REPORT

Tonkin+Taylor

Frank Kitts Park Redevelopment

Geotechnical Report

Prepared for
Wellington City Council

Prepared by
Tonkin & Taylor Ltd

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Executive summary

Design of the Frank Kitts Park redevelopment is to consider earthquake risks to life safety and of damage. Various light and heavy structures, and landscaping is proposed. The Fale building is considered to be Importance Level 3 by the Fale Malae Trust. The remainder of the park and other structures are specified as Importance Level 2 by WCC. Design of the Frank Kitts Park redevelopment will need to consider the following:

- **Resilience, protection of capital investment and continuity of use of the park**
  - There is a risk of liquefaction and lateral spread at the park which could result in considerable damage to the proposed redevelopment. This report provides indications of the likelihood and extent of this damage and proposes some mitigation measures which could be applied to reduce this risk. WCC should consider the acceptability of the residual risk.
  - The potential magnitude of lateral spread reduces with distance from the reclamation edge. To aid the description of lateral spread risk and mitigation, the site has been divided into two zones (refer Section 4.4). Zone B is within 20m of the reclamation edge and at greater risk of lateral spread. Zone A is more than 20m from the reclamation edge.
  - As far as is practical, structures are to be avoided in Zone B or are designed for the effects of lateral spread.
  - The design intent is to improve the resilience within economic constraints and includes:
    - Shallow foundations for light structures in Zone A and landscaping finishes within Zone A and B should be developed to promote ability to tolerate ground deformations and to promote repairability.
    - Foundations for heavy structures and buildings should be designed allowing for the effects of liquefaction and lateral spread. The foundation design should be developed to be independent of the magnitude of ground movement to provide resilience for larger displacements and settlements.
- **Building Code requirements –** Structures which are defined as "Buildings" under the Building Act 2004 will be required to meet Building Code requirements. It is to be agreed with the territorial authority which structures within the development are to be defined as "Buildings". Building Code requirements include:
  - Serviceability limit state (SLS) requirements: In a 25-year return period earthquake the structures shall not experience damage that would prevent the structures from being used as originally intended without repair. For Frank Kitts Park liquefaction, lateral spread and associated damage is not expected in a 25-year return period earthquake.
  - Ultimate Limit State (ULS) requirements: ULS requirements are focussed on life safety. In a ULS earthquake (500-year return period for IL2 and 1000-year return period for IL3 structures) the structure is to have a low probability of instability/collapse and not present a life safety hazard. Structures should have some resilience against collapse in the event of shaking more intense than the ULS case. Liquefaction and substantial lateral spread can be expected at Frank Kitts Park in a ULS event which will need to be allowed for in design. Design allowances for the ULS case are included in this report.
  - Structures not defined as "Buildings" under the Building Act 2004 will not be required to meet Building code requirements. The design of these structures should consider life safety risks.

Table 1 summarises the main conclusions of this report.

Table 1: Conclusions

Topic	Comment	This report reference
Typical Soil profile	**0 to 10 m depth**: Variable end-tipped reclamation fill; soft silt, loose sand and gravel. **10 to 12 m depth**: Marine deposit; very soft to soft clayey silt interbedded with loose silty sand and shell fragments. **12 to 25 m depth**: Alluvium; Dense to very dense silty sand and gravel with occasional medium dense sand and lenses of stiff silt/clay. **Below 25 m depth:** Rock	Section 2.2
Potential for liquefaction	Earthquake shaking slightly more intense than that felt at the park as a consequence of the 2016 Kaikoura earthquake could be expected to trigger liquefaction of the fill's loose sand and gravel which makes up approximately 35% of the total fill thickness. The silt (65% of fill thickness) is assessed to be too cohesive to liquefy. Earthquake shaking to trigger liquefaction is assessed to be 0.2g M7.1, 55% probability of occurrence in the next 25 years.	Section 4.1
Consequences of liquefaction	• Reduced support to shallow foundations. • Lateral spread • Local sand boils • Differential settlement • Lateral spread	Section 4.3
Lateral spread potential	**Applicability:** The likelihood and magnitude of lateral spread is greatest adjoining the reclamation edge and reduces with distance from the reclamation edge. The potential magnitude of lateral spread for any intensity of earthquake shaking cannot be reliably predicted. In this report lateral spread potential is indicated for the two zones highlighted on Figure 4.1. The lateral spread potential can be expected to be greatest at the seaward side of each zone and least at the landward side. Magnitudes of lateral spread for each zone are presented in this report to provide an indication of the risk of lateral spread and not to represent absolute numbers for design. There is considerable uncertainty in these predictions such that actual lateral spread could be 1/3 to 3 times the indicated lower and higher displacement magnitudes respectively. **Trigger for lateral spread damage:** The assessed intensity and likelihood of earthquake shaking which could cause damaging lateral spread (> 50 mm) is assessed to be: **Zone A:** 0.35g M7.1, 25% probability of occurrence in the next 25 years. **Zone B:** 0.25g M7.1, 40% probability of occurrence in the next 25 years. **Magnitude of lateral spread:** Lateral spread as a consequence of a ULS(IL2) event (0.68g M7.7, 10% probability of occurrence in the next 25 years) could be: **Zone A:** 300 mm to 600 mm **Zone B:** metres	Section 4.4
Settlement potential	**Applicability:** In addition to lateral spread, earthquake shaking could result in settlement. Uncertainties in the prediction of settlement are like those for predicting lateral spread (refer lateral spread applicability above). **Trigger and magnitude:** Earthquake shaking of 0.2g M7.1, 55% probability of occurrence in the next 25-years, could result in 100 mm of total settlement and 50 mm differential settlement in either Zone A or Zone B. Additional settlement could be expected in conjunction with the lateral spread described above. In the case of Zone A this additional total and differential settlement could be 20% of the described lateral spread, and for Zone B 50%. I.e., for Zone A in a ULS(IL2) event total settlement is expected to be 100mm plus 20% of 300 mm to 600 mm = approx. 150 mm to 250 mm. Actual settlements could be 1/3 to 3x this indicated lower and higher settlement respectively.	Section 4.5
Design considerations for hard and soft landscaping	Table 5.2 summarises design considerations for soft and hard landscaping finishes associated with the redevelopment of Frank Kitts Park. These finishes should be designed to promote tolerance to ground displacements and repairability.	Section 5.2.1
Design considerations for light structures	Table 5.3 summarises design considerations for foundations for light structures including retaining walls, decorative walls, pergolas associated with the redevelopment of Frank Kitts Park. Table 5.4 provides a discussion of the specific foundation types being considered in the Frank Kitts Park redevelopment.	Sections 5.2.2 and 5.2.3
Design considerations for buildings and heavy structures including the Fale.	Buildings and heavy structures will require specific foundation design. Table 5.5 provides a discussion of the form of foundations that are being considered for the Fale.	Section 5.3.1

1 Introduction

Tonkin & Taylor Ltd (T+T) has been engaged by Wellington City Council (WCC) to provide geotechnical input to support the resource consent application for the redevelopment of Frank Kitts Park, Wellington. This report has been prepared in accordance with our Letter of Engagement dated 15 September 2023[1].

