Frank Kitts Carpark DSA
Detailed Seismic Assessment

Frank Kitts Carpark
15 Jervois Quay
Wellington

Detailed Seismic Assessment Report

Holmes Consulting

Revision 5
11 December 2020
105978.19

Holmes Consulting

Level 2, Chartered Accountants House, 50 Customhouse Quay
PO Box 942
Wellington 6140
holmesconsulting.co.nz

Detailed Seismic Assessment Report
Frank Kitts Carpark DSA

Prepared For:
Wellington City Council

Date:	11 December 2020
Project No:	105978.19
Revision No:	5

Prepared by:

May Teoh
GRADUATE ENGINEER
Holmes Consulting LP

Prepared by:

Rolf Veldboom
DESIGN ENGINEER
Holmes Consulting LP

Reviewed by:

Panos Armaos
SENIOR PROJECT ENGINEER
Holmes Consulting LP

Reviewed and Approved by:

Ian Hills
PROJECT DIRECTOR
Holmes Consulting LP

105978.19DSA2211.002.docx

Report Issue Register

DATE	REV. NO.	REASON FOR ISSUE
8 / 11 / 19	0	DRAFT - For internal communication
22 /11 / 19	1	DRAFT - For internal review
6 / 12 / 19	2	DRAFT - For client review
3/2/2020	3	Issued to Client
24/2/2020	4	FINAL – Minor update for submission to local authority
11/12/2020	5	FINAL – reissued to include appendix B (omitted in rev 4) and to correct Table 1.

CONTENTS

TABLES

FIGURES

EXECUTIVE SUMMARY

Background

Holmes Consulting LP have been engaged by Wellington City Council to complete a Detailed Seismic Assessment (DSA) of Frank Kitts Carpark, at Jervois Quay, Wellington. The DSA has been completed in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017), referred to as the "Engineering Assessment Guidelines" within this report.

Building description

The carpark is a single storey building. The building was designed and constructed in 1989. Its main purpose is to function as a carpark building where the roof is used as a public park. However, it is nowadays also used for to hold weekend markets and festival events.

The floor plan of the carpark comprises No.9 bays in transverse direction (E-W) and No.10 bays in longitudinal direction (N-S). The structure is formed by precast concrete wall panels around the perimeter, precast concrete frames and a flooring system comprising precast concrete double-tee units and cast in-situ slabs. The structure has an approximate area of 3500m^2.

Assessed seismic rating

The results of the DSA indicate the building's seismic rating to be 15%NBS (IL3) assessed in accordance with the Engineering Assessment Guidelines. The seismic rating assumes that Importance Level 3 (IL3), in accordance with AS/NZS 1170.0.2002, is appropriate, since it allows for more than 300 people to congregate during weekend markets or festival events.

Identified structural weaknesses

Structural weaknesses are those elements included in our assessment (the failure of which would constitute a significant life safety hazard) which scored 15% NBS (IL3) or which require other attention. The Critical Structural Weakness is the lowest scoring of these elements. The assessment identified the following Structural Weaknesses (SWs) in the building:

Table 1 - Structural weaknesses

Building Element	Structural Weakness (SW)	%NBS(IL3)
Roof diaphragm	The roof diaphragm contains non-ductile mesh which cannot be relied to provide a load path for the diaphragm forces. Further commentary on the performance of the diaphragm is provided in Sections 5.3 and 10C.4.	15%
Connection between diaphragm and lateral load resisting system	The precast concrete wall panels are connected to the diaphragm at the roof level by starter bars. These starter bars are insufficient for the required shear transfer.	15%
Reclamation fill seismic performance and soil-structure interaction	Onset of liquefaction and lateral spread displacements imposing significant displacements demands on the foundations and structure. This may lead to undesirable behaviour in the superstructure and potential for collapse.	20-30%

Building Element	Structural Weakness (SW)	%NBS(IL3)
Precast concrete wall panels	Yielding of the starter bars of the precast concrete wall panels for in-plane flexural demands. The capacity is limited by the tensile strain of the starter bars and the inelastic rotation capacity of the precast concrete wall panels.	40%*
Double-tee units	The double-tee units forming the roof slab are web-supported. These, when subjected to movement may exceed the capacity of the supporting concrete members in bearing. _Note:_ Visual observations revealed several double-tee units where the seating support had already spalled. The remaining seating of these units may be insufficient during large seismic events and would consequently rate lower than 100%NBS (IL3).	100%

- Shown value corrected in revision 5 to be as per table 3 and as elsewhere in this report.

The Critical Structural Weakness

The Critical Structural Weakness (CSW) is the lowest scoring structural weakness, the failure of which would result in a significant life safety hazard. The CSW was found to be the roof diaphragm that distributes the loads between the lateral load resisting systems. The roof diaphragm is reinforced with non-ductile mesh and consequently does not provide a reliable load path between the lateral load resisting systems.

Further commentary on the performance of the diaphragm is provided in Sections 5.3 and 10C.4.

Engineering assessment summary report

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

1 INTRODUCTION AND BACKGROUND

Holmes Consulting LP have been engaged by Wellington City Council to complete a Detailed Seismic Assessment (DSA) of Frank Kitts Carpark, at 15 Jervois Quay, Wellington.

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

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

A DSA is also used to determine whether or not a building is Earthquake-prone in accordance with the definition for an Earthquake-prone building (EPB) in the New Zealand Building Act.

2 SCOPE

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

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

The scope of work for this project included the following:

- Undertake inspections of the building where required to visually assess and confirm existing conditions of structural elements.
- Undertake a Detailed Seismic Assessment (DSA) of the building using hand calculation methods and 2D structural modelling with supporting calculations and evaluation.
- Collect and analyse existing geotechnical information to assess the response of the site during seismic events that induce liquefaction/lateral spreading of the surrounding soils and how these may affect the structure's performance.
- Identify other geotechnical risks that may affect the structure's performance.
- Prepare a DSA report outlining the assessed seismic rating, summary of likely seismic performance and risks, and also describing analysis methodology, modelling parameters. Include commentary on possible strengthening options to improve performance where appropriate.
- Consideration of the building's secondary-structural and non-structural 'parts' that are classified as "parts that pose a significant life safety hazard".

3 TERMINOLOGY AND KEY DEFINITIONS

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

- **Earthquake rating** – The rating given to a building as a whole to indicate the seismic standard achieved in regard to human life safety compared with the minimum seismic standard required of a similar new building on the same site. Expressed in terms of %NBS.
- **Ultimate Limit State (ULS) shaking demand** – the shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members for the same site.
- **Percent New Building Standard (%NBS)** – The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage.
- **Structural Weakness (SW)** – An aspect of the building structure and/or foundation soils that scores less than 100%NBS.
- **Severe Structural Weakness (SSW)** – A defined SW that is potentially associated with catastrophic collapse and for which the capacity may not be reliable assessed based on current knowledge.
- **Critical Structural Weakness (CSW)** – The lowest scoring structural weakness determined from a DSA.

4 BUILDING DESCRIPTION

The Frank Kitts Carpark building is a single storey carpark structure located on Jervois Quay, Wellington. The building was designed and constructed by 1989. Its main purpose is to function as a carpark building while the roof is used as a public park/open space. The carpark is also used to hold weekend markets and outdoor festivals in the public space above the carpark.

The plan of the carpark comprises No.9 bays in transverse direction (E-W) and No.10 bays in longitudinal direction (N-S). The structure is formed by precast concrete wall panels around the perimeter, precast concrete frames and a flooring system comprising precast concrete double-tee units with a cast in-situ topping.

The flooring system forms the roof to the structure which supports a layer of soil and pavement. The concrete walls extend above the roof around the perimeter forming a concrete upstand.

Circular concrete planter boxes are provided to allow for trees and large bushes. Concrete stair elements are provided at various locations to provide access to the roof structure and in the South-West corner there is access from a footbridge that goes across Jervois Quay.

The precast wall panels and precast concrete frames are supported by concrete foundation beams and pads that are founded on a combination of bored concrete piles and driven precast concrete piles. There is no structural ground floor slab in the building, the bases of the columns are tied together by small concrete tie beams.

Figure 1 - Extract from Google Maps - Frank Kitts Carpark location

4.1 Gravity load resisting system

The floor system is typically formed by precast concrete double-tee units with insitu topping, forming the roof to the building. Around the circular planter boxes and at locations around the concrete staircases the slab is executed as a cast in-situ slab. The double-tee units span between the precast concrete gravity frames and perimeter walls in the North-South direction.

Gravity frame columns are typically sized 300x600, supported by reinforced concrete foundation pads and driven precast concrete piles. The precast concrete wall panels are supported on reinforced concrete foundation beams founded either precast concrete driven piles or reinforced concrete bored piles.

4.2 Earthquake (lateral load) resisting system

Lateral loads are distributed by the suspended concrete roof diaphragm to the precast concrete wall panels at the perimeter of the building then into the wall foundations. There is no structural ground floor slab provided to the building, the base of the columns being linked together with relatively small tie beam members.

The precast concrete frames, in the East-West direction, are not considered part of the primary lateral load system, intended to support gravity loads only.

4.3 Foundations and subsoil

The Geotechnical Assessment is attached in Appendix B. It states that the ground model consists of approximately 12m of loose gravelly fill overlying dense to very dense alluvium.

Existing borehole investigations were not sufficient to differentiate the site subsoil class between C and D. Further geotechnical assessments indicated that the site natural period may lie between a C and D site. Therefore, the recommendation is to adopt a D site, being the more conservative site subsoil for the assessment of the structural response.

The ground water level was determined to be approximately 1.8m below the ground floor level of the structure. However, it will obviously be fluctuating with tidal movements in the harbour.

Liquefaction is considered possible within the loose gravelly fill layer. It was also found that the behaviour of the liquefied soil undergoes a significant step change between 20% and 30% of ULS seismic loading. The consequence of the step change is connected with the expected cyclic displacements and lateral spread. Following the step change the lateral spread displacements at the site may be between 1m to 5m.

The step change in the geotechnical behaviour may result directly into a step change in the structural behaviour, as it does lead to increased demands on the foundation elements of the structure. This is further described in Sections 5.3 and 10C.8.

Therefore, in accordance with Section C4.5.3.2 of the Engineering Assessment Guidelines, the seismic response of Frank Kitts Carpark is considered to be structurally and geotechnically interactive in accordance with the Engineering Assessment Guidelines.

5 ASSESSED SEISMIC RATING

We have assessed the seismic rating for the Frank Kitts Carpark as 15%NBS(IL3)

The results of the DSA indicate the building's seismic rating to be 15%NBS (IL3). The performance of the structure is governed by the roof diaphragm. The diaphragm is reinforced with non-ductile cold drawn mesh which is not considered to provide a reliable load path between the lateral load resisting systems.

Further commentary on the performance of the diaphragm is provided in Sections 5.3 and 10C.4.

Relative earthquake risk compared with a new building

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

Table 2: Grading system for earthquake risks relative to a new building

Percentage of New Building Standard (%NBS)	Grade	Approx. risk relative to a new building	Life-safety risk description
>100	A+	Less than or comparable to	Low risk
80-100	A	1-2 times greater	Low risk
67-79	B	2-5 times greater	Low to Medium risk
35-66	C	5-10 times greater	Medium risk
20-34	D	10-25 times greater	High risk
<20	E	25 times greater	Very high risk

The building is potentially Earthquake-prone

A building with an earthquake rating less than 34%NBS fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-prone Building (EPB) in terms of the Building Act 2004.

A building rating less than 67%NBS is considered as an Earthquake Risk Building by the New Zealand Society for Earthquake Engineering.

As such, Frank Kitts Carpark is categorised as a potential Earthquake-prone Building.

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

The seismic rating of 15%NBS(IL3) requires the provisions of the Earthquake-prone Building requirements of the Building Act 2004 to be met. More information on this can be found in Section 8.

5.1 Identified Structural Weaknesses

A Structural Weakness (SW) is any aspect of the building structure and/or foundation soils that scores less than 100%NBS (IL3). Table 3 lists all the Structural Weaknesses (SW) identified in the Frank Kitts Carpark.

Table 3 - Structural Weaknesses identified in the DSA

Building Element	Structural Weakness (SW)	%NBS(IL3)
Roof diaphragm	The roof diaphragm contains non-ductile mesh which cannot be relied to provide a load path for the diaphragm forces. Further commentary on the performance of the diaphragm is provided in Sections 5.3 and 10C.4.	15%
Connection between roof diaphragm and lateral load resisting system	The precast concrete wall panels are connected to the diaphragm at the roof level by starter bars. These starter bars are insufficient for the required shear transfer.	15%
Reclamation fill seismic performance and soil-structure interaction	Onset of liquefaction and lateral spread displacements imposing significant displacements demands on the foundations and structure. This may lead to undesirable behaviour in the superstructure and potential for collapse.	20 – 30%
Precast concrete wall panels	Yielding of the starter bars of the precast concrete wall panels for in-plane flexural demands. The capacity is limited by the tensile strain of the starter bars and the inelastic rotation capacity of the precast concrete wall panels.	40%
Double-tee units	The double-tee units forming the roof slab are web-supported. These, when subjected to movement may exceed the capacity of the supporting concrete members in bearing. Note: Visual observations revealed several double-tee units where the seating support had already spalled. The remaining seating of these units may be insufficient during large seismic events and would consequently rate lower than 100%NBS (IL3).	100%

Other elements that could present a significant life safety hazard were assessed as having strength and deformation capacities above 100% NBS (IL3).

The Critical Structural Weakness was found to be the diaphragm

The Critical Structural Weakness (CSW) is the lowest scoring structural weakness, the failure of which would result in a significant life safety hazard. The CSW was found to be the diaphragm that distributes the loads between the lateral load resisting systems. The diaphragm is reinforced with non-ductile mesh and does not provide a reliable load path between the lateral load resisting systems.

Further commentary on the performance of the diaphragm is provided in Sections 5.3 and 10C.4.

5.2 Secondary structural and non-structural elements

The secondary structural and non-structural aspects (SSNS) that were considered in this assessment are listed in Table 4. Consideration includes a decision to include or exclude these elements from an overall %NBS Assessed Seismic Rating. The intent of the EPB Methodology is to identify and rate those building parts which, should they lose support or collapse, would present an unavoidable danger that a number of people are exposed to.

Section 2.4.1 of the EPB Methodology is applied, using the guidance in Section A4.3.2 of the Engineering Assessment Guidelines, to determine which parts are included or excluded from the rating. These items are discussed in more detail in Section 10C.9.

Building contents are not considered in this assessment.

Table 4 - Secondary structural and non-structural elements

Building Part	Included in this assessment
Circular planter boxes at several locations in the structure	No
Masonry walls in boat sheds on the Southern end of the structure	No
Statue/Flag/Pole on roof of structure	No
Look-out tower (the Oriel) at the South-West corner of the building	Yes
Wall cap on the perimeter wall	Yes

5.3 Commentary on structural weaknesses

Comparison between original design requirements (1989) and current requirements

The original design calculations and assumptions from the 1989 building consent documentation were available for review during the assessment process. This provided valuable insights in the level of seismic actions that were used for the original design of the structure and to compare these against current design loading.

Frank Kitts Carpark was designed in accordance with the seismic design actions specified in NZS4203:1984 while current seismic design actions are specified in NZS1170.5:2004.

The difference between the seismic design coefficient from each standard are illustrated in Table 5 and Figure 2. It can be seen that the building was originally designed for seismic loads which are approximately 19% of the current seismic design loadings.

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

Parameter	NZS 4203:1984			NZS1170.5:2004		
Ground conditions	C	0.15	Flexible soil T < 0.45s Zone A	$C_h(T)$	3.0	Sub-soil class D T < 0.4s
Structure's fundamental period						
Location				Z	0.4	Wellington
Risk factor	R	1.0	Category 4 [1]	R_u	1.3	Importance level 3
Structure type	S	2.0	Case 6 [2]			
Structure materials	M	0.8	Reinforced non-prestressed concrete			
Near-fault factor				N	1.0	T < 1.5s
Structure ductility				k_μ	1.143	Structural ductility factor approximated at μ = 1.25
Structural response factor				S_p	0.925	
Seismic design coefficient	C_d	**0.24**	C · S · R · M	C(T)	**1.26**	$\frac{C_h(T) \cdot Z \cdot R_u \cdot N \cdot S_p}{k_\mu}$

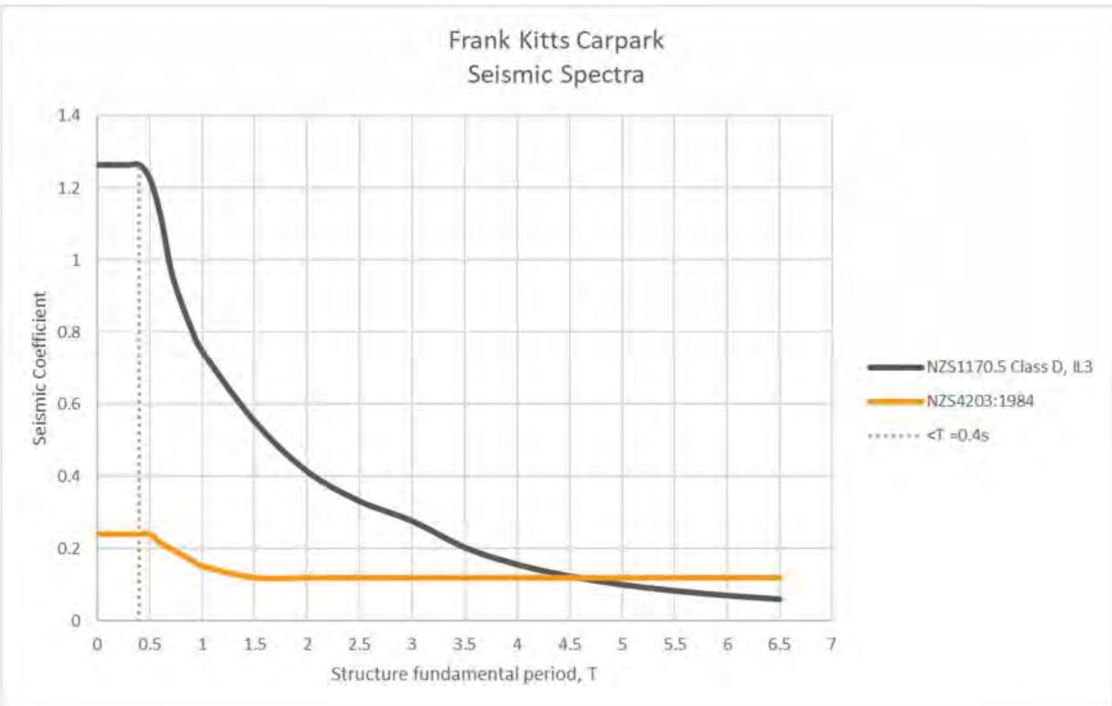


Figure 2 - Seismic spectra comparison between NZS4203:1984 and NZS1170.5:2004

[1] In NZS4203:1984, category 4 buildings included 'buildings with normal occupancy or usage'.

[2] In NZS4203:1984, case 6 buildings included 'single-storey cantilevered buildings supported by face loaded walls constructed of reinforced masonry or concrete.

The capacity of the diaphragm

The roof diaphragm consists of 65mm thick concrete slab that is reinforced with 665 mesh – a cold drawn wire mesh product. The roof diaphragm is required to distribute seismic inertia load of the structure to the lateral load resisting systems.

Cold drawn wire mesh has a non-ductile or brittle behaviour, with low elongation capacity, under tension. Non-ductile mesh has been observed to perform poorly in floor/roof diaphragms in previous New Zealand earthquakes such as the 2010/2011 Canterbury and 2016 Kaikoura events. Failure and fracture of this mesh was observed in buildings in Christchurch and Wellington from these earthquake events.

The 2017 Engineering Assessment Guidelines do not directly comment on the appropriate method to assess diaphragms with this type of non-ductile reinforcing. A crude analysis was undertaken to estimate the performance of the diaphragm, however this resulted in a very low rating of the diaphragm which may be not representative of the actual situation. More comprehensive analysis is not discussed in Chapter C5 of the Engineering Assessment Guidelines, and therefore alternative methods have been considered.

Reference to the latest industry best practice, captured in ASCE 41-17 and the proposed revision of Chapter C5 of the Engineering Assessment Guidelines ("Yellow C5") has been made to provide commentary on the expected seismic performance of this diaphragm. Yellow C5 and ASCE 41-17 state that an elastic analysis can be performed where the total strain captures the effects from the segmental nature of the individual components (e.g. the effects from shrinkage and creep are included in the assessment of strains induced in the mesh reinforcement). Furthermore, Yellow C5 states that 'non-ductile' mesh should not be relied on in a strut-and-tie type of diaphragm analysis.

This provides no solution, since an analysis that includes the effects from shrinkage and creep is deemed impractical and expected to not result in reliable results. This is mainly due to the large number of side-effects that could significantly affect the parameters in the analysis. Attempting to identify and evaluate all these parameters is impractical.

Therefore, the capacity of the diaphragm is considered inconclusive and the Engineering Assessment Guidelines require this to be rated at 15%NBS (IL3)[3].

Performance of foundation system

The lateral load resisting system can be split into bracing lines. A bracing line is a gridline that contains one or more lateral load resisting systems such as the precast concrete wall panels. The distribution of the lateral load between the bracing lines at roof level has been assumed to be through a rigid diaphragm. And has included the effects of accidental eccentricity.

In the assessment of the load paths of the structure it is considered that each bracing line takes a portion of the seismic loading relative to its stiffness. In the analysis, this is done by determining the eccentricity of the shear centre relative to the centre of mass. This results in gridlines with a lot of precast concrete wall panels (such as gridline 8) to take a relatively large portion of the seismic actions, since these are relatively stiffer. For reference, this also results in the precast concrete frames taking nominal earthquake loads since they are much more flexible.

Each bracing line then transfers the lateral load to its foundation and ultimately the base shear is taken out by the piles. On the ground floor level however, there is no positive diaphragm to redistribute the forces to

[3] The Engineering Assessment Guidelines recommend in Section A8.2 that scoring elements below 15%NBS has no practical meaning and may provide an erroneous indication of expected performance. Therefore, it is recommended that a scoring or rating is not quoted as less than 15%NBS.

other gridlines. Therefore, the relative stiffness of the bracing lines above the foundation system determines the load transfer to each individual pile.

This assessment accounts for the participation of both the bored concrete piles and the precast concrete driven piles in the lateral load resisting system. And the assessment process further accounts for inelastic displacement capacity of the foundation system, since connections to the foundation beams allow for a ductile response. Other effects, such as friction between the foundation beams and the ground or passive soil pressures were found to contribute minimally in the assessment.

The assessment methodology is contrary to the assumptions made in the original 1989 design. From the calculations of the consent documentation it is found that the total base shear of the entire structure is assumed to be divided equally over all bored concrete piles, irrespective of their position in the structure. The precast concrete driven piles were not considered in the lateral load resisting system. Inelastic behaviour of the members was also not considered.

From the site reports and the as-built drawings it is found that the location of the bored concrete piles is generally in the Western side of the building. In the Eastern side of the building some bracing lines (such as gridline H) are equipped with precast concrete driven piles only. This represents a significant deviation of lateral load distribution philosophy between the original design and the assessment methodology used here.

The assessment process incorporated in this DSA is considered to be much more comprehensive but results in yielding of the piles at lower levels of seismic shaking then what was assumed in design. Ultimately though, the performance of the foundation system is limited by the geotechnical behaviour as outlined below.

