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LIQUEFACTION HAZARD OF WELLINGTON RECLAMATIONS BASED ON CONVENTIONAL ANALYSIS

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ABSTRACT

Severe liquefaction-induced damage occurred in reclamation fills at the port of Wellington (CentrePort) in the 2016 Kaikōura earthquake, but little or no damage was reported in areas of older and shallower reclamations in central Wellington. Recent studies have therefore primarily focused on understanding the liquefaction hazard of the port, while little is still understood with regards to the fill characteristics and liquefaction potential of the Wellington reclamations outside CentrePort. This study utilizes data from comprehensive field investigations, including 58 new cone penetration tests (CPTs) performed both within and outside the port in the Wellington waterfront area, supplemented with over 100 CPTs from our previous studies at CentrePort, to characterize the liquefaction resistance of the reclaimed fills in Wellington. The geotechnical data is first used to define simplified schematic soil profiles and to determine characteristic CPT parameter values (25th–50th–75th percentiles) for fills encountered in different reclamation areas. These analyses highlight differences in the soil profiles, and the relative similarity in the estimate of liquefaction resistance based on conventional CPT-based assessment, of fills encountered in different reclamation areas despite differences in the age, techniques, and materials employed in the construction of these reclamations. Conventional liquefaction assessments of reclamation fills based on CPT data are then performed over the wider waterfront area for a range of earthquake scenarios and ground motion intensities relevant for Wellington. For recent, past earthquakes, correspondence between predicted and observed severity of the manifestations of liquefaction vary depending on the earthquake event and area of observations. Likelihood of liquefaction occurrence and severity of the effects of liquefaction are then discussed for characteristic return periods, in the context of the seismic hazard of Wellington.

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INTRODUCTION

Land reclamations often comprise thick fills of liquefiable soils with little or no compaction efforts. The frequent use of granular fill materials placed in a loose state result in fills with a high liquefaction potential [1]. In the case of the Wellington waterfront, the reclamations are also in an area of high seismicity and host critical infrastructure that carry significant seismic risk. The inherent vulnerability of these reclamations was aptly demonstrated in the 2016 Kaikōura earthquake, which caused significant liquefaction-induced damage in the reclaimed land at the port of Wellington (CentrePort; [2]). This is not an isolated case, and observations of liquefaction-induced damage to reclaimed land areas accompanied other seismic events, such as the 1989 Loma Prieta earthquake, which caused extensive damage across different sites in the San Francisco Bay area, including hydraulic fill sand and silty sand reclamations at Treasure Island and in the Marina District [3,4]; the 1995 Hyogoken-Nambu earthquake, which caused severe damage to buildings, wharves, quay walls, and lifelines in the port of Kobe and other locations in the Bay of Osaka reclaimed with sands, and gravel-sand-silt fills derived from residual granite soils (e.g., [5-10]); and the 2011 Tohoku, Japan earthquake, where different land performance was observed in reclamations built since the 16th century in the Tokyo Bay area, suggesting that ageing effects increased the liquefaction strength of the oldest man-made deposits, mitigating the consequences of liquefaction [11].

It has to be recognized that the reclamations in the wider Wellington waterfront were built in a staged construction process across several decades (1850s to 1970s), resulting into a variety of types of fills and construction methods employed, and a wide range of fill thicknesses. The significant liquefaction-induced damage during the Kaikōura earthquake in some of the port reclamations starkly contrasted against the none-to-minor levels of damage observed in other reclaimed areas outside the port [12], demonstrating that the liquefaction performance of the fills depends both on the intensity of the earthquake excitation and the characteristics of the reclamations (e.g., soil composition, density state, fabric, thickness, age, and interaction with surrounding structures).

Our previous detailed studies focused on the CentrePort area [13-15], especially in relation to the performance of the reclamations during the 2016 Kaikōura earthquake and 2013 earthquakes. Other previous studies covering areas outside CentrePort [16-19] were based on regional-scale assessments of liquefaction hazard through generic interpretation of geotechnical and geologic data, with no site-specific studies. Though such studies generally identify high liquefaction potential for the reclamations along the Wellington waterfront, they fail to depict important differences between reclamation areas or develop clear understanding of how different reclamations perform under different earthquake scenarios. This highlights the need for greater rigour and sophistication in the evaluation of the liquefaction response, so that not only likelihood of occurrence, but also consequences of liquefaction are more rigorously evaluated in relation to various levels of earthquake excitations. This is especially important in view of the high exposure to seismic hazards of significant assets, important buildings, and critical infrastructure located in the waterfront area of Wellington.

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This study contributes to such improved liquefaction assessment of the Wellington reclamations by developing better understanding of the reclamation fill characteristics in the Wellington waterfront, and by identifying return periods (*RP*s) associated with initial triggering of liquefaction in the fills, as well as ground motion intensities at which severe liquefaction-induced damage is predicted. The paper first provides important background information on the reclamation works in the waterfront area, liquefaction-induced ground damage in reclamations caused by past earthquakes, and comprehensive geotechnical investigations undertaken within our studies. The present work integrates geotechnical data obtained from our earlier (2017-2019) investigation campaigns with 58 new CPTs to first define simplified, schematic soil profiles and provide CPT characteristics for the reclamations. The CPT data is then used to comparatively assess the liquefaction hazard for different reclaimed areas along the Wellington waterfront by performing simplified liquefaction analyses for different earthquake demands, considering both past seismic events and potential future earthquakes based on a probabilistic seismic hazard analysis (PSHA) for Wellington. Liquefaction-induced settlement versus intensity of ground motion relationships are developed for the different areas of reclamation. This provides the basis for a comparative evaluation of liquefaction triggering thresholds, characteristic evolution of the liquefaction response, and its maximum severity for the Wellington reclamations.

RECLAMATION HISTORY OF WELLINGTON WATERFRONT

The port of Wellington and part of the Central Business District sit on reclaimed land that was constructed in a series of land-development efforts since the European settlement in early 1840s, to the 1970s. The reclamation works can be generally grouped into three main phases, as illustrated in Figure 1. The reclamation efforts differ in terms of their area, thickness of fills, soils used in the reclamation process, methods of construction, and period of construction. Further details on the development of the reclamations are shown in Figure 2, derived based on interpretation of the 1892 historic survey map of central Wellington [20], historic aerial photographs [21], and previous studies focussing on the development of Wellington waterfront areas during the second half of the 20th century [22-25].

The first series of reclamations (green hatching in Figure 1) were constructed between 1852 and 1925 by end-tipping of materials obtained from local sources, such as soil from excavation works, demolition rubble, and quarried rocks and gravels. The thickness of these reclamations progressively increases as one moves from the original shoreline towards the sea. The seafront perimeter of these reclamations, and hence, their boundary with the subsequent reclamations, are delimited by concrete gravity seawalls of approximately 6-10 m in height, which approximately indicates the maximum thickness of these fills [24,26].

Another round of significant reclamations (red hatching in Figure 1) was carried out between 1924 and 1932 (along Aotea Quay) by hydraulic placement of sandy and silty sediments dredged from the nearby harbour seabed. The hydraulic fill reclamation is retained along Aotea Quay by a concrete gravity wall of approximately 10 m height [24].

The most recent rounds of reclamations (blue hatching in Figure 1) were undertaken between 1965 and 1976, usually by end-tipping of gravelly soils sourced from nearby quarries. In the Thorndon reclamation area, which was the focus of recent studies, these gravelly fills are well-graded containing 40-80% gravel-size particles by weight, with the remaining fractions being sand and non-plastic silts [2,27]. Cubrinovski [28] and Dhakal et al. [14-15] characterized these fills as gravel-sand-silt (G-S-S) mixtures, with sand and silt being the controlling fraction of the mixture despite the large portion of gravel particles by weight. The Thorndon reclamation is approximately 10 m thick along the old seawall of its northern perimeter and gradually increases to approximately 22 m thickness at the south-eastern corner of the reclamation, which is the deepest reclamation point into the sea. Unlike the older gravelly reclamations and the hydraulic fills, which are retained by quay walls, the Thorndon reclamation is an unconfined fill sloping towards the seabed, with an armoured rock slope-protection layer. The western and eastern edges of the Thorndon reclamation have a 1V:1.5H slope, while the southern face of the reclamation was reworked from an original 1V:1.5H slope to a gentler 1V:2H slope after the 2013 Cook Strait earthquake [13].

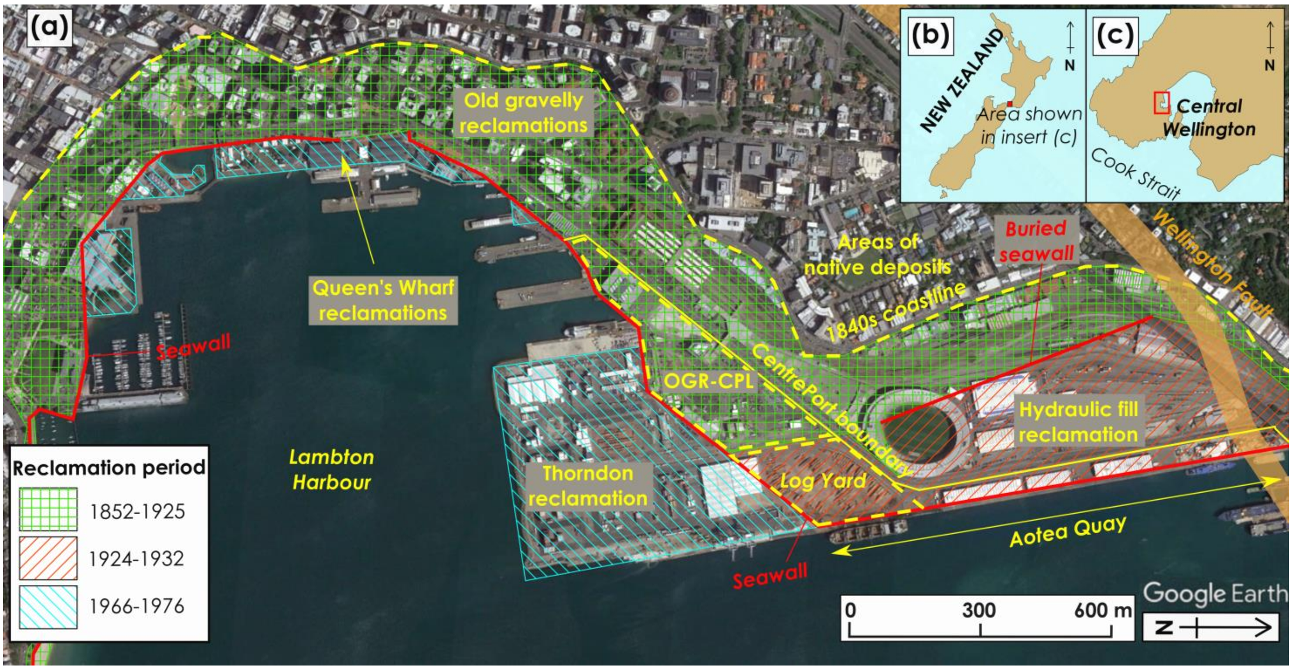

Figure 1: (a) Reclamation areas in central Wellington; (b) inset showing location of New Zealand and Wellington; and (c) inset of location of the central area of Wellington in the North Island of New Zealand.

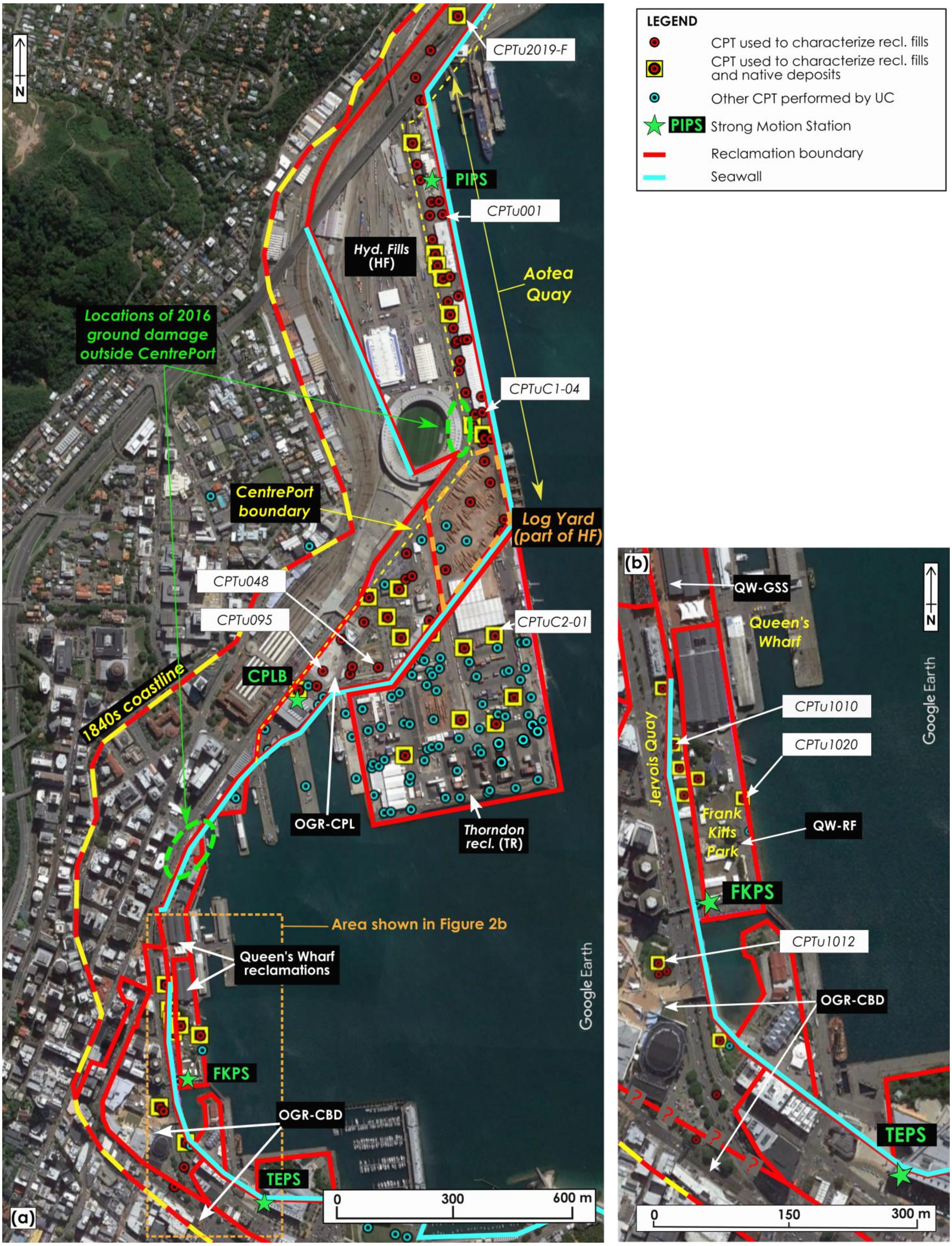

Figure 2: Locations of CPTs and their spatial distribution over (a) all reclamations investigated in this study, and (b) close-up of southern end of the study area.