This report presents the following:

- Summary of ground and groundwater conditions at the site;
- Seismic shaking hazard potential for the site;
- Potential for liquefaction at the site and associated geotechnical consequences;
- A summary of the considerations for the redevelopment of the site; and
- Comment on the engineering approach to address resilience for the site.

Reports previously issued by T+T for the site comprise:

- Geotechnical Factual Report[2] issued 11 February 2022.
- Geotechnical seismic stability assessment report[3] issued 20 July 2022.

Wraight + Associates Ltd (WAAL) is the landscape architect. Dunning Thornton Consultants (DTC) is the structural engineer.

This report has considered the concept layout in WAAL Drawings[4] dated November 2023. This comprises a combination of light and heavy structures.

1.1 Site description

Frank Kitts Park is located within an area of reclaimed land that was constructed in the 1970s along the Wellington waterfront, between Jervois Quay (West) and the Lambton Harbour (East). The site is bound by Queen’s Wharf to the north and the Whairepo lagoon in the south. The site is generally flat at RL +2.4m (WVD1953).

The land-use of the Frank Kitts Park at the time of reporting comprises the following (refer Figure 1.1):

a Harbour promenade/Ara moana at the top and Lagoon/Whairepo Promenade to the right, and a rock armoured batter slope forming the seaward edge of the reclamation fill.

b The children’s playground, currently under reconstruction (excluded from this scope).

c Grassed amphitheatre.

d Underground carpark. The underground carpark also hosts boat sheds, public toilets and several small shops around the interface with the Harbour and Lagoon Promenades.

[1] Tonkin & Taylor Ltd (September 2023). Letter of Engagement (Rev 1). Frank Kitts Park Redevelopment. Engineering services to support resource consent application. T+T Ref: 1018875.4000.
[2] Tonkin & Taylor Ltd (February 2022). Wellington Waterfront Geotechnical Factual Report. T+T Ref: 1018875.0000.
[3] Tonkin & Taylor Ltd (July 2022). Frank Kitts Park – Geotechnical seismic stability assessment report. T+T Ref: 1018875.1000.3F.
[4] Wraight + Associated Ltd (November 2023). *Frank Kitts Park – Preliminary Design*. Drawing number L101.

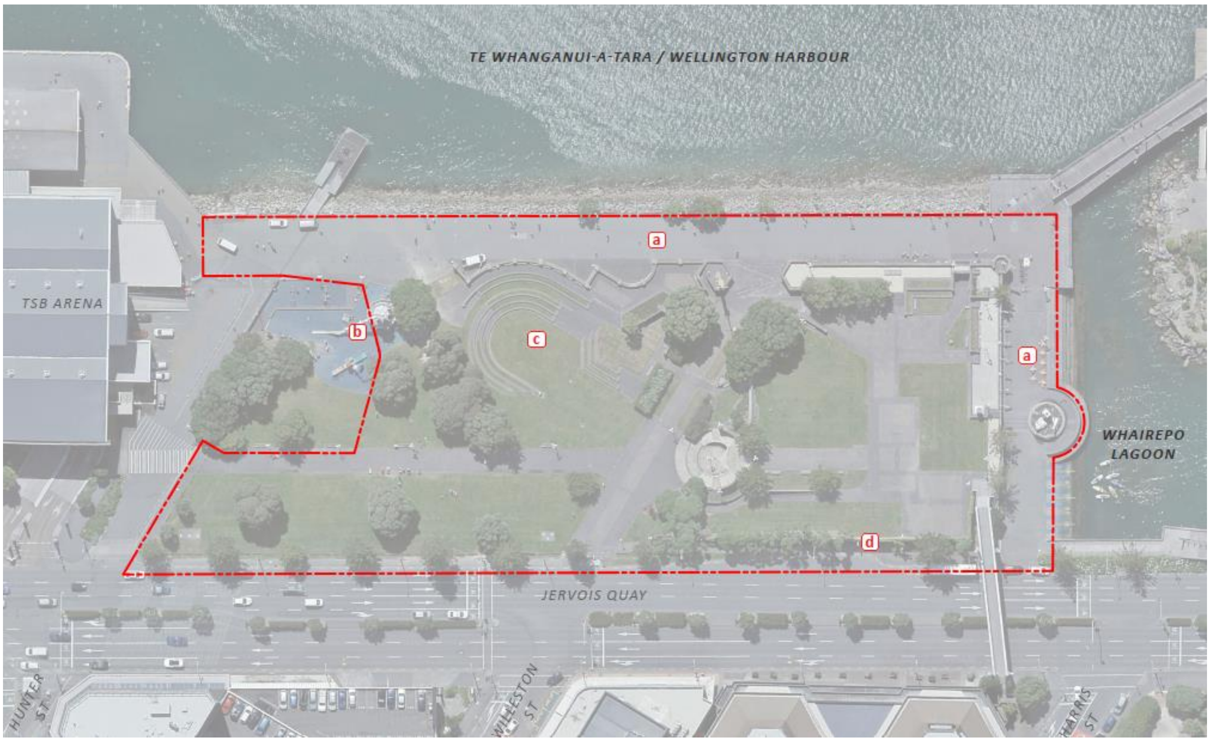

Figure 1.1: Frank Kitts Park and existing carpark building

1.2 Proposed redevelopment

The proposed redevelopment of Frank Kitts Park (refer Figure A1, Appendix A) includes the following:

- Demolition of the existing Frank Kitts carpark building;
- Harbour Promenade;
- Te Papa Whenua (planted area);
- Harbour Lawn;
- Garden of Beneficence;
- Fale Building; and
- Whairepo Lawn and Malae.

Structures proposed as part of the development include the following:

- Garden of Beneficence
  - Free standing concrete walls up to 2.8 m high
  - Concrete retaining walls up to 1.1m retained height
  - Lightweight pavilion structures
  - Pai Lau structure
- Fale Pasifika building (includes a basement) (being developed by the Fale Malae Trust)

Other parts of the redevelopment include hard and soft landscaping finishes (e.g. promenade walkways, garden beds and lawns), Raukura (feather) sculptures, and light poles.

Construction of the updated children's playground has just been completed and is not a relevant component for this Park redevelopment proposal.

1.3 Design considerations

Design of the Frank Kitts Park redevelopment will need to consider the following:

- Building Code requirements
  It is to be agreed with the territorial authority which of the structures are "Buildings" as defined in the Building Act 2004. "Buildings" will be required to meet Building Code requirements. WCC has advised that the Frank Kitts Park redevelopment shall be considered Importance Level 2 (IL2). The Fale building should be considered Importance Level 3 (IL3).
- Building Code requirements include:
  - Serviceability limit state (SLS) requirements:
    In a 25-year return period earthquake the structures shall not experience damage that would prevent the structures from being used as originally intended without repair. For Frank Kitts Park liquefaction, lateral spread and associated damage is not expected in a 25-year return period earthquake.
  - Ultimate Limit State (ULS) requirements:
    ULS requirements are focused on life safety. A ULS event for an IL2 structure is a 500-year event. A ULS event for an IL3 structure is a 1000-year event. In a ULS earthquake the structure is to have a low probability of instability/collapse and not present a life safety hazard. Structures should have some resilience against collapse in the event of shaking more intense than the ULS case. Liquefaction and substantial lateral spread can be expected at Frank Kitts Park in a 500-year and 1000-year event which will need to be allowed for in design. Design allowances for the ULS case are included in this report.
  - We understand from DTC that free standing light poles and sculptural artworks (e.g. Raukura sculptures) are not considered "Buildings" under the Building Act and are therefore not subject to the regulatory issues outlined in the Building Act.
- Protection of capital investment and continuity of use of the park
  Separate to Building Code requirements design of all aspects of the redevelopment should consider protection of capital investment and continuity of use of the park. There is a risk of liquefaction and lateral spread at the park which could result in considerable damage to the proposed redevelopment. This report provides indications of the likelihood and extent of this damage and proposes some mitigation measures which could be applied to reduce this risk. WCC should consider the acceptability of the residual risk.

