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Site characterisation and liquefaction potential of Blenheim gravelly sandy deposits

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Principal Investigator Associate Professor Gabriele Chiaro
Department of Civil and Natural Resources Engineering,
University of Canterbury, Christchurch, New Zealand

Research team

- Dr Mark Stringer, Senior Lecturer, University of Canterbury (Associate Investigator)
- Dr Liam Wotherspoon, Associate Professor, University of Auckland (Associate Investigator)
- Dr Andrew Stolte, Lecturer, University of Auckland (Research Assistant: MAWS testing)
- Mr Abilash Pokhrel, PhD Student, University of Canterbury (Research Assistant: Field and laboratory testing)

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Gravelly soils; liquefaction; field investigations; triggering analysis; dynamic penetration test; Blenheim

Summary

Gravelly soils are a very common feature of the New Zealand's geological setting. From a geotechnical viewpoint, however, such soils are considered problematic being difficult to characterise and because their liquefaction potential is largely unknown.

Contrary to the general belief that gravelly soils do not liquefy, case histories from at least 27 earthquakes worldwide have indicated that liquefaction can actually occur in gravelly soils (either natural deposits and manmade reclamations) causing severe damage to land and civil infrastructures. Three of such case histories are from New Zealand and include the 1929

M_w7.6 Murchison earthquake, the 2010 M_w7.1 Darfield earthquake, and the 2016 M_w7.8 Kaikoura earthquake. The latter case history refers not only to the well-known Wellington CentrePort's gravelly reclamations; in fact, following the 2016 M_w7.8 Kaikoura earthquake, surface liquefaction features were observed also in Blenheim, at sites underlined by loose alluvial sandy gravel deposits at shallow depths.

Worldwide, due to a deficiency of well-documented case histories and very limited availability of field assessment data, the current practice of evaluating the liquefaction resistance of gravelly soils relies on the assumption that liquefiable gravels behave similarly to sands. However, existing empirical correlations based on sands may not work for the characterization of gravelly soils and could be misleading engineering assessment.

In gravelly soils, the standard penetration test (SPT) and the cone penetration test (CPT) are not generally useful because the interference from large-size particles. That is, because of the large particles, the penetration resistance increases and may reach refusal even when the soil is not particularly dense. This limitation often makes it very difficult to obtain a consistent and reliable correlation between SPT or CPT penetration resistance and basic gravelly soil properties. To overcome such difficulties, the Becker penetration test (BPT) has been proposed as a field test to evaluate the liquefaction resistance of gravelly soils. Yet, the high mobilization and testing costs, uncertainty in measuring the BPT resistance and limited worldwide availability, have restricted the use BPT to high-cost investigations. Thus, as a promising cost-effective testing procedure for gravelly soils, the dynamic cone penetration test (DPT) has been introduced and optimised over the last decade or so. DPT can be performed by using commonly available drill rigs with a standard SPT hammer, and reliable DPT-based probabilistic liquefaction triggering curves have been recently developed based on data points collected from 137 sites (with different geological setting) for 10 earthquakes where liquefaction did and did not occur. It is evident that research aimed at developing proper investigation methods for characterising gravelly soils and studying the liquefaction potential of gravelly soils (including developing triggering analysing techniques) is critical not only for the New Zealand context, but worldwide.

Given the aforementioned background, the main objectives of this EQC project were (1) to identify and use reliable and cost-effective field techniques for properly characterise, from a geotechnical viewpoint, typical New Zealand alluvial gravelly soil deposits, and (2) assess their liquefaction potential using adequate liquefaction triggering procedures for gravelly soils. To this scope, three relatively close sites where gravelly soils liquefaction manifestation (i.e. soil ejecta and settlement) was observed or not in Blenheim during the 2016 M_w7.8 Kaikoura earthquake, the 2013 M_w6.6 Cook Strait/Seddon earthquake and the 2013 M_w6.6 Lake Grassmere earthquake were carefully selected and characterised. Detailed soil profiles and soil gradation characteristics were obtained from soil samples retrieved by borehole core drilling. DPT investigations were carried out by using an automatic free-fall 63.5-kg SPT hammer, and the hammer energy delivery was measured by means of an instrumented rod section for the purpose of calibration against the conventional Chinese DPT method. To supplement this dataset, non-invasive V_s measurements were obtained using the multi-channel analyses of surface wave (MASW) method. Hence, DPT and shear velocity (V_s) profiles were developed.

DPT could be driven through the alluvial gravelly profiles (irrespective of the gravel content and maximum gravel particle size) just using the standard 63.5-kg SPT hammer (with an energy efficiency of 85.6%), which is commonly available in New Zealand. DPT correctly predicted liquefaction at sites where liquefaction manifestation was observed or not for the three earthquakes. However, for the Kaikoura earthquake, it also predicted liquefaction in a sandy gravel deposit (with a 3m-thick non-liquefiable gravel capping layer) for which liquefaction features were not observed, suggesting that this false-positive prediction could be the result of the system response of the profile that may have impeded sand ejecta to reach the surface. On the other hand, the non-invasive MASW-based V_S measurements were found to provide less detailed information as compared to DPT, and V_S-based liquefaction triggering analyses were unable to distinguish between liquefaction and no-liquefaction observed behaviours at these sites.

It is important also to mention that CPT were also attempted, but generally performed poorly in the gravelly soils, reaching refusal at shallow depth, even when the soil profile was not particularly dense. This made it not possible to comprehensively characterise the investigated soil deposits and evaluate their liquefaction potential using CPT-based procedures.

This project has confirmed that SPT-hammer-driven DPT represents a reliable and cost-effective technique for measuring the penetration resistance of typical New Zealand alluvial gravelly soils for liquefaction assessment. As such, it should be regarded as a useful field test technique whenever the liquefaction resistance of gravelly soils is of concern. Moreover, it was established that the most recently-developed DPT-based liquefaction triggering procedure by Rollins et al. (2021) for gravelly soils were successful in predicting liquefaction or not for the tested gravelly sites in Blenheim.

The outcomes of this study not only well complement the existing international case history database of gravelly soil liquefaction, but more importantly provide a reliable technique (now available in New Zealand) and analysis for characterising the liquefaction resistance of gravelly soils that are instrumental for defining the impact and consequences of liquefaction of gravely soils on land and infrastructure during expected severe long-duration earthquakes (e.g. Alpine Fault Earthquake) for many critical infrastructure assets across New Zealand.

Introduction

Worldwide, one of the issues that has been constantly brought to attention by the engineering community is the lack of guidance for the geotechnical characterization and evaluation of liquefaction potential of gravelly soils (i.e., gravelly sands, sandy gravels, and uniform gravels). Such soils are often referred to as '*problematic*' because their behavior is poorly understood.

Liquefaction is known to have occurred in gravelly soils (either natural or reclaimed deposits) in a significant number of earthquakes (Table 1), of which at least three case histories are from New Zealand (1929 M_w7.6 Murchison earthquake; 2010 M_w7.1 Darfield earthquake;

and 2016 M_w7.8 Kaikoura earthquake) causing severe damage to land and civil infrastructures. As a result, geotechnical engineers have been frequently called to assess the potential for liquefaction in gravelly soils. Yet, the current practice of evaluating the liquefaction resistance of gravelly soils relies on the assumption that liquefiable gravels behave similarly to sands. However, existing empirical correlations based on sands may not work for the characterisation of gravelly soils and could be misleading the engineering assessment. Therefore, research in studying the liquefaction mechanism and developing proper analysing techniques for gravelly soils is critical to characterise the hazard presented by these materials.

Table 1 – *Case histories of liquefaction in gravelly soil deposits*

Earthquake	Year	M_W	Reference
Mino-Owari, Japan	1891	7.9	Tokimastu and Yoshimi (1983)
San Francisco, USA	1906	8.3	Youd and Hoose (1978)
Messina, Italy	1908	7.1	Baratta (1910)
Murchison, New Zealand	1929	7.6	Berrill et al. (1988)
Fukui, Japan	1948	7.1	Ishihara (1985)
Valdez, Alaska, USA	1964	8.4	Coulter and Migliaccio (1966)
Haicheng, China	1975	7.3	Wang (1984)
Friuli, Italy	1976	6.5	Sirovich (1996); Rollins et al., (2020)
Tangshan, China	1976	7.8	Wang (1984)
Miyagiken-Oki, Japan	1978	7.4	Tokimastu and Yoshimi (1983)
Montenegro	1979	6.9	Kociu (2004)
Borah Peak, Idaho, USA	1983	7.3	Youd et al. (1985); Andrus (1994)
Spitak, Armenia	1988	6.8	Yegian et al. (1994)
Limon, Costa Riga	1991	7.7	Franke and Rollins (2017)
Roermond, Netherlands	1992	5.8	Maruenbrecher et al. (1995)
Hokkaido, Japan	1993	7.8	Kokusho et al. (1995)
Kobe, Japan	1995	6.9	Kokusho and Yoshida (1997)
Kocaeli, Turkey	1999	7.6	Bardet et al. (2000)
Chi-Chi, Taiwan	1999	7.7	Lin and Chang (2002)
Wenchaun, China	2008	7.9	Cao et al. (2013)
Darfield, New Zealand	2010	7.1	Cubrinovski et al. (2010)
Tohoku, Japan	2011	9.0	Tatsuoka et al. (2017)
Iquie, Chile	2014	8.2	Rollins et al. (2014)
Cephalonia, Greece	2014	6.1	Nikolau et al. (2014)
Muisne, Ecuador	2016	7.8	Lopez et al. (2018)
Kaikoura, New Zealand	2016	7.8	Cubrinovski et al. (2017; 2018)
Durres, Albania	2019	6.4	Pavlides et al. (2020)

Many loosely deposited gravelly-reach alluvial soil deposits can be found in New Zealand. Such soil deposits are likely susceptible of liquefaction during strong earthquakes, but to date, they have been poorly characterised, and their liquefaction potential is essentially unknown. For instance, on 14 November 2016, following the M_w7.8 Kaikoura earthquake, liquefaction was a key feature of the seismic performance of alluvial gravelly deposits in Blenheim (Stringer et al., 2017). Similarly, alluvial gravelly deposits liquefied during the 2010 M_w7.1 Darfield earthquake (Cubrinovski et al., 2010) and the 1929 M_w7.6 Murchison earthquake (Berrill et al., 1988).

