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Geotechnical assessments for design of foundations in lower-seismicity liquefaction prone areas – a case study

M. C. McLean, B. O'Loughlin
ENGEO, Tauranga.

ABSTRACT

This paper presents a case study of a Ministry of Education (MoE) building project on potentially liquefiable dune sands at Mount Maunganui, Tauranga. It presents our methodology for assessing liquefaction vulnerability including triggering response and step change behaviour, how these aspects were considered during design in compliance with NZS1170, the MoE Structural and Geotechnical Requirements (SGR), and how project challenges introduced by the release of the National Seismic Hazard Model (NSHM) during design were overcome.

Liquefaction triggering just beyond Serviceability Limit State (SLS1) meant that careful consideration and guidance needed to be given to the level of foundation and structural resilience. Our analyses determined that punching failure and deformations could be addressed to avoid collapse, however significant repair and/or rebuild would be necessary at both SLS2 and Ultimate Limit State (ULS), requiring a resilient solution incorporating shallow ground improvement and a robust ground beam foundation system.

This paper presents the liquefaction and soil structure interaction analyses undertaken to develop a solution to increase seismic resilience and safeguard the building from potential building code updates. It also discusses how achieving minimum code requirements may result in low levels of seismic resilience, and why assessing triggering response and step change behaviour is necessary to inform robust geotechnical and structural considerations for foundations and ground improvements that deliver an appropriate level of seismic resilience.

Liquefaction triggering and foundation design in lower seismicity areas is a relatively common but less so talked about challenge for consultants. With increasing regulatory requirements being introduced by building consent authorities to improve resilience, increasing focus is being placed on triggering assessment to determine liquefaction vulnerability and foundation resilience. This paper introduces the approaches taken, lack of clarity in existing codes and guidelines, and presents the benefits of adopting MOE's SGR for foundation design resilience for the project.

1 INTRODUCTION

1.1 Location and Geological Setting

Mt Maunganui College is located on the eastern side of Maunganui Road within the suburb of Mount Maunganui, Tauranga. Online GIS resources provided by Tauranga City Council (TCC) indicate the site elevation varies between 5.5 and 8.0 metres above sea level (NZVD2016 Datum), generally sloping gently from north to south (although largely near level).

GNS (2010) and Briggs *et al* (1996) map the site as underlain by a sequence of Holocene age fixed foredune sand deposits. The original dune morphology, characterised by repeating ridges and troughs aligned parallel to the beach, is visible in 1940s aerial imagery. These features have since been modified over time by urban development, typically from levelling the dune ridges and infilling of trough systems, locally burying interdune organic rich soils.

1.2 Project Description

ENGEO was engaged as the geotechnical consultant at the Masterplan stage of the project and retained as the geotechnical consultant through the design stages of a teaching block – a new 3450 m^2, two storey Importance Level 3 building proposed over the existing southwestern carparking and adjacent to Maunganui Road. The proposed foundation system included a grillage of ground beams typically 1 m wide and 0.8 m deep, with a 150 mm thick reinforced concrete floor. The building design was ultimately discontinued at 100% Developed Design and the project pivoted to considering three smaller buildings that offered a similar amount of new teaching space, on a similar ground beam foundation system.

1.3 Ministry of Education Structural and Geotechnical Guidance

MoE have an operational Structural and Geotechnical Requirements (SGR, 2020) document setting out key design requirements for Structural and Geotechnical professionals designing new school buildings, which are typically more stringent than the New Zealand Building Code minimum requirements. MoE's Design Review Panel (DRP) made up of industry experts are engaged to independently review the proposed consultant designs against this document and current industry best practice at multiple hold points throughout design process.

The SGR includes a preference for shallow foundation systems. There is no requirement for 'good ground' as presented in NZS3604, with specific engineering design being preferred to standardised acceptable solutions. The seismic performance requirements for MoE assets are outlined in Section 4.2 of the SGR, and are as follows:

- SLS1: The building must be readily repairable with the building able to continue being used for its intended purpose.
- SLS2: The building may suffer tolerable damage where it can continue to be used for its intended purpose with some reduction to amenity, and repairs or reinstatement should be straightforward or rapid.
    - The building must be able to undergo a subsequent ULS event with acceptable performance.
- ULS: The building should not require demolition.
    - The building foundation should not be the weak link in the system and there must be an identified strategy for repair or recovery.

The inclusion of SLS2 performance criteria exceeds the minimum requirements of the NZ Building Code, as does a requirement for foundation repair or recovery following a ULS event.