Performance of structure during liquefaction of the ground profile

The geotechnical assessment concluded that during a seismic event, liquefaction can occur up to a depth of approximately 11m below the foundation pads and beams. Since the piles have a length of approximately 12m, this essentially means that during a seismic event the piles will only have an embedment in non-liquefied ground of 1m.

The geotechnical assessment further states that liquefaction occurs even at very low levels of seismic shaking and that a significant geotechnical behaviour step-change may be expected at seismic shaking of 20-25% of ULS accelerations. This step-change may induce significantly larger cyclic displacements and lateral spread.

To assess the foundations of the structure during a seismic event that triggers liquefaction, 4 scenarios are considered. The geotechnical investigations show that once the seismic shaking reaches the level of shaking that triggers the step-change, then the liquefied scenarios 2, 3 and 4 are expected to significantly impact the structure. The scenarios that are considered are:

1. 100% of inertia combined with non-liquefied soils
2. 100% of inertia combined with liquefied soils
3. 80% of inertia combined with liquefied soils and cyclic displacements
4. Liquefied soils combined with lateral spread

These scenarios are selected based on the guidance by design manuals such as NZGS/MBIE: Module 4: 2016 and the NZTA Bridge Manual version 3.3: 2018. These indicate that it is unlikely that the full dynamic response of the building would be superimposed simultaneously with the full kinematic loading on the pile.

The first scenario is unlikely to occur due to the high proneness of the ground to liquefy. However, the soil may respond initially without liquefaction and the 4 scenarios could occur in sequence. If this is the case, then the assessment shows that the piles would hinge at approximately 10% of ULS seismic loading. The pile has the ability to deform inelastically however and can perform at a higher level of loading, if the ground doesn't liquefy.

When the ground does liquefy (scenario 2, 3 and 4), then the pile is essentially only supported in its embedment 11m below the foundation pads and beams. This results in period elongation and may reduce the seismic inertia from the structure. The embedment length of the pile in approximately 1m of soil is however not enough to generate fixity. And this means that when the piles hinge at the top, there is no reliable load path that provides lateral restraint to the structure.

Furthermore, the embedment length of approximately 1m does not provide a large restraint. It may therefore equally happen that the capacity of this pocket is exceeded and that the pile may 'pop' out of the shallow embedment. Even if the pile has not hinged at the top yet, the consequence of this is similar, as the pile has no reliable load path anymore that provides lateral restraint to the structure. Both resulting scenarios are illustrated in Figure 3.

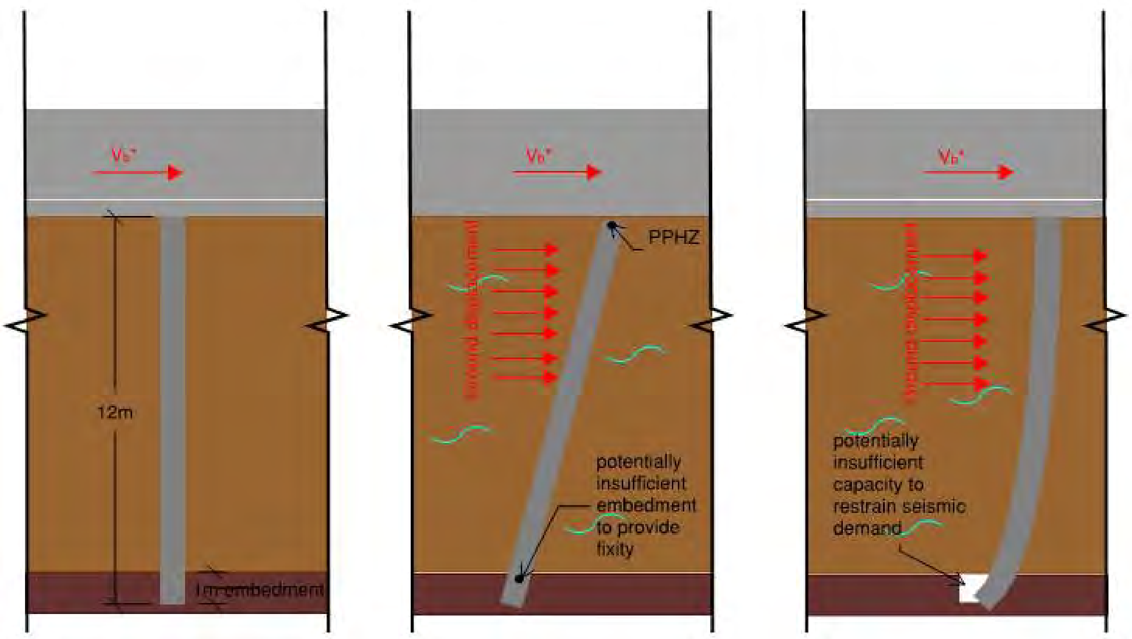


Figure 3 - Illustration of pile behaviour under liquefaction scenarios

where: V^*_b describes the inertia of the structure itself
Ground displacements are caused by cyclic displacements or lateral spread
PPHZ refers to the location of a potential plastic hinge zone in the pile

The assessment has found that there are a lot of uncertainties that come with both the capacity of the structure, particularly the piles, and the demand under liquefied ground conditions. One of the critical parameters that is possible to be determined with more certainty, however, is the step-change in geotechnical parameter that occurs at 20-30% of ULS seismic demands. Exceeding this level of shaking may trigger large cyclic displacements and lateral spread of up to 5m. For this reason, the assessment concludes that the capacity of the piles is 20-30%NBS (IL3) which is limited by the structural performance/capacity of the piles following the onset of liquefaction and lateral spread.

It is noted that a step change in the geotechnical that leads to a step change in the structural behaviour that may lead to a collapse event is required to lower the performance of the structure by a factor of 0.5. However, as described in the Geotechnical Assessment in Appendix B, there is sufficient certainty from the geotechnical assessment to not use this factor in the performance assessment of the foundation elements based on the performance of the structure and the ground during the 2016 Kaikoura Earthquake event.

6 BASIS OF THIS REPORT

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

6.1 Information available for the assessment

The assessment has been based on the following information:

- 1989 complete set of architectural and structural drawings of Frank Kitts Carpark from original designers at BECA
- Construction specifications from 1989 prepared by original designer BECA
- Calculations package from 1989 prepared by original designer BECA
- Incomplete set of shop drawings of precast elements from 1989
- Load assessment report from 2018
- Geotechnical report by Holmes Consulting LP dated 8/11/2019 – Appendix B

The construction drawings appeared to be repeatedly updated during the design process and are considered to be close to the as-built status. However, minor conflicts were found between the consent package, the construction drawings and the shop drawings. Where these conflicts were observed the following order of priority was given to the available information:

1. Site observations
2. Shop drawings
3. Construction drawings
4. Consent documentation

The following information was <u>not available</u> for the assessment:

- Shop drawings of the precast concrete wall panels
- Shop drawings of the double-tee units

6.2 Site observations and investigations

A visit to the Frank Kitts carpark was carried out in September 2019. The purpose of the visit was to visually observe important structural and seismic characteristics, assess their condition and note obvious deficiencies or differences from the documentation. Particular locations that were investigated in more detail were:

- General walkovers were undertaken to confirm the global geometry of the Frank Kitts carpark and to identify secondary and non-structural elements that would influence the assessment of the carpark.
- The connection between precast shear wall panels.

The as-constructed structure appears to be generally consistent with the available drawings. No significant modifications appear to have been made to the primary lateral or gravity systems.

Material testing has not been completed. Material properties based on values in the original documentation are considered sufficient for this assessment. Our observations have been visual only.

6.3 Earthquake demand used in the assessment

The Ultimate Limit State (ULS) seismic demand is derived from the New Zealand Structural Design Actions Standard, NZS1170.5:2004 Incorporating Amd. 1 (Standards New Zealand, 2016). Factors that affect the earthquake demand are described below.

Building importance level

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

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

Seismic hazard zone

Wellington CBD is located in a zone of relatively high seismicity. The Wellington Fault passes within 400 m of the site. We have used a hazard factor of Z=0.40 and near fault factors in accordance with NZS1170.5:2004.

Site and subsoil class

Seismic loads are also dependent on the soil type a structure sits on. In accordance with the recommendations from the geotechnical assessment, a Site Soil Class of D has been used.

7 METHOD OF ASSESSMENT

The method of assessment is selected in accordance with the recommendations of Section C2 of the Engineering Assessment Guidelines, as summarised in Table C2.1 of those Guidelines.

Equivalent Linear Static analysis

The equivalent static lateral force method is a simplified technique to substitute the effect of dynamic loading of an expected earthquake by a static force distributed laterally on a structure for design purposes. The total base shear is evaluated in the two horizontal directions parallel to the main axes of the building. It assumes that the building responds in its fundamental lateral mode. The building is low-rise and is fairly symmetric. Therefore, torsional movement due to ground motions can be captured with reasonable confidence. The use of equivalent static method is used for the primary and secondary and non-structural elements in the assessment.

The analysis has assumed a structural displacement ductility of $\mu = 1.25$. This simulates a nominal ductile structure and allows for minimal inelastic behaviour in the structure. The analysis has shown that there is a number of elements that respond with minimal inelastic deformations which coincides with this assumption.

Further information/technical details on seismic loads, analysis procedures and evaluation can be found in 10Appendix C.

8 STATUTORY REQUIREMENTS FOR EXISTING BUILDINGS

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

Frank Kitts Carpark is considered potentially Earthquake-prone, and therefore, the relevant sections of the New Zealand Building Act 2004 and the Building (Earthquake-prone Buildings) Amendment Act 2016 that need to be considered in relation to the building's structure and strength are set out as follows.

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

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

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

Key features of the act are described below:

Earthquake-prone buildings

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

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

Seismic risk locations

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

- **High seismic risk – Z factor greater than or equal to 0.3**
- Medium seismic risk – Z factor between 0.15 and 0.3
- Low seismic risk – Z factor less than 0.15

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

Priority buildings

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

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

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

Frank Kitts Carpark is not classed as a High Priority Building.

Timeframes for identifying potentially Earthquake-prone buildings

TAs need to assess and identify potentially Earthquake-prone buildings as outlined below.

- **High seismic risk areas:**
  - High Priority buildings January 2020
  - **All other buildings July 2022**
- Medium seismic risk areas:
  - High Priority buildings July 2022
  - All other buildings July 2027
- Low seismic risk areas:
  - All buildings July 2032

Frank Kitts Carpark is considered to be in a high seismic risk area and is a low priority building. Therefore, it needs to be assessed as a potential Earthquake-Prone Building prior to July 2022.

Earthquake-prone building (EPB) notice

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

Engineering assessments can be sent to the TA at any time to determine the EPB status of a building.

Timeframes for strengthening Earthquake-prone buildings

Seismic work must be completed within the following periods, measured from the date the EPB notice was issued.

- **High seismic risk areas:**
  - High Priority buildings 7.5 years
  - **All other buildings 15 years**
- Medium seismic risk areas:
  - High Priority buildings 12.5 years
  - All other buildings 25 years
- Low seismic risk areas:
  - All buildings 35 years

If an Earthquake-Prone Building notice is issued for Frank Kitts Carpark than strengthening will need to be completed within 15 years of notice issue.

Alterations and change of use

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

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

9 LIMITATIONS

Findings presented as a part of this project are issued pursuant to our contract with Wellington City Council dated 27 June 2019 and for the sole use of Wellington City Council in its evaluation of the subject property. The findings are not intended for use by other parties and Holmes Consulting assumes no liability to any party other than Wellington City Council.

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

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

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

10 REFERENCES

ASCE/SEI, 2014. *ASCE/SEI 41-13 Seismic Evaluation and Retrofit of Existing Buildings.* s.l.:American Society of Civil Engineers.

ASCE/SEI, 2017. *ASCE/SEI 41-17 *Draft for Public Comment* Comment Seismic Evaluation and Retrofit of Existing Buildings.* s.l.:ASCE.

ASCE/SEI, 2017. *ASCE/SEI 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures.* s.l.:American Society of Civil Engineers.

Bradley, B. A. & Tarbali, K., 2017. *Default ground motion ensembles for response history analysis of structures in Christchurch, Wellington and Auckland - Revised Draft 22 June 2017,* s.l.: Bradley Seismic Limited.

Fenwick, R., Bull, D. & Gardiner, D., 2010. *2010-02 Assessment of hollow-core floors for seismic performance,* Christchurch: University of Canterbury.

Mondkar, D. P. & Powell, G. H., 1979. *ANSR II Analysis of Non-linear Structural Response User's Manual, EERC 79/17.* Berkeley: University of California.

NEHRP, 2010. *NIST GCR 10-917-5 Nonlinear Structural Analysis for Seismic Design,* s.l.: National Institute of Standards and Technology.

NZ Govt., 2017. *Building Regulations 2005 (SR 2005/32) - as amended by Amendment Regulations 2017 (LI 2017/136)).* Wellington: MBIE.

NZ Govt, 2016. *Building (Earthquake Prone Buildings) Amendment Act 2016.* Wellington: MBIE.

NZSEE, 2017. *The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, Initial Release July 2017.* Wellington: MBIE.

Preistley, M. J. N., Calvi, G. & Kowalsky, M., 2007. *Direct Displacement Based Design of Structures.* Pavia: IUSS Press.

Standards New Zealand, 2007. *NZS 3404:1997 Steel Structures Standard, Incorp. Amd. 1 & 2.* Wellington: Standards New Zealand.

Standards New Zealand, 2008. *NZS 3101:2006 Concrete Structures Standard, Incorp. Amd. 1 & 2.* Wellington: Standards New Zealand.

Standards New Zealand, 2011. *AS/NZS 1170.0:2002 Structural Design Actions Part 0: General principles, Incorp. Amd. 1 to 5.* Wellington: Standards New Zealand.

Standards New Zealand, 2016. *NZS 1170.5:2004 Incorporating Amendment 1 - New Zealand Structural Design Actions Part 5: Earthquake Actions.* Wellington: Standards New Zealand.

Appendices

CONTENTS

Appendix A

Assessment Summary Report

Appendix A Engineering assessment summary report

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

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

Building Information	
Building Name/ Description	The carpark is a single carpark building on Jervois Quay, Wellington. The building was designed and constructed in 1989. Its main purpose is to function as a carpark building where the roof is used as a public park. However, it is now also used for markets and festivals. The floor plan of the carpark comprises No.9 bays in transverse direction (E-W) and No.10 bays in longitudinal direction (N-S). The structure is formed by precast concrete wall panels around the perimeter, precast concrete frames and a flooring system comprising precast concrete double-tee units and cast in-situ slabs. The flooring system is tied together by a concrete layer across the area of the structure. On top of the flooring system there is a typical layer of soil and pavement and around the perimeter there is a concrete upstand. Circular concrete planter boxes are provided to allow for trees and large bushes. Concrete stair elements are provided at various locations to provide access to the roof structure and in the South-West corner there is access from a footbridge that goes across Jervois Quay. The precast wall panels and precast concrete frames are supported by concrete foundation beams and pads that are founded on a combination of bored concrete piles and driven precast concrete piles. The foundation beams and pads are tied together by concrete tie beams.
Street Address	Frank Kitts Carpark, Jervois Quay, Wellington.
Territorial Authority	Wellington City Council
No. of Storeys	1
Area of Typical Floor (approx.)	3341 m^2
Year of Design (approx.)	1989
NZ Standards Designed to	NZS4203:1984, NZS3101:1982
Structural System including foundations	The floor system is typically formed by precast concrete double-tee units. Around the circular planter boxes and at locations around the concrete staircases the slab is executed as a cast ins-situ slab. The double-tee units span between the precast concrete frames and walls in the North-South direction. Gravity loads from the floor system are carried by precast concrete columns, typically sized 300x600, and precast concrete wall panels on the perimeter of the structure. The columns are supported by concrete foundation pads which are founded on driven piles. The precast concrete wall panels are supported on foundation beams that are founded on a combination of precast concrete driven piles and bored concrete piles. The lateral loads are collected by the concrete diaphragm and distributed over the precast concrete wall panels and the precast concrete frames. The frames only run in the East-West direction.

Building Information	
	Lateral load is then taken by the precast concrete wall panels and precast concrete frames on a level of contribution that is relative to their stiffness. The precast concrete wall panels and precast concrete frames transfer to load to the foundations through their connection to the piling system.
Does the building comprise a shared structural form or share structural elements with any other adjacent titles?	No
Key features of ground profile and identified geohazards	The ground model consists of approximately 12m of loose gravelly fill overlying dense to very dense alluvium. Since the depth to bedrock was not determined, the site subsoil class was inconclusive between C and D. For this assessment the geotechnical recommendation was to use the most conservative for the assessment of the structure. The ground water level was determined to be approximately 1.8m below the ground floor level of the structure. However, it was considered to be fluctuating with the tide of the sea. Liquefaction is considered to be occurring in the ground layer consisting the loose gravelly fill. It was also found that the behaviour of the liquefied soil undergoes a significant step change between 20% and 30% of ULS seismic loading.
Previous strengthening and/ or significant alteration	N/A
Heritage Issues/Status	N/A
Other relevant information	-

Assessment Information	
Consulting Practice	Holmes Consulting LP
CPEng Responsible	Ian Hills (on behalf of Holmes Consulting LP) CPEng 179831 (Structural and Civil practice areas). Structural engineer with 20 years experience in buildings and infrastructure structural design. 8 years experience with seismic assessment and strengthening for harbour side buildings on reclaimed land.
Documentation reviewed	▪ 1989 complete set of architectural and structural drawings of Frank Kitts Carpark from original designers at BECA ▪ Construction specifications from 1989 prepared by original designer BECA ▪ Calculations package from 1989 prepared by original designer BECA ▪ Incomplete set of shop drawings of precast elements from 1989

Assessment Information	
	▪ Load assessment report from 2018
Geotechnical Report(s)	Geotechnical desktop investigation by Holmes Consulting LP completed in November 2019.
Date(s) Building Inspected and extent of inspection	A visit to the Frank Kitts carpark was completed in the September 2019. The purpose of the visit was to visually observe important structural and seismic characteristics, assess their condition and note obvious deficiencies or differences from the documentation. Particular locations that were investigated in more detail were: ▪ General walkovers were undertaken to confirm the global geometry of the Frank Kitts carpark and to identify secondary and non-structural elements that would influence the assessment of the carpark. ▪ The connection between precast shear wall panels. The as-constructed structure appears to be generally consistent with the available drawings. No significant modifications appear to have been made to the primary lateral or gravity systems. Material testing has not been completed. Material properties based on values in the original documentation are considered sufficient for this assessment. Our observations have been visual only.
Description of any structural testing undertaken and results summary	N/A
Previous Assessment Reports	Detailed seismic assessment by Aurecon New Zealand Limited – completed in 2013
Other Relevant Information	

Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	IL3
Summary of how Part C was applied,	Equivalent Linear Static Analysis
Other Relevant Information	-

Assessment Outcomes		
Assessment Status	Final	
Assessed Seismic Rating	15%NBS (IL3)	
Seismic Grade and Relative Risk	Alpha rating of E, corresponding to an approximate risk of 25 times greater relative to a new building.	
Secondary Structural and Non-structural elements/ parts identified and assessed	The look-out tower at South-West corner of the building (the Oriel) has been included in the assessment since failure may result in a significant life-safety hazard. The precast capping beam that is provided around the perimeter of the structure is also included, as failure of their connections may result in a significant life safety hazard. Other secondary and non-structural elements were assessed to not pose a significant life safety hazard.	
Describe the Governing Critical Structural Weakness	Diaphragm	
If the results of this DSA are being used for Earthquake-prone decision purposes, and elements rating <34%NBS have been identified (including Parts) :	**Engineering Statement of Structural Weaknesses and Location**	**Mode of Failure and Physical Consequence Statement(s)**
	Roof diaphragm	It is constructed with non-ductile mesh which does not provide reliable load path. In addition, transfer of the load to the lateral load resisting systems fails in shear-friction.
	Reclamation fill seismic performance and soil-structure interaction	Onset of liquefaction and lateral spread displacements imposing significant displacements demands on the foundations and structure. This may lead to undesirable behaviour in the superstructure and potential for collapse.
Recommendations	-	

Appendix B

Geotechnical Assessment Report

Appendix B Geotechnical Assessment Report

Holmes Consulting

Level 2, Chartered Accountants House, 50 Customhouse Quay
PO Box 942
Wellington 6140
holmesconsulting.co.nz

Memorandum

To:	Kelly Crandle
Company:	Wellington City Council
From:	Safia Moniz
Date	6 December 2019 Project No: 105978.19
Subject:	Frank Kitts Carpark DSA – Geotechnical Assessment

1 INTRODUCTION

This project involves the Detailed Seismic Assessment (DSA) of the Frank Kitts Carpark located along the Wellington Waterfront, Jervois Quay, Wellington, 6011 (the site). Holmes Consulting LP (Holmes) has been commissioned by the Wellington City Council (WCC) to provide the structural and geotechnical engineering assessment services to carry out the DSA. This memo describes the geotechnical considerations as inputs to the DSA.

The geotechnical considerations for the detailed seismic assessment were carried out in accordance with: *Seismic Assessment of Existing Buildings. NZSEE. July 2017. Part C4. Geotechnical Considerations* (The NZSEE Guidelines).

The primary objective of the DSA is to obtain an Earthquake Rating for the structure based on life safety issues. The rating is not reflective of serviceability performance (or day to day use of the structure).

2 SITE DESCRIPTION

The Frank Kitts Carpark is located on the Wellington Waterfront on reclaimed land. The eastern end of the building is located approximately 20m from the water's edge. The location of the site is shown in Figure 1.

Figure 1: Aerial photo showing site location

The building is a single-storey carpark structure. The roof is covered with grass and other landscape features and is used regularly as a recreational area. A few times a year it is used as a venue for outdoor concerts. Whilst the building is primarily used for carparking, it is also regularly used to accommodate a market.

The building is constructed of reinforced concrete walls, beams and columns and is founded on both 800mm diameter augered piles as well as 350mm reinforced concrete square driven piles. The foundation plan is presented in Appendix B1.

2.1 Geologic Environment

The surficial geology of the site has been mapped[1] as reclaimed land with fill consisting of domestic waste, sand, boulders, and rock (Q1n), overlying estuarine deposits consisting of poorly consolidated silt, peat, sand and minor gravel (Q1a).

The Wellington Central Business District has also been mapped by Semmens et al (2010)[2] and has mapped the site as being underlain by Reclamation Fill placed during the period 1965 – 1972 by the Wellington Harbour Board. This document also suggests that the depth to bedrock at the site is approximately 20 – 40m.

3 AVAILABLE FOUNDATION AND GEOTECHNICAL INFORMATION

We have been provided with geotechnical information from Beca's[3] archives. This information includes borehole logs and their locations, foundations plans and pile construction records. The information received from Beca, most relevant to the current DSA being carried out by Holmes, are appended to this memorandum (Appendix B2).

Ten borehole logs were received from Beca. One borehole (BH17) was obtained from the New Zealand Geotechnical Database. The names of these boreholes and their approximate locations are presented in Figure 2. The depths of the boreholes range from 15 to 20m below ground level (bgl).

The foundation drawings have been labelled "For Construction". However, we were also provided with detailed notes and site repots taken during construction from the geotechnical engineer. From the information provided it is reasonable to assume that the "For Construction" drawings (along with the construction monitoring notes) be used as as-built information.

[1] Begg, J.G.; Johnston, M.R. (compilers) 2000. Geology of the Wellington area. Institute of Geological and Nuclear Sciences 1:250 000 geological map 10. 1 sheet + 64p. Lower Hutt, New Zealand: Institute of Geological and Nuclear Sciences Limited.