The reclamations at Queen's Wharf (depicted in Figures 1 and 2) were carried out in two stages [23]. Firstly, an area to the south of the wharves was reclaimed in 1968-1970 by end-tipping of variable fill materials sourced from excavation works from around the city. The remaining strip of land, between the seawall along Jervois Quay on the landward side and the 1968-1970 reclamation to the east, was subsequently reclaimed in 1970-1973, concurrently with the area to the north of Queen's wharf, using gravelly material from nearby quarries.

The overall reclamation processes involved deposition of large volumes of soils into the seawater without any compaction efforts, except for a 2-3 m thick roller-compacted layer at the top of the fills (above the water table). Importantly, the loose, fully-saturated fills (below the water table) encountered across all waterfront areas comprise soils susceptible to liquefaction.

LIQUEFACTION OBSERVATIONS FROM PAST EARTHQUAKES

Historical Earthquakes (19th-20th Centuries)

The following historical earthquakes are known to have caused ground damage in central Wellington, possibly due to liquefaction [29-31]: 1848 Marlborough earthquake (estimated Modified Mercalli Intensity, MMI, VIII for the city centre), 1855 Wairarapa earthquake (MMI IX), 1929 Murchison earthquake (MMI VI), and 1942 Masterton earthquakes (MMI VI-VII). The 1848 and 1855 earthquakes are not relevant for the performance of the reclamations as they occurred prior to commencement of the reclamation works in Wellington. The 1929 Murchison earthquake caused insignificant damage in the Wellington reclamations. Finally, historic sources contain clear descriptions of observed surface manifestations of liquefaction in reclaimed land for the 1942 earthquakes, but due to incompleteness of records, the extent and magnitude of ground damage in central Wellington for these events are not clear.

2013 Cook Strait and Lake Grassmere Earthquakes

Unlike the aforementioned historical earthquakes, detailed observations of ground damage in the waterfront area exist for the 2013 Cook Strait seismic sequence [32-34] and the 2016 Kaikōura earthquake [2]. The 2013 Cook Strait earthquake (M_w6.6; source-to-site distance, $R \approx 45$ km) and 2013 Lake Grassmere earthquake (M_w6.6; $R \approx 65$ km) caused low-to-moderate ground shaking in Wellington, with horizontal peak ground accelerations (*PGA*s) at the ground surface in the CentrePort area of approximately 0.22g, and 0.11-0.15g, respectively [35,15]. The ground motions had relatively short significant durations of approximately 11-16 s.

Overall, the 2013 events caused no ground damage outside CentrePort, low-to-moderate (but limited in extent) liquefaction-induced damage in some parts of the port area, with major damage only along the southern perimeter of the Thorndon reclamation [32-34]. The Cook Strait earthquake triggered slumping failure of a 250 m-long section of the slope at the southwestern part of the Thorndon reclamation. The failure, which progressed up to 30 m inland, was accompanied by ground cracking, differential settlements, and traces of possible sand ejecta observed at four locations. The wharf structure along the western edge of the Thorndon reclamation moved laterally approximately 0.25 m, while settlements in the order of 0.05 m were observed in this reclamation. The subsequent Lake Grassmere event caused failure of an additional 50 m-wide section of the southern slope of the Thorndon reclamation. Some localized cracks were also observed in the hydraulic fill near the edge of the seawall along Aotea Quay.

2016 Kaikōura Earthquake

The 2016 Kaikōura (M_w7.8; $R \approx 60$ km) earthquake produced moderate shaking intensity in the Wellington area, with *PGA*s of approximately 0.25g recorded at CentrePort and significant duration of approximately 25-30 s [15], causing significant liquefaction-induced damage at the port, especially in the Thorndon reclamation [2]. Liquefaction resulted in widespread ground cracking, large areas covered by sandy-gravelly ejecta, and settlement of the reclamations of 0.3-0.5 m [14]. The liquefied fills moved laterally towards the sea, with horizontal movements exceeding 1 m along the southern face of the Thorndon reclamation. The movements induced by lateral spreading damaged the wharves along the Thorndon reclamation and resulted in large vertical offsets (differential settlement of up to 0.6 m) between the pile-supported wharf structures and the adjacent reclamation fill. Liquefaction-induced differential settlements and lateral stretching also caused damage to buildings on shallow foundations.

Severe liquefaction manifestations, with sand and silt ejecta, cracking, and ground settlements of up to 0.2 m were observed in the Log Yard hydraulic fills (location depicted in Figures 1 and 2), immediately to the north of the old seawall. Further north along Aotea Quay (location depicted in Figures 1 and 2), the remainder of the hydraulic fill reclamation underwent substantially less damage, with some settlement and cracking parallel to the revetment line, but with no soil ejecta or substantial ground movements.

In the older reclamations within the port (labelled OGR-CPL in Figure 2), ejecta and settlements greater than 0.1 m were reported only in an area of limited extent close to the buried seawall, but no significant liquefaction-induced damage was otherwise observed. Detailed description of the liquefaction-induced damage at CentrePort due to the 2016 Kaikōura earthquake is given in Cubrinovski et al. [2], whereas Dhakal et al. [14] provide liquefaction-damage maps for the 2013 and 2016 earthquakes.

In the waterfront areas outside CentrePort, little or no liquefaction-related damage was observed after the 2016 Kaikōura earthquake [12]. Localized cracking and subsidence were observed in the area between the stadium and the Log Yard, and along the waterfront retaining wall between the port area and Queen's Wharf (locations marked in Figure 2). This damage is generally classified as minor, with the exception of localized area to the north of Queen's Wharf, where Orense et al. [12] reported significant settlement of the fill.

CHARACTERISTIC SOIL PROFILES

CPT Investigations

The construction of reclaimed land in the waterfront area involved huge volumes of soils (e.g., over 3 million cubic metres of fill material for the Thorndon reclamation alone; [22]), which is typical for large-scale reclamation efforts. In addition, there were instances when the construction was performed with urgency, and consequently the reclamation works have relatively poor records of materials and processes used (e.g. [16,36]). As geotechnical investigations following the construction of the reclamations were limited, there was a need to perform detailed geotechnical characterization of the fills through comprehensive field investigations to better characterize the fills and understand their performance during the 2013 and 2016 earthquakes. To this end, the University of Canterbury led a series of comprehensive field and laboratory investigations including:

- Performing over 100 CPTs between 2017 and 2019, the majority of which were in areas significantly affected by liquefaction (Thorndon reclamation and the Log Yard).

However, approximately 30 CPTs were completed in the hydraulic fills along Aotea Quay that exhibited less evidence of liquefaction. These CPTs have been comprehensively studied by Cubrinovski et al. [13] and Dhakal et al. [14-15].

- Conducting additional investigations in 2020 to provide greater density of CPTs in the hydraulic fills along Aotea Quay and in the older reclamations, as well as to obtain more data in areas without liquefaction manifestations during the Kaikōura earthquake at CentrePort. These CPTs also aim to provide data on some of the reclamations along the Wellington waterfront outside CentrePort. In this investigation campaign, a total of 58 CPTs were performed.

The locations of all CPTs performed at CentrePort and in the city centre as part of our studies are shown in Figure 2. The figure also includes 5 CPTs performed by WSP at the northern end of Aotea Quay, so that the entirety of Aotea Quay can be considered. The particular use of the CPTs in this study is indicated in the legend of Figure 2, with Table 1 summarizing the number of CPTs analyzed for each reclamation. The analysis of the CPT subset covering the native deposits underlying the reclamations will be presented in the final section of the paper. All CPTs were performed according to the methodology described in Cubrinovski et al. [13] using 10 cm^2 and 15 cm^2 A.P. van den Berg I-cones after predrilling through asphalt and dense compacted fill cover. For the 58 CPTs performed in 2020, predrilling usually ranged from 1 to 2.5 m, with maximum predrilling depths of up to 4 m. Early refusal in the fills was encountered in 3 CPTs, where cone testing was continued beyond the refusal depth with a deeper predrill. Note that no clear particle-to-probe size effects were observed for the two cone sizes used, apart from occasional spikes in cone tip resistance. These findings reflect the key characteristics of the gravel-sand-silt fills encountered in many areas, namely that they comprise predominantly fine-to-medium gravel particles with angular shape, have matrix controlled by sand and silt fractions [28], and are in a loose state.

The present study utilizes the comprehensive CPT data to establish simplified soil profiles for different reclamations and to perform comparative liquefaction analyses of the reclamations. The following reclamations (indicated in Figures 1 and 2) are considered in this study (listed north-to-south): hydraulic fills (HF, along Aotea Quay and in the Log Yard), old gravelly fills at CentrePort (OGR-CPL), Thorndon reclamation (TR), Queen's Wharf reclamations (QW), and old gravelly reclamations outside CentrePort (OGR-CBD). The characterization of the Thorndon reclamation utilizes the results of the detailed study presented in Dhakal et al. [14-15], whereas details for the remaining reclamations are presented in this paper. For each reclamation, all CPTs within the reclamation were examined and used to develop simplified soil profiles. For reclamations with significant variation in the soil profile characteristics, the reclamation was subdivided into different areas, and typical soil profiles were identified for each area separately.

Table 1: Investigated areas with indication of reclamation period and number of CPTs considered in this study.

Reclamation	**Year**	**No. of CPTs used in this study**
Hydraulic fills (CentrePort; HF)	1924-1932	45
of which are in Log Yard		*7*
Old gravelly reclamation W (CentrePort; OGR-CPL)	1904-1916	17
Thorndon reclamation (CentrePort; TR)	1965-1976	7*
Old gravelly reclamations O-N (CBD; OGR-CBD)	1886-1889	7
Queen's Wharf (CBD; QW)		
Variable fill reclamation (QW-RF)	1968-1970	2
Gravelly fill reclamation (QW-GSS)	1970-1973	3

** These CPTs are representative of a larger data set of 31 CPTs considered by Dhakal et al [14].*

The simplified soil profiles and selected CPTs for each reclamation are shown in Figures 3, 4 and 5, with their locations indicated in Figure 2. The thickness of the fills is shown adjacent to the schematic description of soils throughout the depth of the profile. The CPT profiles include q_c (cone tip resistance) and I_c (normalized soil behaviour type index) plots. In the latter, the coloured bands in the background of the I_c plots correspond to the intervals of I_c = 1-1.6-1.9-2.2-2.6-3.0-4.0 and the soil behaviour type descriptions are based on the CPT-based soil behaviour classification scheme of Robertson [37-38] for similar values of I_c.

Conventionally, soils with $I_c < 2.6$ are considered liquefiable, while those with $I_c > 2.6$ are considered non-liquefiable (e.g. [40]). For the most frequent fills with $I_c < 2.6$ in each reclamation, interquartile ranges of normalized clean sand cone tip resistance, q_{c1Ncs}, have been reported in Figures 3, 4 and 5. The calculation of these q_{c1Ncs} values will be discussed later, after introducing the general features of typical soil profiles for the different reclamation areas.

Typical Soil Profiles of the Hydraulic Fills (HF)

Figure 3 shows the simplified soil profiles and respective selected representative CPT profiles for three areas of the hydraulic fills along Aotea Quay. The relative thickness of liquefiable and non-liquefiable (plastic, fine-grained) soils varies significantly from one location to another, with the total thickness of liquefiable soils ranging from negligible to 9 m. Despite the substantial spatial variability of the hydraulic fills, three different zones in the fills can be generally identified with the following characteristics: (1) fills comprising mostly plastic fine-grained soils with negligible presence of liquefiable soils (profile *HF-PS*); (2) fills with significant presence of sands and non-plastic silts (profile *HF-SS*), and (3) fills containing predominantly gravel-sand-silt mixtures (profile *HF-GSS*). The soil profile characteristics change from one zone to another over complex transition zones.

The *HF-PS* profile (Figure 3a) consists of a compacted gravelly layer, 2-3 m thick, overlying non-liquefiable plastic silts with thickness of up to 9 m. The plastic silts are characterized by low penetration resistance ($q_c < 1$ MPa) and I_c values greater than 2.7-3.0, which are typical for plastic silts and clayey soils. The fills are underlain by natural marine sediments, and an alluvial deposit composed of interbedded gravels and silts. The transition from the plastic silt fill to native marine sediments is often difficult to identify from the CPT data, as the two units have almost indistinguishable CPT characteristics [15]. For this reason, in some cases (e.g., the fills at Queen's Wharf discussed later) they are grouped into a single layer. The alluvium is capped by marginal marine sandy sediments, which are relatively loose in the CPT profile shown.

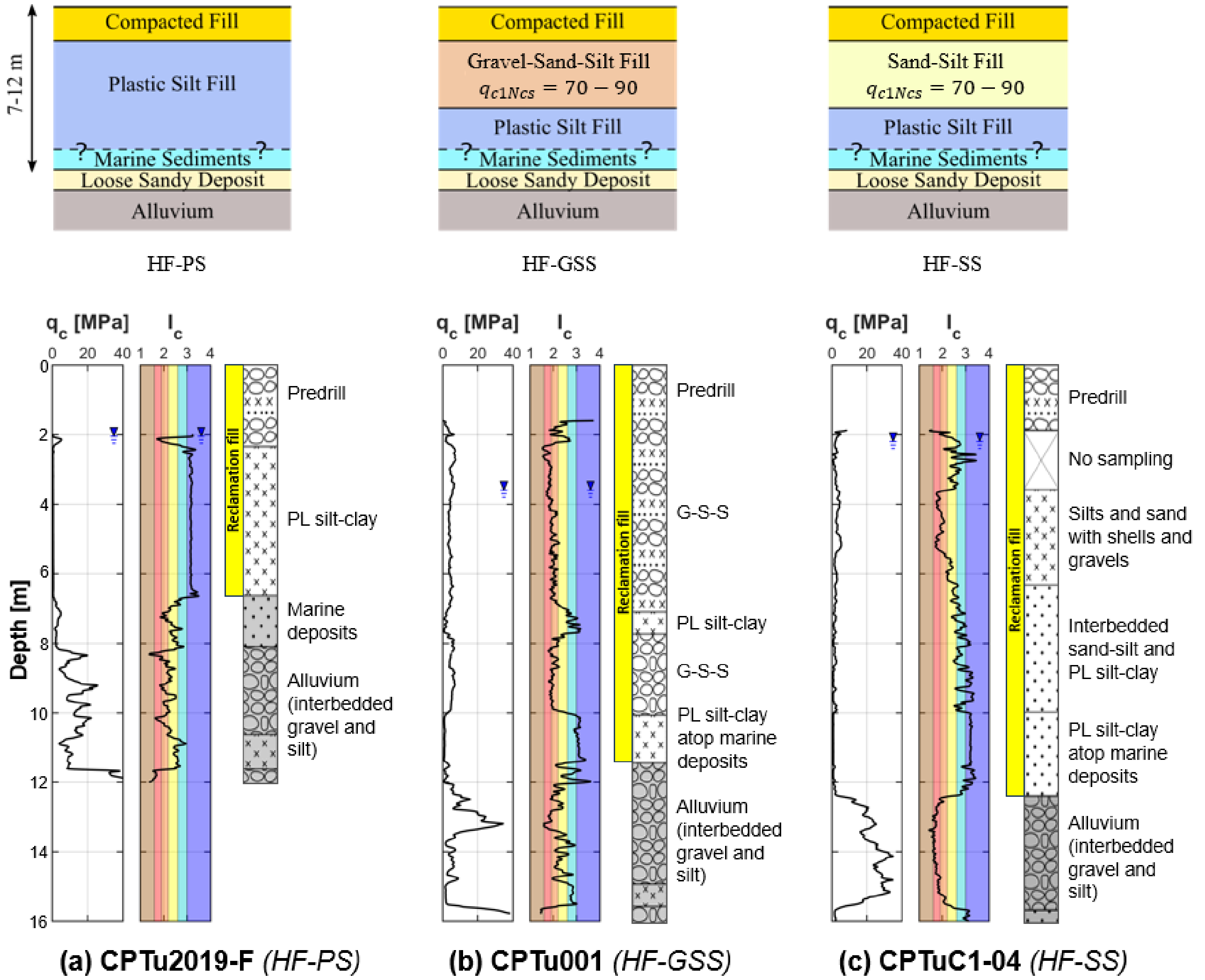

Figure 3: Schematic of typical soil profiles with interquartile range of q_{c1Ncs} for most frequent fills (refer Table 2) and one representative CPT profile in the hydraulic fill reclamation within CentrePort: (a) dominated by plastic silt fill; (b) containing gravel-sand-silt mixtures; and (c) containing silts and sands with shells. G-S-S denotes gravel-sand-silt fill; PL denotes plastic.