2 Ground and groundwater conditions

2.1 Previous geotechnical investigations

The Geotechnical Factual Report[2] prepared by T+T presents:

- Site description.
- Available records from previous geotechnical site investigations.
- Records of recent geotechnical site investigations.
- Commentary on geology and reclamation history.

Refer to Figures A2 and A3 in Appendix A for the site plans presenting:

- Location of investigations undertaken in 1987, 2020 and 2021.
- Boundaries and year of construction of the stages of reclamation.

Refer to Figure A4 in Appendix A for the typical cross-section through the harbour reclamation edges.

2.2 Ground model

The inferred ground model at the site is described in Table 2.1 Refer to Figure A3 in Appendix A for the soil profile cross section.

Table 2.1: Summary of inferred soil profile

Layer	Geological Unit	Typical Description	Depth to Top of Layer (m)	Layer Thickness (m) [1]	Raw SPT N (blows/ 300mm)	CPT qc (MPa)
1a	Upper Reclamation Fill (1970s)	Loose to medium dense GRAVEL with some cobbles. Minor sand and silt.	0	2.0– 2.5	0 – 50+ Typically, 15 – 20	2 – 10
1b	Lower Reclamation Fill (1970s)	Variable end tipped fill. Varying from soft sandy SILT and gravelly SILT to loose silty sandy GRAVEL.	2.0-2.5	4.5 – 10.0	0 – 8 Typically, 4 – 5	2 – 30 Typically, 2 – 6
2	Marine Deposits	Very soft to soft sandy and clayey SILT interbedded with loose silty SAND and shell fragments.	6.5 – 8.0	0 – 4.0	0 – 27 Typically, 3 – 7	0.5 – 5 Typically, 1
3	Alluvium	Dense to very dense silty SAND and GRAVEL with occasional medium dense SAND and lenses of stiff SILT/CLAY.	10.3 – 12.5	3 – 6.5 (proven)	7 – 50+ Typically, silt/clay 15 sand/gravel 30-50+	5 – 30
4a	Completely Weathered Rock / Residual Soil	Silty SAND, some gravel.	16.5-25.0	4.5	50+	N/A
4b	Basement Rock	Highly weathered SANDSTONE (Greywacke)	18.0 [2] – 30.0m [3]	NA	50+	N/A

Notes:

1. Layer thickness has been taken from the typical ground level within Frank Kitts Park (RL +2.4m WVD1953). BH103 was performed at RL +4.08 (WVD1953).
2. The depth to rock was encountered at 18 to 21m depth along the western edge of the site in two boreholes.
3. Rockhead inferred to be up to 30m deep for the rest of the site based on Kaiser et al., 2019.

2.3 Groundwater

Groundwater levels were monitored in two boreholes for a period of one month. In BH102 the groundwater was monitored to vary with the tide between RL 0.0 m and RL +0.7 m. In BH103 groundwater was monitored to vary with the tide between RL -0.3 m to RL +1.0 m.

Over the next 50 years a sea level rise of 500 mm should be considered in accordance with the Ministry for the Environment guidance[5] for sea level rise.

Considering the monitored groundwater results and an allowance for sea level rise we propose a design groundwater level of RL +0.85 m for liquefaction assessment.

3 Seismic shaking hazard

3.1 2022 National Seismic Hazard Model (NSHM) update

In October 2022, GNS Science released the revised National Seismic Hazard Model (NSHM)[6]. This represents the latest scientific knowledge of earthquake hazard in New Zealand and is an important factor for understanding and managing earthquake risk in the built environment.

While the NSHM will inform future design standards, it does not provide information that can be directly applied in design applications. Consequently, the current minimum compliance pathway within the Building Code has not changed[7]. However, important updates to Building Code compliance documents that will be informed by the NSHM are expected to be released between 2024 and 2025.

We have undertaken an initial appraisal of the implications of the 2022 NSHM for geotechnical design. It is uncertain how the updated NSHM will be reflected in future design standards, however it is possible that the code minimum seismic design loadings will increase in some situations.

Seismic hazard models carry an inherent amount of uncertainty, but more important is the uncertainty in what shaking a particular site or building will be subject to during its actual life. This depends on which specific earthquakes actually occur over that time. Therefore, designers and building owners are strongly encouraged to focus on resilient design practices, rather than the specific code minimum demand[8].

Liquefaction triggering and associated consequences are non-linear. Our liquefaction analysis has considered a range of seismic loadings, including values between the current code minimum limit states of SLS and ULS (for further explanation on these, refer to Section 3.4), as well as beyond ULS. This allows us to understand the impact of the uncertainty in seismic loadings on the geotechnical performance of the site, in particular whether there are any step-changes which could be critical. The consequences of this are discussed further in Section 4.3.

3.2 Seismic site subsoil class

The seismic subsoil class in accordance with NZS 1170.5:2004 Section 3.1.3 for the site is considered to be 'Class C – Shallow Soil Sites'.

This assessment is based on published geotechnical information outlined in Section 2.2 and the Geotechnical Factual Report[2]. Greywacke rock is inferred to be up to 30 m deep.

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[5] Ministry for the Environment (2022). Interim guidance on the use of new sea-level rise projections.

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

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

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

3.3 Historic earthquakes

Table 3.1 summarises the assessment of intensity of earthquake shaking which was likely to have been felt at the site in recent earthquakes. The strong ground motion sensor 'FKPS' is located within the site and is likely to represent the felt shaking. The strong ground motion sensor 'CPLB' is located adjoining CentrePort and has been included for comparison.

Table 3.1: Strong motion recording stations data

Earthquake	Magnitude (M)	Peak Ground Acceleration (PGA) at Frank Kitts Park (FKPS)	Peak Ground Acceleration (PGA) at CentrePort (CPLB)
Seddon 2013	6.5	0.11	0.21
Lake Grassmere 2013	6.6	0.10	0.15
Kaikoura 2016	7.8	0.15	0.24

There was some evidence of minor lateral displacement of the reclamation edge at Frank Kitts Park as a result of the 2016 Kaikoura earthquake. A liquefaction sand boil was observed in the lagoon south of Frank Kitts Park.

Considerable ground damage was experienced at CentrePort as a consequence of these earthquakes.

3.4 Ground shaking hazard

The seismic hazard in terms of peak ground acceleration (PGA) and magnitude for the site has been assessed based on NZGS/MBIE Module 1 2021[9]. Table 3.2 presents unweighted PGAs and a corresponding earthquake magnitude for the ultimate limit state (ULS), serviceability limit state (SLS) earthquakes and various earthquakes.