In Blenheim, at Lawnsdone Park, in proximity of the city centre, ejecta material deposited at the ground surface consisted typically of clean sands. Yet, based on available geological, geomorphological and borehole drilling information, it appears that the ground profile at this location consists mainly of gravelly soils, indicating that the liquefied soil layers could have actually been within a gravelly deposit. It is also noted that no manifestation of liquefaction was observed at this site following the 2013 M_w6.6 Lake Grassmere and the 2013 M_w6.6 Cook Strait/Seddon earthquakes.

Based on the aforementioned background, the main objective of this EQC project was (1) to characterise from a geotechnical viewpoint the Lansdowne Park liquefied site – and selected nearby sites that liquefied or not – using reliable field techniques for gravelly soils and (2) evaluate their liquefaction potential. Consequently, three sites (namely S1, S2 and S3) were selected in Blenheim, where liquefaction features (i.e., soil ejecta and settlement) were observed or not during the 2016 M_w7.8 Kaikoura earthquake, the 2013 M_w6.6 Lake Grassmere earthquake and the 2013 M_w6.6 Cook Strait/Seddon earthquake.

The selected sites are relatively close to each other, and even though the soil profile characteristics could be similar, the liquefaction manifestation was quite different. Specifically, site S1 liquefied during the 2016 earthquake (M_w7.8, a_{max} = 0.227 g) but did not liquefy during the 2016 earthquakes (M_w6.6, a_{max} = 0.119 g; and M_w6.6, 0.068 g). Site S2 did not liquefy in 2013, but a localized 10-cm surface settlement suggested that it could have liquefied in 2016, although manifestation of soil ejecta was not observed at the ground surface. At site S3 no manifestation of liquefaction was observed during the three earthquakes. Each site was characterised by performing dynamic cone penetration tests (DPT) and shear wave velocity (V_s) measurements (MASW method). Soil samples for laboratory particle size analyses were obtained from sites S1 and S3 by borehole core drilling. It is important to mention that DPT were carried out by using a drill rig equipped with an automatic standard 63.5-kg SPT hummer and the hammer energy delivery was carefully measured for energy efficiency calibration/correction against the conventional Chinese DPT method. To assess the liquefaction potential of each site, the newly-developed DPT-based liquefaction triggering procedure developed by Rollins et al. (2021) for gravelly soils was employed. An attempt was also made to use currently available Vs-based liquefaction triggering procedures for sands and assess their accuracy for the investigated gravelly soil deposits.

An overview of field-testing methods for characterising gravelly soils

To date, laboratory tests are still not considered as a reliable technique for evaluating the liquefaction potential of gravelly soils primarily because the difficulties in extracting undisturbed samples from gravelly soil deposits. The use of freezing sampling techniques before extraction could improve the sample quality, but the costs are essentially prohibitive for routine projects. Moreover, even though undisturbed samples cold be collected, the change of stress conditions between field and laboratory often would limit the usefulness of the laboratory test results (Rollins et al., 2020).

In gravelly soils, the standard penetration test (SPT) and the cone penetration test (CPT) are not generally useful because the interference from large-size particles (Rollins et al., 2020; 2021). Some researchers have proposed gravel correction procedures or short interval sampling (e.g., Rhinehart et al., 2016), but such approaches are often difficult to apply. That is, because the large particles, the penetration resistance increases and may reach refusal even when the soil is not particularly dense (Dhakal et al., 2019 and 2020). This limitation often makes it very difficult to obtain a consistent and reliable correlation between SPT or CPT penetration resistance and basic gravelly soil properties.

In North American practice, the Becker penetration test (BPT) has become a primary field test to evaluate the liquefaction resistance of gravelly soils. It consists of hammering a close-end 168-mm-diameter, 3-m long double walled casing into the ground so that the penetration resistance is much less affected by particle size. Its penetration resistance is defined as the number of blow to drive the casing through a depth interval of 30 cm. Major problems in using BPT for liquefaction assessment include the high mobilization and testing costs, uncertainty in measuring the BPT resistance due to friction between the driven casing and the surrounding soil, and uncertainties with respect to correlations with sand behavior – derived by (Harder and Seed, 1986) from limited number of parallel BPT and SPT tests (Sy, 1997; Cao et al., 2013; Rollins et al., 2020). Because such limitation, the use BPT has been limited to high-cost investigations, such as earth dams. Furthermore, it is simply not available in most part of the world.

The shear wave velocity (V_s) has also been used as a means to evaluate the liquefaction resistance of sands and gravels liquefaction. Nevertheless, a number of studies have indicated that the liquefaction triggering curves maybe higher for gravels than for sands (Cao et al., 2013; Chang, 2016). Moreover, Menq (2003) and Stokoe (2015) have shown that V_s in higher in gravel than in sands at the same relative densities, due potentially to different soil fabric and structure effects.

Development and optimisation of the dynamic penetration test (DPT) for gravelly soils

As a promising alternative for gravelly soils, the penetration resistance derived from dynamic cone penetration tests (DPT), developed in China, was initially correlated to liquefaction resistance based on field performance data of 19 sites from the M_w7.9 Wenchuan earthquake (Cao et al., 2013). More recently, 137 data points from 10 earthquakes and 7 countries have been used to develop refined probabilistic liquefaction resistance curves (Rollins et al., 2021).

The Chinese version of the DPT consists of a 74-mm-diameter cone tip continuously driven by a 120-kg hammer with a free-fall of 100 mm, using a 60-mm drill rod to reduce friction (Fig. 1). The DPT blow count reduction, N_{120}, represents the number of hammer blows to drive the penetrometer 30 cm into the ground with a 120-kg hammer. Blow counts are typically reported every 10 cm but multiplied by 3 to get the equivalent N_{120} values. Cao et al. (2013) reported that the DPT penetration resistance was able to discriminate liquefaction from no-liquefaction at some sites with maximum particle size of 70 mm, despite the low diameter-to-particle-size ratio. Yet, at 74-mm, the DPT dimeter is 50% larger than that of SPT and 110% larger than a standard 10-cm^2 CPT, and its penetration resistance would be less affected by diameter-to-particle-size ratio as compared to that of SPT and CPT. The effect of gravel size and percentage on the DPT penetration resistance, however, must be studied more comprehensively in future.

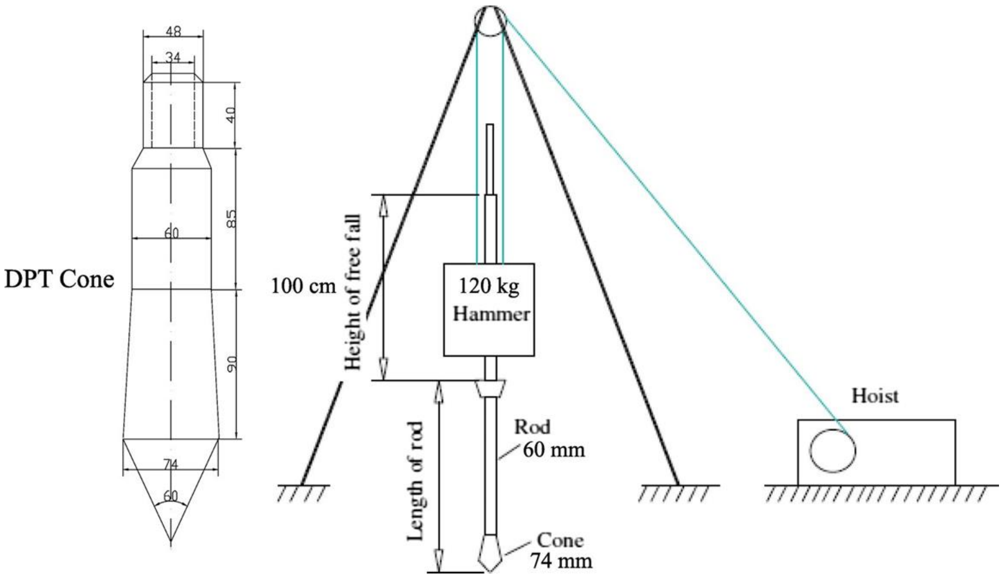


Fig. 1 – *Schematic illustration of the Chinese dynamic penetration test apparatus (adopted from Cao et al., 2013).*