1.4 Seismicity (TCC Bradley Study vs NZGS Module 1 vs NSHM)

TCC commissioned a Probabilistic Seismic Hazard Analysis (PSHA) encompassing Tauranga city (Bradley Seismic Limited, 2019), which allowed peak ground accelerations (PGAs) to be calculated from VS30 profiles obtained across the city and interpolated across broadly similar ground conditions. TCC hosted this information on a series of online, publicly available maps. The study author intended for the findings to be used in residential land damage assessment applications, however for a time, Council allowed (and sometimes recommended) adoption for other land use or building types. The PGAs for typical landforms around Tauranga were lower than those in NZGS Module 1 (2001) or those derived via NZS1170.5:2004

In 2022 TCC recommended returning to PGA values presented in NZGS Module 1, only allowing for the adoption of the Bradley study when supported by peer review by one of a small number of nominated professionals. At that time, our experience with MoE suggested they preferred continuing to use the Bradley study, however for this project the MoE requested ENGEO suspend design work until after the release of the National Seismic Hazard Model (NSHM, 2022), as their Geotechnical Design Review Panel (DRP) panellist expected the NSHM to more closely align with the Bradley Study than Module 1.

Instead, the NSHM update released in November 2022 showed an increase in PGA for Importance Level 3 (IL3) buildings for the SLS2 and ULS cases. MoE recommended adoption of the values presented in the NSHM for future-proofing the design and from a regulatory standpoint. The requested design approach and the consequences of changing PGA during the project are discussed later in this paper.

For comparison, the mean moment magnitude (Mw) and PGA values for Tauranga from NZS1170:2004 (Class D soil), Module 1 and the NSHM (VS30 of 300 m/s, adopted based on site-specific interpolation) for IL2 and IL3 structures are presented in Table 1. The Mw presented in Module 1 was checked against disaggregation of the NSHM data, which indicated the primary influence being shallow crustal (versus deep crustal / subduction margin) earthquakes in the Tauranga region. Accordingly, Mw of 5.9 was maintained from MBIE Module 1.

Table 1: Comparison of Ground Shaking Parameters, IL3 Buildings, Tauranga

	Mw	**SLS (1 in 25 yr)**	**SLS2 (1 in 250 yr)**	**ULS (1 in 1000 yr)**
TCC Bradley Study (2019)	6.1 to 6.3	0.06	0.15	0.24
MBIE Module 1 2021 ed.	5.9	0.07	0.22	0.39
NSHM 2022 (VS30=300 m/s)	5.9	0.05	0.38	0.46

2 GROUND MODEL

Site investigations comprised hand auger boreholes, Cone Penetrometer Tests (CPTs), machine boreholes and groundwater monitoring which showed a consistent soil profile between test locations. The generalised ground model is summarised in Table 2, including separation of the dune sand deposits into three engineering geological units DS1, DS2 and DS3.

Table 2: Generalised Ground Model

Material	Depth Range	Strength Characteristics	Notes
Fill and Topsoil / Organic Rich Interdune Deposits	Up to 1.0 m	Variable / Not Applicable	Non-engineered and variable in composition, but largely sand and aggregate fills overlying 'black sand'. Assumed to be replaced by a dense, well compacted gravel raft.
Dune Sand 1 (DS1)	~1.0 to ~4.5 m	Loose	Laterally continuous lower density layer. Susceptible to liquefaction below ~2.5 m depth when ground shaking exceeds 0.3 g.
Dune Sand 2 (DS2)	~4.5 to ~6.0 m	Medium Dense	Laterally continuous higher density layer. Not liquefiable.
Dune Sand 3 (DS3)	~6.0 to ~15 m	Loose to Medium Dense	Variable density material with higher and lower density layers. Higher density layers are typically laterally continuous. Portions of the layer begin to liquefy when ground shaking exceeds 0.12 g.
Estuarine / Harbour Sediments	15 m +	Firm	Underlying sediments deposited during periods of lower sea levels and since buried. May be very thick. Little to no influence on seismic performance of surface founded structures, other than influencing VS30.

Groundwater was subsequently monitored for a period of six months, which included Cyclone Gabrielle and the Auckland Anniversary Weather events. During this time the groundwater rose from approximately 3 m to 2.3 m below ground level and remained elevated for a period of months. TCC publish modelled long term climate adjusted groundwater levels (and recommend their use for liquefaction assessments), which for this site aligned with our measured elevated water conditions for these events.

3 ANALYSES

Site specific liquefaction analyses used a combination of:

- CLiq (Geologismiki), primarily to estimate free-field settlements, triggering behaviour and surface damage / crust thickness indices.
- Settle3 (Rocscience) to estimate total and differential building settlements under static, co-seismic and post-seismic conditions as part of an iterative soil-structure interaction analyses to support foundation design.
- In-house calculation sheets to evaluate bearing capacities and punching shear under liquefied conditions, and to determine the minimum raft thickness to resist punching shear and maintain foundation deformations in keeping with MoE guidance.