Table 3.2 includes the likelihood of occurrence of the ULS and SLS earthquake as indicated by the NSHM. The likelihood is provided to inform an understanding of seismic risk.

Table 3.2: Design Level Shaking

	Design Case	Return Period	M	PGA (g)	Likelihood in next 25 years based on NSHM 2022
Module 1 (2021)	SLS	25	6.5	0.13	75%
	-	50	6.8	0.19	55%
	-	100	7.1	0.28	35%
	-	250	7.5	0.47	15%
	ULS (IL2)	500	7.7	0.68	10%
	ULS (IL3)	1000	7.7	0.91	< 5%

Note:

Structure design life	50 years – as advised by DTC.
VS30	Approximately 250m/s to 275 m/s based on inferred soil profile (refer section 3.2) and published Vs30 map by Semmens et al. 2010[10]. Relevant to NSHM only.

[9] MBIE/NZGS. Earthquake Geotechnical Engineering Practice, Module 1 (Version 1, 2021): Overview of the Guidelines.
[10] Semmens, S.; Perrin, N.D.; Dellow, G. 2010. It's Our Fault – Geological and Geotechnical Characterisation of the Wellington Central Business District, GNS Science Consultancy Report 2010/176. 52p.

In accordance with MBIE guidance the geotechnical design of new structures shall be in accordance with Module 1 (2021)[9] .

WCC has advised that the Frank Kitts Park redevelopment shall be considered Importance Level 2 (IL2). The Fale building has been specified as Importance Level 3 (IL3) by the Fale Malae Trust.

4 Liquefaction Assessment

4.1 Liquefaction hazard

The site is located within the liquefaction hazard overlay of the Proposed Wellington District Plan[11]. Responding to that hazard has been a focus of the investigations, assessment and concept design as outlined in this report.

As notified, the proposed rules relating to the liquefaction hazard overlay permit the following:

- 'Less hazard sensitive activities' such as park furniture and facilities; and
- 'Hazard sensitive activities' which includes Community Facilities (e.g. Fale Building).

4.2 Liquefaction potential

Liquefaction only occurs in some soils. Liquefaction susceptible soils are typically saturated, non-cohesive and loose or medium dense. Soils which are susceptible to liquefaction require a certain level of earthquake shaking (trigger) to cause them to liquefy. Denser soils require more intense and/or longer duration of shaking (higher trigger) than less dense soil.

The liquefaction susceptibility and trigger for each soil layer has been assessed by the method proposed by Idriss and Boulanger (2014)[12]. The conclusions are summarised in Table 4.1 for each soil layer. The details of the liquefaction assessment are presented in Appendix B of the Assessment Report[3].

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

Table 4.1: Liquefaction assessment summary

Layer	Geological Unit	Liquefaction susceptibility and triggering
1	Reclamation Fill (1970s)	Above groundwater level: Not expected to liquefy Below groundwater level: Typically, 65% of the fill is assessed not to be susceptible to liquefaction as the behaviour of the material has been assessed to be clay-like. 35% of the fill comprises low plasticity silt, sand and gravel and is assessed to be susceptible to liquefaction. Liquefaction of this susceptible soil could be expected to be triggered by 0.2g M7.1 earthquake shaking.
2	Marine Deposits	The clayey silt has been assessed to be clay-like and is not considered to be susceptible to liquefaction. Liquefaction of localised pockets /lenses of non to low plasticity silt and loose to medium dense sand could be expected to be triggered by 0.2g, M7.1 earthquake shaking.
3	Alluvium	Dense to very dense sand and gravel is not expected to be susceptible to liquefaction. Liquefaction of localised pockets/lenses of medium dense sand, gravel and low plasticity silt could be expected to be triggered by 0.25g, M7.1 earthquake shaking.
4a	Completely Weathered Rock / Residual Soil	Not expected to liquefy.
4b	Basement Rock	Not expected to liquefy.

4.3 Liquefaction consequences

Table 4.2 outlines the consequences of liquefaction at the site.

Table 4.2: Liquefaction Consequences

ID	Consequence	Description
1	Lateral spread	• Lateral spread could be expected at the site because of gravity and seismic inertia loads mobilising ground toward a free edge (harbour or lagoon). • Limited in magnitude relative to a site with a continuous thick layer of liquefied soils because the non-liquefiable silt within the fill maintains strength. • Refer Section 4.4 for further details.
2	Reduced support to shallow foundations.	• Liquefaction beneath a shallow foundation will result in loss or substantial reduction in support to that foundation and associated subsidence of that foundations. The magnitude of subsidence will depend on the ability of the structure to re-distribute loads. Assessment of the magnitude of such subsidence would require consideration of soil-structure interaction. • Refer Section 5.2.2 for further details.

ID	Consequence	Description
3	Free field settlement	• In the order of 50 to 100 mm post liquefaction free-field settlement could be expected at the site. • Refer Section 4.5 for further details.
4	Sand boils	• Possible due to thin non-liquefiable crust. • Refer Section 4.5 for further details.
5	Differential settlement	• Differential settlements may be induced by the following mechanisms o Vertical component of lateral spread (ID1) o Post liquefaction free-field settlement (ID3) o Sand boils (ID4) • Refer Section 4.5 for further details.

4.4 Lateral spread potential

An assessment of the magnitude of the lateral spread potential for Frank Kitts Park was undertaken and is presented in the Seismic Stability Assessment Report[3].

The assessment comprised the following:

- Assessing the strength of the soil along which lateral spread could occur allowing for liquefaction effects. Refer Section 8.1 of the Assessment Report[3].
- Applying this strength in a Newmark sliding block (NSB) analysis to assess displacement potential. Refer Section 8.2 of the Assessment Report[3].

The conclusions of the assessment of lateral spread potential are presented in Table 4.3 for the two zones highlighted in Figure 4.1. The two zones have been assessed considering the potential for larger displacements at distances closer to the reclamation edge.

The lateral spread potential can be expected to be greatest at the seaward side of each zone and least at the landward side. Magnitudes of lateral spread for each zone are presented in Table 4.3 to provide an indication of the risk of lateral spread, and not to represent absolute numbers for design. There is considerable uncertainty in these predictions such that actual lateral spread could be 1/3 to 3 times the indicated lower and higher displacement magnitudes respectively.