With the main objectives of providing additional data points defining the liquefaction resistance of gravelly soils as a function of the DPT blow count, Rollins et al. (2020) carried out DPT at several sites in Idaho (USA) and Friuli (Italy) where gravels have and have not liquefied during major past earthquakes. Moreover, to optimize the use of DPT for use in practice, the DPT were performed using in parallel both the Chinese hammer energy and the energy delivered by a SPT hammer (with appropriate energy corrections). It was reported that the DPT could generally be driven through profiles of sandy gravel alluvium with 40 to 60% gravel content using only the conventional SPT hammer energy despite the larger particle sizes. Moreover, liquefaction-triggering correlations based on DPT N'_{120} values (i.e., N_{120} values corrected for the effects of overburden pressure) correctly identified sites where liquefaction manifestation was observed, apart from a soil profile with highly interbedded silt and silty sandy gravel layers that produced no surface evidence despite low blow count. This is likely the result of the system response of the profile, which inhibited the eruption of ejecta as described by Cubranovski et al. (2018b). Such additional studies confirmed that DPT test could provide an important new procedure for characterizing gravelly soils and fill a gap in the present geotechnical practice between SPT/CPT and BPT testing, but additional field testing is still required.

DPT-based probabilistic liquefaction resistance curves

Following the 2008 M_w7.9 Wenchuan earthquake in China, 47 DPT soundings were carried out at 19 sites where liquefaction manifestation was observed (hereafter referred to as "liquefied sites") and 28 nearby sites where liquefaction manifestation was not observed (hereafter referred to as "non-liquefied sites"). Each of the sites consisted of 2-4 m of clayey soils (capping layer) on top of thick gravel deposits. The looser upper layers within the gravel deposits liquefied during the earthquake. Boreholes were drilled about 2 m away from the DPT soundings to obtain continuous samples that otherwise could not be obtained with DPT.

In gravelly deposits, layers with lower DPT resistance were identified as the most liquefiable or critical liquefaction zones. For the 19 liquefied sites the DPT penetration resistance values were generally lower than those obtained for the remaining 28 non- liquefied deposits. Thus, DPT liquefaction resistance curves were proposed (Fig. 2) as a reliable identifier of liquefiable layers (Cao et al., 2013).

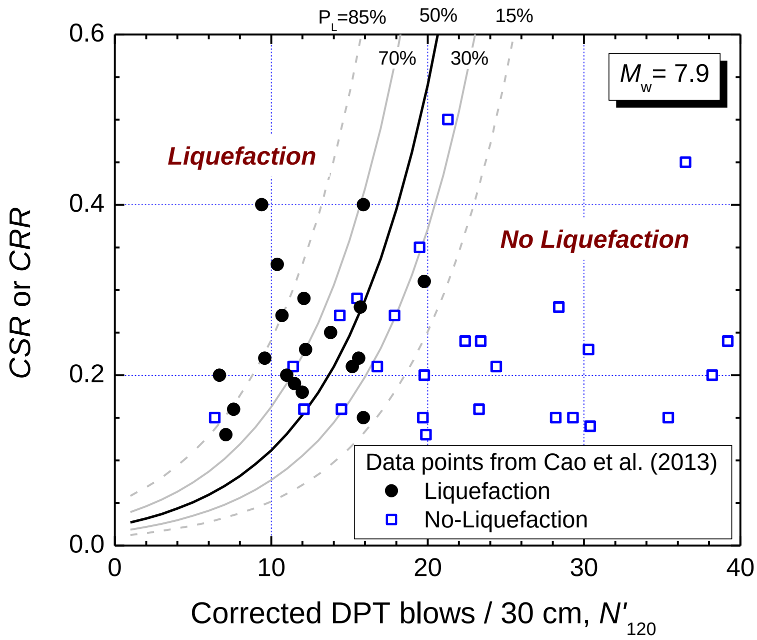


Fig. 2 – *Cao et al. (2013) DPT-based probabilistic liquefaction triggering curves for gravelly soils.*

Yet, the Cao et al. (2013) triggering curves were essentially developed using a single event of M_w7.9 without incorporating any correction for the seismic demand by using a magnitude scaling factor (*MSF*). Thus, the applicability of such curves become questionable for evaluating the liquefaction potential of gravelly soils for other seismic events of different magnitude. To address this issue, Rollins et al. (2021) collected and added more data points in the DPT database for different earthquake magnitudes and different geology settings to develop an improved DPT-based liquefaction triggering procedure. Eventually, an expanded dataset of 137 sites for 10 earthquakes where liquefaction did and did not occur was collected. These data points, including the 47 DPT soundings from Cao et al. (2013) have then been used to develop an enhanced set of probabilistic triggering curves for gravelly soils (Fig. 3) using logistics regression analysis. The new curves include the earthquake moment magnitude as an independent variable which led the development of a new DPT-based *MSF* model exclusively for gravelly soils.

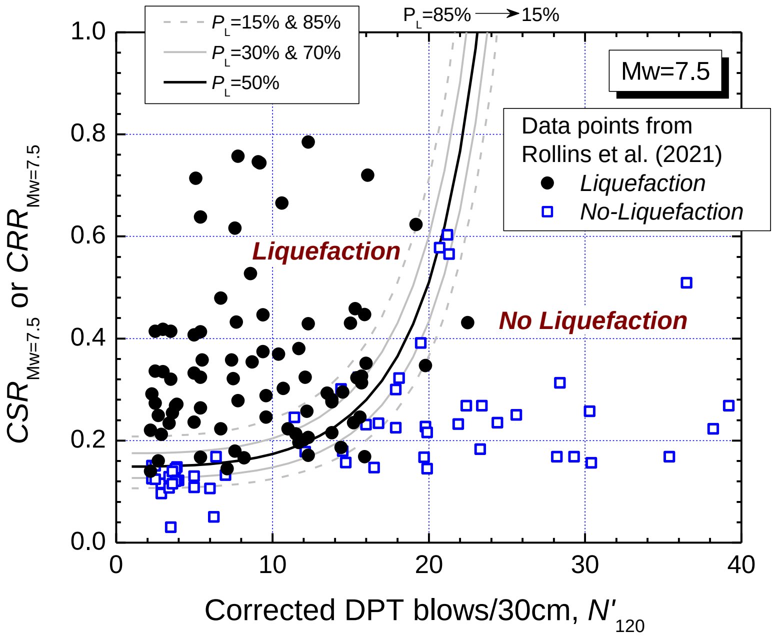


Fig. 3 – *Rollins et al. (2021) DPT-based probabilistic liquefaction triggering curves for gravelly soils.*

Soil liquefaction in Blenheim induced by the 2016 M_W7.8 Kaikoura earthquake

The township of Blenheim is located on the north-eastern corner of the South Island, New Zealand, approx. 5 km from the coast within the Wairau Plain. Typically, the soil deposits are interlayered silts, sands and gravels of alluvial and colluvial Late Quaternary origin (Fig. 1).

Two ground motion stations are present within the Wairau Valley, as shown on Fig. 5. Table 2 provides a summary of the recorded peak ground accelerations (PGAs) during recent strong earthquakes at these two stations. At the time of the 2016 Kaikoura earthquake, at the MCGS station within Blenheim urban area (Site Class D according to NZS1170.5:2004), the geometric mean of the PGA of the two horizontal components of motion was 0.227 g. Further north-west at the BWRS strong motion station, situated on rock (Site Class B) at the base of the hills, the geometric mean PGA was 0.126 g.

Localised liquefaction and associated lateral spreading occurred during the Kaikoura earthquake proximal to the Ōpaoa River within Blenheim as outlined in Fig. 5. Liquefaction and lateral spreading related damage was confined to the inner-banks of meander bends of the rivers or was associated paleo-channels; no damage was observed on the outer-banks of the meander bends. Localised liquefaction also occurred adjacent to the Taylor River within central Blenheim, in an area that was formerly within the river channel prior to modification and straightening in 1969 which subsequently reduced flow levels (Marlborough District Council, 2017).

