WCC Housing Detailed Seismic Assessments

Berkeley Dallard – Detailed Seismic Assessment

Wellington City Council

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Executive Summary

Scope and Basis of Assessment

Aurecon have been engaged by the Wellington City Council (WCC) to provide a Detailed Seismic Assessment (DSA) for the building located at 46 Nairn Street, Mt Cook, Wellington, Wellington. The building is known as the **Berkeley Dallard Apartments.**

The DSA was generally completed in accordance *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (**Red Book**), including the *updated Section C5 – Concrete Buildings – Proposed Revision to the Engineering Assessment Guidelines*, dated November 2018 (the **Yellow Chapter**). These are collectively noted as the **Guidelines**.

The building is an **Importance Level 2 (IL2)** structure, located on a **Site Subsoil Class C** site as defined by NZS 1170.5:2004.

Beca Ltd (Beca) was engaged by Wellington City Council to carry out an independent peer review of this Detailed Seismic Assessment. A copy of their Peer Review letter can be found in Appendix I.

Results Summary

The seismic rating of a building is generally limited by the lowest scoring element; therefore, the buildings achieve an earthquake rating of **30%*NBS*(IL2)** for both North and South Buildings in accordance with the **Guidelines**. This rating is based on the out-of-plane flexural capacity of the reinforced concrete shear walls and roof and level 10 diaphragm connections to the shear walls as Critical Structural Weaknesses (**CSWs**). The buildings also contain other distinct elements that are classified as structural weaknesses (**SW**) summarized in the Tables below.

A **SW** is an aspect of the building structure and/or the foundation soils that scores less than 100*%NBS* and a **CSW** is the lowest scoring structural weakness.

The building lateral resisting is provided by shear walls in both directions. Considering the height of the buildings, due to the existence of multiple number of lateral load resisting elements in both directions their seismic responses are stiff. It is worth noting that these buildings are classified as torsionally irregular as per NZS1170.5:2004.

The Tables below present a summary of the results for the North and South Buildings based on the **Guidelines.**

Table 1, Summary of the North Building elements' %NBS scores

Element:	%NBS(IL2):	Commentary:
RC Shear Walls – Singly Reinforced In-Plane Loading -Longitudinal -Transverse	 >34% >34%	▪ Certain number of RC piers and spandrels have insufficient flexural capacity to resist the expected seismic demands and limits the rating to >34%NBS(IL2) in the longitudinal and transverse directions, respectively (Refer to Appendix H). ▪ The single layer of reinforcement within the walls limits the probable flexural capacity of the 6" walls at the upper levels of the structures.

RC Shear Walls – Doubly Reinforced In-Plane Loading -Longitudinal - Transverse	 >34% >67%	▪ In the longitudinal direction certain number of RC piers and spandrels have insufficient flexural capacity as a result of the flexure – axial interactions to resist the expected seismic demands >34%NBS at IL2 in the longitudinal and transverse directions respectively (Refer to Appendix H). ▪ The irregular openings in walls and spandrel elements result in walls acting as a coupled and inducing large tensile forces in the piers. Tension forces greatly reduce the probable flexural capacity of the shear wall elements. ▪ In the transverse direction the capacity is limited by the shear capacity of the squat walls at the lower levels along gridline 1. ▪
Concrete Diaphragms	55%	▪ The level one, two and eight floor diaphragms are limiting the score to the 55%NBS(IL2). Level nine is scoring 60%NBS(IL2), and the rest of the floors are scoring 65%NBS(IL2) ▪ The concrete diaphragm, reinforced with deformed bars, have insufficient capacity to transfer the diaphragm inertia and transfer loads to the vertical elements of the lateral resisting system. ▪ We note the score is based on specific areas of the floor and not the whole floor. ▪ Intrusive site investigation may help to improve this rating.
Foundations: - Longitudinal - Transverse	 60% 85%	▪ The bored piles lateral capacity in combination with the passive pressure resistance of the ground beams are insufficient to resist 67%NBS of an IL2 event in the longitudinal and 100% in the transverse directions.
Stairs	100%	▪ The stairs are concrete cast in-situ. The connections of the stairs to the landings are fixed with no allowance for sliding or lateral movement of the building. The flights are cast into the face of the landings with deformed bars. Though stairs have a seismic capacity of 100%NBS.
Walls Out-of-Plane	30%	▪ The RC walls above Level 9 are cantilevering to support the roof system. Rating of the new diaphragm added at level 10 affect this. This cantilever is as high as 5.5m. It is not expected that the roof system can provide restraint of the walls for out-of-plane loading. The walls score 30%*NBS*(IL2) for out-of-plane seismic parts loading.
Roof and timber diaphragm at Level 10	<34%	▪ The steel roof is a flexible diaphragm and does not contain steel-cross braces or a plywood diaphragm. Therefore, the DHS purlins must transfer seismic load from the roof to the RC walls by bending out-of-plane. ▪ The timber diaphragm located at level 10 is connected to concrete walls, however, the connections do not possess sufficient capacity to resist the ultimate limit state (ULS) seismic actions. The diaphragm has a seismic capacity score of <34%NBS (IL2). The capacity of connections between the timber floor and the concrete walls using T16 Trubolts with unknown embedment depthlimits the rating.
Water tank at roof level	50%	▪ The reinforced concrete water tank located at the roof level has achieved a seismic performance score of 50%NBS (IL2), based on its walls' out-of-plane capacity when subjected to seismic loading. Considering the location of the water tank and the identified mode of failure, we do not consider it as a significant life safety concern.

Table 2, Summary of the South Building elements' %NBS scores

Element:	%NBS(IL2):	Commentary:
RC Shear Walls – Singly Reinforced In-Plane Loading -Longitudinal -Transverse	 >34% >34%	▪ Certain number of RC piers and spandrels have insufficient flexural capacity to resist the expected seismic demands for 25%NBS and 30%NBS at IL2 in the longitudinal and transverse directions of the south building respectively (Refer to Appendix H). ▪ The single layer of reinforcement within the walls limits the probable flexural capacity of the 6" walls at the upper levels of the structures.
RC Shear Walls – Doubly Reinforced In-Plane Loading -Longitudinal -Transverse	 >34% 100%	▪ In the longitudinal direction, certain number of RC piers and spandrels have insufficient flexural capacity as a result of the flexure – axial interactions to resist the expected seismic demands for 25%NBS and 40%NBS at IL2 in the longitudinal and transverse directions of the South building respectively (Refer to Appendix H). ▪ The irregular opening in walls and spandrel elements result in walls which act as a coupled shear wall, inducing large tensile forces in the piers. Tension forces greatly reduce the probable flexural capacity of the shear wall elements.
Concrete Diaphragms	60%	▪ The level eight floor diaphragm for the South building is limiting the score to the 60%NBS(IL2) for south building. Level nine is scoring 65%NBS(IL2) and the rest of the floors are scoring 67%NBS(IL2). ▪ The concrete diaphragm, reinforced with deformed bars, have insufficient capacity to transfer the diaphragm inertia and transfer loads to the vertical elements of the lateral resisting system. ▪ We note the score is based on specific areas of the floor and not the whole floor. ▪ Site investigation may improve this rating.
Foundations: - Longitudinal - Transverse	 65% 90%	▪ The bored piles and passive resistance provided by the ground beams do not have sufficient lateral capacity for 67%NBS and 100%NBS of an IL2 event in the longitudinal and transverse direction of the South building respectively.
Stairs	100%	▪ The stairs are concrete cast in-situ. The connections of the stairs to the landings are fixed with no allowance for sliding or lateral movement of the building. The flights are cast into the face of the landings with deformed bars. Though The stairs have a seismic capacity of 100%NBS.
Walls Out-of-Plane	30%	▪ The RC walls above Level 9 are cantilevering to support the roof system. Rating of the new diaphragm added at level 10 affect this. This cantilever is as high as 5.5m. It is not expected that the roof system can provide restraint of the walls for out-of-plane loading. The walls score 30%*NBS*(IL2) for out-of-plane seismic parts loading.

Roof and timber diaphragm	<34%	▪ The steel roof is a flexible diaphragm and does not contain steel-cross braces or a plywood diaphragm. Therefore, the DHS purlins must transfer seismic load from the roof to the RC walls by bending out-of-plane. ▪ The timber diaphragm located at level 10 is connected to concrete walls, however, the connections do not possess sufficient capacity to resist the ultimate limit state (ULS) seismic actions. The diaphragm has a seismic capacity score of <34%NBS (IL2). The connections between the timber floor and the concrete walls using T16 Trubolts with unknown embedment depth limits the rating.
Canopies	>67%	▪ The canopies have a seismic score of >67% ULS seismic actions. The failure is due to concrete cone failure and splitting failure.
Water tank at roof level	50%	▪ The reinforced concrete water tank located at the roof level has achieved a seismic performance score of 50%NBS (IL2), based on its walls' out-of-plane capacity when subjected to seismic loading. Considering the location of the water tank and the identified mode of failure, we do not consider it as life safety concern.

We note that the non-structural building elements (ceilings, lightweight partition walls, overhead services were not part of the scope of this DSA. However, a desktop study of the available documentation did not identify any large plant, ceilings, and partitions that would raise concern.

Further Investigations

We recommend that further investigation be carried out to the following elements to provide a more accurate seismic score:

- Investigate the connections of the timber diaphragms at Level 10 to the RC shear walls and the connections at the roof level. The assessment to date has based the score on an assumed connection detail. Further clarity of the connection arrangement is recommended to either confirm the as-built connections and provide a more accurate %*NBS* score.
- To improve the current rating of the diaphragms, Aurecon recommends measuring the as-built lap length of the bottom reinforcement layers by chipping away the bottom concrete cover at both sides of the walls.

Recommendations

We recommend that the building is seismically strengthened considering a two-stage approach. Stage 1 would be to strengthen the building to a minimum seismic rating of greater than **34*%NBS*(IL2).** Based on our review, the seismic strengthening, to achieve greater than 34*%NBS*(IL2), would include:

- Singly reinforced shear walls in out-of-plane directions
- Connections between the timber diaphragm at L10 and roof level to the lateral resisting system.
- Improving the roof diaphragm action

We recommend strengthening the above elements to minimum 67%NBS(IL2) if the final objective is to achieve this rating.

Stage 2 would be to seismically strengthen the building to a minimum rating of 67*%NBS* (IL2). Based on our review, the seismic strengthening to achieve 67%*NBS*(IL2) would include, but not be limited to:

- Singly and doubly reinforced RC shear walls in the in-plane direction
- Water tank
- Foundation in the longitudinal direction
- Diaphragms

As noted earlier, further investigation may reduce the scope of the strengthening required to achieve 67%NBS(IL2). For further explanation refer to **Strengthening** section.

We also recommend that as part of any seismic upgrade or future fitout , the non-structural building elements (ceilings, internal walls, overhead services and plant and equipment etc) is seismically restrained to meet the current standards. It should be noted that no large plant was identified in the building that would need seismic support. No ceilings, partitions were identified while studying the existing documentation that would raise concern.

We further recommend that in designing any seismic retrofit that the building owner should also consider the proposed increase in seismic hazard levels in Wellington. This would insulate the building against further future reductions in the seismic rating.

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1 Introduction

1.1 Background

Aurecon have been engaged by the Wellington City Council (WCC) to provide a Detailed Seismic Assessment (DSA) the Berkeley Dallard buildings located at 46 Nairn Street, Mt Cook, Wellington. The buildings that have been assessed are the South and North Buildings. Refer to **Figure** 1-1 for the site's location and layout. **Figure** 1-2 shows general view of the building.

The DSA focuses on life safety issues as the primary objective. This means that the earthquake scores or rating is based primarily on life safety considerations rather than damage to the building or its contents unless this might lead to damage to adjacent property. The earthquake rating assigned is, therefore, not reflective of serviceability performance.

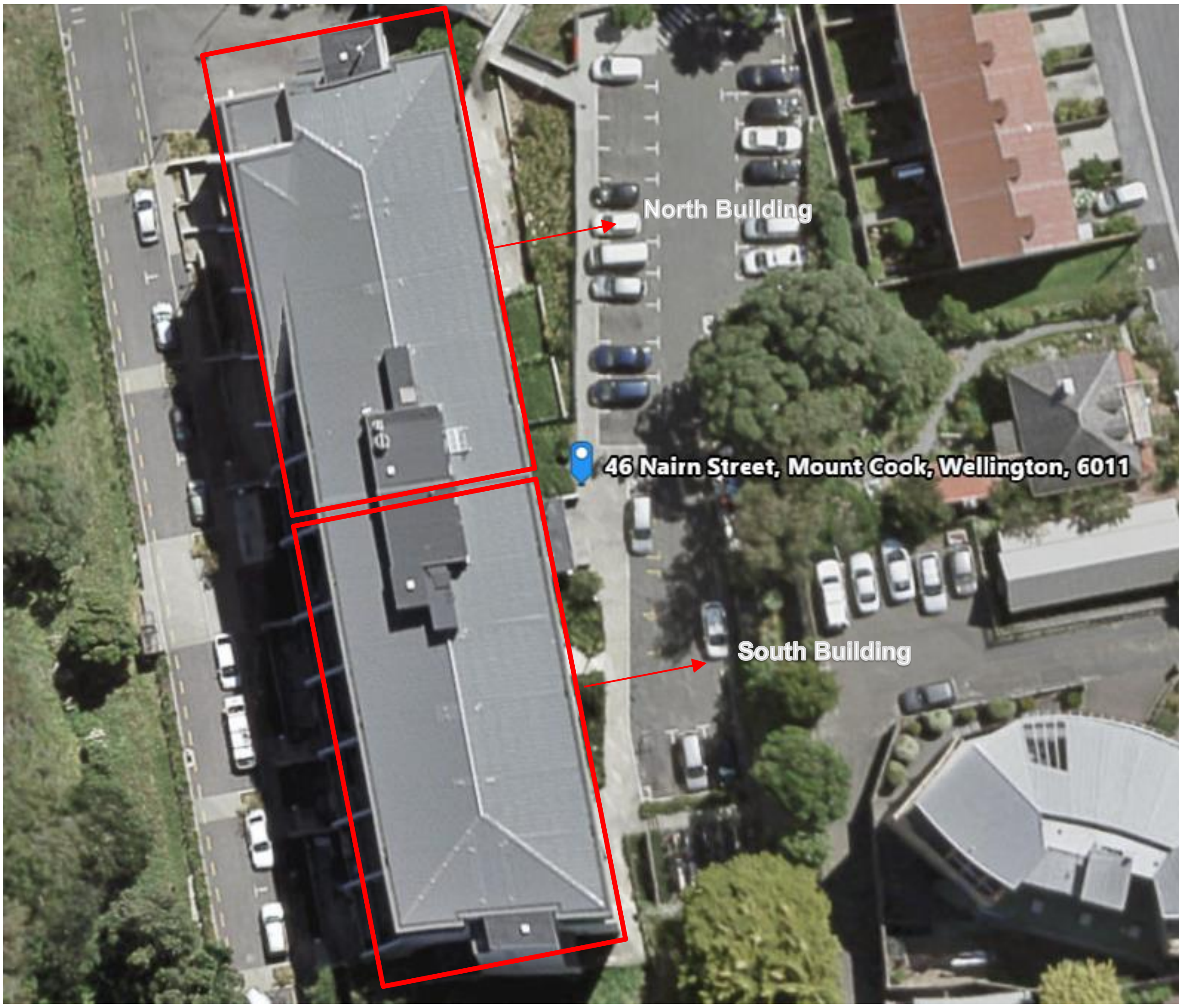

Figure 1-1 Site Layout (Source: ArcGIS)

Figure 1-2 Buildings General View

1.2 Terminology and Key Definitions

See below for key terminology and key definitions as defined by the **Guidelines**. Refer to **Appendix A** for additional definitions.

- ***%NBS* (New Building Standard)*:*** 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. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
- **Design level/ULS earthquake:** Design level earthquake or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004
- **Ductile/ductility**: Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake.
- **Structural weakness (SW):** An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS*.
- **Critical structural weakness (CSW):** The lowest scoring structural weakness determined from a DSA.

1.3 Building Description

The Berkeley Dallard Building is a residential building designed in 1974. It is a reinforced concrete shear wall building with cast-in-situ reinforced concrete floors. The building is seismically separated in the middle of its length to the South and North Buildings. As the buildings are located at the escarpment of the hill, their number of stories varies.

The North Building steps down the hill by two levels in both transverse and longitudinal directions. it has a maximum height of 30m with 11 levels at the North-West corner end and reduces to 7 levels toward the seismic gap at the South-East corner. The South building has 9 levels and steps down by two levels in the transverse direction and has constant levels longitudinally. The building has 9 stories on the West and 7 stories on the East.

The North building is 37.8m long and has the maximum width of 20m at level 3 reducing to 16m above level 6. The South building is 36m long and has the maximum width of 17.3m at level 5 reducing to 16m above level 6.

Each floor houses several individual apartments with an access gallery in the middle running in the North-South direction. Stairs are also provided at the North and South ends of the building complex.

The lateral load resisting system consists of shear walls in both directions generally separating the apartments in the transverse direction. In the longitudinal direction the main two shear walls are located at each side of the passage in the centre and running full height and length of the building.

Thickness of the original shear walls vary between 225mm to 150mm. Generally, walls below level 6 are 200mm and reduce to 175 and then 150mm toward the top of the building. Walls with the thickness more than 150mm are reinforced with 2 layers of reinforcement both ways, both faces. Walls 150mm thick are reinforced with 1 layer of reinforcement in the middle. The dominant type of reinforcement is 10mm mild steel for walls less than 225mm thick and 16mm mild steel for 225mm thick walls.

The floors below Level 10 are 5" (125mm) thick reinforced concrete cast-in-situ flat slabs spanning between the shear walls. **Figure** 1-3 shows the floor plans for Levels 6 to 8.

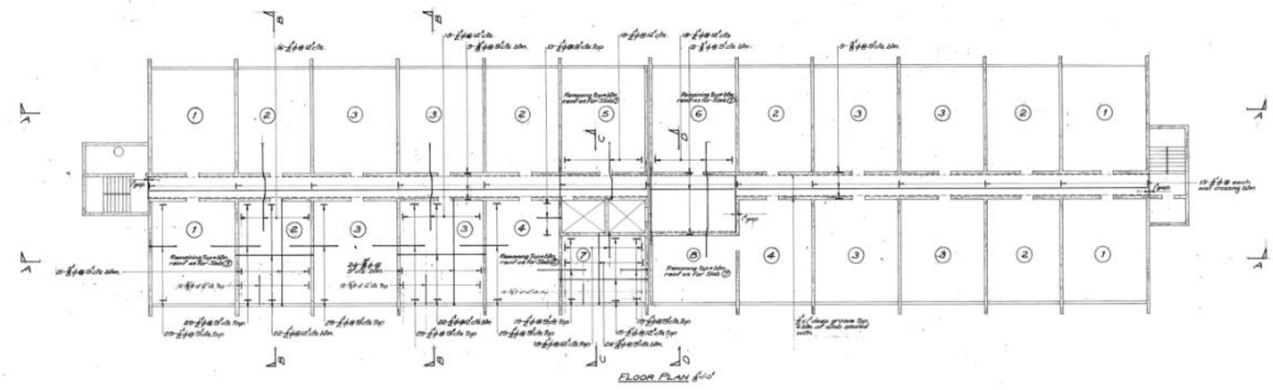

Figure 1-3. Floor plan for L6 to L8

The buildings are supported on a series of stepped ground beams in both the longitudinal and transverse directions supporting the shear walls. The ground beams are supported by the series of bored piles. Further information about the details of the foundation is provided later in this report.

1.4 Previous Assessments

In 2008, Holmes Consulting Group (HCG) issued a report titled "*Berkeley Dallard Seismic Assessment*." The report indicated that the North building achieved a seismic rating of **30-35%*NBS*(IL2)** in the longitudinal direction and **20-25%NBS(IL2)** in the transverse direction. The South building had similar rating in the longitudinal direction and **25-30%NBS(IL2)** in the transverse direction. The assessment was in accordance with the then current 2006 *NZSEE Assessment Guidelines.* Following that in 2009-13 Aurecon has been engaged to do a seismic assessment and strengthening of the building. The DSA rating was similar to the HCG and limited by the capacity of the lintels acting as coupling beams.

Due to the date of the assessment, the assessment was not completed in accordance with *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (commonly known as the "**Red Book**").

Today the Red Book provides mandatory technical guidelines for engineers to use when carrying out seismic assessments of potential earthquake-prone buildings when required by the Territorial Authority. They should also be used by engineers for all seismic assessments.

In 2018, a proposed technical revision to *Section C5 of the Engineering Assessment Guidelines* (referred to as the "**Yellow Chapter**") was released by the engineering sector to provide the latest engineering knowledge on aspects involved in the assessment of concrete buildings, and to reflect what engineers learned from investigations following the Kaikoura earthquake.

1.5 Alterations and Strengthening

The building was subject to extensive refurbishment in 2013 as part of the wider WCC Housing Upgrade project. As part of this refurbishment a number of new doorways are added through the transverse and longitudinal walls. At the same time a number of existing doorways are infilled with concrete.

The roof and the Level 10 of the building are reconfigured. A new lightweight timber floor at level 10 of the buildings and a new lightweight steel roof are added to the building. The new floor and roof are supported on the existing reinforced concrete walls and steel posts extended up from Level 9. A new façade also constructed of lightweight steelwork at the upper-level glazing line to enhance the appearance of the building.

As part of the strengthening, a 125mm thick reinforced concrete skin is added to the transverse walls adjacent to the locations where new openings are made.

In the longitudinal direction, the existing doorway lintels are cut. This helps to have better prediction of the walls' performance. A 250mm thick reinforced concrete skin is also added around the lift core areas of the buildings at either side of the seismic gap.

1.6 Basis of Assessment

1.6.1 General

The DSA was generally completed in accordance with *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (**Red Book**), including the *updated Section C5 – Concrete Buildings – Proposed Revision to the Engineering Assessment Guidelines*, dated November 2018 (the **Yellow Chapter**). These are collectively noted as the **Guidelines**.

1.6.2 Importance Level

The building has been assessed as an **Importance Level 2** (IL2) building and a design life of 50 years, in accordance with the New Zealand Building Code. A return period factor 'R' of 1.0 has therefore been used in accordance with NZS1170.5.

1.6.3 Site and subsoil class

Based on a 2009 geotechnical report, for Berkeley Dallard building the depth to very weak rock is generally greater than 3m and accordingly site subsoil is classified as **C**.

Geotechnical hazards such as liquefaction, landslide and lateral spreading are outside the scope of this assessment.

1.6.4 Hazard Zone Factor

The hazard zone factor Z determines the "seismic risk" area in accordance with NZS1170.5. There are different hazard zones factors depending on the buildings located in New Zealand. From NZS1170.5, we have used a hazard factor of **Z=0.40** for Wellington.

1.6.5 Scope

The assessment included undertaking the following:

- Retrieval and review of structural drawings, reports, calculations, and earlier models
- Conduct a walk through the building to establish that the building is generally in accordance with the plans (No intrusive investigations is allowed for)
- Create a detailed 3D ETABS model for each structure in accordance with the guidelines, based on the existing and strengthening structural drawings.
- Analyses of the superstructure with consideration of site subsoil class and flexibility of foundation and the nonlinearity in vertical behaviour of piles.
- Checking the walls, based on the analysis results and the detailing shown in the drawings.
- Assessment for the flat slab cast-in-situ diaphragms
- Assessment of the foundation including the ground beams and piles
- Review of the secondary elements including stairs, steel roof lightweight floor added at Level 10 and water retaining structure at the roof level.
- Formal in-house verification by CPEng engineer
- Produce and issue a report.
- Liaison and meetings as requested.

Elements that are excluded from consideration and analysis in this DSA include, but are not limited to:

- Non-structural building elements (façade elements, ceilings, internal lightweight walls, overhead services and plant and equipment), although please note our observations with regards to these.

2 Assessed Seismic Risk

The results of the DSA assess earthquake rating to be **30*%NBS*(IL2)** for both the North and South buildings, respectively in accordance with the **Guidelines**. This rating is based on the Critical Structural Weaknesses (**CSW**) as the in-plane flexural capacity of the shear walls and the capacity of the roof and level 10 diaphragm connections to the walls. The buildings also contain other distinct elements that are classified as structural weaknesses (elements that score less than 100*%NBS*). Details of %NBS scoring are provided in , Summary of the North Building elements' %NBS scores and Table 5-2, Summary of the South Building elements' %NBS scores.

Therefore, this is a **Grade D** building following the **Guidelines** grading scheme. This may classify the building as earthquake prone to the New Zealand Building Act, subject to the Territorial Authority. A Grade D building imposes a risk 10 to 25 times greater than a new building. Refer to **Table** 2-1 that shows the relative seismic risk compared to a new building.

Table 2-1 Relative seismic risk

Seismic Grade	*%NBS*(IL2)	Approx. risk relative to a similar new building	Relative life-safety risk description
A+	>100	<1	low risk
A	80 to 100	1 to 2 times	low risk
B	67 to 80	2 to 5 times	low to medium risk
C	33 to 67	5 to 10 times	medium risk
D	20 to 33	10 to 25 times	high risk
E	<20	more than 25 times	very high risk

A building with an earthquake rating less than 34%*NBS*, with the assessment undertaken utilising the Red Book, fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-Prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67%*NBS* is considered as an Earthquake Risk Building (ERB). The building is therefore categorised as an Earthquake-Risk Building and meets one of the criteria that could categorise it as an Earthquake Prone Building by Wellington City Council as the Territorial Authority. We note that our assessment used the **Yellow Chapter**. An assessment using the Red Book would likely result in similar scores to the **Yellow Chapter**.

3 Structural System Description

3.1 Primary Lateral Load Resisting System

3.1.1 Vertical Lateral Resisting Elements

Longitudinal Direction

The lateral resisting system in the longitudinal direction consists of in-situ RC piers and spandrels. Generally below level 6 these elements are 8” (200mm) thick and their thickness reduces to 7” (175mm) up to level 9. These elements are mainly reinforced with two layers of mild steel deformed 3/8” (9.5mm) diameter bars at 12” (305mm) spacing each way. Above level 9 thickness of the walls reduces to 6” (150mm). These walls are reinforced with single layer of mild steel deformed 3/8” (9.5mm) diameter bars at 8” (200mm) spacing each way. The walls do not have end thickenings, but larger diameter trimmer bars are provided around wall openings. These are typically two 3/4” (20mm) diameter bars.

In 2013 strengthening, a new 250mm thick wall is added along the lift cores of the building all the way up. These walls are reinforced with 2 layers of HD16 at 200mm spacing both ways. The wall is connected to the existing walls with HD12 hooked bars at 600 centres each way. During the strengthening certain number of openings in the longitudinal walls are infilled with concrete but new openings were also added to these walls. As part of the strengthening, the existing lintels in the walls in the North-South direction acting as spandrels were completely cut at both sides of the doorways up to the underside of the floor slabs. This converts certain number of walls to a series of cantilever walls. Figure 3-1 shows elevations of the main two longitudinal walls.

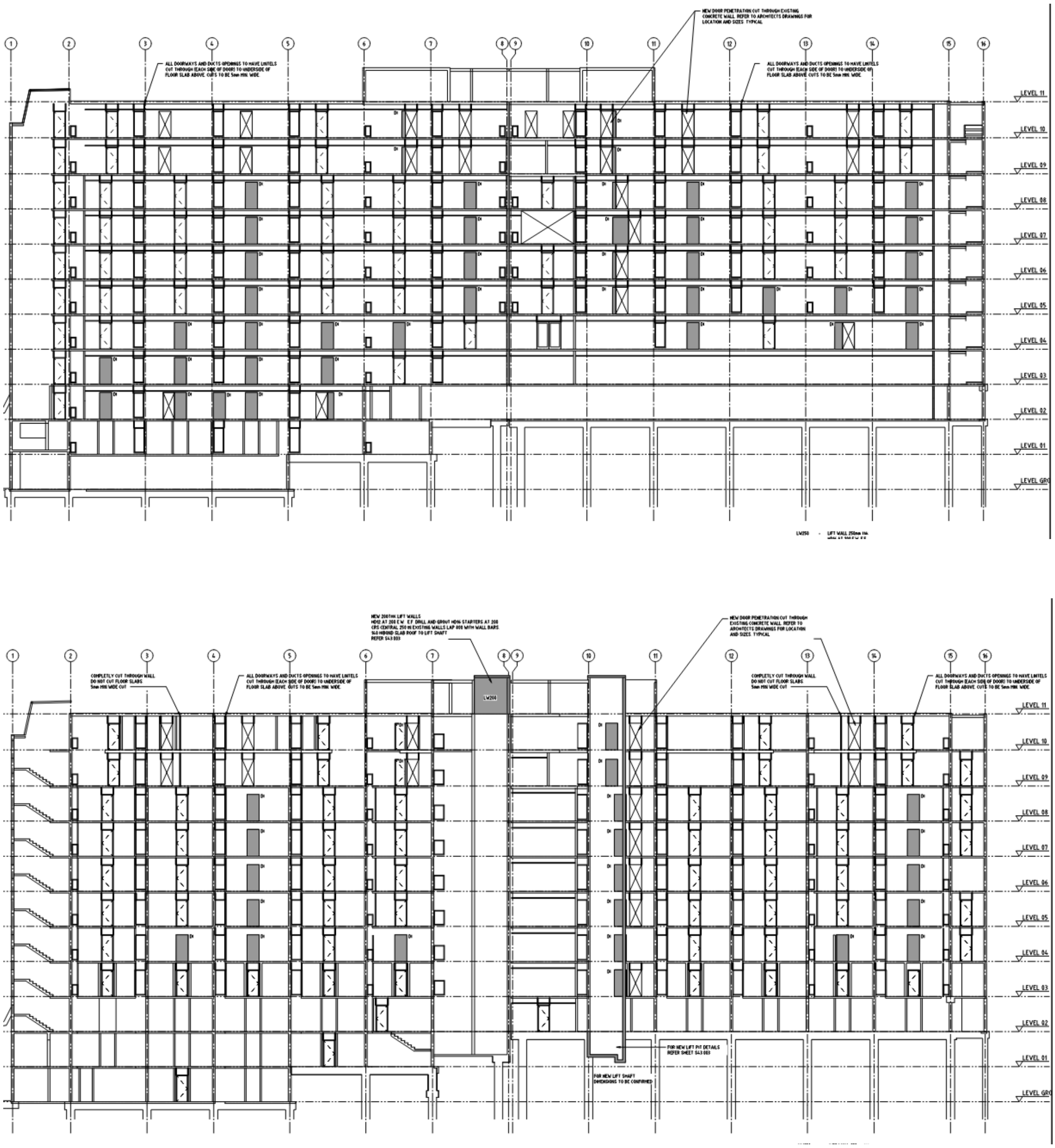

Figure 3-1, Lateral Load Resisting Elements in the Longitudinal Direction

Transverse Direction

The lateral system in the transverse direction consists of a series of shear walls. Generally below level 6 the walls are 8” (200mm) thick. Above this level their thickness reduces to 7” (180mm) up to level 9. The walls up to level 9 are reinforced with 2 layers of deformed bars both faces both ways. The diameter of the bars and their spacing varies among the walls. The walls are generally located between tenancies as well as around the stair cores. The trimmer bars around wall openings are similar to those in the longitudinal direction. Above level 9 certain walls are terminated. The thickness of the rest of the walls extended to the roof level is reduced to 6” (150mm). These walls are reinforced with single layer of the 1/2” (12.7mm) deformed bars at 12” (305mm) spacing both ways.

In 2013 strengthening, a new 125mm skin wall is added at different locations and different levels. These walls are reinforced with a layer of HD12 at 200mm spacing each way. The skin wall is connected to the existing walls with HD12 hooked bars at 600 centres each way. New door penetrations are cut through the existing walls.

Figure 3-2 shows a plan view the lateral load resisting elements in the transverse direction. Location of the new skin walls are highlighted. It is notable that the extent and location of the new skin walls varies among different levels.

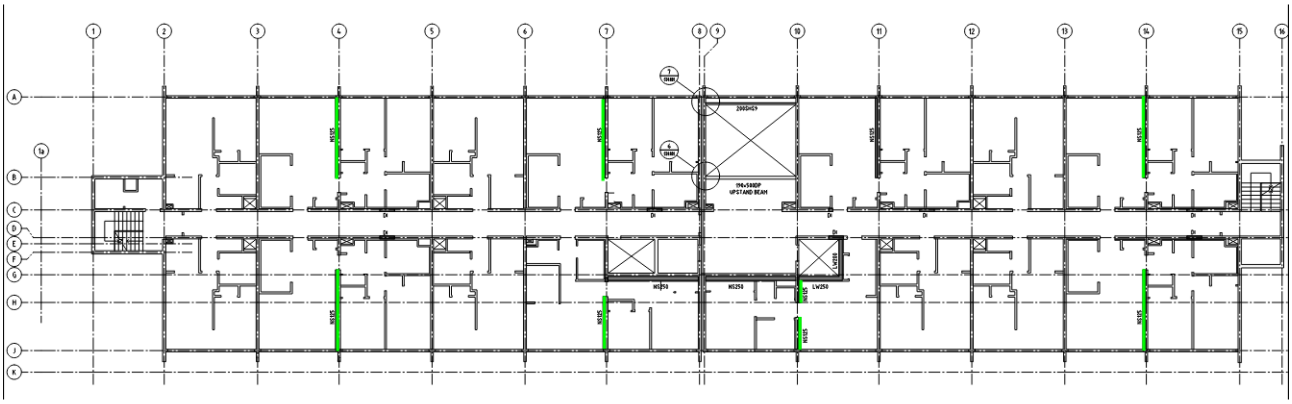

Figure 3-2 Lateral Load Resisting Elements in the Transverse Direction

3.1.2 Horizontal Lateral Resisting Elements

The horizontal lateral load resisting system consists of:

- The typical floor system of the building consists of a 5” (125mm) thick reinforced concrete flat slab spanning in both the longitudinal and transverse direction. The slabs are reinforced with deformed bars. The top reinforcement in the slabs is not continuous over the full span of the slab. Top bars are only located in hogging moment regions. Saddle bars and starter bars connect the floor diaphragm to the shear walls.
- The horizontal load is transferred from the floor slab, by starter reinforcement bars along the RC Shear Walls.
- A new Level 10 added in 2013 uses two layers of 21mm plywood forming a diaphragm supported on the new timber floor joists spanning in the transverse direction between the PFC and timber beams. The timber floor joists are connected with post-installed mechanical anchors to the existing 150mm thick concrete walls to transfer the lateral loads. **Figure** 3-3 shows a section of the floor at level 10.
- The new roof does not have a cross bracing. It relies on the out-of-plane capacity of the DHS purlins spanning between the walls and roof steel beams to resist the lateral loads. **Figure** 3-4 shows a section of the roof.