At other locations along the Aotea Quay, such as the profiles shown in Figures 3b and 3c, the hydraulic fills may be liquefiable (i.e., have I_c < 2.6). These fill materials are located immediately underneath the compacted crust and consist of either gravel-sand-silt mixtures (profile *HF-GSS* in Figure 3b) or sand-silt mixtures, with either frequent or occasional presence of shell fragments (profile *HF-SS* in Figure 3c); in some cases, shells can amount to over 50% of the material by weight. The underlying native marine sediments and alluvial layers are generally present in all profiles. In CPTu001 (Figure 3b), the gravel-sand-silt layer between 2 m and 5 m depth has q_c = 3-5 MPa and I_c mostly in the range 1.8-2.1. Similar values of q_c and I_c are also observed for the sandy/shelly fill between 3.5 and 5.5 m depth in CPTuC1-04 (Figure 3c). The similarities in q_c and I_c values for these two types of fills make it difficult to differentiate them using CPT data alone, and they were distinguished, in the specific case of these two CPTs, using soil cores recovered from nearby boreholes (< 5 m distance from each CPT).

Typical Soil Profiles for Older Gravelly Reclamations at CentrePort (OGR-CPL) and Thorndon Reclamation (TR)

As depicted in Figure 2, the older gravelly reclamations (OGR-CPL) are located north-west of the Thorndon reclamation, north of the old seawall. The fills typically consist of G-S-S mixtures with thickness of 4-10 m, with the top 3 m being compacted. At CPTu048 (Figure 4a), the fill at 3–8 m depth has q_c values of 4-7 MPa (with the exception of a 0.5-m thick layer at 7 m depth, with $q_c \approx 12$ MPa) and I_c = 1.7-2.2. Loose marine sediments and denser native alluvial soils are encountered at 8-9 m depth.

Based on the extensively characterization of the fills of the Thorndon reclamation by Dhakal et al. [14-15], two characteristic soil profiles have been generally identified, the simplified profiles of which are shown in Figure 4b. Most of the Thorndon reclamation fills have the characteristics of the simplified CPT profile *TR-GSS*, with 12 m (at the north end) to 22 m (at the south end) of gravel-sand-silt fills. The fills are generally loose, except for the compacted top 2-3 m. The Thorndon G-S-S fills have characteristic q_c and I_c values of 6.5-8.0 MPa and 1.9-2.3, respectively, as shown by the CPT trace at 3-14 m depth in Figure 4b. One relatively large area within the Thorndon reclamation contains a mound of sandy fill (profile *TR-S*), which is up to 10 m thick. Dhakal et al. [14] provide detailed cross-sections, based on the comprehensive CPT data, delineating the spatial distribution of the sandy fill within the G-S-S Thorndon reclamation.

Typical Soil Profiles of Older Gravelly Reclamations (OGR-CBD) and Queen's Wharf Reclamations (QW)

CPTu1012 (Figure 5a) was performed in an area reclaimed in 1889 (OGR-CBD, Figure 2), before the OGR-CPL reclamations inside CentrePort. The older fill consists of a 4 m thick G-S-S mixture with q_c = 2-6 MPa and I_c = 1.8-2.3.

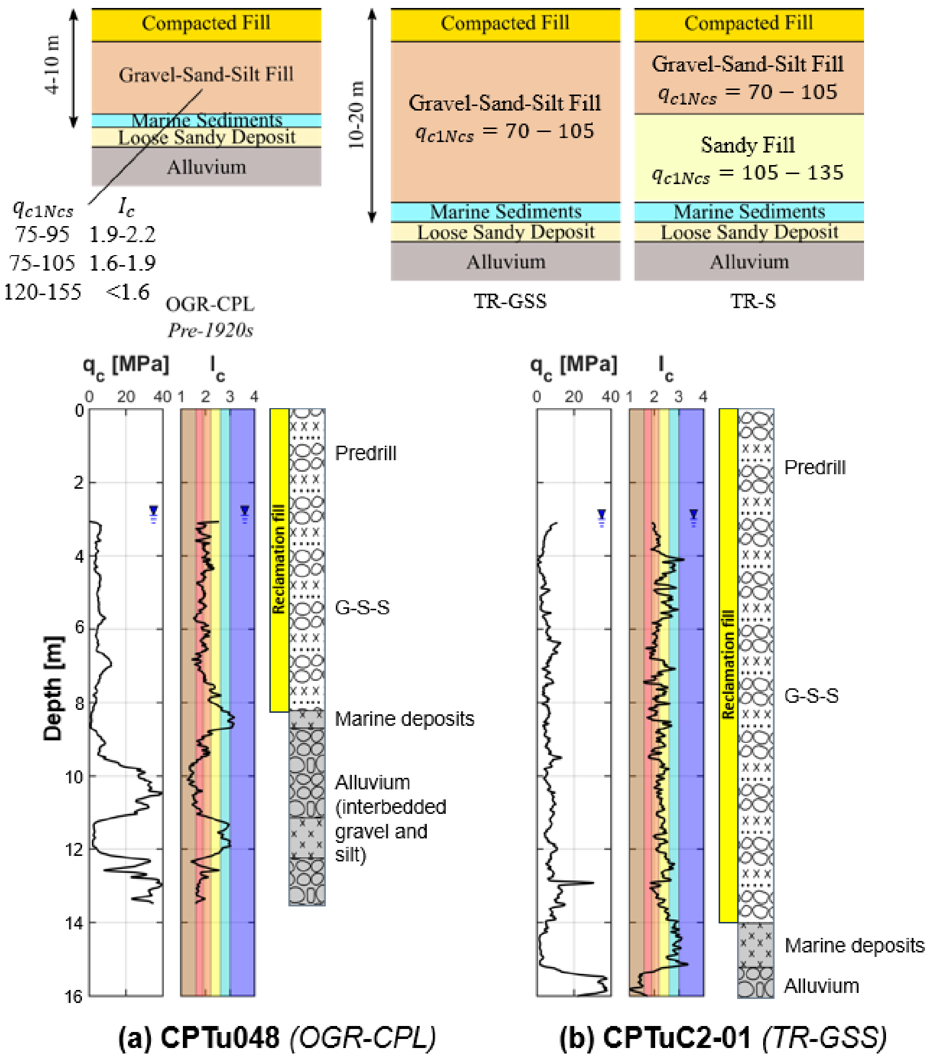

Figure 4: Schematic of typical soil profiles with interquartile range of q_{c1Ncs} for most frequent fills (refer Table 2) and one representative CPT profile in gravelly reclamations within CentrePort: (a) old gravelly reclamation; and (b) Thorndon reclamation. G-S-S denotes gravel-sand-silt fill.

Although it cannot be resolved with certainty, given the distance of the closest boreholes to the CPT, the topmost native layer (at 4-6.5 m depth) consists of either dense beach sands or alluvial gravelly soil mixture, and is underlain by interbedded soft silts and dense alluvial gravels. In this profile, the marine sediments are absent in the CPT trace.

The area immediately to the south of Queen's Wharf was reclaimed in two stages in which different materials were employed (Figure 2b): variable fills from excavation works (*QW-RF*) and quarried gravelly fills (*QW-GSS*). CPTu1020 (Figure 5b) is in the area reclaimed in 1968-1970, and shows alternating layers, with thicknesses ranging from 0.2 m to 2 m, of sands, silts, gravels, and their mixtures, and plastic silty-clayey soils, underlain by marine sediments. CPTu1010 (Figure 5c) is located on the strip adjacent to Jervois Quay reclaimed in 1970-1973 (location depicted in Figure 2b). The fill consists of a G-S-S layer overlying plastic soils with soft marine sediments, which sit atop the alluvium. As discussed before, the similar CPT characteristics (q_c and I_c values) do not allow to distinguish the lower silty-clayey reclamation fill from the underlying native marine sediments, therefore these soil units are combined into a single layer. The smaller thickness of the fill built in 1970-1973 (Figure 5c) with respect to the 1968-1970 reclamation (Figure 5b) is due to the dipping of the seafloor as one moves from the buried concrete seawall towards the sea.

Variability in Soil Profiles

To provide an indication on the variability of CPT parameters with depth in different areas, Figure 6 presents profiles of q_c and I_c for three reclamations: HF, OGR-CPL, and OGR-CBD. For clarity, the plot shows only five profiles for each area, and CPT data only for the reclamation fills and not for the underlying native deposits. Most CPT traces start at 1.5-2 m depth since the site fill at these locations was predrilled.

In the hydraulic fill reclamation (Figure 6a), one can observe that I_c values exhibit significant variations over the whole depth of the fills, ranging from 1.5-1.6 to 3, while q_c ranges from less than 1 MPa to values which, below 4 m depth, rarely exceed 7-8 MPa. It is worth bearing in mind that that the CPT profiles shown here, with fills up to 12 m thick, are representative of ground conditions in the area along Aotea Quay, on the waterfront (see Figure 2), and that the thickness of the reclamation fills gradually decreases to 4-6 m at the inland boundary with the older reclamations [29].

The CPT profiles for the old gravelly reclamations at CentrePort (OGR-CPL, Figure 6b) and in the city centre (OGR-CBD, Figure 6c) show less variability in I_c values, which rarely exceed 2.6, but highly variable cone tip resistance q_c, with spikes greater than 10 MPa likely reflecting the variability of materials used in the reclamation process and localized effects of gravel-sized particles on the CPT cone.

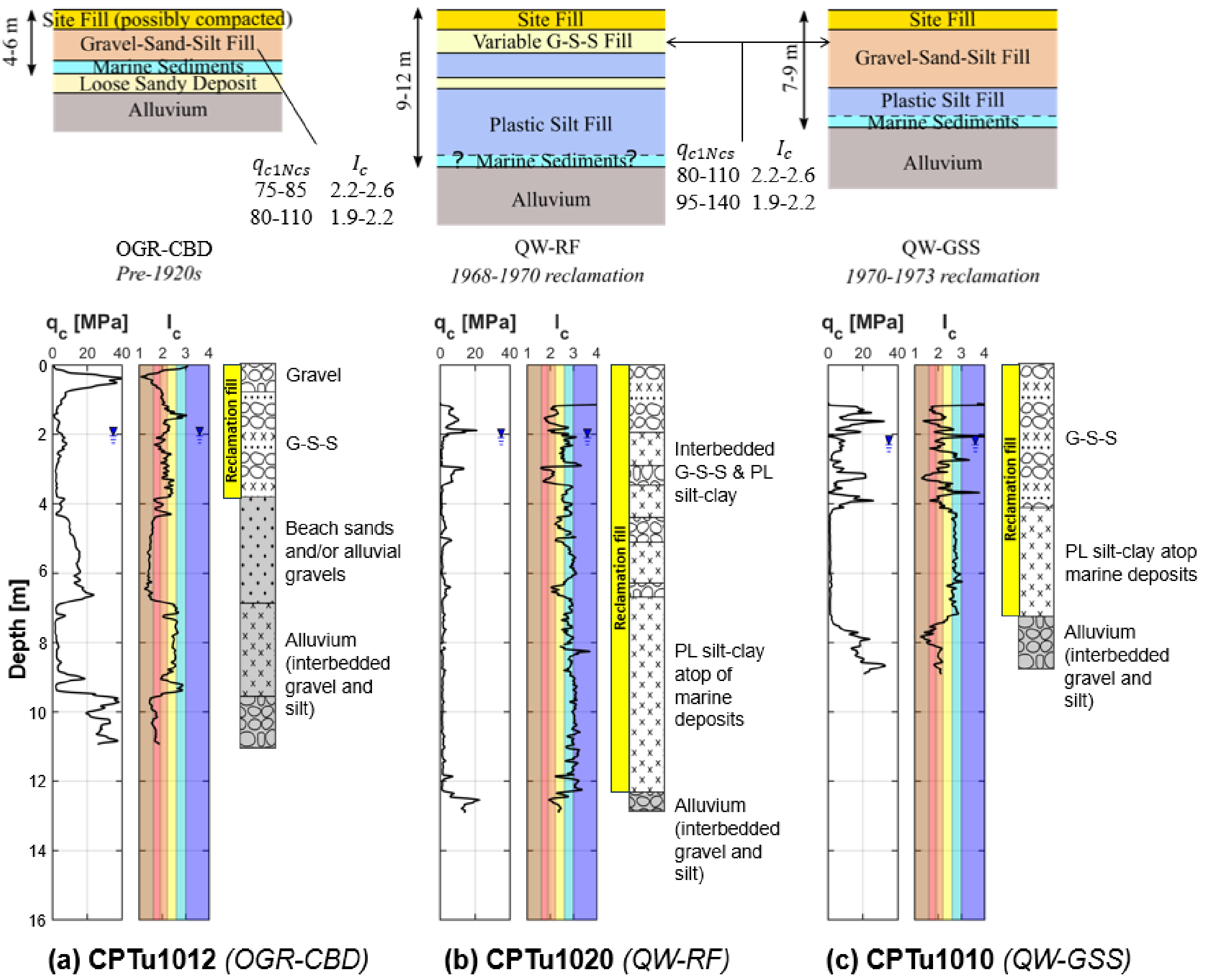

Figure 5: Schematic of typical soil profiles with interquartile range of q_{c1Ncs} for most frequent fills (refer Table 2) and one representative CPT profile in reclamations outside CentrePort: (a) old gravelly reclamation; (b) Queen's Wharf reclamation with variable fills; and (c) Queen's Wharf reclamation with quarried gravelly fill. G-S-S denotes gravel-sand-silt fill; PL denotes plastic.