[2] Semmens, S.; Perrin, N.D.; Dellow, G.D. 2010. It's Our Fault – Geological and Geotechnical Characterisation of the Wellington Central Business District, GNS Science Consultancy Report 2010/176. 52p.

[3] Beca were the structural and geotechnical designers of Frank Kitts Carpark.

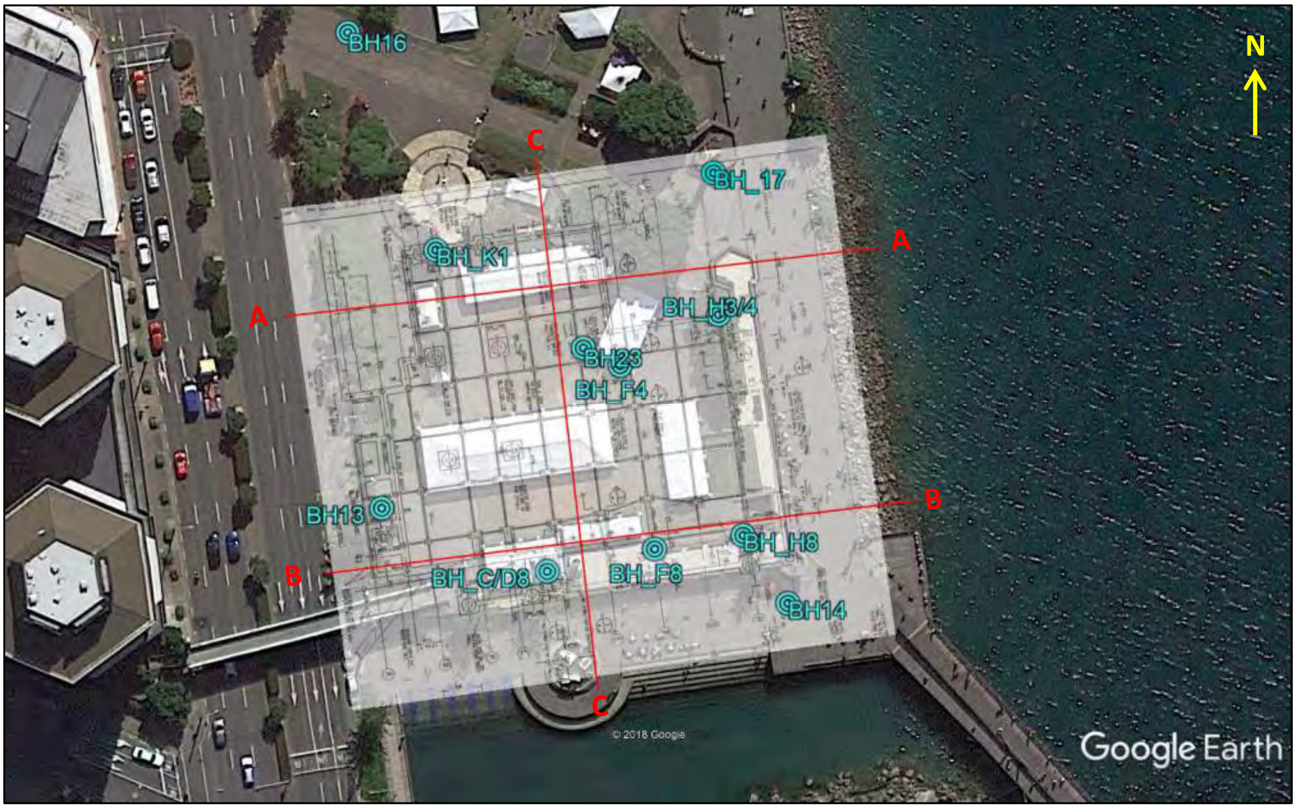


Figure 2: Aerial photo showing approximate locations of (1) boreholes used for this assessment and (2) ground model cross sections.

4 GROUND CONDITIONS

In general, the ground model consists of approximately 12m of loose gravelly fill overlying dense to very dense alluvium. In some boreholes a layer described as Harbour Deposits, consisting of soft black clayey silt and peat, was encountered above the alluvium. This layer was approximately 300mm thick. Three cross sections have been prepared and are shown in Figure 3,Figure 4 and Figure 5.

Uncertainties in the ground model include:

- There are only a few SPT N-values in taken in the Fill layer. We have assumed in our ground model that the consistency of this entire layer is very loose to loose. Given historical accounts that material was placed by end- tipping, we consider that this is a reasonable assumption.
- The depth to rock. This has not been investigated or confirmed. We have relied on Desktop Information[2] for assessing the depth to the bedrock.
- The consistency of the alluvium layer. For the idealized ground model, we have assumed that the consistency of this layer is dense to very dense and is unlikely to be susceptible to liquefaction. However, there was an N value of 26 recorded in this layer. An N-value of 26 in cohesionless material could suggest marginal susceptibility to liquefaction under a ULS earthquake.

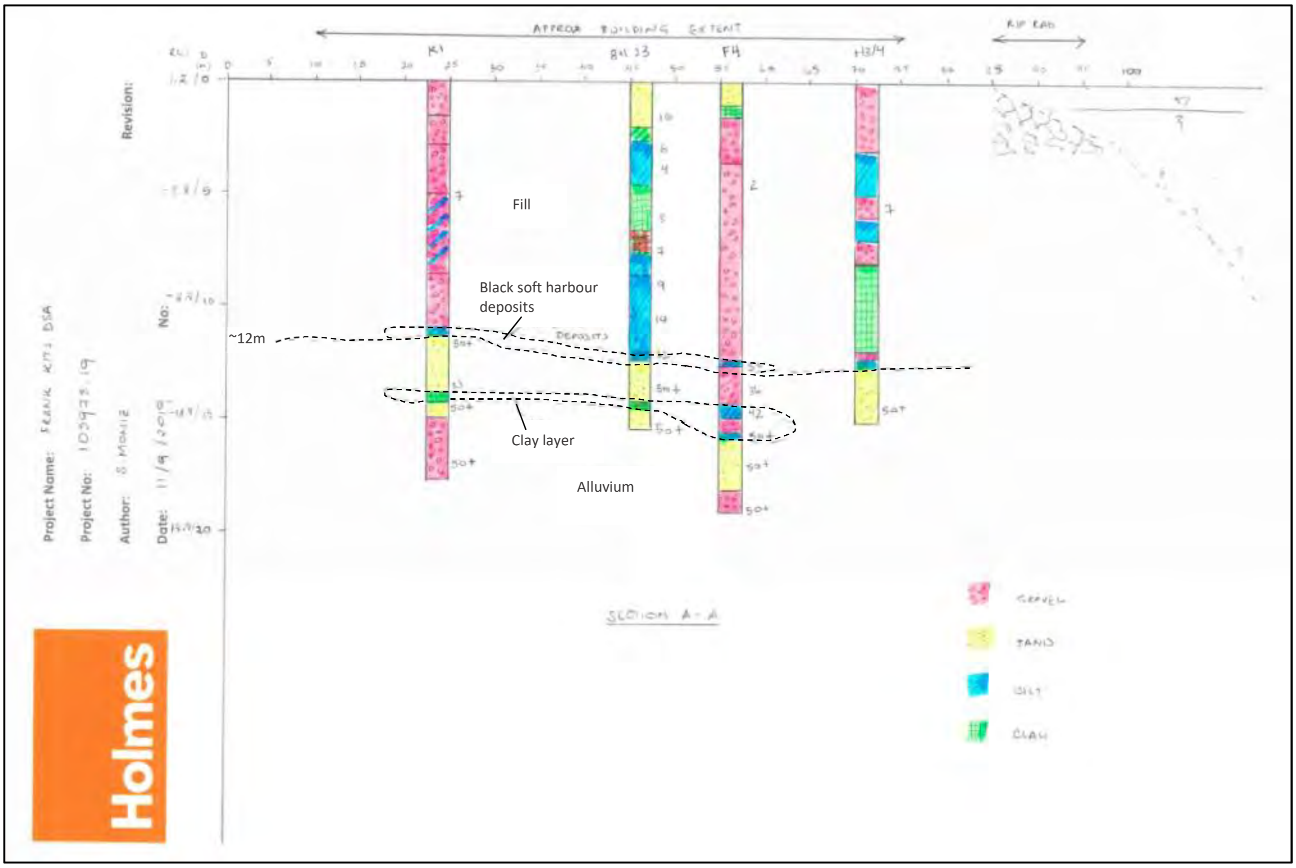


Figure 3: Ground Model Cross Section A-A

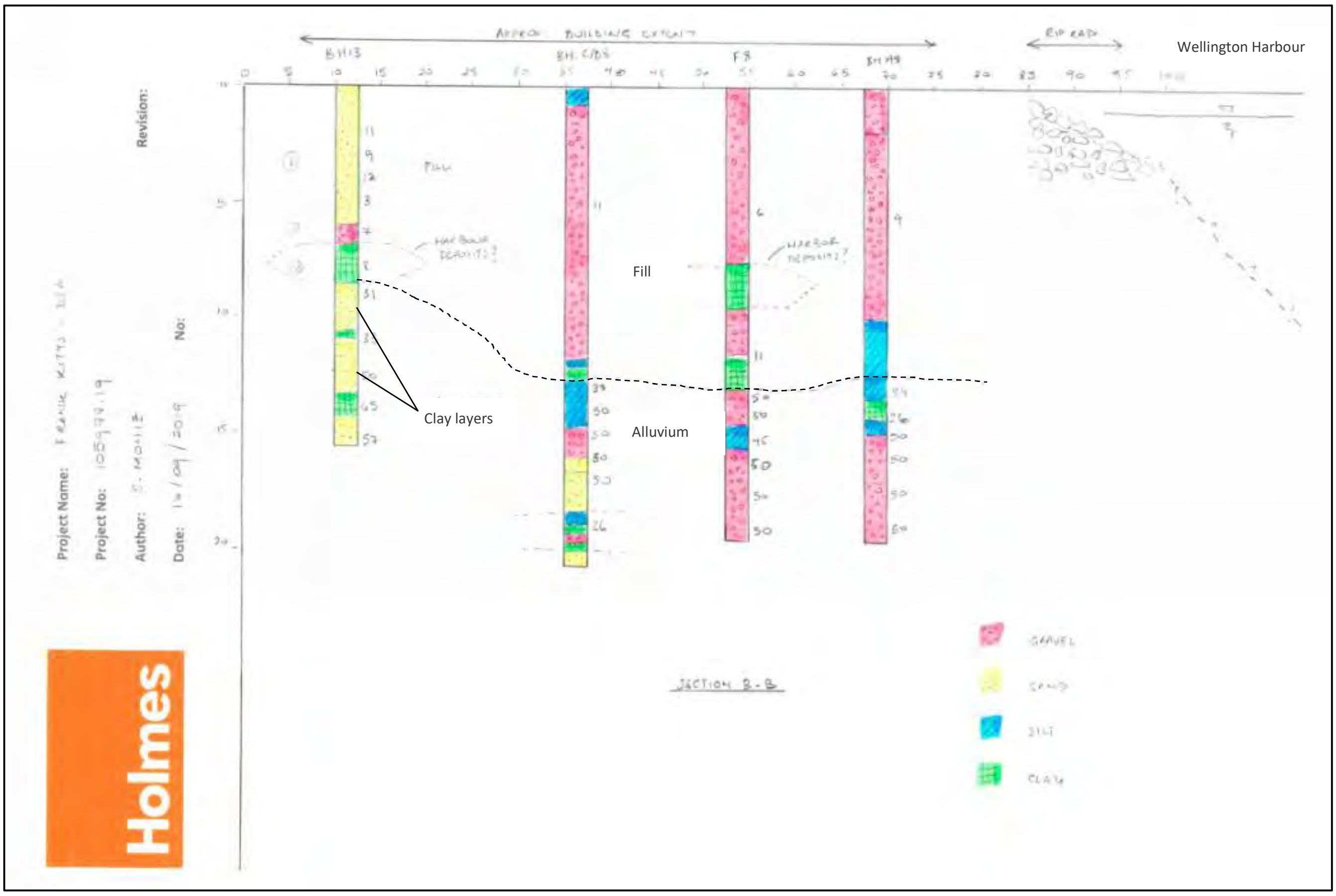


Figure 4: Ground Model Cross Section B-B

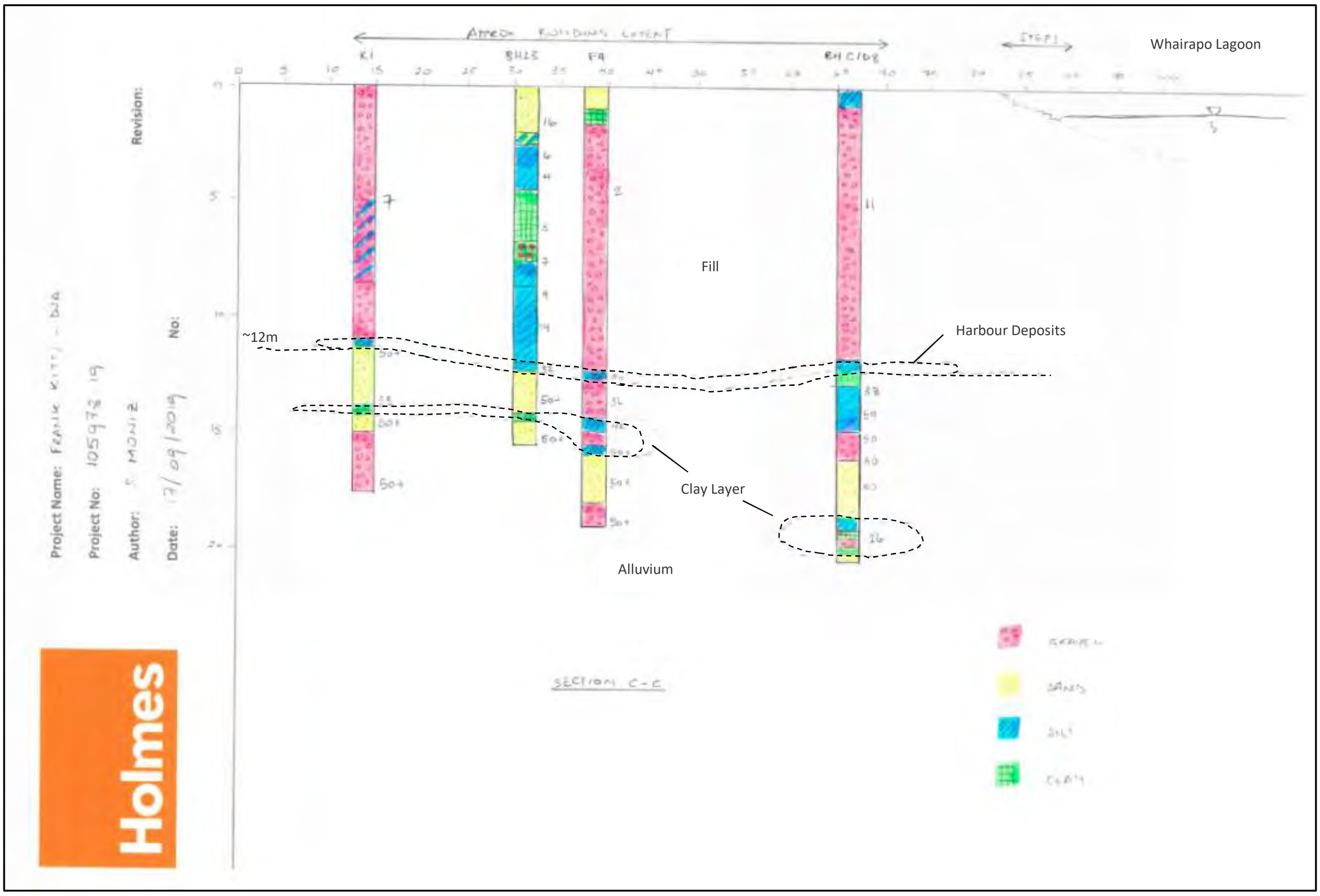


Figure 5: Ground Model Cross Section C - C

4.1 Site Subsoil Class

The site has been mapped by Semmens et al (2010)[2] as Class C but with a 'boundary uncertainty' qualifier (see Figure 6). On this map we can see that immediately to the north and south of the site, boreholes have been advanced and confirmed to be Class C (the two yellow dots within the red dashed square).

The site is underlain by very loose to loose fill with SPT N values ranging from 2 – 16 with an average of 7 (based on the Beca borehole information). Referring to NZS1170.5:2004 Clause 3.1.3.4 and Table 3.2, to be classified as Class C, there must not be any very loose soil (SPT N values <6) present. In this case there were several measurements of SPT N values < 6.

We have also carried out a calculation to estimate the natural period of the site as per NZS1170.5:2004 Clause 3.1.3.7. This resulted in a natural period between 0.51s and 0.75s, spanning across the cut off site natural period for Class C of 0.6s.

There is one data point on Map 6 of Semmens et al (2010) which shows that the site period is 0.5s at the southwestern end of Frank Kitts Park.

In summary, we have some evidence suggesting that it could be Class C, some suggesting Class D and some suggesting uncertainty. In the hierarchy of methods given in NZS1170.5:2004, our calculation of site period based on shear wave velocities derived from the SPT N values is the highest.

Therefore, given the uncertainty in the Site Subsoil Class we have used the most conservative of the two classes for the geotechnical and structural assessments respectively. For the geotechnical analyses we will assume Site Class C as this results in a higher PGA and for the structural analysis, we will assume Site Class D as this is the worst case for the structure. The structural engineering report will discuss the implications of Class C or D on the structure.

To produce a more accurate assessment of the site subsoil class shear wave velocity tests can be carried out.

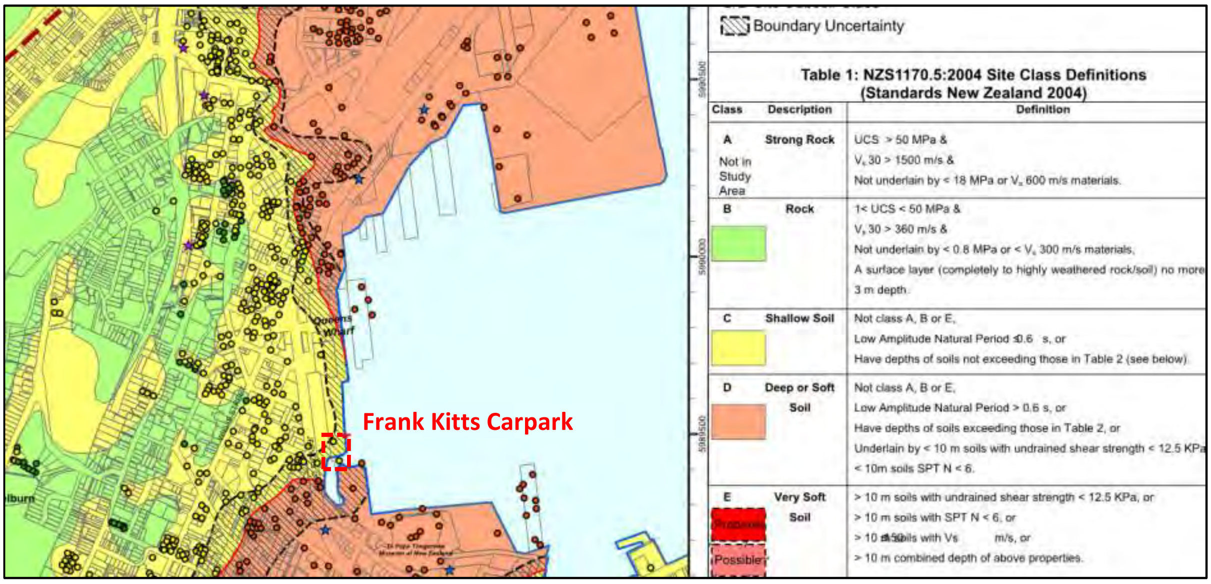


Figure 6: Site Subsoil Map showing Frank Kitts Carpark mapped as 'Class C - Boundary Uncertainty'

4.2 Water Table

Groundwater was encountered at 1.8m below the ground level in BH13. This water level will fluctuate with the tide given the proximity of the site to the harbour.

5 BUILDING DESCRIPTION

The carpark is a single storey carpark building on Jervois Quay, Wellington. The building was designed and constructed in 1989. It has been used as carpark, to host the weekend markets and annual Homegrown festival. The structure is formed by in-situ slab, double tees and precast wall panels. The precast walls are founded on a combination of 800mm diameter augered piles and 350mm driven precast concrete square piles on the perimeter.

6 PROJECT CATEGORISATION

The geotechnical considerations for the detailed seismic assessment were carried out in accordance with: *Seismic Assessment of Existing Buildings. NZSEE. July 2017. Part C4. Geotechnical Considerations*. The guidance states:

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

After compiling the desktop study, understanding the structure and ground model we have compiled a list of the geotechnical hazards as related to the Frank Kitts site. For each geotechnical hazard we have indicated whether it is:

(a) Originating from outside the building footprint and thus not influencing the %NBS rating

(b) Jointly agreed with the structural engineer that, because of the soil and structure's expected behaviour, are not likely to be critical to the assessment of the %NBS rating

(c) Specifically assessed.

The geotechnical engineer met with the structural engineers on a number of occasions to discuss the project including how the geohazards identified below would impact the structure. These are presented in Table 1.

Table 1: Geotechnical Hazards Affecting Frank Kitts Carpark Building

Geotechnical Hazard	Discussion	Not Critical or Specifically Assessed?
Negative skin friction (as a result of liquefaction)	Loss of pile side resistance due to an increase in pore water pressure and a subsequent reduction in effective stress and therefore strength. This can happen regardless of if liquefaction is fully triggered at the site. Bored cast in place piles are particularly susceptible to settlement caused by pore water pressure increase and liquefaction above the base of the piles, because initially most of the gravity loads are resisted by the side friction.	*Not critical.*

Geotechnical Hazard	Discussion	Not Critical or Specifically Assessed?
	With decreased side resistance available settlement will start and continue until the load is resisted by the end bearing of the pile. If excessive settlements occurred this would not directly cause collapse of the structure or lead to a significant life safety hazard.	
Cyclic Axial Load	Cyclic axial loading may cause excessive settlement and potentially loss of capacity during strong shaking. Excessive settlement would not directly cause collapse of the structure or lead to a significant life safety hazard.	*Not critical.*
Pile Settlement	Isolated sand layers may be present within the Alluvium that are susceptible to liquefaction, leading to settlement under both the bored and square piles. Excessive settlement would not directly cause collapse of the structure or lead to a significant life safety hazard.	*Not critical*
Loss of base shear take-out	Base shear take-out may be via 1) friction developed between the underside of the building and the ground and (2) the passive resistance provided by the piles. Static settlement with time and strong shaking can cause settlement of the ground away from the underside of the slab reducing available friction. Additionally, after a few cycles of shaking gapping can occur and passive resistance be lost. The consequence of these phenomenon is that the piles or the connection between the piles and the pile cap may experience higher shear forces that the original designer intended. Structural overload in shear in the pile or pile-cap connection could lead to the building moving off its foundations in an extreme case. This scenario is unlikely to lead to building collapse nor is it considered to create a significant life safety hazard.	*Not critical*
Cyclic (Kinematic) Ground Displacements	Cyclic ground displacement is the relative movement between layers of liquefied and non-liquefied soil over the depth of the pile. These induce additional displacements and forces in the piles which are additive to forces imparted by seismic inertial loading from the building.	*Not critical*
Lateral Spread	Lateral spread displacements occur towards the end of shaking due to elevated porewater pressures and decreased effective stresses and (liquefied) soil strengths	*Specifically assessed*

Geotechnical Hazard	Discussion	Not Critical or Specifically Assessed?
	Horizontal deformations due to lateral spread are typically large where the building is located a few metres from a free face and underlain by deep liquefiable deposits. Further, the presence of a free face toward both the Wellington Harbour and the Whairepo Lagoon will exacerbate displacements. Due to the sudden onset and large predicted lateral spread displacements at this site is likely to lead to a step change in structural performance which could trigger collapse and lead to a significant life safety hazard. See Figure 7.	