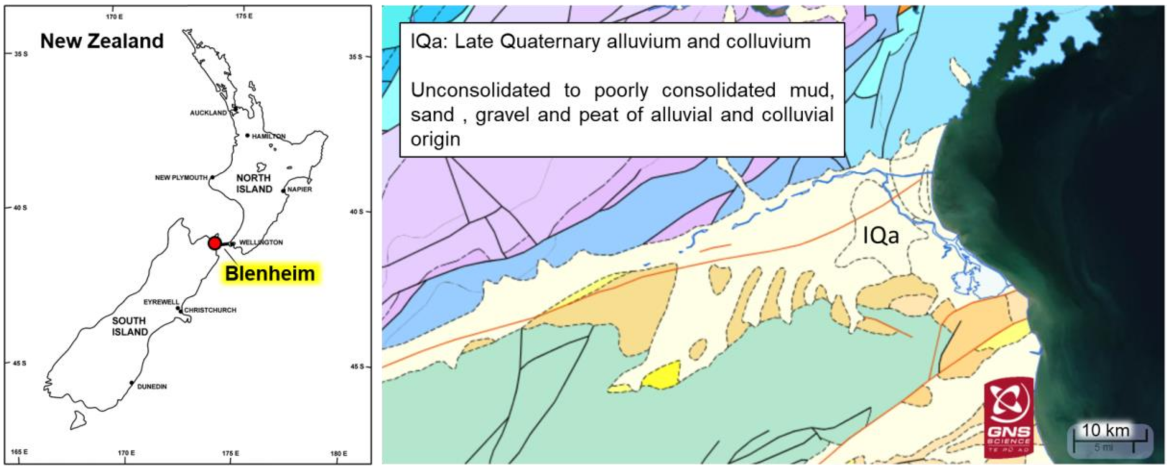


Fig. 4 – *Geology chaarcteristcs of Blenheim (geological map from https://data.gns.cri.nz/geology/)*

Table 2 – *Summary of ground motion characteristics recorded in Blenheim during recent significant earthquakes*

Ground Motion Station	Soil Class	Geometric mean horizontal PGA (g)		
		2016 M_w7.8 Kaikoura earthquake	2013 M_w6.6 Lake Grassmere earthquake	2013 M_w6.6 Cook Strait/Seddon earthquake
MCGS	D	0.227	0.119	0.068
BWRS	B	0.126	0.097	0.085

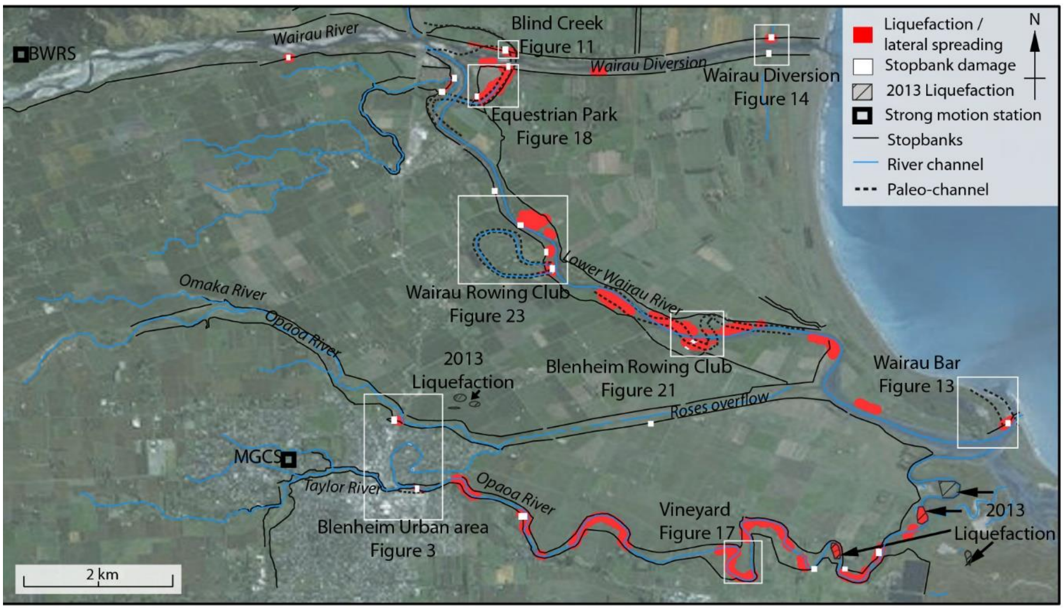


Fig. 5 – *Overview of liquefaction damage in Blenheim and the Wairau Plains resulting from the 2016 M_w7.8 Kaikoura earthquake and 2013 M_w6.6 Lake Grassmere earthquake (adopted from Stringer et al., 2017).*

Sand boils were observed at Lansdowne Park which is located adjacent to the southern bank of the Ōpaoa River on the northern edge of Blenheim (Figs. 6 and 7). A detailed survey of ejecta was conducted in the southern half of Lansdowne Park; the location of ejecta features is shown in red in Fig. 7. Sand boils were typically 1-2 m in diameter and in many cases formed lineaments of aligned sand boils (Fig. 8a). The ejecta material was largely bluish-grey in colour, but there were some features which were light brown in colour.

Wet sieve analyses were performed on 10 ejecta specimens from across Lansdowne Park (locations are marked in Fig. 7 with blue stars). The particle size distributions (PSD), which are summarised in Fig. 8b, can be separated into two groupings. LDP-3, 4, 5, 8, 9, and 10 being fine sands, while the samples LDP-1, 2, 4, 6, and 7 were medium sands.

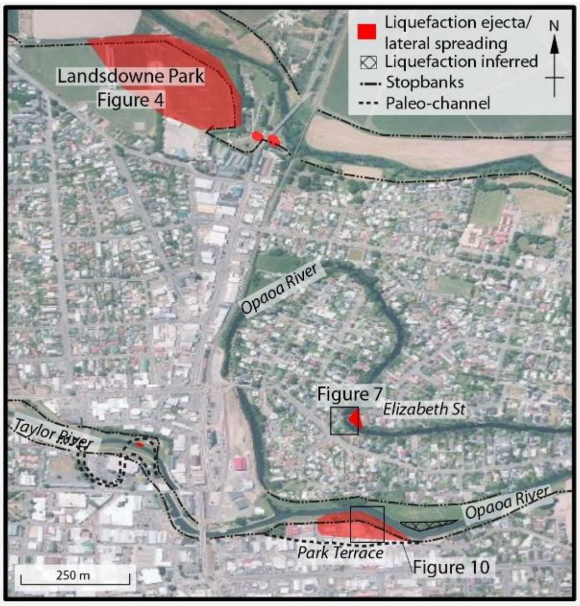


Fig. 6 – *Overview of liquefaction related damage within the Blenheim urban area (adopted from Stringer et al., 2017).*

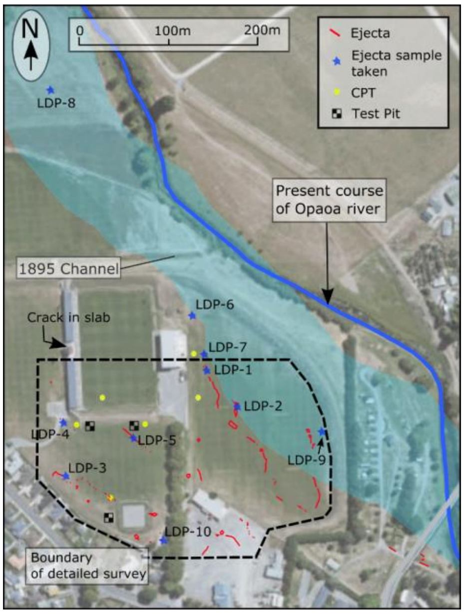
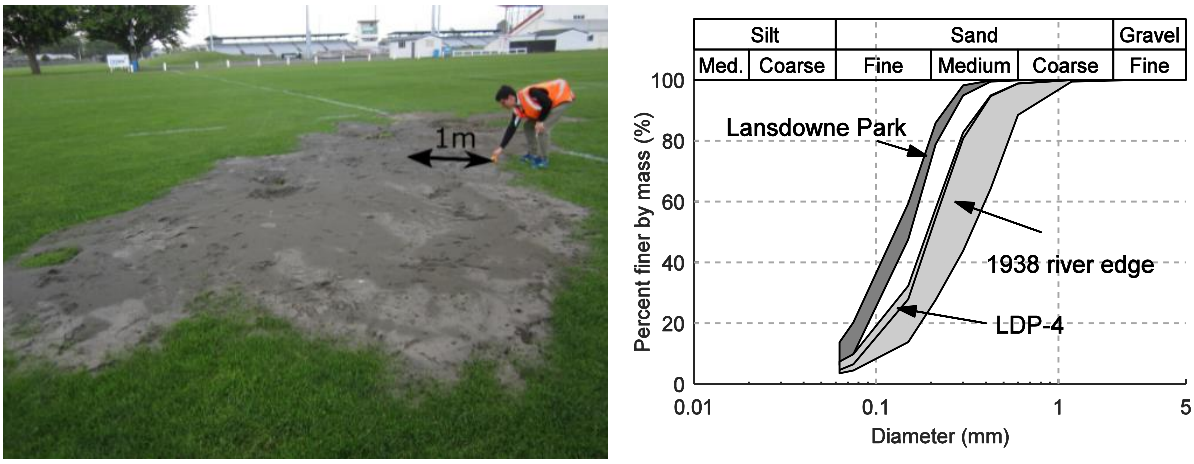


Fig. 7 – *Ejecta deposits mapped within Lansdowne Park shown by red areas (adopted from Stringer et al., 2017)*



(a) (b)

Fig. 8 – *Ground observations at Lansdowne Park: (a) Example of liquefaction ejecta features observed at Lansdowne Park; (b) Particle size distributions of the ejecta obtained from Lansdowne Park (adopted from Stringer et al., 2017)*

Site selection and characterisation in Blenheim

Three sites with shallow gravelly deposits (called here S1 (i.e., LDP6 in Fig. 7), S2, and S3) were selected in Blenheim, as shown in Fig. 9. The soil, DPT and V_S profiles of each site are shown in Fig. 10. The uppermost portion of such soil profiles consist essentially of alluvial sandy gravels with varying gravel content.