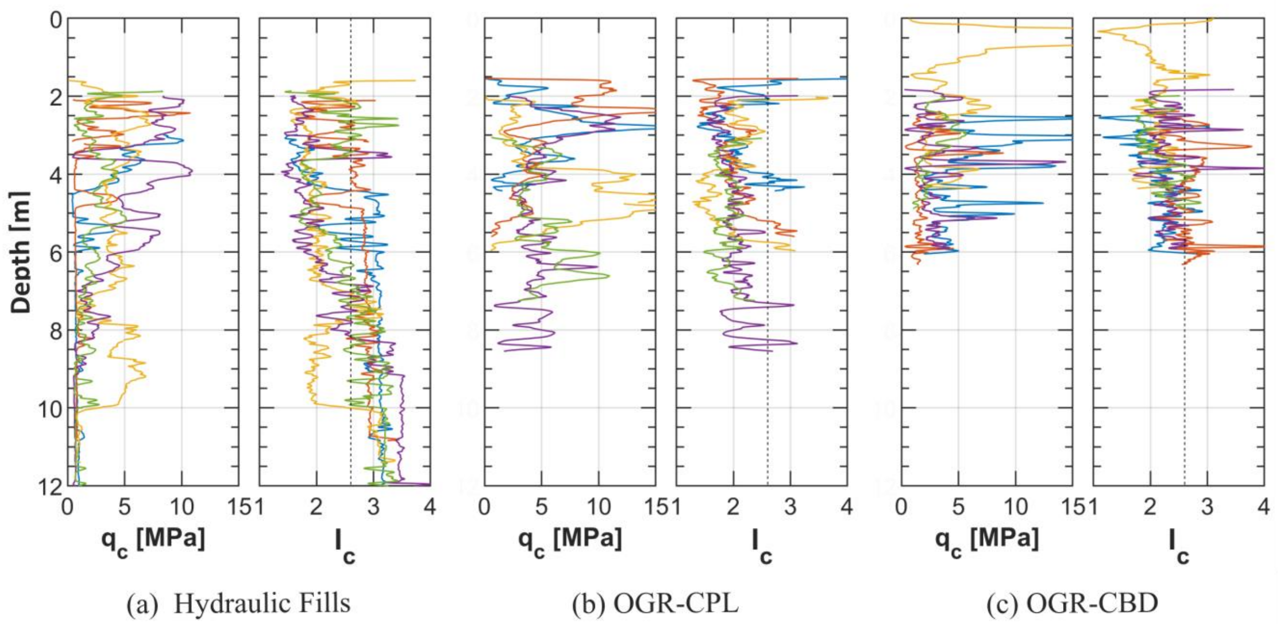

Figure 6: Example profiles of q_c and I_c for fills in: (a) hydraulic fill reclamation; (b) old gravelly reclamation at CentrePort; and (c) old gravelly reclamation outside CentrePort. In each subfigure (a, b and c), q_c and I_c profiles of a given CPT are plotted with the same colour.

While the q_c profiles presented in Figures 6b and c may suggest that the reclamation fills are more variable at CentrePort (OGR-CPL) than in the areas reclaimed before the 1920s in the city centre (OGR-CBD), this feature could be because the OGR-CBD CPT soundings were performed over a relatively limited area compared to the extent of investigated areas of older reclamations inside CentrePort.

Overall Soil Profile Characteristics of the Fills

The presented soil profiles reflect the following general characteristics throughout the depth of the reclamations at the Wellington waterfront:

- 2-3 m thick compacted gravelly layer at the top of the fill. This layer is sometimes not present in CPTs performed outside the port, e.g. in the Queen's Wharf area, and is expected to be inconsistent in other areas of the CBD (depending on details of reclamation works at each site).
- Saturated fill below the sea level, with thickness ranging from 2 m to 20 m; the thickness generally increases with increasing distance from the original shoreline towards the sea. It is noted that the waterfront area is mostly flat-lying. The upper-bound (shallowest) groundwater table, at most CPT sites, ranges from 2.0 to 3.5 m below the ground surface and is controlled by tidal fluctuations, which are in the order of ±1 m [39].
- The older fills (pre-1920s) have the smallest thickness (generally 4-8 m thick); hydraulic fills along the Aotea Quay are up to 10 m thick, whereas Thorndon reclamation fills are 10-22 m thick.
- A wide range of soil types are encountered in the fills including gravel-sand-silt mixtures, sands, non-plastic silts, plastic silts, and clayey soils. The simplified soil profiles in Figures 3, 4 and 5 show general deposit characteristics of the fills.
- The fills are underlain by native marine sediments, sometimes with beach deposits, and interbedded alluvial gravels and silts.

CPT CHARACTERISTICS OF RECLAMATION FILLS

To more rigorously characterize the fills and provide basis for a direct comparison of different reclamation soils, CPT characteristics of the fills were analyzed, interpreted, and quantified using the following approach:

1. Firstly, layers of the reclamation fill were distinguished from the underlying alluvium based on their CPT parameter values.
2. The reclamation fill was then subdivided into layers, so that each layer had well-defined (i.e., nearly constant) values of q_c and I_c. Highly interbedded parts of the fills and transition zones from stiff to soft soils were classified respectively as interbedded or transition units. The above approach was applied to all CPT profiles at locations marked by red symbols in Figure 2.
3. For all established layers with well-defined values of q_c and I_c (i.e., ignoring transition and interbedded layers), CPT data points within each layer were collated as a function of median I_c for different reclamation areas. When collating the data points, thin layers of less than 0.3 m thickness were ignored. This step, together with the exclusion of transition and interbedded layers, was taken to limit the possible influence of thin-layer effects on the CPT readings while using the original data for the analyses, rather than data points altered by thin-layer effects corrections.
4. Following this, a clean sand equivalent normalized cone tip resistance, q_{c1Ncs}, was calculated for each data point (using the q_c and I_c values) based on the liquefaction triggering procedure of Boulanger and Idriss [40]. The advantage of considering q_{c1Ncs} is that it can be calculated for all liquefiable soils and, importantly, q_{c1Ncs} allows to quantify the differences between liquefaction resistances of different reclamation fills.
5. Characteristic values (25th–50th–75th percentiles) of I_c and q_{c1Ncs} were finally computed, as a function of median I_c, for all reclamation fills in a given reclamation.

An example of an interpreted CPT profile using this approach (steps 1 and 2) is shown in Figure 7. Joint considerations of the q_c and I_c data (Figure 7a and 7b) and simplified stratigraphy (Figure 7c) show that the shelly sand-silty sand fill at 3.5-6.2 m depth was subdivided into five distinct layers. Within the highly interbedded zone from 6.2 m to 10 m depth, six distinct layers with well-defined values of q_c and I_c were identified, with the remainder of this zone classified as either "interbedded" or "thin". Finally, two layers with $I_c > 3.0$ were identified in the deeper plastic silt fill/marine sediments above the alluvium between two other layers marked as "interbedded" or "thin".

Table 2 summarizes the computed characteristic values of q_{c1Ncs} (i.e., 25th, 50th, and 75th percentiles) for the considered reclamation fills, organized in terms of I_c. Figure 8 shows the relationship between q_{c1Ncs} and I_c for liquefiable soils of different reclamations. Note that soils with $I_c > 2.6$ are not considered in this analysis as they are assumed to be non-liquefiable, and hence the use of q_{c1Ncs} is not appropriate for these soils. The following observations on the q_{c1Ncs} values reported in Table 2 and q_{c1Ncs}-I_c relationship shown in Figure 8 can be made:

- There is a relatively well-defined correlation between q_{c1Ncs} and I_c for the liquefiable soils of different reclamations, though some scatter is also evident. Generally, q_{c1Ncs} values computed based on Boulanger and Idriss [40] decrease with increasing I_c, but differences in q_{c1Ncs} are not pronounced for $I_c > 1.6$.
- The hydraulic fills (including the Log Yard) and Thorndon gravelly reclamation fills in the range of $I_c = 1.6$-2.6 all have very similar, and generally low, q_{c1Ncs} values in the range between 70 and 90, with a median value of approximately 80-85. This implies that, based on the simplified liquefaction assessment procedure of Boulanger and Idriss [40], these fills should have similar liquefaction resistance, which is also the lowest among all the fill units within the studied area.
- Similarly low q_{c1Ncs} values also characterize the old G-S-S reclamation, but only for higher I_c values (1.9-2.6 at CentrePort and 2.2-2.6 outside CentrePort, see Table 2). For lower I_c values (i.e., < 1.9 at CentrePort and < 2.2 outside CentrePort), approximately 10-15% higher q_{c1Ncs} values (and consequently slightly higher liquefaction resistance) are observed.
- Fills at the Queen's Wharf do not contain low q_{c1Ncs} values such as those encountered at CentrePort (Table 2), based on the limited number of CPTs performed at this location. The q_{c1Ncs} values for $I_c = 2.2$-2.6 are 10-15% larger than for the same I_c range as all other fill units. When $I_c < 2.2$, upper-quartile q_{c1Ncs} values are highest among all liquefiable fills in the areas of study. They are therefore expected to have the highest liquefaction resistances according to the simplified liquefaction triggering procedure of Boulanger and Idriss [40].

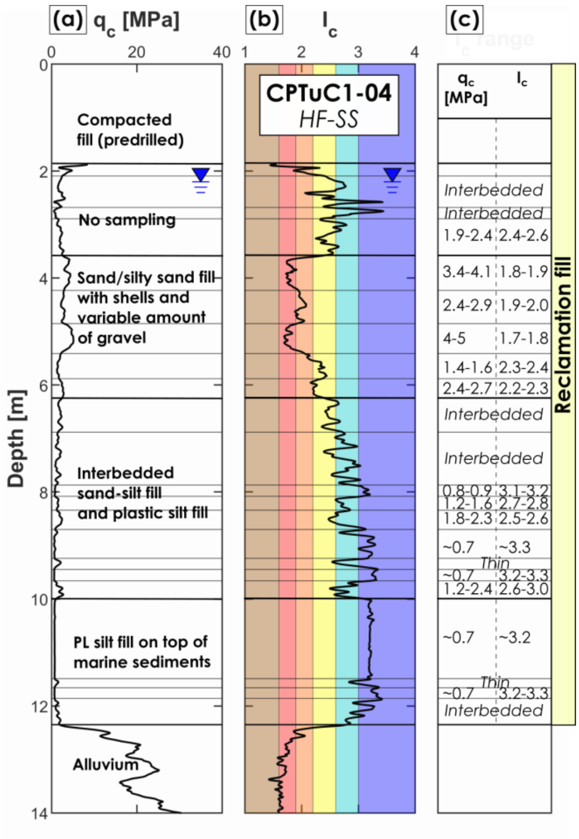

Figure 7: Example of subdivision of CPT profile into layers with near-constant values of q_c and I_c for CPT-based characterization of reclamation fill. Layers marked as interbedded or thin are not considered in the calculation of percentile values of q_{c1Ncs} and I_c.

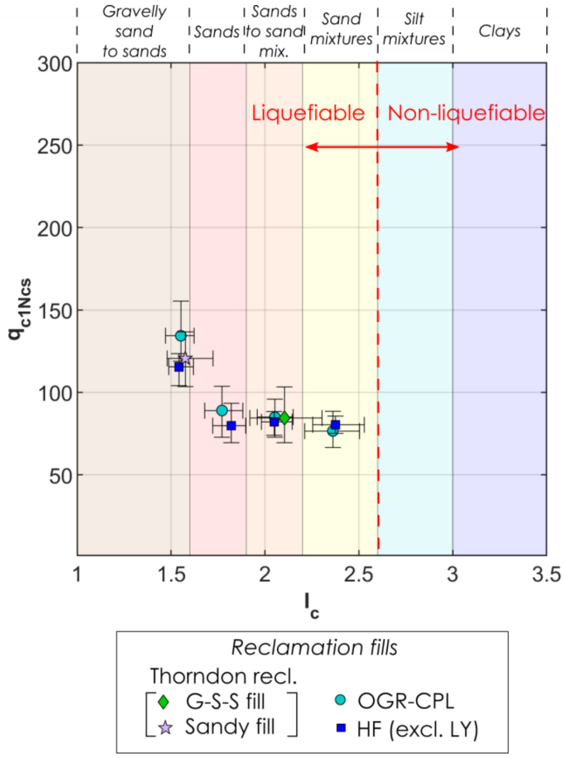

Figure 8: Comparison between q_{c1Ncs} values as a function of I_c for reclamation fills. Error bars indicate interquartile q_{c1Ncs} and I_c ranges; G-S-S denotes gravel-sand-silt fill. Soil descriptions on top of plot are based on CPT-based classification scheme of Robertson* [37]. *Data from the Log Yard (LY) are very similar to the rest of the hydraulic fill and hence are not shown for clarity, and OGR-CBD and Queen's Wharf data are omitted as their statistics are based on a small number of CPTs.

Table 2: Characteristic (25th-50th-75th percentile) values of q_{c1Ncs} as a function of median I_c for fill materials with $I_c < 2.6$ in different reclamation areas.

Reclamation	**q_{c1Ncs}**			
	$I_c < 1.6$ ***Gravelly sand to sands***	**$I_c = 1.6$-1.9** ***Sands***	**$I_c = 1.9$-2.2** ***Sands to sand mixtures***	**$I_c = 2.2$-2.6** ***Sand mixtures***
Hydraulic fill (HF) (excluding Log Yard)	104-116-123*	70-80-93	74-82-88	75-80-85
Log Yard	-	72-83-95	73-83-90	73-80-84
Old gravelly reclamation W (CentrePort; OGR-CPL)	119-134-155**	73-89-104	73-85-96	66-77-88*
Queen's Wharf reclamations (QW)	-	-	94-109-142	79-90-109
Old gravelly reclamations O-N (OGR-CBD)	-	-	81-93-111	76-82-87
Thorndon reclamation(TR)***	103-120-136 (I_c = 1.5-1.6-1.7) *Dense sandy fill (S)*		69-84-103 (I_c = 1.9-2.1-2.3) *Gravel-sand-silt fill (G-S-S)*	

** Limited occurrence of fill with given I_c values in this area.*
*** Excluding cobble/rubble fill encountered at CPTu095 site.*
**** Data reprocessed from Dhakal et al.* [14].

- Dense sandy reclamations encountered in the Thorndon reclamations were associated with I_c = 1.5-1.7 and the largest q_{c1Ncs} values (105~135) among the fills studied. These values imply larger liquefaction resistance than the gravelly reclamation according to the simplified liquefaction triggering procedure of Boulanger and Idriss [40]. Note that some locations in the Thorndon reclamation also contain loose sandy fill with CPT characteristics comparable to the Thorndon gravelly fill, details of which are discussed in Dhakal et al. [14] and Dhakal and Cubrinovski [41].

EVALUATION OF LIQUEFACTION PERFORMANCE OF THE FILLS FOR RECENT EARTHQUAKES

Simplified liquefaction analyses were performed to evaluate the liquefaction response of the Wellington reclamations for the three earthquakes experienced in 2013 and 2016. The analyses included triggering assessment based on the method of Boulanger and Idriss [40] and liquefaction-induced free-field settlements computed as per Idriss and Boulanger [42]. Analyses were performed using the earthquake magnitude for each event (M_w7.8 for the 2016 Kaikōura earthquake, M_w6.6 for the 2013 Cook Strait and Lake Grassmere earthquakes), while the input *PGA* values ranged from 0.05g to 0.50g, at 0.05g intervals. Thus, the applied *PGA* range covers a wide range of ground motion intensities including those recorded during these events. Such an approach provides better understanding of the evolution of the liquefaction response with increasing shaking intensity within the holistic framework of liquefaction assessment [43].