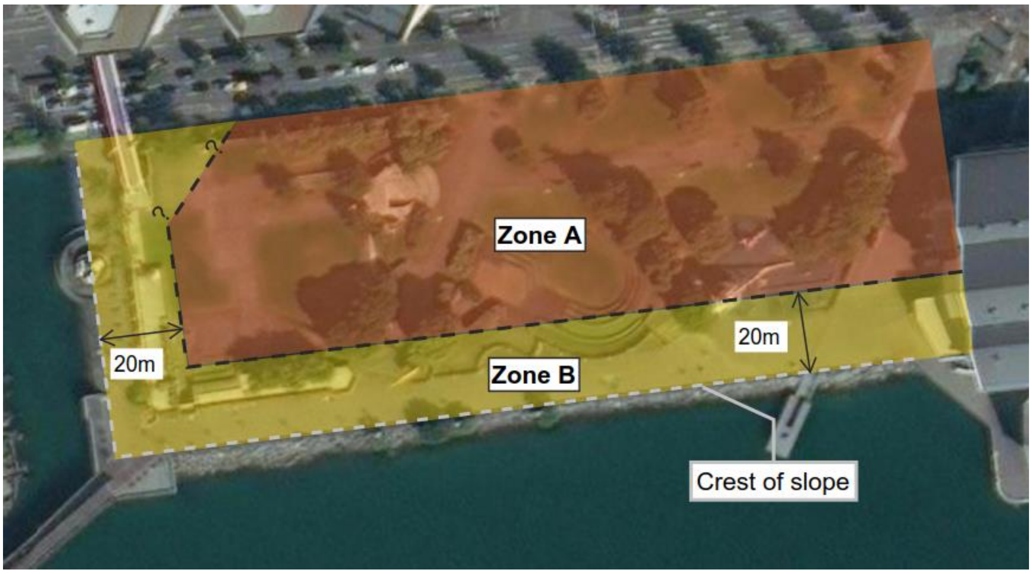

Figure 4.1: Lateral Spread Risk Zones

Table 4.3: Lateral spread potential

Ground shaking hazard					
Return period (Module 1 2021)	50 years	100 years	250 years	500 Years (ULS-IL2)	1000 Years (ULS-IL3)
Magnitude and PGA (Module 1 2021)	M6.8, 0.19g	M7.1, 0.28g	M7.5, 0.47g	M7.7, 0.68g	M7.7, 0.91g
Probability of occurrence in next 25 years (1)	55%	35%	15%	10%	5%
Zone	**Displacement (2)**				
Zone A	< 25 mm	< 50 mm	100 to 250 mm	0.3 to 0.6 m	0.5 to 1.5 m
Zone B	< 50 mm	50 to 300 mm	0.5 to 1.5 m	1 to 3m	Metres

Note:
1. Based on NSHM 2022.
2. Displacements provided are indications of the risk of lateral spread rather than representing absolute magnitudes. There is considerable uncertainty in these predictions such that actual lateral spread for a specific ground shaking hazard could be 1/3 to 3 times the indicated lower and higher displacement magnitudes respectively.

4.5 Potential for settlement and sand boils

Liquefaction could induce settlement of the site by the following mechanisms:

- Vertical component of lateral spread.
- Potential for sand boils.
- Post liquefaction consolidation.

Earthquake shaking of 0.2g M7.1 (55% probability of occurrence in the next 25-years) could result in 100 mm of total settlement and 50mm differential settlement over 10m in either Zone A or Zone B. Additional settlement could be expected in conjunction with the lateral spread reported in Table 4.3. In the case of Zone A this additional total and differential settlement could be 20% of the lateral spread reported in Table 4.3, and for Zone B 50%. There is considerable uncertainty in the prediction such that actual settlement for a specific ground shaking hazard could be 1/3 to 3 times the indicated lower and higher magnitudes respectively. I.e., For Zone A in a 500-year ULS(IL2) event total settlement is expected to be 100 mm plus 20% of 300 mm to 600 mm = 150 mm to 250 mm. Actual settlements could be 50 mm to 750 mm.

5 Geotechnical considerations for redevelopment

5.1 Design parameters

Concept design and subsequent design development of landscaping and structures allowing for ground deformation should be undertaken in consultation with the geotechnical engineer.

Table 4.3 provides an indication of the potential for lateral spread for consideration. Section 4.5 provides an indication of total settlement for consideration. Because of the uncertainty in predicting lateral spread it is proposed that designers apply a risk-based approach as outlined below.

<u>ULS design</u>

In accordance with the building code, structures should have a low risk of collapse at ULS. There should be some resilience at levels of shaking greater than ULS such that collapse would not be expected. Table 5.1 proposes ground deformations to be considered in ULS design.

Table 5.1: ULS design ground deformations

Zone	Design case	Total lateral spread (3)	Total settlement (4)	Total lateral spread (3)	Total settlement (4)
		IL2 structures (i.e. Frank Kitts Park redevelopment)		IL3 structures (i.e. Fale building)	
A	ULS Low probability of collapse	0.6 m	0.25 m	1.5 m	0.4 m
	ULS Collapse not expected (resilience check)	1.8 m	0.75 m	Metres	1.2 m
B	ULS Low probability of collapse	Metres	Metres	Metres	Metres
	ULS Collapse not expected (resilience check)	Metres	Metres	Metres	Metres

Notes:

1 Designs for Building Code requirements are to be developed to have "low probability of collapse" and "collapse not expected" allowing for the respective displacements and settlements reported.

2 Total lateral spread and total settlements are reported. Differentials equal to ½ of these totals are to be allowed for across the width of any foundation. For wide foundations differentials equal to ½ of these totals are to be allowed over any 2 m distance, i.e. the bending effect of differential is to be considered.

3 Total lateral spreads are taken from Table 4.3 i.e. top end of range reported for "low probability of collapse" and 3x top end of range for "collapse not expected". The "collapse not expected" value also provides resilience for >1000year top end of range predicted.

4 Refer Section 4.5 for basis of settlement numbers i.e. 100 mm volumetric strain plus 20% to 50% of lateral spread for "low probability of collapse" and 3x this magnitude for "collapse not expected".

<u>SLS design</u>

Earthquake induced lateral spread and settlement is not expected.

<u>Between SLS and ULS</u>

Designs should be developed to mitigate damage due to lateral spread and settlement as far as is practical. Designers should assess the risk and magnitude of damage at various levels of shaking, communicate this to the client and obtain acceptance of this risk before finalising the design. Table 4.3 and Section 4.5 provides ground deformations at various levels of shaking to inform this risk assessment.

5.2 Frank Kits Park redevelopment

5.2.1 Landscaping

The Frank Kitts Park redevelopment includes hard and soft landscaping finishes, and these are understood to comprise paved surfaces, garden beds, lawn areas and feature rocks. These details are subject to change.

Table 5.2 provides general design guidance in response to lateral spread and settlement potential. Designers will need to develop specific responses for specific landscape proposed in consultation with the geotechnical engineer.

Table 5.2: Geotechnical considerations for landscaping

Landscape finishes	General design considerations	Trigger for damage (Probability in next 25 years) (1)	Post-earthquake repair
Zone A			
Soft landscaping; gardens, lawns, natural boulders	Standard construction	15%	Local repairs of gardens and natural boulders. Possibly relaying of lawns.
Flexible paving; precast concrete pavers, asphalt	Standard construction. As far as practical construct paved surfaces at steeper gradients than standard construction to mitigate the risk of ponding in the event of settlement.	15 to 35%	Local repairs, or relaying following larger events.
Rigid paving; insitu concrete	Individual concrete slabs to be robustly tied together by reinforcing. Joints between slabs to be detailed to allow movement and repair.	15 to 35%	Filling of gaps at joints if this is an acceptable repair, or reconstruction following larger events.
Zone B			
Soft landscaping; gardens, lawns, natural boulders	Standard construction	35%	Reconstruction could be required.
Flexible paving; precast concrete pavers, asphalt	As for zone A. Consider risk of damage and if this is acceptable.	35 to 55%	Reconstruction could be required.
Rigid paving; in-situ concrete	As for zone A. Consider risk of damage and if this is acceptable.	35 to 55%	Reconstruction could be required.

