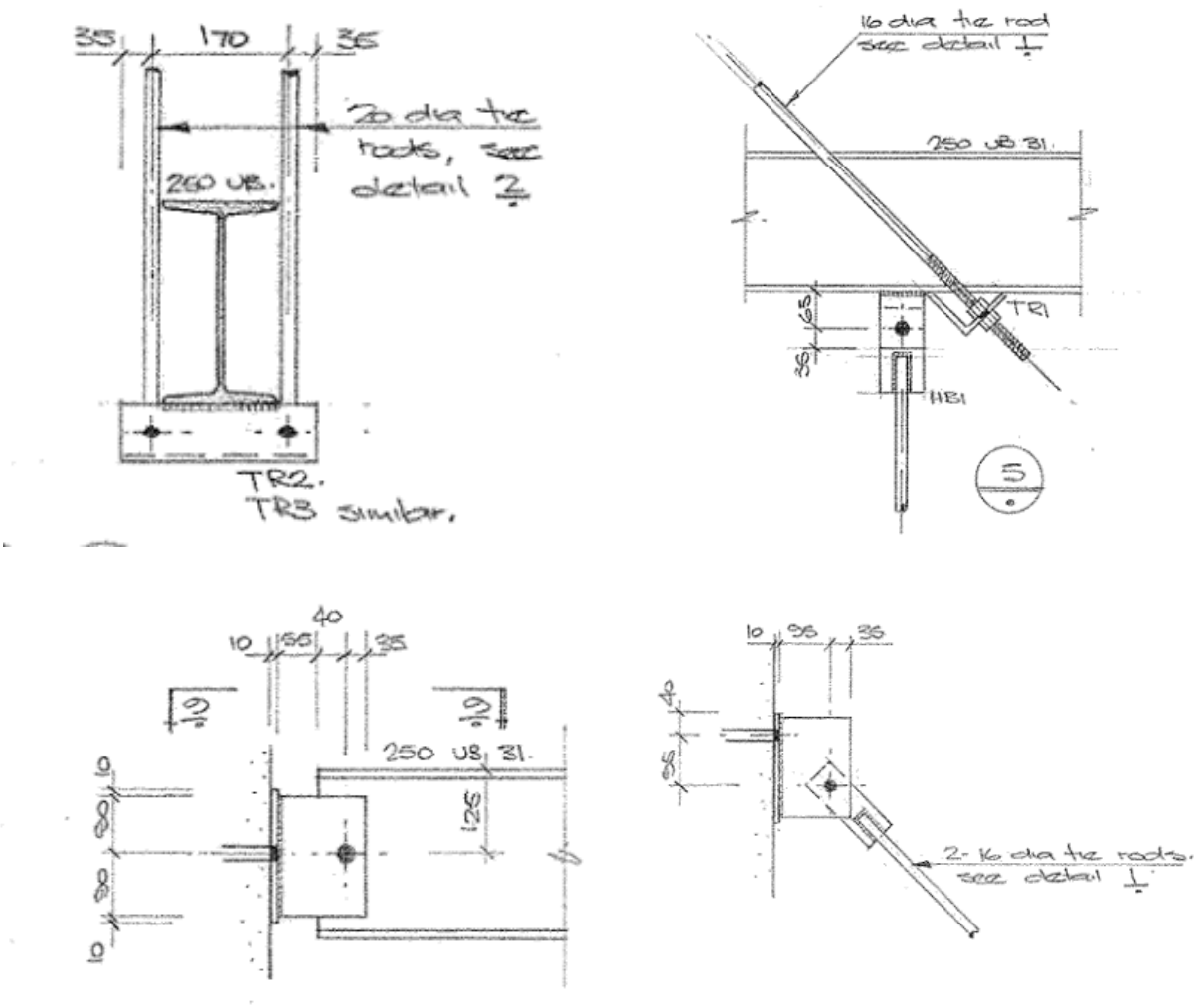

Figure G.5-3: Snapshot from drawing S97/2A of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing typical details of the catwalk structure.

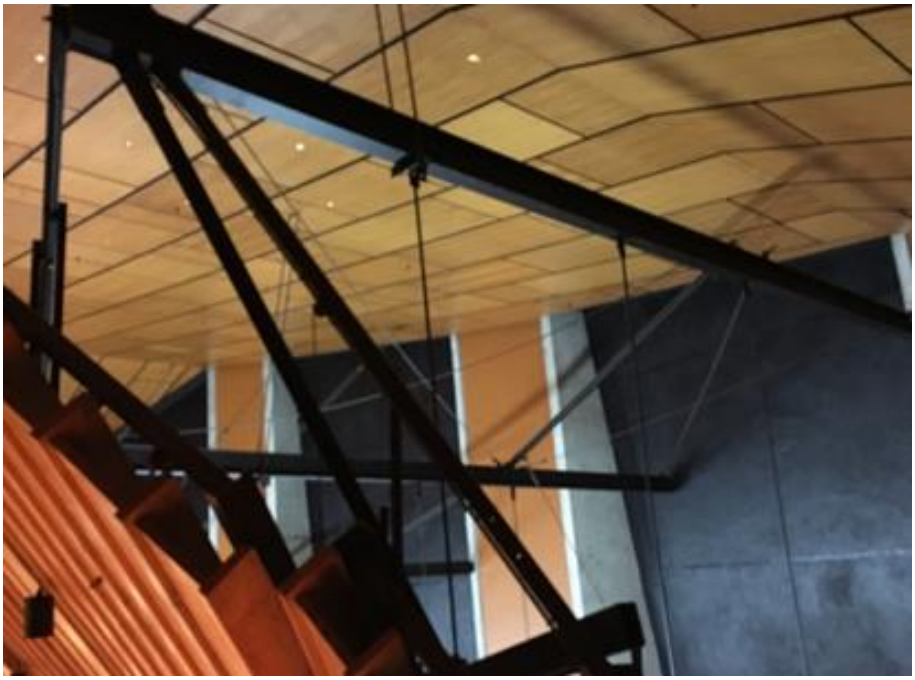

Figure G.5-4: On the eastern side of the Auditorium, the in-plan bracing has been changed to cables in an X-shape configuration. The cables appear to have some slack.

Figure G.5-5: At the eastern side of the Auditorium, the vertical bracing has been altered to a cable (shown in red). Approximately Ø12mm in size.

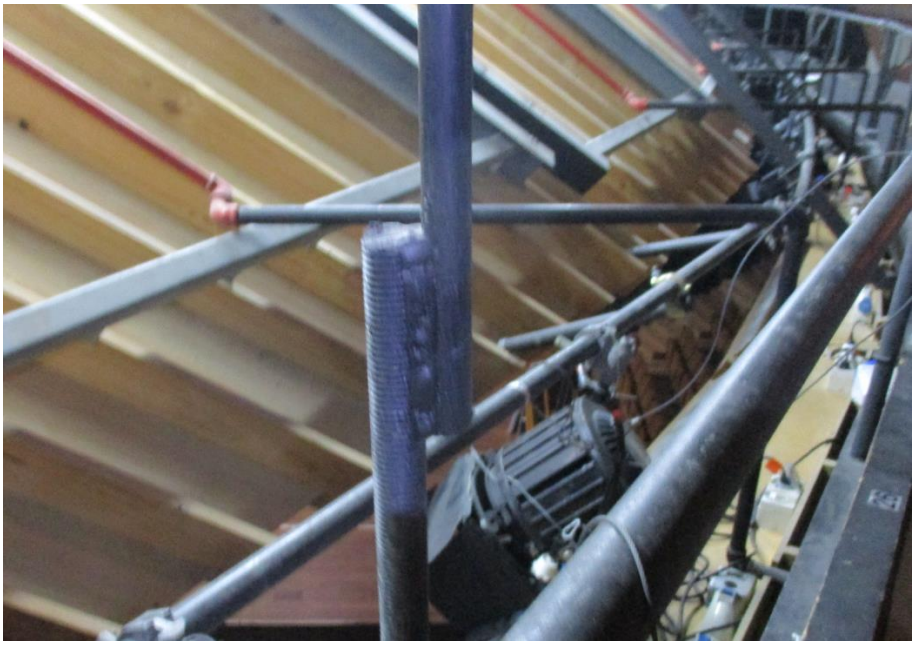

Figure G.5-6: Some (at least 3x observed) of the vertical Ø16mm threaded rods (gravity hangers) have been reinstated with a two-sided fillet weld

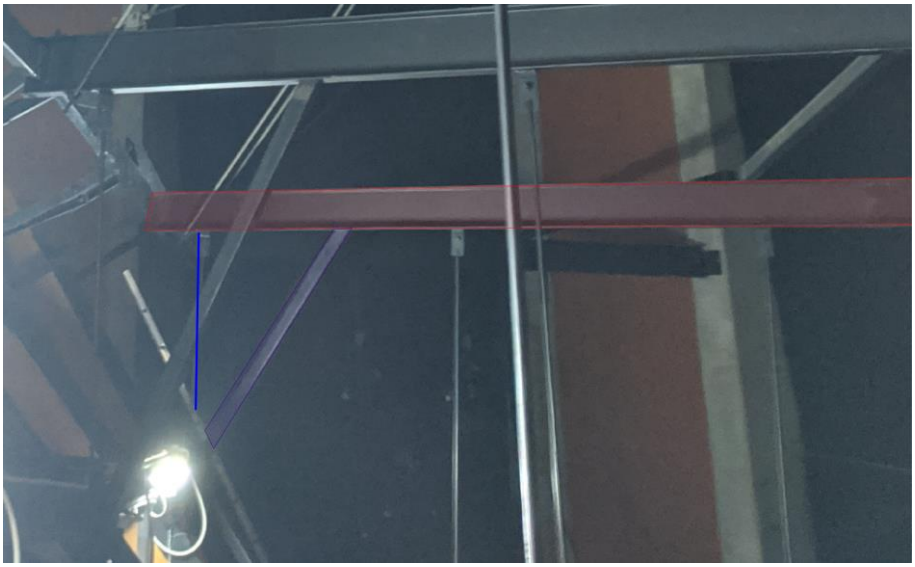

Figure G.5-7: The cantilever frames underneath roof trusses T2 and T3 have been configured slightly different at their ends. The 76x4.0mm SHS does not continue to the end of the UB, however instead it bends halfway and returns to the UB at a location closer to the vertical threaded rod of the walkway

Figure G.5-8: Modification of the 250UB connecting into the PBW system. Showing also a more detailed view of the altered cross bracing in plan.

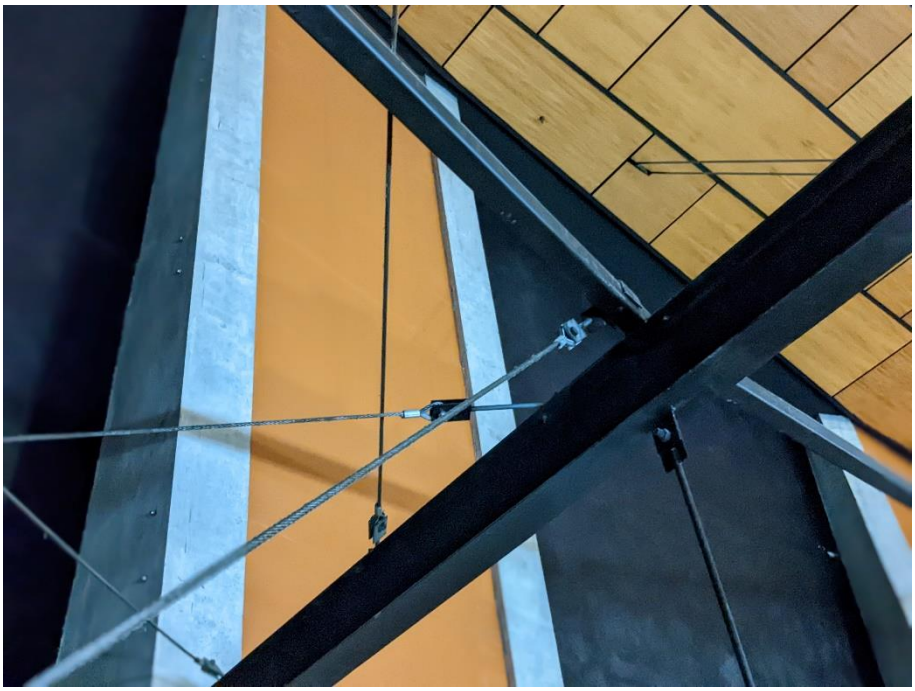

Figure G.5-9: Zoomed view of the altered cross bracing in plan and vertically. Also, noting the eccentric nature of these members

Assessment of catwalk structure

Figure G.5-10 shows the original timber framework intended to go around the catwalk structure. This is assumed to be relatively consistent with the current configuration. Timber density is taken as 4.5kN/m^3 as per table ZZ2.1 of NZS1720.1:2022, which has allowed for a distributed load of the timber built-up to be calculated.

Even though the catwalk structure has a walkway they are assumed to be accessed for maintenance. Hence, $\psi_E = 0$ for the live load (Q) in the seismic case as per NZS1170.0 for roof live loading.

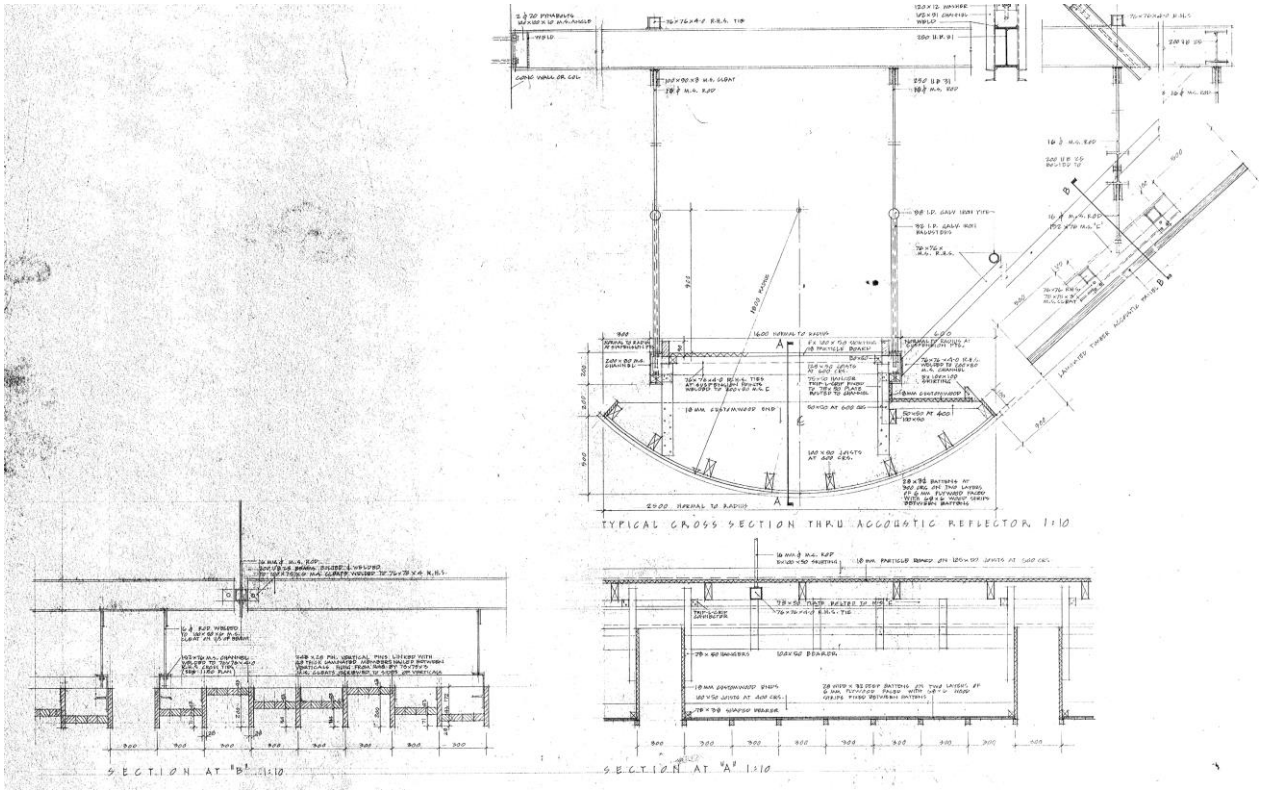

Figure G.5-10: Snapshot from drawing 89 of the 1980's Warren & Mahoney drawing set – showing architectural built-up of the acoustic reflectors in the catwalk structure

The potential life safety risk associated with the induced lateral movement and evaluated in the seismic score is, firstly, that the tie rods may see an exceedance of its capacity in tension. Either along the members themselves or at their bolted, welded or anchored connections. Exceedance of the capacity of these elements could see the catwalk structure show excessive displacements or result in loss of gravity support.

Secondly, there could be flexural demands or tension demands that would exceed the capacity of the UB or its connections. This element is critical in maintaining the gravity support of the catwalk structure.

In the assessment of the catwalk structure the flexural and tension capacity of members is considered to have some level of ductility ($\mu \geq 2.0$). Steel connections that behave more brittle, such as bolts or welds in shear have been assessed with nominal ductility ($\mu = 1.25$). Shallow embedded anchors in the concrete have been assessed with no ductility ($\mu = 1.0$).

The catwalk structure is assessed using a force-based assessment with vertical and horizontal design actions determined in accordance with the provisions for 'Parts and Components' from Chapter 8 of NZS1170.5:2004. A Microstran model has been generated to view the internal forces.

The weight of the timber frame is applied to the 76x4SHS member as shown in Figure G.5-11.

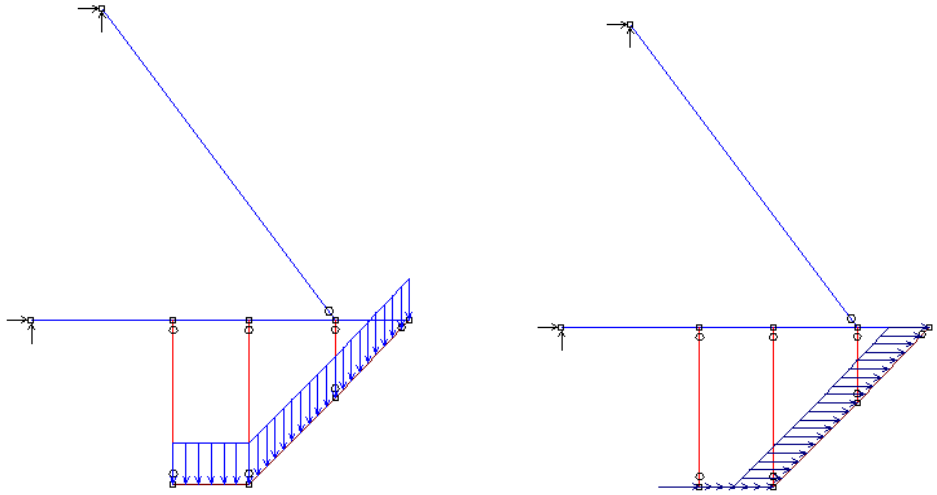

Figure G.5-11: Snapshot from the Microstran model showing the load application in gravity (left) and lateral from seismic shaking (right).

Moving away from the Bay Walls

When the accelerations drive the inertia of the screens away from the bay walls, the primary structural members will initially all act in tension. At some point, the lateral loads from seismic shaking will overcome the gravity demands on the sections. This forces the upper diagonal tie rods and the vertical threaded rods of the walkway to go into compression and to buckle.

The assessment in this direction has considered a fundamental period of the structure of T ≤ 0.75s.

The weakest link in this scenario is the 250UB which flexes relatively early but can deform inelastically at a ductility of μ = 2.0 to achieve a seismic score of 40%NBS_{IL3}.

Within the bounds of the flexural overstrength of the 250UB, its connections to the bay walls are also exceeded at a similar level of seismic shaking (40%NBS_{IL3}).

The subsequent buckling of the diagonal 76x4.0 SHS may also lead to substantial damage but is not considered a significant life safety hazard.

Moving towards the Bay Walls

For the scenario where the accelerations drive the inertia of the catwalk structure towards the Bay Walls, the vertical Ø16mm threaded hanger rods will likely see early buckling. This scenario will likely result in dynamic behaviour which may result in a lot of damping of the actions. Neither the Engineering Assessment Guidelines nor NZS1170.5:2004 provide a lot of guidance on how to deal with this scenario. In our opinion the additional flexibility and damping is best captured by an equivalent structural period of T≥1.5s to reduce the seismic loading.

The critical elements are the bolted connections and the vertical threaded rods. These have been assessed for those dynamic loads, as they 'slam' back to their initial state from the gravity demands.

However, this scenario was not found to be critical and all elements were found to perform at >100%NBS_{IL3}.

Moving in plan

For the scenario where the accelerations drive the inertia of the catwalk structure to move in plan, the bracing in plan is required to transfer the load horizontally to the PBW system.

Reiterating the alterations in the catwalk structures that were observed. Where the east side of the catwalk structure has two sets of cross braces with Ø12mm cables which transfers the load horizontally to a single diagonal brace of Ø12mm cables spanning vertically parallel to the threaded rods of the walkways. While the west side has (as drawn) 76x4.0 SHS spanning horizontally to transfer the earthquake load to the 51x4.0 SHS vertical member.

For the assessment, it is assumed that both systems have a higher system ductility of μ = 2 and will primarily perform in tension. Also, the observations on site revealed a lot of slackness in the bracing members and the connections. Therefore, a longer period of T > 1.5s has been considered as the system is expected to be relatively flexible and 'out-of-sync' with the other members across the plan.

It was found that within these assumptions, that both the vertical bracing along the threaded rods of the walkways and the in-plan bracing are rating at **20-30%NBS_{IL3}**. However, the seismic score is likely on the lower end, as connections are likely going to be critical. As the catwalk system already has a seismic score lower than 33%NBS_{IL3} with a ductility of μ = 2.0, they are not further analysed. Similarly, the concurrency of vertical accelerations will likely result in a lower seismic score, which is not further analysed.

In conjunction with the observations on site, this generally appears to be an accurate representation and fair reflection of the seismic risk. There is no resilient load path, and the subsequent excessive displacements may cause parts of the timber framework to fall or have the catwalk structure lose its structural integrity more globally.

However, it is noted that there are still many unknowns that may affect the seismic score upwards and downwards.

G.5.2 Wall reflectors

Above the Level 4 stepped seating there are also a series of acoustic reflectors that are located just below the catwalk structure. They are individual elements that are independent from the catwalk structure.

The structural configuration comprises a steel frame comprising 75x50x4.0 RHS', which are hung from the bay walls by with Ø16mm threaded rods (as shown in Figure G.5-13). The timber framework screens are attached to RHS frame with regular spaced M10 bolts.

The wall reflectors are connected to the PBW system by post installed anchors. They are connected by two locations for Ø12mm 'Dynabolts' at the lower end and with a single Ø12mm Dynabolt to connect the Ø16mm threaded rods. Refer Figure G.5-14 for the connection details.