Computed Fill Response for the 2016 Kaikōura Earthquake

Figure 9 shows estimated liquefaction-induced settlements at the ground surface (s_v) of the hydraulic fills for a M_w7.8 earthquake, as a function of *PGA*. The results are plotted separately for different soil profiles, which are organized into three different groups based on the cumulative thickness of liquefiable soils within the fill (i.e., t_L = 0.5-2 m, 2-4 m, and 4-9 m). As t_L increases, the soil profiles transition from those dominated by plastic silt fills (*HF-PS*, Figure 3) to those with greater thickness of liquefiable soils (*HF-SS* and *HF-GSS*), noting that intermediate profiles exist between those indicated in Figure 3 (see Figure 6). Settlements at locations within the Log Yard are plotted with red dashed lines. As indicated in the figure, *PGA* = 0.24-0.25g corresponds to the geometric mean *PGA* values recorded during the 2016 Kaikōura earthquake at the strong motion stations CPLB and PIPS (strong motion stations locations depicted in Figure 2).

Variations are observed in the s_v computed for individual CPT traces within each group which reflect differences in the soil profile characteristics. Despite these differences, the threshold *PGA* for triggering of liquefaction is approximately 0.10g for all CPTs. Following the triggering of liquefaction, the computed settlements substantially increase as *PGA* increases from 0.10g to approximately 0.20g, beyond which they attain their maximum value. The latter reflects a particular feature of the simplified liquefaction assessment in which the maximum volumetric strain (and hence settlement) is capped at a specific value, and this cap is a function of the density of the soil. Thus, the results of the simplified analyses essentially imply that, for a M_w7.8 earthquake, triggering of liquefaction would occur at low shaking intensity of *PGA* = 0.10g, and for *PGA* = 0.20g practically all liquefiable soils in the hydraulic fills reach their maximum response in terms of liquefaction-induced settlement. The increase in the predicted maximum settlement across the different groups reflects the larger cumulative thickness of liquefiable soils in the fill.

Similar ranges of maximum settlements are generally predicted for the other reclamations (i.e., OGR-CPL, OGR-CBD, QW, and TR fills), as illustrated in Figure 10. For comparison purposes, shaded areas indicate settlement ranges for hydraulic fills with equivalent thicknesses of liquefiable soils (i.e., the ranges encompassed by the s_v-*PGA* curves plotted in Figure 9). Again, in these plots, the key factor controlling the maximum settlement is the total thickness of liquefiable soils. The following exceptions are noted:

- For one site in the old gravelly reclamation at CentrePort, OGR-CPL (CPTu095, blue line in Figure 10b; location depicted in Figure 2), reclaimed with cobble-sized demolition rubble, the predicted settlements are only 30-40% of those for other sites with t_L > 4.0 m in the same reclamation area.
- For soil profiles with t_L > 4.0 m, the predicted maximum settlements of 0.15-0.35 m along the Aotea Quay (shaded region in Figure 10d) are approximately twice those estimated for a CPT at Queen's Wharf, QW (line in Figure 10d). One contributing factor to the reduced settlement values for the Queen's Wharf site is the greater q_{c1Ncs} values (and hence lower volumetric strains, ε_v) of the liquefiable fills to the south of Queen's Wharf as compared to the other reclamations (Table 2).

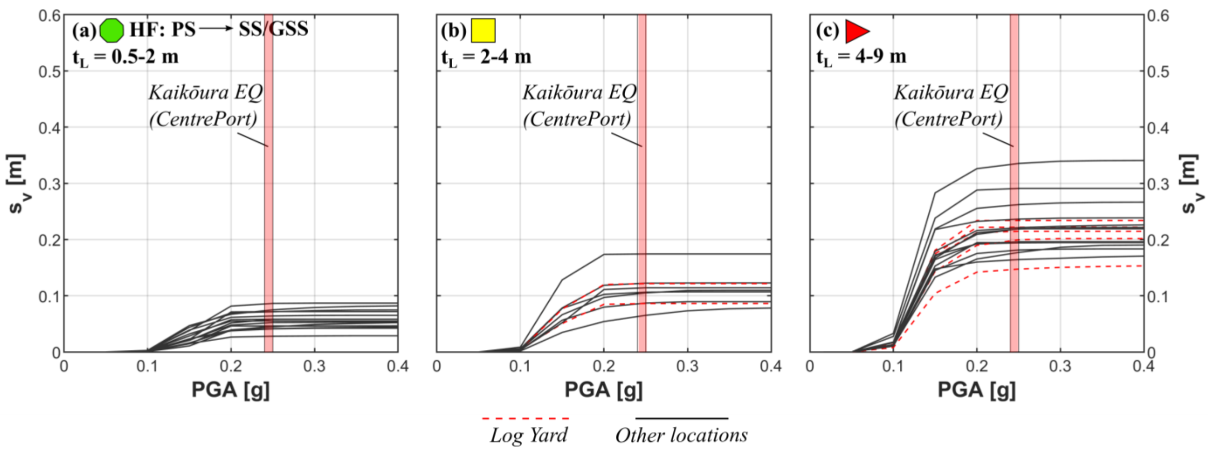

Figure 9: Estimated liquefaction-induced reconsolidation settlements of hydraulic fills (HF) for M_w7.8 earthquake and varying earthquake PGA grouped for profiles with different cumulative thickness of liquefiable soils (t_L): (a) t_L = 0.5-2 m; (b) t_L = 2-4 m; and (c) t_L = 4-9 m.

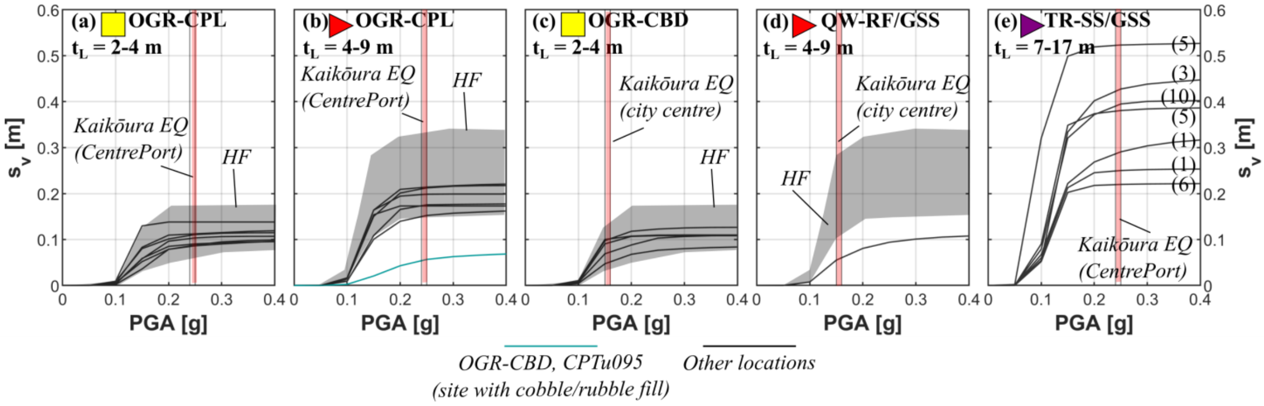

Figure 10: Estimated liquefaction-induced reconsolidation settlements of fills for M_w7.8 earthquake and varying earthquake PGA grouped for profiles with different cumulative thickness of liquefiable soils (t_L) for (a-b) old gravelly reclamation at CentrePort (OGR-CPL), (c) old gravelly reclamation in city centre (OGR-CBD), (d) Queen's Wharf (QW) reclamations, and (e) selected profiles from the Thorndon reclamation (TR). For comparative purpose, shaded zones indicate settlement ranges for hydraulic fills (HF) with equivalent thickness of liquefiable soils (from Figure 9).

Figure 10e shows the predicted settlements for seven CPTs in the Thorndon reclamation (TR), which are representative of a larger data set of 31 CPTs used by Dhakal et al. [14-15] to characterize subsoil conditions and fill properties in this area. The numbers in Figure 10e indicate the number of CPTs with similar characteristics to those used to compute each s_v-*PGA* relationship.

One can notice that the required *PGA* value to produce maximum settlements in the Thorndon reclamation (i.e., approximately 0.20g) is essentially the same as for the other reclamations. Given that the 2016 Kaikōura earthquake generated *PGA*s at the ground surface of approximately 0.25g in the CentrePort area and 0.15g in the city centre (indicated by the red region in Figures 9 and 10), state-of-practice liquefaction assessment procedures estimate the maximum settlement response should have been triggered in most reclamations for this event. Based on the q_{c1Ncs} values of the reclamation fills (Table 2 and Figure 8), the maximum volumetric strain (ε_v) values for the Thorndon fills estimated as per Idriss and Boulanger [42] are expected to be up to 20-50% smaller than ε_v of the fills in the other areas of the port. However, the reduced ε_v values are compensated by the greater thickness of the Thorndon fills.

General Observations from the Simplified Liquefaction Evaluation

Overall, based on the results of Figures 9 and 10, the following key observations and critical review of the predictions by the simplified liquefaction analyses can be made.

There are aspects of the settlement predictions that directly reflect key assumptions and limitations of the simplified analyses. For example, the analyses assume free-field, level ground conditions, which are not satisfied close to the slopes of the Thorndon reclamation or in soils adjacent to a retaining structure. The analyses assume that the maximum settlement potential for a given soil is achieved once the demand exceeds the capacity (liquefaction resistance) by a factor of approximately 2, which is a condition reached for the 2016 Kaikōura earthquake for the liquefiable soils of nearly all analyzed CPTs. This is illustrated by the fact that the predicted settlements corresponding to the 2016 Kaikōura earthquake are generally at the plateau of the settlement relationships shown in Figures 9 and 10.

Given that the liquefiable soils have very similar q_{c1Ncs} values (Figure 8 and Table 2), and that most liquefiable soils reach the maximum settlement potential for the 2016 Kaikōura earthquake demand, the maximum settlement value of the fills simply reflects the cumulative thickness of liquefiable (i.e., liquefied) soils. Figure 11 illustrates this characteristic by comparatively showing the cumulative thickness of liquefiable fills (Figure 11a) and the computed settlement (Figure 11b) for each of the analyzed CPT profiles. Evidently, profiles with thicker liquefiable fill tend to also have the largest computed settlements, and vice versa. This observation also highlights that the calculation of settlement in the simplified procedure requires additional interpretation on damaging effects as depth of liquefaction-induced settlement is not discriminated in the calculation (i.e., shallow and deep liquefaction contribute equally to the calculated settlement).

While the settlement predictions for the Thorndon reclamation are reasonable (given the assumptions and limitations of the simplified analyses), the equally large settlement predictions of > 20 cm for parts of the hydraulic fills along the Aotea Quay are substantially overestimating the observed settlements. Predictions for the old gravelly reclamations at CentrePort (OGR-CPL), are generally consistent with the magnitude and distribution of land damage observed in the area, which was concentrated along the buried seawall to the north of the Thorndon reclamation. At CPT locations in the city centre (OGR-CBD) and at Queen's Wharf (QW), observed damage was minor-to-none [12]; in this case the predicted response, with calculated settlements of 0.05-0.10 m (between the triggering threshold and the plateau of s_v-*PGA* curves), can be interpreted as a slight overprediction of observed performance.

This demonstrates an anomaly of the predictions by the simplified analyses for the Wellington reclamations, namely that simplified analyses frequently fail to discriminate between vastly different liquefaction performances in the field [39,41]. There are a number of potential contributing factors for these shortcomings of the simplified procedures. One major deficiency is that simplified analyses do not consider the dynamic response of deposits and ignore cross-layer interactions or system response effects, which are critically important for the evolution of the response in liquefying deposits [43,44]. Other potentially contributing factors are related to the uncertainties in the characterization of the liquefaction response of soils with complex composition or in the transition range of behaviour [45]. The effects of some of these uncertainties are further explored below.

Sensitivity Study

Sensitivity of the estimated settlements on the position of the ground water table (GWT) and liquefaction resistance of the

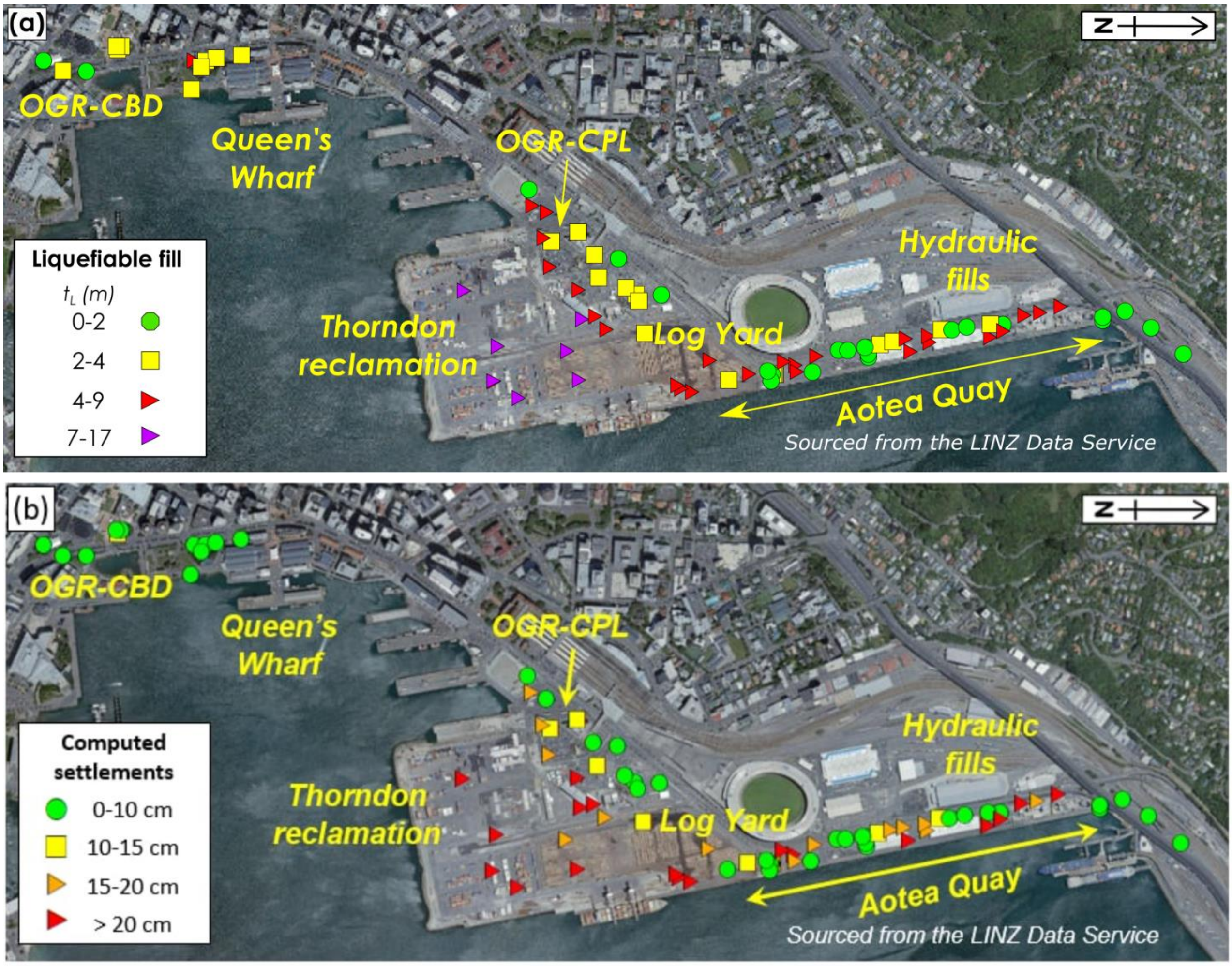

Figure 11: (a) Cumulative thickness of liquefiable reclamation fills (i.e., with I_c < 2.6 below upper-bound level of GWT) at waterfront CPT locations, and (b) estimated liquefaction-induced reconsolidation settlements of reclamation fills for the M_w7.8 Kaikōura earthquake (CentrePort: PGA = 0.25g; CBD: PGA = 0.15g).

fills was investigated using three CPT profiles from the hydraulic fills. The soil profiles are characterized by liquefiable soils with cumulative thickness of t_L = 0.5-2 m, 2-4 m, and 4-9 m, representative of profiles from the groups shown in Figure 9a, 9b and 9c, respectively.