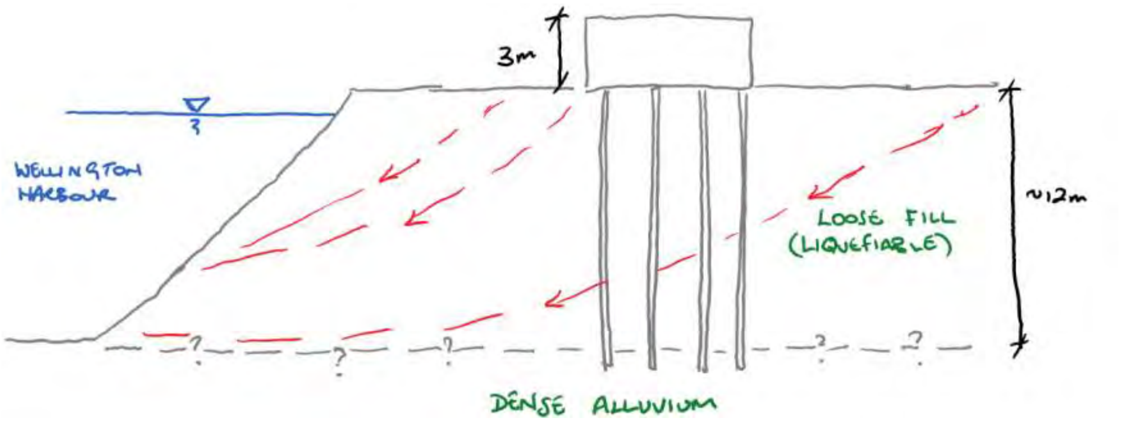


Figure 7: Lateral spread - critical geotechnical issue for Frank Kitts Carpark that could result in a significant structural step change

Following interactive whiteboard sessions with the structural engineers on the 18th and 25th October 2019, the project has been placed in the '**Interactive**' category.

7 GEOTECHNICAL ANALYSES

The objective of the DSA is to determine a %NBS[4] score. This score is the "ratio of the ultimate capacity of the lowest scoring issue compatible with a significant life safety hazard (lateral spread in this case) to the actions expected when the structure is subjected to the demands resulting from the ULS defined loads / deformations for new buildings". Therefore, we need to determine (1) what the PGA of the 100% ULS earthquake is for NBS for the Frank Kitts Carpark Building to compare to (2) the PGA that results in an amount of displacement caused by lateral spread to trigger a significant life safety hazard within the structure.

[4] %NBS = % of the New Building Standard

7.1 Determining the 100% ULS Earthquake

The design life, importance level and return periods for the design earthquake for Frank Kitts Carpark is as follows:

Table 2: Seismic Design Criteria

Design Parameter	Value
Design Life	50 years
Importance Level	3
ULS Annual Probability of Exceedance	1/1000

Based on the results of the ground investigation, the NZS1170.5 Site Subsoil Classification is C, for geotechnical assessment purposes, refer to Section 4.1.

The 100% ULS Earthquake Peak Ground Acceleration (PGA) and Magnitude (presented in Table 3) were derived in accordance with:

- MBIE/NZGS Module 1: Earthquake Geotechnical Engineering Practice
- NZTA Bridge Manual 3rd Edition, Amendment 3.

Table 3: 100% ULS Earthquake

Design Earthquake	Return Period Factor (R_u)	Return Period	PGA	Magnitude
ULS	1.3	1/1000	0.585	7.1

7.2 Liquefaction and Lateral Spread Analysis

Liquefaction analyses using the SPT values from the boreholes were carried out with reference to *MBIE/NZGS Module 3: Identification, assessment and mitigation of liquefaction hazards*. The liquefaction triggering analysis method used was Boulanger & Idriss (2014). Analyses were carried out for borehole BH23 and BH_C/D8 as these were representative of the ground conditions at the site.

The analyses were carried out for a range of accelerations to determine the relationship between liquefaction triggering and the ULS PGA. These results are presented in Figure 8. This plot suggests that a 'step change' in ground performance occurs between 20 – 30%ULS.

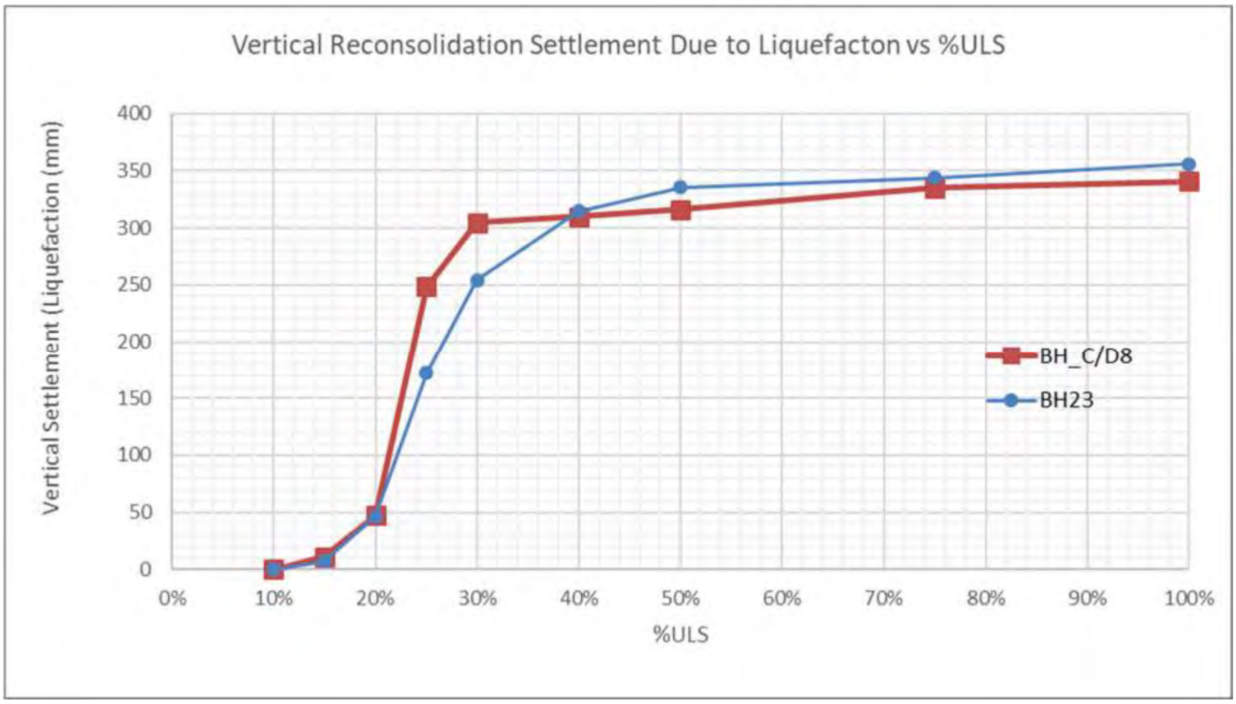


Figure 8: Plot showing relationship between vertical reconsolidation settlement and %ULS

Lateral spread was evaluated using two methods: (1) a semiempirical method utilizing the SPT values, and the length and height of the slope - Zhang et al (2004) and (2) a Newmark sliding block analysis using the program Slide(2018 8.028) which is a limit equilibrium slope stability program. These two approaches are endorsed in the MBIE/NZGS Module 3[5].

Method 1: Zhang 2004

This is a semiempirical method that is based on case study data and considers SPT-N values, the site geometry and liquefaction susceptibility.

The equation used to estimate Lateral Displacement (LD) is given by:

$$\frac{LD}{LDI} = 6 \cdot \left(\frac{L}{H}\right)^{-0.8} \quad (for\ 4 < \frac{L}{H} < 40)$$

Where:

LD = Lateral displacement

LDI = Lateral Displacement Index

L = Horizontal distance from the toe of a free face

H = elevation difference between the level ground surface and the toe of a free face

---

[5] MBIE/NZGS Earthquake geotechnical engineering practice. Module 3: Identification, assessment and mitigation of liquefaction hazards.

We have taken L = 75m which is the distance from the free face to the edge of the building furthest from the Wellington Harbour. This distance runs in a West – East direction which is the more critical case for lateral spread at the site. The height of the free face, H, was more difficult to determine given that we did not have a survey of the seabed. What we know is there is approximately 12m of fill at the site and there is riprap at the crest of the slope which we can observe. Bathymetry surveys[6] for Wellington indicate that the seabed is approximately 5m below chart datum in area nearest to the Frank Kitts Carpark. We have therefore used the ground profile shown in Figure 10 for the lateral spread analysis.

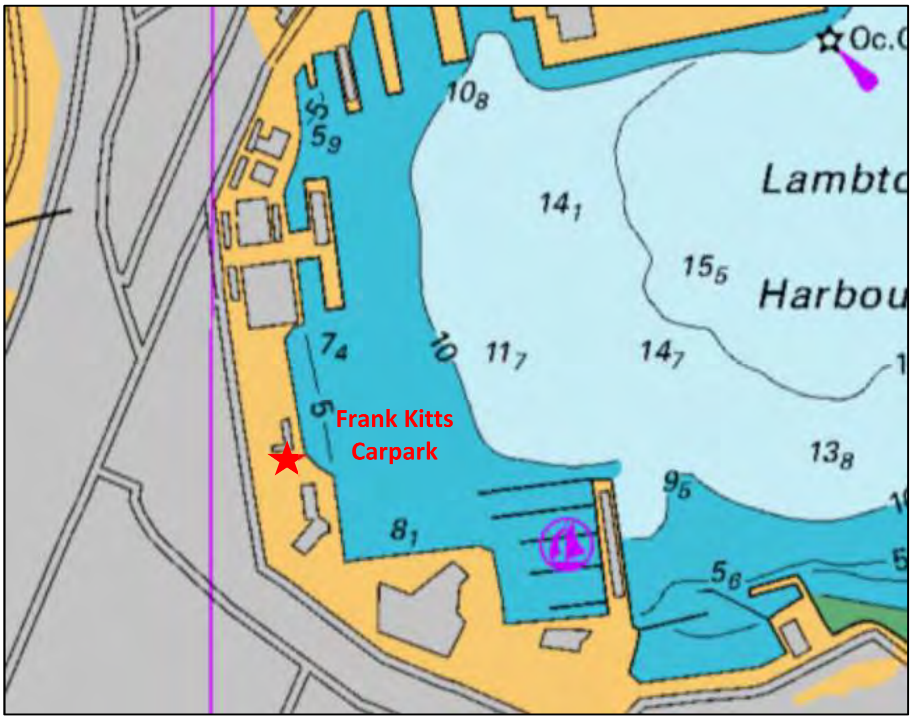


Figure 9: Bathymetry Survey of Wellington Harbour

The results of the lateral spread analysis using Zhang (2004) are shown in Figure 11.

The results of the analyses suggest that the maximum lateral displacement is in the order of 5m. Due to there being less SPT N-Value data points, the plot for BH_C/D8 is a bit cruder. However, with this method there is a 50 – 200% margin of error. Given this margin of error we consider it prudent to carry out another method for comparison.

[6] Bathymetry Chart. Chart NZ 4633 Wellington Harbour. LINZ Data Service.

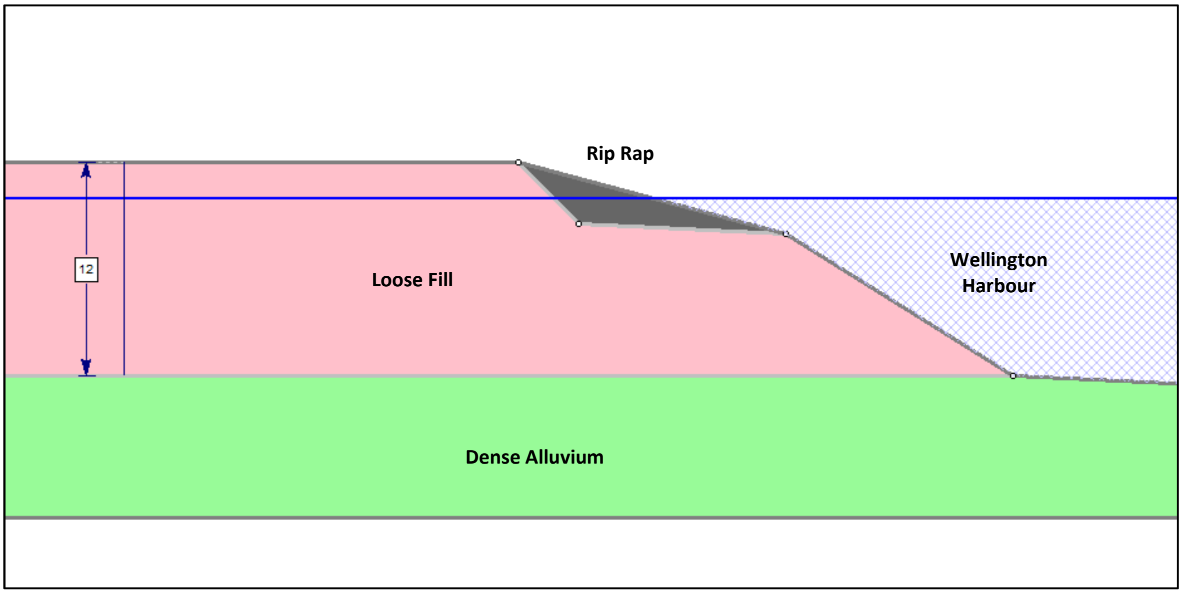


Figure 10: Ground profile used for the lateral spread analysis

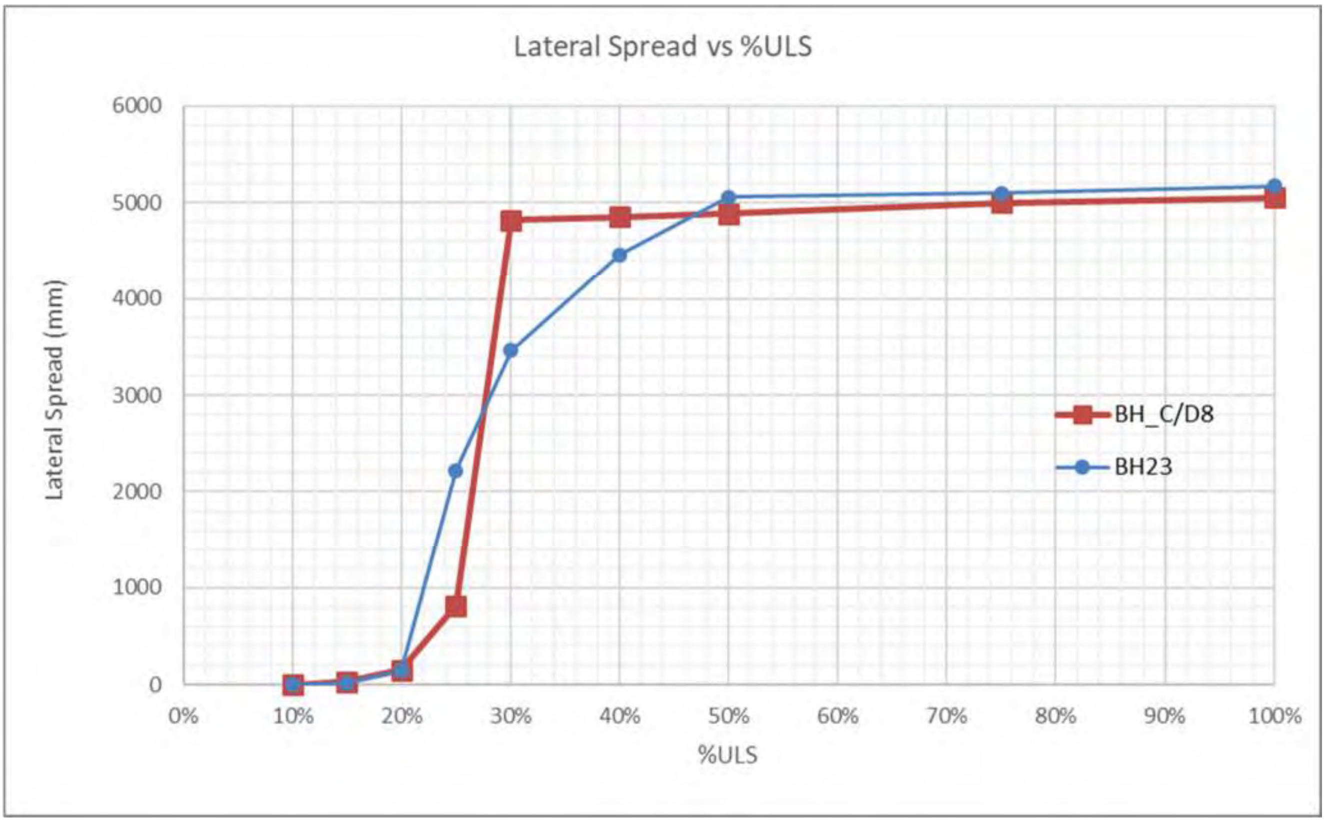


Figure 11: Plot showing the relationship between lateral spread vs %ULS

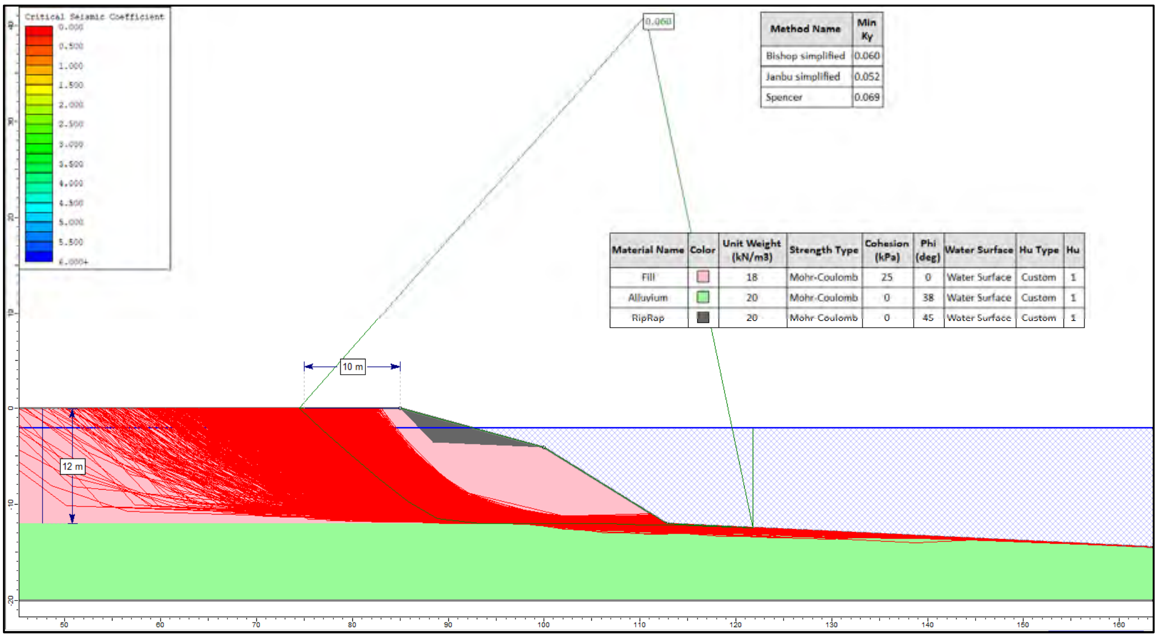
Method 2: Newmark Sliding Block Analysis

The Newmark Sliding Block analysis estimates earthquake-induced permanent lateral ground displacements by subjecting a rigid body to base acceleration. First the yield acceleration is calculated based on the limit equilibrium approach. This was carried out using the slope stability program Slide (2018 8.028). Slope displacement is then calculated by using approximate equations developed from analyses based on a database of ground motions.

The slope stability model was set up as shown in Figure 10. The residual shear strength of the liquefied fill was estimated using Idriss and Boulanger (2014)[7] to be approximately 25kPa and was modelled as a Mohr-Coulomb material. The dense alluvium was modelled as a cohesionless Mohr-Coulomb material with $\phi = 38^o$ and c=0kPa. The Rip-Rap was also modelled as cohesionless Mohr-Coulomb material but with $\phi = 45^o$ and c=0kPa.

The results of the analysis showed that the critical acceleration (a_c) = 0.060g (as an average of the methods used). The results of the analysis are presented in Figure 12.

Using the meta-analysis chart presented in Figure 24 in the paper presented by Wood 2015[8], and taking a_{max} = 0.585g, the displacement was calculated to be about 1m.



Method Name	Min Ky
Bishop simplified	0.060
Janbu simplified	0.052
Spencer	0.069

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Fill		18	Mohr-Coulomb	25	0	Water Surface	Custom	1
Alluvium		20	Mohr-Coulomb	0	38	Water Surface	Custom	1
RipRap		20	Mohr-Coulomb	0	45	Water Surface	Custom	1

Figure 12: Results showing critical acceleration

[7] 2nd Ishihara Lecture: SPT- and CPT-based relationships for the residual shear strength of liquefied soils. I.M. Idriss and Ross W. Boulanger. 2014.

[8] 15th Geomechanics Lecture – Geotechnical Issues in Displacement Based Earthquake Design of Highway Bridges and Walls.

Lateral Spread – Summary:

From Method 1: Zhang 2004 we calculated LD = 5m and from Method 2: Newmark sliding block an LD = 1m was calculated. We have therefore recommended to the structural engineers that the structure be checked for lateral displacements between 1m and 5m. The lateral spread displacement profile presented in Figure 13 should be used for lateral spread assessments.