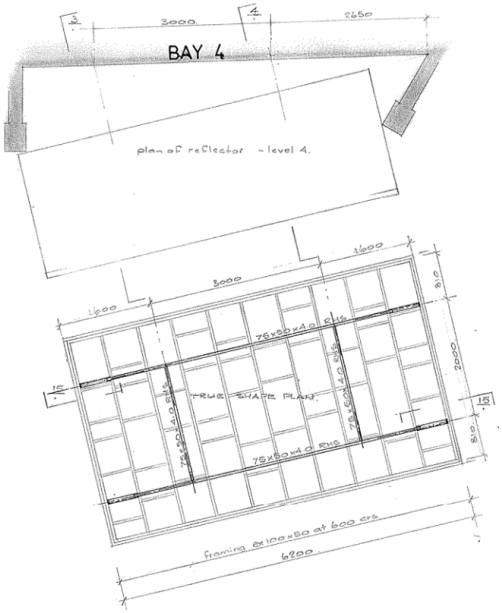

Figure G.5-12: Timber framework of the wall reflectors

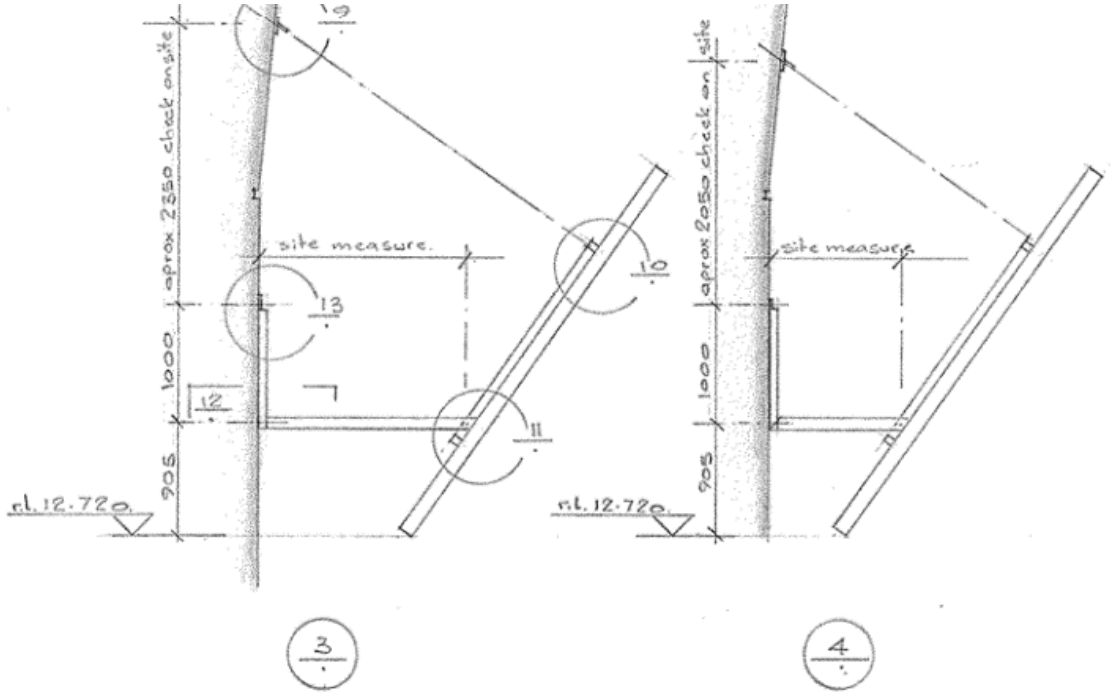

Figure G.5-13: Snapshot from drawing S99 of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing a typical plan and elevation of the wall reflectors

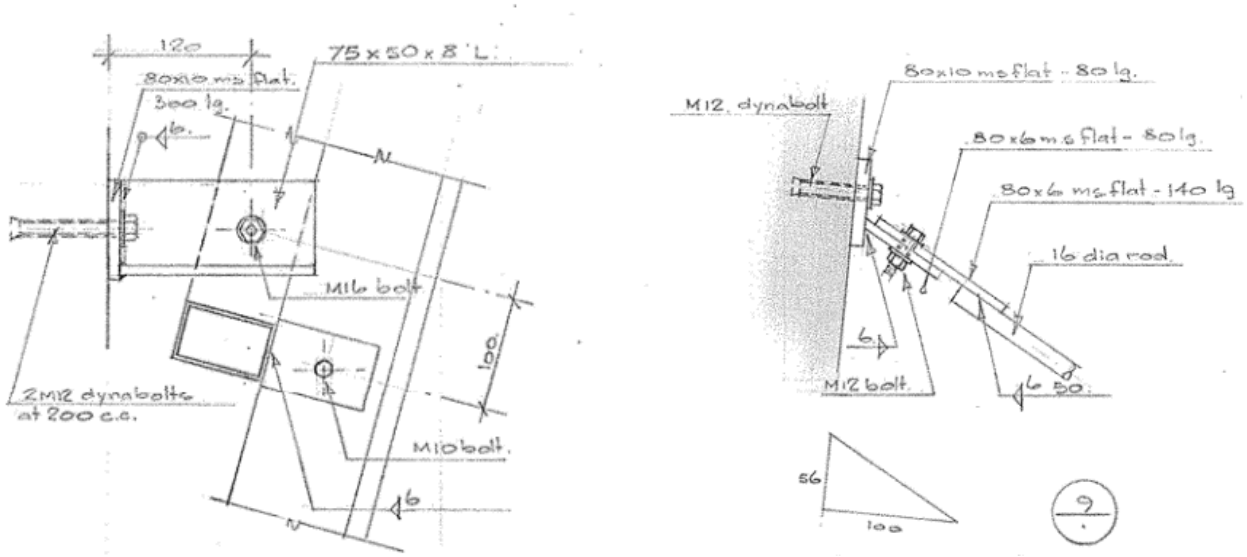

Figure G.5-14: Snapshot from drawing S99 of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing a typical connection from the reflector to the PBW's.

Assessment of wall reflectors

The timber framework has considered timber density taken as 4.5kN/m^3 as per table ZZ2.1 of NZS1720.1:2022. A distributed load of the of the timber built-up is calculated from the built-up as shown in Figure G.5-12. The weight of the timber frame is applied to the 75x50x4RHS members as shown in Figure G.5-15.

The potential life safety risk associated with the induced lateral movement and evaluated in the seismic rating is, firstly, that the threaded rod can lose its connection to the PBW system. This will cause the structure to lose vertical support and swing (or fall) down causing a life safety hazard.

Secondly, though potentially less critical is the loss of the connection of the 75x50x6.0 RHS to the Bay Walls. If the structure loses this connection the whole reflector structure will be supported by the Ø16 tie rods and its connection. The dynamic amplification of this nonlinear behaviour can load the tie rods excessively and can cause the whole structure to fall off when its capacity is exceeded.

In the assessment of the wall reflectors, the 75x50x4.0 RHS and threaded rod members are considered to have a high ductility (they are assessed for a ductility of μ = 3) as they can deform and dissipate energy during a seismic shaking. Steel connections that behave more brittle, such as bolts or welds in shear have been assessed with nominal ductility (μ = 1.25). Shallow embedded anchors in the concrete have been assessed with no ductility (μ = 1.0)..

The wall reflectors are assessed using a force-based assessment with vertical and horizontal design actions determined in accordance with the provisions for 'Parts and Components' from Chapter 8 of NZS1170.5:2004.

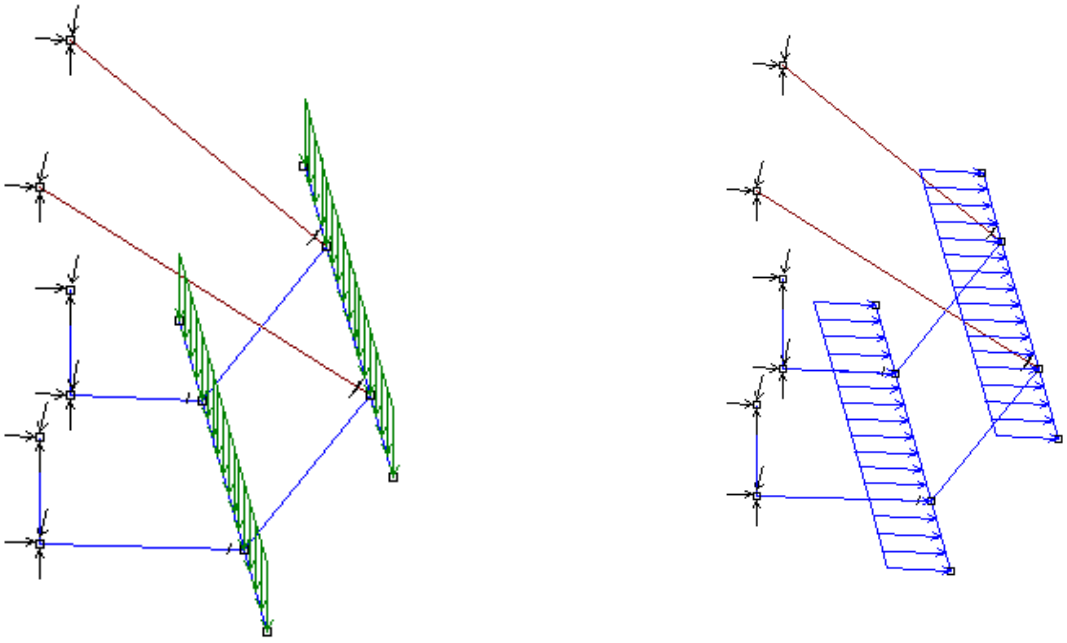

Figure G.5-15: Snapshot from the Microstran model showing the load application in gravity (left) and lateral from seismic shaking (right)

The assessment focusses on the movement of the wall reflectors towards or away of the Bay Walls; however, these load cases are combined with vertical accelerations and movements in plan where

applicable. Concurrency is considered at 100% in the direction under consideration and 30% in all the secondary directions.

Moving towards the Bay Walls

For the scenario where the accelerations drive the inertia of the wall reflectors towards the bay walls, the vertical Ø16mm threaded rods will likely see early buckling. This scenario will likely result in dynamic behaviour which may result in a lot of damping of the actions. Neither the Engineering Assessment Guidelines nor NZS1170.5:2004 provide a lot of guidance on how to deal with this scenario. In our opinion the additional flexibility and damping is best captured by an equivalent structural period of T≥1.5s to reduce the seismic loading.

However, this scenario was not found to be critical, and all elements were found to perform at >100%NBS_{IL3}.

Moving away from the Bay Walls

When the accelerations drive the inertia of the wall reflectors away from the bay walls, vertically or in the plan, the primary structural members will all act in tension.

The weakest link was identified as the horizontal 75x50x6.0 RHS spanning back to the bay walls. They exceed their capacity under combined tension and flexure. When they yield however, a fuse is introduced in the analysis model as the yielding does not cause a life-safety hazard in itself.

The next element in strength hierarchy is the connection of the diagonal threaded rod to the bay walls which show a concrete breakout failure in the Ø12mm Dynabolts at **20%NBS_{IL3}**.

G.5.3 Stage reflector ('Donut')

As explained in the Section 5.1.1.9, the stage reflector supporting structure was altered in 2020. Since the alteration occurred post 2018, they were retrofitted to modern standards. It was also confirmed that a factor of safety (>2) has been applied on the supporting cables and their connection at the roof level.

The cables are now hung (vertically) perpendicular to the 'donut'. Hence, it is considered to act as a pendulum during a seismic shaking. Considering the potential spectral displacements that the 'donut' may be subject to, it is unlikely to hit any other structural element.

The 'donut' is now supported on proprietary F44P trusses instead of directly loading the bottom chord of the truss. The F44P trusses span between the Auditorium roof trusses and thus provides a more uniform distribution of load than a point load on the main trusses.

The F44P trusses are a proprietary product, and their connections to the roof trusses are configured by fabric straps, as designed by the rigging engineer. It is intended to be a temporary solution, as the rigging configuration may change between events. The 2018

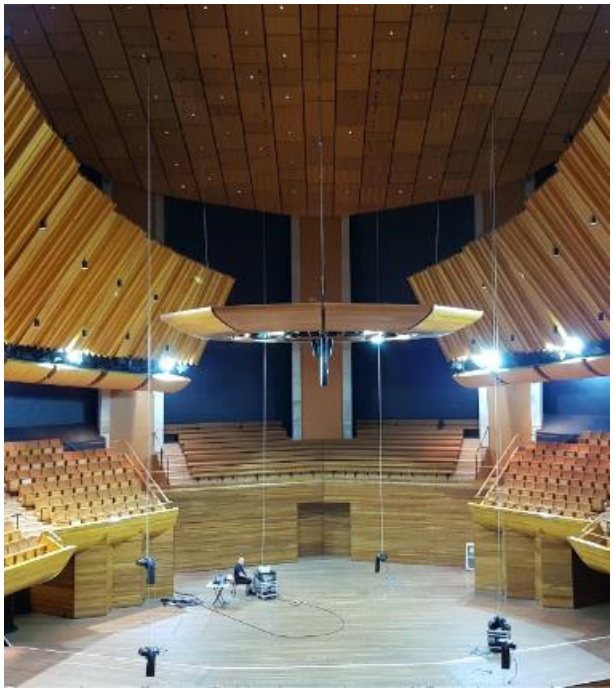

Figure G.5-16: Photograph of the inside of the auditorium showing the 'donut' and its vertical support cables

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Wellington 6140
holmesanz.com

'Service Loading Report' states that the "F44P truss to support beam connection is designed by the rigging engineer to prevent movement of F44P trusses". How this is exactly detailed with the straps, is not confirmed, but the assessment assumes there is a level of tension in the straps.

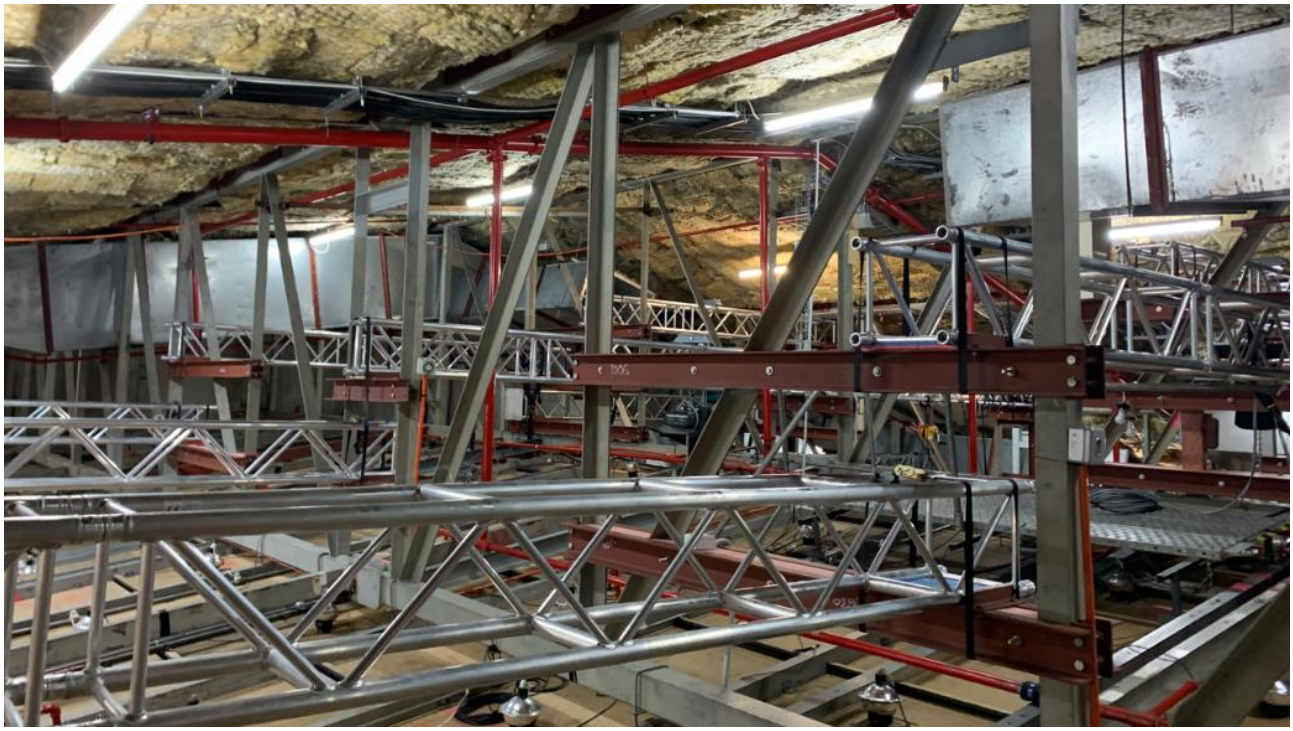

Figure G.5-17: Photograph of the inside of the ceiling space showing the F44P trusses between the Auditorium roof trusses

As shown on Figure G.5-17, there are brackets in place which prevent movement of the F44P trusses along their supporting structure. In the direction perpendicular to the supporting structure, there is also minimal movement expected. Purely from a geometric point of view, as the 'Donut' swings as a pendulum, the expected maximum relative displacement that the 'Donut' will swing is assumed equal to the spectral displacements (S_d). For the Michael Fowler Centre, the (undamped) spectral displacements at 100%NBS_{IL3} are 1.4m. From our information, the 'Donut' hangs at least 4m below the support structure but can be lowered as required. Therefore, the biggest aspect ratio of any vertical to horizontal load is ~3:1. If that is compared against the friction coefficient of aluminium (F44P trusses) on steel, then this is typically at least a factor of μ=0.50. The aluminium F44P trusses are relatively lightweight. On this basis, it is not expected that the F44P trusses move from the inertia of the 'Donut'.

G.5.4 Other rigging points in the building

Within the Auditorium and the Renouf Foyer there are other rigging points which support lighting equipment, camera's, microphones, speakers, and other features. These are considered to have a seismic weight of >25kg and therefore may be considered a life safety hazard.

The equipment in the Auditorium, are typically hung from vertical cables, similar to the 'donut' (see Figure G.5-16). These are expected to perform in a similar manner to the 'donut', by swinging around. They are not expected to impose notable lateral load on their supporting structural systems, and they are also not expected to critically collide with other elements.

In the Renouf Foyer the rigging elements are supported by Ø10mm threaded rods, which are assumed to be of Grade 4.6 quality, these are shown on Figure G.5-18.

The lateral forces in the structural system have been determined in accordance with the requirements on 'Parts and Components' requirements, as specified by Chapter 8 of NZS1170.5:2004. For this assessment, it is assumed that the threaded rods respond in a ductile manner ($\mu_p \geq 3$), and that the part have a period of $T_p \geq 1.5$s.

The 'parts' (speakers, lighting, etc.) are estimated to have a weight of 25kg for each of the Ø10mm threaded rods. The equipment is considered to hang ~0.5m below the ceiling of the Renouf Foyer. The resulting bending moment and shear demands in the threaded rods result in a seismic score of **70%NBS$_{IL3}$**, governed by the flexural capacity.

However, a better seismic performance may be expected, as the threaded rods may sustain larger inelastic displacements and start to act as a pendulum system.

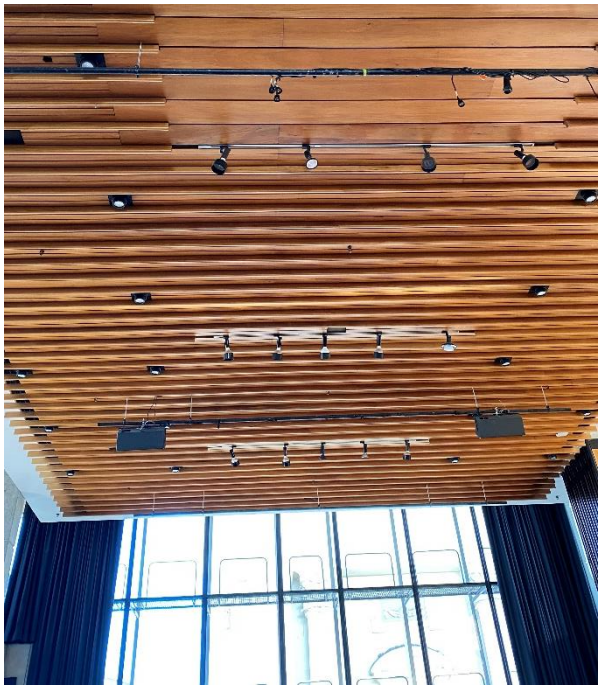

Figure G.5-18: Photograph of ceiling of the Renouf Foyer, showing the rigging points

The connections of the threaded rods were assessed against elastic loading (μ_p=1.0) and were not found to be critical.

Table G.5-1: Parameters used in determining the horizontal 'Parts and Components' acceleration on the rigging points

Parameter	Notation	Value	Commentary
Site hazard coefficient for T=0s	C(0)	1.12	Subsoil class D
Floor height coefficient	C_{Hi}	3.0	$h_i \geq 0.2h_n$
Part spectral shape coefficient	$C_i(T_p)$	0.5	$T_p \geq 1.5$s
Part risk factor	R_p	1.0	Category P.2 and P.3
Part response factor	C_{ph}	0.45	$\mu \geq 3.0$

G.6 PRECAST INFILL CLADDING PANELS

Around the upper precast sections of the Auditorium, several large precast cladding panels are located. They fill the spaces between the bay walls.

The panels are each 250mm thick, approximately 2m wide, and up to 8.3m tall. They are attached to the piers at the ends of the reinforced concrete shear walls using connections with M20 bolts and slotted such that they can move +/- 10mm horizontally and +/- 25mm vertically. The connections are considered to be positioned centrally for the purpose of this assessment.

The precast panels themselves are reinforced with 16mm bars at 300mm spacing in both directions. They are considered to be rigid elements with only nominal deformation capacity. Because of the travel built into the connections, the infill panels are expected to move independently initially while the Auditorium's drifts are not excessive. This means they are not expected to contribute to the lateral load resistance of the structure.