Note:

1. Based on 2022 NSHM.

5.2.2 Light structures

Several light structures are proposed as part of the redevelopment. Table 5.3 provides general design guidance in response to lateral spread and settlement potential. Designers will need to develop specific responses for specific structures proposed in consultation with the geotechnical engineer.

Table 5.3: Geotechnical considerations for light structures

Structure name	**General design considerations**	**Trigger for damage (Probability in next 25 years) [1]**	**Post-earthquake repair**
Zone A			
Shallow foundations for light structures	• Use low structure height to foundation width ratios or bracing to reduce risk of tilting. • Robustly reinforce strip and pad foundations. Tie foundations together where this is practical. • Alternatively provide isolation joints in strip footings at locations where differential displacement can be tolerated by structure. • Because lateral spread and settlement cannot be reliably predicted provide resilience for larger displacements/settlements. • Foundations to be detailed and sized to promote control of damage and to promote repairability in design events (Refer Sections 4 and 5.1) • Foundations to be sized to promote stability (Refer Sections 4 and 5.1). • Foundation design to allow for liquefied soil strengths. • For light structures defined as "buildings" by the Building Act, it is expected that shallow foundations for these structures can be designed to meet the requirements of the Building Code in Zone A (Refer Table 5.1).	15 to 35%	Repair, or reconstruction following larger events.
Zone B			
Shallow foundations for light structures	• As far as practical, structures should be avoided in Zone B. • If structures are to be constructed on shallow foundations in Zone B design considerations should be as for Zone A above noting that the risk of damage and instability will be greater in Zone B. WCC will need to consider the acceptability of the associated damage and life safety risk. • Shallow foundations for structures (defined as "Buildings" by the Building Act 2004) are unlikely to be suitable in Zone B because of inability to meet Building Code ULS design requirements (refer Table 5.1), and are not recommended. • Substantial piles or ground improvement could be considered but the solutions are likely to be cost prohibitive for light structures.	35 to 55%	Reconstruction could be required.

Note:

1. Based on 2022 NSHM.

5.2.3 Specific foundations proposed for Frank Kitts Park

Specific structures and foundations proposed for Frank Kitts Park are discussed in Table 5.4. The foundations have been selected to meet the general design considerations discussed above.

Table 5.4: Specific structures and proposed foundations

Ref. [1]	Structure	Location	Comment
1	Wahine mast memorial	Zone A	• Shallow raft foundation proposed. • Location of structure is near the Zone A/B boundary. To reduce the risk associated with lateral spread, it is proposed to either extend the foundation back away from the edge or tie the foundation back to other foundations further from the reclamation edge.
2	Pai Lau	Zone A	
3	Free standing walls / retaining walls	Zone A	• Shallow raft foundation proposed.
4	Pavilion structure	Zone A	
5	Raukura (feather) sculptures	Zone B	• Shallow raft foundation proposed. • Those that are in Zone B (particularly those on the reclamation edge) could be at risk of toppling. This toppling is likely to be in the seaward direction. The life safety risk associated with these structures will require specific consideration by WCC. • DTC advise that these structures are not subject to building code requirements.
6	Light poles	Zone B	
-	Fale Building	Zone A/B	• Large diameter bored piles or CFA cellular structure proposed (refer Table 5.5).

Notes:

1. Refer Figure A5 in Appendix A for location.

5.3 Fale building

5.3.1 Building foundation options

The foundation options under consideration for the Fale building are discussed in Table 5.5. Either foundation system could support the building to meet building code requirements. The ground beyond the building will remain at risk of liquefaction and lateral spread and consequently a risk of damage to access, services and other facilities connected to the building. This risk to services and access exists for other buildings along Wellington’s reclaimed waterfront to varying degrees. For the Fale building the following measures could be considered during design development to reduce this risk:

- Provide flexible joints for services and access connections to the building.
- Minimise the number of locations where services connect to the building and form these in a manner to facilitate repair.
- Provide either articulated joints or isolation joints between hard landscaping and the building.
- Provide building egress points on the landward side of the building.

Table 5.5: Fale foundation options

ID	Foundation Type	Comment
1	Stiff strong large diameter piles to resist load of soils laterally spreading past them.	Piles of 1.5m diameter. This foundation system can be developed to meet Building Code requirements. Some displacement and damage of the piles could be expected in a ULS design event. The ground adjoining and beneath the building would still be at risk of settlement and lateral spread. Refer sections 4.3 and 4.4.
2	A cellular structure formed by secant CFA piles extending through the fill and marine deposits to found in the alluvium.	This foundation system can be developed to meet Building Code requirements. The ground adjoining the building would still be at risk of settlement and lateral spread. Refer sections 4.3 and 4.4. The ground beneath the building would not be at risk of settlement and lateral spread. The potential for lateral spread of the ground landward of the building would be reduced to some extent.

5.4 Geotechnical suitability to re-use cut material

Cut and fill earthworks are proposed as part of the re-development.

The cut material (excluding topsoil) is typically expected to be of the Upper Reclamation Fill which generally forms the upper 2 to 2.5m. This material is typically gravel with minor sand and silt.

Cut material below the existing carpark for excavation to Fale basement level is expected to be of the Lower Reclamation Fill which can be highly variable (silt, gravelly silt and silty sandy gravel).

It is expected both the cut Upper and Lower Reclamation Fill can be used as landscape fill, e.g. as fill to build up the amphitheatre / Harbour Lawn and demolished carpark / Malae to design levels. The design should consider safe batter angles for slopes constructed.

Subject to detailed design of structures, it is expected that imported structural Fill (e.g. GAP65) will be used below / around structures.

6 Further work

The following further work is recommended:

Table 6.1: Further Work

ID	Topic	Further Work
1	Design of shallow foundations supporting light structures and details for landscaping.	Designs/details to be developed by the project team in line with this report and in consultation with the geotechnical engineer. WCC to consider acceptability of life safety and damage risk and select design options accordingly.
2	Design of foundations to support Fale.	Foundation option selection: Fale Malae Trust to select the preferred foundation option in light of the relative merits of the options reported by the project team. Developed and detailed design: Analysis and development of the design which is likely to require further specific site investigations.

7 Applicability

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

We understand and agree that our client will submit this report as part of an application for resource consent and that Wellington City Council as the consenting authority will use this report for the purpose of assessing that application.

We acknowledge that the Fale Malae Trust will also submit this report as part of an application for resource consent in accordance with the Reliance Statement[13], and that Wellington City Council and Greater Wellington Regional Council as the consenting authority will use this report for the purpose of assessing that application.

Recommendations and opinions in this report are based on data from discrete investigation locations. The nature and continuity of subsoil away from these locations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
....	....
Emily Peebles Geotechnical Engineer	Dr. EngLiang Chin Project Director

Technical review by: Bhavesh Rama (Geotechnical Engineer) and Stuart Palmer (Technical Director)

EMPE
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[13] Tonkin & Taylor Ltd (April 2024), Letter to Fale Malae Trust titled "Reliance Statement – Frank Kitts Park Redevelopment". T+T Ref. 1018875.4.