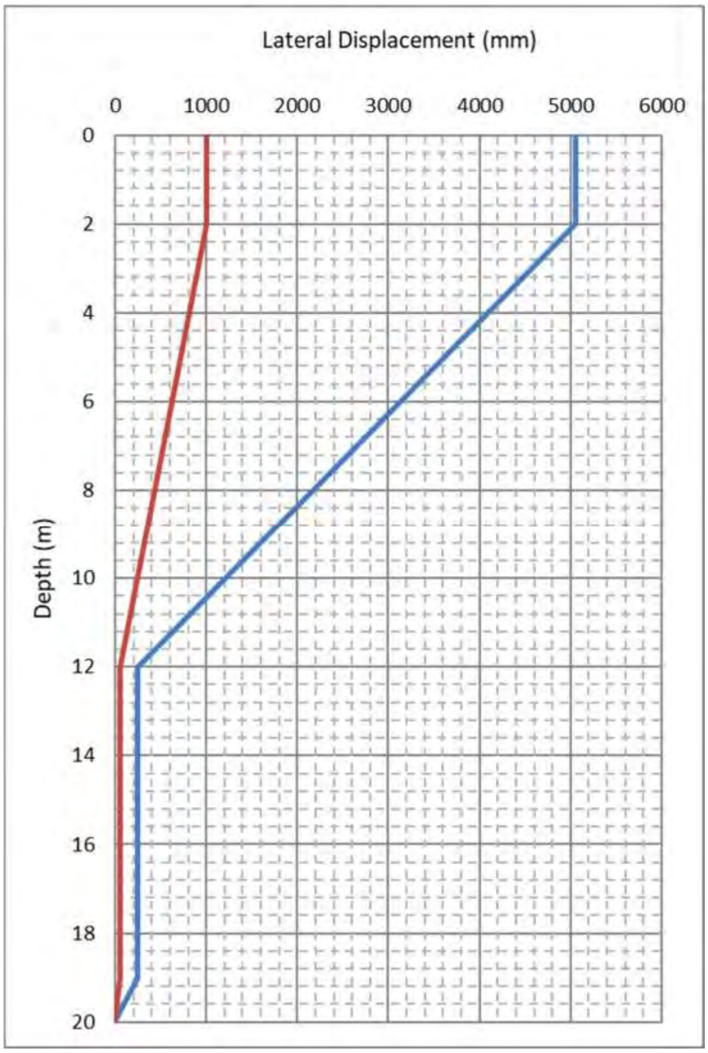


Figure 13: Lateral Spread Displacement Profile for 100% ULS

Holmes

7.3 Pile Analyses

Lateral pile analyses were carried out using the program AllPile. As a joint structural-geotechnical exercise two scenarios were modelled: Scenario 1a and 1b and Scenario 2a and 2b. Analyses were carried out on the 800mm diameter pile as these piles were the most critical piles for the structure. The purpose of this exercise was to understand the load-displacement behaviour of the pile through modelling different load and ground scenarios that would be expected under seismic conditions. Scenario 1a and 1b were carried out for the non-liquefied soil case and Scenario 2a and 2b were carried out for liquefied ground conditions. The resulting load-displacement behaviour will be used by the structural engineers to help inform the performance of the foundations.

Scenario 1a: The purpose of this scenario was two-fold: (1) to serve as a calibration check – i.e. to determine if the structural and geotechnical models were giving roughly the same result and (2) to push the pile to its moment capacity which is approximately 1400kN-m. This was carried out assuming non-liquefied soil conditions.

Axial load: 100 kN

Head Condition: Fixed

Ground Conditions: Prior to liquefaction

Results:

- The flexural capacity at the head of the pile is reached at a lateral load corresponding to 20% of the ULS demand (i.e. 20% ULS =725kN).
- At this point the pile hinges as it has reached its flexural capacity at the pile head.
- The pile head displacement generated was 37mm. We understand from the structural engineers that a similar value was also attained in the structural model.

Scenario 1b: The purpose of this scenario was to model the maximum shear force and displacement of the pile after it has reached its moment capacity as in Scenario 1a. To do this the pile first was modelled as a free-head pile with a shear force of 725kN and a moment of -1400kN-m applied to the pile head. This represents the equivalent state reached in Scenario 1a. (The displacement was 37mm confirming the equivalency to Scenario 1a). An additional shear force was then applied to the pile to investigate the response.

Axial load: 100 kN

Head Condition: Free

Ground Conditions: Prior to liquefaction

Applied Moment: -1400kN-m (at head of the pile to represent flexural capacity of the pile section)

Results:

- The results indicate that as the shear force is increased to 1000kN, a second plastic hinge will form at depth within the pile, with a corresponding pile head displacement of 160mm.
- This corresponds to ~26% of the ULS demand (i.e. 26% ULS =1000kN).

- We understand from the structural engineers that at beyond this point the maximum pile displacement will be controlled by the curvature (rotation) capacity in the plastic hinge region of the pile. The structural shear capacity of the pile should also be checked against the applied load.

Scenario 2a: The purpose of this scenario is the same as Scenario 1a, except assuming liquefied soil conditions.

Axial load: 100 kN

Head Condition: Fixed

Ground Conditions: Liquefaction is fully triggered

Results:

- The flexural capacity at the head of the pile is reached at a lateral load corresponding to 15% of the ULS demand (i.e. 15% ULS =600kN).
- At this point the pile hinges as it has reached its flexural capacity at the pile head.
- The pile head displacement generated was 60mm.

Scenario 2b: The purpose of this scenario was to model the maximum shear force and displacement of the pile after it has reached its moment capacity as in Scenario 2a. To do this the pile first was modelled as a free-head pile with a shear force of 600kN and a moment of -1400kN-m applied to the pile head. This represents the equivalent state reached in Scenario 2a. (The displacement was 60mm confirming the equivalency to Scenario 2a). An additional shear was then applied to the pile to investigate the response.

Axial load: 100 kN

Head Condition: Pinned

Ground Conditions: Liquefaction.

Applied Moment: -1400kN-m (at head of the pile to represent flexural capacity of the pile section)

Results:

- The results indicate that as the shear force is increased to 850kN, a second plastic hinge will form a depth within the pile, with a corresponding pile head displacement of 300mm.
- This corresponds to ~22% of the ULS demand (i.e. 22% ULS =850kN).
- We understand from the structural engineers that at beyond this point the maximum pile displacement will be controlled by the curvature (rotation) capacity in the plastic hinge region of the pile. The structural shear capacity of the pile should also be checked against the applied load.

Summary:

The results of the pile analyses can be summarised in the plot presented in Figure 14. The structural engineers will use this (load – displacement) relationship to investigate the structural performance of the pile foundation. On this plot we have also included the shear capacity of the pile (825kN, as advised by the structural engineers). The structural limitations of pile performance are evaluated and discussed as part of the structural assessment.

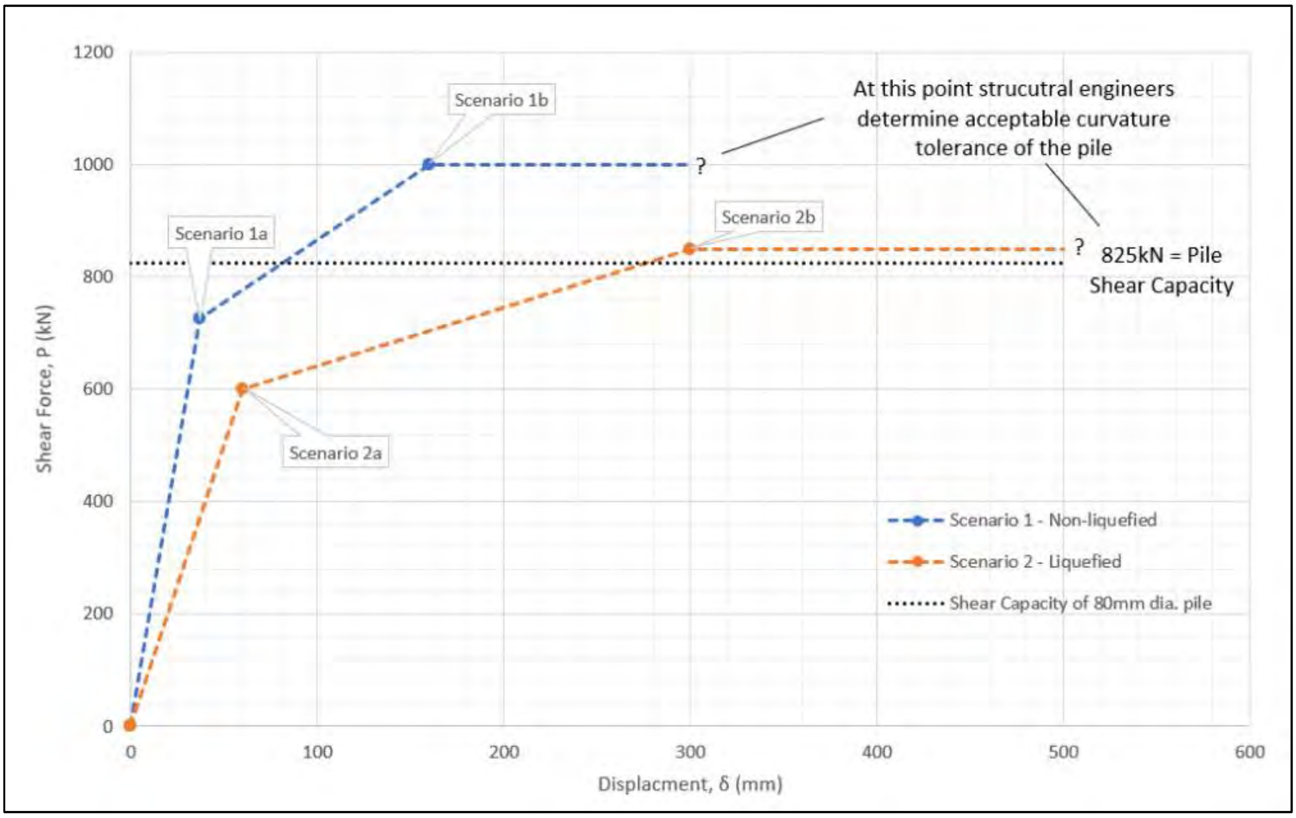


Figure 14: Plot showing load-displacement relationship for the 800mm dia. piles

Discussion:

Section C4.5.3.2 of the NZSEE Guidelines state that a "*Geotechnical step change will only be an issue for setting the earthquake rating if in turn it results in a step change behaviour of the building structure, i.e. a structural step change, and then only one that would result in a significant life safety hazard*."

A geotechnical step change has been identified in the large increase in lateral spread displacements at 20-30%ULS. These large lateral spread displacements will result in a significant step change in structural behaviour of the building that could lead to a significant life safety hazard. We have carried out analyses to quantify the geotechnical aspects in terms of %ULS.

The NZSEE Guidelines (Section C4.7) state that if a geotechnical step change leads to the development of a significant life safety hazard in the building, then a margin of 2 must be applied to the PGA at the step change. This factor is intended to represent the uncertainty involved with geotechnical assessments.

Strong motion data was recorded during the Kaikoura 2016 earthquake by GNS strong motion accelerometers located at Frank Kitts Park (FKPS), very close to the carpark building. The PGA at this site from the Kaikoura 2016 earthquake was recorded to be 0.21g[9]. When magnitude scaled to the ULS earthquake M7.1 for Wellington, this gives a PGA of 0.189g. This is equivalent to approximately 30% of the ULS earthquake for the Frank Kitts Carpark.

Noting that no evidence of liquefaction or lateral spread was observed at Frank Kitts Carpark, we can take this value as the lower bound value for the geotechnical performance where we know the foundation system functioned as intended. That is, we do not consider it necessary to consider and apply the factor of 2 as required by the Guidelines to reflect any uncertainty in the geotechnical assessment.

Given all practical uncertainties considered, the PGA recorded at the Frank Kitts site during the Kaikoura Earthquake falls within the 20 to 30% range of liquefaction triggering calculated and discussed in Section 7.2.

Therefore, the geotechnical assessment has yielded a 20 – 30%NBS rating. This is a combined structural response based on geotechnical issues. (Interactive project).

8 UNCERTAINTY, RISK AND FURTHER WORK

Analyses have been carried out using the most current guidelines and using all the information available to us both published publicly and provided to us by Wellington City Council. The outcome of our analyses is based on sound engineering analyses and was derived using well established methods. However, we are aware that we may not have all the information relevant to the site and that there are inherently limitations and uncertainties in the analyses carried out. As part of the DSA process the NZSEE Guidelines suggest that uncertainties in the analysis are identified. These are as follows:

Ground Model: The uncertainties in the ground model were discussed in Section 4.

Site Geometry: The height and gradient of the slope for the lateral spread analyses was estimated based on observations, published mapped data and our knowledge of the fill placement in the area. The height of the slope has a significant influence on the outcome of lateral spread predictions.

Site Subsoil Class: The uncertainties surrounding the site subsoils class are discussed in Section 4.1.

Liquefaction analyses: Liquefaction analyses were carried out using well established simplified methods. The input for these analyses were based on SPT N values which are an inherently crude input for liquefaction analyses as they are typically carried out at 1.5m intervals. Additionally, there were not many SPTs carried out within the fill layer; in some instances, just a single SPT test was carried out in the fill layer for some of the boreholes. There was also no lab testing available which would have helped refine the liquefaction triggering predictions.

Due to the inherent uncertainty in geotechnical earthquake engineering, we have had to rely on precedent, empiricism, and well-founded judgement to arrive at the predicted range of deformation and foundation performance. Although we have highlighted the uncertainties in the analyses and the accuracy of the outcome may improve if these are addressed, it is a possibility that outcome may be unchanged even if these uncertainties are mitigated.

[9]Data retrieved from: http://spectra.rapidalert.org.nz/detail.jsp?id=4502 and ftp://ftp.geonet.org.nz/strong/processed/Proc/2016/11_Nov/2016-11-13_110256/20161113_110256.CSV

Should WCC wish to investigate any improvement of the accuracy of the geotechnical aspects of the DSA, we recommend the following:

- Shear wave velocity of Horizontal to Vertical Spectral Ratio (HVSR) testing to confirm the site period and Site Subsoil Class.
- Shear wave velocity testing in the fill to improve the liquefaction triggering and lateral spread assessment.
- Undertake a topographical survey of the area including capturing the underwater slopes (and just beyond) at the eastern and southern ends of the site to establish the Site Geometry and subsequent lateral spread predictions.

Safia Moniz
SENIOR PROJECT ENGINEER
Holmes Consulting LP

Limitations Statement

Findings presented as a part of this project are for the sole use of the client in its evaluation of the subject properties. The findings are not intended for use by other parties and may not contain sufficient information for the purposes of other parties or other uses. The information contained in the memorandum is subject to the terms and conditions of our professional services engagement with Wellington City Council.

This report may only be relied upon by the Client and only in relation to the scope of services agreed between Holmes Consulting and the Client under agreement dated [date]. This report may not be relied upon by any third party or for any other purpose without the express written agreement of Holmes Consulting.

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

The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the Client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it should be appreciated that actual conditions could vary from the assumed model.

Appendix C

Technical Summary of Equivalent Linear Static Analysis and Evaluation

Appendix C Technical Summary of Equivalent Linear Static Analysis and Evaluation

C.1 INTRODUCTION

For the analysis of Frank Kitts Carpark an equivalent linear static analysis method has been used by means of hand calculations. Lateral mechanisms were identified prior to the analysis to confirm the expected performance of the assessed structural elements. The expected performance was later confirmed by the outcome of the assessment. Where applicable, for example to determine the in-plane capacity of the precast panels, a direct displacement-based analysis has been carried to determine the performance of the structural elements.

Since the structure is single storey and of rectangular geometry an equivalent linear static analysis is appropriate for the assessment.

C.1.1 Equivalent Linear Static analysis

The equivalent static lateral force method is a simplified technique to substitute the effect of dynamic loading of an expected earthquake by a static force distributed laterally on a structure for design purposes. The total base shear is evaluated in the two horizontal directions parallel to the main axes of the building. It assumes that the building responds in its fundamental lateral mode. The building is low-rise and is fairly symmetric. Therefore, torsional movement due to ground motions can be captured with reasonable confidence. The use of equivalent static method is used for the primary and secondary and non-structural elements in the assessment.

The analysis has assumed a structural displacement ductility of μ = 1.25. This simulates a nominal ductile structure and allows for minimal inelastic behaviour in the structure. The analysis has shown that there is a number of elements that respond with minimal inelastic deformations which coincides with this assumption.

C.2 ANALYSIS INPUT

C.2.1 Building description

Frank Kitts Carpark is a single storey carpark building. The floor plan of the carpark comprises No.9 bays in transverse direction (E-W) and No.10 bays in longitudinal direction (N-S). The structure is formed by in-situ slab, double-tee units and precast concrete wall panels. The precast concrete wall panels are founded on a combination of bored concrete piles and precast concrete driven piles on the perimeter of the structure.

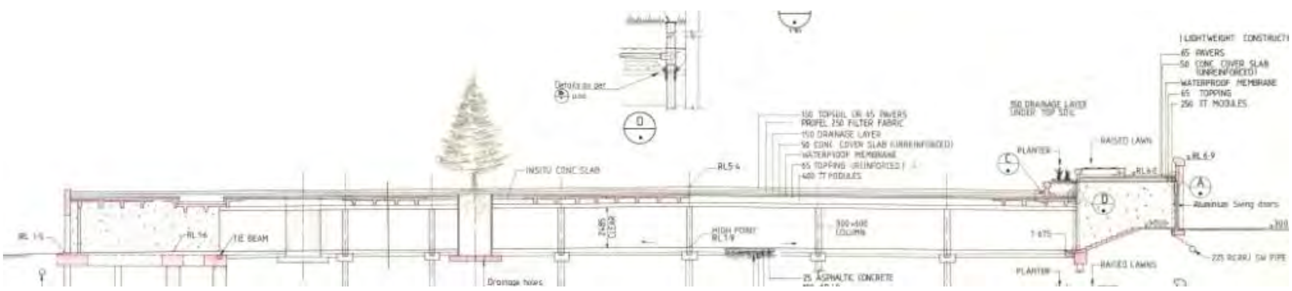


Figure 4 - Section through long direction

C.2.2 Material properties

Material strengths used in the assessment are shown in Table 6 and were based on information of the building consent package from 1989. Generally, strength/capacity reduction factors equal to 1.0 have been used.

Table 6 - Material strengths used in assessment

Parameter	Design parameters (1989) [MPa]	Probable parameters [MPa]
Concrete compression strength	30	45
Concrete modulus of elasticity	-	31500
Yield strength of Grade 275 steel reinforcement	275	324
Yield strength of Grade 380 steel reinforcement	380	455
Steel modulus of elasticity	200000	200000

C.2.3 Building masses, weights, mass eccentricity and P-Delta

The roof structure comprises double-tee units and cast in-situ flooring. These are supported by the beams of the precast concrete frames and the precast concrete wall panels.

Seismic weights were calculated by a summation of the self-weight of the roof structure, superimposed dead loads and live loads. Live load is multiplied by an earthquake-imposed action (live load) combination factor (ψ_E=0.3) and an area reduction factor for uniformly distributed actions (ψ_a=0.5). These factors are determined in accordance with Section 4.2 of NZS1170.5:2004 and Section 3.4.2 of AS/NZS1170.1:2002 respectively. Floor weights are summarised in Table 7.

Table 7 - Floor seismic weight summary

Level	Dead load G [kPa]	Seismic Live Load $Q \cdot \psi_E \cdot \psi_a$ [kPa]	Area of structure A [m^2]	Total M [kN]
Roof	14	0.6	3340	46500

C.2.4 Diaphragm modelling

For the purpose of this assessment the diaphragm is considered to be rigid and loads are distributed along the lateral load resisting systems on the basis of their relative stiffness.

As described in Section 5.3, the diaphragm is constructed using non-ductile mesh. Therefore, no reliable analysis can be carried out on the diaphragm itself.

C.2.5 Foundation/Soil interface modelling

The assessment of the foundation system is described in Section C.8. For the purpose of the assessment of other structural elements the foundations are considered to provide a rigid response. This corresponds to soils that are non-liquefied where resulting deformations in the piles are minimal.

When the ground liquefies from seismic shaking, the residual load paths in the foundations becomes unreliable. Therefore, the resulting performance of the structural elements is not further considered in the analysis for this scenario.

C.3 SEISMIC INPUT AND RESPONSE

The seismic loading parameters used in the assessment are shown in Table 8. For comparison the parameters from the original 1989 design are also shown.

Table 8 - Seismic coefficient parameters for NZS4203:1984 and NZS1170.5:2004

Parameter	NZS 4203:1984			NZS1170.5:2004		
Ground conditions	C	0.15	Flexible soil T < 0.45s Zone A	$C_h(T)$	3.0	Sub-soil class D T < 0.4s
Structure's fundamental period						
Location				Z	0.4	Wellington
Risk factor	R	1.0	Category 4 [4]	R_u	1.3	Importance level 3
Structure type	S	2.0	Case 6 [5]			
Structure materials	M	0.8	Reinforced non-prestressed concrete			
Near-fault factor				N	1.0	T < 1.5s
Structure ductility				k_μ	1.143	Structural ductility factor approximated at μ = 1.25
Structural response factor				S_p	0.925	

The horizontal acceleration response spectra determined in accordance with NZS1170.5 and NZS4203 is shown in Figure 5.

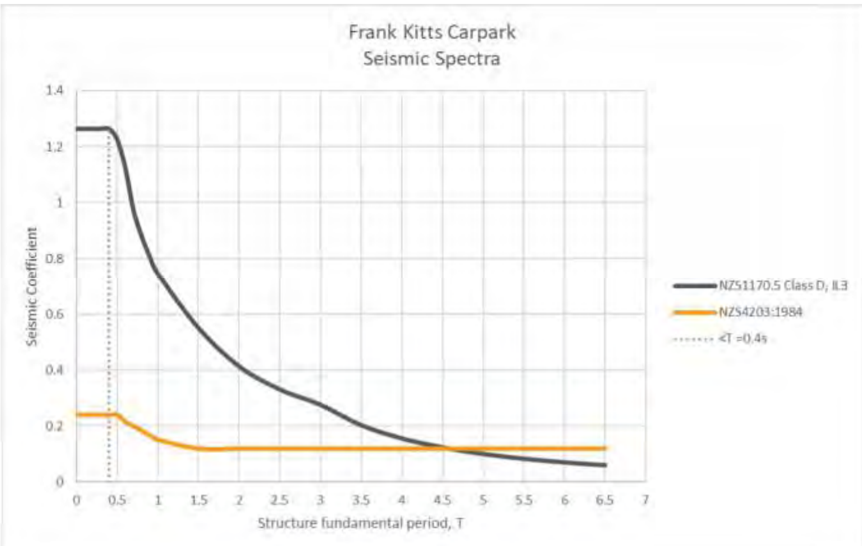


Figure 5 - Seismic spectra comparison between NZS4203:1984 and NZS1170.5:2004

[4] Category 4 buildings included 'buildings with normal occupancy or usage'.

[5] Case 6 buildings included 'single-storey cantilevered buildings supported by face loaded walls constructed of reinforced masonry or concrete.

C.4 ASSESSMENT OF DIAPHRAGM

The diaphragm comprises a 65mm thick cast in-situ topping slab on top of the double-tee units. It is tied into the lateral load resisting systems at the perimeter precast concrete wall panels and at the precast concrete frames. The diaphragm is assumed to be rigid and to distribute forces to the lateral load resisting systems relative to their stiffness.

C.4.1 Connection between diaphragm and lateral load resisting systems

The roof diaphragm is connected to the perimeter precast concrete wall panels using 12mm deformed Grade 275 starter bars. The typical connection to the precast concrete wall panels is shown in Figure 6.

The load transfer from the diaphragm to the lateral load resisting systems is considered to go through a shear-friction mechanism. The capacity of this connection at precast concrete wall panel PC5 corresponds to 14% of ULS seismic loading. In general, the connection between diaphragm and the precast concrete wall panel perform at 15% and 23% of ULS seismic loading.

For reference, the shear stress in the concrete reaches approximately 12 MPa at 100% of ULS seismic loading.

Since there is nominal inelastic contribution considered in the shear-friction form of load transfer, the capacity of this connection is rated at 15%NBS (IL3).

C.4.2 Diaphragm

The roof diaphragm consists of 65mm thick concrete slab that is reinforced with 665 mesh – a cold drawn wire mesh product. The roof diaphragm is required to distribute seismic inertia load of the structure to the lateral load resisting systems.

Cold drawn wire mesh is considered to exhibit non-ductile or brittle behaviour, with low elongation capacity. Non-ductile mesh has been observed to perform poorly in floor/roof diaphragms in previous New Zealand earthquakes such as the 2010/2011 Canterbury and 2016 Kaikoura events. Failure and fracture of this mesh was observed in buildings in Christchurch and Wellington from these earthquake events.

The 2017 Engineering Assessment Guidelines do not directly comment on the appropriate method to assess diaphragms with this type of non-ductile reinforcing.