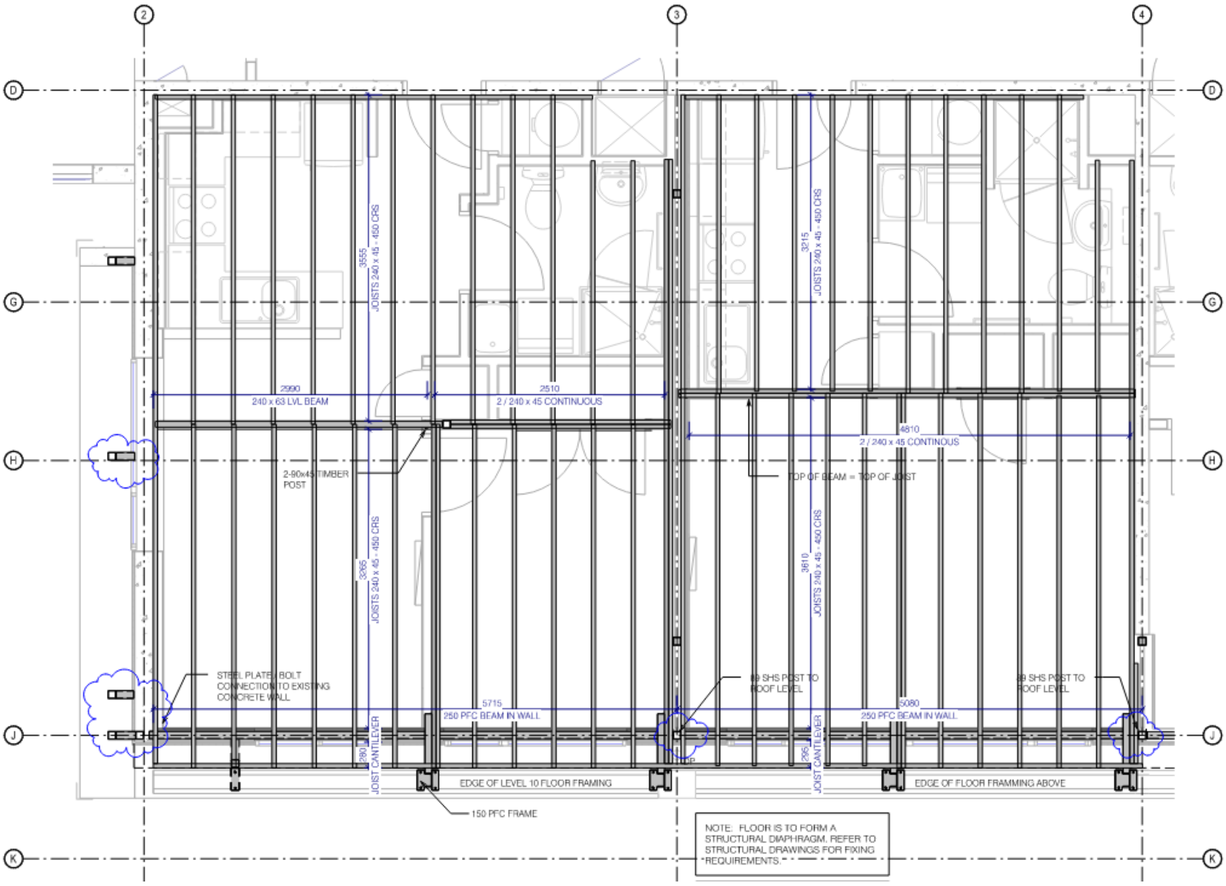

Figure 3-3, Flooring System at Level 10

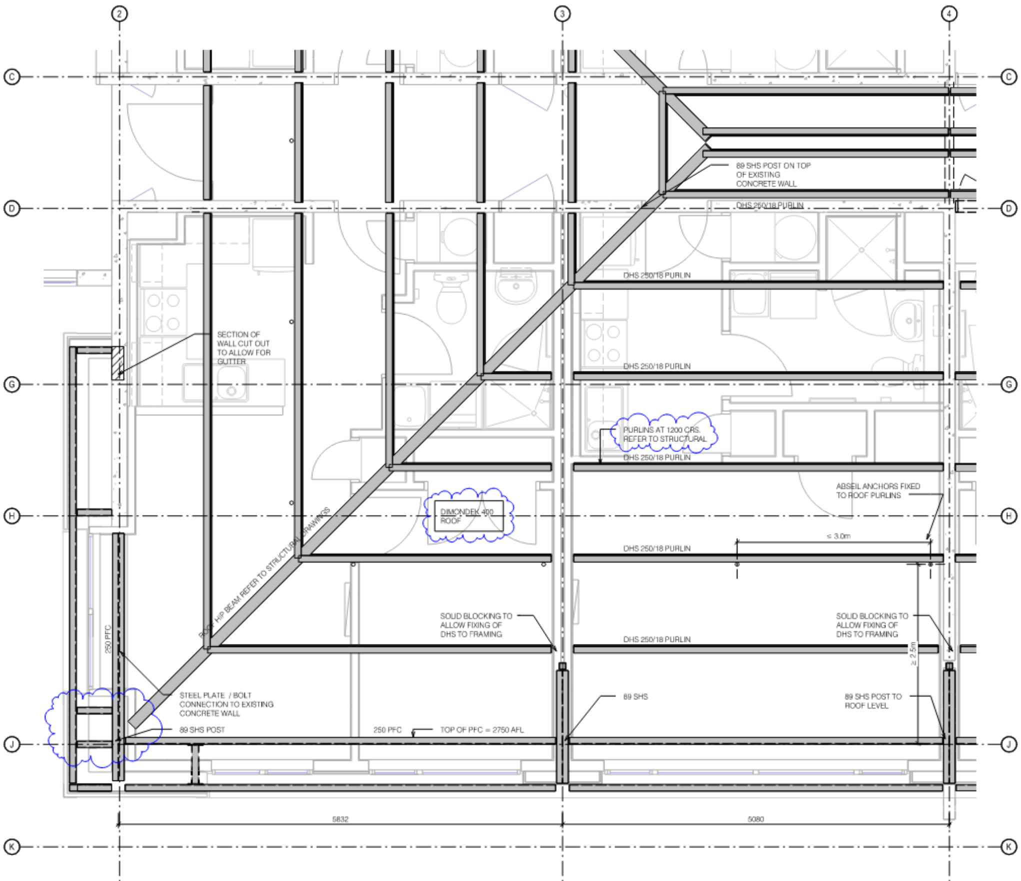

Figure 3-4, Roof Structure

3.2 Gravity System

The typical floor system of the building below level 10 consists of a 125mm thick two-way spanning reinforced flat slab spanning in both longitudinal and transverse directions. The slab is doubly reinforced at the walls and singly reinforced at all slab midspans using deformed bars. The top reinforcement in the slab is not continuous over the full span of the slab. Top bars are only located in the hogging moment regions. Slab is connected to the shear walls with saddle and starter bars. The slabs are typically supported by the RC shear walls from 3 sides and RC moment frames. Refer to **Figure** 3-5 for a section of typical wall to slab interface.

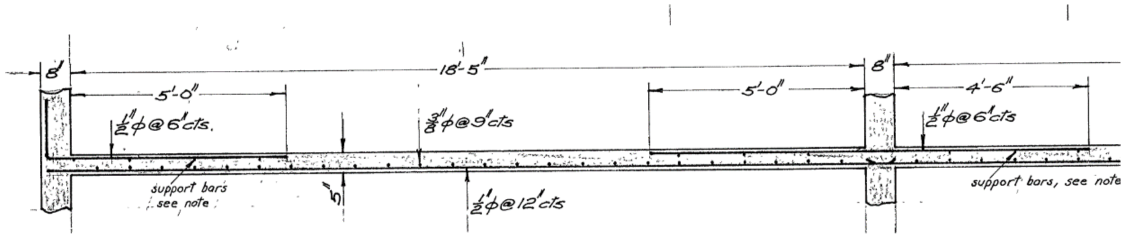

Figure 3-5 Section of typical wall to slab interface

At level 10 the gravity load is transferred from the plywood diaphragms to the timber joists and timber and PFC beams and to the SHS posts extended from level 9 and to RC shear walls.

At the roof level, the gravity load is transferred from the Dimondek metal sheeting to the purlins and transfers through the steel beams to SHS posts extended from level 9 and to RC shear walls.

3.3 Foundations

The buildings are supported on a series of stepped ground beams in both the longitudinal and transverse directions supporting the shear walls. The ground beams are generally 900mm wide and have varying depth between 510mm to 610mm. it is notable that the seismic gap is extended to the ground beam level making the North and South buildings fully separate from each other.

The ground beams are supported by the series of piles. In total 181 bored bulb-end piles are used. 99 piles are of 685mm diameter, and the rest are 480mm in diameter. Depth of the piles varies between 2.5m to 10m. Spacing between the piles varies between 1.3m to 2.2m in the transverse direction. In the longitudinal directions piles are spaced at 5.5m in average.

Foundation beams and piles layout plans are shown in **Figure** 3-6 and **Figure** 3-7.

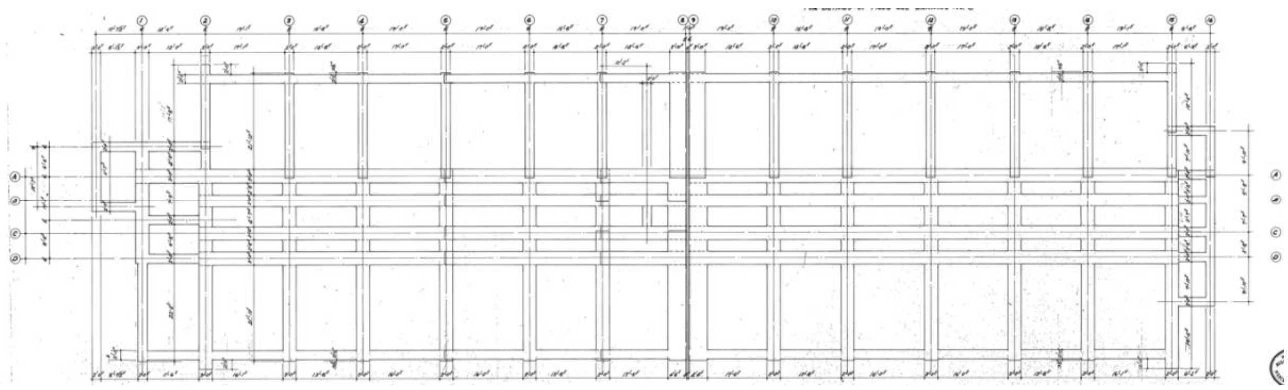

Figure 3-6 Foundation Beams Layout Plan

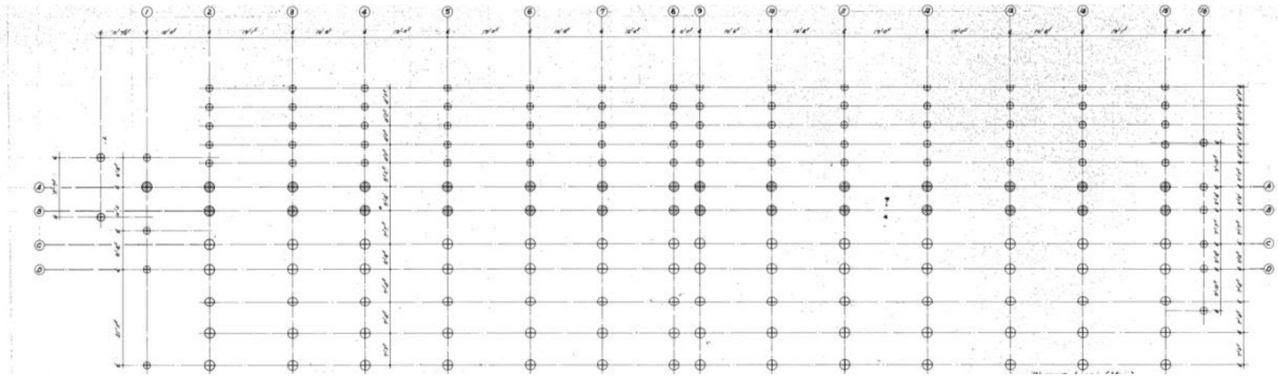

Figure 3-7, Pile Layout Plan

3.4 Subsoil

A geotechnical study was performed as part of the assessment and strengthening in 2009, refer to **Appendix G** for the report. Several active and inactive faults lie near the site, the most important of which is the active Wellington Fault, which lies approximately 2.7km northwest of the site. Based on the two Boreholes on site, it was concluded that the depth to very weak Greywacke rock is generally greater than 3n, and hence the site subsoil has been considered as **Subsoil Class C.**

3.5 Stairs

There are two main stair cores located at the North and South ends of the building. The main entrance lobby is located centrally on the North-West corner of the South building. Some additional stairs are localised to certain number of the tenancies below Level 6. **Figure** 3-8, Stairs Location shows a general floor plan between Level 6 and 10 with the locations of the stairs.

All the main stairs are in-situ concrete stairs with a 5" thick throat. The connections of the stairs to the floors are fixed with no allowance for sliding or lateral movement of the building. **Figure** 3-9 shows an elevation of the stairs.

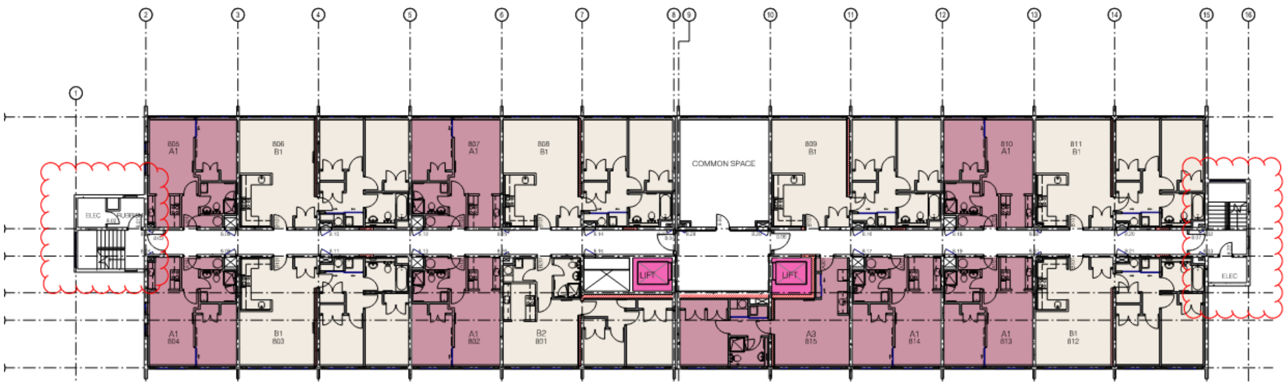

Figure 3-8, Stairs Location

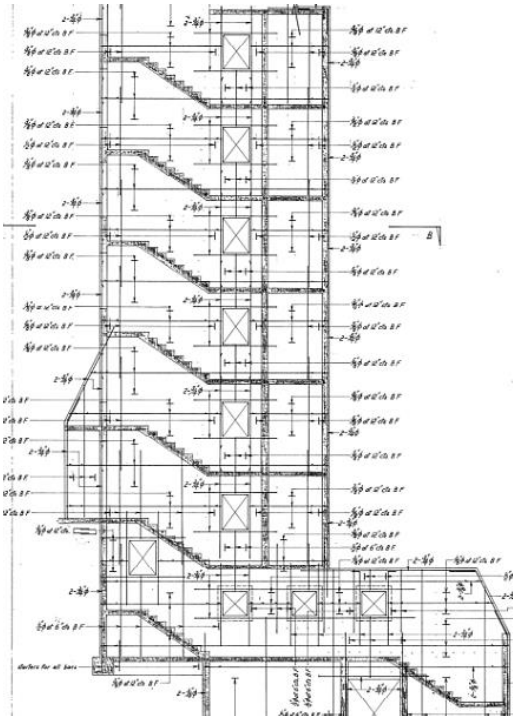

Figure 3-9, Stairs Elevation

3.6 Canopies

New canopies are added at the entrance of the building at the East and West of the building. These canopies are supported on PFC steel posts and beams with lightweight timber and corrugated iron roofing. **Figure** 3-10 shows the West entrance canopy and **Figure** 3-11 shows the East entrance canopy. Additionally, a new two-storey box feature is made of the steel frame and windows at the of the West end of the building. **Figure** 3-12 shows the box feature on the West elevation.

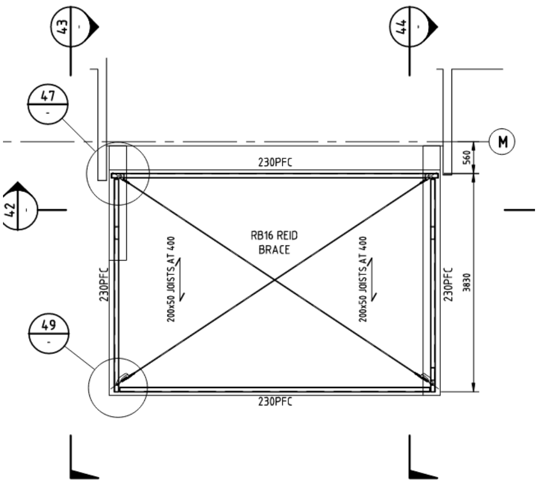

Figure 3-10, West Canopy, Plan View

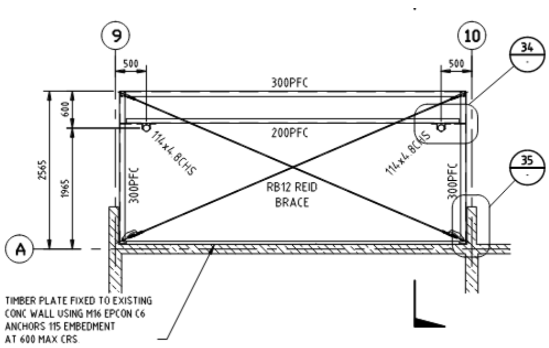

Figure 3-11 East Canopy. Plan View

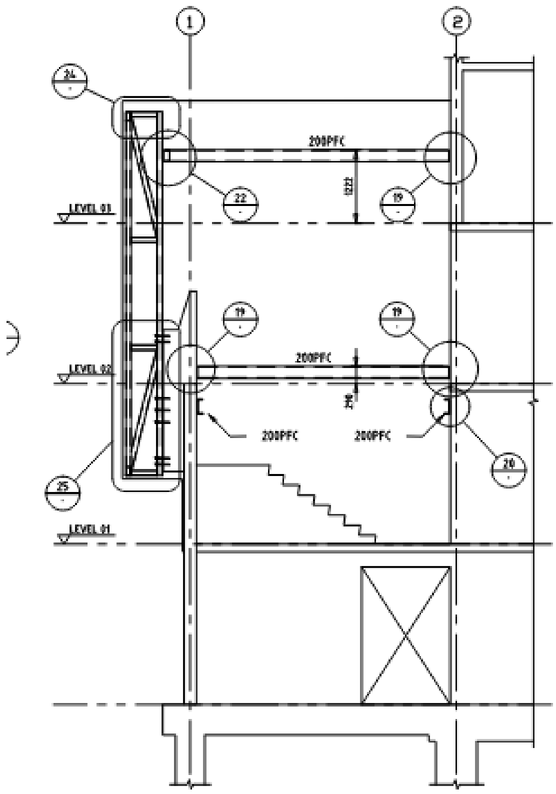

Figure 3-12, Box Feature on the West Elevation

3.7 Non-structural Building Elements

From our recent experience in evaluating similar buildings in Christchurch and Wellington, non-structural building elements (façade, ceilings, internal walls, overhead services etc.) constitute a significant portion of the repair/reinstatement cost following an earthquake. In a moderate seismic event, non-structural element damage may contribute heavily to downtime and repair costs and therefore the performance of these non-structural elements following a moderate seismic event could affect business continuity.

Assessment of these non-structural elements is not part of this DSA. However, a desktop study of the available documentation did not identify any large plant, ceilings, and partitions that would raise concern.

4 Assessment Methodology

4.1 Assessment Description

The DSA was completed in accordance with the **Guidelines.** The Guidelines provide solutions and methods for the assessment of existing buildings and give guidance for strengthening methodologies that are considered acceptable. Refer to **Appendix B** for the Assessment Inputs.

We have undertaken a stepped analysis approach to assess this building. We started with simpler elastic analysis methods and progressed with more complex analysis (including nonlinear static and response spectrum analyses) to determine the seismic performance of the building.

4.2 Computer Modelling

4.2.1 Boundary conditions

A computer model of the structures was developed using the ETABS computer program. Refer to Figure 4-1- North Building ETABS Model and Figure 4-2, South Building ETABS Model for the 3D View of the ETABS Models of the North and South Buildings, respectively. A Simple Lateral Mechanism Analysis (SLaMA) procedure was also undertaken to determine the global capacity of the structure.

To investigate the effect of the soil flexibility on the behaviour of the superstructure, two sets of boundary conditions were considered:

- Models without consideration of soil flexibility: The base of the walls at different levels are fixed in the vertical direction and springs with high stiffness are assigned to both horizontal directions. this is to ensure that the base shear takeout is happening gradually among the founding levels.
- Models with soil flexibility: the supports were modelled with nonlinear springs to capture the vertical and horizontal resistance of the bored piles and account for the soil flexibility. The soil springs' stiffnesses were defined in accordance with the values provided by the 2009 Geotechnical Report. As the piles were closely spaced in the transverse direction, the group effect was also considered in the stiffness calculation of the soil springs in this direction. In the transverse direction, the group effect was not identical in the in positive and negative loading directions. Therefore, two separate models were developed, and the most critical load cases was adopted accordingly for the assessment of different elements. The global structures' behaviour was captured using non-linear equivalent static analysis.

To assess the stair performance, the stairs were added to the 3D ETABS model to determine how much load they would attract given their proximity to shear walls.

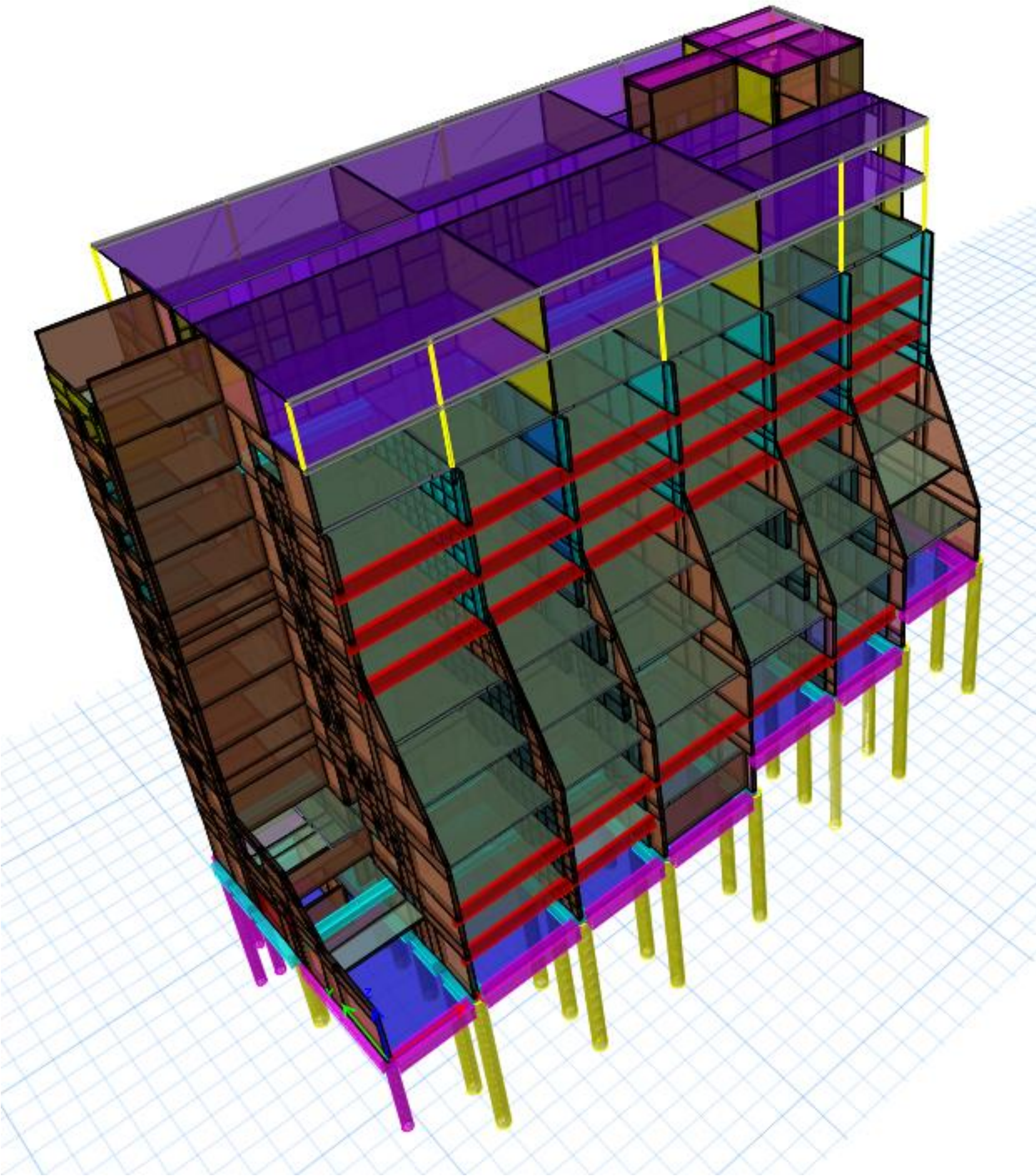

Figure 4-1- North Building ETABS Model

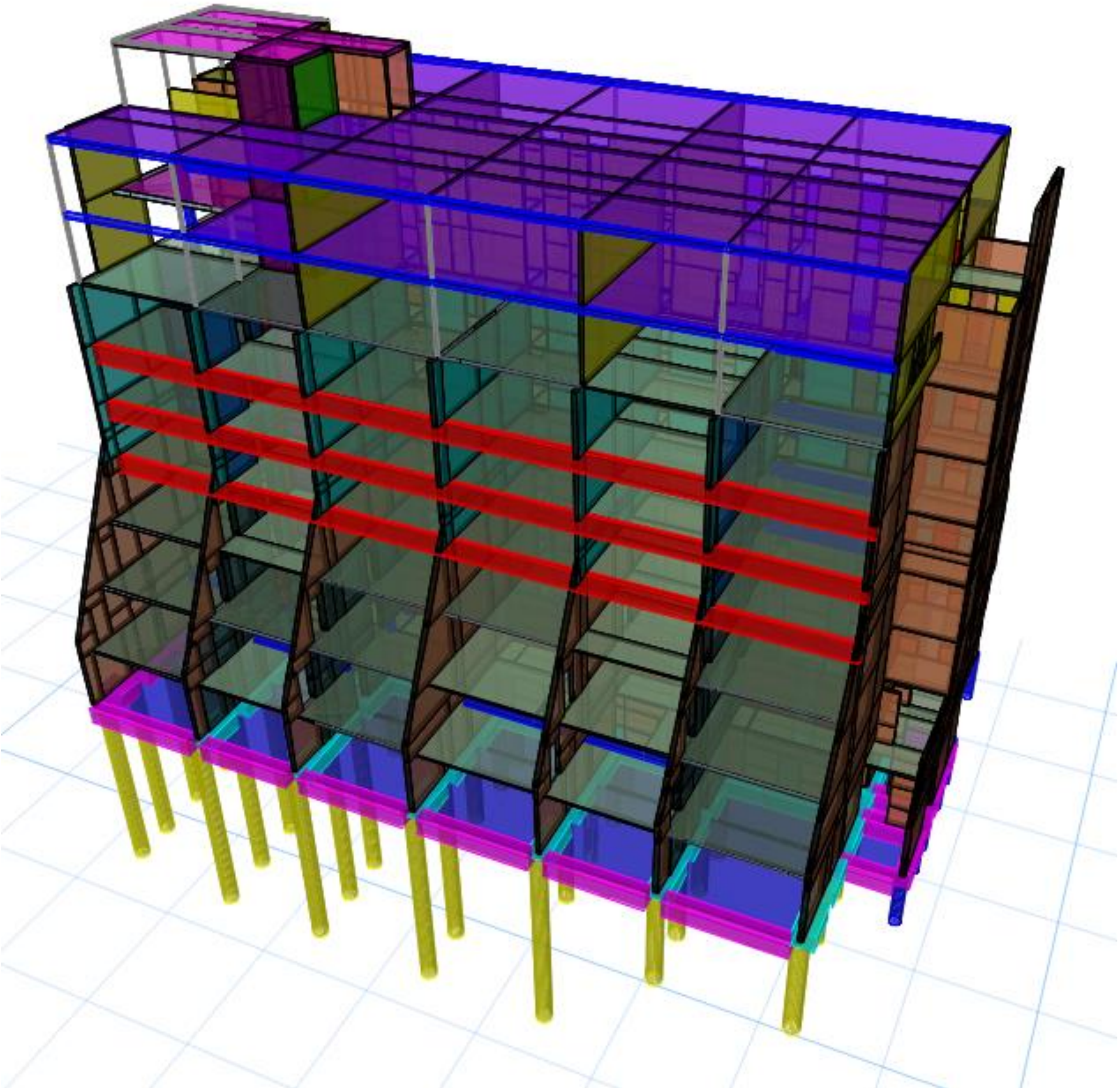

Figure 4-2, South Building ETABS Model

4.2.2 Torsional Amplification

Due to the irregular arrangement of walls over the building's length and height, torsion effects have been considered in the seismic loading. NZS11170.5 2004, Appendix C2F of the Guidelines and Section 6 and ASCE7-22, Section 12 has been used to determine the structures torsional sensitivity and torsional amplification factors.

The North and South structures were identified to be torsionally sensitive under loading in the transverse direction as specified by NZS1170.5.. ASCE7-22 is referenced to determine the Torsional Irregularity Ratio (TIR) and corresponding torsional amplification factor to be used in the Nonlinear Static Analysis.

Torsional modelling of the structure is a combination of two factors, inherent torsion (M_t) and accidental torsion (M_{ta}). Inherent torsion is the torsion associated with the structures centre of mass and centre of stiffness. The accidental torsion accounts for uncertainties in the mass and stiffness.

The torsional amplification factor (A_x) for each story was determined based on the ETABS max over average displacements. These factors were then averaged over the height of the structure and applied to the accidental eccentricity within the relevant ETABS seismic load pattern functions.

4.3 Primary Lateral Load Resisting Elements

4.3.1 Concrete Shear Walls

Reinforced concrete shear walls and lintels make up the primary lateral load resisting system for both buildings in the longitudinal and transverse directions. The longitudinal and transverse walls are typically connected, forming series of T, C, box and other complex shapes, which vary up the height of the buildings depending on their floor layout.

Where the wall sections are less complex, the SLaMA analysis has shown that there is a possibility of potential hinge formation. As the buildings are stepped, Location of hinge formation can vary along the few bottom levels of the structure. For the North Building, the hinges will form on Levels 2 and 3 while for the South Building, the hinges are forming at levels 3 to 5.

Therefore, with some localised hinge formation to achieve a ductility of 2 in both major orthogonal directions. The SlaMA also shows that a considerable number of walls being governed by their out-of-plane lateral instability indicating that the walls will not have resilience beyond their plastic rotational capacity accounted for, to resist the gravity loading.   localised damage (hinges), however in general the structure will remain within the nominal ductile range. In addition, the walls at the upper levels are singly reinforced and are not appropriate to assess using a ductility greater than 1.25 for their in-plane performance. Therefore, the walls in the longitudinal and transverse direction have adopted a ductility of 1.25 for the assessment.

The deformed bars in the walls generally have sufficient lap length and anchorage detailing for the construction period. However, the detailing of the boundary elements of the walls is not compliant with ductile detailing requirements of NZS3101 Amd. 3.

The walls have been modelled as individual piers and spandrels (where appropriate) in each direction to determine expected demands. The walls in the transverse direction have also been separated from the longitudinal walls at their intersections.

Spandrels were assessed initially, where insufficient shear or flexural capacities were identified, the spandrels were cracked. This represented the uncoupling of the walls and allowed for a redistribution of forces. This process was iterated until the remaining spandrels were identified as sufficient to resist the redistributed demands.

The concrete piers were assessed based on their size and reinforcement at each story to determine the critical ratings. In locations where walls were identified as earthquake prone (<67%NBS), a load redistribution of 30% was deemed acceptable. This was as the based on the assumption that as a pier reaches capacity it undergoes softening and causes demands to distribute to the stiffer elements. The critical walls were assessed based on their redistributed demands.

4.3.2 Foundations

Concrete walls at the foundations are supported by reinforced concrete ground beams in the longitudinal and transverse direction. The ground beams in turn are supported by a number of bored piles. The lateral resistance in both directions is predominantly provided by the piles with some consideration given the passive soil resistance provided by the ground beams.

The bored piles have been modelled and assessed based on the 2009 geotechnical report which presents vertical (tension/compression) and lateral capacities for the piles based on site testing and modelling.

The expected passive pressure from the ground beams has been determined based on the equations referenced in the 2009 Geotechnical Report.

Frictional resistance provided by the ground slab and base of the ground beams has not been utilised for the lateral load resistance. This is because the walls are directly supported on piles, resulting in a small percentage of the demands in the ground beams. and reduction of soil ground beam/slab contact. The method of lateral resistance relying on the friction force as a function of weight, deemed not suitable for this assessment.

Also, the passive pressure from the soil retaining structures is not relied on. As shown in **Figure** 4-3 the general construction method was to build a temporary soil retaining structure and then build the shear walls of the building with a gap behind the temporary retaining structure. Therefore, even when the piles fail and the building starts to slide, the passive pressure of the soil retaining structure will not provide immediate resistance until the sliding exceeds the gap width. Therefore, this mechanism is not considered to be reliable to resist the lateral loads.

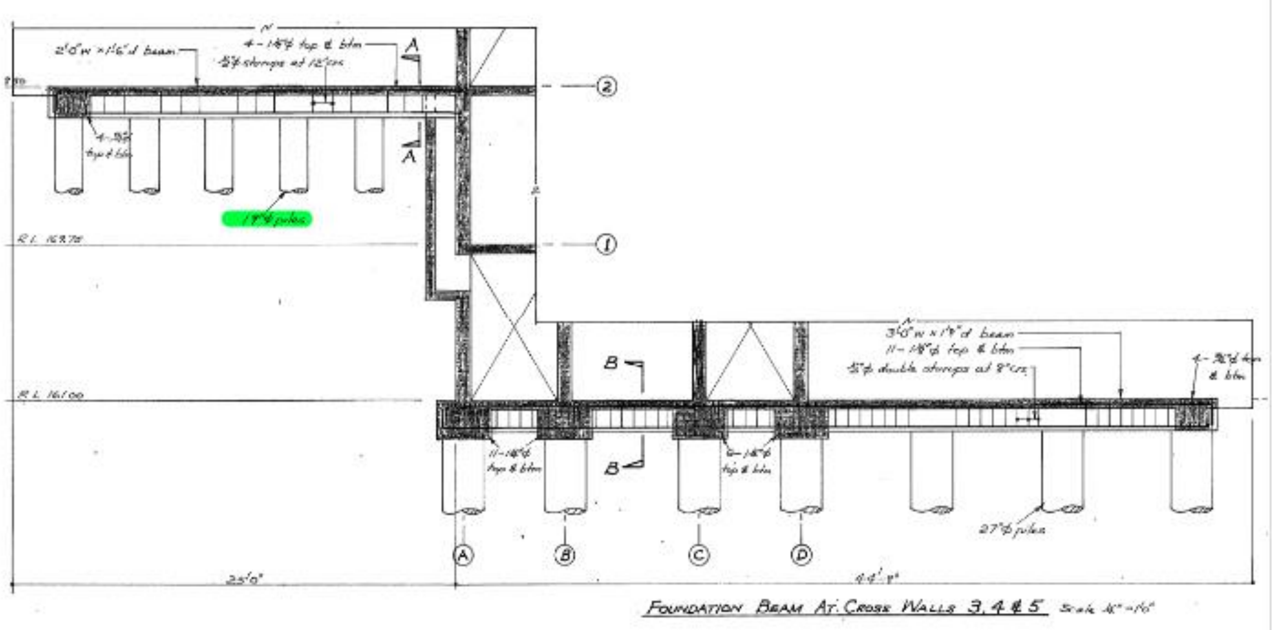

Figure 4-3, Typical Soil Retaining Structure

4.3.3 Diaphragms

The diaphragm acceleration demands were determined by the pESA method as recommended in NZS1170.5 C5.7.2. While the south building has 9 stories, certain area of the North building has 11 stories. While this exceeds the 9 stories as upper bound for pESA loading application, it should be noted that the response of the structures is much stiffer compared with the buildings designed to the current codes. Also, high mass participation in the first mode in each direction confirms that the effect of the higher modes is negligible.

These design accelerations/forces were then applied to the centre of mass of each diaphragm of the 3D ETABS model. For each diaphragm and for each direction of loading, the shear entering/exiting each vertical lateral resisting element (difference in shear above and below the level being considered) was extracted.

Due to the complexity of the diaphragms the diaphragm demands were assessed using the Grillage Method as recommended in the Guidelines. It is essentially an automated strut and tie analysis method to obtain demands. Capacities were determined using Appendix A of NZS 3101:2006. Refer to Figure 4-4 Grillage model of Level 8 for the South Building.

The diaphragm capacity is calculated based on the residual capacity of the hogging reinforcement following application of the gravity load. The hogging reinforcement have sufficient length to transfer the load based on non-contact lap splice. It is noted that the lap length for the bottom layer of the reinforcement is significantly lesser than what is required. Therefore, the capacity of the bottom layer reinforcement at wall location are ignored. To improve the current rating of the diaphragms, Aurecon recommends measuring the as-built lap length of the bottom reinforcement layers by chipping away the bottom concrete cover at both sides of the walls.