Even though these sites are relatively close to each other, and the soil profile characteristics are similar, the liquefaction manifestation was quite different. Specifically:

- *Site S1* liquefied during the 2016 M_w7.8 (a_{max} = 0.227 g) earthquake but did not liquefy during the 2013 M_w6.6 (a_{max} = 0.119 g and 0.068 g) earthquakes.
- *Site S2* did not liquefy in 2013, but ground features such as a localized 10 cm surface settlement suggests that it could have liquefied in 2016, although manifestation of soil ejecta was not observed at the ground surface.
- At *Site S3* no liquefaction features were observed following the three selected earthquakes.

At sites S1 and S3 samples for gradation testing were obtained from 120-mm-diametre boreholes using a sonic drilling rig. The PSD curves and photographic images of soil samples retrieved from the identified critical liquefaction layers are reported in Fig. 11. A part from samples S1-4 and S1-6 that can be classified as gravelly sands/sands, most of the soils are sandy gravels/gravels with gravel content ranging from 54% to 92% (based on a gravel size greater than 2 mm, as per NZS 4402.2.8.1:1986). Their PSD fall well within the gradation boundaries reported by Rollins et al. (2021) for case histories of liquefied gravelly soils.

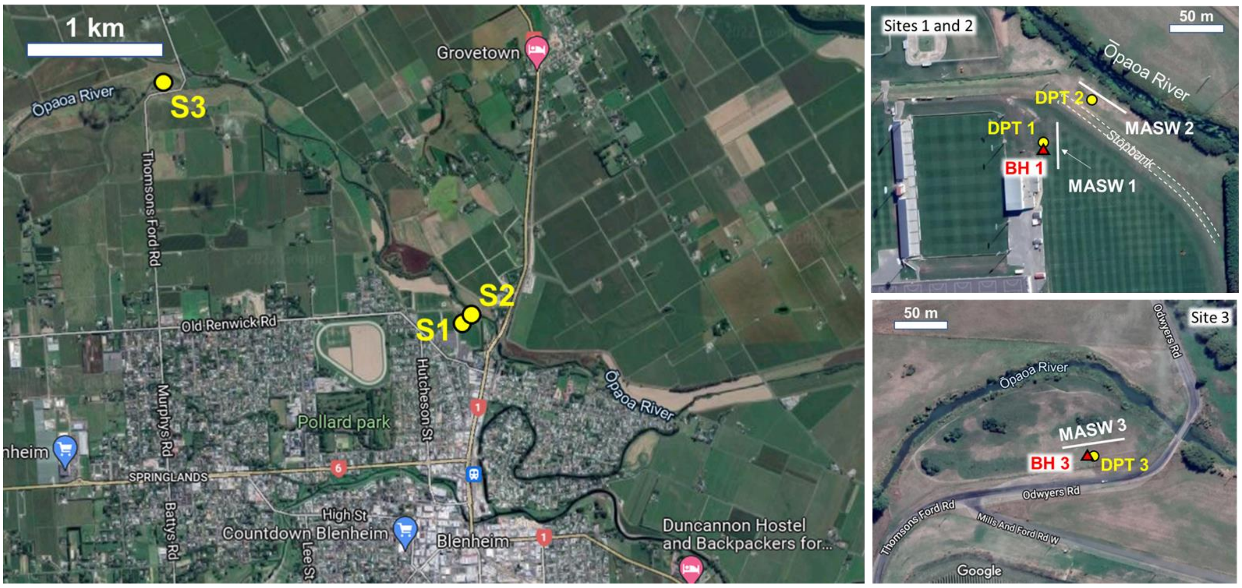


Fig. 9 – *Location of geotechnical investigations in Blenheim.*

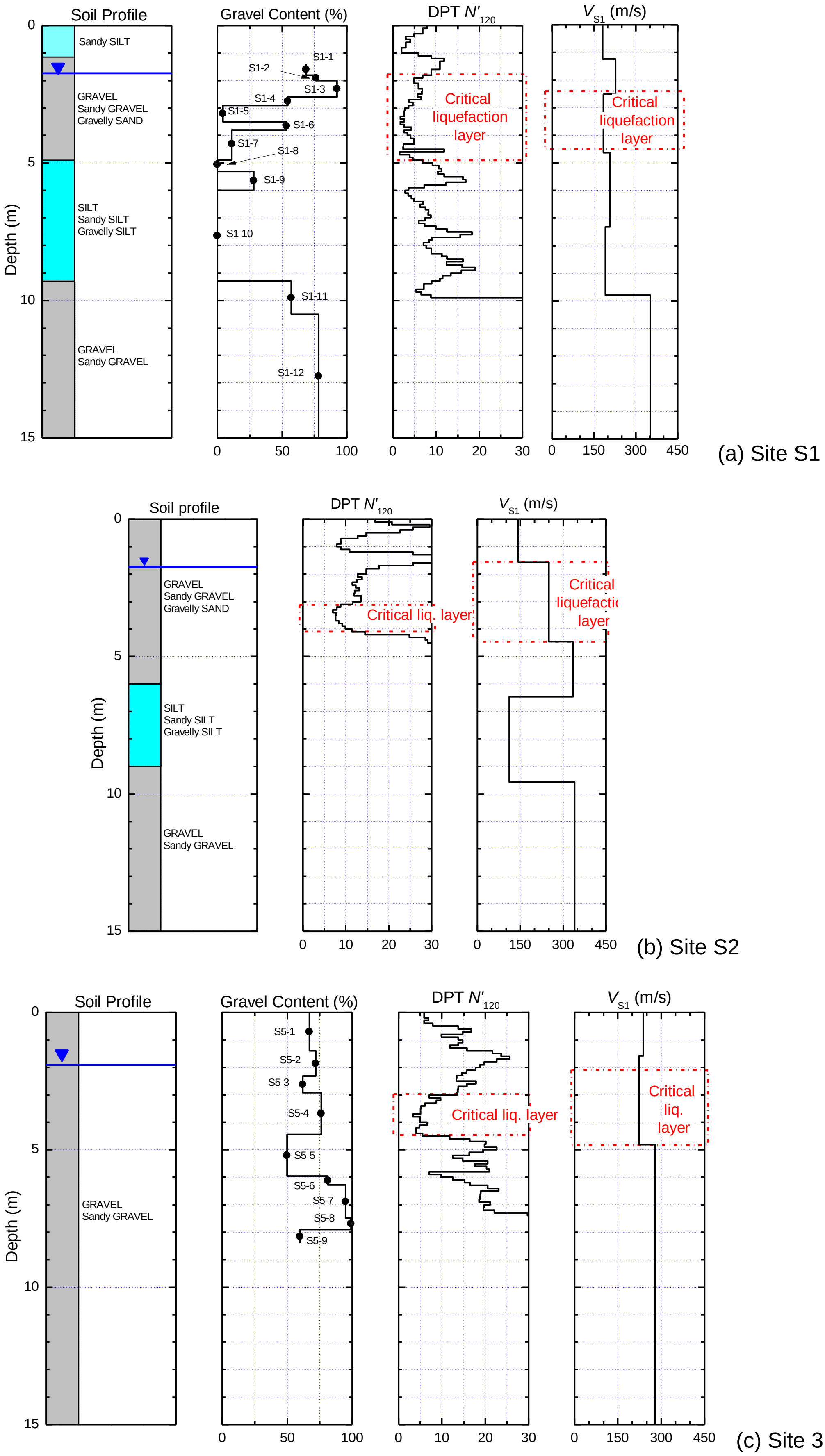


Fig. 10 – *Soil, gravel content, DPT N'*$_{120}$ and *Vs1 profiles of investigated sites in Blenheim.*