Reference to the latest industry best practice, captured in ASCE 41-17 and the proposed revision of Chapter C5 of the Engineering Assessment Guidelines ("Yellow C5") has been made to provide commentary on the expected seismic performance of this diaphragm. Yellow C5 and ASCE 41-17 state that an elastic analysis can be performed where the total strain captures the effects from the segmental nature of the individual components (e.g. the effects from shrinkage and creep are included in the assessment of strains induced in the mesh reinforcement). Furthermore, Yellow C5 states that 'non-ductile' mesh should not be relied on in a strut-and-tie type of diaphragm analysis.

This provides no solution, since an analysis that includes the effects from shrinkage and creep is deemed practically impossible to result in reliable results. This is mainly due to the large number of side-effects that could significantly affect the parameters in the analysis. Attempting to identify and evaluate all these parameters is impractical. Therefore, the capacity of the diaphragm is considered inconclusive and the Engineering Assessment Guidelines require this to be rated at 15%NBS (IL3).

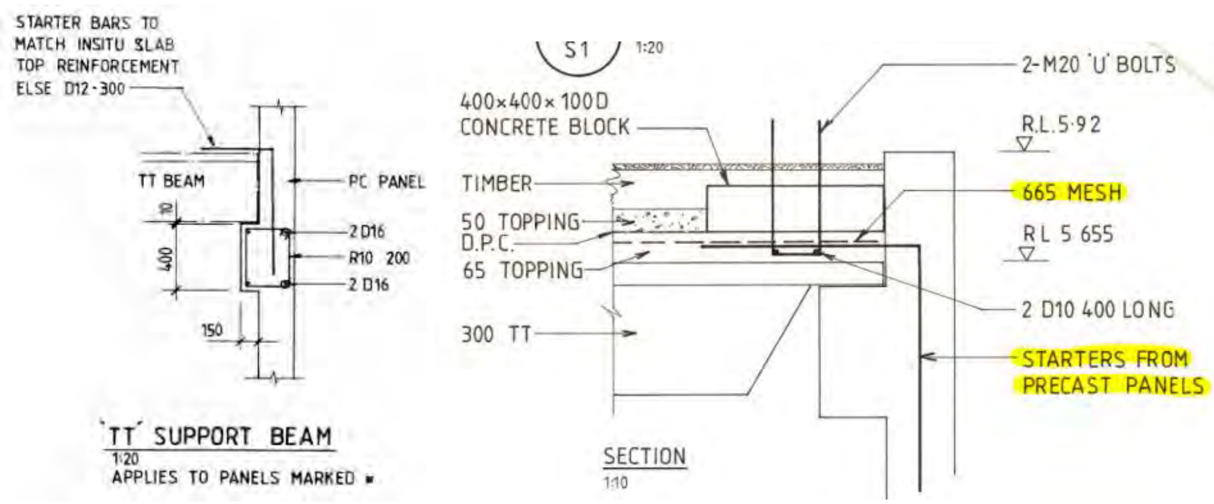


Figure 6 - Extract from as-built drawing S8 and S16 - diaphragm to wall connection

(note that the extract on the left shows a full web-height support and the extract on the right shows a flange hung double-tee, however this has not been observed on site)

C.5 DOUBLE-TEE UNITS

The 300TT and 400TT type double-tee units have been used in the design of the roof slab. They typically span in the North-South direction between the precast concrete frames and the precast concrete wall panels. The double-tee units are web-supported, as indicated in Figure 7. The double-tee units have been assessed for the seating length and bearing capacity.

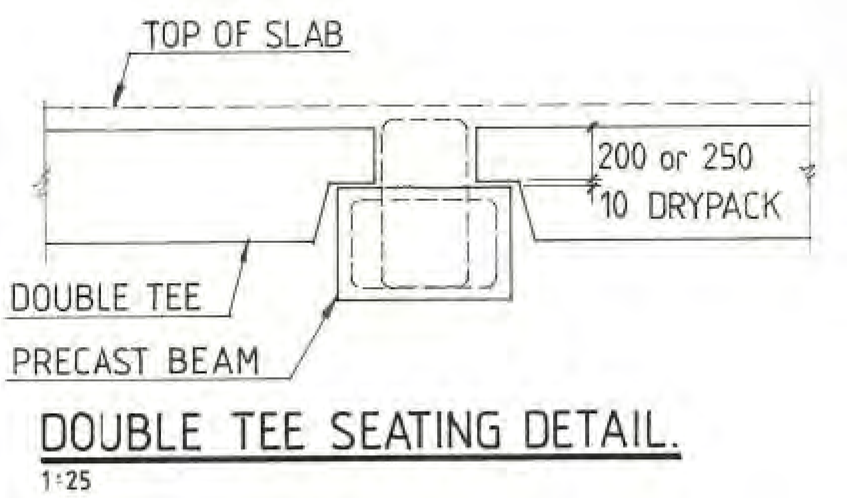


Figure 7 - Double tee units support - extracted from drawing S2 of the as-built drawings

The initial seating length for the double-tee units was intended to be 125mm according to the drawings. Site observations indicate that some double-tee units show some spalling at the end of the precast unit. With reduced seating length, some units may be prone to unseating or loss of support or the bearing capacity of the support being exceeded.

An assessment for the seating capacity of the double-tee units was undertaken in accordance with Appendix C5E of the Yellow C5 of the Engineering Assessment Guidelines, since this is the latest industry practice for the assessment of precast flooring systems. This assessment takes the initial seating length and has subtracted the relative distances for long-term effects, construction tolerances and the drifts during seismic shaking, these values are given in Table 9.

Table 9 - Considerations for loss of seating

Considerations	Depth
Initial spalling due to construction issue	Worst case found on site: About ¾ of length spalled = 95 mm
Construction tolerance	20 mm *(Maximum observed on site, although some photos show that double-tee units sit directly against beams.)*
Spalling based on drift	Ledge spalling = 0 mm Unit spalling = 15 mm
Creep and shrinkage shortening	5 mm
Drift	2 - 5 mm

For the double-tee units this means that they generally have enough residual seating length to cater for expected building drifts (displacements) for >100%NBS (IL3) level of loading.

However, for damaged units, where initial spalling has occurred there is limited seating length remaining. The area remaining for bearing at the ends of the double-tee units will be reduced. For the double-tee units that were found with initial spalling, the potential remaining contact length is within the concrete cover of the beams. Bearing capacities have been checked in accordance with Section 16.3 of NZS3101:2006 Amendment 3. It was found that the residual bearing area is expected to be adequate for building drifts (displacements) up to 40%NBS (IL3) level of loading.

Figure 8 - Double tee support conditions - (Left) double-tee units positioned directly against the beam and (right) initial spalling at the back of the double-tee unit

C.6 ASSESSMENT OF PRECAST CONCRETE WALL PANELS

Precast concrete wall panels are located on the perimeter of the structure. Individual precast concrete wall panels are typically around 4m in length with a vertical joint between each panel. Therefore, the precast wall panels are considered to act as independent cantilever wall elements.

The precast concrete wall panels are connected to the roof structure and the foundation beams by 12mm diameter starter bars. Typical details are provided in Figure 6 and Figure 10 for the connection to the roof and the foundation beam respectively.

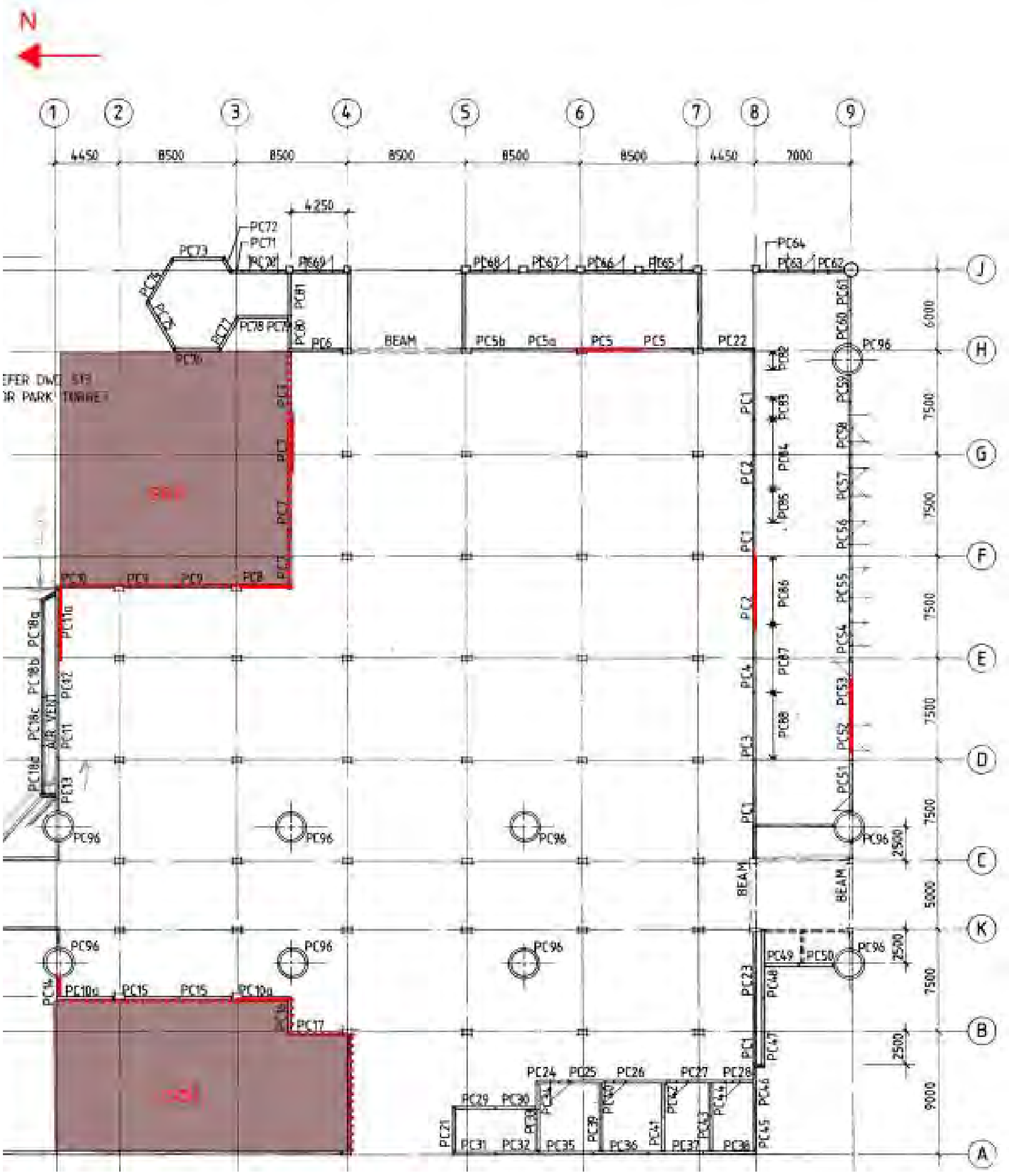


Figure 9 - Extract of the as-built drawing S7 - showing the precast concrete wall panels around the perimeter of the structure

C.6.1 In-plane Flexural Strength Capacity

150mm thick and 200mm thick precast concrete wall panels are used as the main lateral load resisting system on the perimeter of the carpark. Precast panel vertical flexural reinforcing and starter bar details are setup similar for each panel but varies depending on the panel thickness (refer Figure 11). The probable in-plane flexural capacity of the precast concrete wall panels has been checked against moment demand from ULS seismic loading according to C5 Concrete Building, Section C5.5.2.1.

Starter bars connect the precast concrete wall panels to the foundation beams as shown in Figure 10. Starter bars from the foundation beams are grouted into corrugated ducts cast into the base of the precast wall panels. As the precast concrete wall panels experience in-plane flexure, the starter bars will have to transfer this to the foundation beams in shear and tension from flexure. This has been found to be the first mechanism to yield under in-plane demands.

The development of the starter bars has been checked for the configuration in the corrugated ducts and the assumption that there is no direct contact with the vertical reinforcement of the wall. It was found that this has been developed appropriately to develop the yield capacity of the starter bars.

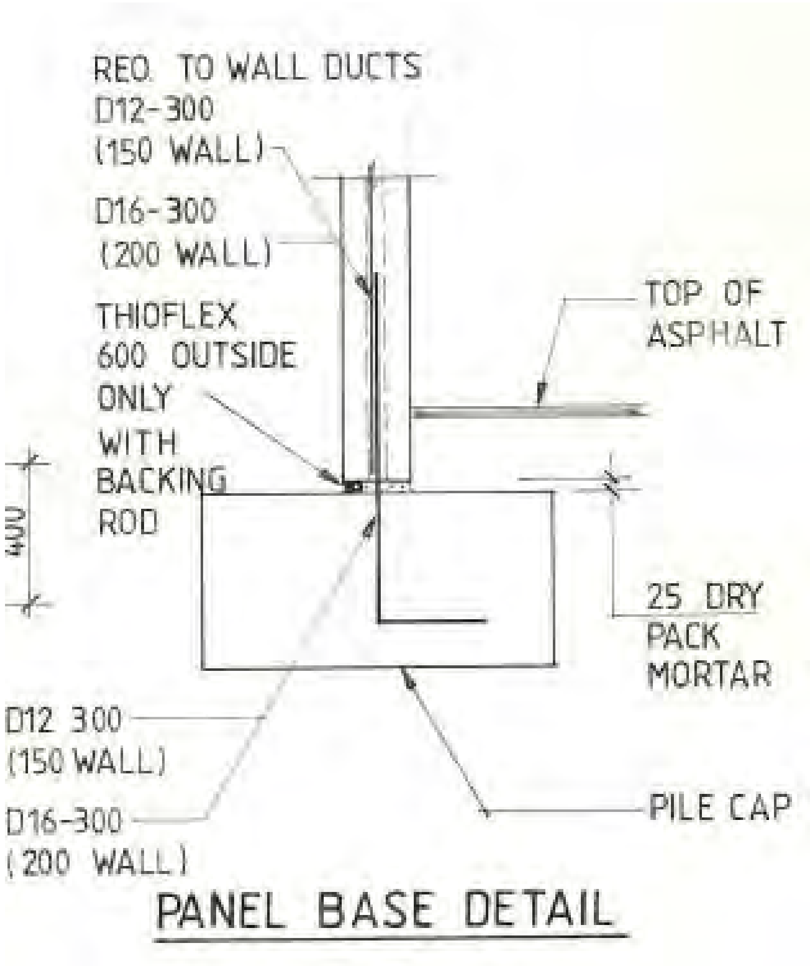


Figure 10 - Extract from the as-built drawingS7 - showing the connection between precast concrete wall panel and footing

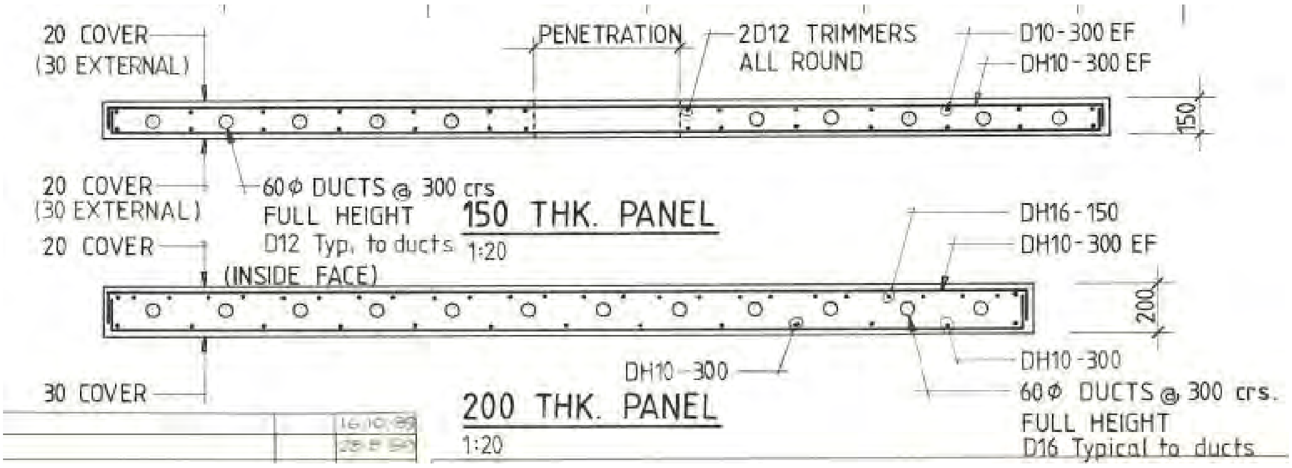


Figure 11 - Extract from the as-built drawing S7 - showing the precast concrete wall panels in section

Several walls around the perimeter of the building were reviewed, denoted as red lines on the building plan in Figure 9. These walls have been selected as they are repetitive and located on representative bracing lines around the building. They also represent a good variation in panel heights and widths which gives an appropriate indication on how the precast concrete wall panels around structure can be expected to perform. ULS seismic load demands on each bracing line were determined using a distribution of total lateral load in proportion to the stiffness of the wall panels along each bracing line. Expected precast wall panel performance is as follows:

Table 10 - Precast wall panel in-plane flexure

Panel	Panel thickness	Grid location	Yield loading
PC1 (similar to PC2, PC3 and PC4)	150mm	Grid 8	20 %ULS
PC5	150mm	Grid H	55 %ULS
PC7	200mm	Between Grids 3 & 4	55 %ULS
PC8	200mm	Between Grids E & F	100 %ULS
PC10a	200mm	Between Grids B & K	100 %ULS
PC11a	150mm	Grid 1	20 %ULS
PC14	150mm	Grid 1	55 %ULS
PC52	150mm	Grid 9	20 %ULS
PC53	150mm	Grid 9	15 %ULS

In the main section of the building, used as the carpark, PC2 and PC11a have the lowest performance rating. These precast concrete wall panels yield at 20% of ULS seismic loading. The 150mm thick precast wall panels were limited by the flexural capacity of starter bar connection, as this is the weakest link in the chain. Simply, the reinforcement area of the starter bar connection is less than the that of the panel itself.

The resultant performance of PC52 and PC53 are particularly low as well, this is due to the opening in these panels. PC52 and PC53 are located on the outer perimeter which has been used as boat shed. The analysis has however shown that yielding of the precast concrete wall panels at the boat sheds can be redistributed along the remainder of the structure.

C.6.2 Redistribution potential when yielding

The analysis has found that the precast concrete wall panels on gridline 8 generally attract the most loading for seismic shaking in East-West direction and for torsional effects for seismic shaking in any direction. This meets the expectations as this gridline generally has the largest stiffness of all the bracing lines attached to the diaphragm.

Gridline 8 consist of several panels which are referred to as PC1 (4x), PC2 (2x), PC3 and PC4, which are all of similar size and have similar flexural capacity. As described in Section C.6.1, these panels all yield at 20% of ULS seismic loading. Upon yielding these precast concrete wall panels will have the ability to deform inelastic and redistribute the loads to the other bracing lines.

To investigate the response of the structure after this initial yielding of gridline 8, these precast concrete wall panels (PC1, PC2, PC3, PC4 and other) are considered to respond with reduced stiffness in further shear resistance of the structure.

Initiating this step in the analysis has found that the centre of stiffness, is moving approximately 15m to the North to redistribute the loads between the remaining elastic bracing lines. Upon this redistribution however, it is found that under the same level of loading (~22% of ULS seismic loading) the other primary gridlines 1 and gridline 3/4 are also initiating yielding due to the additional load on the walls from torsional demands. The total system is ultimately able to redistribute the loading to other gridlines to increase the overall shear resistance of the system and the whole system yields in the East-West direction at 24% of ULS seismic loading. The North-South direction has more walls to redistribute the forces along and initiates yielding of the precast concrete wall panels at higher seismic loading.

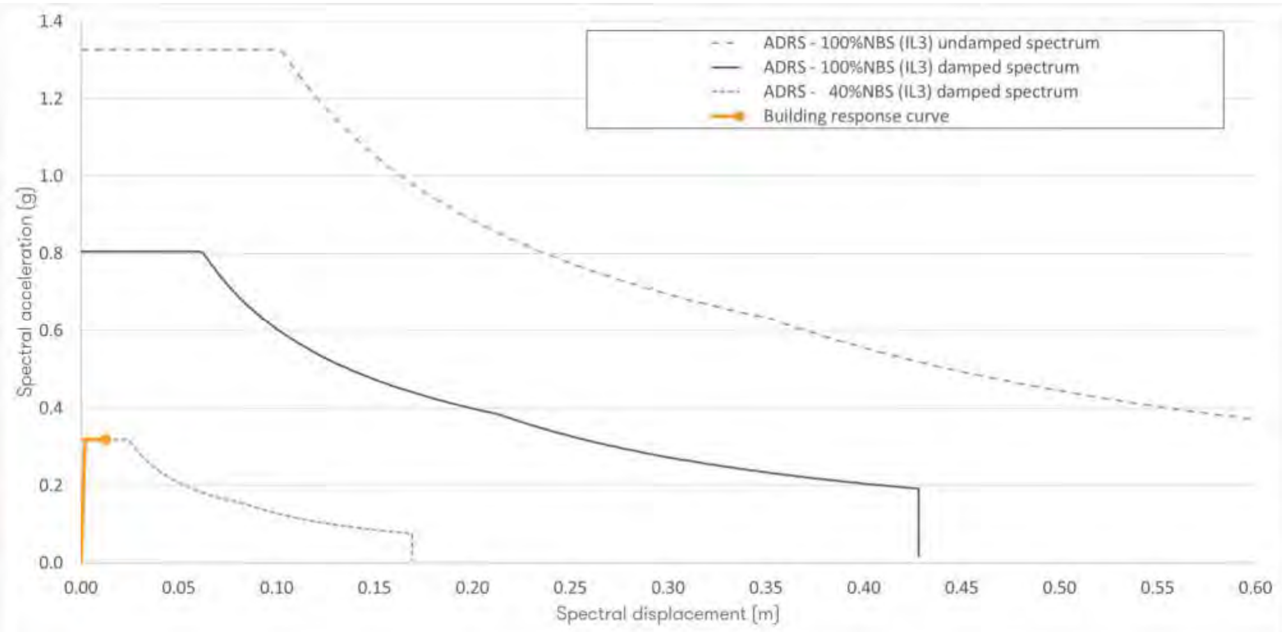


Figure 12 - Building response curve of the precast concrete walls in the East-West direction

The precast concrete panels on gridline 1 (PC11-PC14) and gridline 3/4 (PC7 (4x) are plotted in the ADRS of Figure 12 alongside the panels of gridline 8 to create a building response curve for the performance in the East-West direction. For this analysis, the seismic loading has taken an initial system ductility of $\mu = 1.0$, as the ductility is determined following the building response curve.

This response is eventually limited by the inelastic rotation capacity at precast concrete wall panel PC1. The starter bars have the ability to develop their tensile strain capacity in full before eventually failing, which theoretically gives the starter bars a member ductility of $\mu_{mem} \sim 25.0$. However, other factors, such as bar buckling and bond failure are limiting this, since the starter bars have nominal confinement when the panel starts performing inelastically. Therefore, a ductility of the overall system of $\mu = 6.0$ is set as a boundary condition. The precast concrete panels respond with a yield drift of $\delta_y \sim 2$ mm and then can deform up to an inelastic displacement of $\delta_{cap} \sim 12$ mm.

The ADRS is furthermore set up using the recommendations from Appendix C2D of the Engineering Assessment Guidelines to determine the system inherent damping and the system hysteric damping from the inelastic behaviour. The system inherent damping is set at a typical 5%. The system hysteric damping is determined in accordance with Table C2D.1 of the assessment guidelines for a concrete wall system at 12%.