Settlement results are presented in Figure 12a for three combinations of parameters: (i) shallowest water table and liquefaction resistance corresponding to P_L = 16% (which produces the largest liquefaction response for the investigated parameters) in the triggering method of Boulanger and Idriss [40]; (ii) deepest water table and liquefaction resistance corresponding to P_L = 50% (which produces the smallest liquefaction response for the investigated parameters), and (iii) an intermediate case using the deepest water table and P_L = 16%. The difference between the shallowest and deepest GWT was 1.37 m for all three CPTs.

Comparing the two cases with P_L = 16% (i.e., (i) and (iii) above), the predicted settlements for *PGA*s > 0.20g are reduced by approximately 4-6 cm for the cases with the deepest GWT, for all three CPTs. The relative difference in the maximum settlement is greater for profiles with thinner liquefiable fill (i.e., 50% reduction for t_L = 0.5-2 m as opposed to 15% reduction for t_L = 4-9 m) since a greater proportion of fill transitions from being liquefiable to non-liquefiable for t_L = 0.5-2 m than for t_L = 4-9 m when increasing the depth to the GWT by 1.37 m. The *PGA* levels at which settlements are triggered (~0.10g) and at which maximum settlements are attained (~0.20g) remain unaffected by variations in the position of the GWT.

The use of a median liquefaction resistance (P_L = 50%) instead of the conventionally used value for forward analysis (P_L = 16%) causes an increase in both *PGA* thresholds, i.e., *PGA* for triggering and attainment of maximum s_v, by approximately 0.05g. While the sensitivity of results on both depth of GWT and P_L is apparent in Figure 12a, the overall liquefaction responses and severity of liquefaction effects (i.e., magnitude of settlement) computed in the simplified analyses are generally not altered by the variation of these parameters.

Figure 12b comparatively presents equivalent results for two different earthquake magnitudes of M_w7.8 and M_w6.6 representative of the 2016 and 2013 earthquakes that affected the Wellington reclamations. *PGA* values recorded by strong motion stations at CentrePort during the two earthquakes in 2013 are also indicated in the figure (Cook Strait: 2013 CS EQ; Lake Grassmere: 2013 LG EQ). Comparisons of results for M_w7.8 and M_w6.6 show small effects of earthquake magnitude on the computed settlements for the three CPT profiles. This is unsurprising given that the characteristic values of the magnitude scaling factor, which accounts for the effects of M_w, are relatively small for liquefiable soils with predominantly q_{c1Ncs} = 70-90 [46].

For the 2013 Cook Strait (CS) earthquake (recorded *PGA* = 0.22g at CentrePort), nearly the maximum settlement response is predicted for all three soil profiles, which is inconsistent with the observed performance of the fills during this event. As previously discussed, the damage caused by the 2013 earthquakes was confined to the southern end of the Thorndon reclamation.

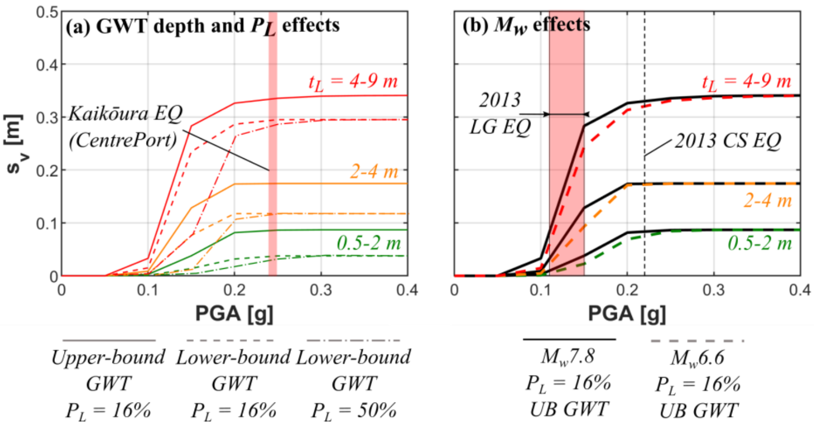

Figure 12: Estimated liquefaction-induced reconsolidation settlements of hydraulic fills (HF) for varying earthquake PGA and (a) moment magnitude M_w7.8, and different water table depths and P_L values, and (b) different moment magnitudes M_w.

In the hydraulic fills, only minor cracking was observed along the edge of the Aotea Quay seawall after the second (smaller) 2013 Lake Grassmere (LG) earthquake. Thus, the liquefaction-induced damage in the hydraulic fills with thick liquefiable soils is significantly overpredicted for the 2013 Cook Strait earthquake (see further evidence in [14]). In the case of the 2013 Lake Grassmere earthquake, recorded *PGA* values at CentrePort are close to the threshold for settlement triggering, and these results are generally consistent with the observations.

Considering the negligible influence that moment magnitude M_w has on computed settlements for the 2013 Cook Strait intensity of shaking (Figure 12b), the disagreement between predictions and observations for this earthquake is consistent with the corresponding poor performance of the conventional liquefaction assessment noted for the 2016 Kaikōura earthquake. More generally, the different accuracy of predictions of the simplified method for the hydraulic fill reclamation for the two 2013 seismic events are in line with the previous observations that simplified analyses frequently cannot discriminate between significantly different liquefaction performances in the field [39,41,43].

PREDICTED LIQUEFACTION PERFORMANCE OF RECLAMATIONS IN THE CONTEXT OF WELLINGTON HAZARD

PGA Hazard for Central Wellington

Wellington is a well-known hotspot of potential seismic activity in New Zealand. There are major fault systems in the region, with some local faults capable of producing large magnitude earthquakes ($M_w \geq 7.5$) traversing the city. Moreover, the Hikurangi Subduction zone dips beneath the city [47]. These factors contribute to a high seismic hazard for the city and define several distinct but equally relevant earthquake scenarios from an engineering assessment viewpoint. Significant ground motion amplification effects due to complex sedimentary basin structures underlying the waterfront area [48] further influence the ground motion hazard, as it has been evident in recent events [35,49-50].

One of the principal objectives of this study is to comparatively examine the liquefaction performance of different reclamations in the context of the seismic hazard of Wellington using the simplified liquefaction assessment procedure. Results from PSHA are required as input in the simplified procedure. Bradley et al. [51] and Cubrinovski et al. [52] present outputs from PSHA which was adopted for use in this study. Figure 13a shows the *PGA* hazard curves in terms of annual rate of exceedance of *PGA* values at sites with time-averaged shear-wave velocity in the top 30 m of $V_{s,30}$ = 200 m/s and 300 m/s. It is noted that the reference PSHA uses $V_{s,30}$ to account for one-dimensional site-amplification effects; basin effects are not explicitly considered as they require a site-specific study. The differences between the two hazard curves are relatively small, with *PGA* values for a given exceedance rate generally varying within 10%. As $V_{s,30}$ values in the reclamations are usually between 200 and 300 m/s [53], PSHA results for $V_{s,30}$ = 300 m/s were adopted in the analyses of this study. Figure 13 also shows as dashed lines the hazard curve according to the 2022 National Seismic Hazard Model [54] for Wellington, for $V_{s,30}$ = 300 m/s.

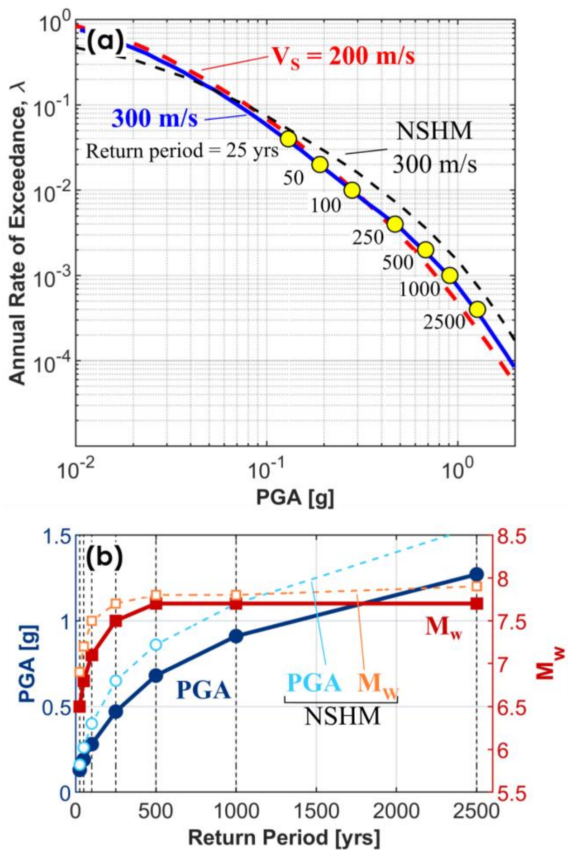

Figure 13: (a) PGA hazard curve for sites in Wellington with $V_{s,30}$ = 200 m/s and 300 m/s (modified after Bradley et al [51]); (b) PGA and M_w values for different return periods of seismic action for sites with $V_{s,30}$ = 300 m/s (values from Cubrinovski et al. [52]). The median hazard from the 2022 NSHM for Wellington for $V_{s,30}$ = 300 m/s [54] is plotted for reference using dashed lines.

For the purposes of liquefaction assessment using simplified method, the differences with the PSHA employed in the present study do not significantly affect the analyses presented herein, given the comparable levels of PGA and moment magnitudes at a given return period for the two hazards.

The earthquake load is defined in terms of *PGA* and M_w in the simplified liquefaction analyses, hence representative *PGA* and mean magnitude values were extracted from the hazard curves for return periods (i.e., inverse of the annual rate of exceedance) *RP* = 25, 50, 100, 250, 500, 1000, and 2500 years. As illustrated in Figure 13b, *PGA* values increase from 0.13g at *RP* = 25 years to 1.27g at *RP* = 2500 years, while the mean M_w takes values from 6.5 at *RP* = 25 years to 7.7 at *RP* ≥ 500 years. The *PGA*-M_w combinations shown in Figure 13b were used to define the relevant earthquake loading in the simplified liquefaction analyses of Wellington reclamations. One should note that the 2013 Cook Strait earthquake and the 2016 Kaikōura earthquake produced *PGA* values at the port of Wellington that approximately correspond to *RP* = 25-50 years and *RP* = 50-100 years, respectively.

Predicted Liquefaction-Induced Settlement of the Fills

The holistic assessment approach introduced previously was applied to the investigated reclamations using the *PGA* and M_w combinations from the Wellington hazard as input in the simplified analyses. Figure 14 shows the ranges of computed liquefaction-induced settlements for the considered reclamations as a function of the input *PGA*. Each input *PGA* corresponds to a different return period (indicated by labels in the plots) and is associated with a respective earthquake magnitude M_w. The plotted grey shaded zones show the range of results for the hydraulic fills, Thorndon reclamation, old gravelly fills at CentrePort (OGR-CPL), and the fills at the city centre (OGR-CBD and QW). The plotted ranges for the native deposits (shown in blue) are discussed in the subsequent section.

The 100 mm and 200 mm free-field settlement values marked in red are indicative of thresholds for moderate and severe liquefaction-induced building damage for buildings on shallow foundations (consistent with the demarcations in Figure 11b). These threshold values refer to total free-field settlements, and are indicative of the severity of liquefaction effects on shallow foundations, given the proximity of liquefied soils to the ground surface. As previously noted, the groundwater table in the waterfront area varies between 2 and 3.5 m below the ground surface, and liquefiable reclamation fills, if present (e.g., not for *HF-PS* profiles), are generally located in the upper part of the soil profiles (see Figures 3, 4 and 5). Therefore, as total settlements due to shallow liquefaction increase, differential settlements, which drive building damage, are also expected to increase. Note that shear-induced settlement due to building oscillation, SSI and ratcheting effects will be in addition to the reconsolidation settlements presented in this paper (e.g., [55]). The magnitude of shear-induced settlement will strongly depend on the contact pressure (i.e., weight of the building), thus the use of s_v only as an indicator of the severity of damage. For the above reasons, the reconsolidation settlement damage thresholds marked in Figure 14 do not apply to buildings on deep foundations, given the assumptions behind the estimate of s_v, (i.e., integration of reconsolidation strains across the soil profile [42]).

Although the magnitude of predicted settlements changes significantly from one CPT to another, the triggering threshold for settlement is similar for all fills and locations. Importantly, liquefaction-induced settlement of the fills is triggered at low *PGA*, corresponding approximately to *RP* = 25 years (*PGA* = 0.13g and M_w6.5). For *RP* = 50-100 years (*PGA* = 0.19-0.28g and M_w6.8-7.1), the settlement values reach the maximum level that can be computed by the simplified method. Further increase in the demand results in a steady settlement response at its maximum level, which is a by-product of the simplified analyses, as discussed previously. Thus, simplified analyses indicate that significant liquefaction-induced settlements in the fills will occur frequently, with probability of occurrence between 63% and 98% for structures with 100 years design life.

The predictions based on the simplified liquefaction assessment would suggest a relatively uniform response of the different reclamation fills as the level of shaking increases, with similar thresholds for liquefaction triggering, and also for the attainment of the maximum settlement values in different areas. There are currently no observations of land performance for levels of ground shaking corresponding to events with *RP* greater than 100 years. However, comparisons between observed and predicted liquefaction-induced settlements for 2013 and 2016 seismic events (with recorded PGA levels at CentrePort corresponding to *RP* = 25-100 years), suggest that land performance will vary significantly across the waterfront for levels of shaking between the onset of triggering (around *PGA* = 0.10g) up to at least the plateau of the s_v-*PGA* response.