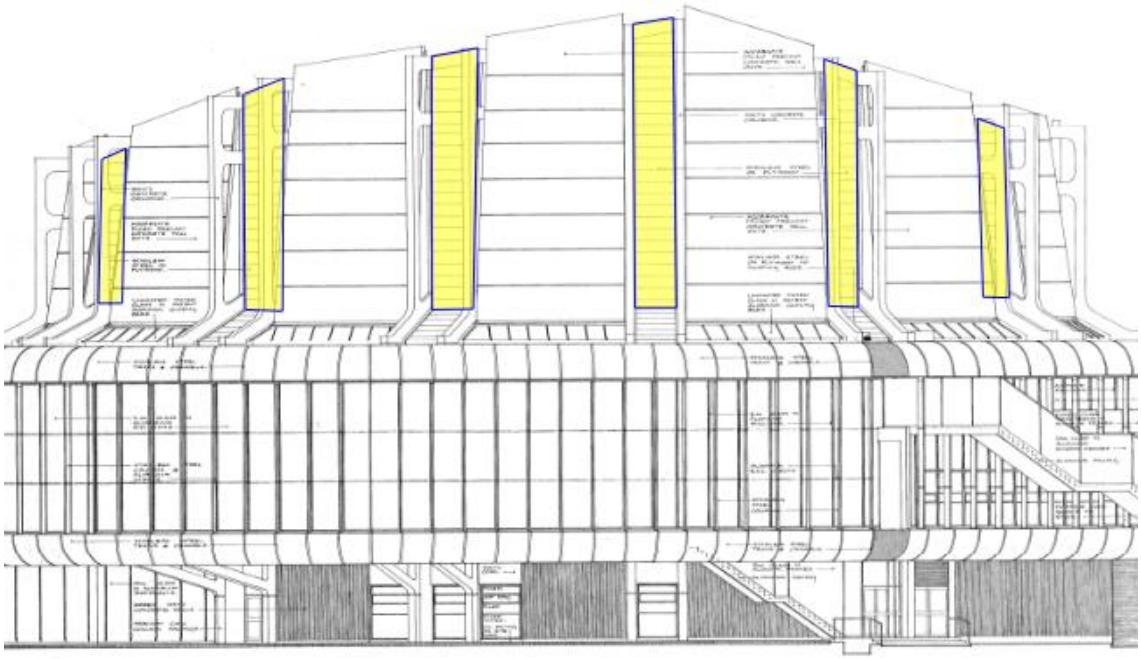

Figure G.6-1: Snapshot taken from drawing 27 of the 1980's Warren & Mahoney architectural drawing set - showing the East auditorium elevation with the precast infill panels highlighted yellow.

Deformation-based assessment

A deformation-based assessment has been used to determine the available in-plane drift capacity in the infill panels. Two issues in particular have been checked:

- At what drift level do the slotted bolted connections reach the end of their travel. Given the size and weight of the solid concrete panels, the bolts are likely to shear once the end of travel has been reached and the system "lock's up". This could lead to the panels losing their support.
- At what drift level do the edges of the panels start contacting the adjacent bay walls. I.e. does the system "lock up" before the travel in the bolted connection is used up. If this occurs before failure of the connections, the primary lateral load resisting system would stiffen and create an

unintended load path, that would induce large shears in the Bay Walls. The geometric effects could potentially push the panels out of the building.

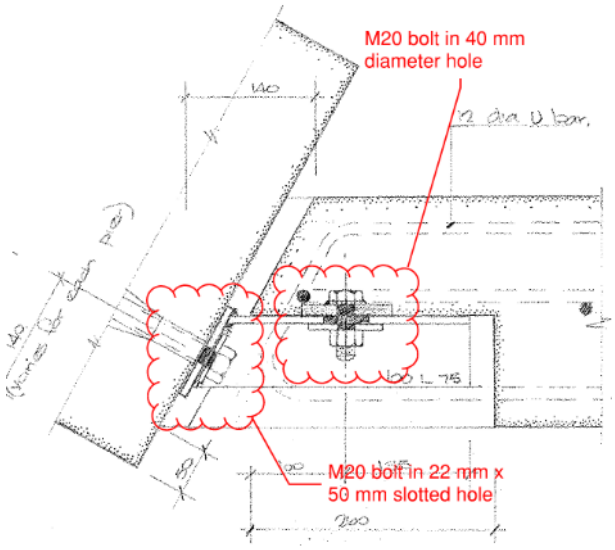

Figure G.6-2: Snapshot taken from drawing S81/8 of the 1980's Holmes, Wood, Poole & Johnstone drawing set - showing the connection detail between the precast buttresses and the precast cladding infill panels.

Assuming that the bolts are initially centred in the oversized holes, using a simple geometric analysis, the Auditorium can drift 0.33% before the bolts touch the edges of the holes. At this drift level, the panels have also used up 13.9 mm out of the available 25 mm gap to the internal columns of the precast buttresses. This means that the displacement capacity in the connection is expected to be exceeded before the infill panels "lock up" between the 'Terriers'.

The maximum inter-storey drift demand between Level 4 and the roof structure is 0.93% in the transverse (Z) direction of the Auditorium, however these drifts are not linear. At 35%ULS seismic loading, the drifts at the roof level are 0.43%. Therefore, the precast infill panels have a seismic score of **25%NBS$_{IL3}$**.

A construction tolerance has not been accounted for and the bolts are initially considered to be centred in the slotted hole. Assuming a construction tolerance would result in a lower seismic score, however, since this is already below <33%NBS$_{IL3}$, this is not further investigated.

Force-based assessment

A force-based assessment has been used to determine the ability of the connections to support the panels under seismic face loading. Calculations have been undertaken using loading to the 'Parts & Components' section Chapter 8 of NZS1170.5:2004 and assuming that the connection due to the mild steel angle brackets are able to respond with a ductility of μ_p = 2.0. Steel connections that behave more brittle, such as bolts or welds in shear have been assessed with nominal ductility (μ_p = 1.25). Shallow embedded anchors in the concrete have been assessed with no ductility (μ_p = 1.0).

This gives the connections a seismic score of 100%NBS$_{IL3}$.

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G.7 STAIRS

The stairs in the Michael Fowler Centre were not explicitly modelled as part of the primary structure due to the number of elements and the fact they span between multiple seismically separated structures. Generally, they were not expected to contribute significantly to overall building behaviour or influence the performance of lateral load-resisting systems. Where they were expected to influence performance, e.g. Stairs 1/2 in the independent staircase structures, they were included as part of the analysis.

G.7.1 Method of Assessment and Seismic Rating

The stairs were generally assessed in accordance with Section C10 of the Engineering Assessment Guidelines. The following checks were carried out considering the location and configuration of each stair structure.

Force-Based Assessment

Inertial seismic demands were checked in accordance with NZS1170.5:2005, with stair structures connected to the primary structure assessed in accordance with the provisions for 'Parts and Components' to Chapter 8 of NZS1170.5:2004. Inertial demands were checked where significant mass eccentricities existed, typically at sliding connections where no translational restraint was provided at one end of the stair span.

Displacement-Based Assessment

The capacity for stair structures to undergo inter-storey drifts under seismic loading was checked in accordance with Section C10.4.5 of the Engineering Assessment Guidelines. The earthquake score was assessed using Eq. C10.1 of the Engineering Assessment Guidelines, where an element performance factor of, $Q = S_p/2$ was used for stair spans supported on ledges. Estimated inter-storey drift demands at varying levels of earthquake shaking used for the stair assessments are summarised in Table G.1-3.

Consequences during lateral spreading of the ground

Several stair structures were assessed against potential consequences of the lateral spreading due to liquefaction triggering on site. The critical stair structures typically spanned between independent or poorly tied foundation systems where differential ground movement could impose displacement demands on the stair structure.

G.7.2 Stair 1/2

Stairs 1 and 2 are located withing the Independent Staircase Structures and span from Level 1 in the Auditorium to Level 2 in the Renouf Foyer. The stairs are integrally cast into the primary wall of the Staircase Structures and the foundation beam at Level 1. The top of the stair flight cantilevers over the secondary stair wall (W66) and is seismically separated from the Renouf Foyer, sitting below the Level 2 floor slab.

As there is no significant mass eccentricity and the stairs are seismically separated with no displacement compatibility issues. Stairs 1 and 2 are not considered to pose an individual structural weakness under earthquake shaking.

G.7.3 Stair 3/4

Stairs 3 and 4 are located in the Auditorium next to Bays 3(a). The stairs span between the ground floor level and Level 2 with an intermediate landing supported by adjacent bay wall. The stairs are cast in-situ with integral connections into both floor slabs and starter bars spanning out from the bay walls at regular intervals.

Considering the low drift demands expected at the base of the bay walls and the additional reinforcing tying the stair into the bay walls, the stairs are not considered to pose a structural weakness due to

minimal displacement demands under seismic loading. Their rating is tied to the performance of the bay walls at Bays 3(a).

As the stairs are integrally connected at the top bottom, there is minimal mass eccentricity under inertial seismic loading; Because of this, a force-based assessment was not considered.

G.7.4 Stair 5/6

Stairs 5 and 6 are located in the Auditorium next to Wall 1 (east and west). The stairs span between the ground floor level and Level 2 with an intermediate landing supported by Wall 1.

Similar to Stairs 3 and 4, they are of cast in-situ construction, with reinforcing tied into the both the floor slabs and the adjacent Wall 1. As such, they are also not expected to pose a significant life safety hazard due to the low drifts in a seismic event. Their rating is tied to the performance of Wall 1

As the stairs are integrally connected at the top bottom, there is minimal mass eccentricity under inertial seismic loading. Because of this, a force-based assessment was not considered.

G.7.5 Stair 7/8

Stairs 7 and 8 are located next to the exterior wall of each Independent Staircase Structure on their external faces. The stairs span from the ground floor level under the Renouf Foyer to Level 2 of the Auditorium with an intermediate landing connected to the Independent Staircase Structures. The stairs were retrofitted in 2016, where an additional concrete support frame was attached to the intermediate landing to provide seating to a sliding joint detail as shown in Figure G.7-1.

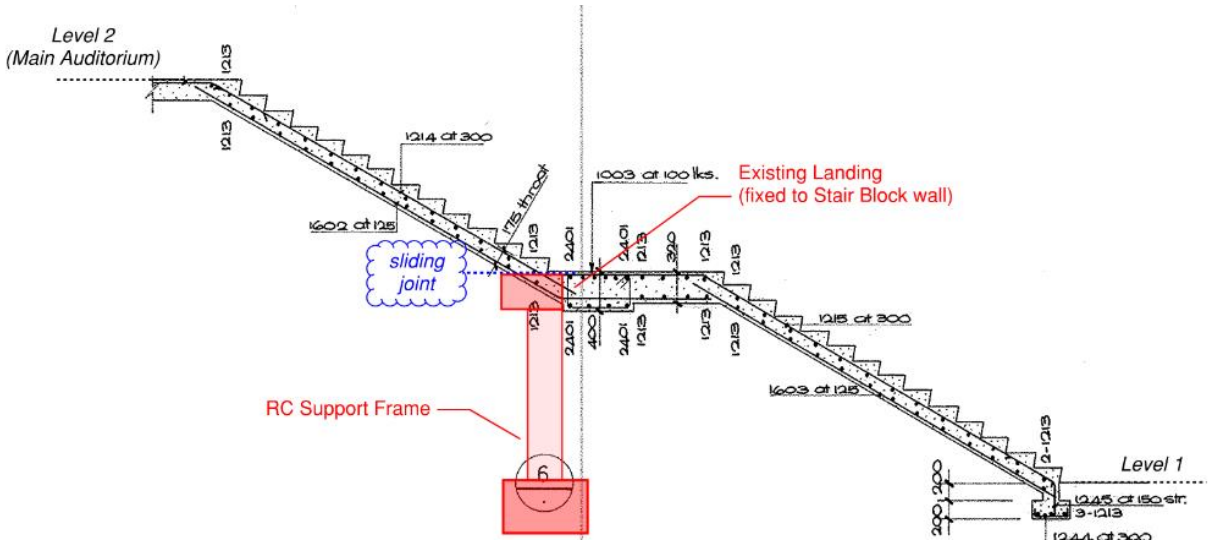


Figure G.7-1: West elevation of Stairs 7 (Stair 8 opp. hand), with indicative retrofitted RC frame and sliding joint shown.

The upper and lower stair flights were seismically separated in 2016, with the 'new' frame providing seating to the upper flight. During construction, the seating detail was modified based on the on-site stair dimensions, resulting in an extended concrete corbel detail as shown in Figure G.7-2.

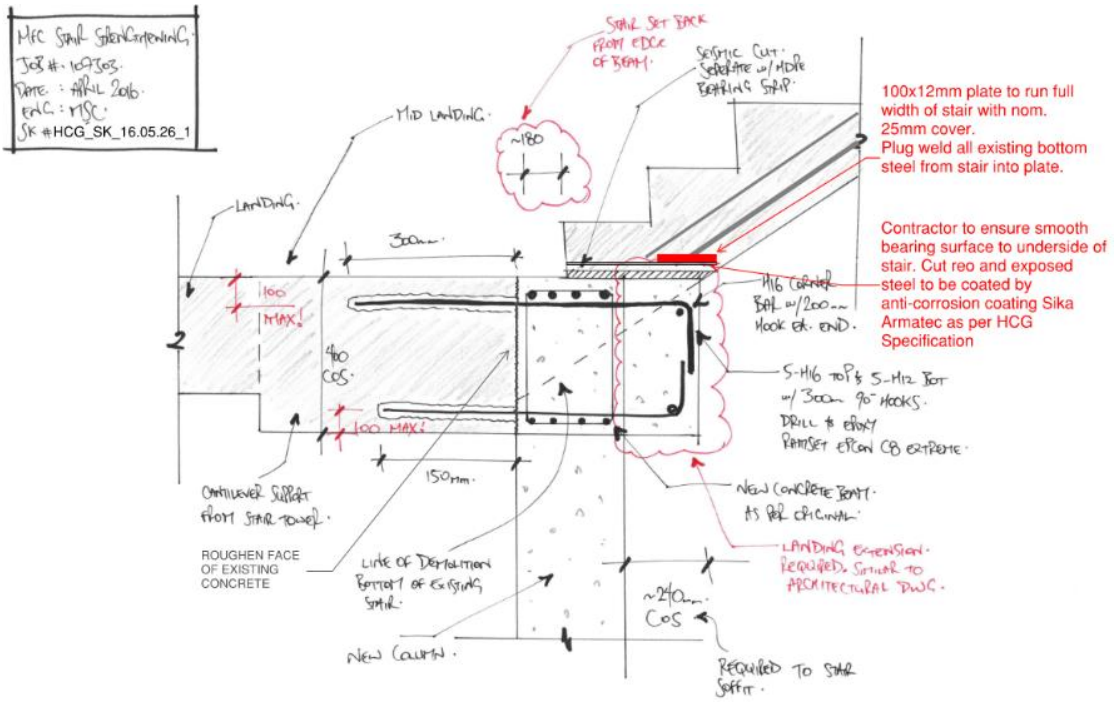


Figure G.7-2: Holmes detail (HCG-SK16.05.26-1) showing extended corbel detail supporting the upper flight of stairs 7 and 8.

Displacement-based assessment

Based on the sliding joint detail, the full 175 mm width of the stair throat is landing on the concrete corbel. Considering a stair inclination angle of 30 degrees, the available seating length could be estimated considering 20 mm cover to the stair reinforcing as, $\delta_{capacity} = \frac{175\ mm}{\cos(30°)} - 20mm = 330\ mm$.

The seating capacity was evaluated considering displacements in the Independent Staircase Structures and the Auditorium under earthquake shaking the N-S (X) direction. The inter-storey drift demands at 100%NBS$_{IL3}$ as shown in Table G.1-3 were used to estimate the displacement demand on the sliding joint as 22 mm (0.5% drift) and 63 mm (3.0% drift) in the Auditorium and the Independent Staircase Structures. The possible concurrency of displacement demands between the Auditorium and Staircase Structure was considered using Equation C2B.1 from the Engineering Assessment Guidelines shown below.

$$S = \sqrt{\delta_1^2 + \delta_2^2}$$

Where: δ_i = the lateral deflection of building 'i' relative to ground, and S = the separation between the buildings. The displacement demands were also increased by 1/S$_p$ (building-specific) before calculating the likely total displacement demand corresponding to S$_p$ = 1.0 using Equation C2B.1. This resulted in an estimated displacement demand of, δ_{demand} = 95 mm.

The seating length could be assessed considering an element performance factor of, Q=0.5, in accordance with Section C10.4.5 of the Engineering Assessment Guidelines. The earthquake score of the sliding joint could therefore be determined as, $\frac{Q \cdot \delta_{cap}}{\delta_{demand}} \cdot 100\%NBS(Il3) = \frac{0.5 \cdot 330\ mm}{95\ mm}$ >100%NBS$_{IL3}$.

Force-based assessment

The upper stair flight is effectively horizontally cantilevering from the fixed connection to the Auditorium at Level 2. Considering lateral loading defined using the provisions of 'Parts & Components' from Chapter 8 of NZS1170.5:2004, the transverse moment demand could be determined. The upper flight of the stair had an estimated seismic weight ($G + \psi_E Q$), of 8.8 kPa (61 kN total). The horizontal design coefficient was determined as, $C_{(part)} = 1.7$ considering a nominally ductile response ($\mu = 1.25$), $h_i = 4.2$ m and, $h_n = 9.2$ m. Based on the horizontal stair span of 3.9m, this corresponded to in-plane bending demand of 205 kNm at the top of the upper flight at level 2.

Moment redistribution was considered to redistribute any minor axis bending of the stair slab at the top support such that the reinforcing could be relied on to solely resist seismic demands. This was considered reasonable as there was sufficient bottom reinforcement in the stair flight to take the full gravity ($G + \psi_E Q$) moment demand assuming no flexural restraint from the supports (pinned-pinned conditions).

The in-plane bending capacity of the stair flight was assumed to be limited by the 6-D12 slab reinforcing bars which lap with the top reinforcing of the stair flight. The probable in-plane flexural capacity based on the slab reinforcing could be determined as, $M_{IP,prob} = 160$ kNm. The earthquake score could then be determined based on the capacity-to-demand ratio as, $\%NBS_{IL3} = 160/205 = 80\%NBS_{IL3}$. The shear strength was similarly evaluated and found not to be critical.

Consequences during lateral spreading of the ground

Stairs 7/8 run underneath Stairs 11/12 at the same location, but Stairs 11/12 have smaller seating provisions. A potential structural weakness at Stairs 11/12 could lead them to fall onto Stairs 7/8, which is considered a life-safety hazard. Therefore, Stairs 7/8 is given the same consideration for the potential consequences during lateral spreading of the ground as Stairs 11/12.

For Stairs 11/12 no step change reduction was considered, however, the displacement capacity associated with a life-safety hazard was exceeded and so the seismic score for Stairs 7/8 is $>45\%NBS_{IL3}$.

Consequences of Independent Staircase Structures overturning

The Independent Staircase Structures were found to exceed the overturning capacity of the foundation system at around 25%ULS base shear demand, as the piles exceed their tension capacity.

As the Independent Staircases uplift northwards, this uplifts the flights of Stairs 7 and 8. The lower flights of the stairs are lifted off their seating, which will put a hogging flexural demand on the flights at the location of the mid-height landing. For the upper flights, this will put a sagging flexural demand on the flights at the upper most connection. In the opposite direction, as the Independent Staircase Structures uplift southwards, it is expected that the opposite occurs. Though this will be to a much smaller extend, as the mid-height landing is located further south on the walls.

For the flights of Stairs 7 and 8, it is expected that the uplift of the Independent Staircase Structures will result in a flexural mechanism across the flights, as the displacements will put the sections beyond their yield strength. This will not result in a life safety hazard, as gravity support of the stairs is maintained through dowel action of the reinforcement in the stairs that has not yielded. The stair flights will become simply supported, and do not rely on the fixity at their upper end.

For the mid-height landings, the assessment has considered the combined flexural, shear and torsion demands from flexural overstrength of the stair flights. This found that the mid-height landings can accommodate this uplift and maintain support of the gravity demands from stair flights.

Neither of the mechanisms result in a life safety hazard.

G.7.6 Stair 9/10

Stairs 9 and 10 are located next to gridline 7 at the eastern and western of the Renouf Foyer. The stairs are of cast in-situ concrete construction consisting of two stair flights and an intermediate landing as shown in Figure G.7-3. Gravity loads from the landing are supported by a concrete cantilevered post, which continues as a Ø450mm 'Franki' pile. The cantilevered post was found to have negligible rotational fixity in the north-south direction with restraint in the east-west direction provided by a 400x400 mm foundation beam spanning back to the Independent Staircase Structures. The lower stair flight is integrally connected to the Renouf Foyer foundation beam at the ground floor level. There upper stair flight has a sliding connection at Level 2, including a steel 'catch bracket' installed in 2016 with the detail shown in Figure G.7-4 below.

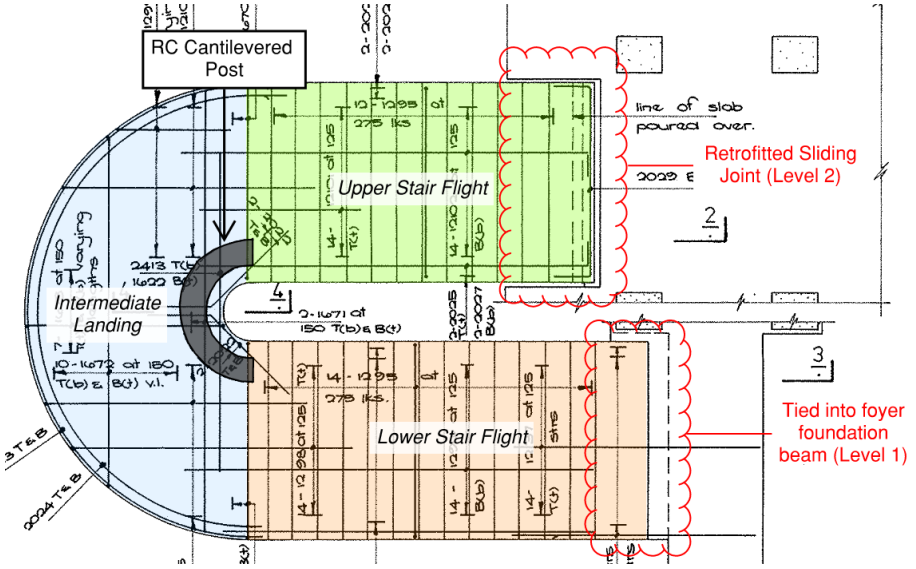


Figure G.7-3: Plan layout of Stair 9 (Stair 10 opp. hand)

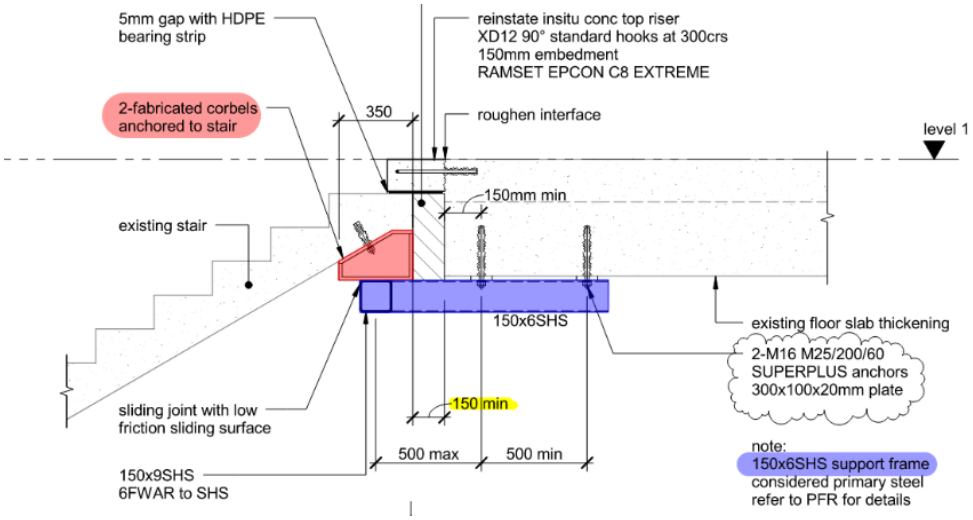


Figure G.7-4: Retrofitted sliding joint detail at Level 2 using an SHS 'catch bracket' with reinforced steel corbel

Displacement-Based Assessment

The retrofitted catch bracket has a minimum gap and seating length of 150mm based on the retrofitted detail in Figure G.7-4. The stairs are orientated in the transverse direction. Table G.1-3 shows that an inter-storey drift of 1.6% or 70 mm occurs at 67%NBS_{IL3}. The seating length could be assessed considering an element performance factor (Q), in accordance with Section C10.4.5 of the Engineering Assessment Guidelines. A structural performance factor of, S_p = 0.7 was used when determining the drifts at 67%NBS_{IL3}, resulting in an element performance factor of, Q = 0.35.