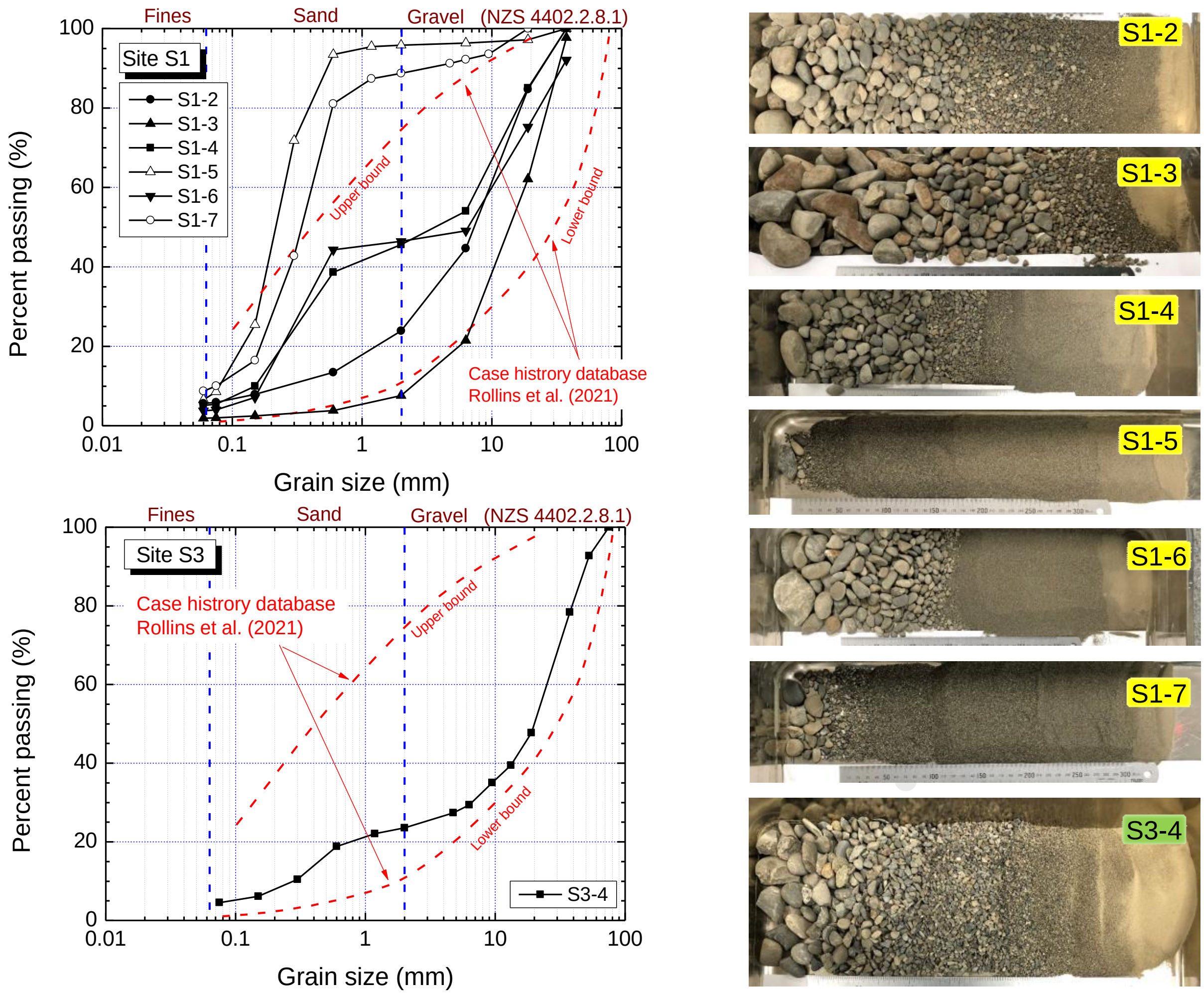


Fig. 11 – *Particle size distribution curves and photographic images of soil samples retrieved from the identified critical liquefaction layers at sites S1 and S3.*

As described henceforward in more details, DPT were carried out at each site with the primary purpose of site characterisation for liquefaction assessment. Shear wave velocity (V_S) measurements were also conducted using a non-invasive geophysical testing method, specifically the multi-channel analysis of surface waves (MASW) method (Park et al., 1999).

It is important to mention also that CPT were attempted but fundamentally performed poorly in the gravelly soils, reaching shallow refusal at shallow depth, even when the soil profile was not particularly dense, and making it not possible to exhaustively characterise and evaluate the liquefaction potential of the investigated soil deposits.

DPT investigations

DPT were carried out at each site. The 74-mm DPT cone was advanced by using a drill rig equipped with an automatic free-fall 63.5-kg SPT hummer dropped from a height of 76 cm (Fig. 12). Hammer energy delivery measurement were made using an instrumented rod section and Pile Driving Analyser (PDA) device. The energy measurement indicates that the SPT hammer delivered an average of 85.6% of their theoretical free-fall energy (Fig. 13).

(a) (b) (c

Fig. 12 – *Field testing equipment used in this investigation: (a) drill rig with 63.5-kg free-fall SPT hammer; (b) 74-mm DPT cone; and (c) geophone array and sledge hammer for MASW-based Vs measurements.*

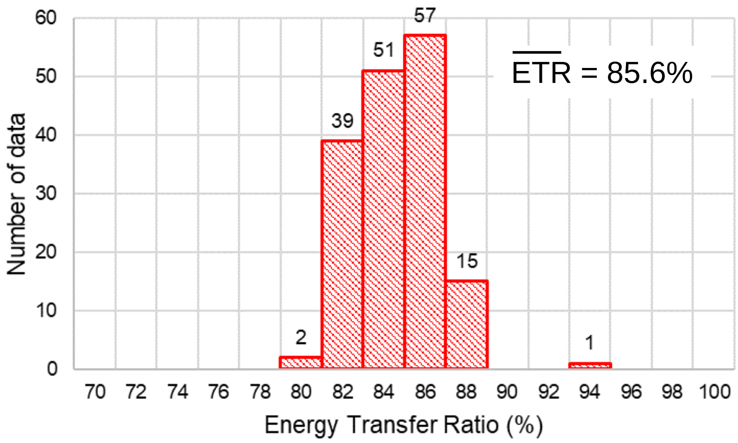


Fig. 13 – *Frequency diagrams showing number of hammer drops with ETR values as measured in this investigation.*

Cao et al. (2012) found that the Chinese DPT (with a weight of 120 kg dropped from a height of 100 cm) provided an average of 89% of the theoretical free-fall energy, on the basis of 1200 energy hammer energy measurements. Since the energy delivered by the 63.5-kg SPT hammer (E_{SPT} = 473 Nm) used in this study was less than the energy typically supplied by a Chinese DPT hammer (E_{DPT} = 1176 Nm), it was necessary to correct the measured blow count. Based on Rollins et al. (2020, 2021) findings, the correction could be made using the simple liner reduction reported in Eqn. (1), originally proposed by Seed et al. (1985) for SPT testing and found valid for DPT:

$$N_{120} = N_{\mathrm{SPT}}(E_{\mathrm{SPT}}/E_{\mathrm{DPT}}) \quad (1)$$

where N_{SPT} is the blows per 30 cm of penetration obtained with the SPT hammer which delivers an energy of E_{SPT}.

Based on 165 hammer energy measurements obtained in this study (Fig. 13), the ratio of the energy delivered by the SPT hammer divided by that of the Chinese DPT hammer was found to be 0.39 (= 0.856 E_{SPT} / 0.89 E_{DPT}).

Furthermore, to account for the effects of overburden on the DPT penetration resistance, Cao et al. (2013) recommended an overburden correction factor (C_n) to obtain the normalised N'_{120} values as indicated by Eqn. (2):

$$N'_{120} = N_{120}\, C_n \tag{2}$$

As originally proposed by Liao and Whitman (1986), C_n was estimated using Eqn. (3):

$$C_n = \sqrt{100/\sigma'_0} \leq 1.7 \tag{3}$$

where σ'_0 is the initial vertical effective stress in kN/m^2. Following the recommendation from Rollins et al. (2020), a limiting value of 1.7 was added to be consistent with the C_n factor used to correct penetration resistance from other in-situ tests (Youd et al., 2001). It should be noted that Eqn. (3) was originally recommended for sandy soils with a wide range of gradation and densities. Yet, for gravelly deposits with a considerable percentage of sand, as is usually the case of liquefiable gravelly soils, the use of Eqn. (3) to estimate C_n is a reasonable approximation (Rollins et al., 2020). Plots of the energy-corrected N'_{120} versus depth profiles are reported in Fig. 10.

Shear wave velocity measurements

As part of this EQC project, V_S profiles were also developed using a multi-channel analyses of surface wave (MASW) method (Park et al., 1999). The MASW measurements were carried out nearby the DPT sites using linear arrays of 24 vertical and horizontal Geospace GS11D 4.5 Hz 4000 ohm geophones, oriented to measure Rayleigh and Love surface waves, respectively. At all three sites, the geophones were spaced 1 metre apart such that the total array length from first to last geophone was 23 metres. A vertical strike of a 12 lb sledgehammer on a steel strike plate was used to excite Rayleigh waves and a horizontal sledgehammer strike on steel capped shear beam was used to excite Love waves. A total of six source offsets were used three at -5, -10, and -15 metres from the first geophone and three off the other end of the geophone array at +5, +10, and +15 metres from the last geophone. At each source offset, five sledgehammer strikes were stacked to increase the signal-to-noise ratio. A Geometrics Geode 24-channel seismograph was used to digitise and record the dynamic signals. For theses tests, the sampling frequency was 1,000 Hz, the record length was 4 seconds, and the pre-trigger delay was 0.5 seconds.