The shortcomings of simplified analyses and their inability to consistently differentiate liquefaction responses of different severity (i.e., deposit characteristics) have been emphasized by Cubrinovski et al. [44] and Cubrinovski and Ntritsos [43], and

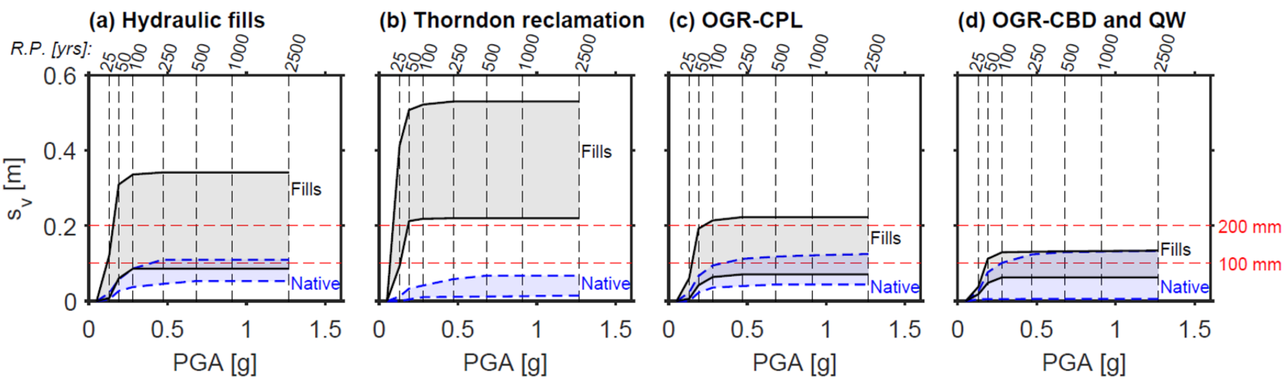

Figure 14: Predicted liquefaction-induced reconsolidation settlements at selected CPT locations as a function of PGA for reclamation fills and native deposits in: (a) hydraulic fills reclamation; (b) Thorndon reclamation; (c) old gravelly reclamations at CentrePort (OGR-CPL), and (d) reclamations at the city centre (OGR-CBD and Queen's Wharf). PGA values correspond to different return periods (labelled), with the associated M_w for the given return period (in years) according to Figure 13.

have been attributed predominantly to the fact that interactions and system response effects are ignored in simplified analyses. It is also important to note that simplified analyses assume free-field, level ground conditions, and hence, do not account for effects of retaining structures, slopes, buildings and piled wharves, which also contribute to the liquefaction responses observed in the field. The presented results of simplified analyses should be interpreted with these limitations in mind. A more rigorous evaluation of the seismic response of the Wellington waterfront reclamations including various port structures using nonlinear dynamic analysis is currently under way.

CONTRIBUTION OF UNDERLYING NATIVE SOILS TO LIQUEFACTION-INDUCED SETTLEMENT

Although this study focuses on the reclamation fills, a few observations will also be made on the contribution of underlying native soils to liquefaction-induced settlement of the reclamations. As previously outlined, the waterfront reclamations are usually underlain by marine and beach deposits, and alluvial, colluvial, and swamp deposits, all of which are of Holocene age. These native soils sit on top of Pleistocene alluvium, and colluvial gravelly deposits, with interbedded silt and clay lenses which become less frequent as depth increases [25,36].

Geotechnical investigation boreholes available in Wellington are rarely dated [36]. Past studies have therefore correlated the top, looser sediments (SPT blow count N = 5-60, but mainly 10-20) to post-glacial Holocene deposits, and the underlying denser sediments (N = 30-120 for gravelly layers, and 10-70 for silts and clays) to Pleistocene deposits (e.g. [36,48,56]). A selected subset of 28 deeper CPTs pushed through native soils (indicated in Figure 2 as symbols with yellow boxes), covering different reclamation areas and characteristic soil profiles, was employed for the liquefaction assessment of the native deposits. For this purpose, marine silty sediments ($I_c > 2.6$), and the shallowest silty ($I_c > 2.6$) and sandy ($I_c < 2.6$) deposits with low q_c values located atop the native deposits were separated from the dense underlying alluvium (consistently with the abovementioned studies based on SPT data). The alluvium was then subdivided into layers and characterized following the same procedure previously described for the reclamation fills (i.e., based on I_c as summarized in Table 2 and Figure 8).

It is noted that, for reclamation fills in all areas apart from Queen’s Wharf, interquartile ranges (i.e., 25th and 75th percentile values) of normalized friction ratio (F_r; [38]) increase from approximately 0.3-1% at I_c = 1.9-2.2, to 1-2% at I_c = 2.6-3.0. For Queen’s Wharf fills, F_r interquartile ranges are 0.8-2.2% at $I_c < 2.2$, and 2-3% for $I_c > 2.2$. A significantly broader range of F_r values was observed in the native soils as compared to the fills with similar values of I_c for I_c = 1.9-3.0. For this reason, an additional subdivision based on a threshold value F_r = 3% for soils with I_c = 1.9-3.0 was introduced for the native deposits.

For the native soils, characteristic values of q_{c1N} (i.e., normalized cone tip resistance to a reference overburden stress of 100 kPa) were classified in terms of their I_c value, as shown with symbols in Figure 15. The error bars indicate the interquartile range (i.e., 25th and 75th percentile values). The shaded blue region indicates the interquartile range observed in the fills (from data shown in Figure 8), with a dashed black line indicating the median relationship for the fills. The following observations are apparent from Figure 15:

- Median q_{c1N} values in the loose shallow marine sediments (yellow triangles) and alluvial deposits with $I_c > 2.6$ are similar to the reclamation fills with similar I_c values.
- In general, liquefiable native soils (based on $I_c < 2.6$ criteria) exhibit substantially higher q_{c1N} values compared to the respective q_{c1N} values of the fills. The larger q_{c1N} values in the alluvial deposits reflect their higher density and older age, both of which imply substantially higher liquefaction resistance.

To quantify the effects of native soils on the liquefaction response, liquefaction analyses were performed for the selected 28 CPT profiles including the native soils beneath the fills. In these analyses, for all native soils apart from the shallowest cover layer of loose sandy sediments, a liquefaction resistance corresponding to P_L = 50% was used as a first approximation to reflect the effects of their older age on their liquefaction strength, as compared to the fills, bearing in mind the limited availability of ageing data for the Wellington native deposits previously noted [36].

Results of these analyses are shown in Figure 14, which illustrates the contribution of native soils to liquefaction-induced settlements (blue shaded ranges) as compared to the settlement from the liquefied fills (grey shaded ranges). The settlement due to liquefaction effects in the native soils is much smaller than that the respective settlement of the fills for all reclamations with significant thickness of liquefiable fills. The only exception to this trend is seen in the profiles in the city centre (OGR-CBD and Queen’s Wharf; Figure 14d), for which the native soils in some cases cause settlements comparable to those of fills with small thickness of liquefiable soils. Note that, in these cases, the maximum settlement values remain relatively small i.e., in the range between 0 to 10 cm.

Details of the results for RP = 500 years in Figure 14 (which reflect the computed maximum settlement values) are shown in Figure 16 in terms of increment of reconsolidation settlement (Δs_v) in native soils against the thickness of the native soils (t_{NS}) considered in the calculation. The results imply that generally the native soils contribute to the overall vertical reconsolidation settlement by 0.5%-2% (on average approximately 1%) of their thickness. It is important to recognize that deeper native soils will cause global settlements, which are substantially less damaging for shallow foundations and buried infrastructure than effects of shallow liquefaction which typically results in substantial differential settlements.

CONCLUDING REMARKS

This paper discusses the assessment of liquefaction hazard of reclaimed land in central Wellington using CPT-based simplified liquefaction triggering analyses. The investigated sites primarily include the area of CentrePort and the southwestern end of the harbour in the city centre. To facilitate the interpretation of the liquefaction analyses, reclamation fills were first characterized using 58 CPTs performed as part of this study, supplemented with selected CPTs performed in previous investigations. Examination of CPT data allowed to establish simplified soil profiles for each reclamation area. Subsequently, soil layers were identified in each CPT profile using values of cone tip resistance, q_c, and normalized Soil Behaviour Type Index, I_c. The discretization of soil profiles into distinct layers allowed to compute 25th-50th-75th percentiles values of q_{c1Ncs} as a function of I_c. The main findings from the soil characterization are:

- The thickest reclamation fills (10-22 m thick) are encountered in the Thorndon reclamations, which largely consist of gravel-sand-silt mixtures, with some profiles also containing sand fills.
- Generally, the thicknesses of the fills decrease as the age of reclamation increases, and as one moves inland from the current revetment line (e.g., ranges from a few metres to 10-12 m thick in older reclamation zones).

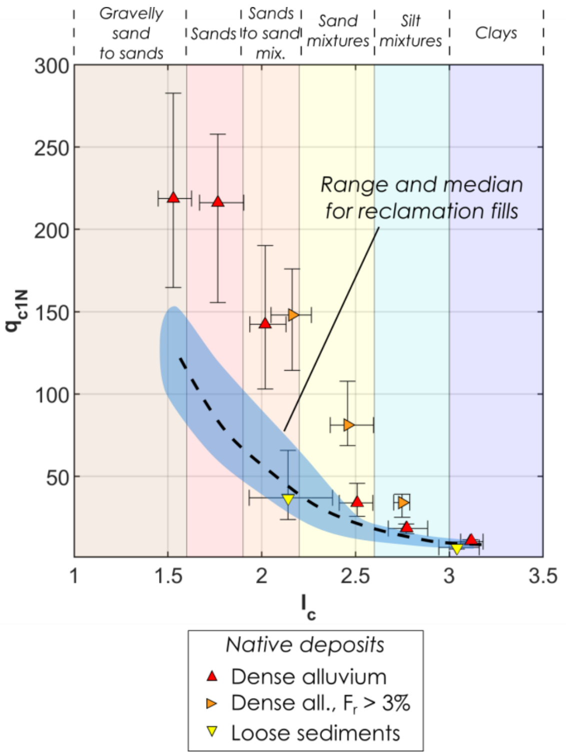

Figure 15: Characteristic q_{c1N} values of native soils with different I_c values (shown with symbols) in relation to respective values for reclamation fills (indicated by shaded zone); error bars indicate interquartile ranges of q_{c1N} and I_c. Soil descriptions on top of plot are based on CPT-based classification scheme of Robertson **[37].**

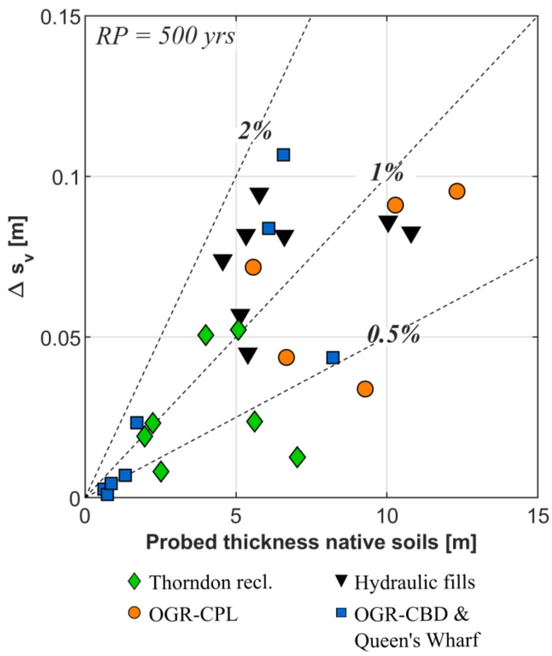

Figure 16: Increment in reconsolidation settlements at the ground surface due to liquefaction of native deposits at selected CPT locations for earthquake with RP = 500 yrs (P_L = 16% for loose native sediments, P_L = 50% for dense alluvium).

- The older reclamation areas have more vertically discontinuous profiles compared to the Thorndon reclamation, and are characterized by layers of gravelly and plastic silt-clay fill, reflecting the more segmented construction processes employed in the early stages of reclamation.
- The hydraulic fills contain three major soil units: plastic silt-clay fill, liquefiable sand-silt fill, and liquefiable gravel-sand-silt mixtures. The fills are generally 4-9 m thick.
- The Thorndon gravelly fills and the hydraulic fills have lowest q_{c1Ncs} values of 70-90, while the older gravelly fills have 10-15% higher q_{c1Ncs} values, all generally with $I_c > 1.6$. This reflects the loose density state of the fills due to the processes employed in their construction. The largest q_{c1Ncs} values (100-150) are only encountered for some denser fills associated with lower I_c values (i.e., as low as 1.5).

Simplified liquefaction analyses were performed to assess the liquefaction response of the reclamations for the two 2013 and 2016 earthquakes. This assessment was performed within the holistic framework of liquefaction assessment. Analyses were performed for a wide range of input *PGA* values, allowing for identification of *PGA* values where liquefaction triggers, liquefaction effects (damage) evolve, and when a maximum response is reached. The main findings from the analyses are:

- All fills generally trigger liquefaction at $PGA \approx 0.10$g, and the computed settlements increase quickly to their maximum value, which is attained at $PGA \approx 0.20$g. The attainment of the maximum response for PGA > 0.20g reflects a feature of the simplified liquefaction assessment in which the maximum volumetric strain (settlement) is capped once a certain seismic demand is exceeded. These observations are largely insensitive to changes in M_w.
- The simplified analyses therefore imply that, for the seismic demand of the 2016 Kaikōura earthquake, all liquefiable soils in the fills should have reached their maximum response in terms of liquefaction-induced settlement.
- There is generally good agreement between predicted and observed liquefaction-induced damage for the Kaikōura earthquake, except for Aotea Quay, where the magnitude of liquefaction-induced damage was overpredicted. The response is generally overpredicted in all fill types for the 2013 Cook Strait earthquake.
- Since the liquefiable soils have very similar q_{c1Ncs} values and reach the maximum settlement potential for the 2016 Kaikōura earthquake demand, the maximum settlement of the fills reflects the cumulative thickness of liquefiable soils. In other words, profiles with thicker liquefiable fill tend to also have the largest computed settlements, and vice versa.

The liquefaction performance of reclamations was also examined for different earthquake excitations (expressed as PGA-M_w pairs) based on the seismic hazard of Wellington. Results show that liquefaction in the reclamation fills is predicted to trigger in a *RP* = 25 years event, with maximum settlement in the fills attained at a *RP* = 50-100 years event. This implies that liquefaction-induced damage in the Wellington reclamations (as originally constructed) will occur frequently (i.e., with probability > 60%) within the design life of buildings and infrastructure.

Finally, some observations were also made on the contribution of underlying native soils to liquefaction-induced settlement of the reclamations. Most liquefiable native deposits contain layers with q_{c1N} values that are approximately twice as large as those of the reclamation fills. For higher *PGA* values producing the maximum response, settlement contributions of native liquefiable soils can be approximated to be (on average) 1% of their cumulative thickness.

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REFERENCES

1 Cubrinovski M and Dhakal R (2021). "Identification and mitigation of seismic hazards from inherited vulnerabilities". *17th World Conference on Earthquake Engineering*, 27 September-2 October 2021, Sendai, Japan.