The performance of the diaphragms at L10 and roof level are limited by their connections to the vertical resisting elements. The existing connections are Trubolts. The anchors are not compliant with the current NZS3101 C2 requirements. They also have insufficient embedment depth.

The rating of the elements will be determined based on the floor accelerations resulted from RSA. For the assessment of these specific elements, the nonlinear vertical springs at the foundation level are replaced with equivalent linear springs.

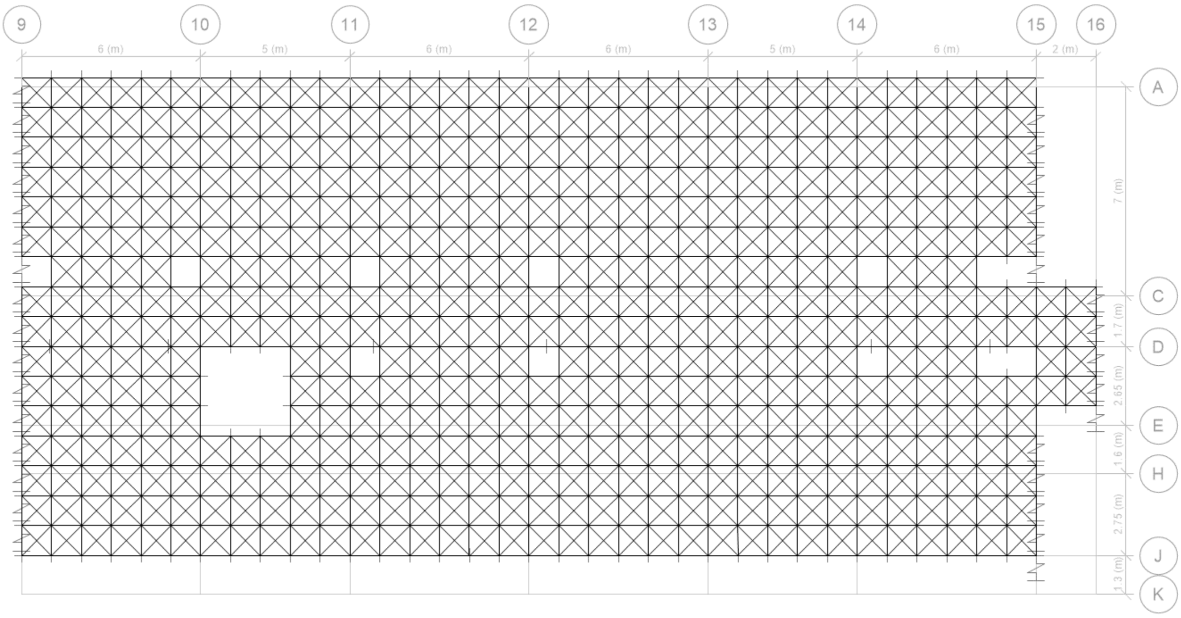

Figure 4-4 Grillage model of Level 8 for the South Building

4.4 Secondary elements

4.4.1 Stairs

To assess the stair performance, the stairs were added to the 3D ETABS model to determine how much load they would attract given their proximity to shear walls.

4.4.2 Water Tank

The water tank has been evaluated based on its walls' out-of-plane capacity when subjected to seismic loading using NZS1170.5 Part coefficients, and a comparison has been made with the NZS3106:2009 loading.

5 Assessment Results

5.1 Assessment Results Summary

The results of the DSA indicate that the buildings' earthquake rating to be **15%*NBS*(IL2**) in accordance with the Guidelines. The earthquake rating is based on the lowest scoring element shown in Table 5-1 and Table 5-2, Summary of the South Building elements' %NBS scores.

Table 5-1, Summary of the North Building elements' %NBS scores

Element:	%NBS(IL2):	Commentary:
RC Shear Walls – Singly Reinforced In-Plane Loading -Longitudinal -Transverse	 >34% >34%	▪ Certain number of RC piers and spandrels have insufficient flexural capacity to resist the expected seismic demands and limits the rating to >34%NBS(IL2) in the longitudinal and transverse directions, respectively (Refer to Appendix H). ▪ The single layer of reinforcement within the walls limits the probable flexural capacity of the 6" walls at the upper levels of the structures.
RC Shear Walls – Doubly Reinforced In-Plane Loading -Longitudinal -Transverse	 >34% >67%	▪ In the longitudinal direction certain number of RC piers and spandrels have insufficient flexural capacity as a result of the flexure – axial interactions to resist the expected seismic demands >34%NBS at IL2 in the longitudinal and transverse directions respectively (Refer to Appendix H). ▪ The irregular openings in walls and spandrel elements result in walls acting as a coupled and inducing large tensile forces in the piers. Tension forces greatly reduce the probable flexural capacity of the shear wall elements. ▪ In the transverse direction the capacity is limited by the shear capacity of the squat walls at the lower levels along gridline 1. ▪
Concrete Diaphragms	55%	▪ The level one, two and eight floor diaphragms are limiting the score to the 55%NBS(IL2). Level nine is scoring 60%NBS(IL2), and the rest of the floors are scoring 65%NBS(IL2) ▪ The concrete diaphragm, reinforced with deformed bars, have insufficient capacity to transfer the diaphragm inertia and transfer loads to the vertical elements of the lateral resisting system. ▪ We note the score is based on specific areas of the floor and not the whole floor. ▪ Intrusive site investigation may help to improve this rating.
Foundations: - Longitudinal - Transverse	 60% 85%	▪ The bored piles lateral capacity in combination with the passive pressure resistance of the ground beams are insufficient to resist 67%NBS of an IL2 event in the longitudinal and 100% in the transverse directions.
Stairs	100%	▪ The stairs are concrete cast in-situ. The connections of the stairs to the landings are fixed with no allowance for sliding or lateral movement of the building. The flights are cast into the face of the landings with deformed bars. Though stairs have a seismic capacity of 100%NBS.
Walls Out-of-Plane	30%	▪ The RC walls above Level 9 are cantilevering to support the roof system. Rating of the new diaphragm added at level 10 affect this. This cantilever is as high as 5.5m. It is not expected that the roof system can provide restraint of the walls for out-of-plane loading. The walls score 30%*NBS*(IL2) for out-of-plane seismic parts loading.

Roof and timber diaphragm at Level 10	<34%	▪ The steel roof is a flexible diaphragm and does not contain steel-cross braces or a plywood diaphragm. Therefore, the DHS purlins must transfer seismic load from the roof to the RC walls by bending out-of-plane. ▪ The timber diaphragm located at level 10 is connected to concrete walls, however, the connections do not possess sufficient capacity to resist the ultimate limit state (ULS) seismic actions. The diaphragm has a seismic capacity score of <34%NBS (IL2). The capacity of connections between the timber floor and the concrete walls using T16 Trubolts with unknown embedment depthlimits the rating.
Water tank at roof level	50%	▪ The reinforced concrete water tank located at the roof level has achieved a seismic performance score of 50%NBS (IL2), based on its walls' out-of-plane capacity when subjected to seismic loading. Considering the location of the water tank and the identified mode of failure, we do not consider it as life safety concern.

Table 5-2, Summary of the South Building elements' %NBS scores

Element:	%NBS(IL2):	Commentary:
RC Shear Walls – Singly Reinforced In-Plane Loading -Longitudinal -Transverse	 >34% >34%	▪ Certain number of RC piers and spandrels have insufficient flexural capacity to resist the expected seismic demands for 25%NBS and 30%NBS at IL2 in the longitudinal and transverse directions of the south building respectively (Refer to Appendix H). ▪ The single layer of reinforcement within the walls limits the probable flexural capacity of the 6" walls at the upper levels of the structures.
RC Shear Walls – Doubly Reinforced In-Plane Loading -Longitudinal -Transverse	 >34% 100%	▪ In the longitudinal direction, certain number of RC piers and spandrels have insufficient flexural capacity as a result of the flexure – axial interactions to resist the expected seismic demands for 25%NBS and 40%NBS at IL2 in the longitudinal and transverse directions of the South building respectively (Refer to Appendix H). ▪ The irregular opening in walls and spandrel elements result in walls which act as a coupled shear wall, inducing large tensile forces in the piers. Tension forces greatly reduce the probable flexural capacity of the shear wall elements.
Concrete Diaphragms	60%	▪ The level eight floor diaphragm for the South building is limiting the score to the 60%NBS(IL2) for south building. Level nine is scoring 65%NBS(IL2) and the rest of the floors are scoring 67%NBS(IL2). ▪ The concrete diaphragm, reinforced with deformed bars, have insufficient capacity to transfer the diaphragm inertia and transfer loads to the vertical elements of the lateral resisting system. ▪ We note the score is based on specific areas of the floor and not the whole floor. ▪ Site investigation may improve this rating.
Foundations: - Longitudinal - Transverse	 65% 90%	▪ The bored piles and passive resistance provided by the ground beams do not have sufficient lateral capacity for 67%NBS and 100%NBS of an IL2 event in the longitudinal and transverse direction of the South building respectively.

Stairs	100%	■ The stairs are concrete cast in-situ. The connections of the stairs to the landings are fixed with no allowance for sliding or lateral movement of the building. The flights are cast into the face of the landings with deformed bars. Though The stairs have a seismic capacity of 100%NBS.
Walls Out-of-Plane	30%	■ The RC walls above Level 9 are cantilevering to support the roof system. Rating of the new diaphragm added at level 10 affect this. This cantilever is as high as 5.5m. It is not expected that the roof system can provide restraint of the walls for out-of-plane loading. The walls score 30%*NBS*(IL2) for out-of-plane seismic parts loading.
Roof and timber diaphragm	<34%	■ The steel roof is a flexible diaphragm and does not contain steel-cross braces or a plywood diaphragm. Therefore, the DHS purlins must transfer seismic load from the roof to the RC walls by bending out-of-plane. ■ The timber diaphragm located at level 10 is connected to concrete walls, however, the connections do not possess sufficient capacity to resist the ultimate limit state (ULS) seismic actions. The diaphragm has a seismic capacity score of <34%NBS (IL2). The connections between the timber floor and the concrete walls using T16 Trubolts with unknown embedment depth limits the rating.
Canopies	>67%	■ The canopies have a seismic score of >67% ULS seismic actions. The failure is due to concrete cone failure and splitting failure.
Water tank at roof level	50%	■ The reinforced concrete water tank located at the roof level has achieved a seismic performance score of 50%NBS (IL2), based on its walls' out-of-plane capacity when subjected to seismic loading. Considering the location of the water tank and the identified mode of failure, we do not consider it as life safety concern.

5.2 Structural Weaknesses

A structural weakness (**SW**) is an aspect of the building structure and/or the foundation that scores less than 100%*NBS*(IL2). The Critical Structural Weakness (**CSW**) is the lowest scoring structural weakness determined in the assessment. Based on the results of the DSA, the CSW for this building is:

- RC walls -out-of-plane flexural capacity
- Roof and level 10 diaphragm connections to shear walls

See below for the other structural weaknesses for the elements considered in this DSA:

- 6” walls out-of-plane capacity
- Concrete diaphragm capacity
- Combined lateral resisting capacity of the foundation elements
-
- Canopy connections
- Water tank out-of-plane capacity

5.3 Severe Structural Weaknesses

A Severe Structural Weakness (SSW) is a defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge.

There are no SSWs identified for this building.

5.4 Displacement and Inter-storey Drift

The building displacements up the height of the building obtained from our analyses for 100%ULS shaking are shown in **Figure** 5-1 below and **Figure** 5-2.

Table 5-3 and Table 5-4 shows the structures time periods, global ductility demand at 100%ULS and the maximum inter-storey drift under 100%ULS shaking. The storey drift allows for the kdm modification factor and P-delta effects. In both directions, the drift is less than the design code limit of 2.5%.

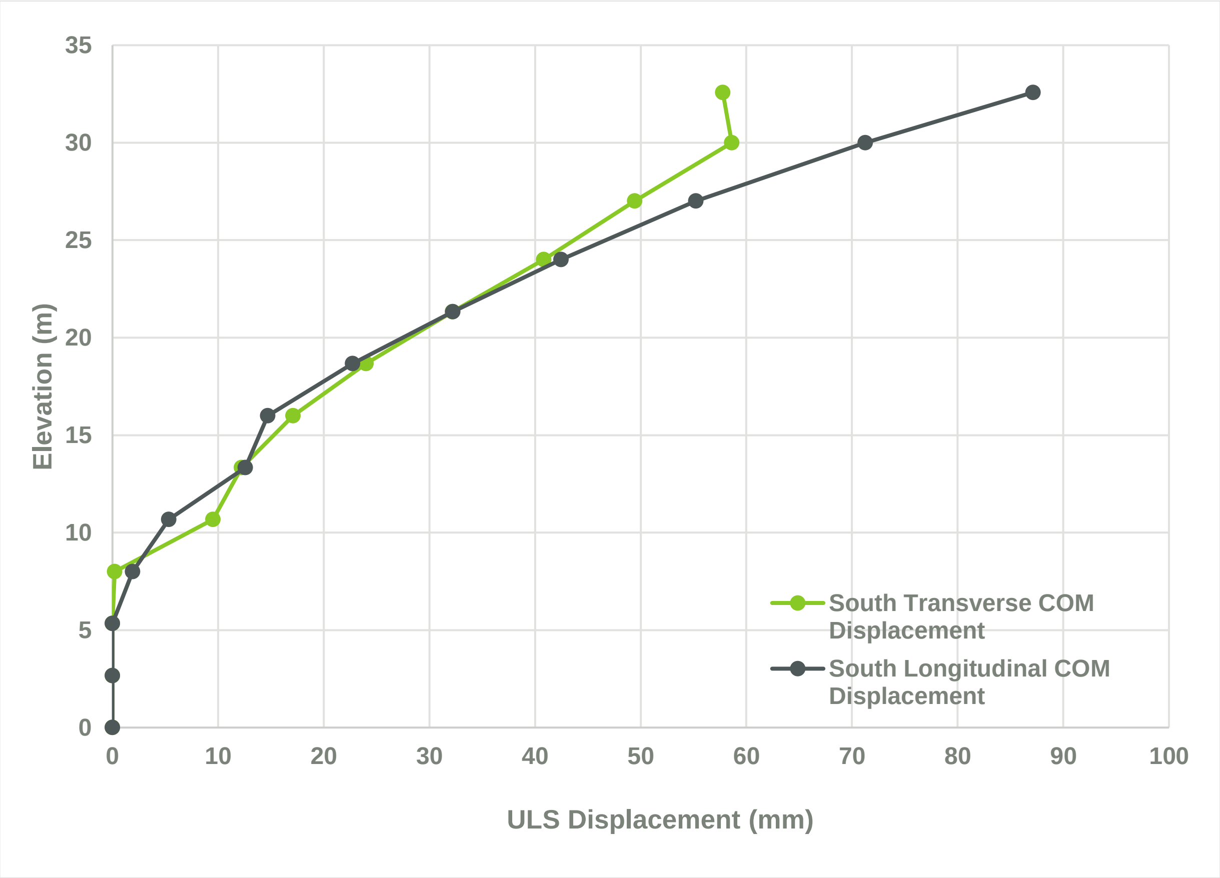

Figure 5-1 Estimated South Building Displacements for 100% ULS shaking

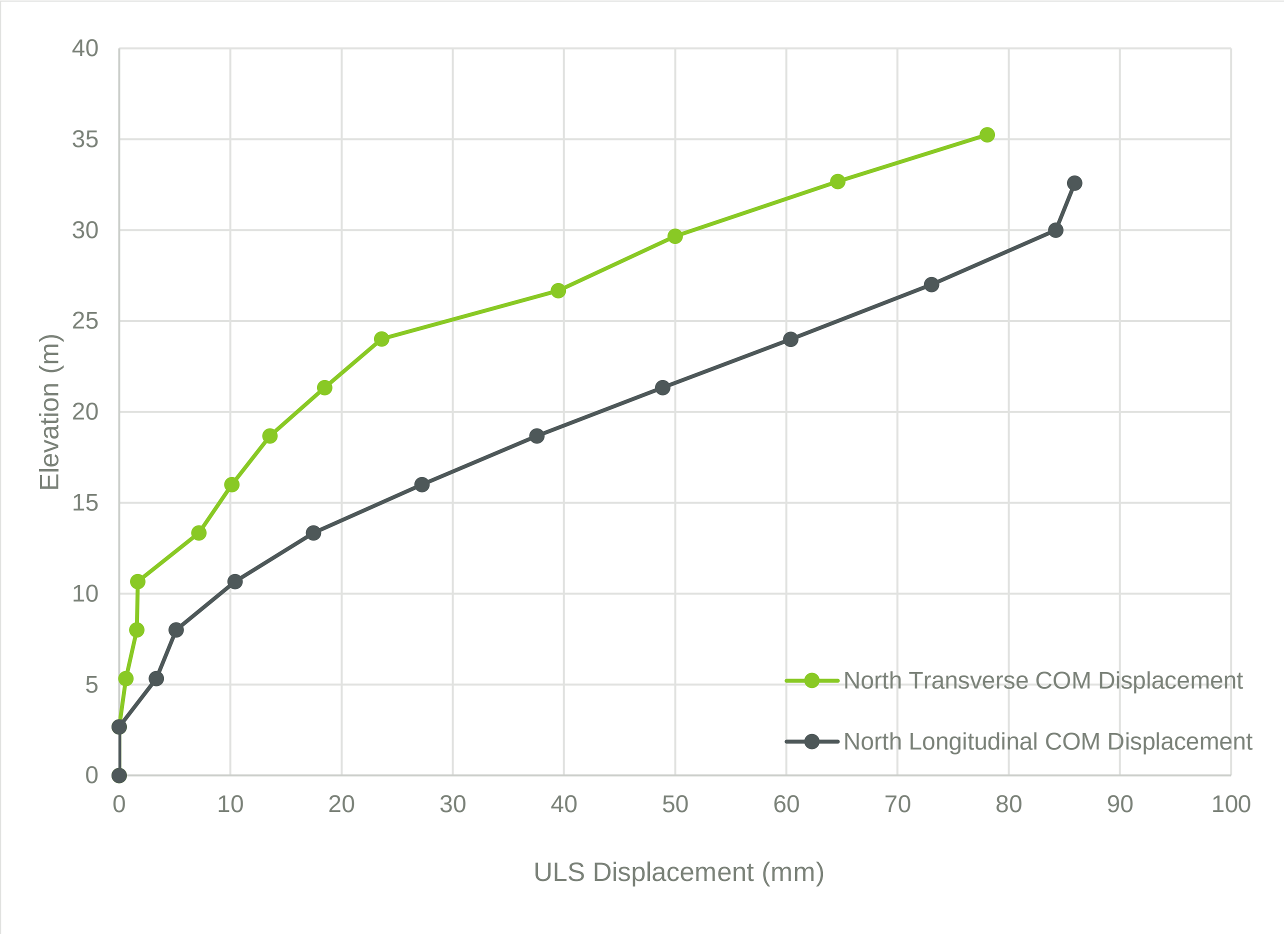

Figure 5-2 Estimated North Building Displacements for 100% ULS shaking

Table 5-3 South Building Estimated Time Periods, Global Ductility and Maximum Inter-Storey Drift for 100% ULS Shaking

Direction	Fundamental Time Periods	Global Ductility	Maximum Inter-storey Drift
Longitudinal	<0.4s (0.58s*)	~1.25	0.5%
Transverse	<0.4s (0.52s*)	~1.25	0.3%

* Numbers in brackets are periods considering soil flexibility

Table 5-4 North Building Estimated Time Periods, Global Ductility and Maximum Inter-Storey Drift for 100% ULS shaking

Direction	Fundamental Time Periods	Global Ductility	Maximum Inter-storey Drift
Longitudinal	<0.4s (0.69s*)	~1.25	0.5%
Transverse	<0.4s (0.65s*)	~1.25	0.4%

* Numbers in brackets are periods considering soil flexibility.

5.5 RC Shear Walls

The building was constructed in the 1970s during a time where there were limited seismic requirements. The understanding of seismic engineering has vastly improved since the building was designed and the loading demand has increased significantly.

Longitudinal and Transverse Direction

The in-plane capacity of RC piers and spandrels which make up the shear walls have been identified as a **Structural Weakness** (SW). The wall ratings for the North and South structures in the longitudinal and transverse directions are summarised in Table 5-5.

Table 5-5, summary of lowest %NBS rating for wall elements

	Direction	%NBS(IL2)
Singly Reinforced Walls		
North Building	Longitudinal	>34%
	Transverse	>34%
South Building	Longitudinal	>34%
	Transverse	>34%
Doubly Reinforced Walls		
North Building	Longitudinal	>34%
	Transverse	>67%
South Building	Longitudinal	>34%
	Transverse	>67%

While the lowest value of interstorey drift is limited to 0.7% for majority of the floor this is over 1%. Shear is the dominant failure mode for the spandrels and the maximum rotational capacity for these spandrels before the loss of lateral load support is limited to 0.3%. Also, spandrel's rotational demands are proportional to the length of the connecting walls vs. the length of the spandrels. Therefore, a considerable number of concrete spandrels are expected to crack and fail in shear at the early stage of lateral displacement. The failure of these spandrels is not expected to pose a concern for life safety as they are supported by the lintels and slab starter bars against fall. Therefore, this type of failure is not considered as the limiting factor for the %NBS rating. A load redistribution considered to capture the behaviour of the structure, in particular piers following the spandrels' failure.

The concrete piers which make up the primary lateral load resisting system have varying performance based on their size and connections to surrounding piers. The critical piers are typically in locations where bond beams (spandrels) are present which cause coupling effects, inducing large tension and compression forces in the piers. Tension loads significantly reduce the probable flexural capacity of the walls.

Some localised walls in the transverse direction were also identified to have insufficient shear capacity for 67%NBS of the expected seismic demand during an IL2 event. However, these walls did not govern the ratings of the global system.

Refer to **Figure 5-3, Critical Transverse Direction Wall – North Structure** and **Figure 5-4, Critical Longitudinal Direction Wall – North Structure** for the critical gridline ratings in the longitudinal and transverse walls in the North Building and **Figure 5-5,** Critical Transverse Direction Wall – South Structure and **Figure 5-6** for the critical longitudinal and transverse walls in the South structure. All critical walls with their scoring are shown in **Appendix H**.

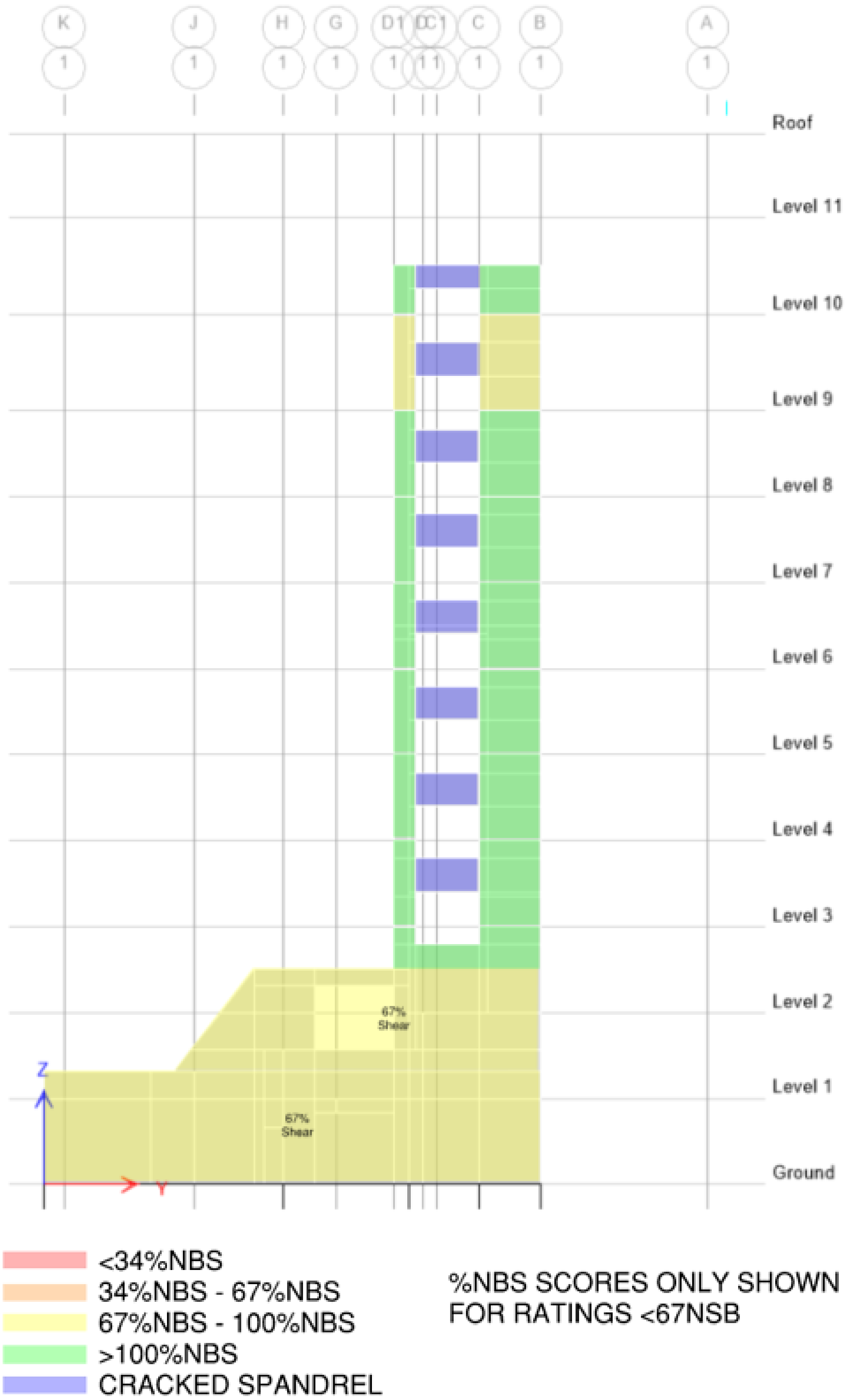

Figure 5-3, Critical Transverse Direction Wall – North Structure

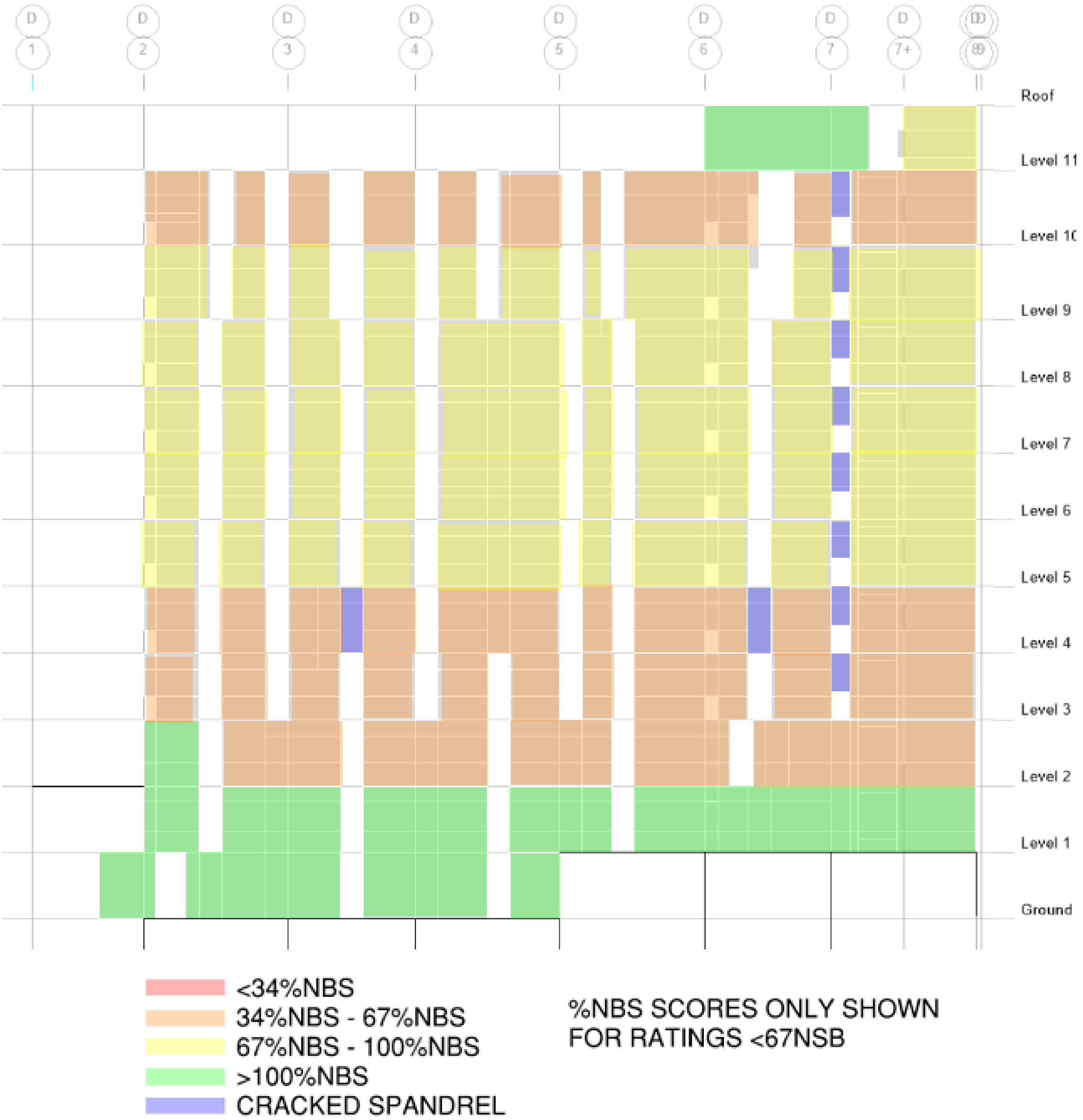

Figure 5-4, Critical Longitudinal Direction Wall – North Structure

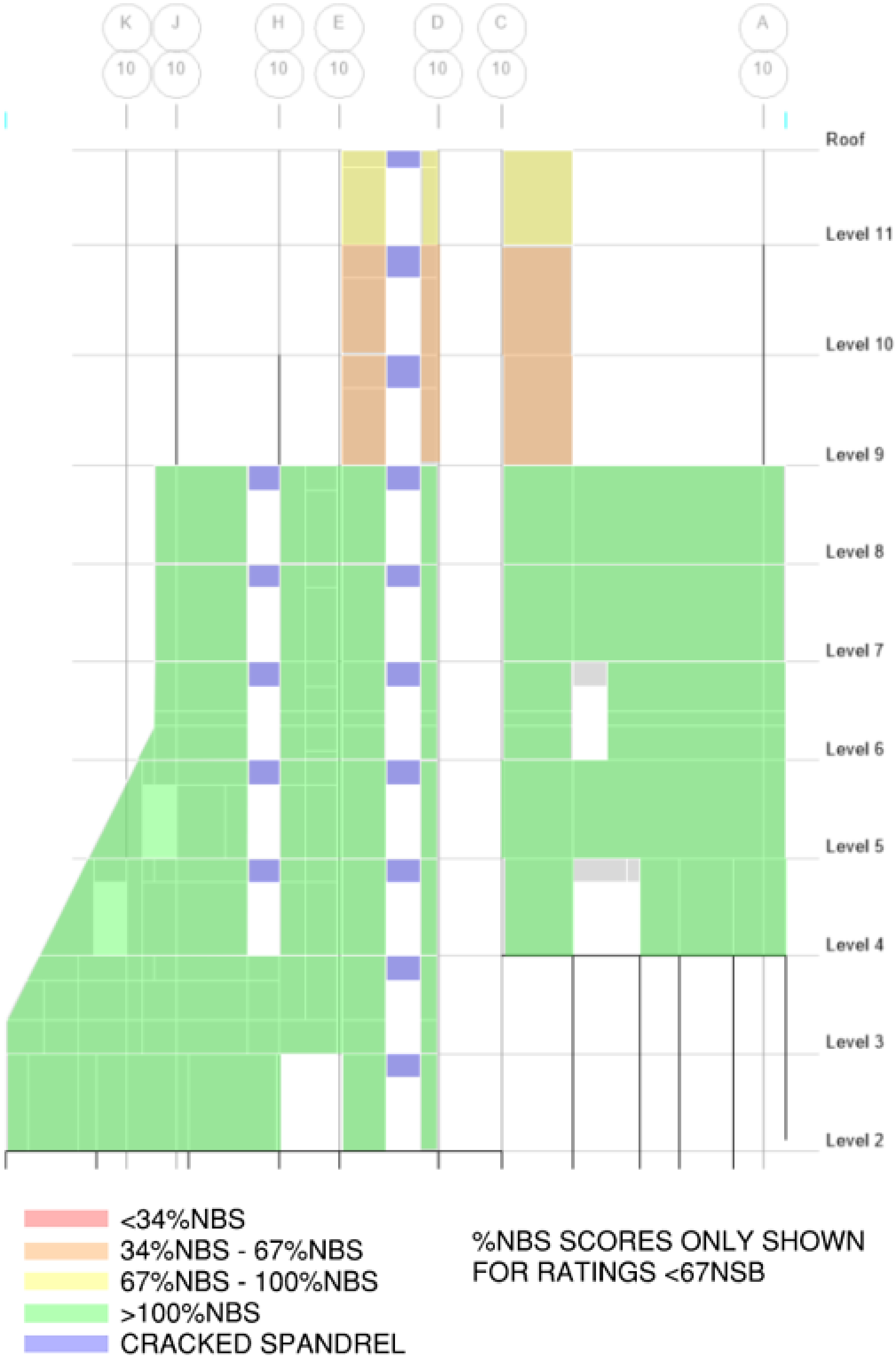

Figure 5-5, Critical Transverse Direction Wall – South Structure

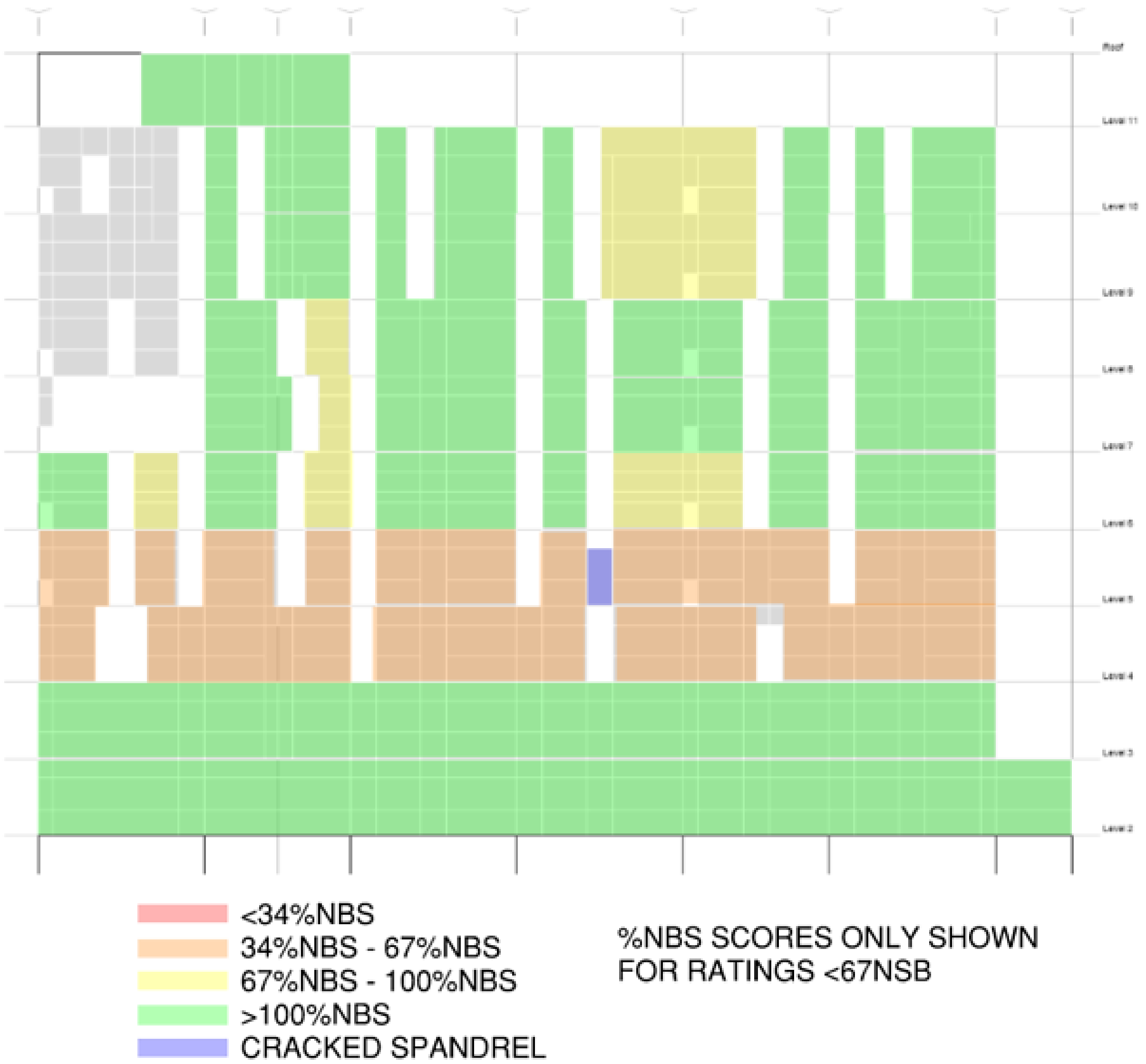

Figure 5-6, Critical Longitudinal Direction Wall – South Structure

5.6 Diaphragms

The diaphragm tension capacity was identified as a structural weakness (SW). The score of the diaphragm is **60%NBS(IL2)** for South building and **55%NBS(IL2**) for North building.