The active-source MASW data were processed using the Frequency Domain Beamformer (FDBF) method in combination with the multiple-source offset technique (Zywicki, 1999; Cox and Wood, 2011). The use of multiple source offsets allows for quantifying dispersion uncertainty and the identification of near field contamination. The dispersion data from each offset was cleaned and combined to develop a single composite experimental dispersion curve.

The open-source software package Geopsy (Wathelet 2008) was used to perform a multi-mode, joint inversion of the combined active and passive dispersion data. The forward modelling calculation methods were originally developed by Thomson (1950) and Haskell (1953) and later modified by Dunkin (1965) and Knopoff (1964). As the surface wave forward modelling problem is ill-posed and non-unique, tens of thousands of possible profiles were considered in each inversion and any of the models with sufficiently low misfit to the experimental data may be representative of the velocity structure at the site.

The analyst defined constraints or layer parameterization for the inversion are V_S, V_P, depth, Poisson's ratio, density, and the number of layers in the soil profile. The use of a parameterization in Dinver aids the inversion process by reducing the size of the solution space from which velocity profiles can be generated. Exiting a priori data incudes the nearby CPT and Chinese DPT testing. These data were used to constrain the near surface layering and velocities. For each parametrisation, 60,000 models with corresponding theoretical Rayleigh wave and Love wave dispersion curves were generated in an effort to obtain the best dispersion curve fit. Within Geopsy, the misfit or the overall 'closeness' between the experimental and theoretical dispersion curve is computed for each model. For the purposes of the liquefaction assessment, the single "best fit" V_S profile was extracted for each site.

The normalised overburden stress-corrected shear wave velocity (V_{S1}) profiles obtained at each site are reported in Fig. 10. The V_{S1} values were estimated using the expression reported Eqn. (4), which was proposed by Sykora (1987), Robertson et al. (1992) and Kayen et al. (1992) and adopted by Youd et al. (2001):

$$V_{S1} = V_S\ (P_a/\sigma'_0)^{0.25} \tag{4}$$

where P_a is the atmospheric pressure approximated to 100 kPa.

Liquefaction triggering analyses

The cyclic stress ratio (*CSR*) induced by the earthquake was computed using the following simplified equation proposed by Seed and Idriss (1971):

$$CSR = 0.65\ (a_{max}/g)\ (\sigma_0/\sigma'_0)\ r_d \tag{5}$$

where a_{max} is the peak ground acceleration (PGA), σ_0 and σ'_0 are the total and effective initial vertical stresses, respectively, and r_d is the depth reduction factor due to soil deformability as defined by Youd et al. (2001). The PGA were obtained from the nearby MGCS strong motion station as indicated by Table 2.

To facilitate comparison with data points obtained from earthquakes of different magnitude, *CSR* was converted to *CRS* at M_w=7.5 ($CSR_{Mw=7.5}$) as indicated by Eqn. (6):

$$CSR_{Mw=7.5} = CSR/MSF \tag{6}$$

where the *MSF* was evaluated as recommended by Rolling et al. (2021), who developed a specific *MSF* expression for gravelly soils:

$$MSF = 7.258\,\exp(-0.264\,M_{\mathrm{w}}) \tag{7}$$

Comparison with DPT-based liquefaction triggering curves

At each site, the critical liquefaction layer was selected as the layer most likely to trigger and manifest liquefaction at the ground surface (Cubrinovski et al., 2018b), which corresponded with the lowest N'_{120}. The average of values N'_{120} and CSR $_{\mathrm{Mw=75}}$ obtained based on the critical liquefaction layers are reported in Table 3. As indicated in Fig. 10, the critical liquefaction layers consisted of shallow gravelly deposits below the water table with an average N'_{120} between 4.3 and 8.5 Table 3.

The $CSR_{\mathrm{Mw=75}}$ versus DPT N'_{120} data points are plotted in Fig. 14 for comparison with the liquefaction triggering curves developed by Rollins et al. (2021), which were determined using the expression reported in Eqn. (8):

$$CRR = \exp\left[\frac{0.0008\,(N'_{120})^3 - 1.3\,M_{\mathrm{w}} - \ln\left(\frac{1-P_{\mathrm{L}}}{P_{\mathrm{L}}}\right)}{5.2}\right] \tag{8}$$

where *CRR* is the cyclic resistance ratio and P_{L} is the probability of liquefaction occurrence. In Eqn. (7), subsisting M_{w} =7.5 and P_{L} for various liquefaction probability values, the triggering curves reported in Fig. 14 were obtained.

In Fig. 14, the vertical lines associate with each data point indicate the range of *CSR* values estimated for the critical liquefaction layer, while the horizonal lines specify the range of N'_{120} values measured with the critical liquefaction layer (Fig. 10).

For site S1, the critical liquefaction layer is located at a depth between 1.75 m and 4.9 m below the ground surface. The CSR-N'_{120} data points plot above the P_{L} = 85% triggering curve for the 2016 Kaikoura earthquake and below the P_{L} = 15% triggering curve for the smaller-magnitude 2013 Lake Grassmere and Cook Strait/Seddon earthquakes. Therefore, correctly predicting both liquefaction and no liquefaction occurrence at this site. It is important to mention that granulometric analyses have confirmed consistency between the PSD curve of the sand fraction of the gravelly deposit within the critical liquefaction layer and that of the soil ejecta samples collected at the ground surface after the Kaikoura earthquake.

For site S2, the critical liquefaction zone is located at a depth between 3.0 m and 3.9 m below the ground surface. The CSR-N'_{120} data points plot just below the P_{L} = 85% triggering curve for the 2016 Kaikoura earthquake and well below the P_{L} = 15% triggering curve for the 2013 Lake Grassmere and Cook Strait/Seddon earthquakes. As mentioned earlier, site S2 did not liquefy in 2013, but ground features such as a localized 10-cm surface settlement suggests that it could have liquefied in 2016, although manifestation of soil ejecta was not observed at the ground surface. Therefore, it appears that the DPT-based procedure, employed in this study, correctly predicted both liquefaction and no liquefaction occurrence at this site as well.

For site S3, the critical layer of liquefaction is located at a depth between 3.0 m and 4.4 m below the ground surface. Alike sites S1, the CSR-N'_{120} data points plot above the P_L = 85% triggering curve for the 2016 Kaikoura earthquake and below the P_L = 15% triggering curve for the 2013 Lake Grassmere and Cook Strait/Seddon earthquakes. Yet, at this site, liquefaction manifestation at the surface was not observed following the three seismic events. Therefore, it seems that the employed DPT-based liquefaction triggering procedure incorrectly predicted liquefaction occurrence at this site for the 2016 Kaikoura earthquake. Yet, as shown by the PSD curve reported in Fig. 11 for sample S3-4, the PSD curve falls well within the boundaries of gravelly soils that liquefied in past earthquakes, suggesting that liquefaction could have occurred within this layer, but without manifestation at the surface. Other researchers have observed similar false-positive predictions (i.e., liquefaction predicted without liquefaction manifestation, e.g., soil ejecta, settlement and lateral spreading) in deposits consisting of highly interbedded silt and sand layers (e.g., Youd et al., 2009; Cubrinovski et al., 2018b; Rollins et al., 2020). This is obviously not the case of site S3, but a review of the soil profile shown in Fig. 9 indicates that above the identified critical layer there is a 3 m-thick non-liquefiable sandy gravel layer with high penetration resistance (potentially indicating a dense soil deposit) that may have acted as a less-permeable capping layer, contributing therefore to the development of high excess pore water pressures leading to liquefaction, and at the same time preventing sand ejecta to reach the ground surface and liquefaction manifestation to be observed.

Table 3 – *Summary of average soil properties and earthquake characteristics in the identified critical liquefaction layers*

Site	Soil properties				Earthquake characteristics		
	Avg. depth (m)	Avg. σ_0 (kPa)	Avg. σ'_0 (kPa)	Avg. N'_{120} (blows per 0.3 m)	M_w7.8	M_w6.6	M_w6.6
					MSF = 0.93	MSF = 1.27	MSF = 1.27
					a_{max} = 0.227g	a_{max} = 0.119g	a_{max} = 0.068g
					$CSR_{Mw=7.5}$		
1	3.35	62.0	46.2	4.3	0.206	0.077	0.044
2	3.45	63.8	47.0	8.5	0.212	0.079	0.045
3	3.70	66.6	47.4	5.6	0.217	0.081	0.046