Following the inelastic response, which is limited by the available tensile strain in the starter bars, the precast concrete wall panels perform at 40%NBS (IL3) for in-plane flexure. It is noted that those panels where the likely flexural mechanisms form first, are not exposed to out-of-plane loading from earth pressures.

C.6.3 Out-of-plane flexural strength capacity

The precast concrete wall panels experience out-of-plane flexure due to earth pressure acting on them. These walls are denoted with red dotted line on Figure 9. PC10a between gridline B and gridline K, PC7 between gridline 3 and gridline 4 and JW20 on gridline 4 have been selected for analysis, which gives a good indication of the overall performance of the precast concrete retaining walls that experience seismic earth pressure.

Precast panels retaining soil are assumed to be pinned at top and bottom of the wall, resisting out-of-plane loadings as simply supported members.

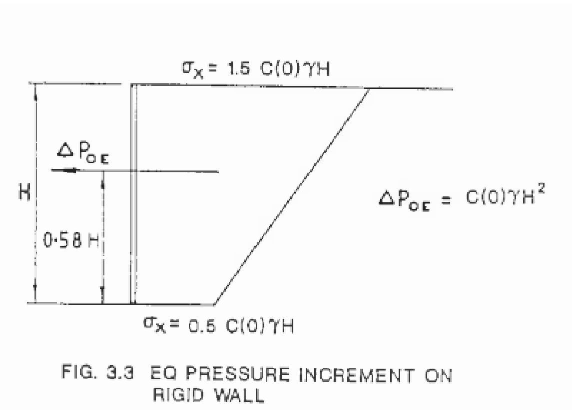


Figure 13 - Extract from Wood and Elms showing earthquake pressure on rigid wall

The probable out-of-plane flexural capacity has been checked against moment demand due to earth load next to Jervois Quay according to C5 Concrete Building, Section C5.5.2.1. Rigid wall approach as shown in Figure 11 has been adopted to obtain the earthquake pressure acting on precast concrete wall panels. This has been combined with the triangularly distributed static earth pressures.

The assessment found that the precast concrete wall panels are adequate to support out-of-plane loads at or beyond 100% ULS seismic loading.

C.6.4 Combination of in-plane and out-of-plane flexural behaviour

Under orthogonal seismic loading, precast wall panels that are required to carry soil retaining loads will be subject to concurrent in-plane and out-of-plane actions. An assessment was carried out by applying 100% ULS seismic loading in-plane and 30% seismic earth pressure out-of-plane and vice versa.

Stress in the wall panel reinforcement were found under concurrent loading by summing reinforcement stresses under each direction of loading. The resultant ULS seismic loading that precast concrete wall panels can resist are as follows: -

Table 11 - Precast wall panel concurrent loading flexure

Panel	Panel thickness	Grid location	%ULS IP + 30% OOP	%ULS OOP + 30% IP
PC10a	200mm	Between Grids B & K	50 %ULS	65 %ULS
JW20	200mm	Between Grids 3 & 4	70 %ULS	85 %ULS
PC7	200mm	Between Grids 3 & 4	55 %ULS	70 %ULS
PC9	200mm	Between Grids E & F	65 %ULS	80 %ULS
IP = in-plane loading, OOP = out-of-plane loading				

As in-plane seismic loading on each panel is higher than out-of-plane seismic earth pressures, the wall panels can be seen to perform better loading in the out-of-plane direction. To conclude, the precast concrete wall panels under concurrent loading perform at 50%NBS(IL3).

C.6.5 Shear capacity

Analysis of the shear capacity has found that the out-of-plane shear demands are insignificant as compared to the in-plane shear demand. The shear capacity in the precast concrete wall panels help resist ULS seismic loading that is applied through the 12mm starter bars from the diaphragm of the roof structure. The probable shear capacity of the precast concrete wall panel has been determined based on Section C5.5.2.3 of the Engineering Assessment Guidelines.

The precast concrete wall panels are considered to perform at a curvature ductility smaller than $\mu_\phi = 2$, therefore the concrete shear strength degradation factor was considered to be $\gamma = 0.29$. The axial load component was found to be minimal.

The wall panels are reinforced using Grade 380 horizontal bars, 10mm in diameter (DH10) at 300mm centres each face over the height of the walls. The typical lay-out of the reinforcement of the walls is given in Figure 11.

In total there are 102 number of precast concrete wall panels in the structure, of the total walls there are:

- 17 panels that can resist between 42% and 67% of ULS seismic loading

- 6 panels that can resist between 68% and 100% of ULS seismic loading
- 79 panels that can resist more than 100% of ULS seismic loading

For this assessment the walls rate at 42%NBS (IL3) on precast concrete wall panel PC11a. However, the panels generally have sufficient shear reinforcement to respond flexurally and behave in a ductile way.

C.7 ASSESSMENT OF PRECAST CONCRETE FRAME

Precast concrete frames are provided to support gravity loads from the roof structure. The frames are constructed from precast beams and precast column elements, stitched at each beam-column location. The beams were precast part depth, with the upper portion of the beam cast as part of the roof topping slab. The columns are supported by concrete foundation pads that are founded on single or double driven piles.

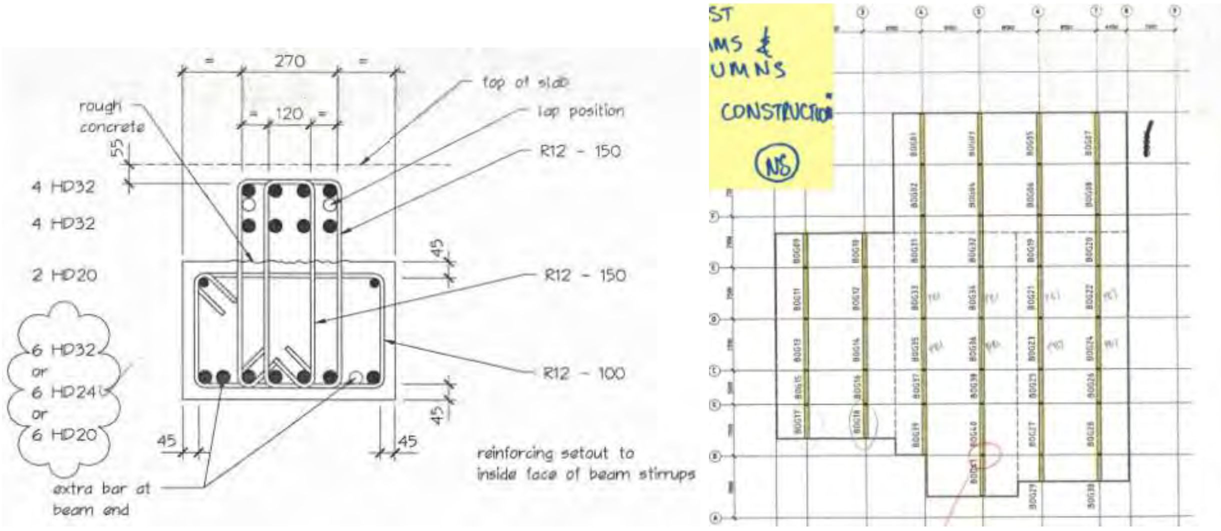


Figure 14 - Extract from as-built drawing - plan of precast concrete frame with foundation; extent of beam of frame

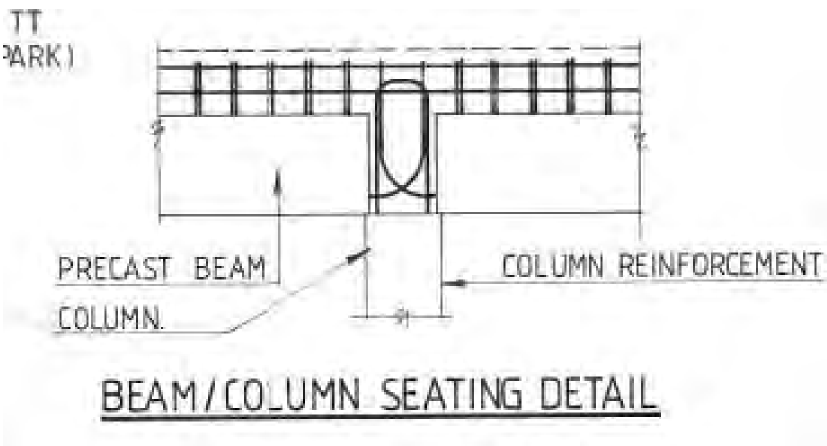


Figure 15 - Extract from as-built drawing S2 - showing beam-column seating detail

The connection between column and beam shown in the following figure enables moment to be transferred at the rigid connection.

At the West end of the structure, the precast concrete frames are supported on the precast concrete wall panels, this detail is shown in Figure 16. A sliding connection is provided, and the precast concrete wall panels are therefore not subject to large forces. In addition, the out-of-plane performance of the precast concrete wall panels is more flexible than the columns due to its relatively low stiffness. Therefore, even if loads would transfer along this connection, they would only be small portion of the ULS seismic loading. It is therefore represented by roller support in analysis.

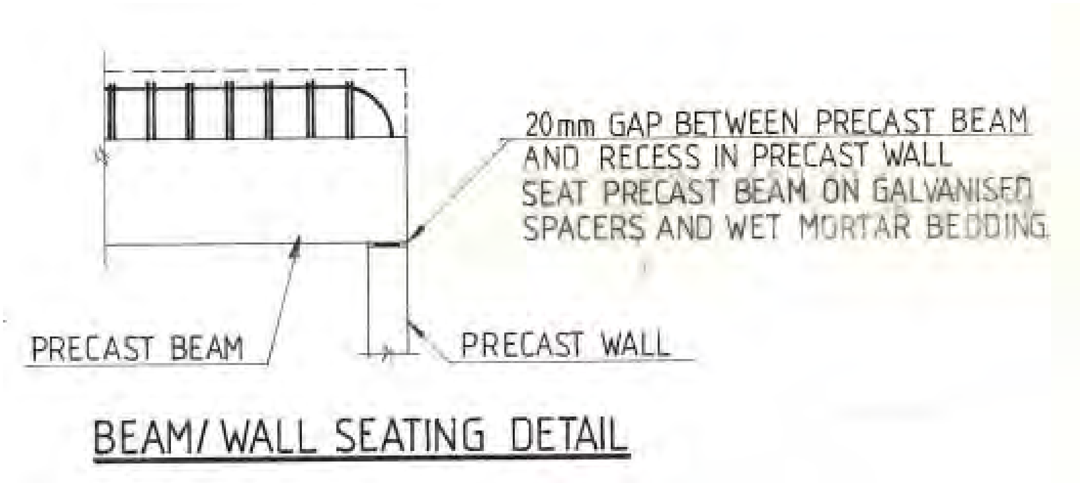


Figure 16 - Extract from as-built drawing S2 - showing beam-wall seating detail

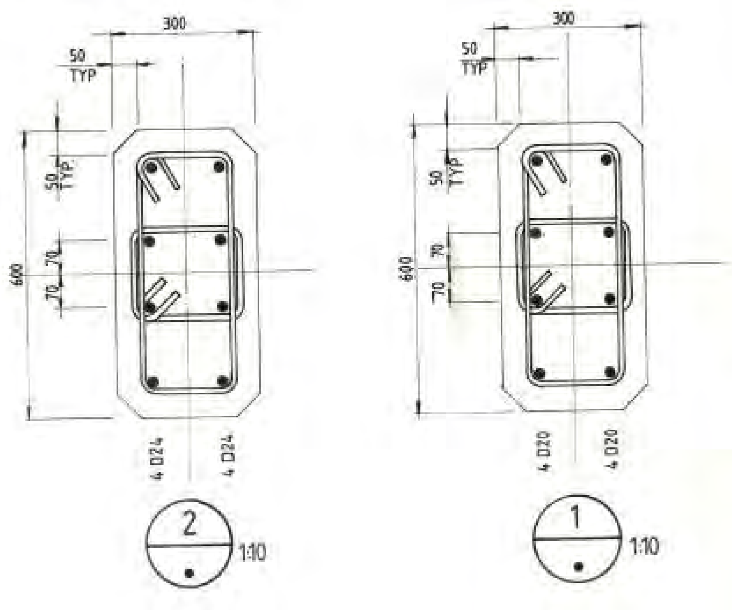


Figure 17 - Extract from as-built drawing S6 - showing column reinforcement detail

The columns are supported by either single or double precast concrete driven piles. It is not directly obvious from the drawing why a distinction has been made between these two scenarios. However, for the purpose of this assessment the connection with the foundation system is modelled as a pinned connection if the columns are supported by a single pile and a fixed connection if the column is supported by 2 piles.

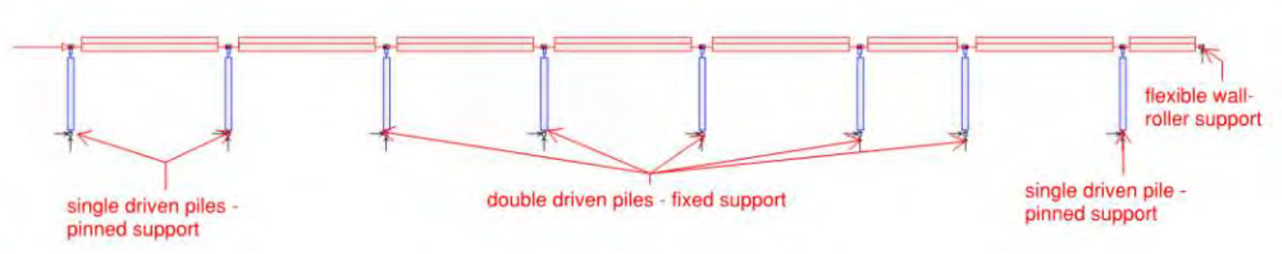


Figure 18 - Extract from Microstran model - showing gridline 6 precast concrete frame model

Based on a rigid diaphragm assumption, the frames are not subjected to any significant lateral load demands, the diaphragm distributing lateral loads to the perimeter walls. As diaphragm is not expected to perform adequately under seismic loading, the frames can be shown to have minimal resistance to support lateral loads. This further supports the notion that the frames are provided as gravity load only elements and not intended to function or contribute significantly as part of the lateral load resisting system.

C.8 ASSESSMENT OF FOUNDATIONS

The assessment of the foundations has been carried out in collaboration with the findings of the geotechnical assessment (Appendix B).

Performance of the foundations has considered the following scenarios which account for the liquefaction and lateral spread behaviour of the soils which are expected at the site:

1. 100% structural inertia combined with non-liquefied soil properties
2. 100% structural inertia combined with liquefied soil properties
3. 80% structural inertia with liquefied soil properties and cyclic displacements
4. Lateral spread effects, with liquefied soil properties

These scenarios are selected based on the guidance by design manuals such as NZGS/MBIE: Module 4: 2016 and the NZTA Bridge Manual version 3.3: 2018. These indicate that it is unlikely that the full dynamic response of the building would be superimposed simultaneously with the full kinematic loading on the pile.

C.8.1 Foundation system setup

The bored concrete piles are Ø800mm and are embedded for 1m in the denser alluvium ground layer approximately 12m below ground. All other piles of the structure are precast concrete driven piles which are square 400mm which are founded in the same layer.

The bored concrete piles are generally located around the perimeter of the building beneath the walls of the building, except for the Eastern end of the building (seaward side) where the walls are supported on driven piles. Columns of the precast concrete frames are supported on driven piles.

At ground floor level there is no structural floor slab provided to the building. The base of the columns within the building are tied together with relatively small tie beams. These tie beams are not able to distribute loads to adjacent foundation elements. Hence, the assessment is continued on the basis that the loads on each bracing line are to be resisted by the piles located on that bracing line.

The piles are tied in with the foundation beams and foundation pads as shown in Figure 19. From the 1989 consent documentation it is understood that the longitudinal bars of the piles are developed in the foundation beams and foundation pads using hooks in accordance with NZS3101.

The piles and the connections to the foundation beams appear to be designed in accordance with capacity design principles. The piles and the foundations beams have sufficient transverse reinforcement to develop the flexural capacity of the piles.

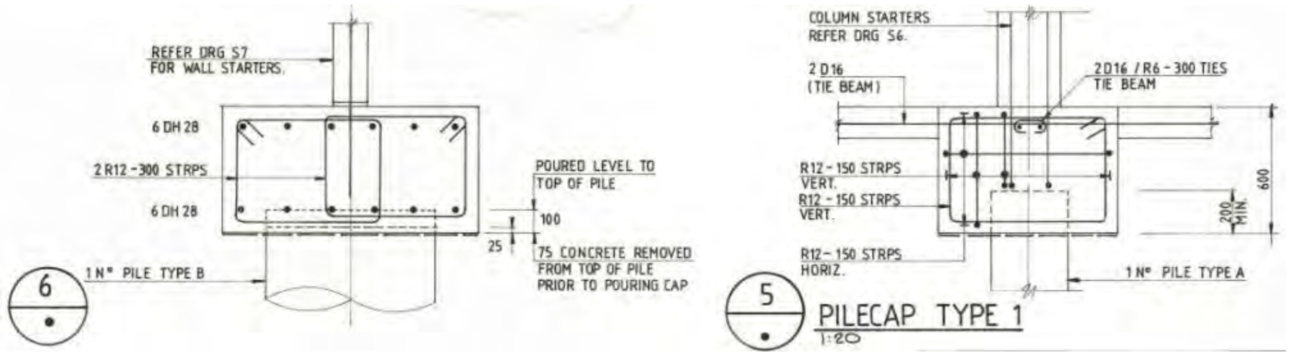


Figure 19 - Extract from as-built drawings - showing foundation beams and pile caps

C.8.2 Response of the foundations in non-liquefied ground

In accordance with the geotechnical recommendations, it is unlikely that the ground will remain non-liquefied during strong seismic shaking, however the initial response may be non-liquefied. During this scenario the piles are modelled with a 'fixed head' and they are found to yield at approximately 20% of ULS seismic loading. The detailing of the foundation system however allows the pile to perform inelastically and form plastic hinges. Once the pile is yielded, the piles are analysed with a 'pinned head' and an applied bending moment that represents the strength of the plastic hinge. The push-over profile of this scenario from the geotechnical analysis is given in Appendix B.

The inelastic rotation capacity however may be limited due to shear degradation from the rotations in the potential plastic hinge zones (PPHZ). The geotechnical analysis has shown that the shear demand in the pile is close to its capacity. Hence, on onset of the shear strength degradation at a member curvature ductility of $\mu_\phi \geq 3.0$, the members are becoming less resistant of shear forces and are becoming much more flexible. This results in significant displacements as illustrated in Figure 20, which is a modified version of the graph in Appendix B, limited by the structural capacity of the piles.

Scenario 1a represents the performance up to forming the plastic hinge at the top and scenario 1b represents the capacity until second hinge is formed deeper in the ground. Scenario 2a and 2b are similar but correspond to the scenario with liquefied ground conditions respectively. For reference, the shear capacity of the pile (V_n = 825 kN) corresponds to approximately 25% of the ULS seismic loading.

While the piles have got the inelastic rotation capacity to undergo these inelastic displacements there is less certainty in how much this dissipates the energy of the seismic shaking and how these large displacements may affect the superstructure. However, the piles themselves perform around the 40-50%NBS mark for scenarios 1 and 2.

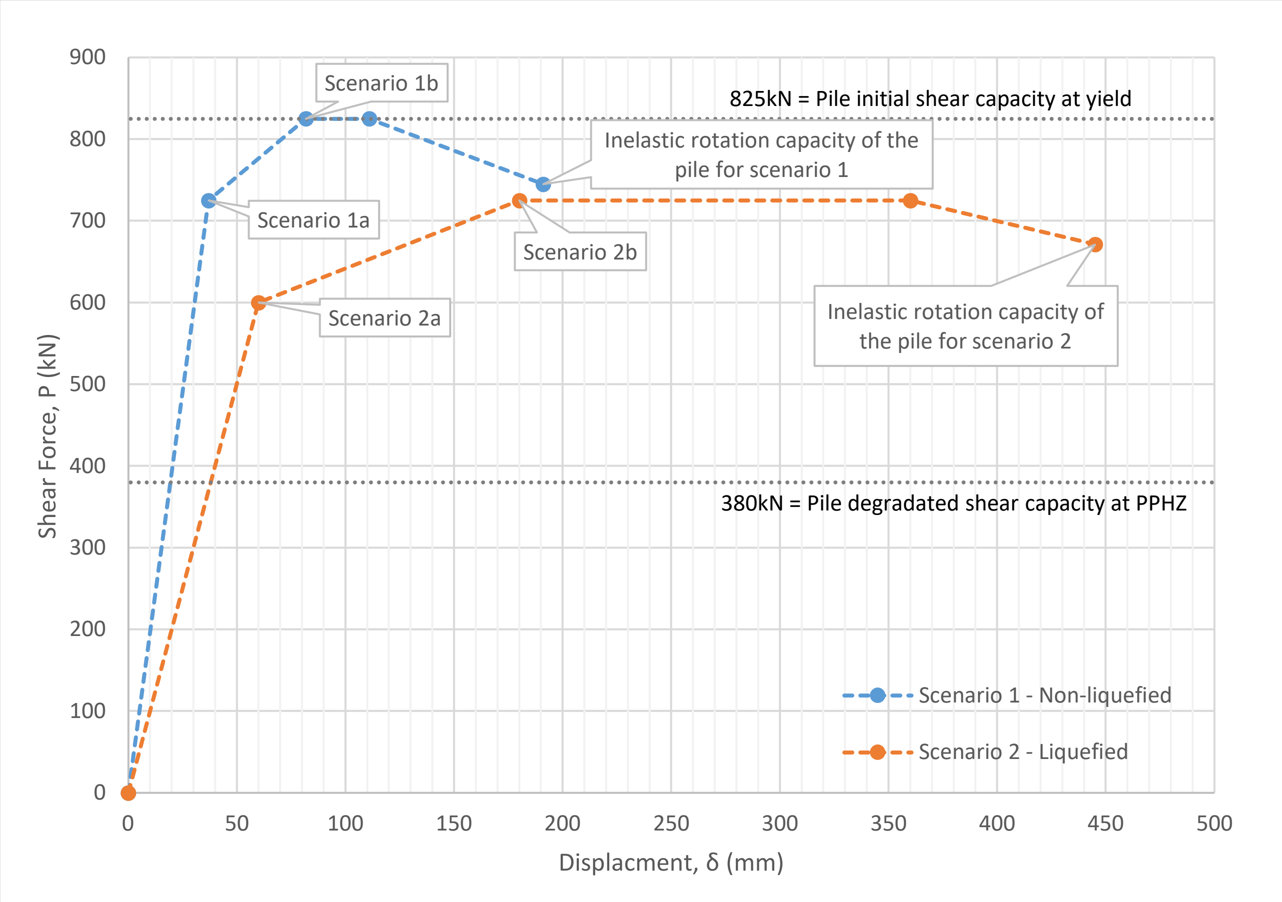


Figure 20 - Pile response curves

C.8.3 Response of the foundations in liquefied ground

The geotechnical recommendations note that a significant step change in ground behaviour may be expected at 20% to 30% of ULS seismic loading, due to triggering of liquefaction and a rapid increase in lateral spread displacements within the liquefied material.

The limited embedment of the piles into the alluvium layer provides limited resistance at the toe of the pile to lateral loads and only nominal pile base fixity. This may result in the following scenarios when liquefaction develops. For reference the image of Figure 21 is added to illustrate the behaviour during these scenarios.