The purpose of a diaphragm is to connect the discrete vertical elements of a structure together in the horizontal plane at regular intervals and be capable of transferring inertia and transfer forces to the lateral elements. The importance and behaviour of diaphragms was often overlooked until the Christchurch Earthquake in 2011, so it is common to find them deficient in older structures. In this building however, the diaphragm is 125mm thick cast in-situ with ductile deformed bars, allowing the diaphragm to stretch and redistribute the load. Also, in general the concrete walls are regularly spaced, which reduces the forces that the diaphragm is required to transfer.

As the number of stories in the building is within the limitation of pESA recommended in NZS1170.5 C5.7.2, the building's acceleration profile was developed using the pESA method. From our analysis, peak ground acceleration (PGA) governs except for few the top levels.

The demand of the diaphragm and the load distribution among the grillage members is affected by:

- Newly added 250mm thick shear wall along grid G added to the existing 8" thick shear wall (Refer to Appendix H)
- Axially stiff spandrel beams on grids A and J.
- Building torsional response and effect of the transfer forces from the levels above.

In our analysis, since the diaphragm was reinforced with the ductile deformed bars, we allowed for the force redistribution. It is noted that the capacity of the bottom layer reinforcement at wall locations are ignored as the lap length for the bottom layer of the reinforcement is significantly less than what is required. This was due to having a poor lap splice detail. **Figure** 5-7 shows the typical detail of the bottom layer lap splice that lacks evidence on the lap length or how the bottom bars are lapped. This was understood as an unreliable load path, and given the uncertainty, we ignored the contribution.

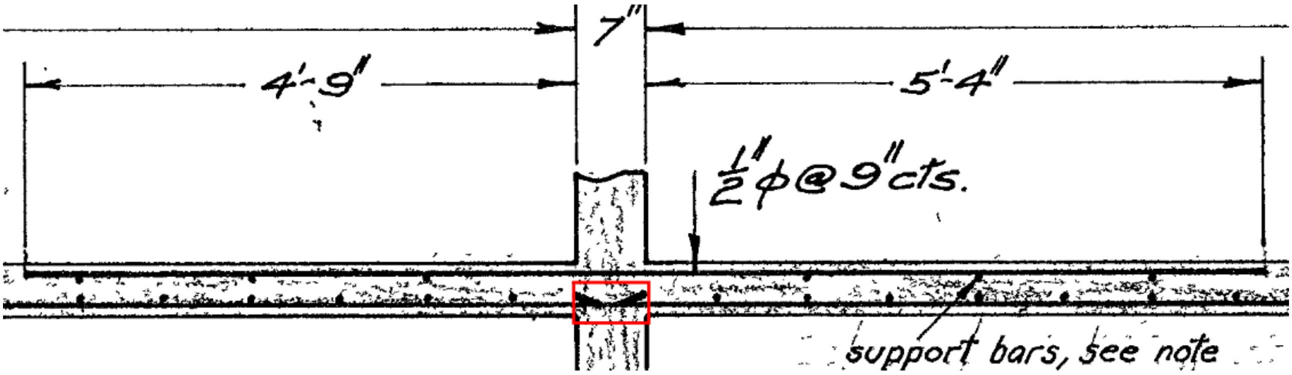

Figure 5-7, Typical bottom reinforcement lap detail in walls

5.6.1 Typical Diaphragm

The diaphragms in the longitudinal direction have insufficient capacity to reliably transfer 100% ULS inertia loads and therefore, was identified as a Structural Weakness. Localised damage around the longitudinal shear walls located in the corners is expected in a severe earthquake. Refer to **Figure** 5-8 for the expected localized damage for the positive earthquake loading in the longitudinal direction. Comparable localized damage is expected for the North building as indicated in **Figure** 5-9 .

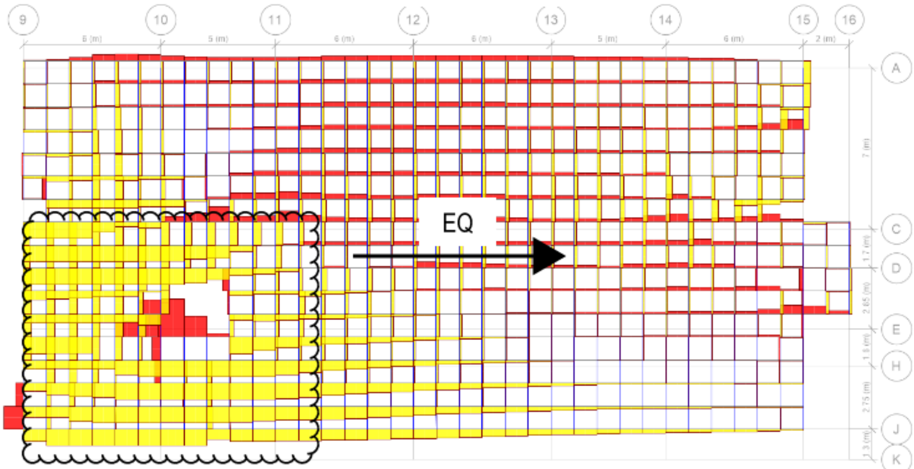

Figure 5-8, Expected Localized Damage around Corner Walls in the South building

(Diagonal Members not Shown for Clarity)

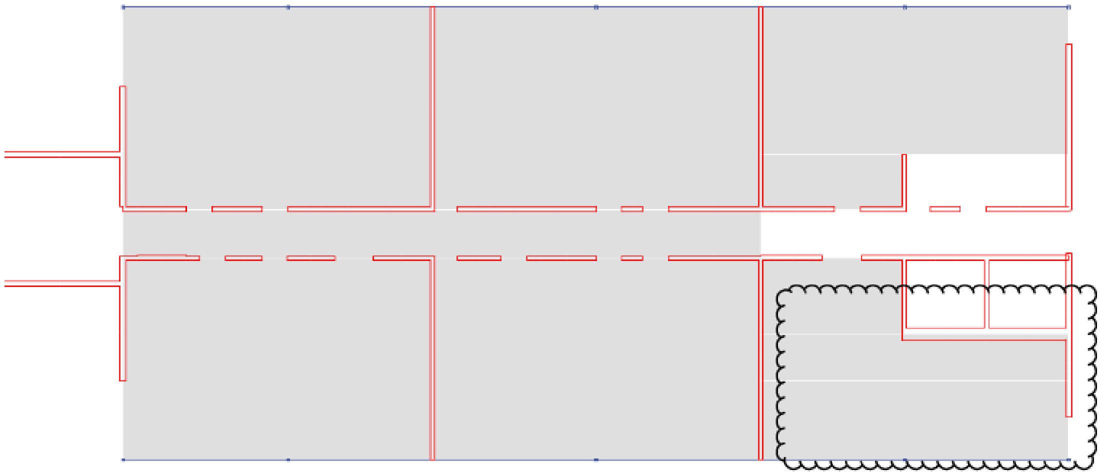

Figure 5-9, Expected Localized Damage around Corner Walls in the North building

5.7 Foundations

The North and South buildings are founded on RC bored piles which are connected together by a grid of RC ground beams at different levels between Ground and Level 4. To assess the foundation, the modes with soil flexibility consideration are referenced (refer to 4.2.1). Ground beams and piles have been identified as **Structural Weakness** (SW) of the structure.

Due to the geometry of the walls and opening in both structures, a number of instances occur in which a wall edge is located between the two piles (at the mid-point of a ground beam)**.** The resulting effect is significant shear forces induced into the ground beams during a seismic event. As the ground beams are typically reinforced with D12 stirrups at 305mm centres they have been determined to have insufficient shear capacity for 67%NBS.

A shear failure is a brittle mechanism which prohibits load redistribution among the foundation elements and where there's no other viable load path, it acts as a fuse, not letting certain piles to contribute to lateral load Accordingly, shear governed ground beams are eliminated as a viable load path and a redistribution of load was considered. Refer to **Figure** 5-10 for the locations where the ground beams which are expected to have a shear failure mechanism.

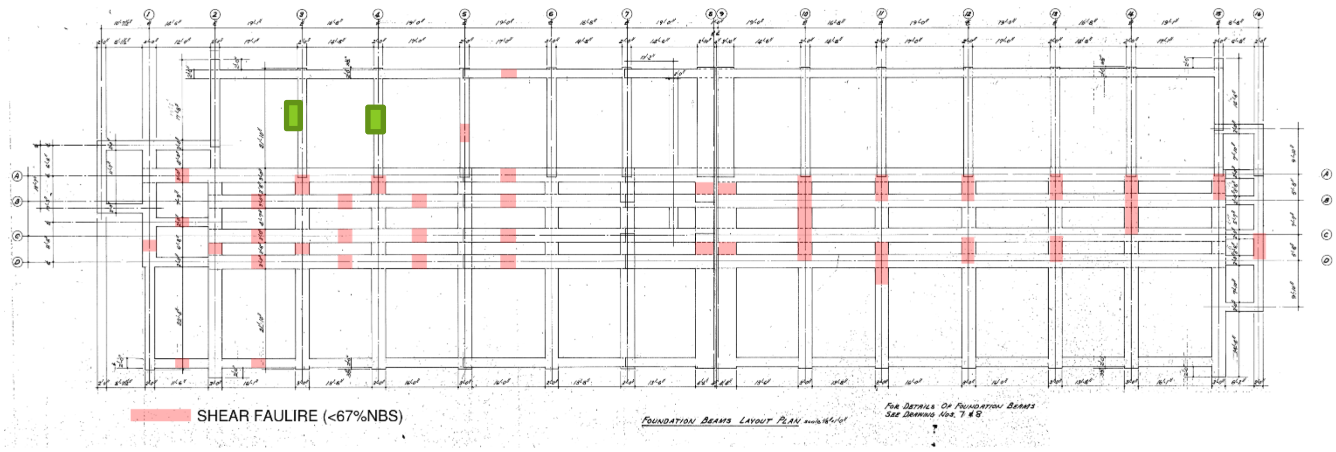

Figure 5-10, Ground Beams with Highlighted Shear Failure Locations (Left: North Building, Right: South Building)

The remaining ground beams are tied into the walls above which are expected provide a combined transfer mechanism to the piles.

The seismic demands at the foundations are predominantly resisted by the lateral capacity of the bored piles with limited additional resistance provided by the ground beams passive pressure. The bored piles were assessed using the established pile information reported in the 2009 geotechnical report, which specified maximum tension, compression, and lateral capacities for the piles.

Similarly, passive soil resistance on the sides of the ground beams was determined to be appropriate due to sufficient soil mobilisation. This provides some additional capacity to the system lateral capacity of the foundation system. It was determined that the foundation system has insufficient base takeout capacity for 67%NBS of the seismic demands during an IL2 event. Refer to Table 5-6 for the summary of the rating for the North and South Building foundation elements.

Table 5-6: %NBS rating for foundation elements

	%NBS(IL2)
North Building	
Longitudinal	**60%**
Transverse	**85%**
South Building	
Longitudinal	**65%**
Transverse	**90%**

5.8 Stairs

The Department of Building and Housing (now MBIE) issued their Practice Advisory 13 in response to concerns about stair collapse and damage observed in the Christchurch earthquake. The primary concern of this Practice Advisory is stairs with sliding support details in mid to high-rise buildings. For these types of stairs, the recommendation is that the stair flights be detailed so that the stairs are free to slide but with sufficient sliding ledge support width available.

The stairs are concrete cast in-situ. The connections of the stairs to the landings are fixed with no allowance for sliding or lateral movement of the building. The flights are cast into the face of the landings with deformed bars.

The stairs were added to the 3D ETABS model to determine how much load they would attract given their proximity to shear walls. The analysis results revealed that the entrance stairs attract some moment and axial demands, even though the stairs are surrounded by RC shear walls. Therefore, the entrance stair does act as an unintentional strut in a design level earthquake. The entrance stair scores 100*%NBS* based on the stairs tension and moment capacity at the stairs knee joint. Refer to **Figure** 5-11 that shows the stair's behaviour during ULS earthquake shaking.

The south and north end of the stairs scores 100% *NBS* (IL2). These stairs are only 1100mm wide and therefore does attract significant seismic load.

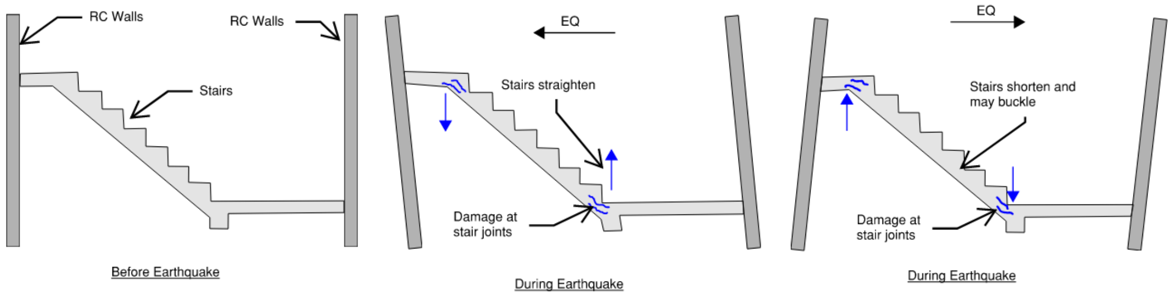

Figure 5-11, Stair's Behaviour During ULS Earthquake Shaking

5.9 Concrete Walls Out-of-plane

The building's concrete walls cantilever up from Level 9 to roof level, providing support for the lightweight roof and ceiling. The RC walls above Level 10 are cantilevering to support the roof system. The rating of the new diaphragm added at level 10 affects this. Following the failure of the diaphragm connections at L10, this cantilever is as high as 5.5m. It is not expected that the roof system will provide restraint of the walls for out-of-plane loading. The walls score 30% NBS (IL2) for out-of-plane seismic loading.

The concrete walls at level 11 are considered cantilevers, as the walls have been assessed based on the roof structure not effectively tying the walls together at high level. The roof structure would need to form a reliable diaphragm to restrain the walls out-of-plane. The existing roof structure, as discussed in the section below, has DHS Purlins connected to the timber blocking and the blocking is bolted to the concrete walls, as shown in **Figure** 5-13. Existing timber floor structure has anchored to the existing concrete walls, as shown in **Figure** 5-14. Two options were shown and option 1 of the timber floor connections were assessed at this stage.

The walls score 30%*NBS*(IL2) out-of-plane seismic parts loading. We note that if these walls were restrained at roof level and level 10, they would score greater than 67%NBS (IL2).

Refer to **Figure** 5-12 that shows the RC shear wall stress distribution based on out-of-plane seismic parts loading.

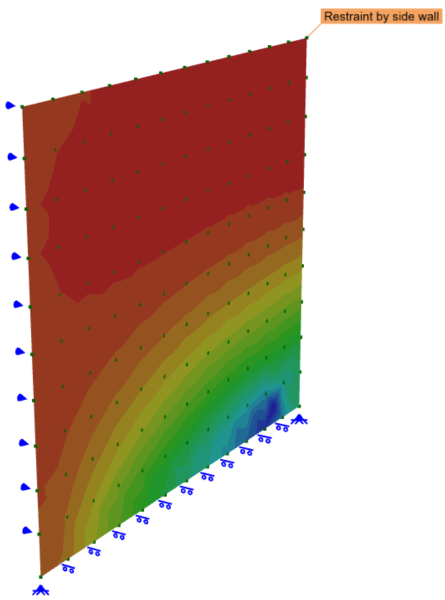

Figure 5-12, 6" RC Shear Wall Stress Distribution Based on Out-of-plane Seismic Parts Loading

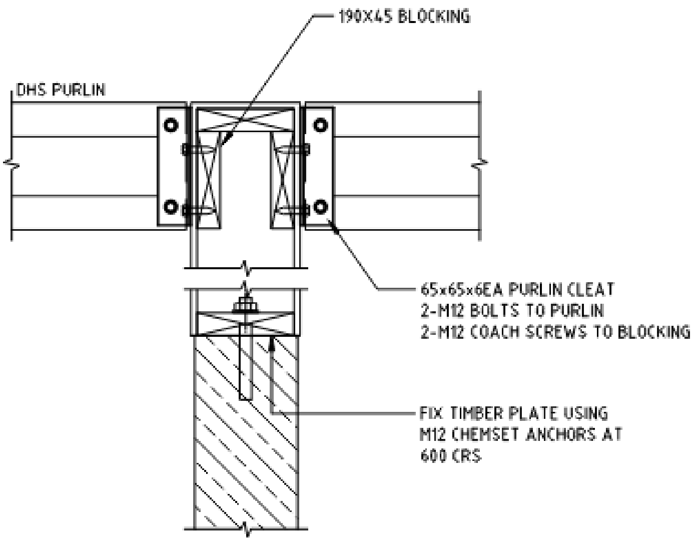

Figure 5-13, Typical DHS Purlins to Timber Wall to Concrete Wall Connections

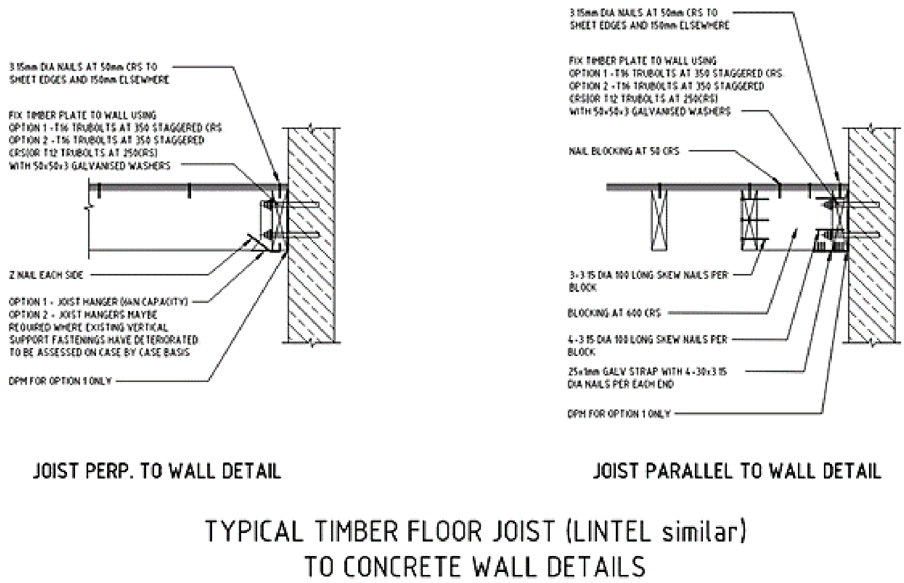

Figure 5-14 , Typical Timber Floor Joist to Concrete Wall Connection Details

The building's roof comprises of DHS Purlins spanning in the building's transverse direction between concrete walls. The roof consists of Dimondeck colorsteel sheeting and DHS purlins, but they are not considered to form an effective diaphragm to transfer the lateral loads into the shear walls. The roof joists have been assessed based on tributary area, and therefore are required to bend out of plane to resist lateral loads in the longitudinal direction of the building.

The timber diaphragm located at level 10 is connected to concrete walls, however, the connections do not possess sufficient capacity to resist the ultimate limit state (ULS) seismic actions. The diaphragm has a seismic capacity score of <34% of the New Building Standard (NBS) . The connections between the timber floor and the concrete walls were made using T16 Trubolts at 350mm spacing withan embedment depth of 90mm. These anchors do not comply with the current C2 requirements of the current concrete code. In the event of a failure of these connections, the unsupported length of the 6-inch concrete walls in the out-of-plane (OOP) direction will increase, spanning from level 9. The joists are connected to a timber end plate running along the concrete shear walls. Further site investigation is recommended to confirm the type of the connection and their spacing. It is important to note that the possible failure of these connections is closely related to the score of the concrete walls out-of-plane.

5.10 Other Secondary Structural Elements

The reinforced concrete water tank located at the roof level has achieved a seismic performance score of 50%NBS(IL2), based on its walls' out-of-plane capacity when subjected to seismic loading using AS/1170.5 Part coefficient. The assessment of the tank was conducted under the assumption that it would be filled with liquid during a seismic event. It is expected that the water tank will experience two-way out-of-plane bending, resulting in cracking and potential leaks. However, it is not anticipated that these issues will pose a life safety risk.

Two canopies, West and East, were analysed for their seismic performance. The East canopy has a seismic score of 100%NBS(IL2). However, the West canopy has a seismic score >67% for ULS seismic actions. The potential failure modes for the West canopy are concrete cone failure and splitting failure. It is important to note that the severity of the failure will depend on the magnitude and duration of the seismic event. In a seismic event, uplift would occur and could cause the structure to move or shift, which can in turn cause the connections between the structure and its supporting wall structure below to become overstressed, thus lead to concrete cone failure and splitting failure. Concrete cone failure occurs when the force of the uplift is concentrated on a small area of the concrete, causing it to deform and eventually break off in the shape of a cone. Splitting failure, on the other hand, occurs when the force of the uplift causes the concrete to split or crack along its length. However, it is not anticipated that these issues will pose a risk to life safety.

6 Strengthening

We recommend that the building is seismically strengthened considering a two-stage approach. Stage 1 would be to strengthen the building to a minimum seismic rating of greater than **34*%NBS*(IL2).** Based on our review, the seismic strengthening, to achieve greater than 34*%NBS*(IL2), would include:

- Increase the RC walls out-of-plane capacity by installing a new roof diaphragm with new connections to the concrete walls. The roof diaphragm can be in the form of steel cross braces and steel beams. The timber floor at level 10 would need to be strengthened with new connections to the concrete walls.
- Strengthening of the connections between Level 10 and Roof to wall elements by replacing the connections with code-compliant anchors or bolt-through connections. It is recommended to confirm the as-built connections via site investigations.

We recommend strengthening the above elements to minimum 67%NBS(IL2) if the final objective is to achieve this rating.

Stage 2 would be to seismically strengthen the building to a minimum rating of 67*%NBS* (IL2). Based on our review, the seismic strengthening to achieve 67%*NBS*(IL2) would include:

- Strengthening of the diaphragms using steel plates. **Further Investigations** may eliminate or reduce the scope of the strengthening for diaphragms.
- Increase the RC walls in-plane performance through the separation of bond elements causing coupling effects and inducing tension forces. Additional localised skin strengthening of wall members with insufficient capacity with reference to the sections of the wall with low %NBS ratings in Appendix H. where the walls are required to be strengthened, the strengthening is to be extended all the way down to the foundation.
- .
- Increasing the capacity of the foundation by strengthening/adding ground beam elements and utilising higher passive pressure capacities. As currently foundation elements are rated 60 and 65 %NBS(IL2) for the North and South Buildings respectively, further geotechnical investigation may help to improve the current ratings and eliminate the foundation strengthening requirement to achieve 67 %NBS(IL2).

The strengthening options recommended are only at a descriptive level and a detailed design will be required for Building Consent and construction documents. We envisage that the strengthening work would be completed in stages (i.e., floor by floor or groups of floors) to minimise occupant disruption. We note that the noise due to drilling and other construction activities will have impact on the building occupants.

We also recommend that part of any seismic upgrade or future fitout that the non-structural building elements (façade glass, ceilings, internal walls, overhead services and plant and equipment etc) is seismically restrained to meet the current standards.

We further recommend that in designing any seismic retrofit that the building owner should also consider the proposed increase in seismic hazard levels in Wellington. This would insulate the building against further future reductions in the seismic rating.

7 Future Code Changes

7.1 Hazard Zone Factor

The results of the updated National Seismic Hazard Model (NSHM) were released in October 2022. The previous update to the NSHM was in 2010. Since then, the science behind estimating earthquake rates and understanding and complexity of ground motion modelling have significantly advanced.

The NZSM provides the basis for setting the seismic demands in the design code NZS1170.5. Although the results are not a design standard or design loadings standard, they provide an indication of how the code may reflect the updated seismic hazard in future revisions. A possible outcome of this review will be an increase in the hazard zone factor, Z, for the Wellington region. This factor is used to determine the seismic risk for the area and hence the design standard for new buildings.

A future increase in the Hazard Factor will lead to an increase in the design level for new buildings in Wellington and potentially increase the standard required for existing buildings to achieve 100%NBS when assessed against that new standard.

7.2 Basin Edge Effects

The 2016 Kaikōura earthquake exposed the concept of the “basin edge effects.” The basin edge efforts cause amplification of ground shaking due to the presence of soft soils in the sedimentary basin and cause larger peak ground accelerations than expected. The edge efforts are currently not incorporated in the Earthquake actions design code NZS 1170.5.

The basin edge effects have the potential to significantly increase the design standard for new buildings in particular locations in Wellington and potentially may increase the standard required for existing buildings to achieve 100%NBS (IL2) when assessed against that new standard. The “basin edge effects” is currently being discussed and reviewed by industry experts with no fixed timeframe when it will be introduced into the design standards. We note that the Hanson St housing complex location is less likely to be impacted by basin edge effects than other sites in the Wellington.

7.3 Seismic Guidelines

The **Yellow Chapter**, dated November 2018, provides the latest engineering knowledge on aspects involved in the assessment of concrete buildings, and to reflect what engineers learned from the Kaikōura earthquake.

However, its impact to the industry to still being assessed before it can be incorporated into regulation. Therefore, some aspects of the Guidelines may potentially change and hence affect the standard required for existing buildings to achieve 100%*NBS*(IL2).

8 Conclusions and Recommendations

8.1 Conclusion

The results of the DSA indicate the buildings' earthquake rating to be **30*% NBS*(IL2)** in accordance with **The Guidelines***.* This rating is based on the Critical Structural Weakness (CSW) of RC walls out-of-plane capacity at the roof level and Level 10 to resist seismic parts loading and the capacity of the roof and Level 10 diaphragm connections to the shear walls. The buildings also contain other distinct elements that are classified as structural weaknesses.

8.2 Recommendations

To achieve a minimum rating of **67*%NBS*(IL2),** we consider the building structure must be seismically strengthened. The seismic retrofit would include strengthening elements as described in **Section 6.**

We further recommend that in designing any seismic retrofit that the building owner should also consider the proposed increase in seismic hazard levels in Wellington. This would insulate the building against further future reductions in the seismic rating.

9 Explanatory Notes

- The information contained in this report has been prepared by Aurecon at the request of Wellington City Council and is exclusively for Wellington City Council's use and reliance. It is not possible to make a proper assessment of this review without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to and the assumptions made by Aurecon. The report will not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. Aurecon accepts no responsibility or liability to any third party for any loss or damage whatsoever arising out of the use of or reliance on this report by that party or any party other than our "Client".
- This report contains the professional opinion of Aurecon as to the matters set out herein, in the light of the information available to it during preparation, using its professional judgment and acting in accordance with the standard of care and skill usually exercised by professional engineers providing similar services in similar circumstances. Aurecon is not able to give any warranty or guarantee that all possible damage, defects, conditions, or qualities have been identified.
- The report is based on information that has been provided to Aurecon from other sources or by other parties. The report has been prepared strictly on the basis that the information that has been provided is accurate, complete, and adequate, except where otherwise identified during site investigation inspections. To the extent that any information is inaccurate, incomplete, or inadequate, Aurecon takes no responsibility and disclaims all liability whatsoever for any loss or damage that results from any conclusions based on information that has been provided to Aurecon.
- The inspections of the building discussed in this report have been undertaken to inspect the structure and confirm the adequacy of the existing drawings. This report does not address building defects. Where site inspections were undertaken, they were restricted to visual inspections with intent to determine existing building main structural elements only.
- We have not undertaken a review of secondary elements such as ceilings, building services, plant, and partitions.

A

Appendix A - Definitions and Acronyms

Definitions and Acronyms

ADRS	Acceleration-displacement response spectrum
Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable yield capacity is exceeded. A brittle structure has little tendency to deform before it fractures.
Critical Structural Weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential CSWs.
Damping	The value of equivalent viscous damping corresponding to the energy dissipated by the structure, or its systems and elements, during the earthquake. It is generally used in nonlinear assessment procedures. For elastic procedures, a constant 5% damping as per NZS 1170.5:2004 is used.
Design Level or ULS earthquake	Design level earthquake or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004 (refer to Section C3)
Detailed Seismic Assessment (DSA)	A seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor or ceiling or a braced roof structure) that is strongly connected to the vertical elements around it and that distributes earthquake lateral forces to vertical elements, such as walls, of the primary lateral system. Diaphragms can be classified as flexible or rigid.
Ductile or Ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Elastic Analysis	Structural analysis technique that relies on linear-elastic assumptions and maintains the use of linear stress-strain and force-displacement relationships. Implicit material nonlinearity (e.g. cracked section) and geometric nonlinearity may be included. Includes equivalent static analysis and modal response spectrum dynamic analysis.
Flexible diaphragm	A diaphragm which for practical purposes is considered so flexible that it is unable to transfer the earthquake loads to shear walls even if the floors/roof are well connected to the walls. Floors and roofs constructed of timber, and/or steel bracing in a URM building, or precast concrete without reinforced concrete topping fall in this category. A diaphragm with a maximum horizontal deformation along its length that is greater than or equal to twice the average inter-storey drift. In a URM building a diaphragm constructed of timber and/or steel bracing.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.

Nonlinear analysis	Structural analysis technique that incorporates the material nonlinearity (strength, stiffness and hysteretic behaviour) as part of the analysis. Includes nonlinear static (pushover) analysis and nonlinear time history dynamic analysis.
Non-structural item	An item within the building that is not considered to be part of either the primary or secondary structure. Non-structural items such as individual window glazing, ceilings, general building services and building contents are not typically included in the assessment of the building's earthquake rating.
OTM	Overturning moment.
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also incorporate the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Rigid diaphragm	A diaphragm that is not a flexible diaphragm
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but, nevertheless, is required to transfer inertial and vertical loads for which assessment/design by a structural engineer would be expected. Includes precast panels, curtain wall framing systems, stairs and supports to significant building services items
Serviceability limit state (SLS)	Limit state as defined in AS/NZS 1170.0:2002 (or NZS 4203:1992) being the point at which the structure can no longer be used as originally intended without repair
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge
Simple Lateral Mechanism Analysis (SlaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (pushover) relationship for the building as a whole
Single-degree-of- freedom (SDOF)	A simple inverted pendulum system with a single mass
Structural element	Combinations of structural members that can be considered to work together; e.g. the piers and spandrels in a penetrated wall, or beams and columns in a moment resisting frame

Structural member	Individual items of a building structure, e.g. beams, columns, beam-column joints, walls, spandrels, piers
Structural sub-system	Combination of structural elements that form a recognisable means of lateral or gravity load support for a portion of the building: e.g. moment resisting frame, frame/wall. The combination of all of the sub-systems creates the structural system.
Structural system	Combinations of structural elements that form a recognisable means of lateral or gravity load support; e.g. moment resisting frame, frame/wall. Also used to describe the way in which support/restraint is provided by the foundation soils.
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100%NBS. Note that an aspect of the building structure scoring less than 100%NBS but greater than or equal to 67%NBS is still considered to be a SW even though it is considered to represent an acceptable risk.
Ultimate Limit State (seismic)	A term defined in regulations that describes the limiting capacity of a building for it to be determined to be an earthquake-prone building. This is typically taken as the probable capacity but with the additional requirement that exceeding the probable capacity must be associated with the loss of gravity support (i.e. creates a significant life safety hazard).
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings.
XXX%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. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
XXX%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

B

Appendix B – Assessment Inputs

Assessment Inputs

Structural Layout

The building layout, member sizes, detailing and material grades have been taken from available design drawings and calculations. A site inspection of the interior and exterior was carried out to confirm that the drawings and documentation was generally in accordance with the as-built configuration. The following drawing documentation was available at the time of the assessment:

- Existing Structural drawings titled "BERKELEY_DALLARD_ORIGINAL_DRAWINGS_STRUCTURAL" dated 1974.
- Strengthening Structural drawings titled "BERKELEY_DALLARD_STRENGTHENING_DRAWINGS_ STRUCTURAL" dated 2013.
- Existing Architectural Drawings titled "BERKELEY_DALLARD_APARTMENTS" by CCM Architects dated 2013.

Dead, Superimposed Dead Loads and Live Loads.

See Table Below for the Dead, Superimposed dead loads and Live Loads used in the assessment. The self-weight of the walls, stairs and slabs are calculated by the structural analysis program based on the input section size and unit weight. The design live loads were adopted as indicated as per structural drawings and in accordance with NZS1170.1 loading.

Table: Dead, Superimposed dead loads and Live Loads used in the assessment

Load Type	Load
Dead Load	Calculated by the structural analysis program based on the input section size and unit weight
Super Imposed Dead Load	0.5 kPa – Level 11 to Roof 0.85kPa – Level 2 to Level 10 0.35kPa – Stairs and Corridors 0.5kPa – Façade
Live Load	0.25kPa for inaccessible roof 5kPa for plantroom 1.5kPa for apartment levels 2kPa Stairs and Corridors

Seismic Weight

The seismic mass was calculated based on the NZS 1170.5:2004 loading combination W = G + ΨEQu, where ΨE = 0.0 for roof. Where applicable, an area reduction factor was also applied to the live load in accordance with clause 3.4.2 of AS/NZS 1170.1:2002.

Wind Loads

Consideration of wind loads is outside the scope of this assessment.

Seismic loading

The seismic loads were determined in accordance with NZS1170.5 with the following parameters.

Table: Seismic parameters for building assessments

Parameter	Value
Design Working Life	50
Importance level	2
Site Subsoil Classification	C
Hazard Factor (Z)	0.4

Material Properties

The following material properties and corresponding characteristic and probable strengths were used as per the Assessment Guideline Tables C5.3, C5.4 and Section C6. No material specification regarding the concrete and steel used at the time was found in the structural drawings. No physical materials testing has been undertaken to validate the assumed material properties.

Table: Material properties

Item	Characteristic Design Strength (MPa)	Assessment (Probable) Strength (MPa)
Concrete	20 MPa	30 MPa
Structural Steel	275 MPa	324 MPa

Geotechnical Parameters

The geotechnical parameters are taken form the Geotechnical Report for Berkely Dallard Apartment dated 15/10/09.