The earthquake score of the staircase sliding joint could therefore be determined as 50%NBS_{IL3} as shown below.

$$\frac{Q \cdot \delta_{cap}}{\delta_{demand}} \cdot 67\%NBS_{IL3} = \frac{0.35 \cdot 150\ mm}{70\ mm} = 50\%NBS_{IL3}$$

Force-Based Assessment

Based on Geotechnical Assessment Report, negligible base shear takeout will occur through either passive lateral resistance of the 400mm deep foundation beam (in the X direction) or through the pile itself. Based on this, lateral loads were assumed to be resisted through the cast in-situ connection between the lower stair flight and the Renouf Foyer foundation beam which will provide the stiffest lateral load path.

This stair flight connection to the foundation beam will be subject to combined bending and axial demands under seismic loading due to the mass eccentricity of approximately 1.4m under N-S loading (X direction) and 3.6 m under E-W loading (Z direction). The seismic demands were calculated considering the seismic hazard parameters per Section 6.3. The stair structure was assumed to have a short fundamental period (T_1 < 0.5s) and nominal ductility (μ = 1.25); This resulted in a horizontal design coefficient of C_d= 1.22 for the ULS earthquake load case. The internal actions were determined using an elastic model considering a total seismic weight of, W_t = 230 kN. The critical demands occurred at the base of the lower stair flight with the combined in-plane flexural and axial demands shown in Figure G.7-5 below. The earthquake score could be determined by reducing the seismic demands until the internal actions aligned with the interaction curve below. This resulted in a critical earthquake score of **45%NBS_{IL3}** in the Z direction.

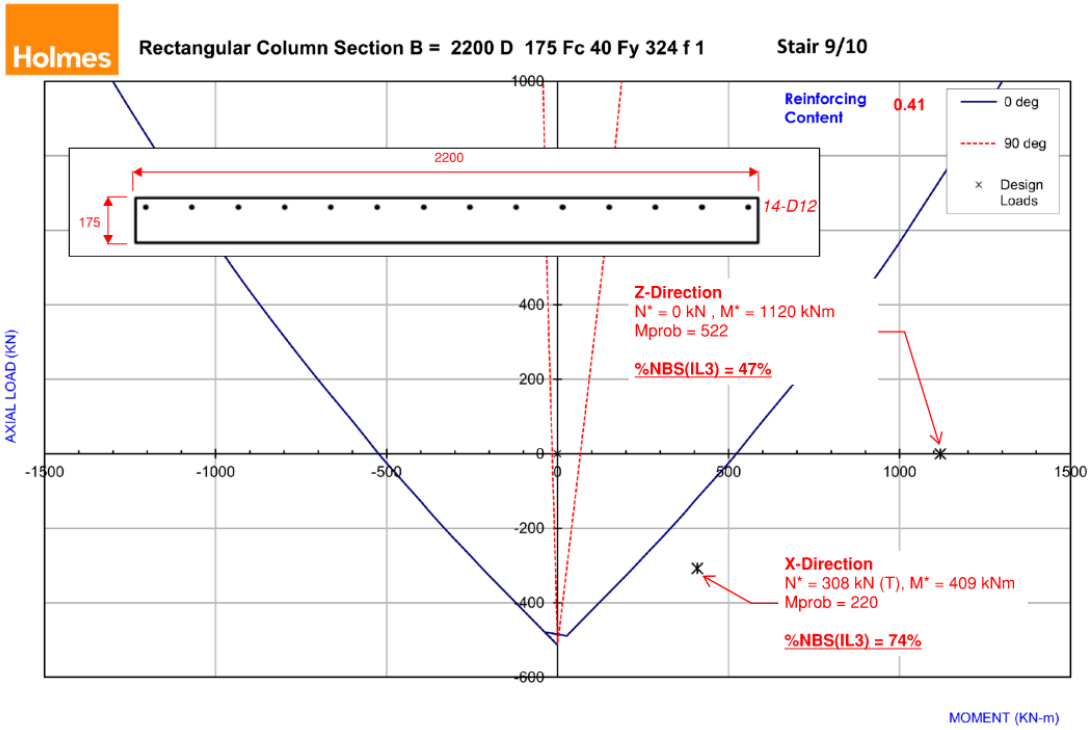


Figure G.7-5: Moment-Axial interaction curve for bottom flight of Stairs 9/10 compared with ULS seismic demands.

Consequences during lateral spreading of the ground

Stair 9 is located at the East end of the Renouf Foyer and is connected to both the Renouf Foyer foundations (through the lower stair flight) and the Independent Staircase Structure foundations (through the square 400mm ground beam). These connections, however, are too flexible to restrain differential ground movement between the foundation systems.

Localised ground movement due to lateral spreading was considered between the cantilever post supporting the Stair 9 Landing and the sliding connection at the Foyer. It was assumed that the Renouf Foyer foundation system remained tied together and a scarp formed around the perimeter of the Renouf Foyer foundation system.

The Stair 9 sliding connection has a total capacity of 150mm. In accordance with Section 8.8 of NZS1170.5:2004, this corresponds to an acceptable displacement of, δ_{SLSH} = 75mm under ULS demands.

This concluded that the differential lateral spread at this interface is not enough to cause a structural step-change and the associated seismic score is **>100%NBS$_{IL3}$**.

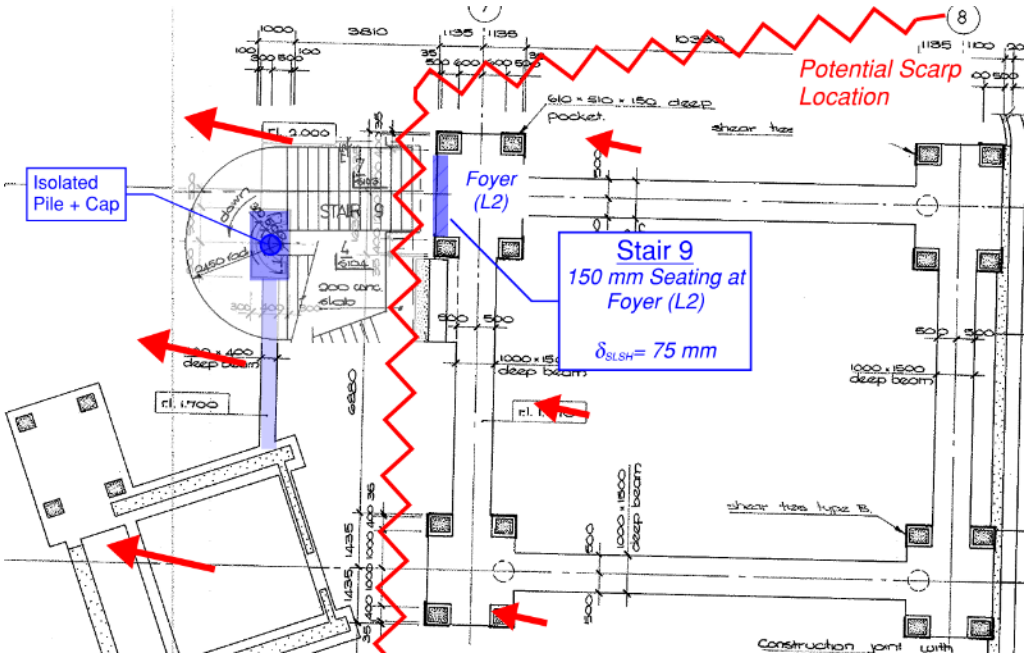


Figure G.7-6: Potential location of surface scarp due to lateral spreading that could impose displacements on the Stair 9 sliding connection.

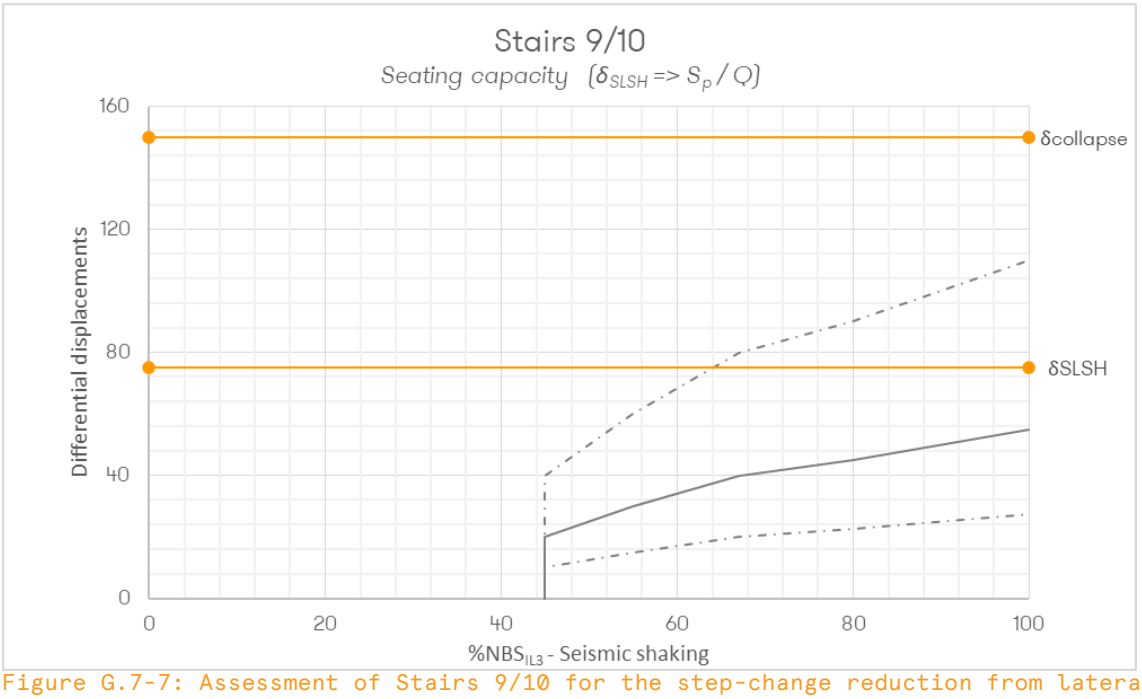


Figure G.7-7: Assessment of Stairs 9/10 for the step-change reduction from lateral spread

G.7.7 Stair 11/12

Stairs 11/12 are located next to the exterior wall of the Independent Staircase Structures on their external faces. The stairs span from Level 2 of the Renouf Foyer to Level 4 of the Auditorium with an intermediate landing connected to the Staircase Structures at Level 3. Both flights of the stairs were designed with sliding joints at the base, using a steel flat plate detail as shown in Figure G.7-8 below.

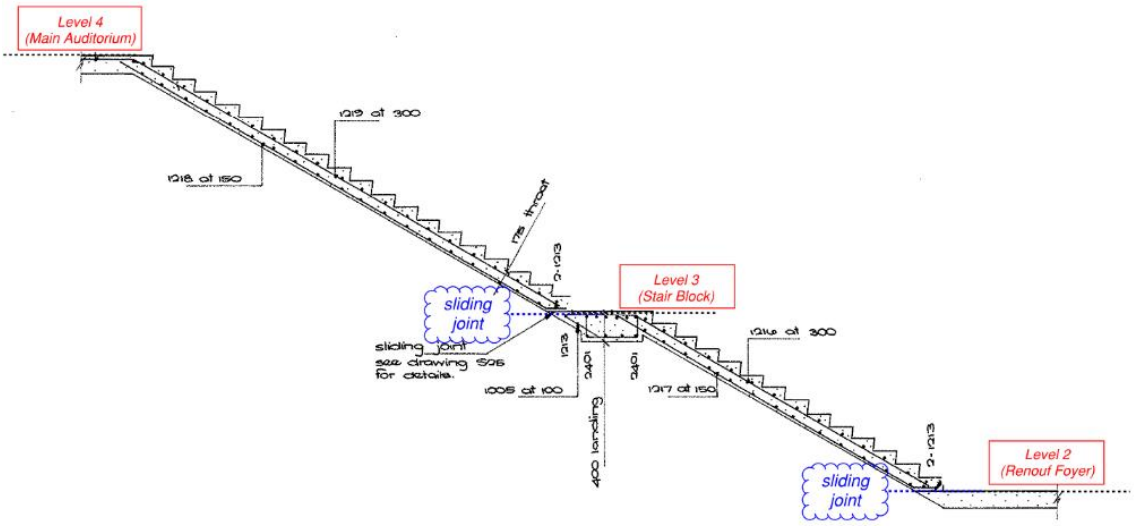


Figure G.7-8: Section of Stair 11 (Stair 12 opp. hand) showing location of sliding joint details.

Displacement-Based Assessment

The sliding joint details were assessed in accordance with Section C10.4.5 of the Engineering Assessment Guidelines. The typical detail for the sliding joint detail is shown in Figure G.7-9 below, with 150mm wide steel plates included to limit friction at the interface. Based on the detail, an available seating length of 200mm was conservatively assumed, ignoring the cover concrete forming the stair soffit due to possible spalling.

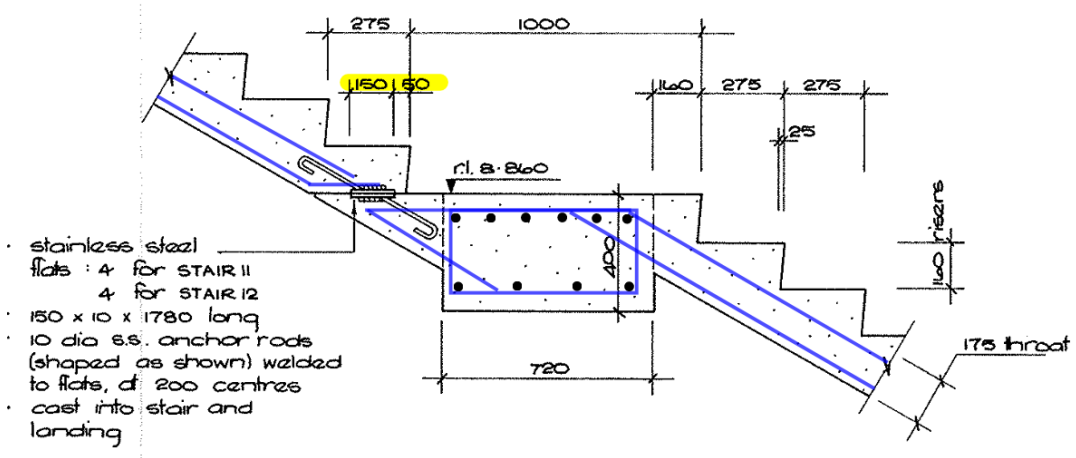


Figure G.7-9: Sliding joint detail for upper stair flight (sim. to sliding joint at Level 2). Indicative reinforcing layout also shown.

The seating capacity will be critical under N-S (X direction) displacements. The drift demands and corresponding displacement demand of the sliding joints at 67%NBS$_{IL3}$ and 34%NBS$_{IL3}$ are shown in Table

G.7-1. It was noted that significant non-linear displacements are expected in the 34-67%NBS_{IL3} range, especially within the Independent Staircase Structures. This led to a bounded approach to establish a reasonable range of possible displacement demands and consequently earthquake ratings for the stair seating ledges.

Table G.7-1:Drift demands (X Direction) on sliding joint connections for Stair 11 and 12.

Structure	Height of Connection to Stair 11/12 m	34%NBS_{IL3}		67%NBS_{IL3}	
		Inter-Storey Drift Ratio %	Displacement mm	Inter-Storey Drift Ratio %	Displacement mm
Auditorium	9.8 (Level 4)	0.3% (S_p = 1.0)	29	0.34% (S_p = 0.7)[1]	32
Staircase Structures	6.6 (Level 3)	0.7% (S_p =0.97)	46	1.34% (S_p = 0.7)	89
Renouf Foyer	4.32 (Level 2)	0.9% (S_p= 0.95)	38	1.6% (S_p = 0.7)	68

[1]*Calculated considering 0.67 times the peak Auditorium drift at 100%NBS_{IL3}*

The displacement demands at both sliding joints could be determined considering the possible concurrency of displacements between structures. It was noted that the displacement demands were increased by 1/S_p (building-specific) before calculating the likely total displacement demand corresponding to S_p = 1.0 using Equation C2B.1. An element performance factor of Q = 0.5 was used to assess possible seating loss (critical) in accordance with C10.4.5 of the Engineering Assessment Guidelines. The displacement demand, available seating and earthquake scores are shown in Table G.7-2 below. The displacement-based assessment of Stairs 11/12 resulted in an earthquake score of between **40-55%NBS_{IL3}** and 50-60%NBS_{IL3} for the lower and upper stair flights, respectively.

Table G.7-2: Earthquake score of Stairs 11 and 12 sliding joint connections.

Connection	Displacement Demand, δ_{demand} mm [S_p = 1.0]		Available Seating, $Q \cdot \delta_{capacity}$ mm	%NBS_{IL3}
	34%NBS_{IL3}	67%NBS_{IL3}		
Upper Sliding Joint (Level 3)	56	135	100	50-60%
Lower Sliding Joint (Level 2)	62	160	100	40-55%

Force-Based Assessment

Both stair flights are effectively horizontally cantilevering from their top connection to the sliding joint at the base of each flight which is unrestrained in the transverse direction. Considering lateral loading defined using the provisions of 'Parts & Components' from Chapter 8 of NZS1170.5:2004, the transverse moment demand could be determined.

The upper stair flight was considered critical as it has a longer span (larger mass eccentricity). The upper stair flight had an estimated seismic weight ($G + \psi_E Q$), of 8.8 kPa (85 kN total). The horizontal design coefficient was determined as, $C_{(part)}$ = 2.51 considering a nominally ductile response (μ = 1.25), h_i = 9.2 m

and, h_n = 9.2 m. Based on the horizontal stair span of 5.4m, this corresponded an in-plane bending moment demand of 580 kNm.

Moment redistribution under seismic loading was considered to redistribute any minor axis bending of the stair slab at the top support such that the reinforcing could be relied on to solely resist seismic demands. This was considered reasonable as there was sufficient bottom reinforcement in the stair flight to take the full gravity (G + ψ_EQ) moment demand assuming no flexural restraint from the supports (pinned-pinned conditions).

The in-plane bending capacity of the stair flight was assumed to be limited by the 7-D12 reinforcing bars which tie into the level 4 slab. The probable in-plane flexural capacity could be determined based on the reinforcing layout as, $M_{IP,prob}$ = 220 kNm. The earthquake score could then be determined based on the capacity-to-demand ratio as, $\%NBS_{IL3}$ =220/580 = 38$\%NBS_{IL3}$. The shear strength was similarly evaluated and found not to be critical.

Consequences during lateral spreading of the ground

Localised ground movement due to lateral spreading was considered to occur either:

- Between the Staircase Structure and the Renouf Foyer, imposing displacements on the lower flight sliding connection.
- Between the Staircase Structure and the Auditorium, imposing displacements on the upper flight sliding connection.

The upper and lower flights have sliding connections with similar seating. However, the magnitude of differential lateral spread is considered to be the greatest at the interface between the Auditorium and the Independent Staircase Structures.

The Stairs 11/12 sliding connections have a total capacity of 200mm. In accordance with Section 8.8 of NZS1170.5:2004, this corresponds to an acceptable displacement of, δ_{SLSH} = 100 mm under ULS demands. Figure G.7-11 shows that there is a margin to collapse and that therefore, no step-change has to be applied.

However, the lateral spread still causes a life-safety hazard, and therefore, the rating of **45$\%NBS_{IL3}$** is assigned to Stairs 11/12.

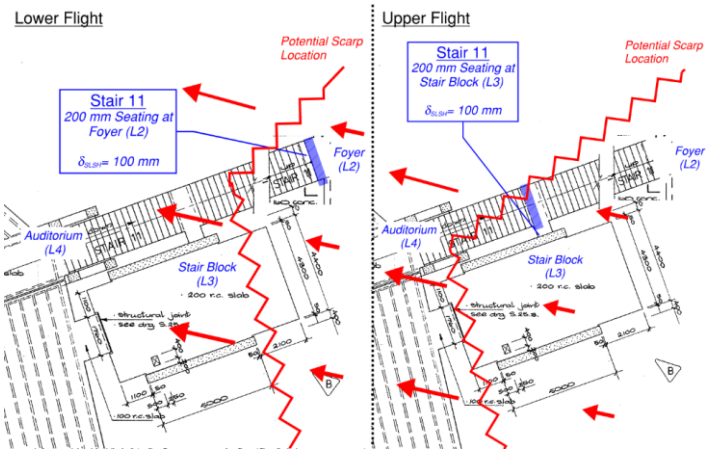


Figure G.7-10: Potential location of surface scarp due to lateral spreading that could impose displacements on the lower (left) and upper (right) Stair 11 sliding connection.

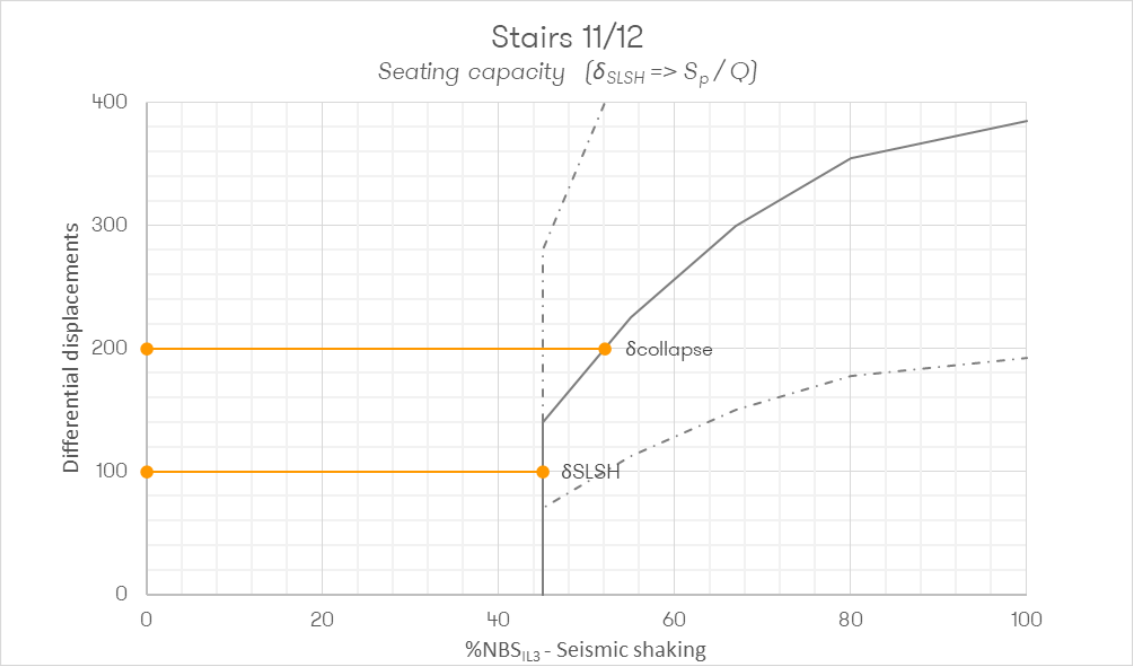


Figure G.7-11: Assessment of Stairs 11/12 for the step-change reduction from lateral spread

Consequences of Independent Staircase Structures overturning

The Independent Staircase Structures were found to exceed the overturning capacity of the foundation system at around 25%ULS base shear demand, as the piles exceed their tension capacity.