2 Cubrinovski M, Bray JD, de la Torre C, Olsen MJ, Bradley BA, Chiaro G, Stocks E and Wotherspoon L (2017). "Liquefaction effects and associated damages observed at the Wellington CentrePort from the 2016 Kaikōura earthquake". *Bulletin of the New Zealand Society for Earthquake Engineering*, **50**(2): 152-173. https://doi.org/10.5459/bnzsee.50.2.152-173

3 USGS (1990). "*Effects of the Loma Prieta Earthquake on the Marina District, San Francisco, California*". Open File Report 90-253, Dept. of the Interior, U.S. Geological Survey, Menlo Park, CA, United States. . https://doi.org/10.3133/ofr90253

4 Seed RB, Dickenson SE and Idriss IM (1991). "Principal geotechnical aspects of the 1989 Loma Prieta earthquake". *Soils and Foundations*, **31**(1): 1-26. https://doi.org/10.3208/sandf1972.31.1

5 Akamoto H and Miyake M (1996). "Earthquake-induced settlement in Naruohama reclaimed land". *Soils and Foundations*, **36**(Supp): 161-167. https://doi.org/10.3208/sandf.36.Special_161

6 Hamada M, Isoyama R and Wakamatu K (1996). "Liquefaction-induced ground displacement and its related damage to lifeline facilities". *Soils and Foundations*, **36**(Supp): 81-97. https://doi.org/10.3208/sandf.36.Special_81

7 Kamon M, Wako T, Isemura K, Sawa K, Mimura M, Tateyama K and Kobayashi S (1996). "Geotechnical disasters on the waterfront". *Soils and Foundations*, **36**(Supp): 137-147. https://doi.org/10.3208/sandf.36.Special_137

8 Shibata T, Oka F and Ozawa Y (1996). "Characteristics of ground deformation due to liquefaction". *Soils and Foundations*, **36**(Supp): 65-79. https://doi.org/10.3208/sandf.36.Special_65

9 Ishihara K, Yoshida K and Kato M (1997). "Characteristics of lateral spreading in liquefied deposits during the 1995 Hanshin-Awaji earthquake". *Journal of Earthquake Engineering*, **1**(1): 23-55. https://doi.org/10.1080/13632469708962360

10 Tokimatsu K and Asaka Y (1998). "Effects of liquefaction-induced ground displacements on pile performance in the 1995 Hyogoken-Nambu earthquake". *Soils and Foundations*, **38**(Supp): 163-177. https://doi.org/10.3208/sandf.38.Special_163

11 Towhata I, Taguchi Y, Hayashida T, Goto S, Shintaku Y, Hamada Y and Aoyama S (2017). "Liquefaction perspective of soil ageing". *Géotechnique*, **67**(6): 467–478. http://dx.doi.org/10.1680/jgeot.15.P.046

12 Orense RP, Mirjafari Y, Asadi S, Naghibi M, Chen X, Altaf O and Asadi B (2017). "Ground performance in Wellington waterfront area following the 2016 Kaikōura earthquake". *Bulletin of the New Zealand Society for Earthquake Engineering*, **50**(2): 142-151. https://doi.org/10.5459/bnzsee.50.2.142-151

13 Cubrinovski M, Bray JD, de la Torre C, Olsen M, Bradley B, Chiaro G, Stocks E, Wotherspoon L and Krall T (2018). "Liquefaction-induced damage and CPT characterization of the reclamations at CentrePort, Wellington". *Bulletin of the Seismological Society of America*, **108**(3B): 1695-1708. https://doi.org/10.1785/0120170246

14 Dhakal R, Cubrinovski M and Bray JD (2020). "Geotechnical characterization and liquefaction evaluation of gravelly reclamations and hydraulic fills (Port of Wellington, New Zealand)". *Soils and Foundations*, **60**: 1507-1531. https://doi.org/10.1016/j.sandf.2020.10.001

15 Dhakal R, Cubrinovski M, Bray JD and de la Torre C (2020). "Liquefaction assessment of reclaimed land at CentrePort, Wellington". *Bulletin of the New Zealand Society for Earthquake Engineering*, **53**(1): 1-12. https://doi.org/10.5459/bnzsee.53.1.1-12

16 Kingsbury PA and Hastie WJ (1993). "Liquefaction Hazard Wellington. "*Seismic Hazard Map Series: Liquefaction Hazard Map Sheet 1 Wellington (1st ed.) 1:50000*", Map + notes to accompany. Pub. No. WRC/PP-T-93/72, Wellington Regional Council, Wellington, New Zealand.

17 Brabhaharan P, Hastie WJ and Kingsbury PA (1994). "Liquefaction hazard mapping techniques developed for the Wellington region, New Zealand". *Annual NZNSEE Conference*, 18-20 March 1994, Wairakei, New Zealand.

18 Dellow GD, Perrin ND and Ries WF (2018). *"Liquefaction Hazard in the Wellington Region"*. GNS Science report 2014/16, Lower Hutt, New Zealand, 71 pp. https://doi:10.21420/G28S8J

19 Griffin AG, Pradel GJ, Abbott ER and Hill MP (2020). "*Liquefaction Susceptibility Verification for Wellington City Council*". GNS Science Consultancy Report 2020/109, Lower Hutt, New Zealand, 67 pp.

20 Wellington City Council (2013). Wellington Thomas Ward Maps. Accessed 31 January 2021. https://data-wcc.opendata.arcgis.com/maps/wellington-thomas-ward-maps/explore

21 Retrolens Historic Image Resource. www.retroloens.co.nz

22 Hutchison AJH (1973). "Reclamations in Wellington Harbour with special reference to recent developments". *New Zealand Engineering*, **28**(8): 217-224.

23 Beca Carter Hollings and Ferner Ltd. (1988). "*Underground Car Parking. Stage 2 Site Investigation Report*". Report prepared for Lambton Harbour Development Project Board, Job No: 2802007, Wellington, New Zealand.

24 Palmer SJ (1995). "Wellington regional stadium site assessment". *Proceedings of the 2nd Australia-New Zealand Young Geotechnical Professionals Conference*. 29 November-2 December 1995, Auckland, New Zealand, 151-156.

25 Murashev A and Palmer S (1998). "Geotechnical issues associated with development on Wellington's waterfront". *IPENZ Transactions*, **25**(1/CE): 38-46.

26 Tonkin and Taylor (2006). "*Harbour Quays Development Factual Geotechnical Report*". T+T Ref: 83725.004 Rev. 2. Report prepared for CentrePort Wellington Limited, Wellington, New Zealand.

27 Tonkin and Taylor (2012). "*Thorndon Container Wharf Seismic Assessment Geotechnical Factual Report*". T+T Ref: 85369.001. Report prepared for CentrePort Wellington Limited, Wellington, New Zealand.

28 Cubrinovski M (2019). "Some important considerations in the engineering assessment of soil liquefaction". *2019 NZGS Geomechanics Lecture, Proceedings of 13th Australia New Zealand Conference on Geomechanics*, 1-3 April 2019, Perth, Australia.

29 Dellow GD and Perrin ND (1991). *"Wellington Railway Yard Assessment of Liquefaction Potential During Earthquake Shaking"*. DSIR Contract Report 1991/4, Lower Hutt, New Zealand.

30 McMinn J, Brabhaharan P and Jennings DN (1993). *"Liquefaction Hazard Study Wellington Region: A Review of Historical Records of Liquefaction*". Works Consultancy Services Limited Contract Report prepared for Wellington Regional Council, Wellington, New Zealand.

31 Grapes R, Downes R (1997). "Sources, ground motion and structural response characteristics in Wellington of the 2013 Cook Strait earthquakes". *Bulletin of the New Zealand Society for Earthquake Engineering*, **30**(4): 271-368. https://doi.org/10.5459/bnzsee.30.4.271-368

32 Tonkin and Taylor (2013). "*Walkover Inspection of Seawalls and Reclamations Following the 21 July 2013 Cook Strait Earthquake*". T+T Ref: 85670.0010. Report prepared for Wellington Waterfront Limited, Wellington, New Zealand, 4 pp. + appendix.

33 Tonkin and Taylor (2014). "*Damage Report – Earthquake Events Dated 21 July and 16 August 2013*". T+T Ref: 85726/Rev. 0. Report prepared for CentrePort Limited, Wellington, New Zealand.

34 Van Dissen R, McSaveney M, Townsend D, Hancox G, Little TA, Ries W, Perrin N, Archibald G, Dellow G, Massey C and Misra S (2013). "Landslides and liquefaction generated by the Cook Strait and Lake Grassmere earthquakes: a reconnaissance report". *Bulletin of the New Zealand Society for Earthquake Engineering*, **46**(4): 196-200. https://doi.org/10.5459/bnzsee.46.4.196-200

35 Holden C, Kaiser A, Van Dissen R and Jury R (2013). "Sources, ground motion and structural response characteristics in Wellington of the 2013 Cook Strait earthquakes". *Bulletin of the New Zealand Society for Earthquake Engineering*, **46**(4): 188-195. https://doi.org/10.5459/bnzsee.46.4.188-195

36 Semmens S, Perrin ND and Dellow GD (2010). "*It's Our Fault – Geological and Geotechnical Characterization of the Wellington Central Business District*". GNS Science Consultancy Report 2010/176, Lower Hutt, New Zealand, 52 pp.

37 Robertson PK (1990). "Soil classification using the cone penetration test". *Canadian Geotechnical Journal*, **27**(1): 151-158. https://doi.org/10.1139/t90-014

38 Robertson PK (2009). "Interpretation of cone penetration test – a unified approach". *Canadian Geotechnical Journal*, **46**(11): 1337-1355. https://doi.org/10.1193/031813EQS070M

39 Dhakal R (2022). "*Liquefaction Assessment Methodologies for Reclaimed Land*". PhD Dissertation, University of Canterbury, Christchurch, New Zealand.

40 Boulanger RW and Idriss IM (2016). "CPT-based liquefaction triggering procedure". *Journal of Geotechnical and Geoenvironmental Engineering*, **142**(2): 04015065. https://doi.org/10.1061/(ASCE)GT.1943-5606.0001388

41 Dhakal R and Cubrinovski M (2024). "Liquefaction response of gravel-sand-silt reclamations from 1D effective stress analysis". *8th International Conference on Earthquake Geotechnical Engineering*, Osaka, Japan, 7-10 May.

42 Idriss IM and Boulanger RW (2008). "*Soil Liquefaction during Earthquakes*". ISBN #978-1-932884-36-4, Earthquake Engineering Research Institute, Oakland, CA, United States.

43 Cubrinovski M and Ntritsos N (2023). "8th Ishihara lecture: Holistic evaluation of liquefaction response". *Soil Dynamics and Earthquake Engineering*, **168**: 107777. https://doi.org/10.1016/j.soildyn.2023.107777

44 Cubrinovski M, Rhodes A, Ntritsos N and van Ballegooy S (2019). "System response of liquefiable deposits". *Soil Dynamics and Earthquake Engineering*, **124**: 219-229. https://doi.org/10.1016/j.soildyn.2018.05.013

45 Dhakal R, Cubrinovski M and Bray JD (2022). "Evaluating the applicability of conventional CPT-based liquefaction assessment procedures to reclaimed gravelly soils". *Soil Dynamics and Earthquake Engineering*, **155**: 107176. https://doi.org/10.1016/j.soildyn.2022.107176

46 Boulanger RW and Idriss IM (2015). "Magnitude scaling factors in liquefaction triggering procedure". *Soil Dynamics and Earthquake Engineering*, **79**: 296-303. http://dx.doi.org/10.1016/j.soildyn.2015.01.004

47 Langridge RM, Ries WF, Litchfield NJ, Villamor P, Van Dissen RJ, Barrell DJA, Rattenbury MS, Heron DW, Haubrock S, Townsend DB, Lee JM, Berryman KR, Nicol A, Cox SC and Stirling MW (2016). "The New Zealand Active Faults Database". *New Zealand Journal of Geology and Geophysics* **59**(1): 86-96. https://doi.org/10.1080/00288306.2015.1112818

48 Kaiser AE, Hill MP, Wotherspoon L, Bourguignon S, Bruce ZR, Morgenstern R and Giallini S (2019). "*Updated 3D Basin Model and NZS 1170.5 Subsoil Class and Site Period Maps for the Wellington CBD: Project 2017-GNS-03-NHRP*". GNS Science Consultancy Report 2019/01, Lower Hutt, New Zealand, 48 pp. + appendix. https://www.gns.cri.nz/static/download/NHRP/NHRP-Kaikōura-Kaiser.pdf

49 Bradley BA, Wotherspoon LM and Kaiser AE (2017). "Ground motion and site effect observations in the Wellington region from the 2016 M_w7.8 Kaikōura, New Zealand earthquake". *Bulletin of the New Zealand Society for Earthquake Engineering*, **50**(2): 94-105. https://doi.org/10.5459/bnzsee.50.2.94-105

50 Bradley BA, Wotherspoon LM, Kaiser AE, Cox BR and Jeong S (2018). "Influence of site effects on observed ground motions in the Wellington Region from the M_w 7.8 Kaikōura, New Zealand, earthquake". *Bulletin of the Seismological Society of America*, **108**(3B): 1722–1735. https://doi.org/10.1785/0120170286

51 Bradley BA, Cubrinovski M and Wentz F (2022). "Probabilistic seismic hazard analysis of peak ground acceleration for major regional New Zealand locations". *Bulletin of the New Zealand Society for Earthquake Engineering*, **55**(1): 15-24. https://doi.org/10.5459/bnzsee.55.1.15-24

52 Cubrinovski M, Bradley BA, Wentz F and Balachandra A (2022). "Re-evaluation of New Zealand seismic hazard for geotechnical assessment and design". *Bulletin of the New Zealand Society for Earthquake Engineering*, **55**(1): 1-14. https://doi.org/10.5459/bnzsee.55.1.1-14

53 Kaiser AE, Manea E, Hill M, Wotherspoon L, Lee R, de la Torre C, Bora S, Stolte A, Bradley B and Gerstenberger M (2022). "*Revision of the National Seismic Hazard Model for New Zealand: Overview of Site/Basin Directions, Including a Case Study of the Wellington Basin*". GNS Science Report 2022/56, Lower Hutt, New Zealand, 46 pp. http://doi.org/10.21420/3XXY-T303

54 Gerstenberger MC, Bora S, Bradley BA, DiCaprio C, Van Dissen RJ, Atkinson GM, Chamberlain C, Christophersen A, Clark KJ, Coffey GL, de la Torre C, Ellis SM, Fraser J, Graham K and Griffin J (2022). "*New Zealand National Seismic Hazard Model 2022 revision: Model, Hazard and Process Overview*". GNS Science report 2022/57, Lower Hutt, New Zealand, 106 pp. https://doi:10.21420/TB83-7X19

55 Bray JD and Macedo J (2017). "6th Ishihara lecture: simplified procedure for estimating liquefaction-induced building settlement". *Soil Dynamics and Earthquake Engineering*, **102**: 215-231. http://dx.doi.org/10.1016/j.soildyn.2017.08.026

56 Perrin ND and Campbell HJ (1992). "Subsurface quaternary sediments of Wellington City". *Recent Advances in Wellington Earth Science – Extended Abstracts*, 8-9 July 1992. Editor: Begg JG, NZGS report G166:85-91, Institute of Geological and Nuclear Sciences, Wellington, New Zealand.