C

Appendix C – Importance Level Description

Importance Level Description

Importance Levels for Building Types – New Zealand Structures

Importance Level:	Comment:	Example:
1	Structures presenting a low degree of hazard to life and other property	Structures with a total floor area of <30 m2 Farm buildings, isolated structures, towers in rural situations Fences, masts, walls, in-ground swimming pools
2	Normal structures and structures not in other importance levels	Buildings not included in Importance Levels 1, 3 or 4 Single family dwellings and Car parking buildings
3	Structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds	Buildings and facilities as follows: a) Where more than 300 people can congregate in one area b) Day care facilities with a capacity greater than150 c) Primary school or secondary school facilities with a capacity greater than 250 d) Colleges or adult education facilities with a capacity greater than 500 e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities f) Airport terminals, principal railway stations with a capacity greater than 250 g) Correctional institutions h) Multi-occupancy residential, commercial (including shops), industrial office and retailing buildings designed to accommodate more than 5000 people and with a gross area greater than 10 000m2 i) Public assembly buildings, theatres and cinemas of greater than 1000m2 Emergency medical and other emergency facilities not designated as post-disaster Power-generating facilities, water treatment and wastewater treatment facilities and other public utilities not designated as post-disaster Buildings and facilities not designated as post-disaster containing hazardous materials capable of causing hazardous conditions that do not extend beyond the property boundaries

4	Structures with special post-disaster functions	Buildings and facilities designated as essential facilities Buildings and facilities with special post-disaster function Medical emergency or surgical facilities Emergency service facilities such as fire, police stations and emergency vehicle garages Utilities or emergency supplies or installations required as backup for buildings and facilities of Importance Level 4 Designated emergency shelters, designated emergency centres and ancillary facilities Buildings and facilities containing hazardous materials capable of causing hazardous conditions that extend beyond the property boundaries
5	Special structures (outside the scope of this Standard-acceptable probability of failure to be determined by special study)	Structures that have special functions or whose failure poses catastrophic risk to a large area (e.g. 100 km2) or a large number of people (e.g., 100 000) Major dams, extreme hazard facilities

Annual Probability of Exceedance

Design Working Life:	Importance Level:	Annual probability of exceedance for ultimate limit states			Annual probability of exceedance for serviceability limit states	
		Wind	**Snow**	**Earthquake**	**SLS1**	**SLS2 Importance level 4 only**
Construction equipment	2	1/100	1/50	1/100	1/25	-
Less than 6 months	1 2 3 4	1/25 1/100 1/250 1/1000	1/25 1/50 1/100 1/250	1/25 1/100 1/250 1/1000	- 1/25 1/25 1/25	-
5 years	1 2 3 4	1/25 1/250 1/500 1/1000	1/25 1/50 1/100 1/250	1/25 1/250 1/500 1/1000	- 1/25 1/25 1/25	-
25 years	1 2 3 4	1/50 1/250 1/500 1/1000	1/25 1/50 1/100 1/250	1/50 1/250 1/500 1/1000	- 1/25 1/25 1/25	- - - 1/250
50 years	1 2 3 4	1/100 1/500 1/1000 1/2500	1/50 1/150 1/250 1/500	1/100 1/500 1/1000 1/2500	- 1/25 1/25 1/25	- - - 1/500
100 years or more	1 2 3 4	1/250 1/1000 1/2500 *	1/150 1/250 1/500 *	1/250 1/1000 1/2500 *	- 1/25 1/25 1/25	- - - *

D

Appendix D – Assessment Summary

Assessment Summary

1. Building Information	
Building Name/ Description:	Berkeley Dallard Apartments
Street Address	46 Nairn Street, Mt Cook, Wellington
Territorial Authority	Wellington City Council
No. of Storeys	11 Stories North Building and 9 Stories South Building
	Approximately 575m^2
Year of Design (approx.)	1974-5
NZ Standards designed to	N/A
Structural System including Foundations	Lateral system consists of RC shear walls, spandrels, and piers. Foundation system is bored piles tied with the ground beams
Does the building comprise a shared structural form or shares structural elements with any other adjacent titles?	No
Key features of ground profile and identified geohazards	The site subsoil classification, in terms of NZS1170.5:2004 Clause 3.1.3, is Class C.
Previous strengthening and/ or significant alteration	2013 strengthening of RC walls, Alterations of L10 and Roof by Aurecon
Heritage Issues/ Status	N/A
Other Relevant Information	N/A

2. Assessment Information	
Consulting Practice	Aurecon NZ Ltd
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings	s(7)(2)(a) ▪ I have more than 25 years of experience in design management, structural design, assessment, and construction monitoring of low and medium rise buildings.
Documentation reviewed, including: • date/ version of drawings/ calculations • previous seismic assessments	▪ Existing Structural drawings titled "*Central Park Flats, Stage Two, High Rise*" dated 1975 ▪ Existing Structural Drawing titled *"WCC Housing Upgrade, Berkeley Dallard Strengthening"* by Aurecon dated 2013 ▪ Existing Architectural Drawings titled "*Berkeley Dallard Apartments"* by CCM Architects dated 2013.
Geotechnical Report(s)	Geotechnical Report for Berkely Dallard Apartment dated 15/10/09. Geotechnical desktop study Appendix G
Date(s) Building Inspected and extent of inspection	10/2023 Visual internal and external. Where the elements are rated below 34%NBS(IL2) further intrusive investigation were carried out.
Description of any structural testing undertaken and results summary	N/A
Previous Assessment Reports	2009-13 Aurecon DSA report.
Other Relevant Information	N/A

<table>
<tr><th colspan="2">3. Summary of Engineering Assessment Methodology and Key Parameters Used</th></tr>
<tr><td>Occupancy Type(s) and Importance Level</td><td>2</td></tr>
<tr><td>Site Subsoil Class</td><td>C</td></tr>
<tr><td><u>For a DSA:</u>
Summary of how Part C was applied, including:
• the analysis methodology(s) used from C2
• other sections of Part C applied</td><td>SLaMA, Nonlinear Static and response Spectrum Analyses
The DSA was generally completed in accordance The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments, dated July 2017 (Red Book), including the updated Section C5 – Concrete Buildings – Proposed Revision to the Engineering Assessment Guidelines, dated November 2018 (the Yellow Chapter).</td></tr>
<tr><td>Other Relevant Information</td><td>N/A</td></tr>
</table>

<table>
<tr><th colspan="3">4. Assessment Outcomes</th></tr>
<tr><td>Assessment Status</td><td colspan="2">Final</td></tr>
<tr><td>Assessed %NBS Rating</td><td colspan="2">30%</td></tr>
<tr><td><u>For a DSA:</u></td><td colspan="2"></td></tr>
<tr><td>Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed</td><td colspan="2">Non-structural elements have not been assessed at this stage.</td></tr>
<tr><td>Describe the Governing Critical Structural Weakness</td><td colspan="2">RC Shear Wall flexural Capacity</td></tr>
<tr><td>If the results of this DSA are being used for earthquake prone decision purposes, <u>and</u> elements rating <34%NBS have been identified (including Parts):</td><td><u>Engineering Statement of Structural Weaknesses and Location:</u>
▪ RC out-of-plane capacity
▪ Roof and L10 capacity</td><td><u>Mode of Failure and Physical Consequence Statement(s):</u>

The mode of failure is the exceedance of out-of-plane flexural capacity in the singly reinforced walls on the two upper levels under seismic loading. This failure results in excessive out-of-plane displacement of the walls and loss of connections to the roof and timber diaphragm.
Once the walls' capacity is exceeded, and the earthquake changes direction in due course, requiring the walls to resist in-plane loading, there is no lateral stiffness or strength left to counter the in-plane forces. This lack of resistance causes the roof to become unstable, leading to excessive displacements. Once again, these displacements can result in the roof losing gravity support, creating a life safety hazard."</td></tr>
<tr><td>Recommendations
(Optional for EPB purposes)</td><td colspan="2">Strengthening should be undertaken to increase the structure’s rating to a minimum of 67%NBS(IL2) if feasible.</td></tr>
</table>