As the Independent Staircases uplift northwards, this uplifts the flights of Stairs 11 and 12. The lower flights of the stairs are lifted off their seating, which will put a hogging flexural demand on the flights at the location of the mid-height landing. For the upper flights, this will put a sagging flexural demand on the flights at the upper most connection. In the opposite direction, as the Independent Staircase Structures uplift southwards, it is expected that the opposite occurs. Though this will be to a much smaller extend, as the mid-height landing is located further south on the walls.

For the flights of Stairs 11 and 12, it is expected that the uplift of the Independent Staircase Structures will result in a flexural mechanism across the flights, as the displacements will put the sections beyond their yield strength. This will not result in a life safety hazard, as gravity support of the stairs is maintained through dowel action of the reinforcement in the stairs that has not yielded. The stair flights will become simply supported, and do not rely on the fixity at their upper end.

For the mid-height landings, the assessment has considered the combined flexural, shear and torsion demands from flexural overstrength of the stair flights. This found that the mid-height landings can accommodate this uplift and maintain support of the gravity demands from stair flights.

Neither of the mechanisms result in a life safety hazard.

G.7.8 Stair 13/14

Stairs 13/14 are in the Auditorium between gridlines 2E/W and 3E/W spanning between Level 2 and Level 3. The stairs are of cast in-situ concrete construction consisting of a single circular stair flight as shown in Figure G.7-12 with a centreline radius of 1.4 m and throat dimensions of 1.2x0.175 m. The stairs are integrally connected to the floor slabs at Level 2 and Level 3 (via a cast in-situ landing at Level 3) with no sliding connections to accommodate any displacement demands due to inter-storey drifts.

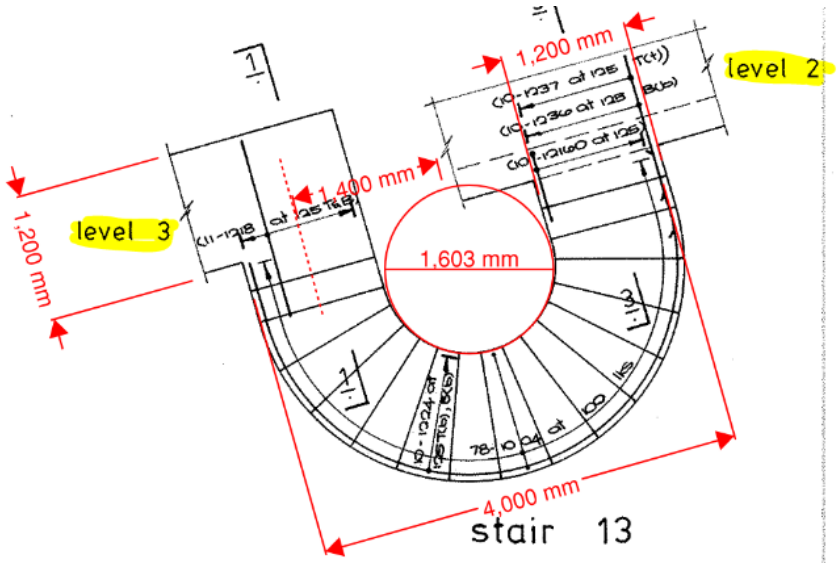


Figure G.7-12: Plan layout of Stair 13 (Stair 14 opp. hand)

Displacement-Based Assessment

To assess the capacity of the stairs to undergo inter-storey drifts between Level 2 and 3, an elastic Microstran model was developed to model the stairs as a flexural element. The stair throat was modelled considering the effective sections properties and set out as shown in Figure G.7-13 below. Mesh refinement was used to ensure the behaviour of the curved staircase was adequately captured by the elastic model.

The seismic weight (G + ψ_EQ load case) was applied to the model alongside imposed inter-storey drifts in both principal directions. Biaxial bending was found to be critical with the slight increase in axial load due to compression (N* < 50 kN (C)) conservatively ignored. The drifts were increased until biaxial bending was estimated to occur within +/- 20% as shown in Figure G.7-14. The stairs were seen to yield at drifts of approximately 1.0-1.1% and 0.2-0.3% in the X and Z directions, respectively. In-plane shear was also checked but, due to the presence of Ø10 stirrups at 100mm centres, shear was not critical.

The drift demands at the northern end of the Auditorium can be seen in Section C3.3, where drifts range between 0.1 to 0.2% between level 2 and 3 at 34%NBS_{IL3}. Considering the stairs flight should have some nominal ductility as flexural is estimated to occur, Stairs 13/14 are considered to pose a life safety hazard between **40-60%NBS_{IL3}**. The range of this earthquake score reflects the uncertainty in the performance of the curved stair system, whilst recognising the low drift demands at 34%NBS_{IL3} are unlikely to pose a significant life safety hazard.

Force-Based Assessment

As the stairs are integrally connected at the top bottom, there is minimal mass eccentricity under inertial seismic loading. Because of this, a force-based assessment was not considered.

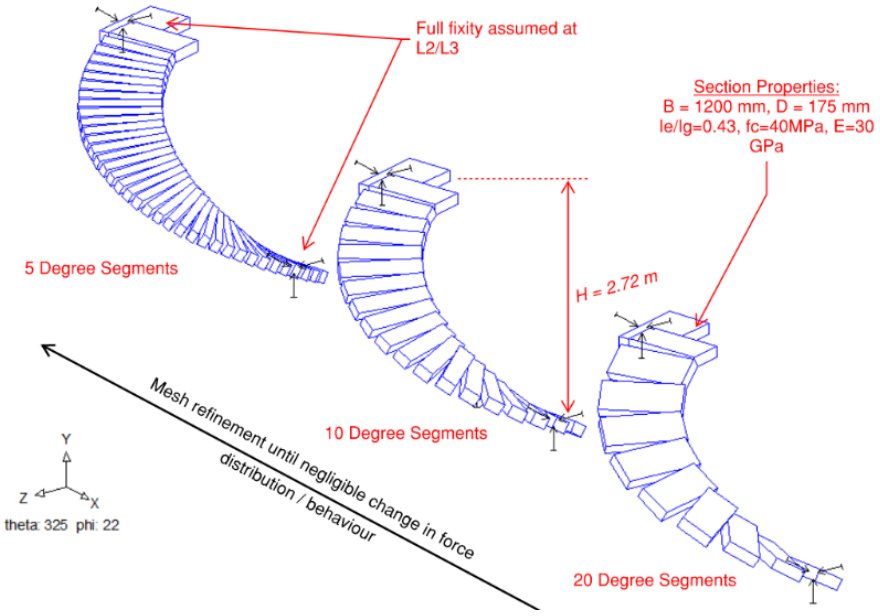


Figure G.7-13: Setout of MicroStran V10 model of Stair 13/14 to assess displacement capacity

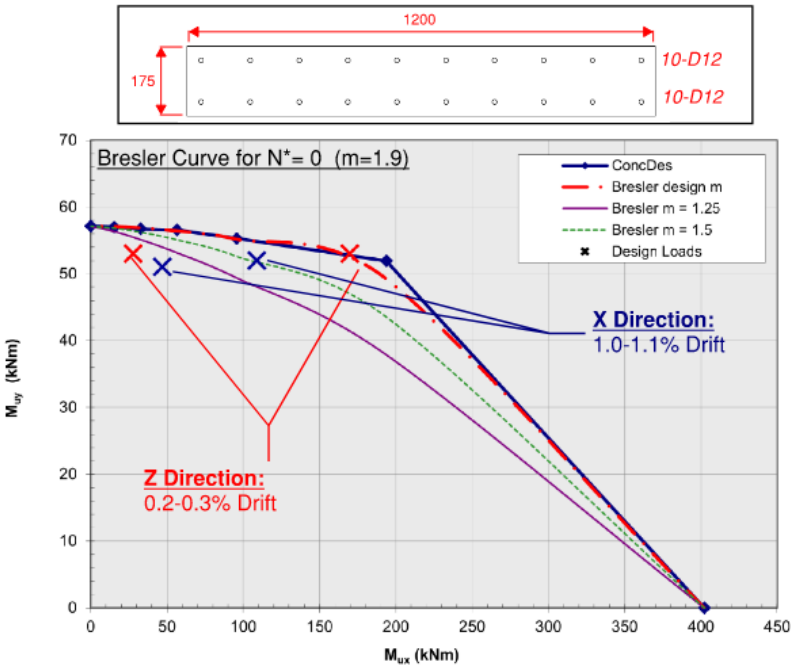


Figure G.7-14: Bresler biaxial bending interaction curve for Stair 13/14 with demands at first yield under imposed inter-storey drifts

G.7.9 Stair 15/16

Stairs 15/16 are enclosed stairs exterior to Auditorium between gridlines 1-2E/W spanning between Level 2, Level 3 and Level 4. They are located within the cladding framework referred to as the 'embryo'. The stairs are integrally connected at each level through cast in-situ landings which cantilever out from the floors at each level.

The stairs were retrofitted as part of remedial works carried out in 2016 as shown in Figure G.7-15 below. The remedial work included:

- Strengthening of the in-situ landings using flat steel plate and M16 bolts through the slab to provide additional hogging moment capacity and shear capacity.
- Installation of RHS 'catch frames' which span between landings to provide gravity support in case the in-situ stair flights are substantially cracked due to displacement demands under a seismic event.

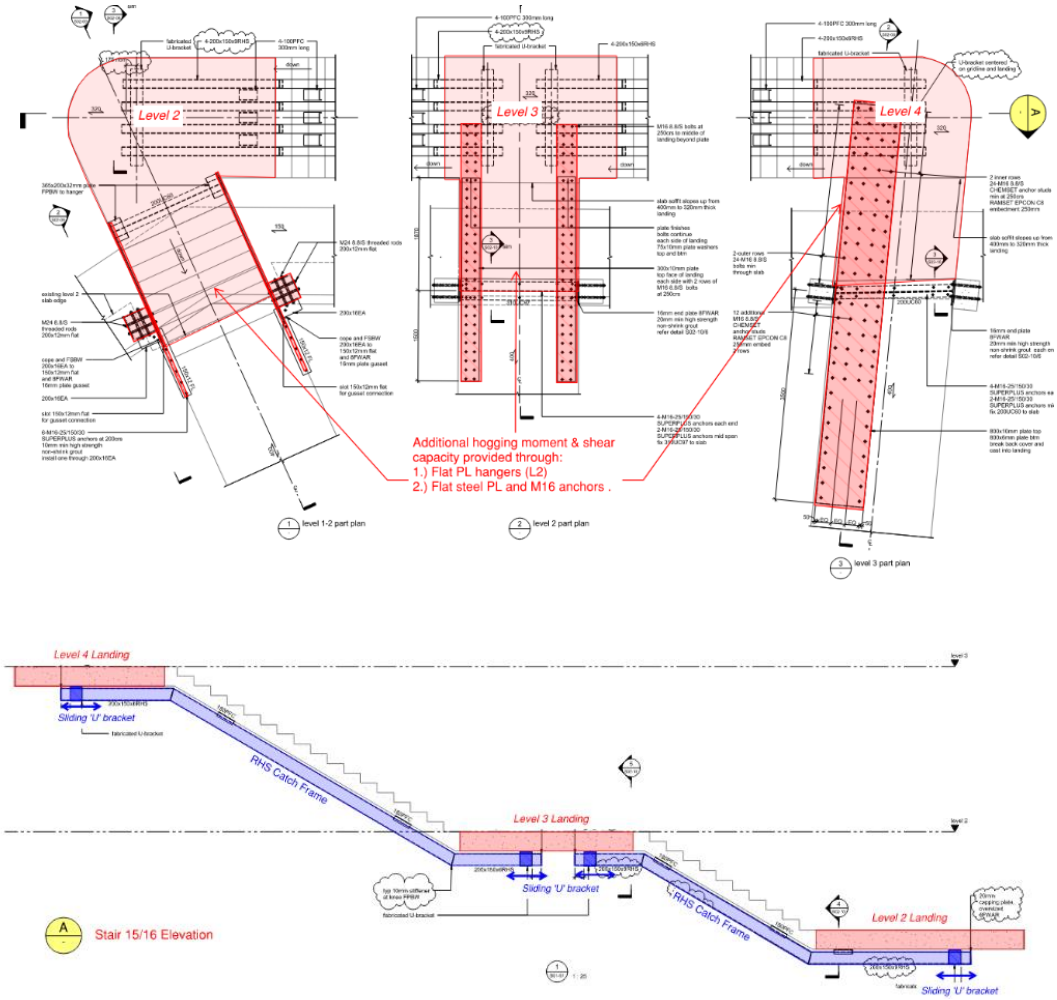


Figure G.7-15: Snapshot of Holmes Structural Drawings (S02-07 & S02-08) for 2016 MFC Stair Strengthening Work.

Although the remedial works do not explicitly allow for relative displacements between each floor level, there will be some capacity to sustain deformations beyond initial axial-flexure-shear of the stair flight due

to the works carried out. The retrofitted landings do appear to have sufficient reinforcement to maintain integrity in a torsionally cracked state. The RHS catch-frames should allow Stairs 15/16 to provide a viable egress route if damage to the stair flight begins becomes substantial. The stair flights can, however, still form an axial strut after initial cracking occurs, which will eventually lead to failure of the stair landings, mostly likely the central Level 3 landing due to the imposed forces from both stair flights. The displacement demands at which a significant life safety hazard may be estimated to occur due to twist of the landings is difficult to predict.

Considering the previous remedial work carried out, it was considered reasonable that the retrofitted landings and consequently Stairs 15/16 will not pose a significant life safety hazard under earthquake shaking below >34%NBS_{IL3} but the performance cannot be reliably predicted at higher earthquake shaking levels.

G.7.10 Stair 17/18

Stairs 17 and 18 are in the Auditorium between gridlines 6E and 6W spanning from Level 4 to the Director's area at Level 5. The stairs are of in-situ construction consisting of two stairs flights and an intermediate landing. The lower stair flight is integrally connected to the bay walls at Bays 7(a) with reinforcing spanning both longitudinally between wall flanges and transversely from the bay wall web. The upper stair flight spans between the intermediate landing and the Level 5 floor slab.

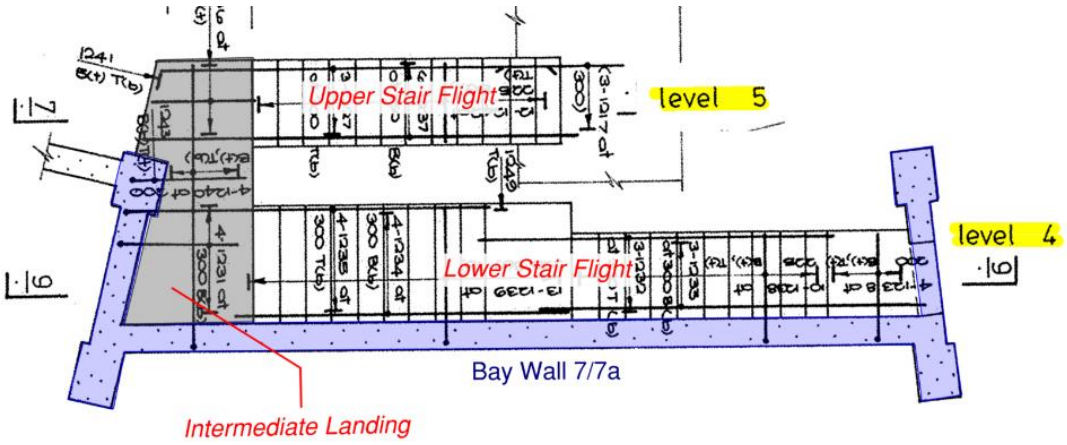


Figure G.7-16: Plan layout of Stair 17 (Stair 18 opp. hand).

Displacement-Based Assessment

The lower stair flight was found to be able to fully support its own seismic weight (G + ψ_EQ) through cantilever action from the adjacent concrete shear wall. As such, it was not considered to pose a life safety hazard due to displacement demands under seismic loading.

The upper flight spans 2.6m vertically between the intermediate landing and the Level 5 floor. Both the intermediate landing and Level 5 floor are supported by the bay walls and so the stairs should follow the displacement profile of these systems. Considering the location of the stair flight at the top of the bay walls, most drift demands will be due to rigid-body rotations (curvature of the wall tending to zero), which will limit the stresses imposed into the stair flight. Based on the above, the upper stair flight should not pose a significant life safety hazard due to limited drift demands under seismic loading.

Force-Based Assessment

As the stairs are integrally connected at the top bottom, there is minimal mass eccentricity under inertial seismic loading. Because of this, a force-based assessment was not considered.

G.8 CEILING PANELS

As mentioned in section 5.1.1, spanning between the bottom chords are a series of rail profiles that support the acoustic ceiling panels. The rail profiles comprise custom section fabricated out of an L-shape section and a T-shape section. The acoustic ceiling panels are fixed to these rail profiles with small angle brackets and 'tek' screws.

A force-based assessment was undertaken to assess the demands on the 'tek' screws and test their vulnerability. The inertia loading was determined in accordance with the provisions for 'Parts and Components' to Chapter 8 of NZS1170.5:2004. Their seismic score was found to be >100%NBS_{IL3}.

An assessment on the rail profiles also showed low seismic demands as they have a 'free-moving'/sliding connections to the bottom chords of the roof trusses. They were not considered to be the critical element.

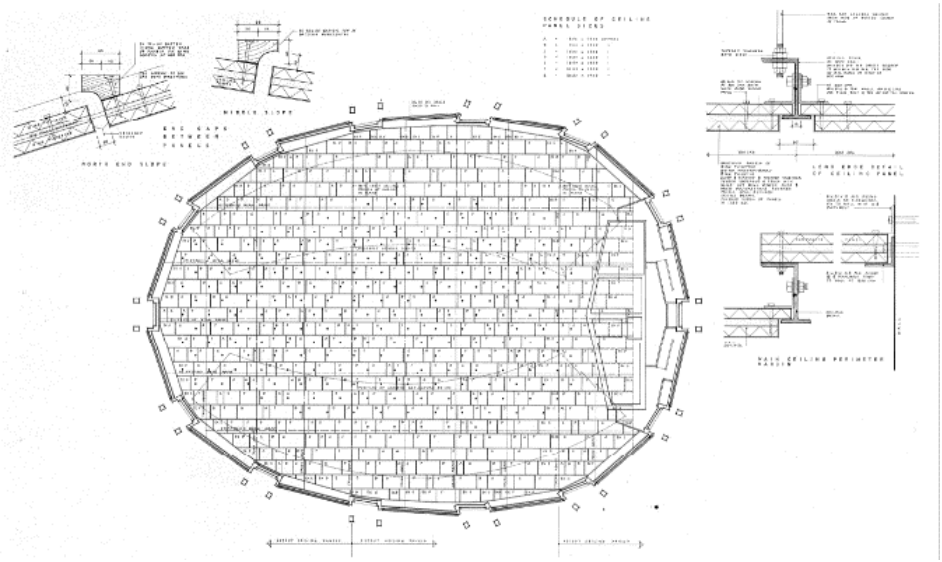


Figure G.8-1: Snapshot taken from drawing no. 17 of the 1980's Warren & Mahoney architectural drawing set - showing the Auditorium reflected ceiling plan.

Figure G.8-2: Photograph of the inside of the ceiling - showing the ceiling fixings

G.9 OPERABLE WALL

There are four operable sliding walls in the Renouf Foyer, refer Figure G.9-1 for a site photo. They are hung underneath roof trusses with no track at the floor level, a motor rolls out a chain in order to open or close the doors. There are two types of connections seen on site to hang the door track and casing, which are shown on Figure G.9-2. The doors on Truss 5 & 6 are supported with detail a) of and the doors connected to Truss 3 & 4 are using detail b).

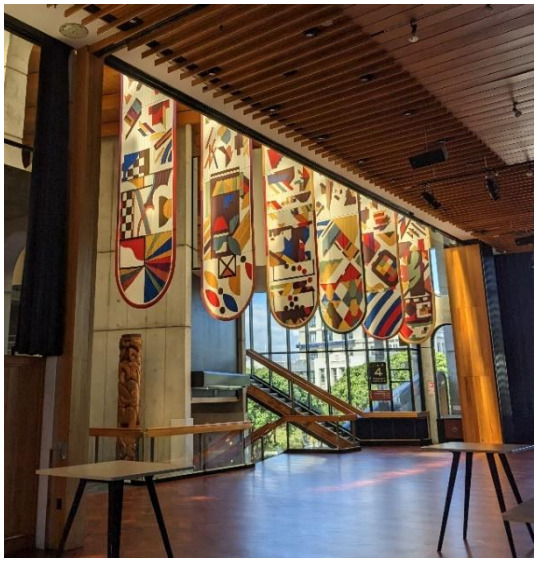


Figure G.9-1: Photo of Operable Wall

Detail b) to Truss 3 & 4 is not documented in the existing structural set and no direct attachment to the steel truss could be seen. The clamping system is considered to rock and bend around the bottom chord of the truss. Not leading to a life safety hazard.

In the plane of the truss 5 & 6, various mechanisms could occur: racking between truss and floor, motor chain lock up, or chain failure and rolling along track. Evaluation of these mechanisms is not conducted as part of this DSA. The swinging out-of-plane pendulum behaviour of the door is considered to dominate the response.

The expected maximum relative displacement that the wall will swing is estimated to be equal to the spectral displacement (S_d). For the Michael Fowler Centre site, the spectral displacement at 100%NBS_{IL3} for the period of the door is ~1.3m at the centre of mass (translating to 2.6m at the floor level).