If the piles have hinged at the top, under structural inertial loads, and embedment at the toe of the pile is insufficient to develop pile flexure, there is no practical mechanism to resist lateral loads through the piles (pin-pin column situation). A limit to lateral load resistance of 25% of ULS seismic loading is expected as mentioned above.

If piles have yet to hinged at the top and embedment of the pile at the toe can provide lateral resistance, the pile may act in cantilever action. Development of a plastic hinge at the top of the pile along with inelastic rotations will allow lateral loads to be resisted, to a point. A review of the capacity of the shallow pile embedment at the base to lateral loads show this to be exceeded at approximately 10% of the ULS seismic loading. Therefore, this load path can also not be relied upon beyond this point.

Both scenarios have thus far excluded effects of cyclic displacements or lateral spread imposing additional displacement demands on the piles. Since these effects are also significantly increased during liquefied ground response, this supports the suggestion that response of the pile post-liquefaction is unlikely to be reliable and will exceed the inelastic rotation capacity of the piles. It is not possible to reliably determine the structure's response if significant expected lateral spread displacement demands of 1m to 5m develop at this site.

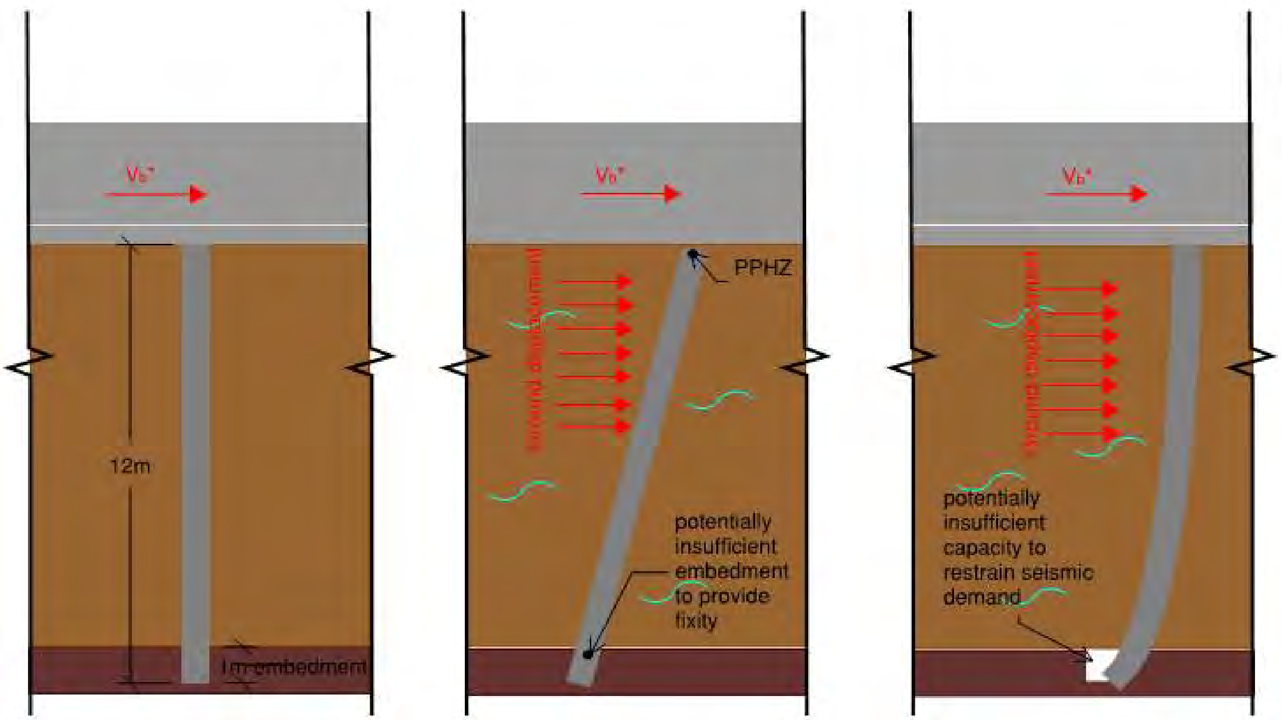


Figure 21 - Illustration of pile behaviour under liquefaction scenarios where:

- V^*_b describes the inertia of the structure itself
- Ground displacements are caused by cyclic displacements or lateral spread
- PPHZ refers to the location of a potential plastic hinge zone in the pile

C.8.4 Conclusion

In conclusion, the piles have the ability to develop their flexural capacity and perform inelastically during seismic events. However, once liquefaction develops across the site, there is limited resistance to lateral loading available from the foundations. Additionally, demands imposed on the foundation system due to cyclic displacements and lateral spread displacements are likely to quickly exceed the capacity of the foundations.

The step change in geotechnical response of the site may lead to a significant change in the response of the structure. Large displacements or differential displacements within the foundation system may lead to unexpected behaviour other structural elements, such as loss of gravity load-paths or the like. It is possible that this situation could lead to life safety issues in or near the building. However, this cannot be determined through calculation methods. The onset of liquefaction is therefore considered to be the limiting factor for the ground performance and the soil-structure interaction.

It is understood that further geotechnical investigations or analysis or further structural analysis is unlikely to significantly improve on these conclusions. Following Section C4.5.3.2 of the Engineering Assessment

Guidelines, the possible step change in structural response, due to a step change in geotechnical behaviour requires any %NBS ratings to be factored by 0.5. This is primarily required due to uncertainty in determining the geotechnical response.

As discussed in the Geotechnical Assessment (Appendix B), response of the site during the 2016 Kaikoura Earthquake event provides evidence to reduce the level of uncertainty in the geotechnical assessment such that this 0.5 factor can be excluded.

The foundations and soil-structure interaction therefore perform between 20-30%NBS (IL3).

C.9 SECONDARY STRUCTURAL AND NON-STRUCTURAL ELEMENTS (SSNS)

The secondary structural and non-structural elements (SSNS) that were considered in this assessment are listed in Table 12. Consideration includes a decision to include or exclude these elements from an overall %NBS Assessed Seismic Rating. A more detailed description on the expected performance of each element is given below the Table. The carpark area is considered a space class II, an open space with minimal furniture.

The intent of the EPB Methodology is to identify and rate those building parts which, should they lose support or collapse, would present an unavoidable danger that a number of people are exposed to, e.g. it is a significant life-safety hazard (SLSH).

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

Table 12 - Secondary structural and non-structural elements

Building Part	Included in this assessment
Circular planter boxes at several locations in the structure	No
Masonry walls in boat sheds on the Southern end of the structure	No
Statue/Flag/Pole on roof of structure	No
Look-out tower (the Oriel) at the South-West corner of the building	Yes
Wall cap on the perimeter wall	Yes

Circular planter boxes

The planter boxes are thin-walled circular concrete boxes that allow for trees or large bushes. They are included in the footprint of the structure at several locations.

Where the planter boxes are located, a cast in-situ concrete slab has formed the connection with the remainder of the structure around it. There appears to be no positive connection between the planter boxes and the diaphragm. The planter boxes have their own foundation pads, which are not tied in with the foundation system of the rest of the structure.

When the structure moves due to seismic shaking, the planter boxes are forced to follow, since they are locked in by the roof structure. Excessive drifts of the roof structure may cause cracking (and potentially failure) of the thin-walled concrete shell. Failure will likely lead to the sudden spread of soil throughout the carpark area that may cover people and cars in a layer of soil.

The planter boxes are considered an appendix or ornament to the structure. In accordance with Table A4.1 of the Engineering Assessment Guidelines, these are only an SLSH if they are located above space class I.

Masonry walls in boatsheds

In the boatsheds on the South and the West ends of the structure, masonry walls are formed. These walls have no obvious structural intent, they are not providing gravity or lateral support for the structure. They are estimated to be 1.5m in height and are considered to be lightly reinforced.

The area in the boatsheds can be considered to be a space class V, storage areas that are not expected to be occupied during an earthquake. In accordance with Table A4.1 of the Engineering Assessment Guidelines, heavy partitions are only an SLSH if they border a space class I, II, III or IV.

Statues, flags and other poles on the roof structure

On the roof of the structure several ornaments or appendages are built as scenery elements. In accordance with Table A4.1 of the Engineering Assessment Guidelines, these are only an SLSH if they are heavy and are above a space class I.

Look-out tower (the Oriel)

The look-out tower (the Oriel) is located at South-East corner of the structure. The look-out tower is connected to the rest of the structure at the step to the slab as illustrated in Figure 22 and Figure 23.

The slab of the tower is connected to the rest of the structure through reinforcement at the steps as shown in Figure 33. The reinforcement connecting the tower to the main structure would resist most of the ULS seismic loading as compared to the foundation pad. This connection acts as a support to the tower. As ULS seismic loading acts on the tower, it acts like a cantilever being supported at the connection at the steps.

While this portion of the slab is connected to main structure, it may respond with more accelerations due to its locally concentrated mass and configuration. The design action on look-out tower was therefore calculated partly on the basis on Section 8 of NZS1170.5:2004. The horizontal response factor was assumed to be 0.85 corresponding to a ductility of μ=1.25, the part risk factor is set at 1.0 and its fundamental period was considered to be less than 0.75s. The height of the attachment of the part and the height from the base of the structure to the uppermost seismic mass has been taken as 3.97m. The weight of the part was calculated to be 220kN, considering only the top part of the structure.

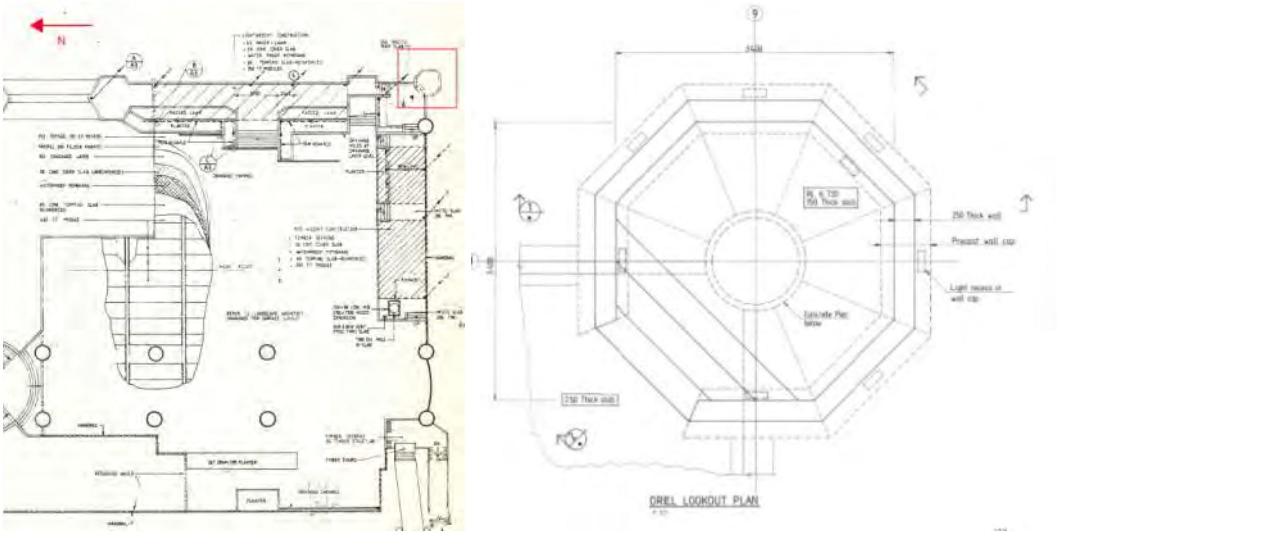


Figure 22 - Extract from as-built drawing A1 and S10 - location and plan of look-out tower

Analysis shows that connection capacity at the step is limited by its flexure capacity, which initiated yielding at 35% of ULS seismic loading. The slab section is taken at the connection at the step with the 200mm thickness and one layer of 16mm diameter bottom reinforcement at 150mm centre.

From here, the Oriel can deform inelastically and the displacement is traced in an acceleration-displacement response spectrum. It was found that the Oriel performs >100%NBS (IL3).

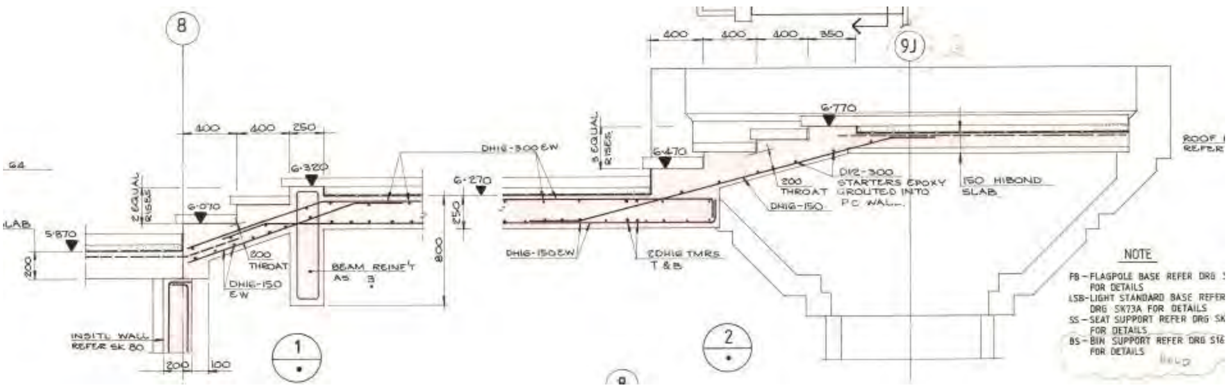


Figure 23 - Extract from as-built drawing S30 - look-out tower connection to the rest of the structure

Wall cap

The wall cap is located on top of the perimeter precast concrete wall panels. Wall cap is connected to the wall through pocket sized 100x100x60 by 16mm diameter bar at 300mm spacing or 600mm spacing. There are two types of typical wall cap which are shown in the following figure.

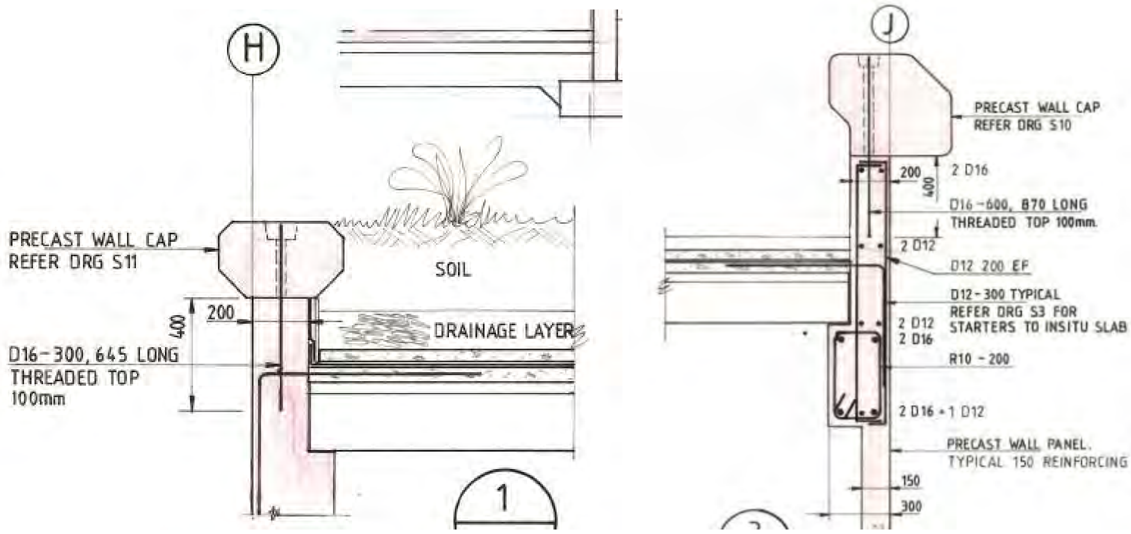


Figure 24 - Extract from as built drawing S4 - showing typical wall cap types and connection detail between the wall cap and precast concrete wall panel

The design action on look-out tower was calculated in accordance to Section 8 of NZS1170.5:2004. The horizontal response factor was assumed to be 0.85, 1.0 for part risk factor to represent a hazard to human life outside of the structure and its period is less than 0.75s. The height of the attachment of the part and the height from the base of the structure to the uppermost seismic mass has been taken as 1.88m. The weight of the part was calculated to be 3.1kN and 7.5kN for the respective wall cap type shown in the figure above.

As seismic action acts on the centre of wall cap, the weakest part would be the connection whereby the starter bar would experience tension. The stress in the starter bar has been checked against its probable yield strength. The wall cap with D16 at 300mm spacing would yield at 231% ULS seismic loading while the wall cap with D16 at 600mm spacing would yield at 97% ULS seismic loading.

The shear-friction mechanism as per Section 7.7.4.1 of NZS3101:2006 Amendment has been used to determine the shear capacity of the starter bar at the interface between wall cap and top of precast concrete wall panel. D16 at 600mm spacing bar would shear at 144% ULS seismic loading.

To conclude, the wall caps perform at 100% NBS(IL3).

C.10 RETAINING WALL

The assessment for retaining walls has been carried out outside of the agreed scope for the DSA. The retaining walls are located next to Jervois Quay. These walls are precast concrete wall panels of 250mm thickness, with heights tapering down towards the North end. The precast concrete wall panels are connected to a cast insitu footing by starter bars. JW1 and JW10 have been selected for the assessment, as they represent two various heights and the two typical reinforcement details of the precast wall panels and starter bar connection to the footing.

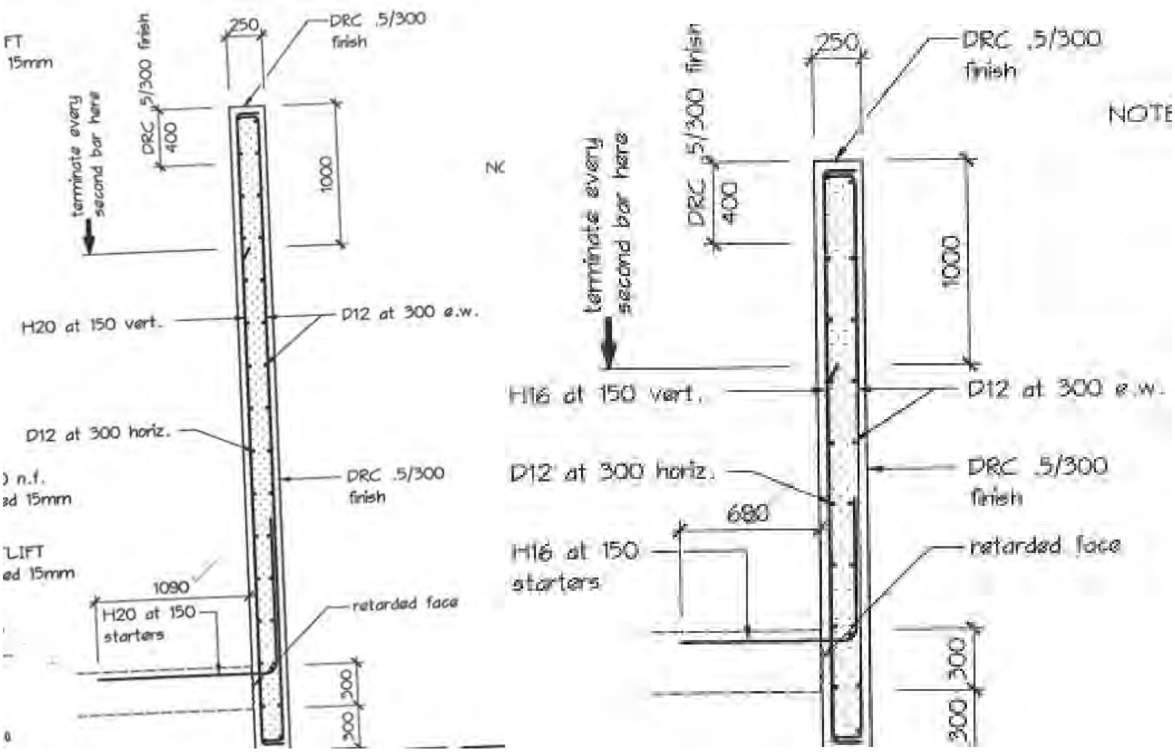


Figure 25 - Extract from as-built drawing - showing the reinforcement detail of precast concrete wall panel and start bar connection to the footing

The assessment of retaining walls was undertaken in accordance to Module 6: Earthquake resistant retaining wall design. The stiff wall approach has been adopted to obtain seismic earth pressure acting on the retaining wall as shown in Figure 13. The retaining walls were assessed for its stability, sliding capacity, bending capacity and starter connection under both gravity load and ULS seismic loading respectively. K_A, active earth pressure coefficient of 0.321, K_P, passive earth pressure coefficient of 4.24, K_{AE}, seismic earth pressure coefficient was adopted in accordance to Mononobe-Okabe method.

Stability has been checked for resisting moment against overturning moment. The self-weight of the structure and soil contribute to the resisting moment, while overturning moment is made up of the seismic earth pressure and inertia of wall elements under ULS seismic loading. Similarly, sliding action is made up of components which contribute to the overturning moment. Sliding resistance is provided by the passive pressure of soil and friction under footing. The bending strength of wall based on the reinforcement details is shown on Figure 24. The starter bar connection at the footing was determined based on Section C5.5.2.3 of the Engineering Assessment Guidelines, as a typical shear mechanism of concrete is achievable.

Analysis shows that JW1 would lose sliding resistance at 34% ULS seismic load and yield at 40% of ULS seismic load. Comparably, JW10 would slide at 69%. JW1 is the highest precast wall panel and retains the most soil, therefore taking large seismic earth pressures. It is therefore expected that the smaller retaining walls will perform better. Both these failure modes – sliding and flexural yielding – are very beneficial for the overall performance of the system as they allow for a lot of energy dissipation. They may slide and lean towards Jervois Quay for a fair distance, but failure that may collapse is not expected during seismic events with shaking less than 100%NBS (IL3).

The retaining walls are not included in the performance rating of the structure since they are technically not connected to the main building. However, since their performance may influence the occupancy of Frank Kitts Carpark, they have been provided to give context.

C.11 SUMMARY OF IDENTIFIED STRUCTURAL WEAKNESSES

Structural Weaknesses are those deficiencies identified which have an assessed seismic rating of less than 100% NBS. These are listed in Table 13.

Table 13 - Seismic rating of potential structural weaknesses

Building Element	Structural Weakness (SW)	%NBS(IL3)
Roof diaphragm	The roof diaphragm contains non-ductile mesh which cannot be relied to provide a load path for the diaphragm forces. Further commentary on the performance of the diaphragm is provided in Sections 5.3 and C.4.	15%
Connection between roof diaphragm and lateral load resisting system	The precast concrete wall panels are connected to the diaphragm at the roof level by starter bars. These starter bars are insufficient for the required shear transfer.	15%
Reclamation fill seismic performance and soil-structure interaction	Onset of liquefaction and lateral spread displacements imposing significant displacements demands on the foundations and structure. This may lead to undesirable behaviour in the superstructure and potential for collapse.	20-30%
Precast concrete wall panels	Yielding of the starter bars of the precast concrete wall panels for in-plane flexural demands. The capacity is limited by the tensile strain of the starter bars and the inelastic rotation capacity of the precast concrete wall panels.	40%
Double-tee units	The double-tee units forming the roof slab are web-supported. These, when subjected to movement may exceed the capacity of the supporting concrete members in bearing. Note: Visual observations revealed several double-tee units where the seating support had already spalled. The remaining seating of these units may be insufficient during large seismic events and would consequently rate lower than 100%NBS (IL3).	100%