E

Appendix E – Building Photographs

Figure E-1, East Elevation

Figure E-2, North Elevation

Figure E-3, West Elevation

Figure E-4, Cast-in-Situ Stairs

F

Appendix F – Sample Building Plans

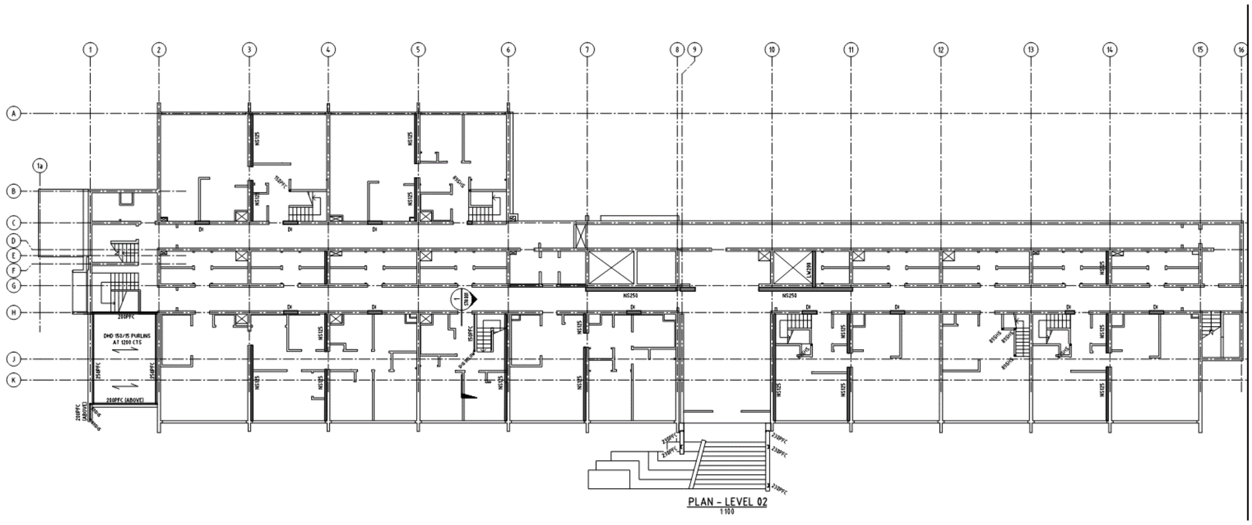

Figure F-1, Plan View L02

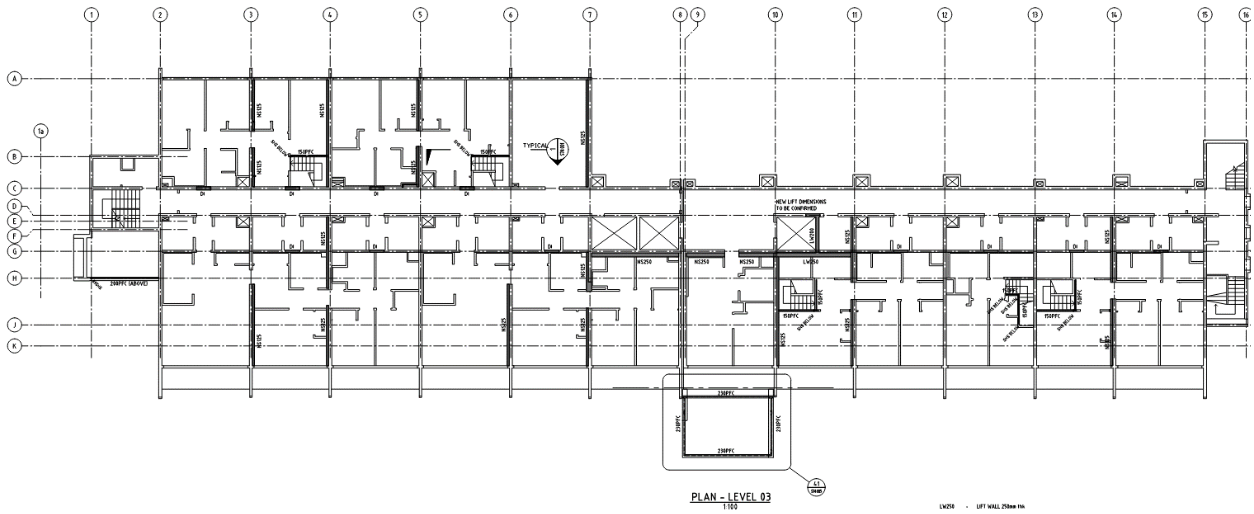

Figure F-2, Plan View L03

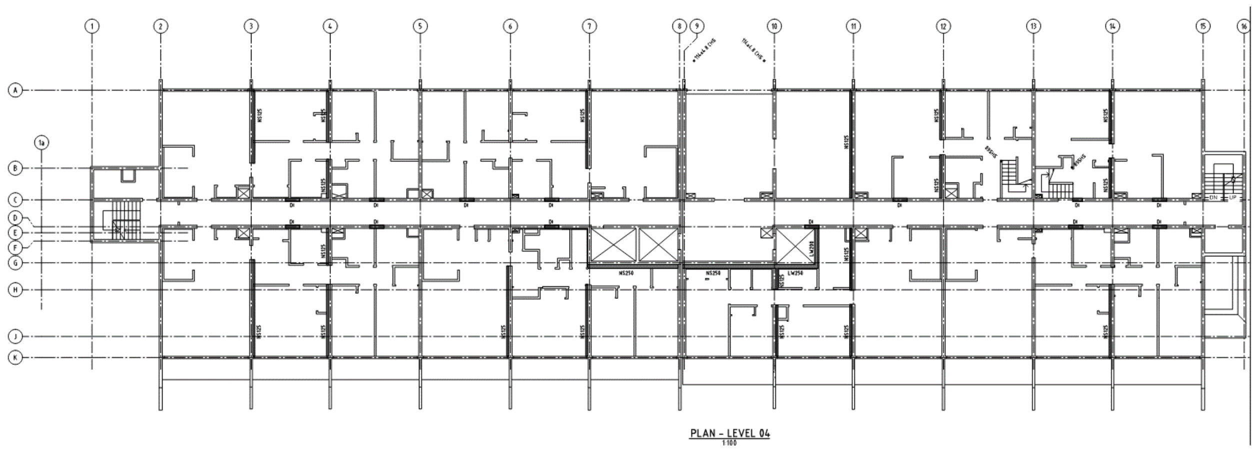

Figure F-3, Plan View L04

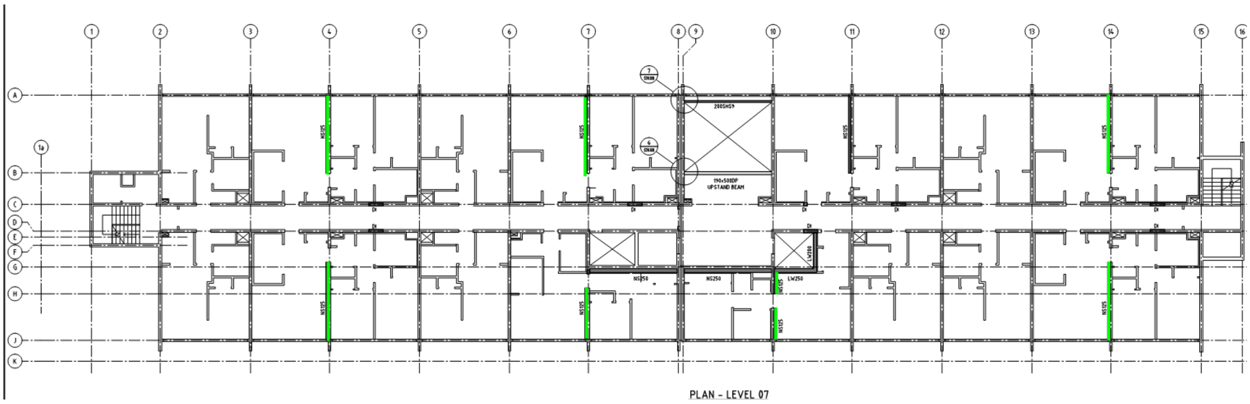

Figure F-4, Typical Plan View for L05-L08

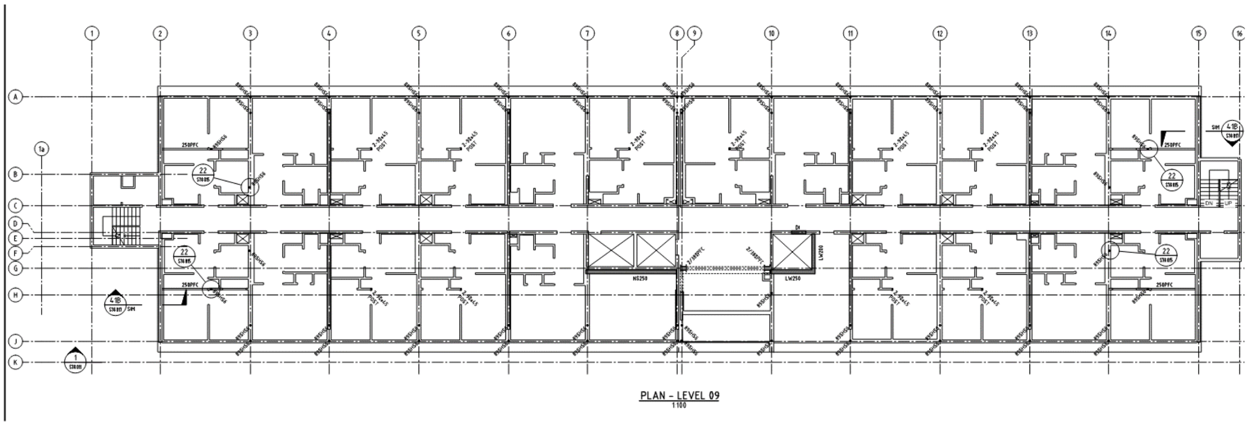

Figure F-5, Plan View for L09

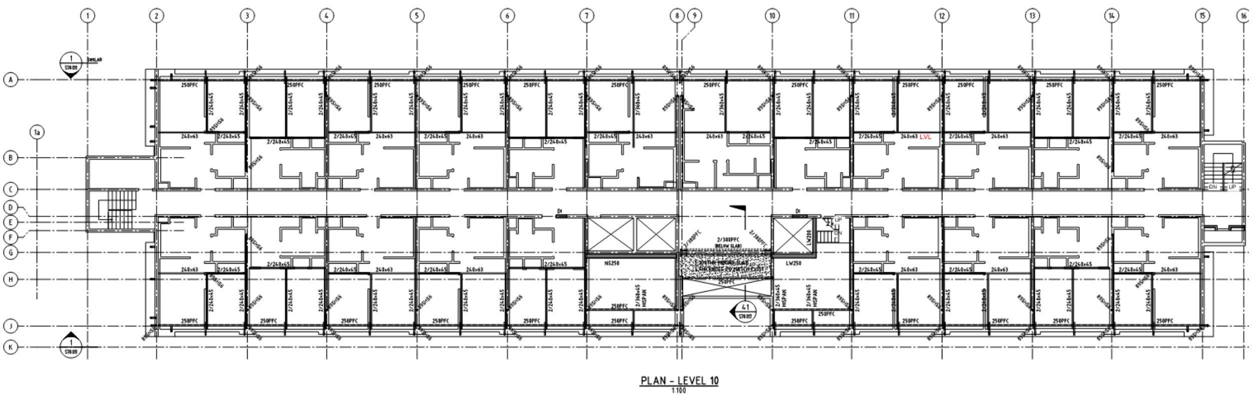

Figure F-6, Plan View for L10

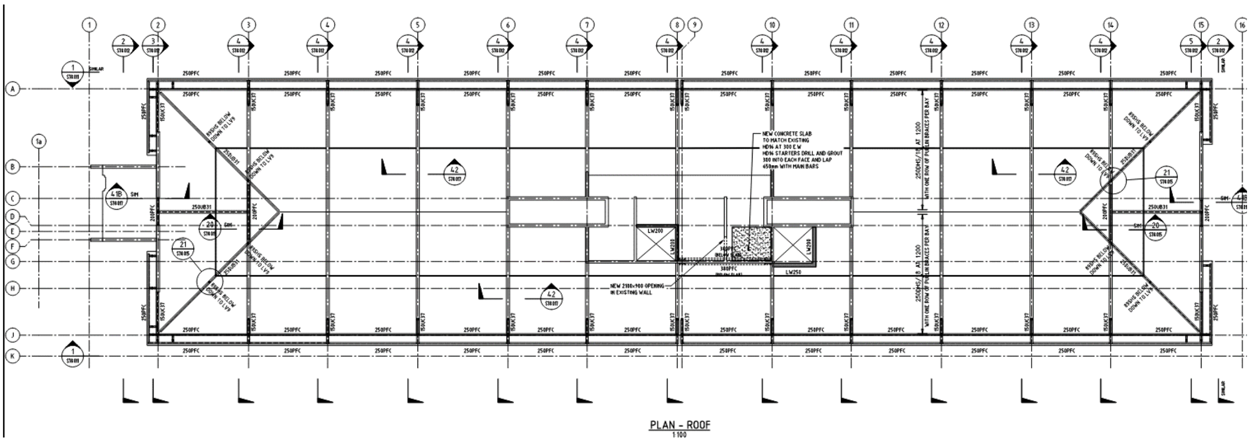

Figure F-7, Plan View for Roof

G

Appendix G – Geotechnical Report

H

Appendix H – Critical Walls In-Plane %NBS Rating

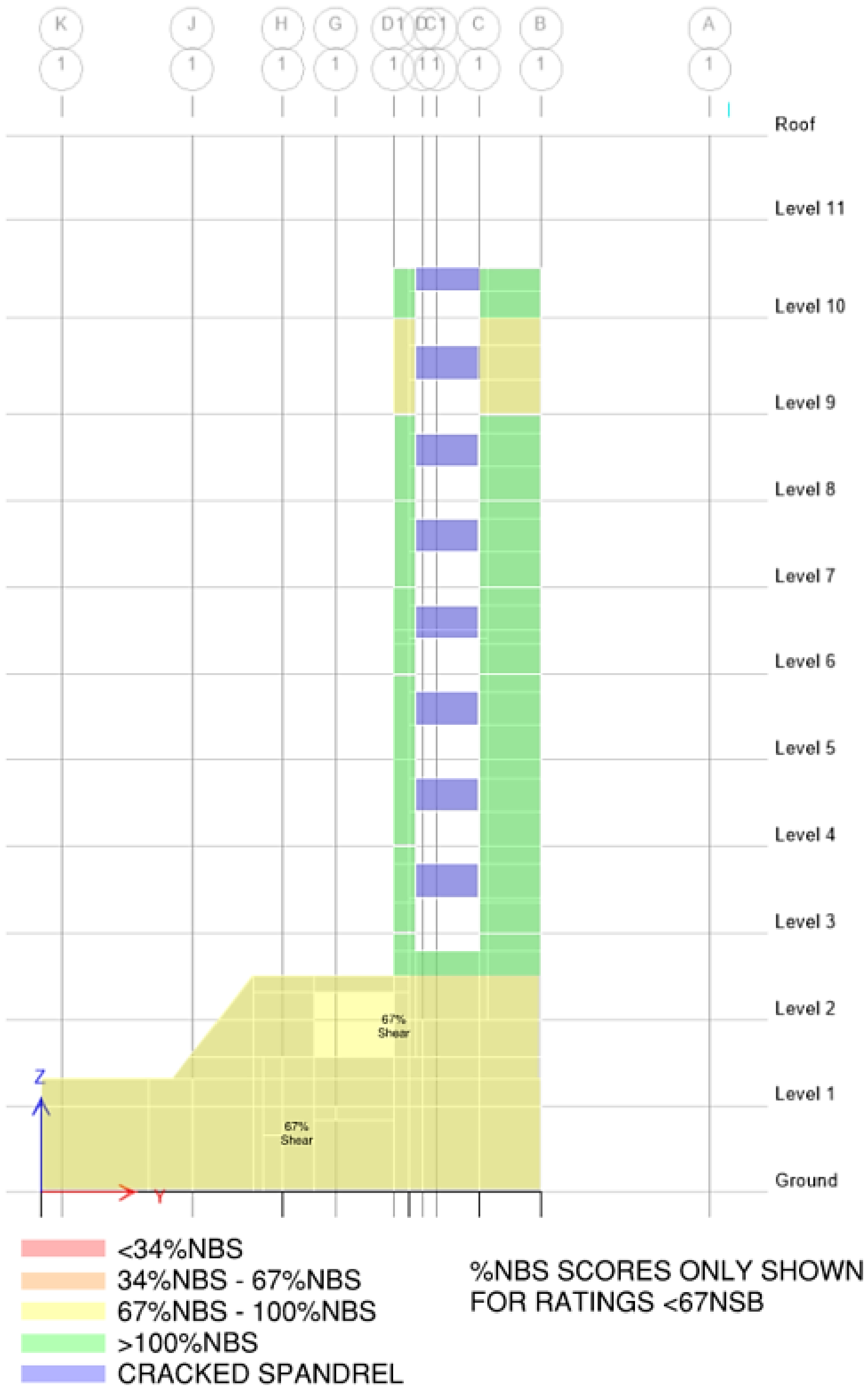

Figure H-1, Gridline 1 Transverse Direction Wall – North Structure

Figure H-2, Gridline 2 Transverse Direction Wall – North Structure

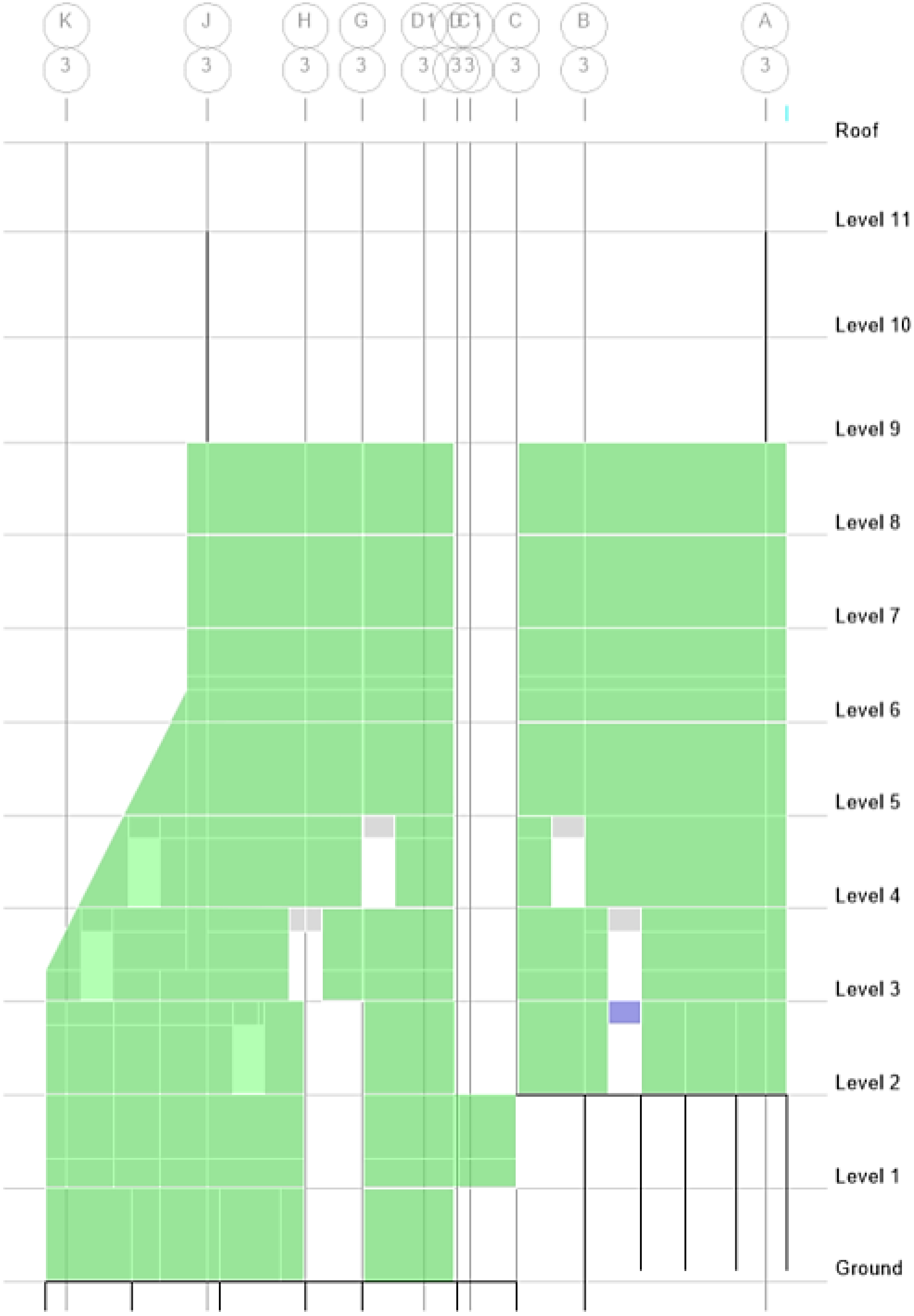

Figure H-3, Gridline 3 Transverse Direction Wall – North Structure

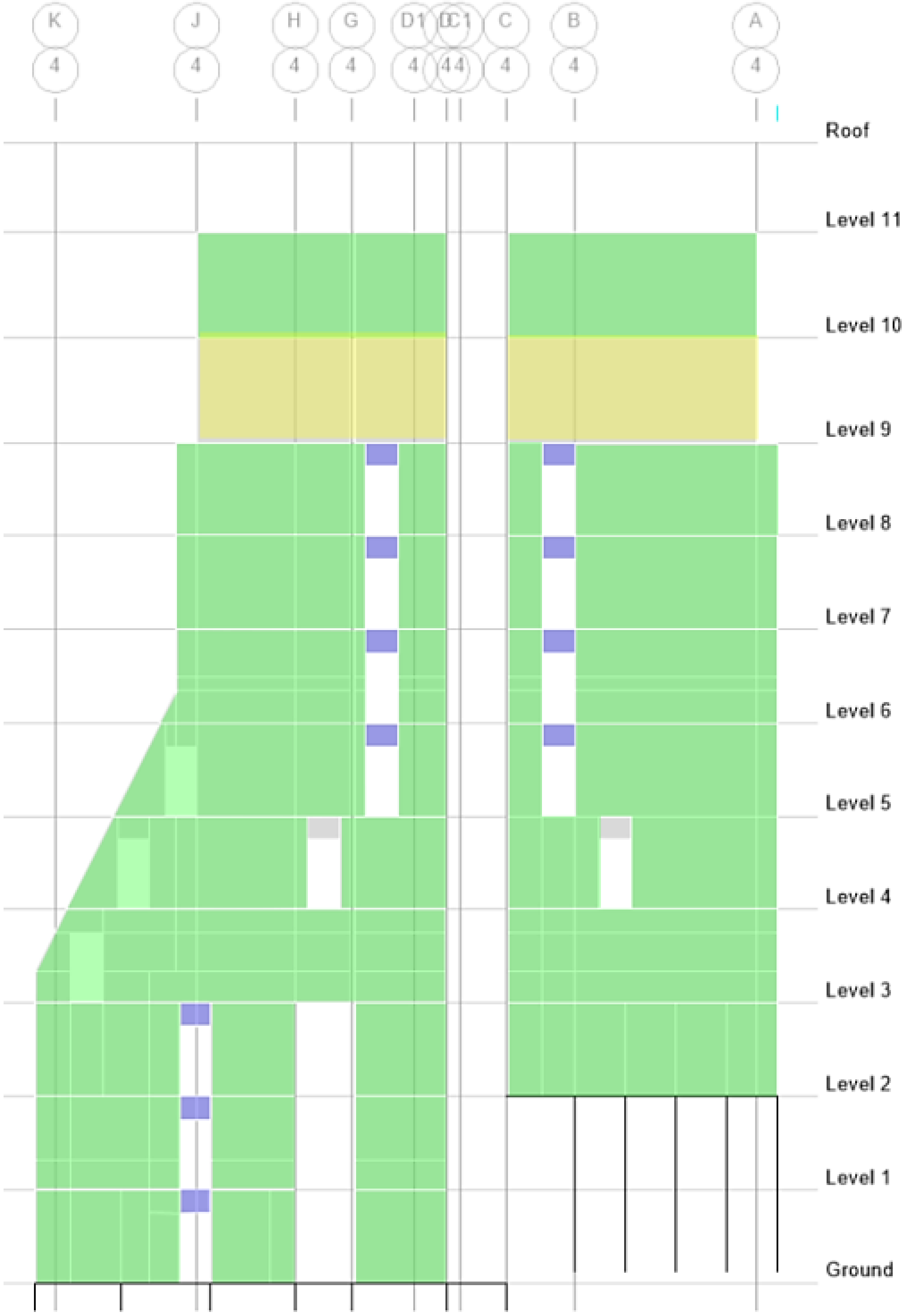

Figure H-4, Gridline 4 Transverse Direction Wall – North Structure

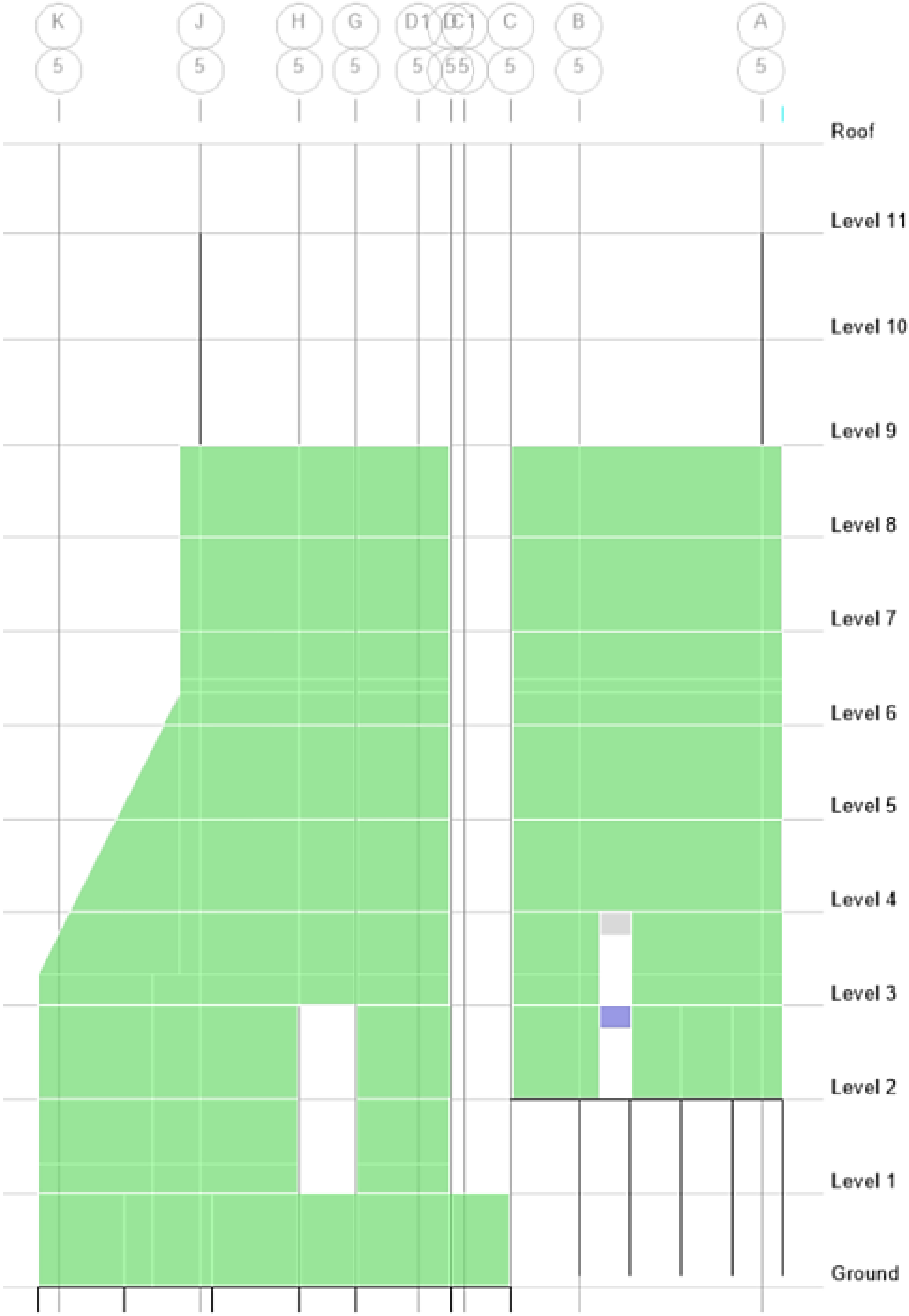

Figure H-5, Gridline 5 Transverse Direction Wall – North Structure

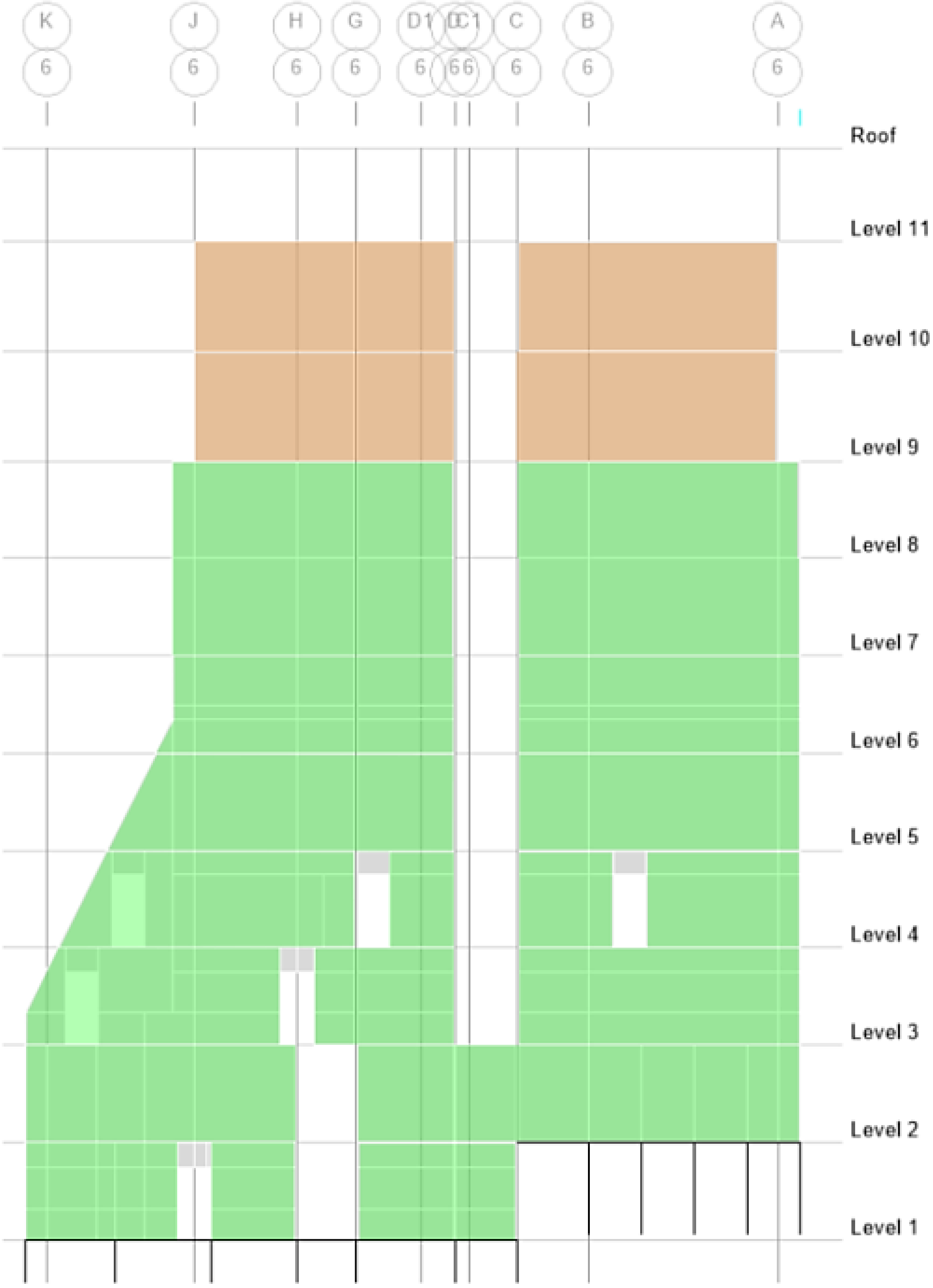

Figure H-6, Gridline 6 Transverse Direction Wall – North Structure

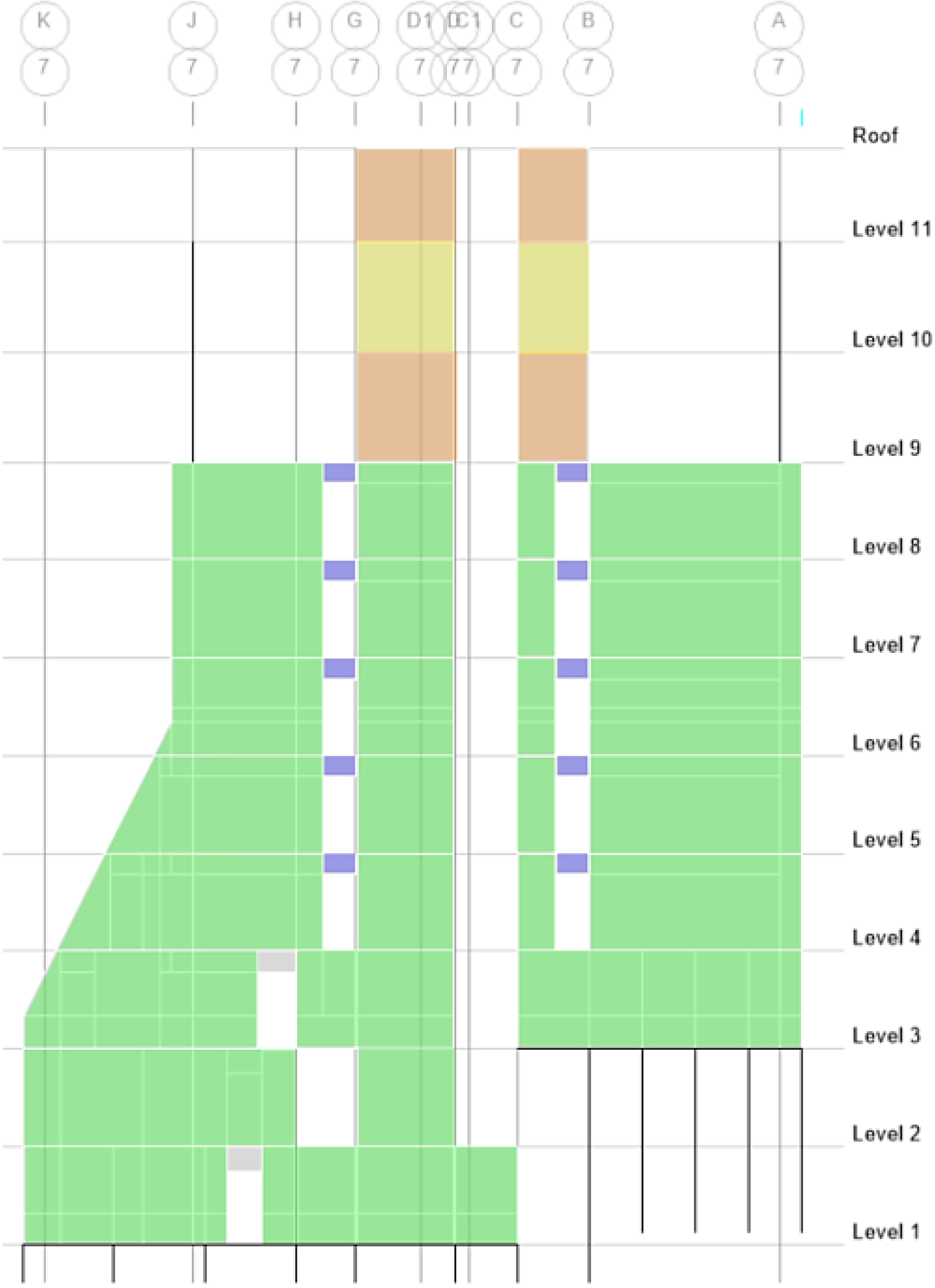

Figure H-7, Gridline 7 Transverse Direction Wall – North Structure

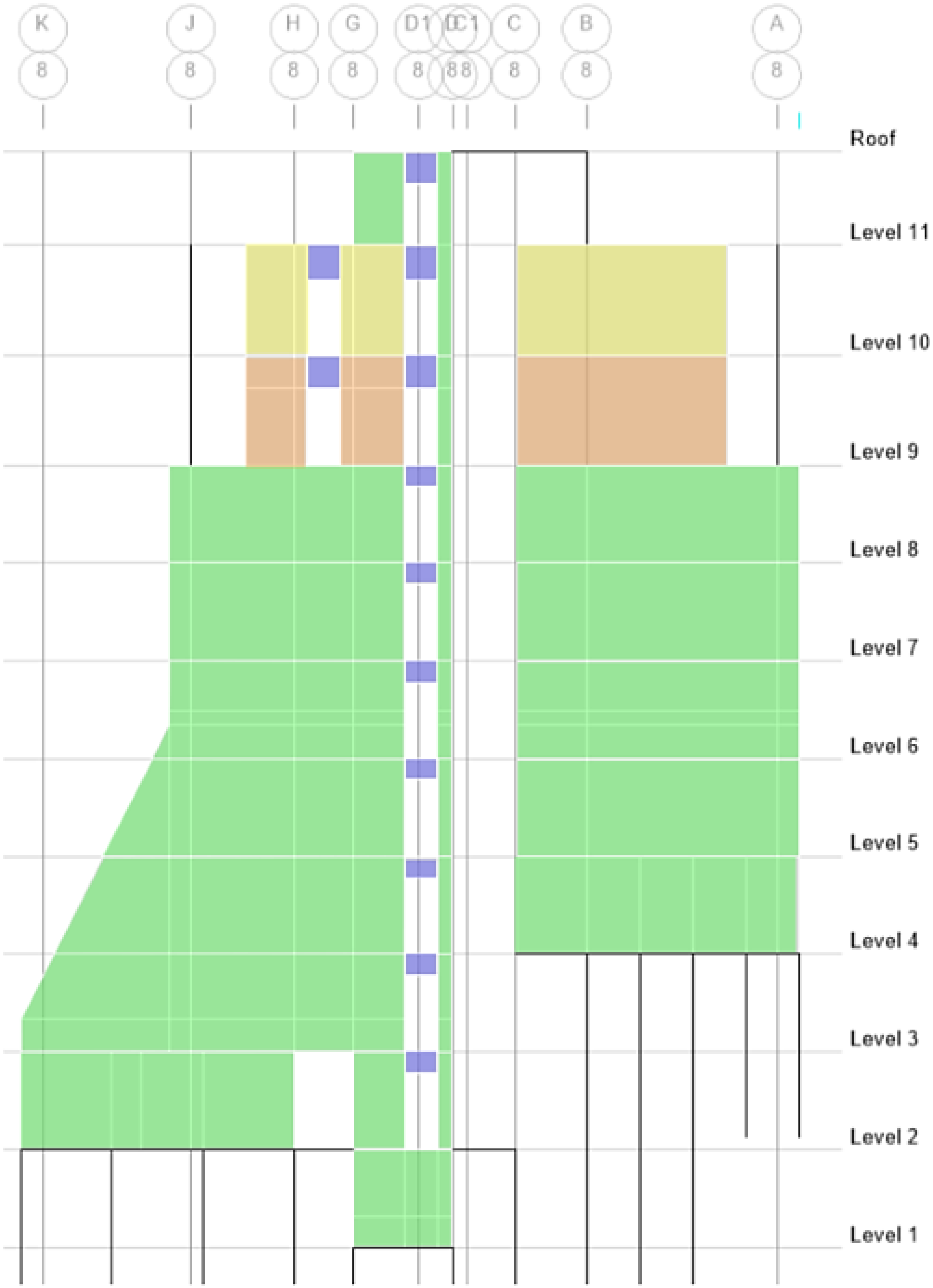

Figure H-8, Gridline 8 Transverse Direction Wall – North Structure

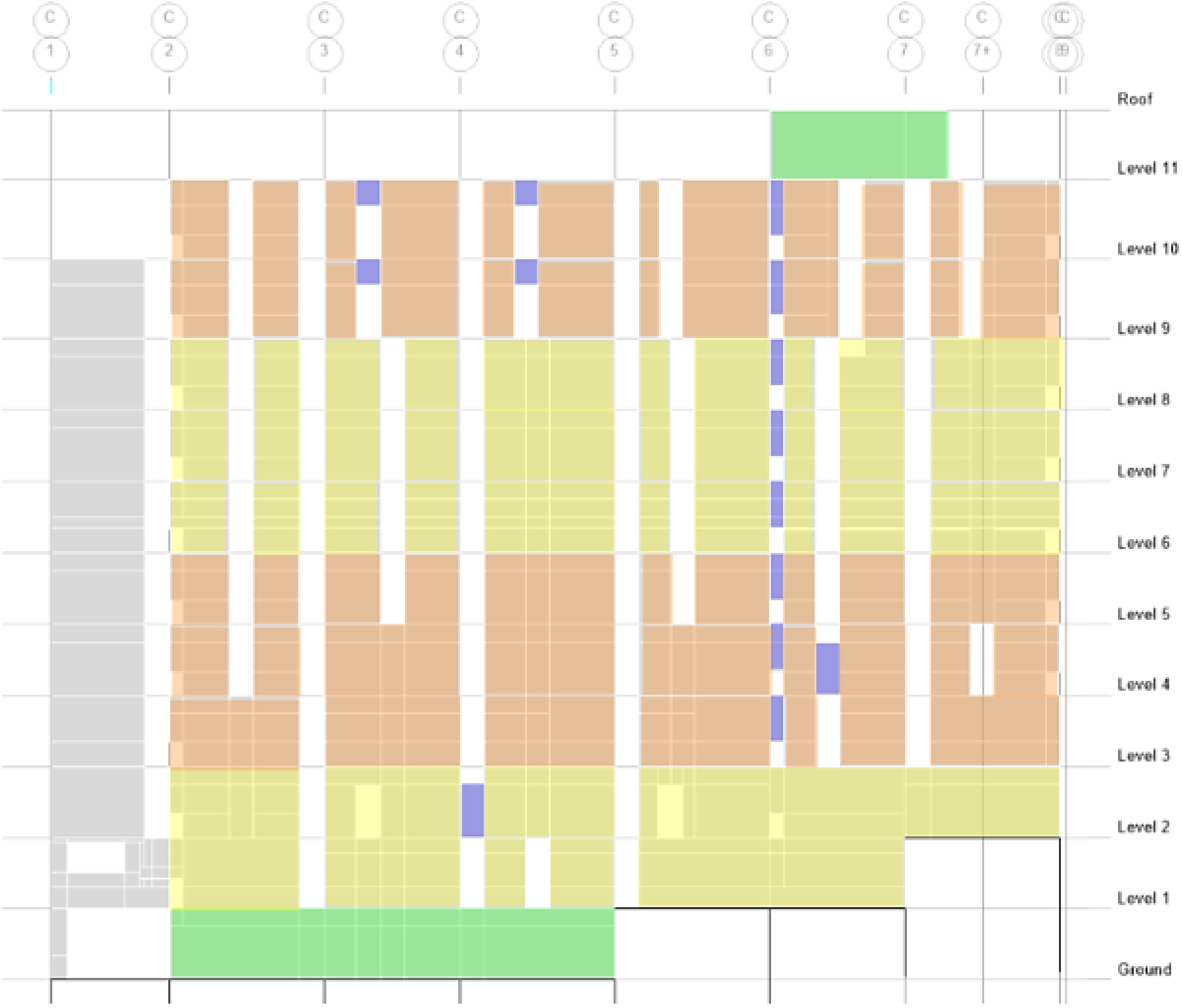

Figure H-9, Gridline C Longitudinal Direction Wall – North Structure

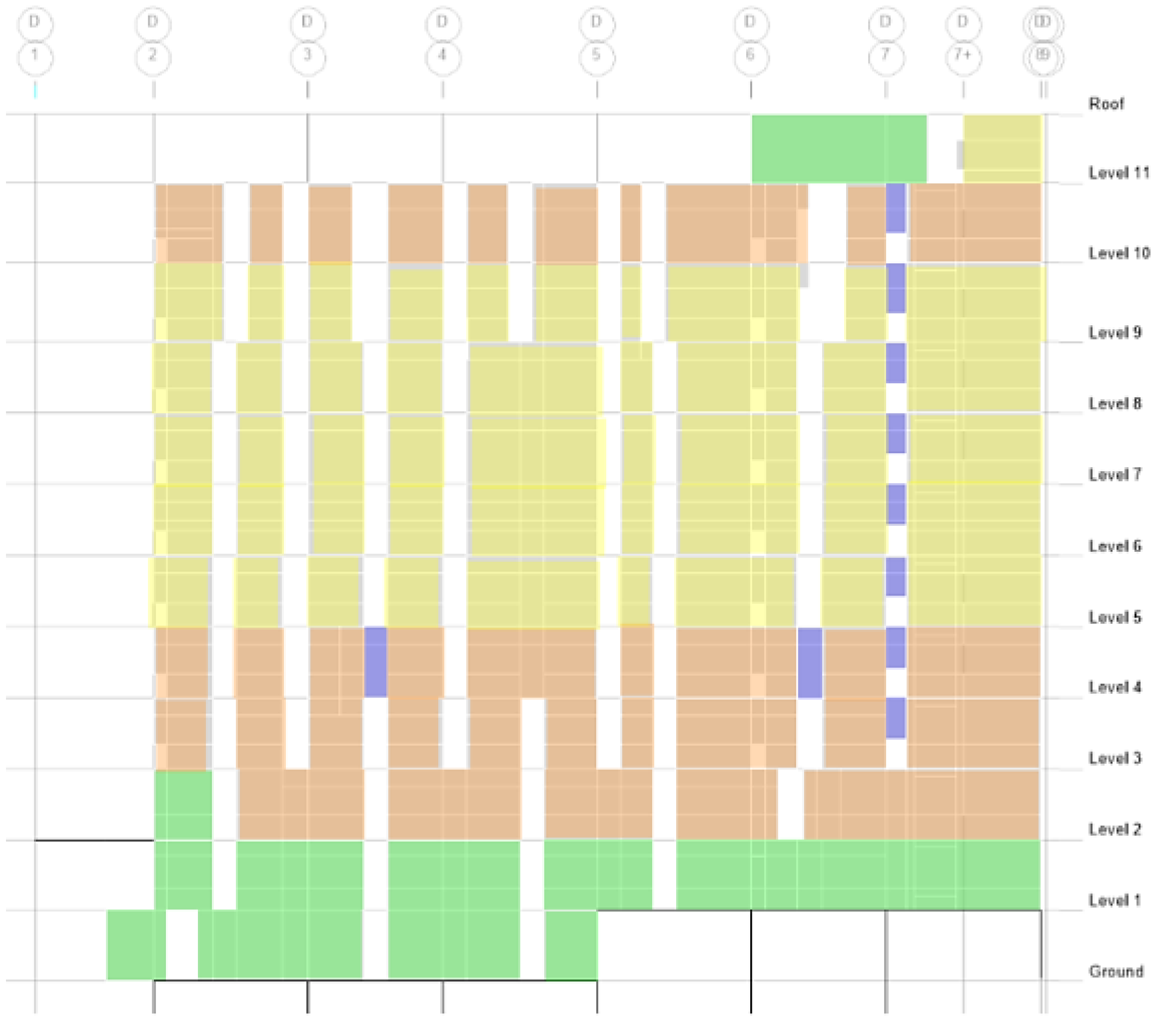

Figure H-10, Gridline D Longitudinal Direction Wall – North Structure

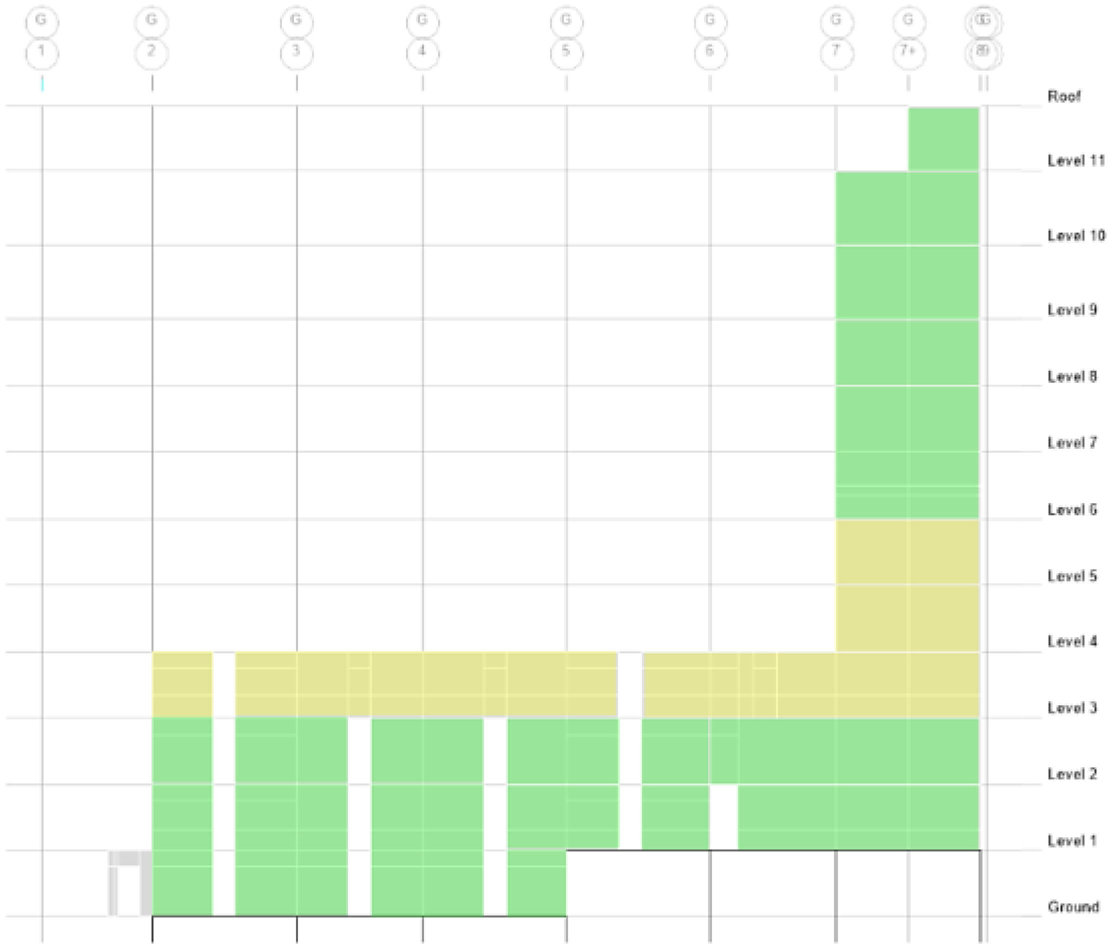

Figure H-11, Gridline G Longitudinal Direction Wall – North Structure

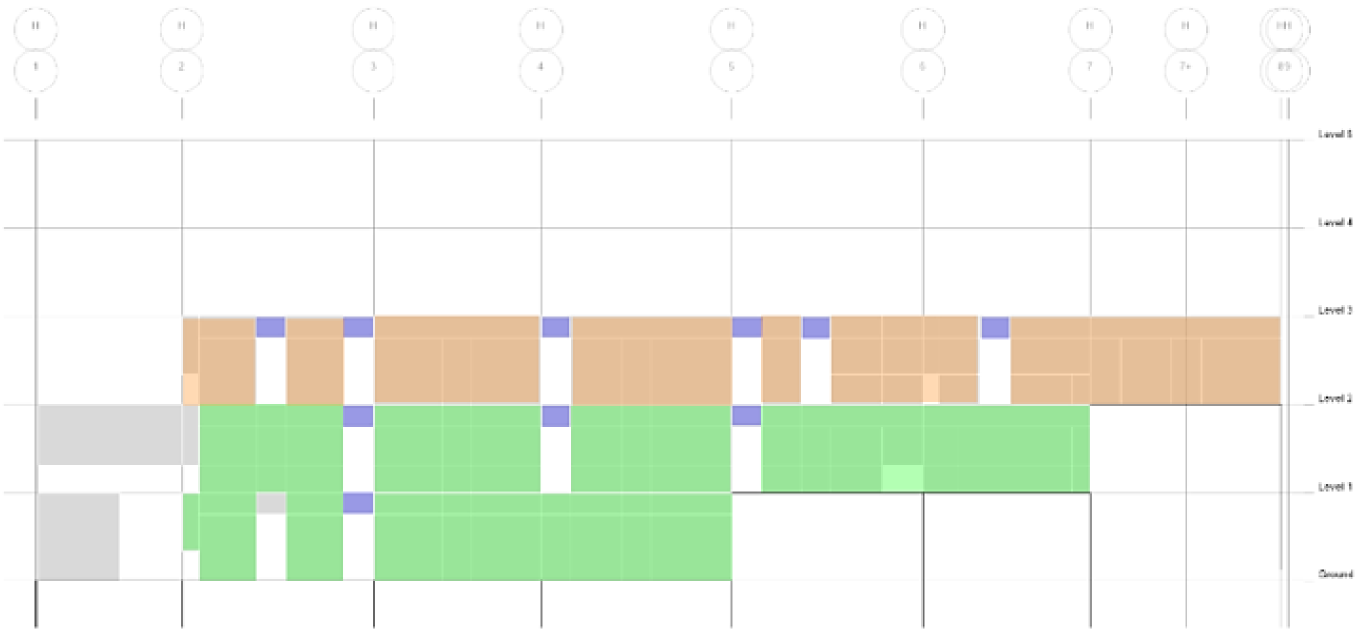

Figure H-12, Gridline H Longitudinal Direction Wall – North Structure

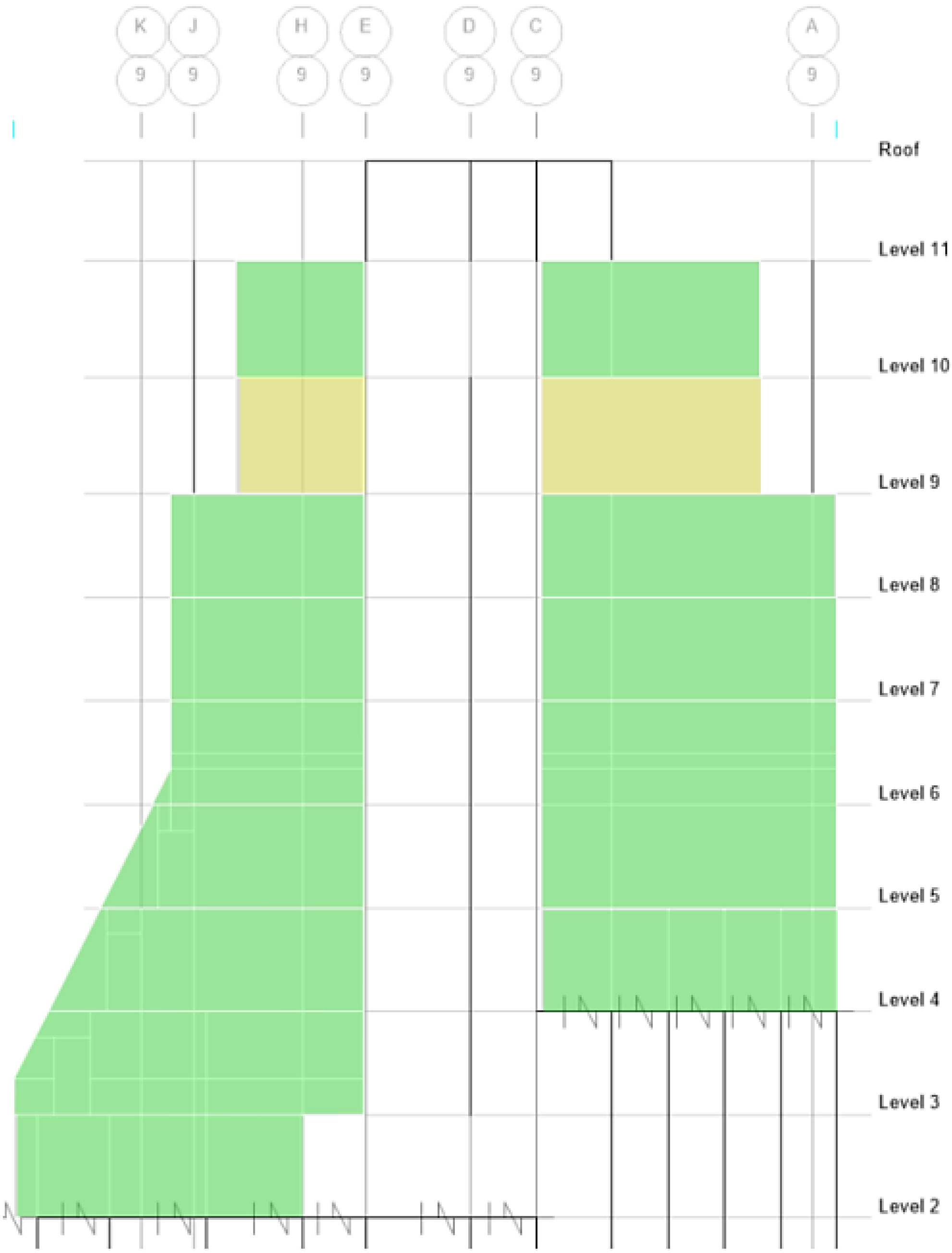

Figure H-13, Gridline 9 Transverse Direction Wall – South Structure

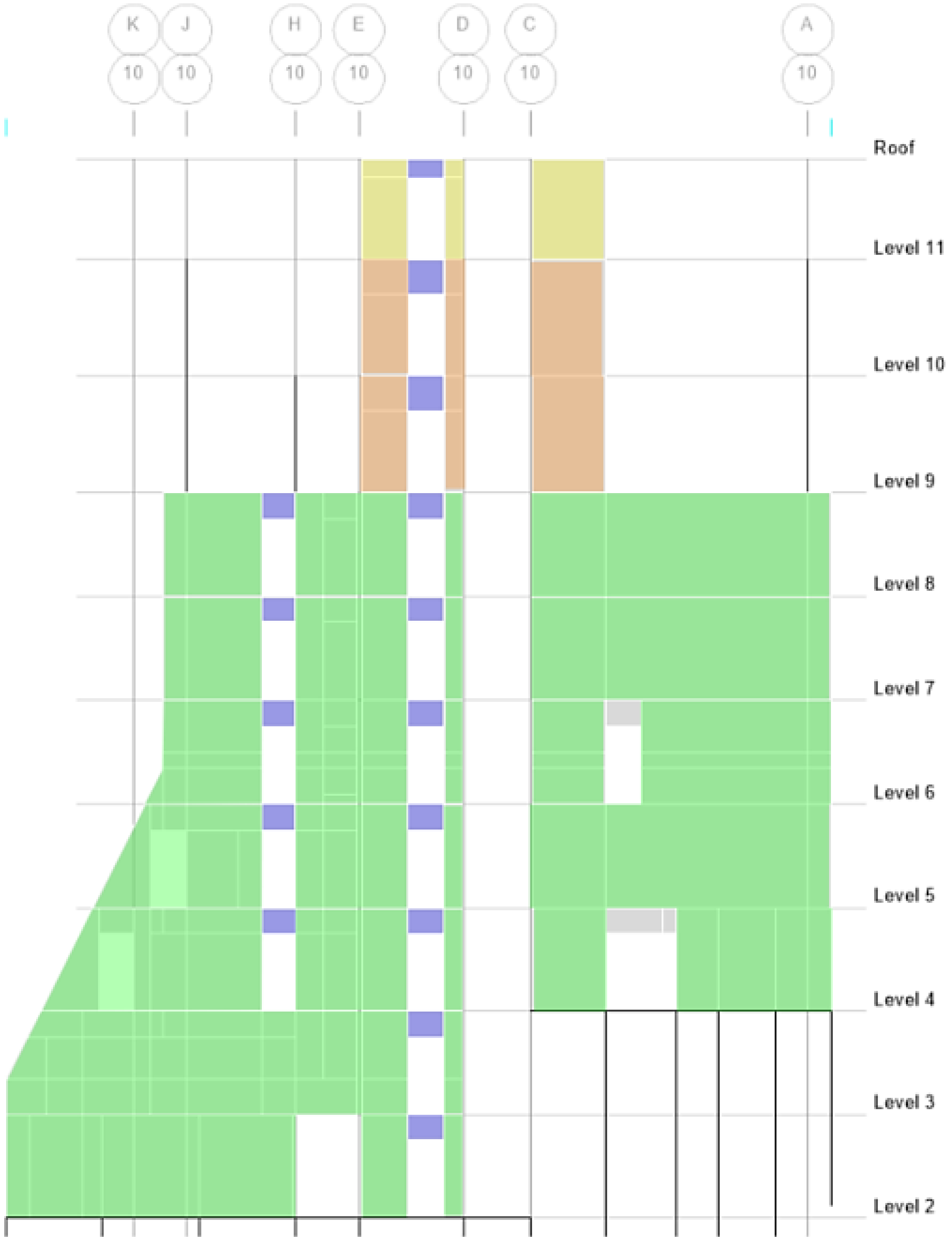

Figure H-14, Gridline 10 Transverse Direction Wall – South Structure

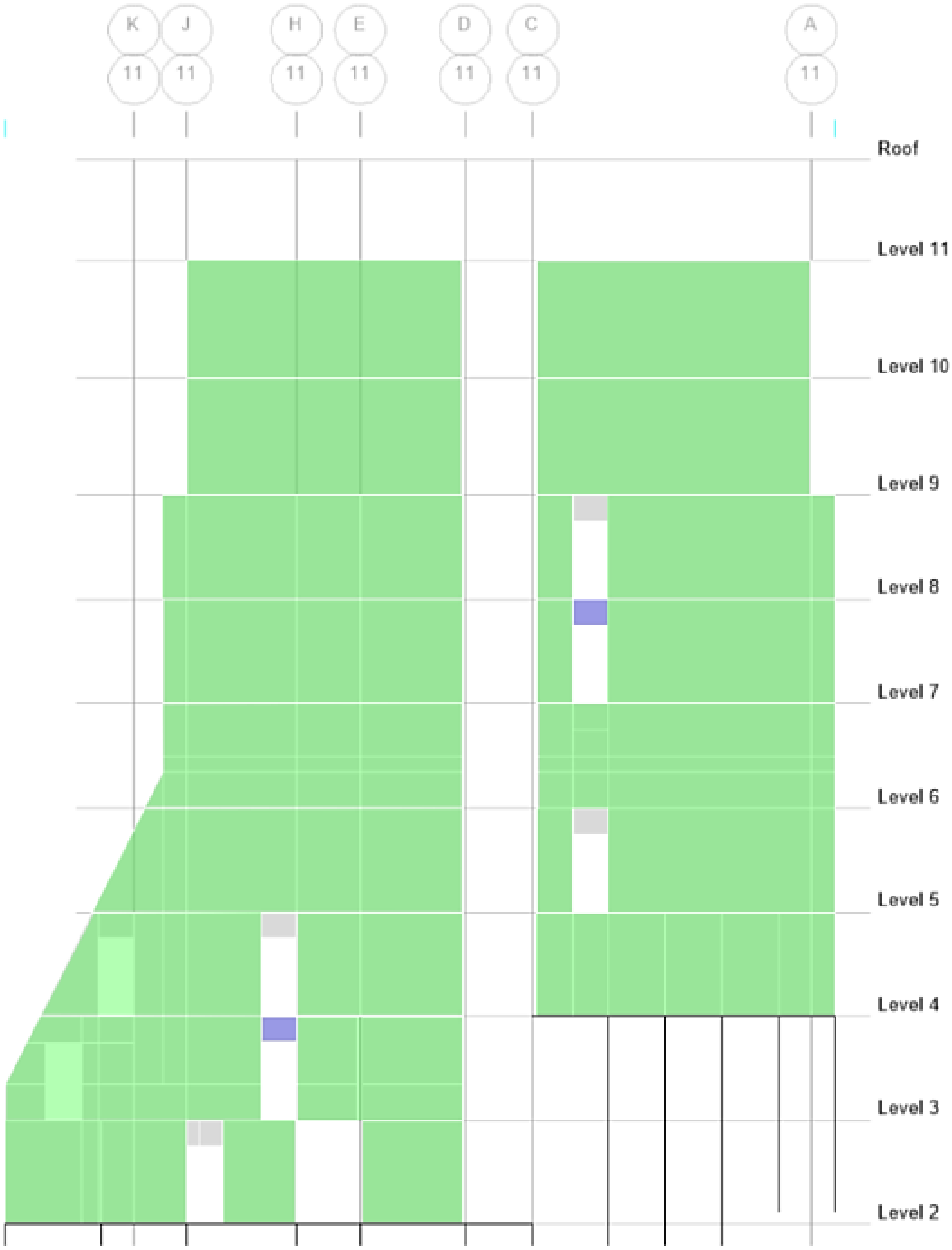

Figure H-15, Gridline 11 Transverse Direction Wall – South Structure

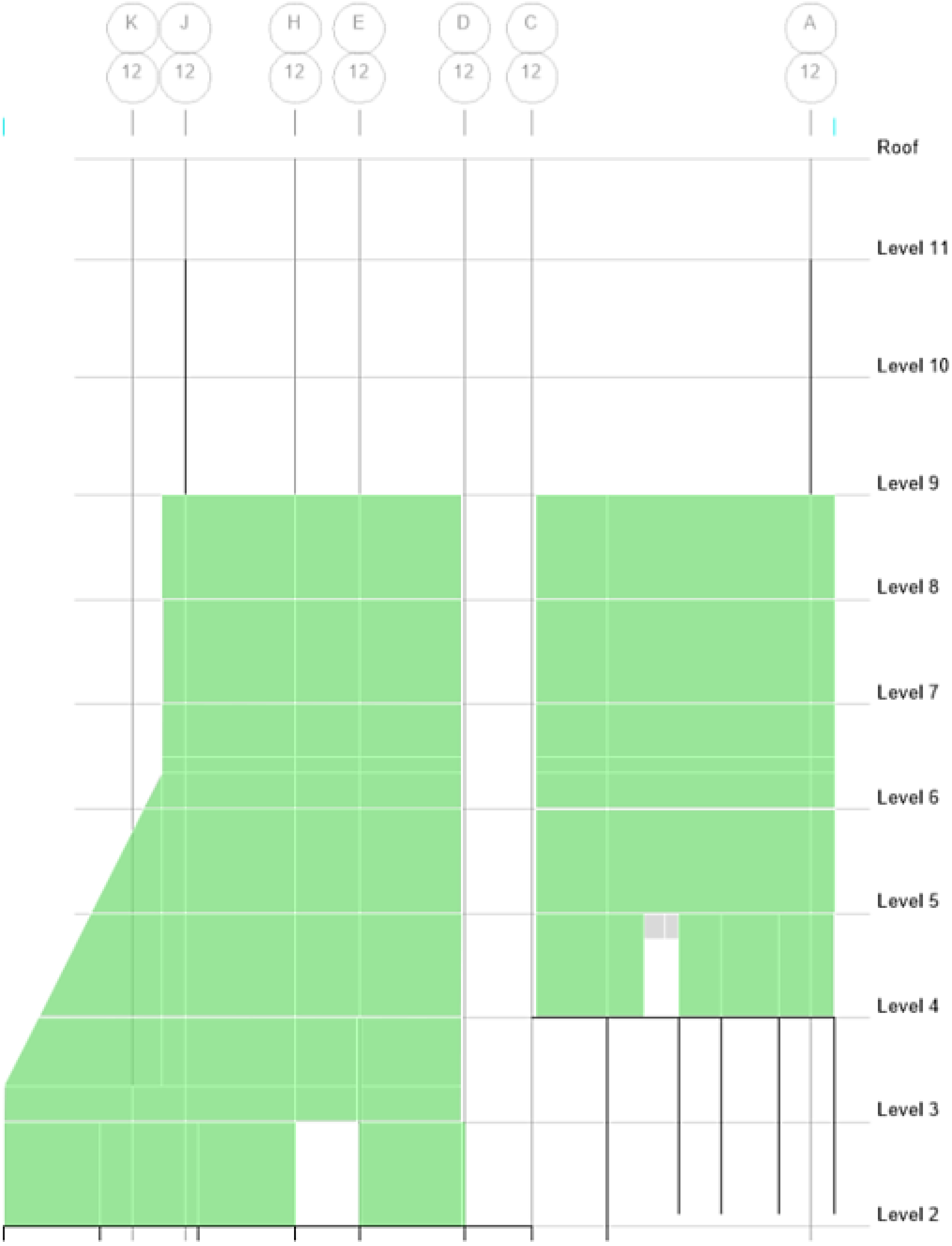

Figure H-16, Gridline 12 Transverse Direction Wall – South Structure

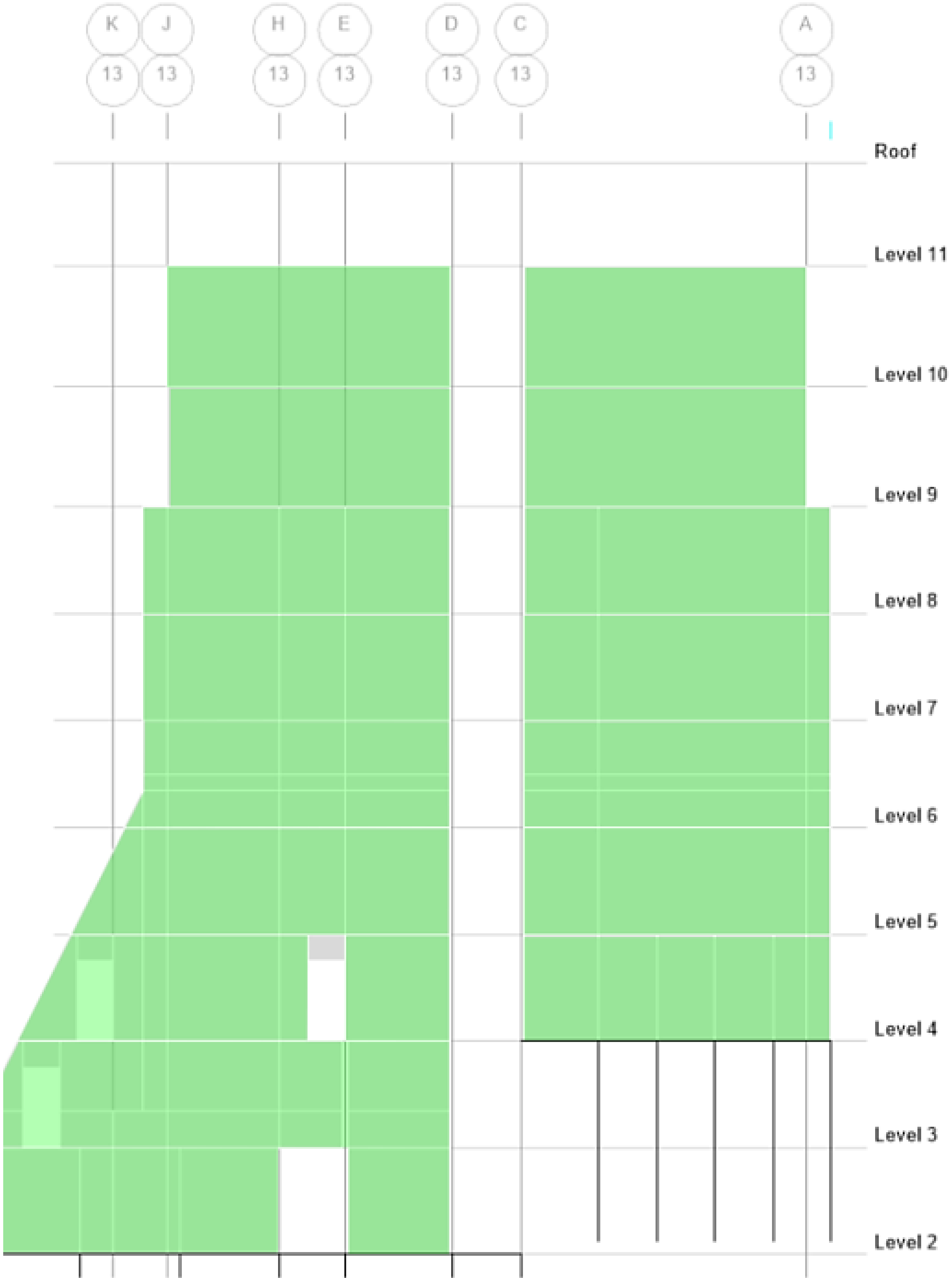

Figure H-17, Gridline 13 Transverse Direction Wall – South Structure

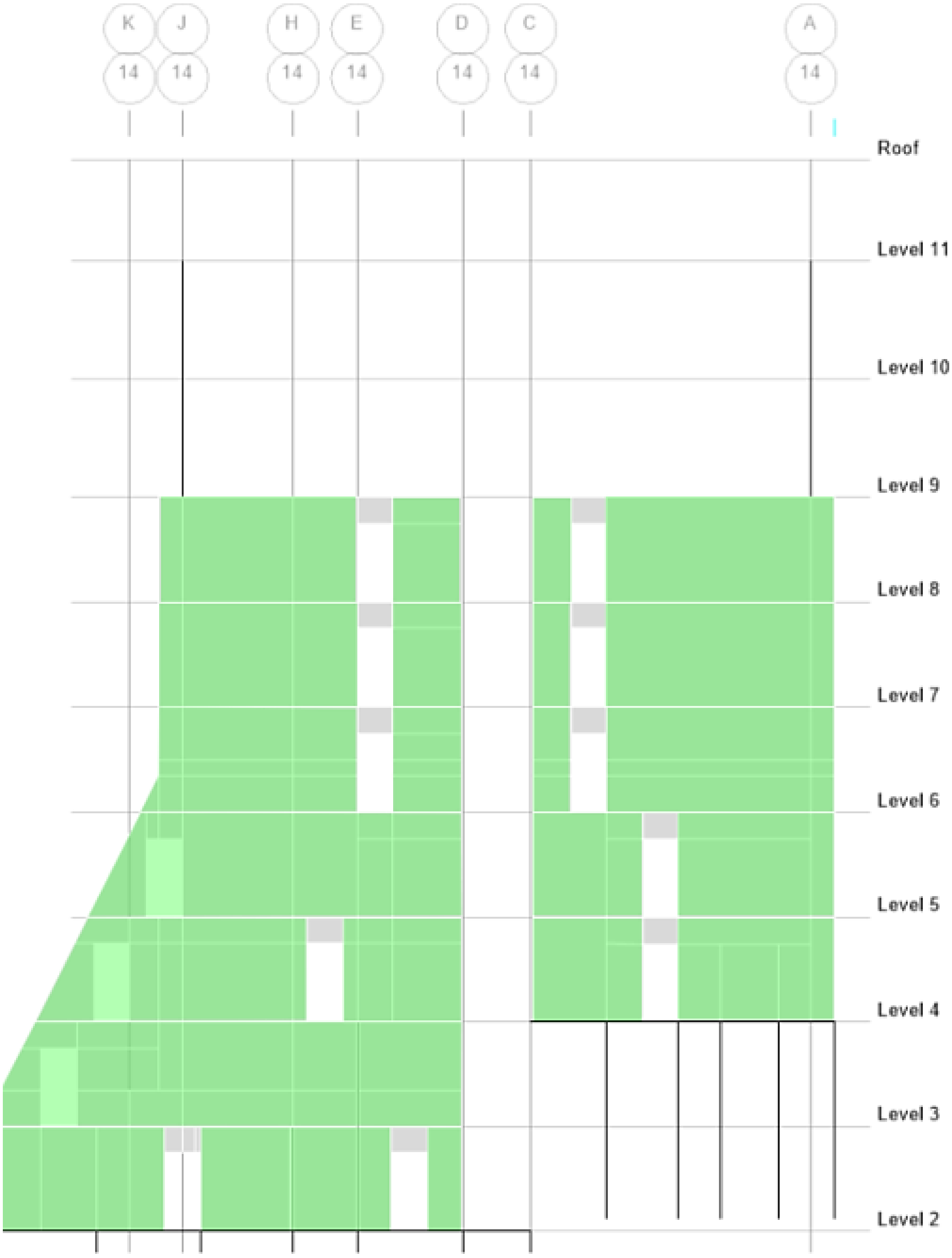

Figure H-18, Gridline 14 Transverse Direction Wall – South Structure

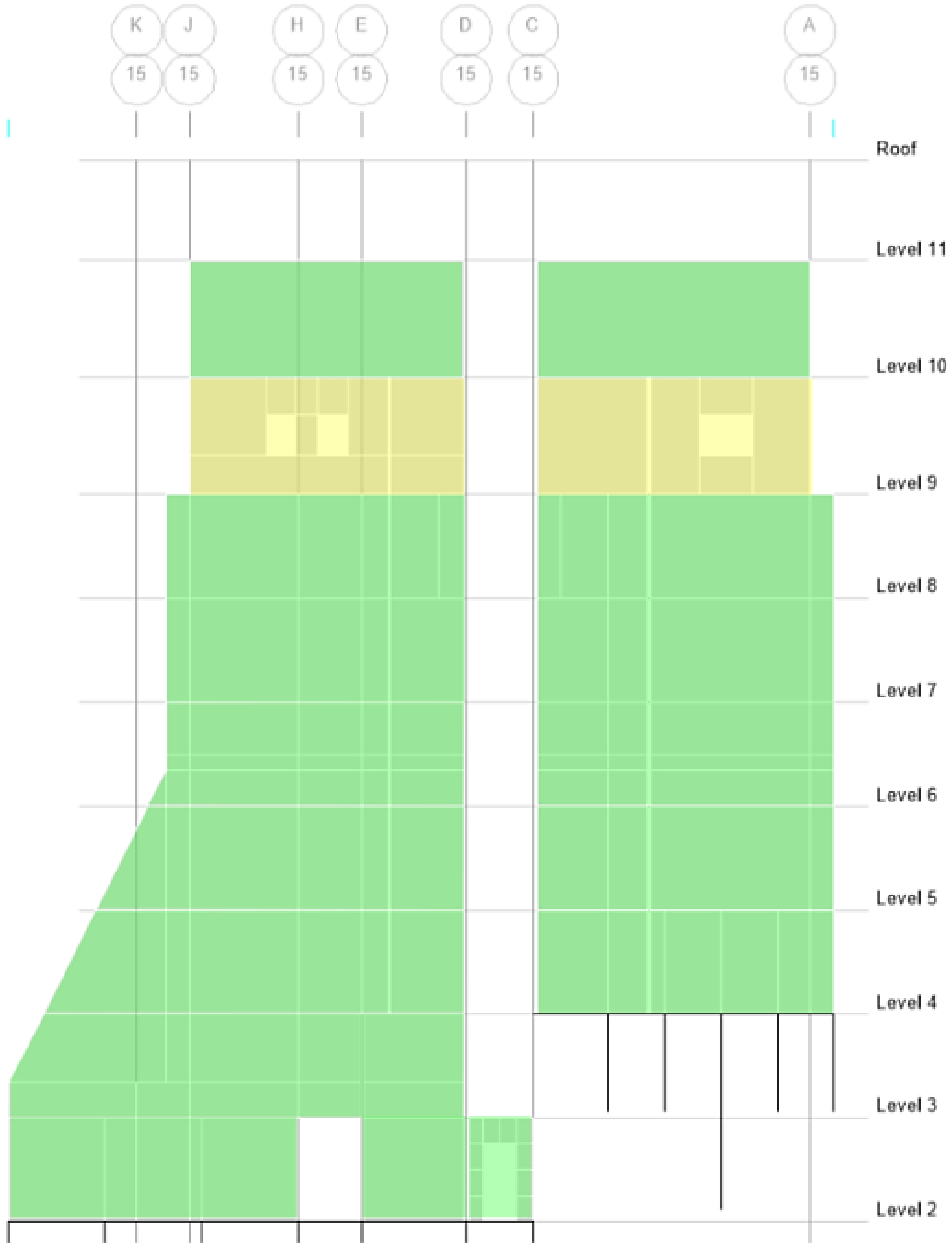

Figure H-19, Gridline 15 Transverse Direction Wall – South Structure

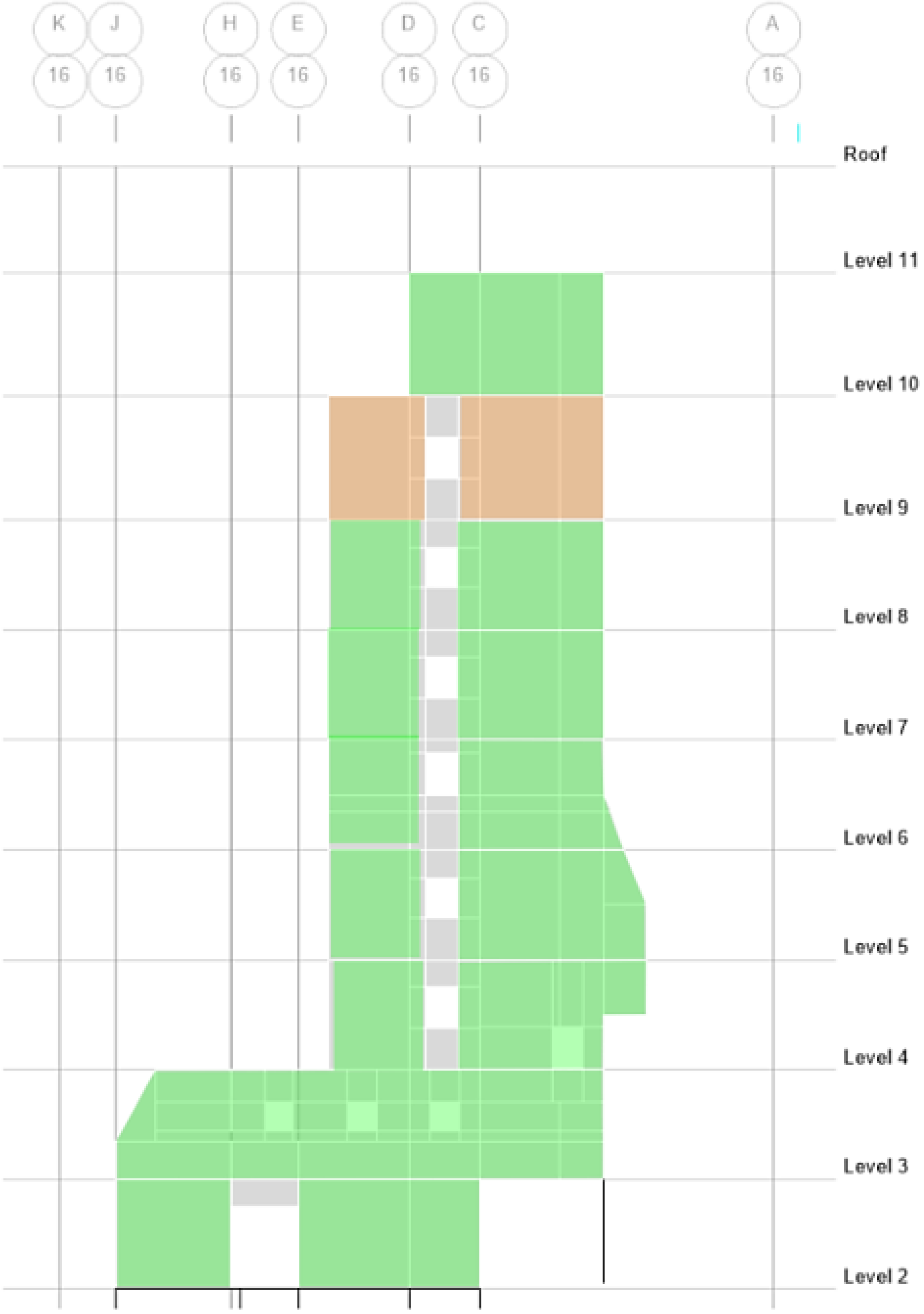

Figure H-20, Gridline 16 Transverse Direction Wall – South Structure

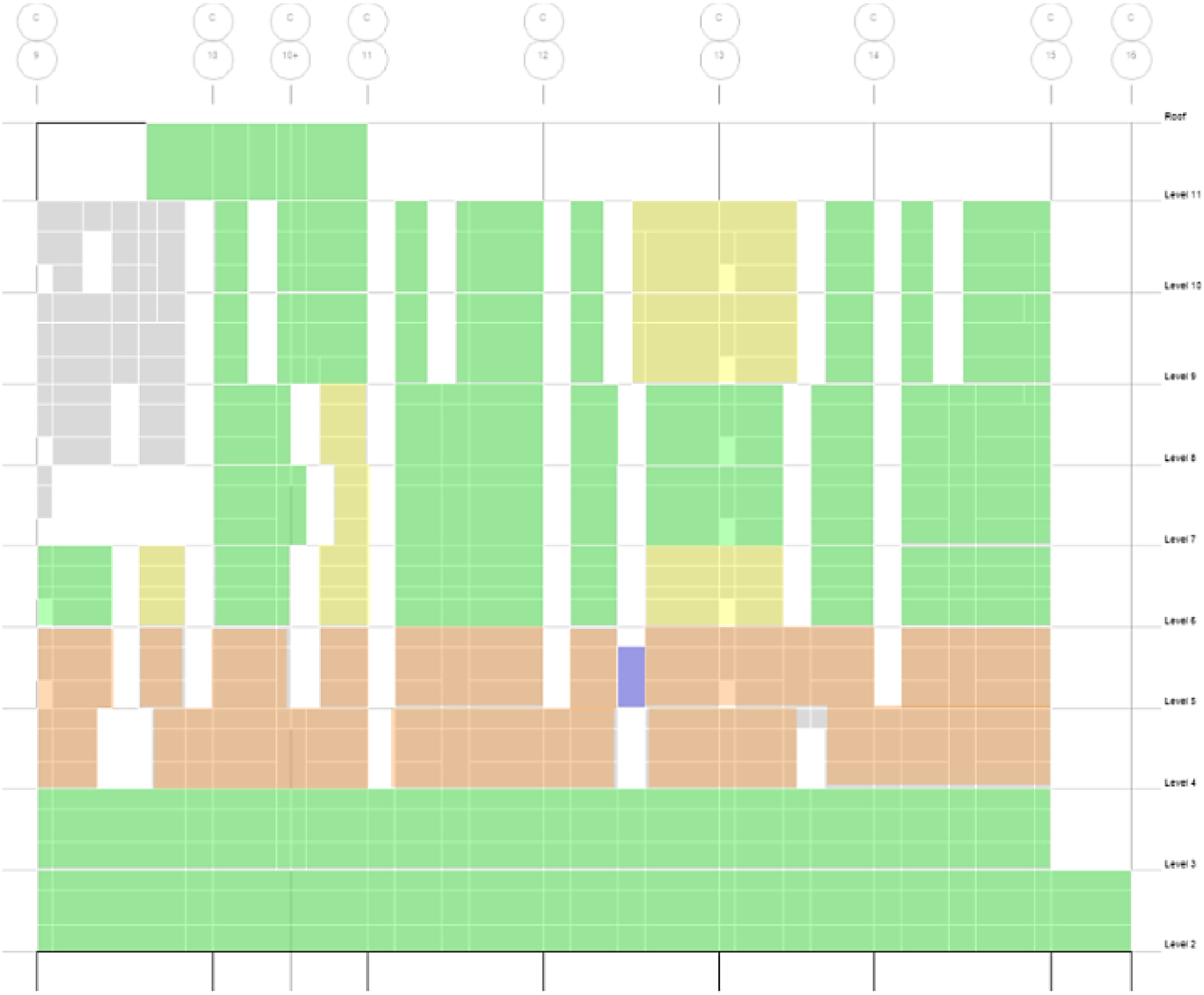

Figure H-21, Gridline C Longitudinal Direction Wall – South Structure

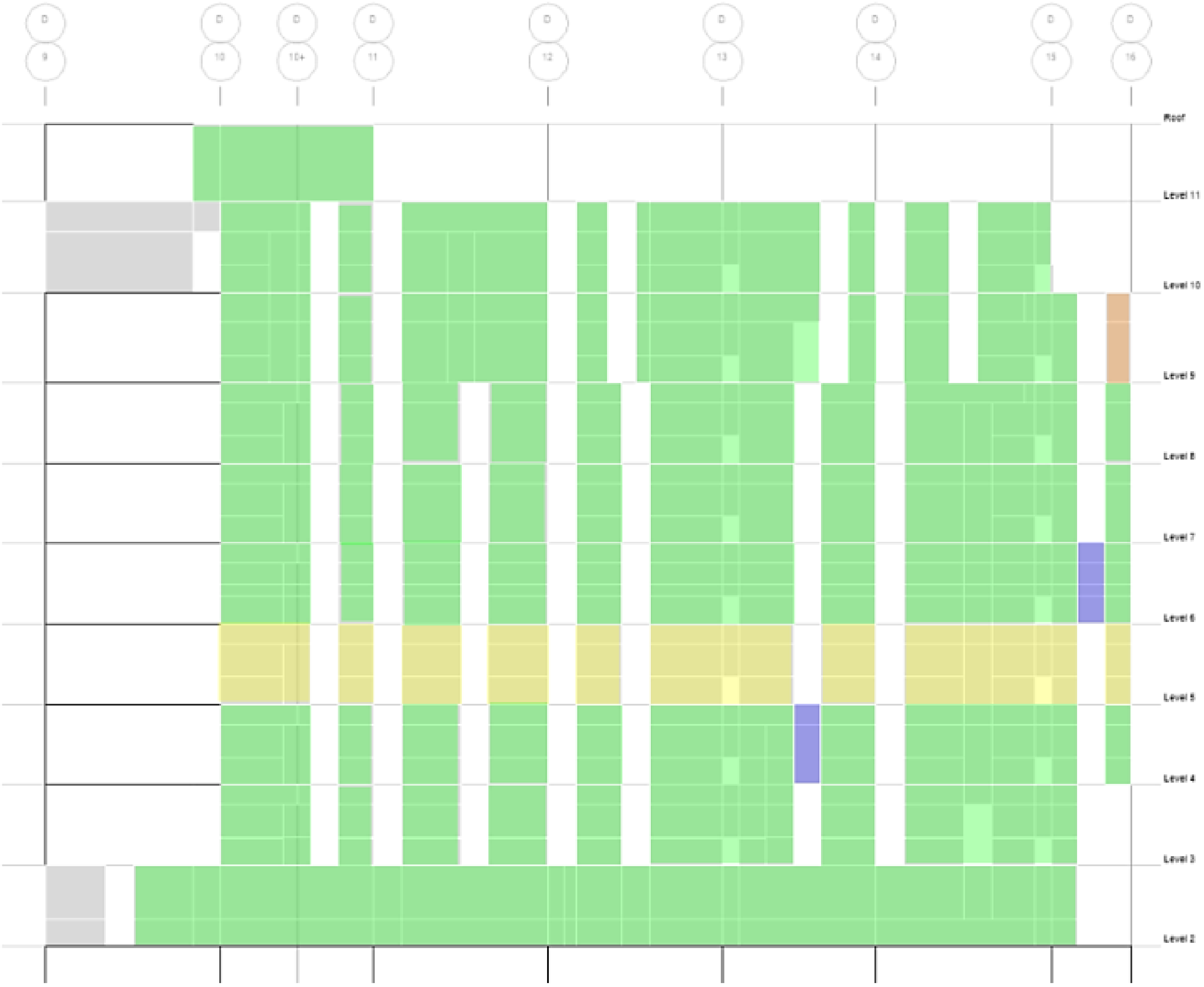

Figure H-22, Gridline D Longitudinal Direction Wall – South Structure

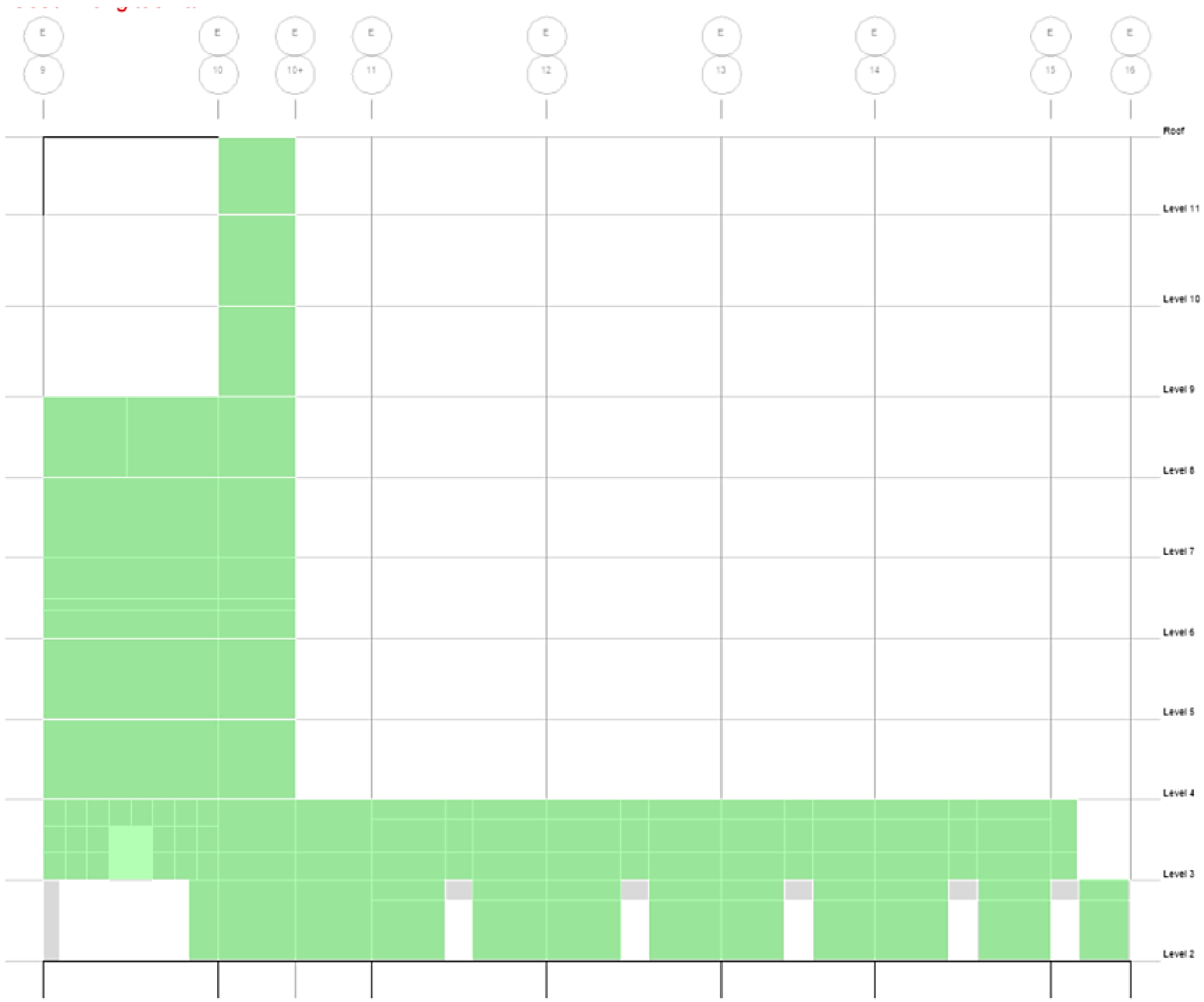

Figure H-23, Gridline E Longitudinal Direction Wall – South Structure

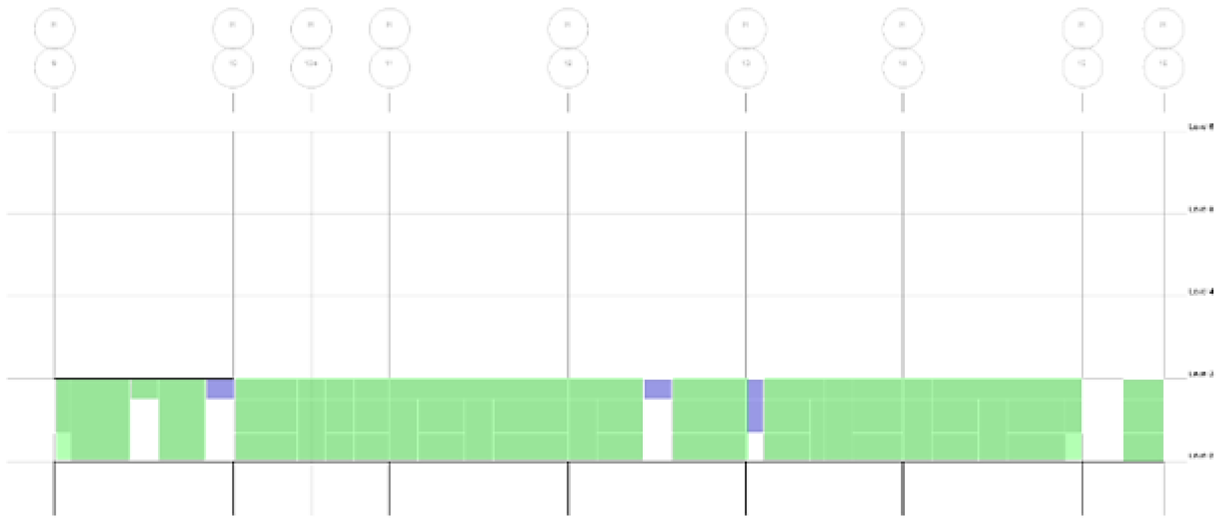

Figure H-24, Gridline H Longitudinal Direction Wall – South Structure

I

Appendix I – Beca Peer Review Letter

85 Molesworth Street,
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6140, New Zealand
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E: info@beca.com // www.beca.com

Peter Mora, Mario Venter, Casey Zhang
Wellington City Council
P.O. Box
Wellington Central

12 December 2023

Attention: Peter Mora, Mario Venter, Casey Zhang

Dear Peter

Peer Review DSA Berkeley Dallard Apartments, Mt Cook

Beca Ltd (Beca) was engaged by Wellington City Council to review the Detailed Seismic Assessment (DSA) carried out by Aurecon for the Berkely Dallard Apartments located at 46 Nairn Street, Mount Cook, Wellington. The apartment building consists of two structurally independent buildings, the north building and the south building.