The assessment found that the welds from the hanging plate to the bottom of the truss of detail a) are sufficiently strong to resist the plate expected plastic capacity. This is the only brittle mechanism seen to be present in the pendulum behaviour. Plate bending and bolt elongation will be extensive but is considered to have sufficient ductility capacity to not be considered a life safety hazard. This steel softening could mean that the door sits down on the Renouf Foyer floor and the 2.6m expected displacement would not occur.

a)

b)

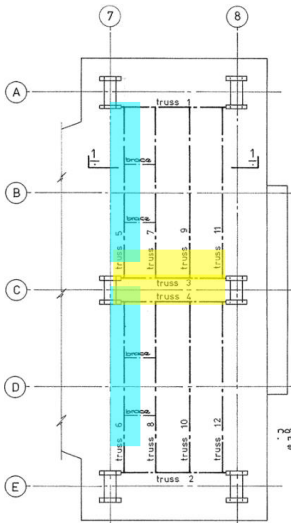


Figure G.9-2: Plan of operable wall locations and details a) for Truss 5 & 6 and b) for Truss 3 & 4

G.10 SUMMARY OF IDENTIFIED STRUCTURAL WEAKNESSES

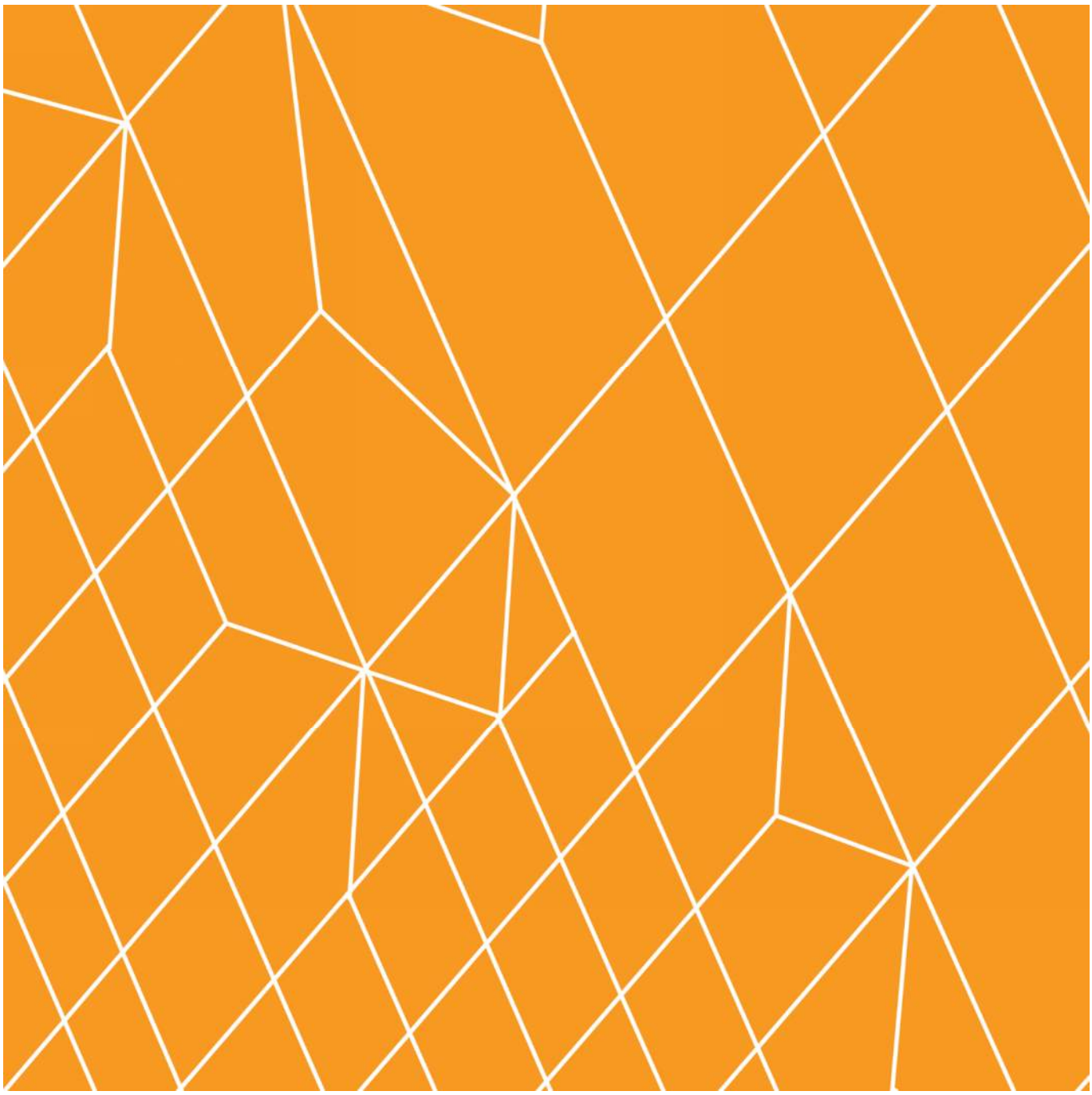
Structural Weaknesses are those deficiencies identified which have an assessed seismic rating of less than 100% NBS. These are listed in Table G.10-1.

Table G.10-1 Seismic Rating of potential Structural Weaknesses in the secondary and non-structural elements

Building Element	Structural Weakness	%NBS$_{IL3}$
Catwalks structures	Exceedance of capacity of in-plan bracing.	20-30
Wall reflectors	Concrete break-out from excessive forces on the post-installed anchors	20
Promenade roof & façade system	Exceedance of deformation capacity of existing glazing suite	25
Precast infill panels	Limited displacement compatibility with the primary structure of the Auditorium	25
Stairs 15/16	Loss of integrity of the landings at large inter-storey drifts.	35
Stairs 11/12	▪ Exceedance of flexural capacity from in-plane loading. ▪ Lack of displacement compatibility of lower and upper stair flight sliding connections. ▪ Differential ground movement from lateral spreading of the ground exceeding the seating capacity.	40-45
Stairs 7/8	Differential ground movement from lateral spreading of the ground exceeding the seating capacity.	45
Stairs 13/14	Integral cast insitu connection leading to excessive flexure of stair flight under imposed displacements.	45-60
Concrete masonry block walls	Exceedance of out-of-plane capacity of block walls at Level 4 around services shaft	50
Stairs 9/10	Exceedance of both displacement capacity and structural capacity at comparable seismic demand due to: ▪ Lack of displacement compatibility of level 2 sliding connection. ▪ Exceedance of flexural capacity of lower stair flight due to significant mass eccentricity under inertial loads.	50
Rigging points in the Renouf Foyer	Exceedance of flexural capacity under 'Parts and Components' loading	70
Precast double-tee flooring at Level 2 of the lobby area in the Auditorium	Loss of seating due to excessive drifts of the primary structure	80

Appendix H

Precast Flooring Assessment



Wellington City Council
Michael Fowler Centre - Seismic Strategy: Auditorium Promenades/Precast Seating Units

111 Wakefield Street
Wellington

Report

Revision 0
18 October 2019
137071.01

Holmes Consulting

Report
Michael Fowler Centre - Seismic Strategy: Auditorium Promenades and Precast Seating

Prepared For:
Wellington City Council

Date: 18 October 2019
Project No: 137071.01
Revision No: 0 - Draft

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1 EXECUTIVE SUMMARY

Holmes Consulting LP (Holmes[1]) have undertaken a review of the Auditorium promenades and precast seating units of the Michael Fowler Centre (MFC) for the purposes of providing high-level recommendations and options on next steps to take to reduce potential seismic risk associated with structural response of the Auditorium promenades and precast seating units. This report is not a Detailed Seismic Assessment (DSA) and does not seek to update the seismic rating of the structure. Global building structural capacity has previously been assessed as above the earthquake prone threshold in a 2014 seismic assessment (1) along with a number of localised structural vulnerabilities. Several of the localised structural vulnerabilities, identified in the 2014 seismic assessment, were addressed as part of strengthening works undertaken in 2015 (2).

The Promenades of the MFC consist of mixture of in situ slabs and precast double-tee units. Precast double-tee units are located on Levels 2 and 4. Units span between precast concrete beams which cantilever from precast concrete piers. The double-tee units are topped with a concrete topping slab. The majority of the double-tee units are web-supported with end units being flange-supported. Both web-supported and flange-supported units have adequate seating for a performance >67% New Building Standard for Importance Level 3 (NBS_{IL3}) for a loss of support failure. Flange-supported units are susceptible to a secondary "bird's mouth" failure at the support. This mode of failure necessitates the addition of supplementary supports for all flange-supported units.

There are two distinct sections in the Entry Foyer of the Auditorium. On the southern side, 500-deep double-tee units are continuous over "outrigger" beams which cantilever from precast concrete piers. The extreme ends of the units are anchored into an in situ "edge beam" and the units are post-tensioned. These units are not vulnerable to a loss of support failure. The northern side of the Entry Foyer consists of 350-deep double-tee units seated on a precast concrete beam or pier and are anchored into the edge beam. When assessed for a loss of support failure, the units perform adequately with performance >67%NBS_{IL3}. However, differential displacements between the Auditorium and Renouf Foyer could potentially unseat these units, hence necessitating installation of supplementary supports at these locations.

Additional precast seating units are present on Level 2 of the Auditorium. These units are similar in form to a double-tee unit and are web-supported on precast concrete beams. Seating units are held in place with a holding down bolt detail. A loss of support failure is not expected and the assessed performance is likely to be >67%NBS_{IL3} without any intervention.

2 INTRODUCTION

Holmes Consulting LP have been engaged by Wellington City Council to provide a report with high-level recommendations and options to reduce the seismic risk of the promenades and precast seating units of the Michael Fowler Centre (MFC) Auditorium. This report is not a Detailed Seismic Assessment (DSA) and is intended only to inform Council of possible options and recommendations for next steps.

The MFC is a performing arts centre in central Wellington. It was designed in 1980 to NZS4203:1976 seismic loadings that are between one third and half of the seismic loadings that would be required under current seismic design codes. The MFC comprises three structurally separate structures (Auditorium, Stair Blocks, and Foyer). For the purposes of this report, only the Auditorium is considered. The Auditorium promenades of the MFC consist of a mixture of in situ slabs and precast double-tee units spanning to concrete beams at Levels 2 and 4. These beams typically cantilever from precast concrete piers. Double-tee units vary in size between the two levels are typically topped with a concrete topping slab. Precast seating units are present

[1] Any reference to "Holmes" in this report refers only to Holmes Consulting LP, unless specifically noted otherwise.

on Level 2 of the Auditorium. These units are similar in form to a double-tee unit and are web-supported on precast concrete beams. Those seating units are held in place with a holding down bolt detail.

The building's use dictates that it is an Importance Level 3 (IL3) structure, meaning that the seismic loads are 30% higher than an Importance Level 2 structure.

3 SCOPE OF WORK

This workstream is part of a larger project at the MFC which is evaluating various elements of the building as identified in the Holmes proposal of 2 May 2019.

The scope of work for this workstream included the following:

1. Review of existing documentation;
2. Preparation of inspection strategy;
3. Inspection and documentation;
4. Assessment of these components to the NZSEE Guidelines section C5 (2018 update[2]);
5. Preparation of recommendations, including schematic details, to NZSEE Guidelines section C5 (2018 update).

4 LIMITATIONS

Findings presented as a part of this project are for the sole use of the client in its evaluation of the subject properties. The findings are not intended for use by other parties and may not contain sufficient information for the purposes of other parties or other uses.

Our observations have been visual only and are limited to representative samples, as described in our record of observations. Our observations have been restricted to structural aspects only. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been inspected or reviewed, and secondary elements such as windows and fittings have not generally been reviewed.

Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

5 PREVIOUS INVESTIGATIONS

Holmes undertook a qualitative and detailed seismic assessment of the MFC in 2011 and 2014, respectively (3) (1). The seismic assessment in 2014 reported a global building rating of approximately 50-60% New Building Standard, Importance Level 3 (NBS_{IL3}) across the MFC complex. The Auditorium structural performance is governed by overall strength capacity of the building. Other potential localised vulnerabilities, which may limit building (or component) seismic performance, were identified, including geotechnical issues, seismic performance of stair elements and concerns over the Auditorium roof detailing. The seismic issues associated with the stair elements were addressed through strengthening works completed in 2015 (2), to raise these to a performance level greater than the assessed global building rating.

[2] The revised (yellow) version of C5 released in November 2018 represents the latest information available to engineers on precast concrete flooring and other aspects of existing concrete buildings. The yellow version of C5 should be used when carrying out all building assessments, with the exception that the July 2017 Version of the Engineering Assessment Guidelines (also known as the Red book) should be used when establishing whether or not a building is earthquake-prone under the Building Act 2004.

Note, due to changes introduced in 2017, the 2014 report does not fully meet the technical requirements for it to be considered a Detailed Seismic Assessment (DSA), for the purposes of determining a building's seismic rating following The Seismic Assessment of Existing Buildings: Technical Guidelines for Engineering Assessments (the Guidelines) (4).

6 AUDITORIUM OBSERVATION

Representatives from Holmes visited the MFC Auditorium on 19 August 2019 to observe the conditions and any damage observed. The area of observation was limited to the Entry Foyer of the MFC Auditorium. Visual observation of the double-tee units in the Entry Foyer did not identify any damage to precast units.

Holmes did not conduct any detailed measurements or inspections. Any further work to this area as recommended in Section 8 of this report should be done in consultation with a suitably qualified contractor, with input from the owner/operator (WCC/Venues Wellington).

7 ANALYSIS AND ASSESSMENT FINDINGS

7.1 Assessment of Precast Double-Tee Units to C5 Guidelines (2018 update)

The limiting drift capacity of precast flooring units have been assessed under the NZSEE Guidelines section C5 2018 update (5). These guidelines provide assessment procedures for a variety of precast concrete flooring, including double-tee units. Precast double-tee units can be supported on either the flange or the web. Both conditions are present in the Auditorium of the MFC. Flange-supported units were typically detailed using a "loop bar" or "pigtail" support detail as shown in Figure 1. Double-tee units are assessed for two failure types: loss of seating (both flange-supported and web-supported units) and a "bird's mouth" failure, where the corner of the unit cracks and opens (flange-supported units only).

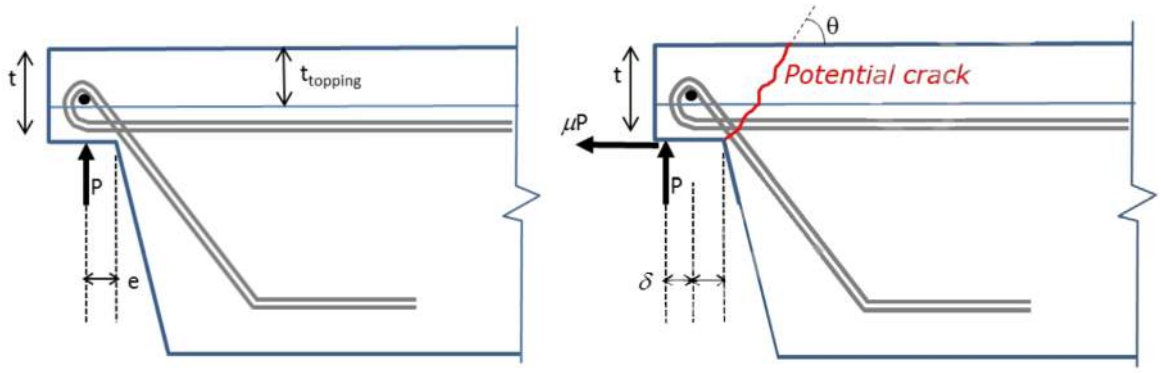

Figure 1: Typical pigtail hanger detail for a flange-supported double-tee unit with "bird's mouth" failure shown on right.

Loss of seating of a double-tee unit is influenced by a combination of several factors including:

- movement of precast floor units relative to the supporting ledge as a result of elongation and rotation of support beams with seismic actions;
- construction practices when built, which may have reduced seating from what is shown in the original construction drawings;
- spalling of concrete from the front face of the support ledge and the back face of the precast floor unit;
- creep, shrinkage, and thermal movement of the floor;
- crushing of concrete resisting the support reaction due to high bearing stresses.

A “bird’s mouth” failure can occur where the overhanging flange intersects with the web, the “bird’s mouth”. If the “bird’s mouth” opens, the unit can fail. The “pigtail” detailing of flange-supported units of this era make these units particularly susceptible to failure due to the lack of reinforcement to resist the opening of the crack. However, there is not practical guidance on how to quantify this failure and obtain a limiting drift. It is therefore recommended to provide some form of supplementary support where flange-supported detailing is present.

Per the NZSEE Guidelines, findings are calculated and reported in terms of a limiting drift, which is the overall building drift that results in the specific failure mode being assessed. The calculated limiting drifts are then compared to the expected building drift at earthquakes corresponding to a %NBS. A summary of unit limiting drifts and corresponding %NBS can be found in Table 3 Section 7.3.

7.1.1 Level 2 Promenades

On Level 2, the promenade wraps around the auditorium between gridlines 3E and 3W, with the areas between 5E and 5W being precast double-tee units. The double-tees are highlighted in Figure 2. At these areas, the promenade system consists of 500-deep double-tee units with a 50 mm concrete topping. These precast units are seated on precast concrete beams which cantilever from concrete shear walls. Units are typically web-supported except for units seated on grids 5E and 5W, where units are flange-supported. A 300x800 beam runs parallel to units on the outer edge of the promenades. The promenade is symmetrical about gridline C.

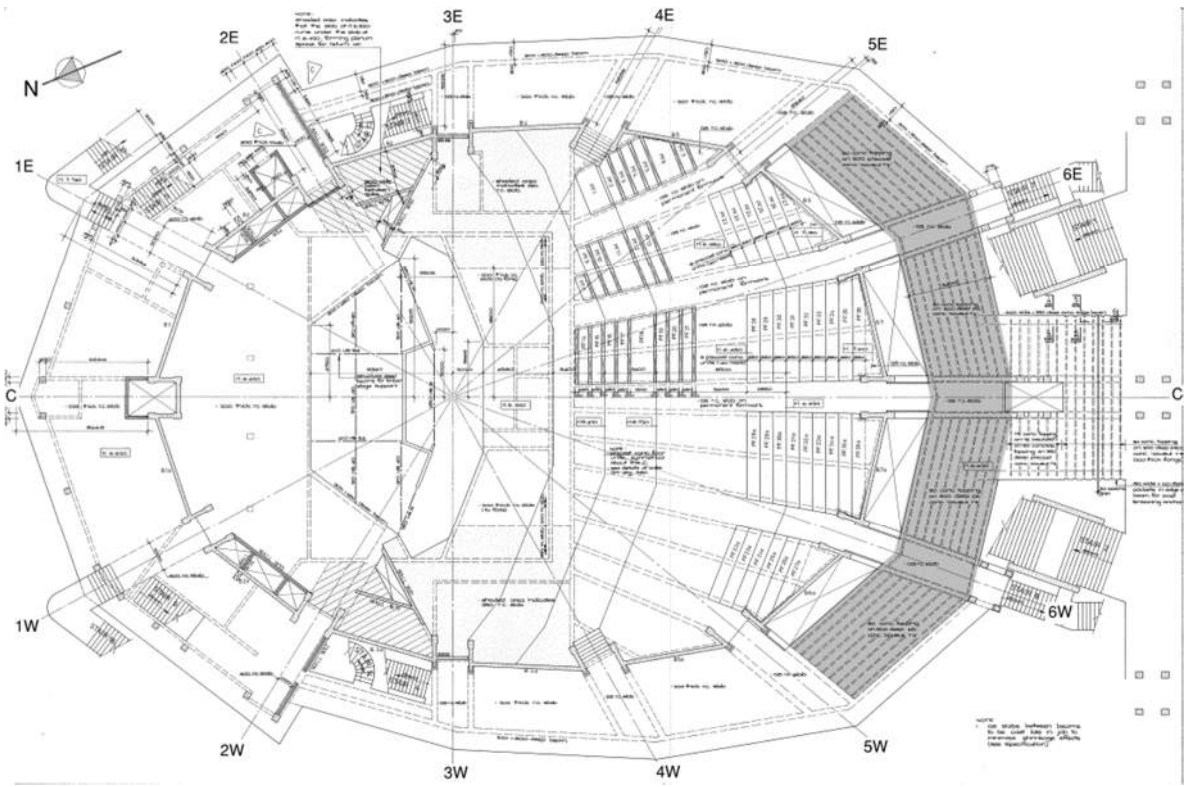

Figure 2: Level 2 part plan showing location (grey highlight) of Promenade precast double-tee units.

To assess the Level 2 promenades to the C5 guidelines (2018 update), several representative units were selected. In general, longer units are more critical for a loss of support condition as the increased span leads to a larger seating demand for bearing. Consequently, two units were selected for evaluation, which

are shown in Figure 3. Unit 1 is flange-supported on one end and web-supported on the other. It is oriented closest to the North-South direction. Unit 1a is web-supported on both ends and has a longer span. The unit is oriented in the East-West direction which was identified as the critical direction for building drift in the 2014 Detailed Seismic Assessment (1). Seating details for the two units are shown in Figure 4.

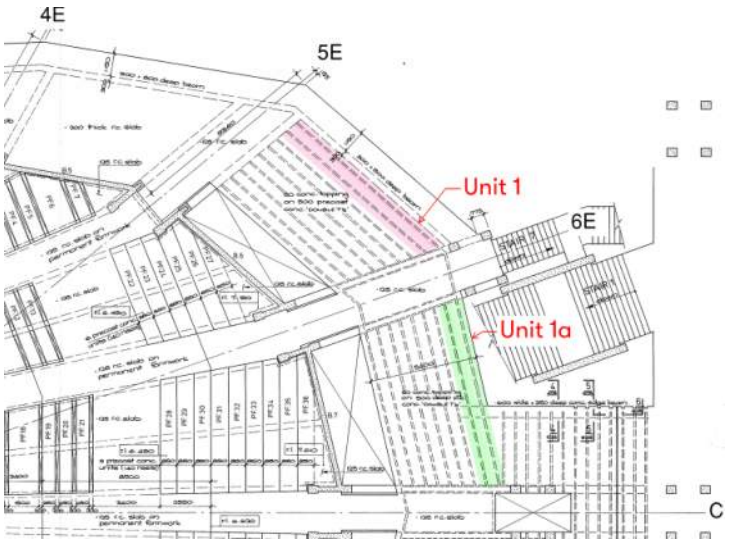

Figure 3: Level 2 part plan showing locations of typical promenade units.

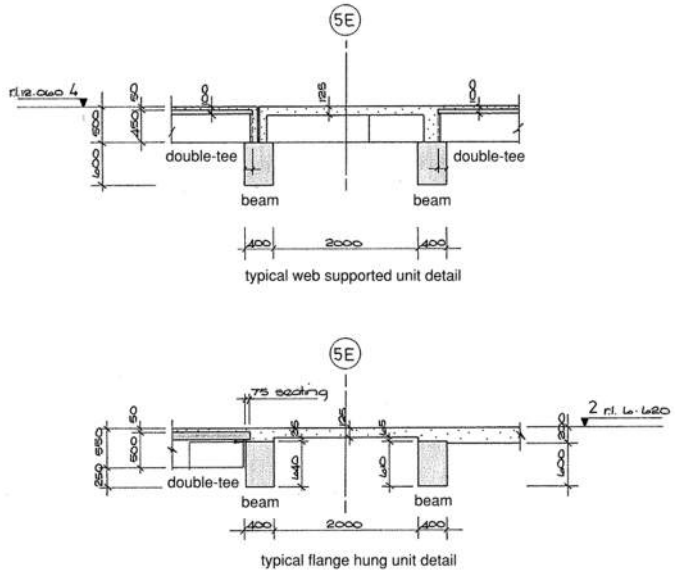

Figure 4: Typical unit seating details for Level 2 Promenades.