Appendix A Figures

- **Figure A1. Proposed redevelopment of Frank Kitts Park**
- **Figure A2. Site plan**
- **Figure A3. Reclamation plan**
- **Figure A4. Typical harbour cross section**
- **Figure A5. Proposed structures part of redevelopment of Frank Kitts Park**

LEGEND

A - Harbour Promenade
B - Children's playground
C - Te Papa Whenua (planted area)
D - Harbour Lawn
E - Garden of Beneficence
F - Fale Building
G - Whairepo Lawn + Malae

REFER L000 FOR FOR PLAN LEGENDS

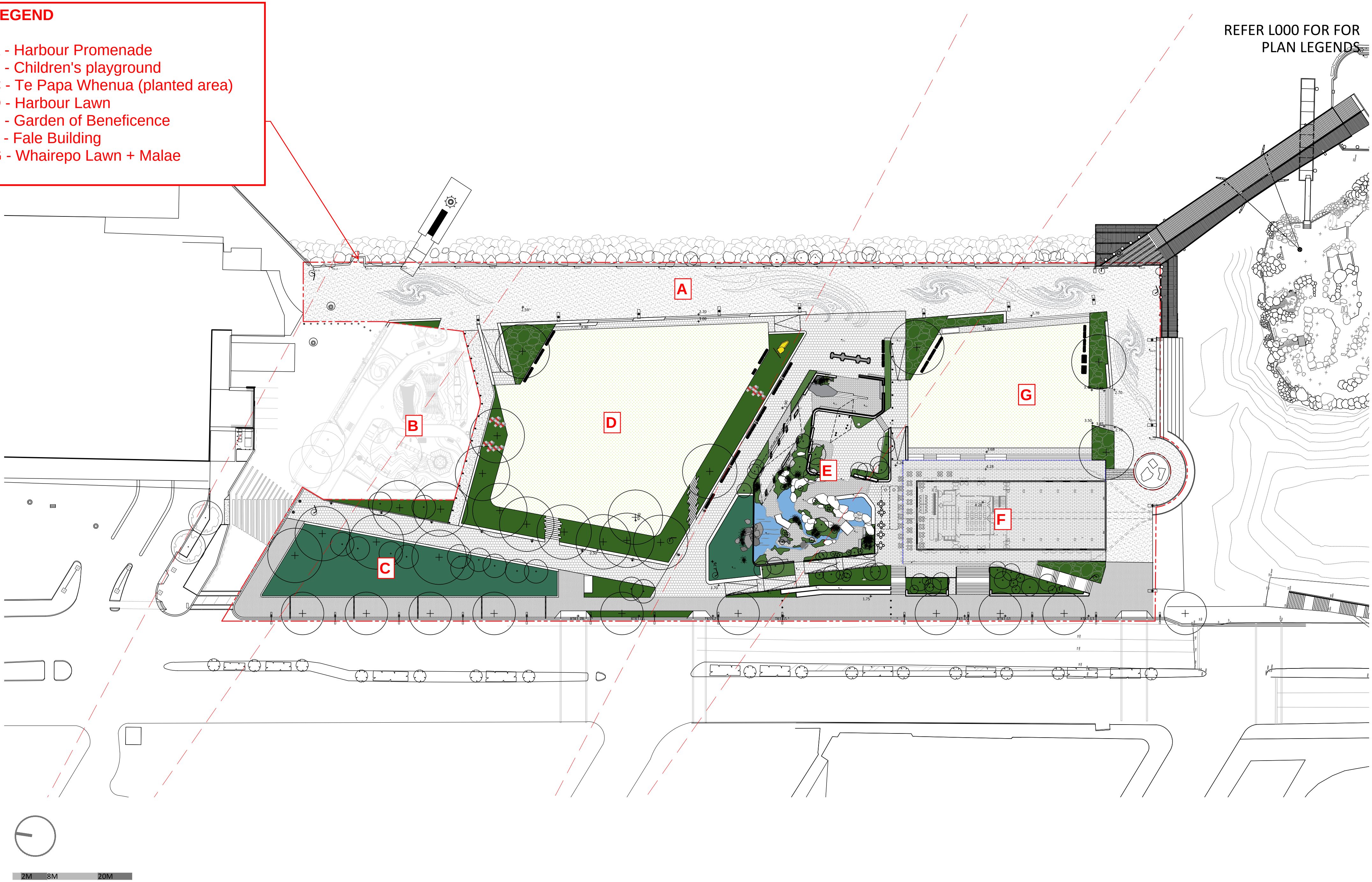

Figure A1. Proposed redevelopment of Frank Kitts Park

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DRAFT

NOT FOR CONSTRUCTION

DO NOT SCALE DRAWING.

issue	amendment	by	checked	date
-	FOR PRELIM COSTING	NT	NT	02.11.23

project name: FRANK KITTS PARK

project phase: PRELIMINARY DESIGN

project no: 0727

client: WCC

drawing title: FKP SITE PLAN

scale: 1:400 @ A1, 1:800@A3

drawing no. L101

LEGEND

1987 BCHF Investigation

Machine Borehole

2020 University of Canterbury Investigation

Cone Penetration Test

November 2021 Site Investigation

Borehole (Wellington Museum)

Borehole (Frank Kitts Park)

Borehole (Fale)

CPT (Wellington Museum)

CPT (Frank Kitts Park)

CPT (Fale)

Extent of carpark

Site boundary

A4 SCALE: 1:2,500

0 25 50 75 100 (m)

N

1. Property Boundary,Street Name,Street Number sourced from the LINZ Data Service and licensed for re-use under the Creative Commons Attribution 3.0 New Zealand licence.
2. World Imagery Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

Created On:	20/12/2021
Created By:	EPeebles
Approved By:	
TT Proj Ref:	1018875
TT Map Ref:	TTMAPREF1433463001.902

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WELLINGTON CITY COUNCIL
WELLINGTON WATERFRONT INVESTIGATION
Site Plan

FIGURE No.
A2

LEGEND

- Approximate extent of 1853 - 1889 reclamation fills
- Approximate extent of 1889 reclamation fill
- Approximate extent of 1970s reclamation fill
- Reclamation boundary
- Pre european shoreline
- Approximate location of mass concrete seawall.
- Approximate location of buried batter slope
- Site Boundary

Reclamation boundaries from Semmens, S. et. al. 2010. *It's Our Fault - Geological and Geotechnical Characterisation of the Wellington Central Business District.* GNS Science Consultancy Report, 2010/176. 52p.