Fig 1: Image of the Apartment building from the north

make everyday better.

1.1 Information Received

Revision	1	2	A
Berkely Dallard – Detailed Seismic Assessment Report	17/5/2023	6/12/2023	
Berkeley Dallard Foundation Calculations – North Structure			26/5/2023
Berkeley Dallard Foundation Calculations – South Structure			26/5/2023
Berkeley Dallard Concrete Wall Calculations – North Structure			26/5/2023
Berkeley Dallard Concrete Wall Calculations – South Structure			26/5/2023
Berkeley Dallard Diaphragm Calculations			4/2023
Berkeley Dallard Secondary Structure Calculations			4/2023
1973/74 Original Structural drawings by Wellington City Corporation Works Department			
2013 Strengthening Structural Drawings by Aurecon			
2013 Strengthening Architectural Drawings by CCM Architects			
2009 Geotechnical Report by Aurecon			
Additional calculations and notes in Email from Aurecon dated 11 July 2023 including sLaMA Calculations, RC Spot Checks and Geotechnical Considerations.			
Additional calculations and notes in Email from Aurecon dated 6 November 2023 including ratcheting calculations			
Additional notes in Email from Aurecon dated 8 November 2023 discussing capacity of Trubolts			

1.2 Scope of Beca's Review

Beca was asked to undertake a peer review of the DSA with focus on identifying what items are above and below 34%NBS.

1.3 Building Description

The two buildings that form the Berkeley Dallard Apartments were designed in 1974 and are rectangular structures. The north building is a maximum of 11 storeys high reducing to 7 storeys as the site slopes up to the south- east. The south building is a maximum of 9 storeys reducing to 7 storeys at the site slopes up to the East. The lateral load resisting system consists of reinforced concrete shear walls with in-situ concrete slab on a piled foundation. The level 10 floor and roof structures are lightweight timber and cold-form steel construction. Aurecon were commissioned by WCC in 2013 to strengthen the building to 67% NBS (IL3), The strengthening works primarily consisted of construction of new reinforced concrete skins to the existing shear walls and decoupling of some of the shear walls by removing the spandrel beams.

1.4 Aurecon's Seismic Assessment Results

Aurecon has determined that the buildings achieve the following earthquake ratings

	Rev 1 – May 2023	Rev 2 – Dec 2023
North Building	15%*NBS* (IL2) Critical Structural Weakness: longitudinal walls in-plane capacity	***30%NBS* (IL2)** After increasing redistribution of loads in-plane. Critical Structural Weakness: Reinforced concrete walls cantilevering out-of plane above level 9 and level 10 diaphragm connections. Relatively straight forward to bring above 34%NBS.
		> 34%*NBS* (IL2) Main structure of the building (other than the floors 9 and 10)
South Building	15%*NBS* (IL2) Critical Structural Weakness: longitudinal walls in-plane capacity	***30%NBS* (IL2)** After increasing redistribution of loads in-plane. Critical Structural Weakness: Reinforced concrete walls cantilevering out-of plane above level 9 and level 10 diaphragm connections. Relatively straight forward to bring above 34%NBS.
		> 34%*NBS* (IL2) Main structure of the building (other than the floors 9 and 10)

These buildings were assessed in accordance with the guideline document 'The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments', dated July 2017, C5. The earthquake rating assumes that the building is Importance Level 2 (IL2), in accordance with the Joint Australian/New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002, is appropriate.

1.5 DSA Peer Review

From our review of the available information and our discussions with Aurecon the below is a summary of our key comments and the outcomes of the discussions on these points

Reinforced Concrete Walls In-plane

We initially raised some questions around how the non-linear nature of the ductile reinforced concrete walls had been considered in the elastic analysis undertaken by Aurecon. This approach was effectively limiting the capacity when the first wall panel failed however there was significant additional capacity in the adjacent panels if redistribution of loads was allowed.