In accordance with the C5 guidelines (2018 update), Units 1 and 1a were assessed for loss of support as a result of frame elongation, rotation of support beams, construction tolerance, spalling, creep, and bearing failure. A limiting drift of 1.6% and 1.8% were found for each unit respectively. Unit 1 was also identified as being susceptible to a “bird’s mouth” failure on gridlines 5E and 5W. We can extrapolate these results to the other units in each of these bays, and those of the symmetrical bays across gridline C.

7.1.2 Level 2 Entry Foyer

The Level 2 Entry Foyer, highlighted in Figure 5, joins the main Auditorium promenades to the seismically separate Renouf Foyer. A series of post-tensioned 500-deep double-tee units with a 50 mm concrete topping are located between the Renouf Foyer and lift void, immediately above the glazed doors. Between the lift void and Auditorium, units are 350-deep double-tees with a 75 mm concrete topping, 75 mm insulation layer and 50 mm concrete screed. These 350-deep double-tees are not post-tensioned.

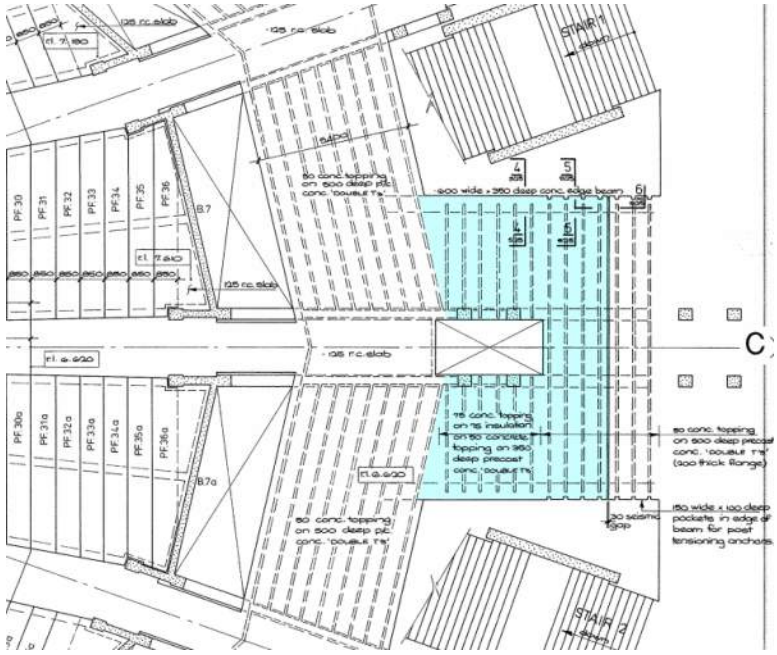

Figure 5: Level 2 part plan showing Entry Foyer precast double-tee units.

The 500-deep double-tee units are web-supported and continuous over “outrigger” beams. These beams cantilever from the main Auditorium precast concrete piers either side of grid C (around the lift void). The external ends of the precast double-tee units are anchored into an in situ “edge beam” as shown in Figure 7. As units are continuous over internal support beams, they are not subject to a loss of support condition. Drawings indicate pockets for post tensioning anchors which appear to tie the edge beam and 500-deep double-tee units together.

350-deep double-tee units are seated on an internal beam/pier and anchored into the in situ “edge beam”. Seating on precast concrete piers is provided by 85 mm deep concrete pockets. The edge beam is supported by the post-tensioned 500-deep double-tee system on the south side and the back span of the edge beam into the auditorium promenade area on the north side.

With the system described above several areas were identified for assessment. Loss of support was checked for the pier seated 350-deep double-tee units (Unit 2) shown in Figure 6. This unit was selected due to the minimal seating length provided. Assessment for a loss of support failure along the C5 guidelines (2018 update) included consideration of rotation of support beams, construction tolerance, spalling, creep and bearing failure. No beams lie parallel to units in question and consequently frame elongation has not been considered. Based on this assessment a limiting drift of 2.1% was determined for Unit 2.

Units in the Entry Foyer could be subject to the effects of differential displacement between the Auditorium and Renouf Foyer in the East-West direction. A shear connector detail bridges the 30 mm seismic gap between the Auditorium and Renouf Foyer, as shown in Figure 7. This shear connection has the potential to

shift the Auditorium double-tee units on the gridline C support beams due to differential displacements of the Renouf Foyer building. This situation is similar to a loss of support condition with a similar requirement for supplementary supports at the ends of the units closer to the lift void.

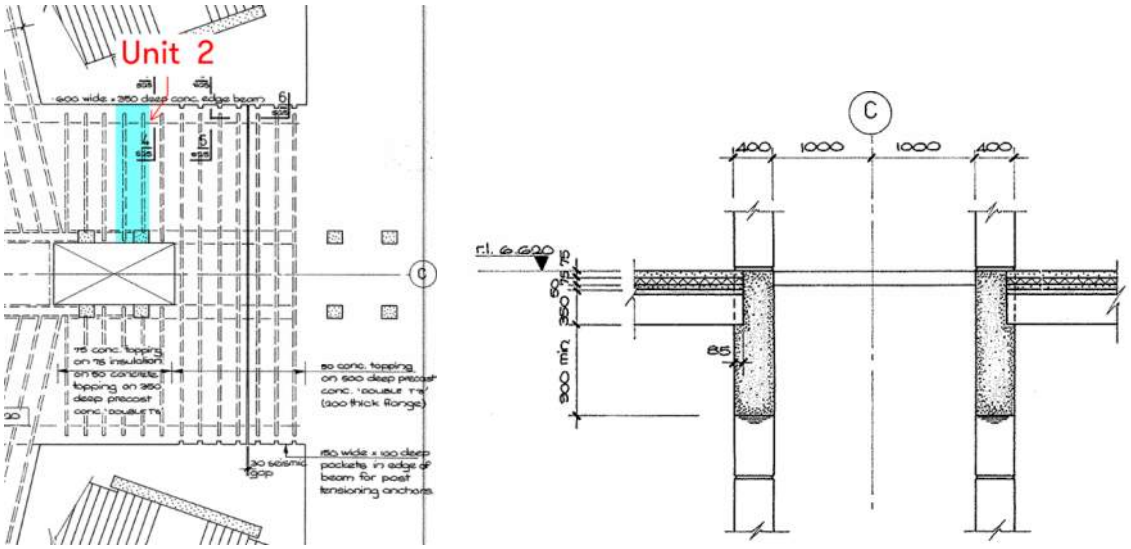

Figure 6: Left - Level 2 part plan showing typical Entry Foyer 350 double-tee unit.
Right - detail of unit supported on precast concrete pier.

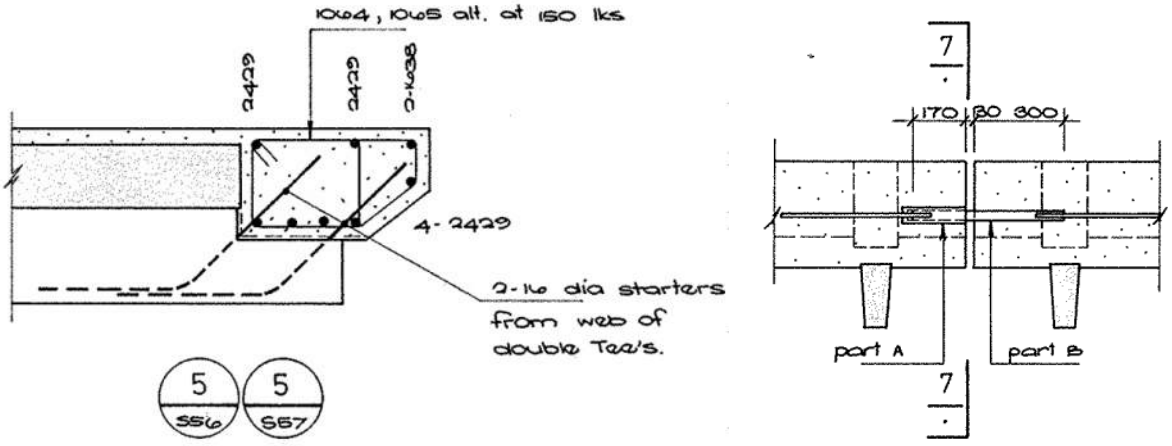

Figure 7: Left - Connection of double-tee units to in situ edge beam.
Right - Shear connector detail.

Additionally, the connection between the double-tee units and perpendicular edge beam was evaluated in accordance with the C5 guidelines (2018 update). In this connection, the development length of the plain diagonal starter bars from the double-tee units to the edge beam was identified as critical. Development length of a bar is critical in developing the full tensile capacity through friction with the surrounding concrete. As the provided development length was found to be less than that required by current standards, the tensile capacity of the starter bars was then scaled down proportionally as detailed in the

C5 guidelines. This lowered capacity was then compared to the demand due to gravity and horizontal and vertical seismic loads. The capacity of the starter bars in the connection was found to be adequate at this level of loading.

7.1.3 Level 4 Promenades

The promenades on Level 4 wrap around the auditorium between gridlines 2E and 2W, as shown in Figure 8. In these locations, 450-deep double-tee units with a 50 mm concrete topping are seated on precast concrete beams. These beams cantilever from the precast concrete piers located either side of each gridline. Units on GL 2E and 2W are flange-supported while all others are web-supported. The promenade is symmetrical about gridline C.

In order to assess the Level 4 promenades to the C5 guidelines (2018 update), several representative units were selected. Longer units in the direction of building maximum drift are the most critical for a loss of support condition.

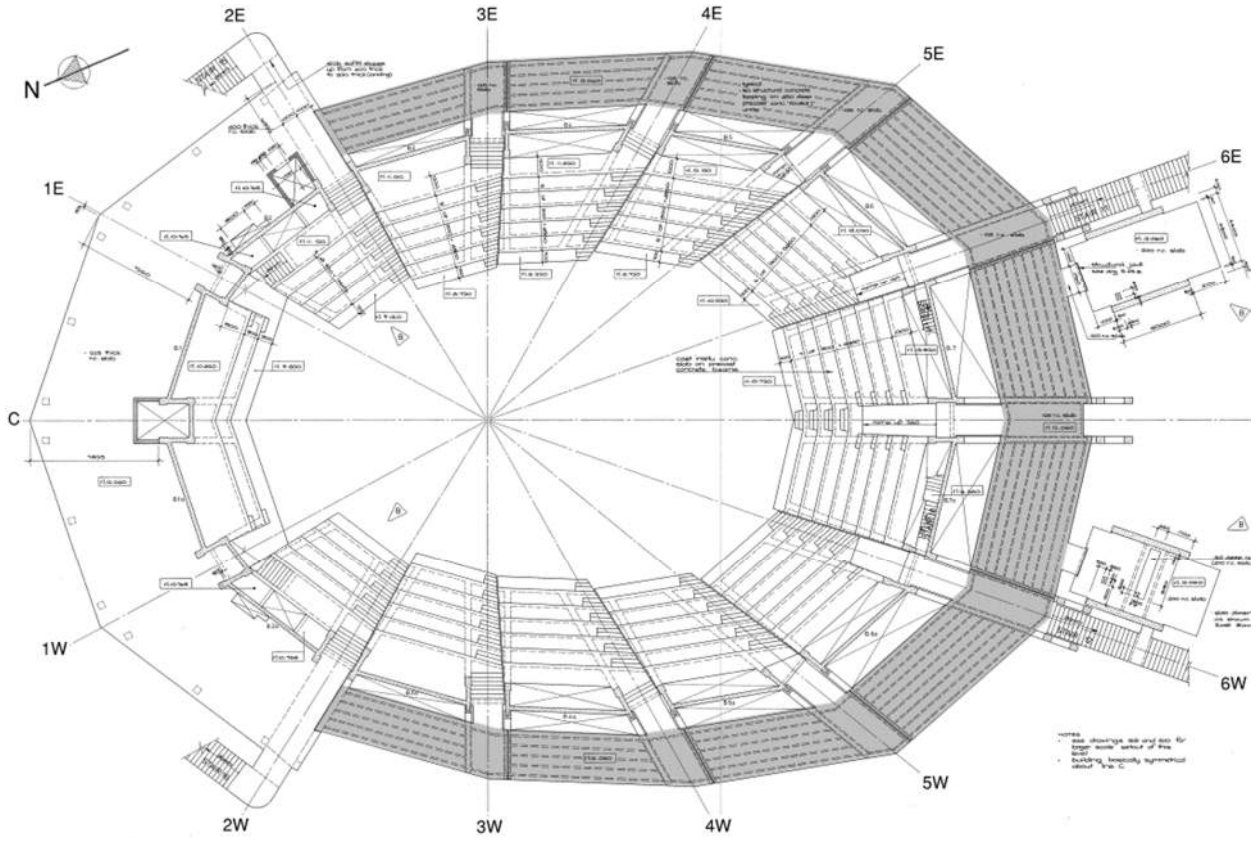

Figure 8: Level 4 plan showing location of precast double-tee units (grey highlight).

Unit 3 is located between GL 2E and 3E and is flange-supported on one end and web-supported on the other. This unit is identified Figure 9. The unit is orientated parallel to the north-south direction. There are no beams running adjacent to the unit and as a result frame elongation was not considered for a loss of support failure. For the flange-supported end of the unit, the limiting drift calculation considered rotation of the support beam, construction tolerance, spalling, creep, and bearing failure. This limiting drift was found to be 1.7% for a loss of support condition. Assessing the other end of the unit as a web-supported condition gave a limiting drift of 2.0%. As units on GL 2E and 2W are flange-supported, they will be susceptible to a "bird's mouth" failure on that end as discussed in Section 7.1.

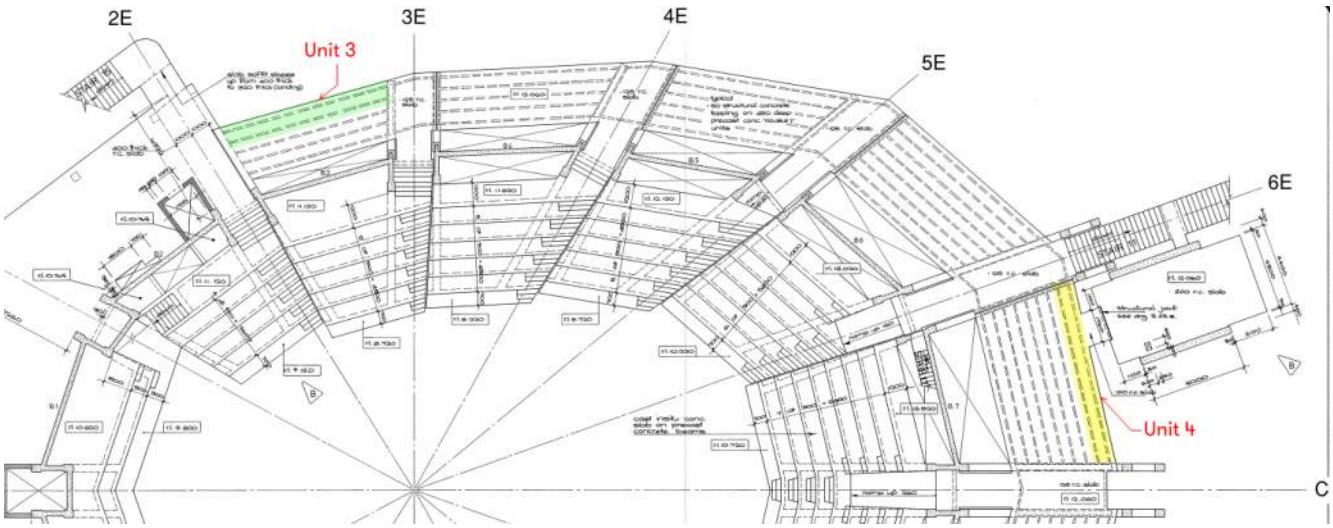

Figure 9: Level 4 part plan showing location of typical units.

Another double-tee unit was selected as a typical web-supported situation. Unit 4 is the longest unit located between GL 6E and GL C and is web-supported on either end. This unit is identified in Figure 9. The unit was selected due to its length and orientation parallel to the east-west direction which makes it most susceptible to a loss of support failure condition. Assessing the double-tee unit in accordance with the C5 (2018 update) guidelines included consideration of rotation of support beams, construction tolerance, spalling, creep, and bearing failure. As there are no adjacent beams, frame elongation has not been considered. Using the outlined procedure, the limiting drift for Unit 4 was calculated as 2.5%.

7.2 Level 2 Precast Seating Units

Part of the interior of the Auditorium at Level 2 is framed using precast seating units, which are identified in Figure 10. These are located between GL 5E and 5W are symmetrical about GL C and are supported on in situ concrete beams and precast piers. The connection for a precast seating unit seated on a pier thickening is most critical due to minimal seating and reinforcing. Of the pier-seated units, longer units are more critical for a loss of support condition due to the increased span and corresponding increase in required bearing length. A typical unit seated on a pier thickening was selected for assessment (precast seating unit PF.35 as notated in the original drawings).

Holding down bolts are cast in situ to the concrete beams or piers and are anchored through the flange of the precast seating units. These details are shown in Figure 11. A u-bar restrains the holding down bolt in both the concrete ledge and the “flange” of the precast seating unit. To assess the connection, the capacity of the bolt, capacity of the u-bar reinforcing and anchorage of the bolt were checked.

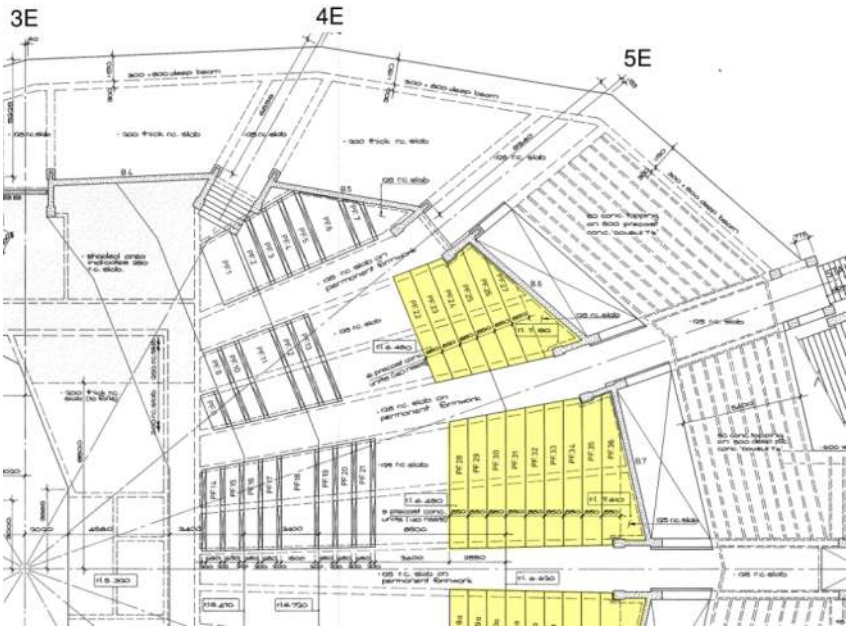

Figure 10: Level 2 part plan showing location of precast seating units (mirrored around grid C).

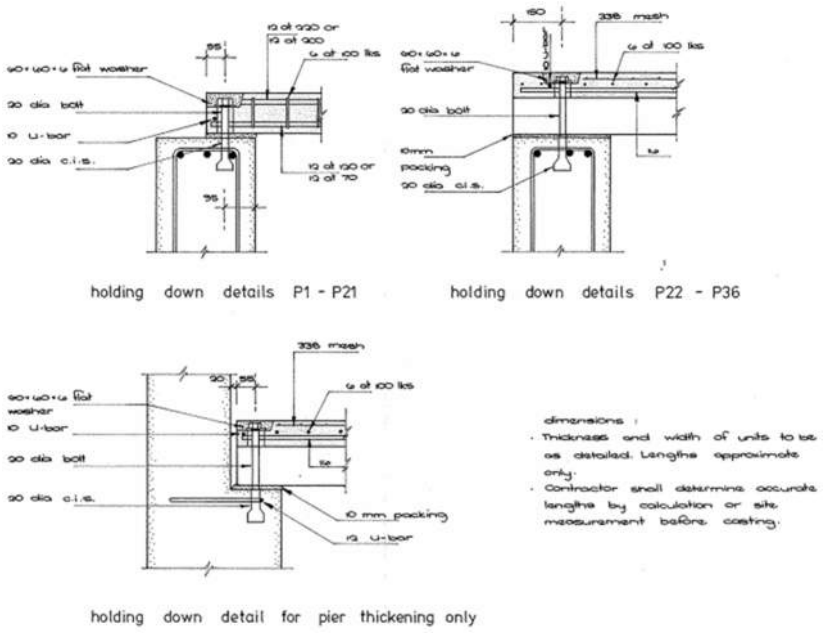

Figure 11: Holding down bolt details for precast seating units.

The u-bar was found to have adequate capacity preventing any concrete breakout failures of the holding down bolt. The bolt is expected to yield in bending with little or no damage to the cast in section. Following bolt yielding, the precast seating unit will not be rigidly fixed and a loss of support failure must be checked. Assessing the precast seating unit for loss of support in line with the C5 guidelines (2018 update) included losses due to rotation of support beams, construction tolerance, spalling, creep, and bearing failure. As no beams sit adjacent to the precast seating units frame elongation was not been considered. The limiting drift

for a loss of support condition for the precast seating units was calculated as sufficiently large drift to not require remediation.

7.3 Summary of Limiting Drifts

Based on the assessment of the precast flooring units as per the guidance outlined in the NZSEE Guidelines section C5 2018 guidelines update (5), the limiting drifts for each unit were calculated and are summarised in Table 3. Expected building drifts were derived from the 2014 detailed seismic assessment (1) and used to determine a likely earthquake score for precast flooring elements. Please note that we are using the drifts derived from this analysis as it is convenient and representative of the expected building performance. However, this analysis has not been updated to the latest NZSEE Guidelines, which changed in 2017 and 2018. This report is not a DSA and the scores listed below are indicative of likely scores for these units should a DSA be performed.

The expected drifts reported in the 2014 detailed seismic assessment in are presented in terms of the building primary X and Z axes. The X and Z axes correspond to the North-South and East-West directions of the building, respectively. At different levels of $\%NBS_{IL3}$ the expected drifts are shown in Table 1.