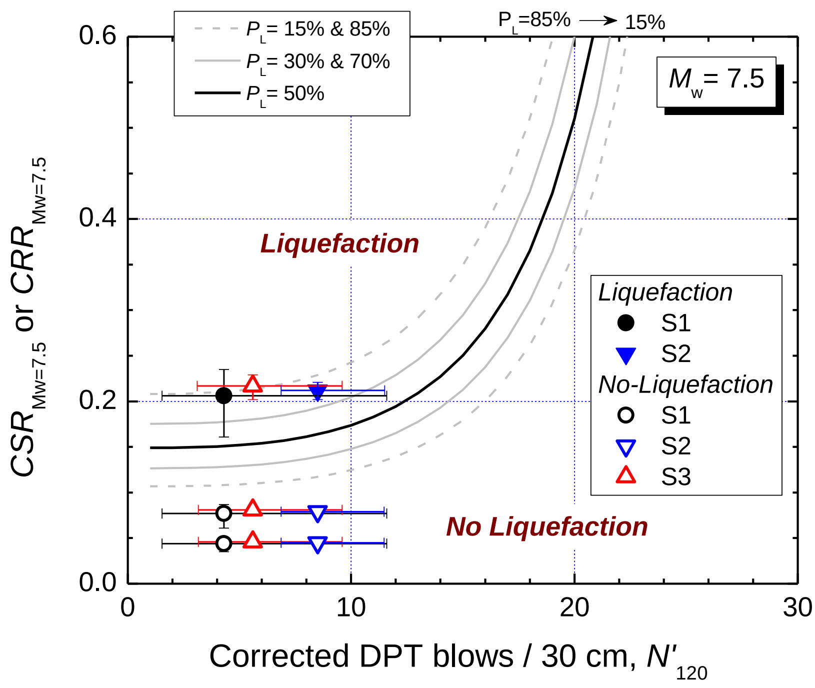


Fig. 14 – *Comparison between the DPT data points obtained in this study for liquefied and no-liquefied sites in Blenheim and DPT-based probabilistic liquefaction-triggering curves for gravelly soils (adjusted at M_w = 7.5) proposed by Rollins et al. (2021).*

Comparison with V_S-based liquefaction triggering curves

Procedures based on overburden stress-corrected shear wave velocity (V_{S1}) by Andrus and Stokoe (2000) and Kayen et al. (2013) were used to conduct additional liquefaction triggering analyses for the Blenheim gravelly sites. Such procedures have been developed for sands and in this study an attempt is made to evaluate their suitability/accuracy for Blenheim sandy gravels.

Critical liquefaction zones for each site were selected based on the V_{S1} profiles reported in Fig. 10. They are generally consistent with those defined but DPT measurements, but not identical. This is due mainly to MASW-based *V*s measurements being less detailed than the DPT blow count ones.

The *CSR*-V_{S1} datapoints are plotted in Fig. 15 and compared with both the Andrus and Stokoe (2000) and Kayen et al. (2013) liquefaction triggering curves. All the nine Blenheim data points plot below the triggering curves for both methods, indicating the inability to predict the liquefaction potential for such sandy gravelly deposits.

Other researchers have reported similar inaccuracies in predicting liquefaction of gravelly deposits using Vs-based curves developed for sands (e.g. Cao et al., 2013, Rollins et al, 2020) and suggested that some adjustments are required for gravelly soil profiles.

Another consideration is that non-invasive geophysical methods (e.g., MASW) average the material properties underneath the horizontal extent of the geophone array and not well suited to the identification of relatively thin soil layers. Non-invasive geophysical methods are better suited to the measurement of relatively high depth resolution V_S profiles. However, direct-push invasive geophysical methods such as seismic cone penetration would have difficult penetrating the gravel layers at these sites and borehole-based method are relatively high-cost.

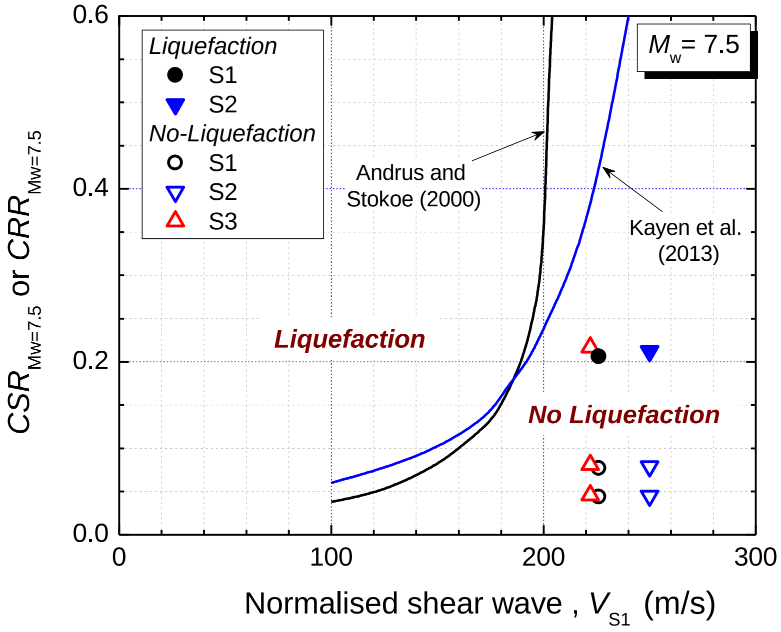


Fig. 15 – *Comparison between the V_{S1} data points obtained in this study for liquefied and no-liquefied sites in Blenheim and V_{S1}-based liquefaction-triggering curves proposed by Andrus and Stokoe (2000) and Kayen et al. (2013) for sands.*

Conclusions and key findings

Based on the field investigations conducted in Blenheim, the following conclusions can be drawn:

- DPT could be driven through the alluvial gravelly profiles (irrespective of the gravel content and maximum gravel particle size) just using a conventional 63.5-kg SPT hammer (energy efficiency of 85.6%).
- The DPT-based liquefaction triggering procedure developed by Rollins et al. (2021) was found to be correct in predicting liquefaction or not at sites S1 and S2 where liquefaction features (i.e., sand ejecta and/or settlement) were observed in sandy gravelly deposits.
- The same procedure by Rollins et al. (2021) was incorrect in predicting liquefaction at sites S3 in a soil profile that produced no surface evidence of liquefaction despite low blow count N'_{120}. Yet, this false-positive case is likely a result of the system response of the profile, due to the presence of a 3m-thick non-liquefiable sandy gravel capping layer, which likely inhibited eruption of sand eject at the surface, in as similar way observed by other researchers in previous studies for highly interbedded silt and sand profiles.
- MASW-based V_S measurements were found to provide less detailed information as compared to DPT. Moreover, available V_{S1}-based liquefaction triggering procedures

were unable to distinguish between liquefaction and no-liquefaction observed behaviours at these sites.

It is important to emphasise also that CPT were attempted but generally performed poorly in such alluvial gravelly soils, reaching refusal at shallow depth, even when the soil profile was not particularly dense. Therefore, it was not possible to comprehensively characterise and evaluate the liquefaction potential of the investigated soil deposits using CPT-based procedures.

This project has, therefore, confirmed that SPT-hammer-driven DPT represents a reliable and cost-effective technique for measuring the penetration resistance of typical New Zealand alluvial gravelly soils for liquefaction assessment. As such, it should be regarded as a useful field test technique whenever the liquefaction resistance of gravelly soils is of concern. Moreover, the newly-developed DPT-based triggering procedure developed by Rollins et al. (2021) was generally successful in predicting liquefaction or not for Blenheim alluvial gravelly soil deposits.

Impact of the study

The outcomes of this study not only well complement the existing international case history database of gravelly soil liquefaction, but more importantly support the use of a reliable field testing technique (now available in New Zealand because of this EQC project) and a gravel-specific DPT-based procedure for characterising the liquefaction resistance of gravelly soils that are instrumental for defining the impact and consequences of liquefaction of gravely soils on land and infrastructure during expected severe long-duration earthquakes (e.g., Alpine Fault or Hikurangi Subduction Zone Earthquake) for many critical infrastructure assets across New Zealand.

Ongoing and future work

Field testing

The field investigations to characterise Blenheim gravelly soils deposits has been completed. Yet, more sites could be tested in future for a more comprehensive characterisation of the liquefaction potential of gravelly soils in Blenheim.

Future work of the research team can be summarised as follows:

- Further characterisation of NZ alluvial gravelly soils deposits and their liquefaction potential using DPT, with primary focus on gravelly soil deposits that liquefied during past earthquakes in Darfield and Murchison.
- Characterisation of gravelly soil deposits of glacial origin and their liquefaction potential using DPT in Queenstown and similar areas.
- Create a gravelly soil database for NZ, and type-specific liquefaction triggering curves for NZ alluvial and glacial gravelly soils deposits.

Laboratory testing

In parallel to detailed field investigations, the research team is currently carrying out also advanced liquefaction tests in the geotechnical laboratory of the University of Canterbury to better characterise the liquefaction potential of gravelly soils under well-controlled testing conditions (e.g., density, fabric and structure, stress conditions).

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Outputs and Dissemination

Chiaro et al. (2022). DPT-based site characterisation for liquefaction assessment of alluvial sandy gravel deposits in Blenheim (in preparation). To be submitted for review and possible publication to the *Bulletin of NZSEE*.

Pokhrel, A., Chiaro, G., Cubrinovski, M., and Kiyota, T. (2022). Liquefaction potential of sand-gravel mixtures: experimental observations. *Proc. of the 2022 NZSEE Annual Technical Conference*, 27-29 April, Wellington, NZ, pp. 10 (to appear).

List of key end users

- Marlborough District Council (MDC)
- Earthquake Commission (EQC)
- Ministry of Business, Innovation and Employment (MBIE)
- New Zealand Geotechnical Society (NZGS)

- Professional engineers