Following discussion on around the approach taken, Aurecon have revised their calculations and all of the walls now score in excess of 34%NBS in-plane.

Diaphragms

The building has reinforced concrete diaphragms at the lower levels which we would expect to be relatively robust, Aurecon's initial assessment scored these elements at 60% NBS. We raised some questions about the load distributions in the grillage analyses undertaken which appeared to show high strut and tie forces. We don't believe that the forces should be as high as they appear in the model, however we have not pursued this further as these not critical elements and increasing the score for these elements will not have an overall impact on the buildings score.

At level 10 the floor diaphragm consist of a flexible plywood diaphragm which was installed as part of the 2013 works and is fixed to the reinforced concrete walls using Ramset Trubolts (post installed mechanical anchors). The roof framing is also fixed to the top of the walls using similar anchors. As these anchors are not C1 or C2 seismic rated anchors (they were installed prior to C1 and C2 anchors were required in New Zealand) it is Aurecon's view that they should be rated <34%NBS, regardless of their capacity to resist static loads. It is Beca's view that this is very conservative and improved ratings could be achieved if a rational reduced capacity was used for these anchors. We accept that this is Aurecon's position and there has been on change to the score for these elements on this basis. These elements are now part of the Critical Structural Weakness that govern the overall rating for the buildings.

We would recommend that testing is undertaken on the existing anchors to better understand their capacity which may allow the score for these elements to be increased.

Reinforced concrete walls Out-of-plane

We raised some comments with Aurecon around the support conditions to the reinforced concrete walls above level 9. On the basis that Aurecon have rated the post-installed diaphragm and roof connections at <34%NBS and unable to provide restraint to the reinforced concrete walls out of plane. Accepting Aurecon's view on the diaphragm connections (as discussed above) we agree that the walls have insufficient capacity to cantilever out-of plane with parts loading.

Secondary structures

In their initial assessment Aurecon had assessed the external canopies at 40*%NBS*. In our initial comments we raised the concern that the loads being used to asses the canopy were excessively high as the canopy had been assessed as a part when it is actually an independent structure. Aurecon agreed and updated their calculations, the score for this area has increased to >67*%NBS.*

SSW

- Both consultants, Beca and Aurecon have not identified any Severe Structural Weakness.

1.6 Conclusion

After completion of the peer review both the North and South Buildings are rated less than 34*%NBS* (IL2), however all CSWs are now limited to elements in the top two floors of the building. We have prepared a peer review log attached and all items are now closed out. We have no further comments.

1.7 Clarifications

This review is of defined scope and is for reliance by Wellington City Council only, and only for this commission. Beca accepts no responsibility or liability to any third part for loss or damage whatsoever arising out of the use of or reliance on this review by that party or any party other than our Client.

While Beca has reviewed the DSA, we do not accept any responsibility for the assessment; this will reside with Aurecon. This review is necessarily reliant on the accuracy, currency and completeness of the information provided to us.

Our review has been limited to structural scope only and does not include a review of geotechnical hazards, non-structural secondary elements or the proposed strengthening options.

If you have any queries, please contact the undersigned.

Yours sincerely

[redacted]

Technical Director - Structural Engineering

on behalf of

Beca Limited

[redacted]

Beca Structural Peer Review

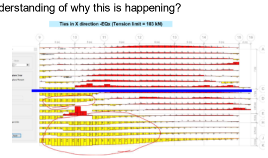
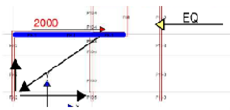
Project:	**WCC Berkeley Dallard - DSA Peer Review**
Project number:	**5275386**
Lead Reviewer:	s(7)(2)(a)
Reviewer (this form):	
Date Updated:	**Friday, 8 December 2023**
Review Phase:	**DSA stage**
Input Documents:	**DSA calculations and report**

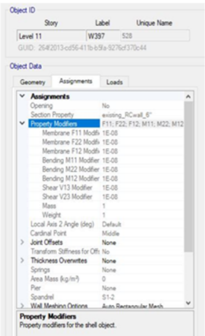
Insert further columns in here if multi-stage response required

Ref No	Item	Reviewers Comment	Designers Response	Reviewers Followup Comment	Designers Followup Response	Reviewers Close out Comment	Status
1.0	**General**						
1.1	Boundary conditions	It is noted that two boundary conditions (fixed base and pile springs) have been considered and the worst has been taken. Is this appropriate or should the more realistic scenario be the one considered? i.e not fixed base. Has a sensitivity study been undertaken in the variance of outcome.	We did not consider the worst case. The effect of the soil flexibility on the behaviour of the superstructure was not significant on the wall elements above level 4. below level 4 and where the terraced foundation levels started, the effect of soil flexibility lowers the NBS scoring of the wall elements by 5-10%. The rating currently reported in Appendix H for the wall elements are for the fixed base scenario. In general, considering the soil flexibility would not change the "Alpha rating" and Potential Building Status. Consideration of soil flexibility and foundation limits was mainly utilised for rating of the piles			**Noted, No Further Comments**	CLOSED
1.2	Modelling/assessment approach	It is noted in the summary that a SLaMa has been undertaken for the building. Could you provide more detail on how this has been done and provide the calculations for this?	We have completed the SLaMA for both of the buildings. The calculations are appended to this. In our SLaMA we have now incorporated the Method C. Method B has some restrictions with regards to the nonlinearity level and requiring the diaphragm maintaining its structural integrity. As shown in the calculations, the TIR ratio is in excess of 1.4 for almost all the floors and Method C above is more penalizing compared to the ASCE7 approach. Therefore, we believe that we did not penalize the building for its torsional sensitivity beyond what deemed necessary as per the guideline	It seems that the mechanism identifed has not followeed through to the latter analysis and calculations. Please provide further explaination of this. It appears to demonstrate that a force based assessment is not appropriate for this building.	Refer to 1.4	**Noted, refer below, No Further Comments**	CLOSED
1.3	Ductility	How have you arrived at the ductility of 1.25 used for the assessment? Given that the failure modes are predominantly flexure governed and the walls are doubly reinforced with deformed bars and adequete laps the guidance in the guidelines would suggest that you could consider a ductility of 2 for this building.	As stated in the Memo sent on 27th April, in general the walls in the longitudinal and transverse directions are connected, forming a series of T, C, box or even more complex geometries at either end of the building complex. Accordingly, it is expected that in general the overstrength factor of the walls to be higher than 3 in both directions. while the ductility demand will be lower than this. The statement in your email dated 15th of June is valid for the cantilever walls. Where the walls have complex geometries the eccentricity between the centroid of walls and the compression block will always amplify the overstrength factor. An example of the overstrength factor calculation for a C shaped wall is supplied for further clarification. for further details refer to CNZLS Seminar Notes TR70, 2018. Despite that, we have revised our calculations with the assumption that where the walls are less complex, (like T shape sections) with the assumption that for these walls there is a possibility of PH formation and achieving a ductility. It is found that we may get close to the ductility of 2. The ratings are revised accordingly where applicable.	The calculations that you've shown are not applied correctly when considering the assesment of a building. In an assesement the only rlevance of the overstrength capacity is when considering whether the wall is flexurally or shear governed. If the wall capacity is in excess of the demand (by some margin as shown in your checks, how can it be deemd to be less than 100%? Why have you not allowed more distribution to allow this element to carry more load? This is an example of where no using a displacement based assessment (pushover or similar) approach is penalising the building unessesscarily.	The nature of the DBA and FBA are different. Though as agreed during our meetings on the 24th of August and 31st of October, we considered further distribution of the load between the elements. It is also notable that the performance of the structure in two orthogonal directions is significantly diffrent, it can be observed that walls are mainly yileidng in their web direction and this limits the rating for the walls in the longitudinal direction. However, in the transverse direction, majority of the walls in their flange direction are having suffieint capacity.	**Although we don't agree with the way that Aurecon have calculated the overstrength factors for the walls, we don't belive further discussion on this point will yield a material differnece to the seismic rating. No further comment**	CLOSED

1.4	Failure mechanisms	From the calculations and report provided to us it doesn't appear that the failure mechanism for the building have not really been identified. From your force based assessment it appears some piers have been identified which cannot sustain the elastically applied flexural demand, however we would expect this to redistribute to stronger elements as these begin to yield.	As stated in your comment in 1.5, we considered redistribution between the wall elements. In addition, the SLaMA is now undertaken for the building and the failure mechanisms are accordingly identified.	Having reviewed the SLaMA in relation to the force based assessment it appears that the failure mechanism has not really been identified in the SLaMA and is not consistent with the mechanisms identifed in the force based procedure. Based on the methodology in the guidelines, this would suggest that you should revisit the analysis method used (as previously discussed we would have expected that a pushover analysis would be more appropriate for this type of strutural arrangement.	As discussed during the meetings on the 24th of August and 31st of October and further relevant correspeondence, it is agreed that the SLaMA results in conjuction with the results of the RSA based on the calculated ductility of 2 and full-redistribution is providing sufficient insight about the building performance and the governing mechanism in each direction. For doubly reinforced walls the ratings in the longitudinal direction is expected to be 35%-40% NBS(IL2) for both buildings. In the transverse direction, the ratings are 100% NBS(IL2) for the South Buiding. For the North Building, basded on the results of SLaMA and its proximity to the RSA values the rating>67%NBS is accetable. Accordingly, further DBA analysis is not going to make an step change in the revised ratings attached and deemed unnecessary.	**Although we don't nessecaily agree with the approach taken to get to these results, we broadly agree with the outcome, no further comment**	CLOSED
1.5	Plastic Distribution of Loads	The 30% redistribution that was assumed is based on the code requirements for a new building and is not really relevant for assesment to the guidelines. How have the loads redistibuted in the SLaMA analysis?	We adopted the force-based assessment approach and the 30% is the upper bound of the redistribution as in this method, the rotational capacity is not directly calculated. With regards to the SLaMA, this is accounted for based on the calculated plastic rotation for each element. The main intent of the redistribution was to see the feasibility of achieving higher NBS rating for the wall elements. With reference to the revised sketches appended to this and accounted for 30% redistribution, it can be observed that the longitudinal walls are scoring low. Therefore, redistribution, won't help with achieving higher NBS% rating, irrespective of its percentage. In the transverse direction, considering the higher level of ductility, and revising the pier labelling in accordance with your email on 15th of June and revising the level of ductility as stated above would help with achieving higher NBS% ratings in most of the cases.	What you have described here is the reason why the results you are getting are inconsistent. You have lines of wall elements rigidly connected together with one of the multitude of elements having a low capacity in flexure. See image attached as an example. This does not appear to constitute failure of the wall system. We would suggest that even if you pinned this pier and the remianing piers would have sufficent capacity. In the results provided it is almost always the smallest piers in a line which 'fail' first. Only these piers begin to yield they will very quickly lose stiffness and the load will redistribute to the stiffer elements adjacent. In this example it appears like complete failure of these piers wouldn't actually consitute any real failure as there are pependicular walls which would continue to maintain grarivuty support of the slab.	We have revised the ratings for the elements based on consideration of further redistribution between the elements. Revised wall rating sketches are attahced for reference.	**Noted, no further comment**	CLOSED
1.6	Seismic Gaps	Structural drawings indicate that the gap provided is less than the ULS drift of the buildings. What consideration has been given to pounding between the two structures in regards to life safety?	The buildings are of similar height, mass and stiffness and the floors are on the same elevations. With referecne to C2B, When adjacent buildings are of similar height and mass and have matching or similar floor levels, it is not expected that engineers need to account for the effects of pounding, irrespective of the provided separation clearances.			**Noted and agreed**	CLOSED
1.7	Seismic Drift	The South building in particualar appears to have a very high interstorey drift over the first level? What is the cause of this and is this an issue (e.g. irregularity of stiffness)? How do the stairs accommodate this kind of movement?	The lower section of the graphs is not referring to the displacement at first level and are showing the deformation of the horizontal springs at the base of the structure. With reference to the Appendix H, it is shown that South building starts from Level 2. We will revise these graphs based on the revised calculations and exclude the spring deformation to better picture the interstorey drifts.			**Noted, this looks much more reasonable now**	CLOSED
1.8	Strengthening Design Intent	Have you discussed the design intent of the strengthening works with the original designer?	Yes, we did. The intent of the strengthening at the time was the primary structural elements for 70% NBS (IL2) considering architectural upgrade requirements.	Based on your assessment have the strengthening works improved the strength of the building or reduced it? The current score is limited by the primary structural elements so the strengthening is directly relevant.	Addition of the new skin walls in 2013 improved the rating of the building compared with its earlier stage.	**Noted, No further comment**	CLOSED

1.9	SLaMA Analysis	As noted, the difference between 2006 and 2017 assessment guidelines is the requirement to undertake non linear checks, even for elastic-based analysis. Refer clause C2.1.1 "A significant change from the previous edition of the 2006 guidelines, is the emphasis on understanding the nonlinear behaviour of the structural systems present even when elastic-based procedures are being used. For this reason, these guidelines recommend using the SlaMA procedure as a first step in any assessment" We have undertaken a SlaMA for a couple of the walls, from this analysis its fairly clear that a PHZ forms at L2 and can achieve a ductility of mu=2, an example shown below. Note our results are indicative and won't be 100% correct as we haven't been able to accurately capture the axial loads on the walls but should capture the behavior.	We have carried out the SLaMA for the buildings with the assumption that the less complex parts of the wall system have the potential of PH formation. While 7% hysteresis damping can be considered in compliance with the ductility of 2, we do not believe that this should be considered simultaneously in conjunction with Sp=0.7. the Sp factor needs to be adjusted in accordance with the expected failure mechanism of the anticipated ductile elements and details incorporated to the model. Noting that when nonlinear analysis is being undertaken the Sp factor is not allowed to be reduced. In addition, the effect of soil flexibility is explicitly modelled resulting in period elongation. Therefore, while in the RSA the Sp is based on the ductility, for the SLaMA, the factor is adjusted as per the above. While we have carried out the SLaMA for being compliant with the Guidelines few things are required to be noted about this: •The buildings are stepped, and the effective height of the building cannot be accurately captured. •We do not believe that the location of hinge formation is being clear as stated above. Location of hinge formation can vary along the few bottom levels of the structure. For the North Building, the hinges will form on Levels 2 and 3 while for the South Building, the hinges are forming at levels 3 to 5. •Where the walls are of more complex geometry, as discussed in the peer review log, the walls will not behave beyond their nominally ductile performance and considering the length of the walls, they are being classified as Squat. •We noted that almost all the walls in the longitudinal directions and 30-40% of the walls in the transverse direction are being governed by their out-of-plane lateral instability (P. C5-90)	We understand that the building is complex to undertake the SlaMA analysis for. In this instance when it was proving difficult to obtain the same mechanism from the Slama and the force based assemsnet this would have indicated that it would likely be more appropriate to move to a pushover analysis. You note the the lateral instability of the trnasverse walls is critical, at what (%NBS) demand does this become an issue?	Refer to 1.4.	**Noted, No further comment**	CLOSED
2.0	**Loading**						
2.2	Rooftop water tank	For the assessment it has been assumed that the water tank is full of water which presumably represents a significant seismic mass at roof level. Has it been confirmed whether the tank is in use and is full of water?	We had a meeting with Peter Mora, briefing him about the outcome of the DSA including the water tank on the 30th of May. He had no objection about the assumptions for the water tank. A separate email is sent to him on the 13th of June to confirm the use of the water tank. We received the confirmation that the water tank is in use as per the snip below from the Operating and Maintenance Manual of Berkeley Dallard. **2.2 DOMESTIC COLD WATER SYSTEMS** The building is a supplied by an existing 75mm diameter mains pressure cold water supply (MPCW) that enters the building in line with grid 3. This supply is from the council water main within Nairn Street and enters the ground floor services tunnel at high level from the duct at the intersection between grids 3 & A. The water is then reticulated through the services tunnel with branches to ducts, two 50mm dia supplies feed the two tanks located on the roof. The MPCW supplies the kitchen sink in each apartment and the hot water cylinders (HWCs) on Levels 9 & 10 only. HWCs on all other levels are supplied via the low pressure water system (LPWS) from the tanks on the roof. Considering the location of the water tank and the identified mode of failure, we do not consider it as life safety concern, though we're required to assess this element as part of the DSA.			**Noted, and we agree that this would not be a life safety concern.**	CLOSED
2.3	Part Loading on L10 Diaphragm	Given that pESA loads have been used elswhere in the building, why were Parts loading being used on the L10 diaphragm? This appears overly conservative when compared to the pESA loads (just one floor below with a significant increase in loads and a step change) used for the other floors given that the plywood provided is reasonably rigid.	Multiple reasons are considered to adopt the Parts loading for this level: -pESA method is developed based on limited research only for reinforced concrete floor diaphragms. -L10 is added later to the building and with reference to the drawings, the stiffness of the building is significantly reducing specially in the Transverse direction, where all other walls are terminated below this level. - A significant portion of the load into the diaphragm elements is based on the out-of-plane loading for the concrete walls which needs to be accounted for using parts loading. - NZS1170.5 C8.1.1 classifies elements such as "Secondary Structures supported by the primary structure such as roofs, ..." as "parts". we believe that the diaphragm at level 10 based on the above explanations fall into this category.	Have you looked at what the accelerations at this level would be when you derive these from a modal response spectrum analysis? This may give you a much reduced loading at the level 10 diaphragm. Accepting the limitations of the methodologies used, however it seems implausible in reality that there is such a drastic step-change in floor accelerations between L9 and L10 which is overly penalising the L10 diaphragm.	Comparing the storey accerlations in the transverse directions are 1.9g and 1.3g for the L10 of the North and South buildings, respectively. While these numbers are 60% and 40% of the parts loading accerlation, it is notable that 69% and 66% of the load into the diaphramg is from out-of-plane loading from the concrete walls in the North and South buildings, respectively. This portion of the load is to be treated as parts as explained earlier. Considering that the critical element for rating the L10 is their connection to the walls, considering the reduced load based on the floor acceleration does not change the rating for this critical element.	**Noted, although it appears overly conservatively the loading is probably the most appropriate given the limitations of the current codes and guidelines. No further comment**	CLOSED

2.4	Retaining Loads	From the calculations provided it is unclear how retaining loads on the building have been considered? Please provide details	The retaining loads are not being imposed on the building. This is explained in part 4.3.2 of the report. These assumptions are validated based on the correspondence with the Geotechnical team at the time of the assessment. Attached is the correspondence with the Principal Geotechnical Engineer.	Has the retaining wall been assesed? Can it carry seismic loads without relying on the building. It seems like a high risk assumption that the retaining wall doesn't load the building given its proximity.	Refer to the attahced calculations and correspondence with Principal Geotechnical Engineer. The relative displacement between the top and bottom of the retainig wall is very small. With reference to any credible reference, such as EN 1997-1 Table C.2 the passive pressure load on the wall is relative to the movement of the wall. Also, Table C4.B.3 is not fully applicable to this case. This clause is for the case where the building is founded on the retained soil and as shown the piles are shorted than the retained height. in this case, the maximum height of the retaining wall is 5.35m while the height of the piles are approximately 9.1m. Also, these retaining walls are not part of the primary structure.	**Noted and thankyou for providing the additional correspondance. Geotechincal engineering is outside our scope of engagement so we take the view of you geotechnical engineers as read, no further comment.**	CLOSED
3.0	**Diaphragms**						
3.1	L10 Floor Diaphragm	Do you have details of the length of the M16 Trubolts used to connect to the walls? This doesn't appear to be on the drawings and will have an impact on the score for these elements.	There is no detail showing the depth of the M16 Trubolts, though based on the wall thickness of 150mm, the embedment depth of this anchors assumed to be 100 mm.			**Noted. No further comment**	CLOSED
3.2	L10 Floor Diaphragm	When assesing an existing building the check that the bolts comply with C1 or C2 as per a new building design does not seem appropriate for Life safety measures. The assessement of the capacity should be undertaken in accordnace with the guidelines. If these bolt demands are compared to the ramset capacity tables it suggests that the capacity would be in excess of 100% NBS.	We do not have this interpretation from the assessment guideline. Page C5-62 of the assessment guideline refers to the outdated ACI318-14 for assessment of post-installed anchors. But also states that " Strength reduction factors for anchors should be implemented following the principles outlined in NZS 3101:2006." It also recommends performing pull-out test on anchors where there is reason to doubt the capacity of anchors. Through the discussion about the pull-out test with Hilti, it is advised that for the Hilti products, those which are not C2 compliant, had a significant reduction in their pull-out capacity. it is expected that this would be the case for the products from other manufacturers. To conclude, we believe Considering the seismicity of the building location, low ratings for the substrate elements, it is very likely that the crack width under seismic loading exceeds 0.8mm threshold. therefore, we believe that the adopted assessment methodology is appropriate. Note: during the NZCLS Seminar on Retrofitting of the precast floor on the 28th of June, the issue of C2 compliance was discussed and there was a general agreement that this is required to be considered where the crack width is possible to exceed 0.8mm threshold.	What is the crack width in the walls at the level in question? In general terms anchors, when tested for C2 loading conditions see a strength reduction in the order of 4. If you made an assumption similar to this you would still find that the anchor capacities have a higher rating than what you have noted. Essentailly you are suggesting that all buildings with post installed anchors, installed prior to C1 and C2 approved anchors being introduced in NZ would be earthquake prone. If this is not the case, based on your review of the strengthening design how was teh use of these anchors justified as compliant in the original design?	Irrespective of the category C1 or C2 and the crack width, the type and diameter of the anchor used is not suitable for this application. The current scope is to deal with the exsitnig structure of Berkely Dallard and in our opinion the connections are not appropriate. We believe the current assigned rating <34%NBS is reflecting that We suggest to carry a site investigation and confirm the diameter of the anchors as alternative T12 at closer spacing is proposed on the drawings. It would be also required to remove at least one of the anchros to ensure about the embedment depth which is currently assumed and is unknown. Also, If you have any evidence of better performance of these specific diameter of anchors tested for the C2 from the specific supplier and for the limited substrate thickness, provide the material for further discussion.	**In our opinion, the Aurecon position on the assessment of post installed anchors is very conservative but can accept that this is their poistion. No further comment**	CLOSED
3.3	RC Diaphragm Assessment	For diaphragm loading in the longitudinal direction, we would expect to see some degree of symmetry across the building (about the blue line below) rather than the significant assymentry that we are seeing. Do you have an understanding of why this is happening? 	With reference to Strengthening Drawing S.25.009, it can be observed that during strengthening, a new 250mm thick wall was added to the existing 8" thick wall along Grid G. This converted the wall to an 8.7m, 450mm thick wall and by far the stiffest element over the entire floor plate in the longitudinal direction. Therefore, as expected the wall absorbed the load and the distribution of floor tie loads is asymmetric.	While we recognise that the strengthened wall will be very strong, the forces in the diphragm are still highly assymetric, and do not appear to align with all of the force tracking to one elements. For example, why are the orthogonal elements in compression at GLA but in compression at GLK for loading in the x direction. We would expect these to be similar and at least the same direction (comp or tens)	The reason for having the large force in GLA lies in two facts: 1) Due to having the thick wall on grid E. More specifically,while most of the load is being channelled to this wall, an inclined strut should form as below. The force in the strut is resisted by the tie elements on GLA and GL9. 2) Spandrel beams exist on GLA and GLJ. These axially stiff elements also tend to grab larger force as compared to other tie elements. 3) The building response is torsional due to the what's explained earlier and also by considering the effect of the transfer forces from the level above. this is explaned further in 3.4. 	**Noted, no further comments**	CLOSED

3.4	RC Diaphragm Assessment	In the longituindal direction the walls along a common grid line appear to be fighting against one another. Do you have an understanding of what is causing this to happen? Do you believe this is realistic?	The length and thickness of the wall elements are varying at every floor and above level 8, there is a significant change in the length of the walls, these changes are causing transfer forces and as a result load from one wall moves to the other.	Is your grillage model a multi storey model? This doesn't appear to be the case and would be a somewhat uncommon way of undertaking this analysis. If it is not multi storey how is it capturing the transfer forces than you note?	The grillage model is 2D. However, the Transfer forces/shear out forces are determined using a 3D model and then manually input in the grillage model. In this 3D model, the lateral force at L8 is equal to the L8 diaphragm load while the lateral forces for other floors are based on ESA but rather scaled in a way that the base shear is constant.	**Noted, no further comments**	CLOSED
4.0	**Secondary Structure**						
4.1	Canopy Parts Coeffcients	West canopy loads appear to have been calculated based on parts, however the structure appears to be an independent structure so parts loading would be overly conservative for this situation and will significantly reduce the seismic rating for life safety.	We agree that the assumption was incorrect, and we updated our calculations for this canopy accordingly. Based on the response to 3.2 and considering the location and lightweight of the structure, we believe that using Category C1 is justifiable and the rating for Canopy will increase to >67%NBS limited by the connection at the base.			**Noted, we are surprised the canopy is not above 100% as we would expect this very lightweight structure to be governed by wind loading.**	CLOSED
4.2	Rooftop Water Tank	As per 2.2 above, if the water tank is no longer used for water storage the seismic rating will be significnatly higher. Please clarify whether the use of the tank has been confirmed.	Refer to 2.2			**Noted**	CLOSED
4.3	Anchors from Diaphragm to Wall	As above, how have the restraints provided by the L10 diaphragm fixings been assessed? This will affect the restraint conditions of the cantilever walls and hence the out of plane capacity.	Refer to 3.2	This is related but the response is not directly covered by your resposne above.	As per 3.2, restraints at L10 are unable to provide sufficient capacity and the wall height will be 5.5m. Irrespective of the ductility for the out-of-plane, the wall capacity does not exceed 34%NBS.	**Noted, taking your opinion on the anchors into consideration, no further comments**	CLOSED
5.0	**Primary Lateral Assessment**						
5.1	Reinforced Concrete Spandrels	You note that if the spandrels had insuffcient shear capacity for 67% NBS they were 'cracked'. What was done to the 'cracked' elements in the model? Why was elements <67% chosen as the point where you would do this? Why not 34% or 100%? Presumably all of the piers and spandrels appropriately have been cracked to begin with? Spandrels with a %NBS score < 67% in shear or flexure will be considered an unreliable load path and cracked section modifiers will be applied within ETABS to redistribute the seismic demands. This process will be iterated until the remaining spandrels are determined to have sufficient capacity for the expected seismic demands.	Initially property modifiers are assigned to all the pier and spandrel elements in accordance with NZS3101, Table C6.5. The 67%NBS was selected based on the desirable performance rating for the building and the previous NBS% target of the building follwing the works in 2012. During few cycles of analysis, the stiffness modifiers of spandrels with unsatisfactory performance are reduced significantly to discount them from the model. 	This is an overly conservative way of accounting for cracking in the spandrels. You essentially discount any capacity in these elements which is overly conservative since the still have capacity after they crack and will continue to carry load and contribute to global building capacity. Given that the building doesn't reach 67% NBS would it be more appropriate to soften off the spandrels at 34% loading as this is the more likely target performnce point.	As discussed, the lowest value of interstorey drifts is limited to 0.7% and for majority of the floors this is over 1%. As the failure of most of the spandrels is shear governed, the maximum plastic rotation before the loss of lateral load support is limited to 0.3%. On the other hand, the Θcb=(1+Lw/Lcb) and in the best-case scenario the value of interstorey drift is to be multiplied to 3 to convert is to the coupling beam/spandrel rotation. Therefore, irrespective of the targeted NBS% rating (34% or 67%) the spandrels fail at the very early stage of lateral displacement. Though due to the existence of multiple walls in each direction, the failure of the spandrels does not lead to loss of support for the slabs. From the life-safety point of view failure of spandrels are not important as they are being supported by the opening lintels below and slab starter bars above. Accordingly, we do not consider the failure of these elements to be a Structural Weakness	**Noted, no further comments**	CLOSED
5.2	Reinforced Concrete Spandrels (Transverse Reinforcement)	You appear to have used the vertical wall reinforcement to calculate the shear cpacity of the spandrels however in a number of locations this reinforcement has been cut and won't be properly anchored. Would you agree that the contribution of these bar should be ignored?	Agreed. Discounting the shear reinforcement contribution will reduce the shear strength of the spandrels and for those spandrels the shear NBS% scoring will be lower. Though as per the above explanation this will not have an impact on the global performance/rating of the structure as these elements have already discounted form the model.			**Noted, however they should not be discouted entirely**	CLOSED

5.3	Separation of Concrete Walls	Why were the walls at the intersections been seperated when they are connected together in reality? How have you done this without fundamentally changing the behaviour of either the longitudinal or the transverse walls?	Walls at their current state have complex configurations, these got even more complex as a result of the strengthening. the thickness and length of walls is changing almost at every level and assessing their capacity based on the current configuration is inconclusive. Majority of the walls in the longitudinal directions are separate from each other as part of the strengthening. This is shown on strengthening drawings S34.011 and S34.012. As explained in the Memo sent on 27th of April, separation of the walls in the transverse direction, will slightly increase the period of the structure by less than 0.1 seconds and in order to avoid underestimating the demand on the walls, the period of the structure is "user-defined" in accordance with the current connected wall configuration. We have also carried out a comparison of mass participation between the two models to ensure that the separation will not have significant impact on MPR and global response of the building. With the revised version with higher level of ductility, we considered the walls to be subjected to the seismic load in their in-plane direction rather than considering biaxial loading for complex walls being part of the two-way resisting system. In addition, we have spot checked that for those walls which are scoring low, consideration of the connected configuration will not improve the rating.	Noted, and understand that it is probably acceptable way of modelling the building. The period of the building given this arrangement appears particularly low given the slender, uncoupled nature of the piers, especially in the transverse direction. What is the mass participation in the first two fundamental modes (we assume these are both translational modes)? Can you provide any commentary as to why the period is so low? The period being low results in conservative assumptions regarding the demand, resulting in lower %NBS scores.	We do not believe that periods reported in the brackets of Table 5-3 and Table 5-4 are low considering the wall lateral resisting system for the buildings. With reference to NZS1170.5 C4.1.2.1, an estimation of the period can be determined. Using the average height of the buildings in the transverse direction yiled the period 0f 0.63 and 0.69 for the South and North Buildings respectively. It can be observed the reported periods has consistency with the values in brackets in Tables 5.3 and 5.4 . Also, it should be noted that the length of the walls in these buildings as the main contributor to stiffness is far greater than the length of the wals in a to-date design. using the more accurate Kt as per the clause above will provide a lower period for both directions of the building. For both of the buidlings, the first two fundamental modes are translational. but considering the torsioanl sensitivity of the building, each of these modes has a considerable mass participation in both translational directions. for the North Building the MPR in the first two modes in approximately 75% in both directions. for the South Building the MPR in the first two modes is 75% in the longitudinal and 89% in the translational direction.	**Noted, no further comments**	CLOSED
5.4	Chosen Ductility	What is the basis for selecting the ductility of 1.25? How have you assesed the ductility capacity of the system?	Refer to 1.3	Refer to reposnses above. The SLaMA doesn't appear to identify the likely global mechanism.		**Noted, refer to commentary above**	CLOSED
5.5	Capacity of Walls	How have you calculated the capacity of walls with both Grade 300 and Grade 500 reinforcing? Have you checked the ultimate strain in the two types of bars?	Yes, we have calculated the capacity of the walls based on the equivalent area of reinforcement. Noting that with reference to the C5, table C5.4, the εsu for grade 300 is 0.15 and for Grade 500 is 0.10 while this is limited to 0.06 in our calculation as per the Guideline. Also, based on the revised ductility consideration, all the strengthened walls are rated 100%NBS. Therefore, strain incompatibility is not an issue.			**Noted, no further comments**	CLOSED
5.6	Calculation of Spandrel cpacities.	The method that was used to calculate the spandrel capcities appears to be significantly overestimate the flexural capacity. An independant calculation based on the example calc you have done in section 7.3 of the south buidling calculatons find the spandrel to have a capacity which is < 1/3 of the capacity that you calculated making the beam much more flexurally governed.	We agree that there was a cell reference error in our spreadsheet and rectified that. Though considering comment 5.2, this does not have an impact on the global response of the building.			**Noted, no further comments**	CLOSED
5.7	Calculation of Pier Capacities	As above the method for the flexural capacity of the piers does not appear to be fully correct. It would not be quite appropriate to simplify the flexural capacity calculation in this way.	This is not the case for the flexural capacity of piers. Considering that in most of the elements the reinforcement is uniformly distributed along the length of the element, the simplified methos provides is accurate enough to estimate the capacity of the elements. we spot checked few elements and the difference in the worst case was ~6%.	The method you have used is far to simplistic for a **detailed** assesesment. This simplication will give an approximate capacity which would be suitable for concpetual design purposes to get a ball park figure. We would expect that at a minimum if this approach is adopted that more accurate calculations are undertaken for critical elements. Particulary when this may influence whether the element is close to the 34% threshold.	As can be seen for the critical elements, where the demands are in the order of 1000KN.m to 11000kN.m the diffrence is in the order of 1% to 3%. In addition, for doubly reinforced shear walls and based on the agreed assumption of full load redistribution between the walls there is no element with the rating below 34% NBS. The elements in the longitudinal direction are just above this threreshod. therefore, we do not think further detailed assessment makes any step change in these walls.	**Noted, no further comments**	CLOSED
5.8	Overstrength Capacity of Walls	We have reviewed what we discussed around overstrength of flanged wall and can't see how you get such a large overstrength factor. Based on what you showed on the screen during the meeting you appeared to be calculating the overstrength factor by comparing a nominal demand to the overstrength capacity. This is not appropriate for an assessment and should be calculated based on the overstrength capacity vs the nominal capacity for a given axial loading and direction. Based on the fo = 1.35, provided in the guidelines for G500 reinforcing (or fo = 1.25 for G300 reinforcing) it should be possible to get an overstrength of 1.35 or 1.25 (depending on the reinforcing)	The above argument is only valid for cantilever walls. What is stated in only material overstrength and is only valid where the cantilever wall is being assessed and in the first place, it is designed for the strength matching the demand. Please refer to the response to Q. 1.3 regarding ductility and additional clarification provided via example.	The way you have deomnstrated this with the calc is not correct. The overstrength is only relevant to the yeilding element therefore the overstrength capacity should be compared to its nominal capacity, as if this is limiting the capacity of the building, this should be equal to the demand in a given loading direction. An overstrength value of the number you have calculated indicates that either the building is greater the 100%NBS or the mechniasm has not been correctly identifed.	With reference to the additional calculation, and color coding for the walls, it can be observed that in the transverse direction (flange direction) of the walls, majority of the walls are not yielding and have the capacity >100%NBS. So, the results of the RSA analysis based on ductility of 2 are validating what's been explanied about the wall capacity in the flange direction.	**Although we don't agree with the way that Aurecon have calculated the overstrength factors for the walls, we don't belive further discussion on this point will yield a material differnece to the seismic rating. No further comment**	CLOSED
5.9	Dynamic Magnification	Regarding your comments around dynamic magnification factors during our first meeting, as per section C2.5.10 of the guidelines, this is only required if considering cantilevered shear wall with a ductility of 3 or more so would not be required for mu = 2.	Based on the above and with the assumption that the ductility close to 2 somehow can be achieved for the structure, we did not consider the dynamic magnification factor and only amplified the shear by an overstrength factor slightly greater than the material overstrength.	Please clarify why you have mangnified the shear by a factor greater than the material overstrength? If you have ideifed that a plastic hinge forms at the base, the strength beyond this point can only be increased by the material overstrength	In the transverse direction as stated above and specially for the South building it can be observed that the walls are rated >100%. Therefore, we adopted an overstrength of 1.5. This does not alter the ratings.	**Noted, no further comments**	CLOSED
5.1	Pier Labelling	Pier labelling. As discussed, you should also review how you have labelled the piers in the etabs model if this is going to be used to provide meaningf results, as an example for the wall shown below, the pier shown below should have a single label as they will behave as a single wall. We would expect this to improve the scores of these elements as the tensions in the small pier are indicative of a ductile failure mechanism.	The pier labelling is reviewed and where applicable, the wall sections at either side of the openings are assigned the same pier.		Noted, Few locations like what is shown on this figure are spotted on Grirds 4, 10 and 15. Though, for these piers the capacity exceed demand and the current ratings are >100%NBS(IL2).	**Noted, this appears to be mostly correct now, no further comments**	CLOSED

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