Table 1. Seismic Building Drifts at Varying %NBS

	% NBS_{IL3}			
Direction	67%NBS_{IL3}	90%NBS_{IL3}	100%NBS_{IL3}	125%NBS_{IL3}
X (North-South)	0.36%	0.58%	0.68%	1.92%
Z (East-West)	0.32%	1.01%	1.07%	1.91%

The NZSEE Guidelines section C5 (2018 update) specify that when assessing brittle failure modes of precast floors, such as loss of seating failure or "bird's mouth" failure, the expected displacement demands should be increased by a factor of 2. Therefore, the factored drifts for comparison to the calculated limited drifts are:

Table 2. Factored Seismic Building Drifts at Varying %NBS

	% NBS_{IL3}			
Direction	67%NBS_{IL3}	90%NBS_{IL3}	100%NBS_{IL3}	125%NBS_{IL3}
X (North-South)	0.72%	1.06%	1.36%	3.84%
Z (East-West)	0.64%	2.02%	2.14%	3.82%

Below is a table which correlates the calculated limiting drifts with the corresponding %NBS.

Table 3: Summary of limiting drifts for a loss of support condition.

Level and Location	Unit (direction)	Limiting Drift	Expected Earthquake Score	Comment
Level 2 Promenade	**Unit 1 (X)**			
	GL 5E – flange-supported	1.6%	>100%NBS_{IL3}*	*This score is based on loss of seating failure **only** and a score corresponding to "bird's mouth" failure has not been assessed. However, support is needed in anticipation of this failure mode.
	GL 6E – web-supported	1.8%	>100%NBS_{IL3}	
Level 2 Promenade	**Unit 1a (Z)**			
	GL 6E/C – web-supported	1.9%	67-90% NBS_{IL3}	
Level 2 Entry Foyer	**Unit 2 (Z)**			
	GL C – pier web-supported	2.1%	90-100% NBS_{IL3}	
Level 2 Auditorium	**Precast seating unit P35 (Z)**			
	CL 6E – pier web-supported	sufficiently large	>100% NBS_{IL3}	
Level 4 Promenade	**Unit 3 (X)**			
	GL 2E – flange-supported	1.7%	>100%NBS_{IL3}*	*This score is based on seating failure **only** and score corresponding to "bird's mouth" failure has not been assessed. However, support is needed in anticipation of this failure mode.
	GL 3E – web-supported	2.0%	>100% NBS_{IL3}	
Level 4 Promenade	**Unit 4 (Z)**			
	GL 6E/C – web-supported	2.5%	>100%	

8 CONCLUSIONS AND RECOMMENDATIONS

We have made an assessment of the precast flooring units in the Auditorium building of the MFC considering both loss of support and "bird's mouth" failures for precast double-tee units. Of these, only the loss of support failure was quantified and assessed. It was found that none of the units have a limiting drift less than that corresponding to 67%NBS_{IL3} for a loss of support failure. However, in certain locations identified on the plans in Appendix A: Schematic Details, supplementary support brackets are prudent to mitigate the risk posed by "bird's mouth" failure or unseating of the precast double-tee units.

As noted in previous strategy guidance provided by Holmes (6), if WCC were to commission a Detailed Seismic Assessment (DSA) in accordance with the 2017 NZSEE Guidelines (4) before the remaining areas addressed in the 2014 seismic assessment (1) are addressed, we believe seismic rating could be an undesirably low rating. We suggest that any update of the DSA should wait until after the remaining areas identified in the 2014 seismic assessment (1) are addressed, as well as the improvements noted in this report and other seismic upgrade work that has already been designed and detailed. Consideration should also be given in the DSA to the implications of potential site geotechnical performance, as identified in the 2014 seismic assessment.

As a result of the indicative-level assessment conducted and discussed in this report, we believe there are two main workstreams that should be performed, in priority order:

1. Addition of supplementary support seating brackets to provide secondary support to flange-supported double-tee units. This is restricted to the double-tee units on gridlines GL 5E/5W on Level 2 and GL 2E/2W on Level 4. The addition of these supplementary support brackets is similar to the details designed for the support of double-tee units in the Renouf Foyer. These brackets would reduce the risk posed by a "bird's mouth" failure of flange-supported units in an earthquake.

2. Supplementary support seating angles to be provide additional seating to 350-deep double-tee units either side of the Entry Foyer lift shaft on GL C. The addition of these supplementary support brackets is also similar to details designed for the support of double-tee units in the Renouf Foyer. These brackets would reduce the risk posed by a loss of support failure of the units in an earthquake.

Schematic details of supplementary supports can be found in Appendix A: Schematic Details.

9 REFERENCES

1. **Holmes Consulting Group.** *Michael Fowler Centre Detailed Seismic Assessment Report (Project Reference 107303).* 2014.

2. —. *Michael Fowler Centre - Stair Strengthening (Project Reference 107303).* 2015.

3. —. *Michael Fowler Centre Qualitative Seismic Review (Project Reference 107303).* 2011.

4. **New Zealand Society for Earthquake Engineering (NZSEE).** *The Seismic Assessment of Existing Buildings.* 2017.

5. —. *The Seismic Assessment of Existing Buildings: Technical Guidelines for Engineering Assessments, C5 Update.* 2018.

6. **Holmes Consulting Group.** *Michael Fowler Centre - Seismic Strategy: Auditorium Roof.* 2018.

APPENDIX A: SCHEMATIC DETAILS

Additional supports necessary

Additional supports recommended

NOTE:
shaded area indicates that the slab at rl. 5.320 runs under the slab at rl. 6.420, forming plenum space for return air.

NOTE:
precast conc. floor units, symmetrical about line C
- see details of units on drg. 380

NOTE
- 125 slabs between beams to be cast late in job to minimise shrinkage effects (see Specification)

Warren & Mahoney
Registered Architects
Christchurch

Holmes Wood Poole & Johnstone
Consulting Civil & Structural Engineers
Christchurch & Wellington

WELLINGTON TOWN HALL

LEVEL 2 FLOOR PLAN

Scale	1 : 100
Drawn	R. I. S.
Traced	S. L. S.
Approved	

PROJECT: MFC - Auditorium
JOB NO: 137071.01 DATE: 18/10/2019
DRAWN BY: SSW SCALE: NTS
SSK No: 12_01 REV: 0
SSK Name: Level 2 Floor Plan

Additional supports necessary

slab soffit slopes up from 400 thick to 920 thick (landing)

400 thick r.c. slab

· 125 r.c. slab

typical
· 50 structural concrete topping on 450 deep precast conc. 'double T' units

· 225 thick r.c. slab

cast in-situ conc. slab on precast concrete beams

· 200 r.c. slab

· structural joint
· see drg S.25.a.

· 100 r.c. slab

· 160 deep recess (200 r.c. slab under)

· slab dimensions as shown for East Block

NOTES
- see drawings 89 and 510 for larger scale setout of this level
- building basically symmetrical about line C.

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Holmes Wood Poole & Johnstone
Consulting Civil & Structural Engineers
Christchurch & Wellington

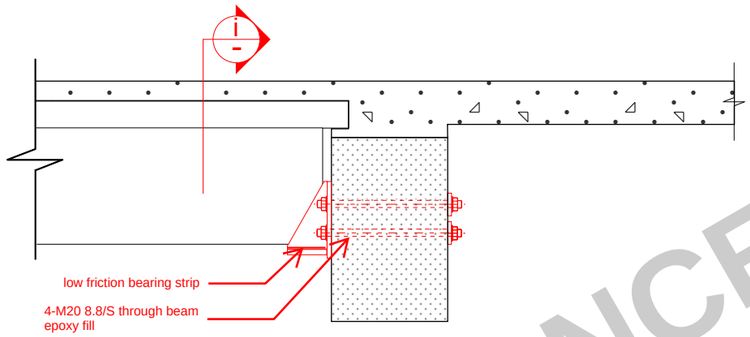
WELLINGTON TOWN HALL

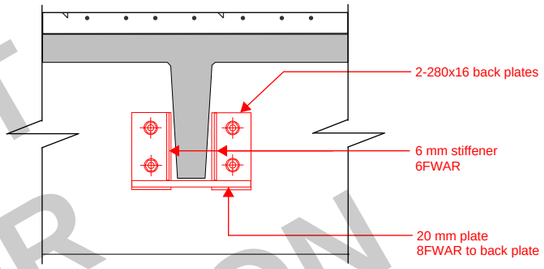
LEVEL 4 FLOOR PLAN

Scale	1 : 100
Drawn	R. I. S
Traced	S. L. S.
Approved	

Holmes

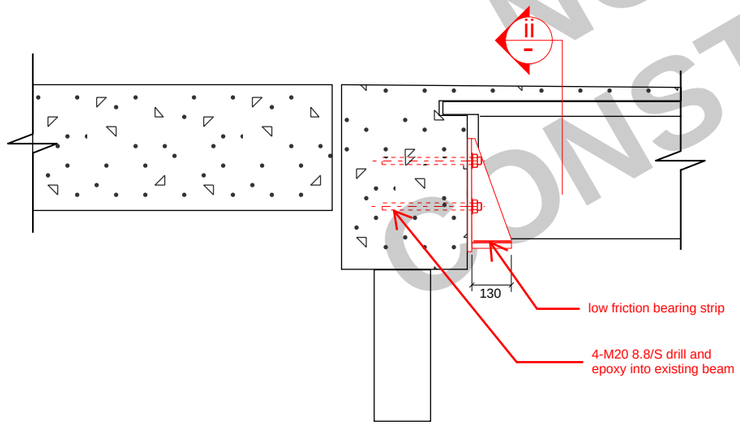
PROJECT: MFC - Auditorium
JOB NO: 137071.01 DATE: 18/10/2019
DRAWN BY: SSW SCALE: NTS
SSK No: 12_02 REV: 0
SSK Name: Level 4 Floor Plan

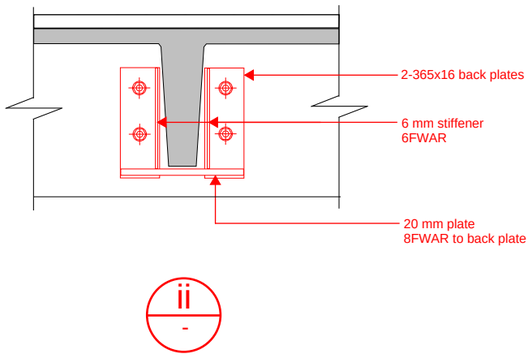




A
-
typical Level 2 flange-hung unit support

i
-





B
-
typical Level 4 flange-hung unit support

ii
-

CONCEPT
NOT FOR
CONSTRUCTION

Holmes

PROJECT: MFC - Auditorium
JOB NO: 137071.01 DATE: 18/10/2019
DRAWN BY: SSW SCALE: NTS
SSK No: 12_03 REV: 0
SSK Name: Supplementary steel support details

Appendix J

Visual Observations

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Appendix J Visual Observations

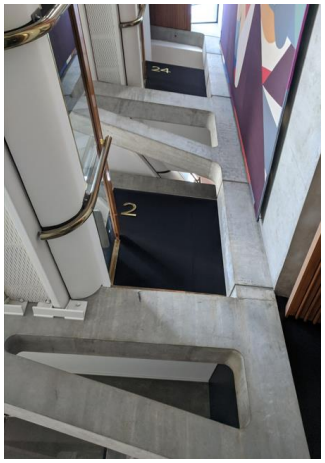
J.1 INTRODUCTION

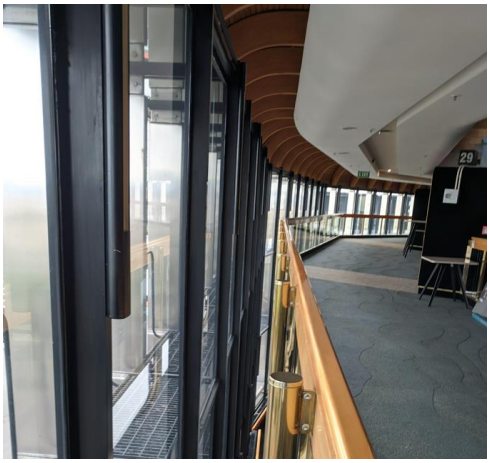
Site visits to the Michael Fowler for Stage A were undertaken on four occasions as noted in Section 6.2.

This appendix highlights some of the main areas observed during these visits.

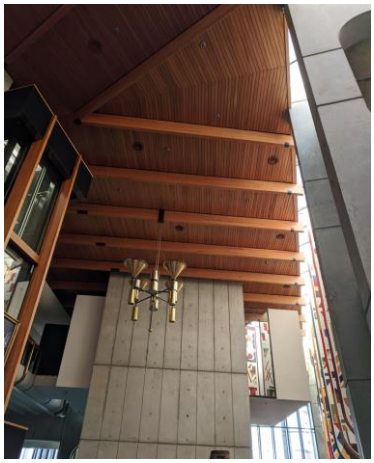
In addition to recent site visit, during carpet replacement works in December 2021, various cracks were observed in the floors. A few pictures of those conditions are also provided.

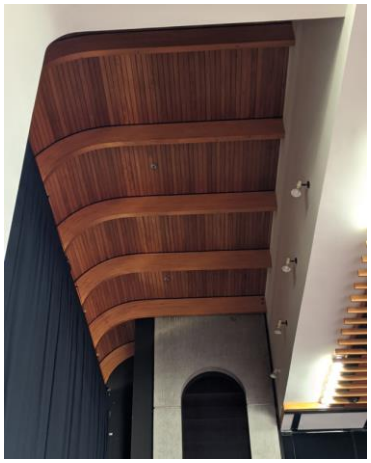
Structure	Location	Observations
Auditorium	Promenade Levels	The Precast Cantilever Beams are hung off the exterior of the buttress walls to create the perimeter Promenades. The façade flies past the Level 4 promenade, with glazing continuous from Level 2 to the timber roof.



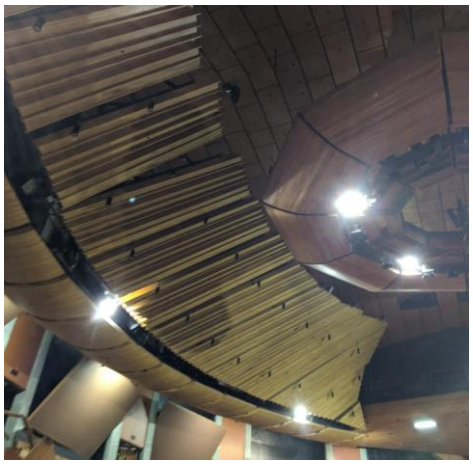


Structure	Location	Observations
Renouf Foyer	Entry way and roof	Glulam beams are continuous over the entry way and the Renouf Foyer. At the Renouf Foyer these beam bend 90 degrees to become the façade columns.





Auditorium	Interior stage	There are two levels of seating in the Auditorium, both the lower and upper angle downwards towards the stage. Hanging from the roof trusses are acoustic reflectors, catwalks, and stage lighting supports.



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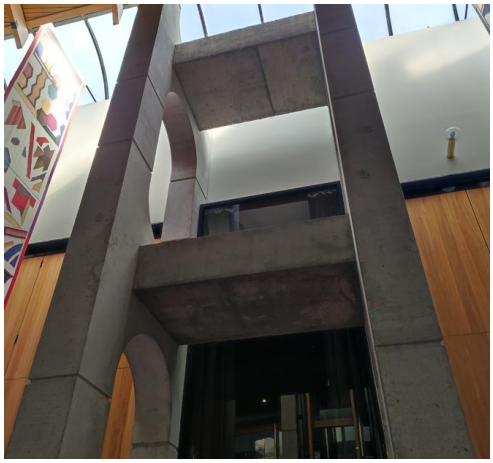
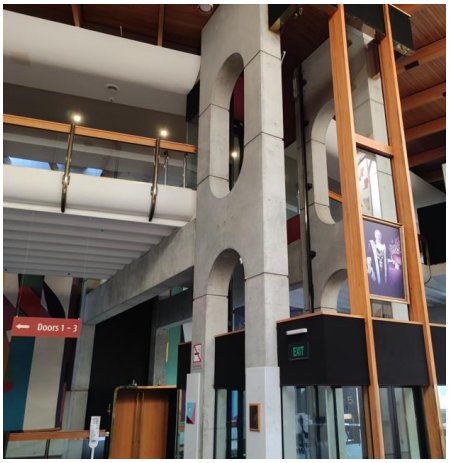
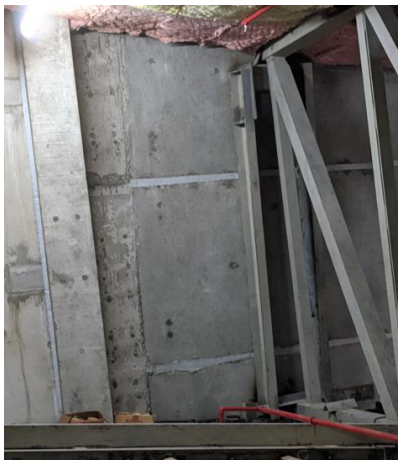
Structure	Location	Observations
Auditorium and Renouf Foyer	Entryway	Parallel pairs of precast concrete frames are in the Auditorium entry way and Renouf Foyer. The frames in the Renouf Foyer have a shallow link beam in the transverse direction.

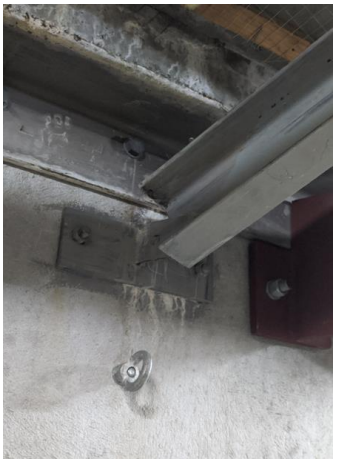




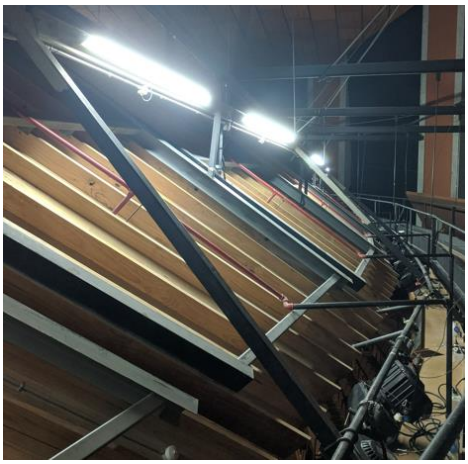
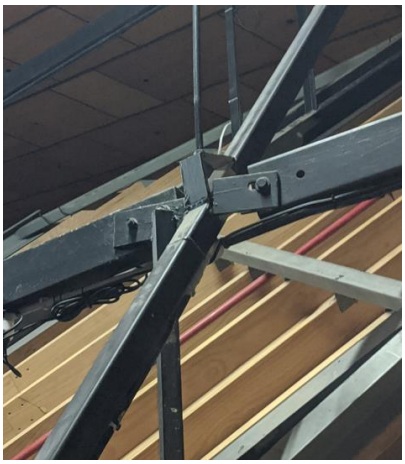
Auditorium	Roof space	Roof trusses span in the transverse direction with chord stiffening elements at the top and bottom chord. They attached to the concrete structure at the precast frames of the sloped precast panels. The connects vary by location and vary to the original structural documentation.



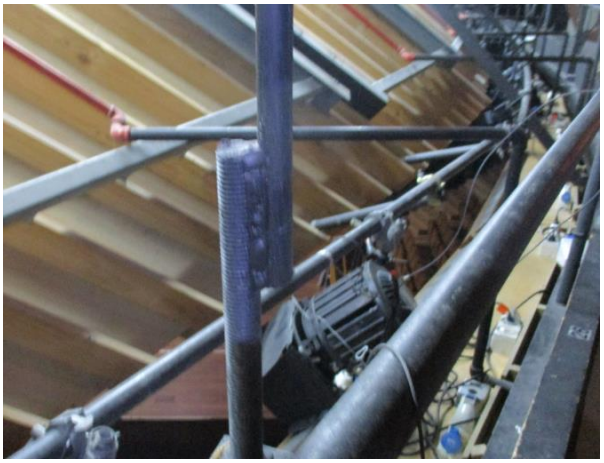
Structure	Location	Observations
Auditorium	Roof space	The chord stiffening elements have a nominal bolted connection to the sloped precast panels. The top and bottom chord of the main trusses have a pinned connection at their midspan



Auditorium	Stage catwalks	The acoustic reflectors and catwalks are propped off the PBW systems and roof trusses. This steel frame structure varies greatly to the details documented on the original drawings. The geometry of the structure creates unique connection nodes

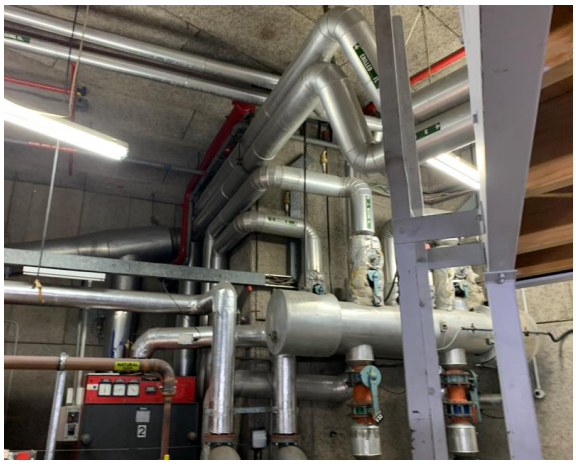
Structure	Location	Observations
Auditorium	Stage catwalks	Obvious site construction changes were made at the time of construction, including a hanging rod needing to be cut and weld spliced to itself. The catwalk structure is largely supported by cable bracing back to the precast walls.

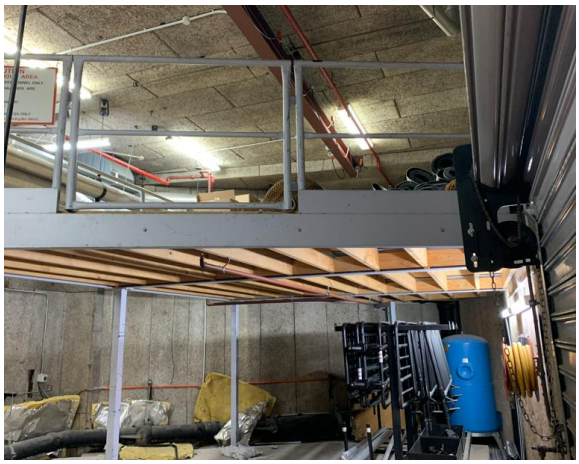


Auditorium	Roof space	One of the catch bracket remediations installed in 2020. The ceiling panels above the stage are clipped to the bottom to the roof trusses.

Structure	Location	Observations
Renouf Foyer	Ceiling space	Showing the plant in the ceiling space and typical connections of the roof trusses

Auditorium	Basement structure	Plant in the basement of the Auditorium





Structure	Location	Observations
Auditorium	Lobby area at Level 2	Cracks were observed at the top of slab – likely from shrinkage

Auditorium	Entry area	Cracks around the web of the double T panel elements were observed above the beams