A4 SCALE: 1:2,500

0 25 50 75 100 (m)

N

1. World Imagery Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

Created On:	08/12/2021
Created By:	EPeebles
Approved By:	
TT Proj Ref:	
TT Map Ref: TTMAPREF1433463001.902	

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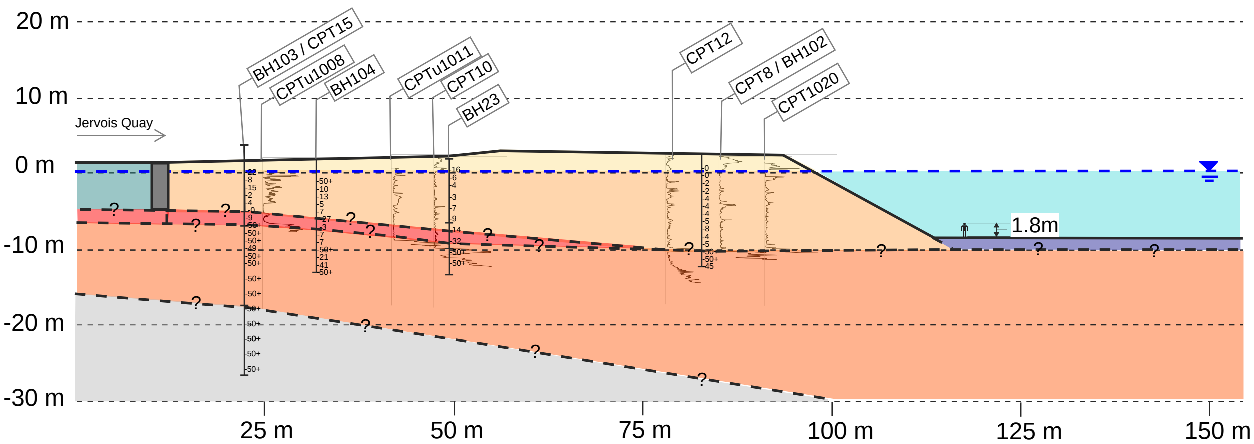
FRANK KITTS PARK

RECLAMATION FILLS PLAN

FIGURE No. A3



LEGEND

- 1970's Reclamation Fill (Upper)
- 1970's Reclamation Fill (Lower)
- 1900's Reclamation Fill
- Marine Deposits
- Alluvial Deposits
- Bedrock (HW or better)
- Harbour bed deposit
- Groundwater Level
- Buried seawall

A4 SCALE: 1:1000

0 20 40 (m)

Created On:	08/12/2021
Created By:	EPeebles
Approved By:	
TT Proj Ref:	

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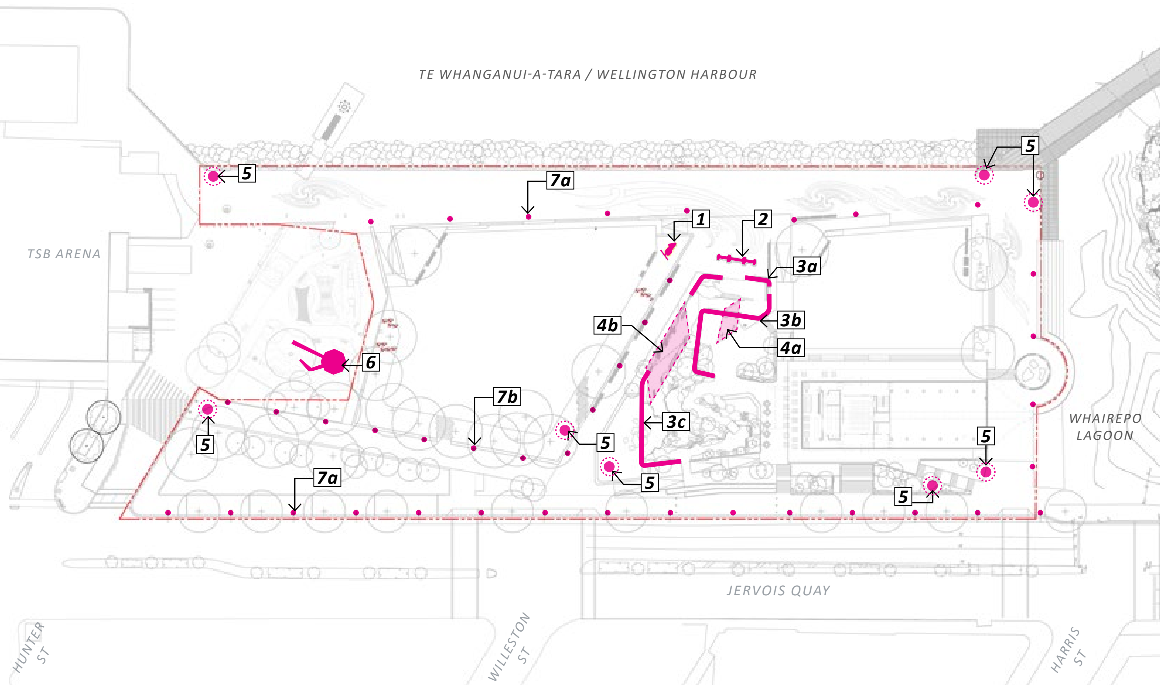
FRANK KITTS PARK

Typical Harbour Cross-section

Figure A4

STRUCTURES



ref	structure	height	quantity
1	wahine memorial mast	16.0m	1no.
2	pai lau	8.50m	1no.
3	walls		
3a	free standing type 01	2.50m	24.5lm
3b	retaining type 01	2.50m	33.0lm
3c	retaining type 02	3.70m	33.5lm
4	pavilion structures		
4a	pavilion 01	3.50m	1no.
4b	pavilion 02	5.00m	1no.
5	raukura feather sculptures	6.0m	4no.
		4.8m	4no.
6	lighthouse slide	10.80m	1no.
7	lightpoles		
7a	lightpole type 01/02	9.00m	27no.
7b	lightpole type 03	7.00m	12no.

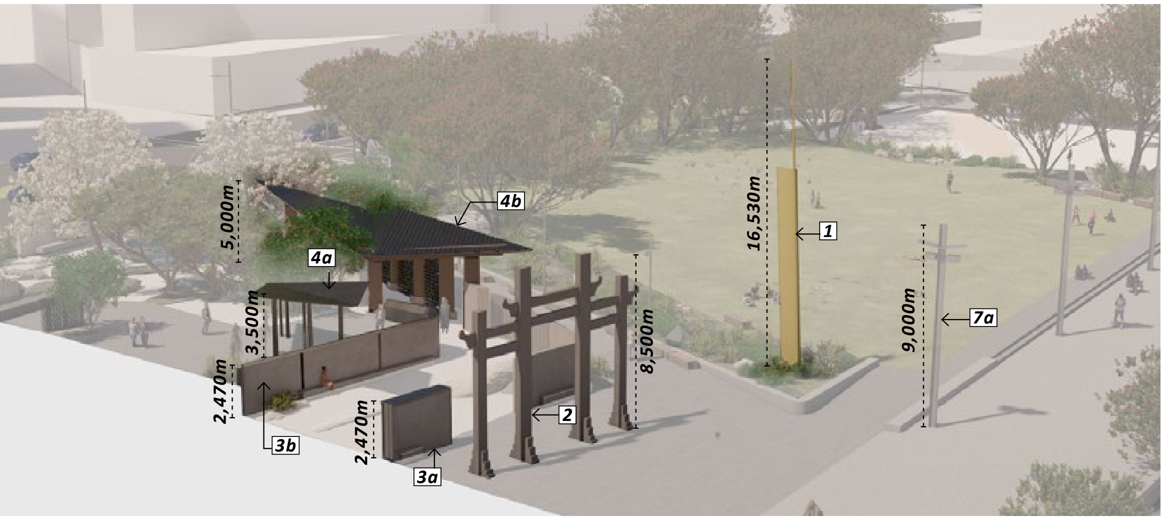


Figure A5. Proposed structures part of redevelopment of Frank Kitts Park www.tonkintaylor.co.nz