3.1 Triggering Assessment and Free-Field Settlement

Liquefaction analyses were undertaken in accordance with NZGS / MBIE in Module 3 adopting the Boulanger and Idriss (2014) analysis method, and simplified methods for calculation of liquefaction induced settlement in accordance with Zhang (2002). Inputs into CLiq included data sourced from six proximal cone penetration tests (CPTs) with three of these located within the building footprint. Mw and PGA were adopted from NSHM (Table 1), with groundwater taken as 2.3 m below ground level. Liquefaction triggering plots are included in Figure 1, which include comparisons between NSHM, Module 1 and TCC Bradley Study for future discussions. Our findings are summarised as follows:

- Liquefaction is not expected to occur under the SLS1 seismic case.
- Liquefaction is beginning to occur within portions of DS3 at PGA between 0.12 g to 0.15 g, albeit with low-likelihood and negligible surface effects (Liquefaction Potential Index (LPI) less than 5) and Liquefaction Severity Number (LSN) less than 1).
- From around 0.30 g liquefaction begins triggering in the shallower soil layers located just below the groundwater table (DS1 - within influence of foundations). The triggering behaviour is continuous, with LPI, LSN and calculated settlement within this layer increasing with increasing PGA.
- At NSHM SLS2 (0.38 g, comparable to the Module 1 ULS case of 0.39 g), liquefaction is predicted to trigger within DS1 from around 2.5 m to 4.5 m depth. Additional layers are also shown to trigger below 6 m depth (DS3). Free field settlements are predicted to range between 70 to 100 mm. The LSN range is narrow, between 8 and 13 predicting little to minor surface expression of liquefaction. When plotted on an Ishihara chart, the majority of CPTs show insufficient crust to prevent free-field surface expression (assuming minimum contributing layer thickness of 0.5 m to filter out 'thin layer' effects).
- Additional free field settlements are predicted as the PGA rises from NSHM SLS2 to ULS. Free field settlements of 85 to 115 mm are predicted to occur under NSHM ULS conditions. Calculated LSN is quite consistent, between 10 and 16, indicating minor surface expression of liquefaction damage and when plotted on an Ishihara chart provide insufficient crust thickness to prevent surface manifestation.
- If ground shaking exceeds NZHM ULS, a degree of additional total settlement may occur, due to additional denser sand layers liquefying within DS3. DS2 is resistant to liquefaction beyond ULS.

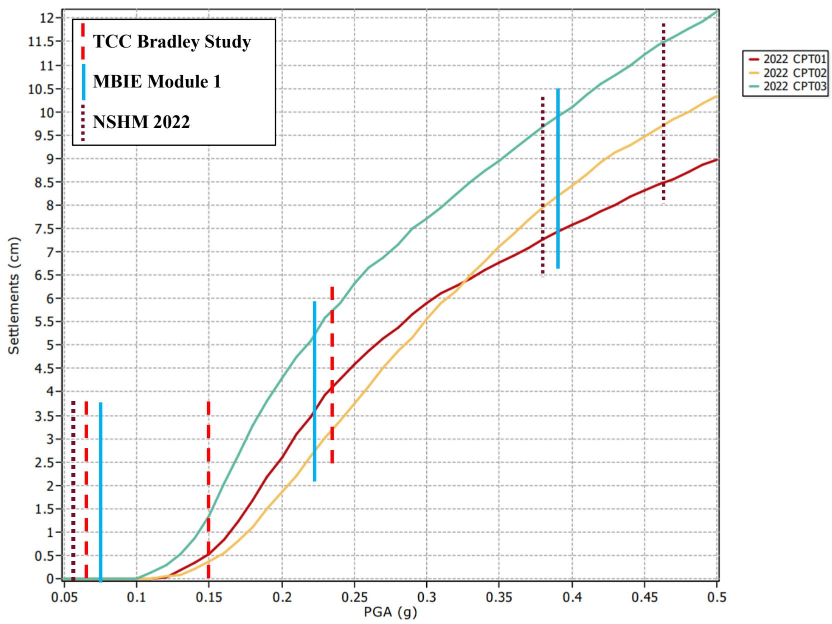

Figure 1: Parametric settlement analysis with SLS, SLS2 and ULS PGA values marked for comparison.

3.2 Building Induced Settlement Analyses (Soil Structure Interaction)

As LSN values are relatively low and quite consistent between test locations, we considered that a large portion of differential settlement or tilt experienced by the structure will be driven by building loading and the interaction of this loading with the liquefied soil, as opposed to variability in how free-field settlement expresses.

The soil structure interaction (SSI) effects have been considered using Settle3D software (Version 5.013). The analytical model for the pseudo static (liquefied) case comprises a 1 m thick dense engineered geogrid reinforced gravel raft underlain by: non-liquefied DS1 to 2.5 m, liquefied DS1 from 2.5 to 4.5 m depth, non-liquefied DS2 from 4.5 to 6 m depth, and liquefied DS3 from 6 m to 15 m depth. The top of alluvium (15 m depth) was taken as the base of the liquefaction model.

Under static conditions, DS1 is assumed to have an initial stiffness (Elastic Modulus (Es)) of around 20 MPa (accounting for compaction by earthworks plant), reducing to between 3% and 10% of this value below 2.5 m depth under liquefied conditions, or Es of 600 to 2,000 kPa. We also checked the residual liquefied shear strength ratio (Su/σv') of the liquefied soil column which is estimated to range between at 0.08 to 0.20 for DS1 and DS3, which resulted in an undrained shear strength (Su) ranging between 4 and 16 kPa. Appropriate soil stiffnesses for a material with this typical value of Su are around Es 800 to 3,200 kPa (calculated by multiplying the undrained shear strength by 250).

The above model and parameter assumptions were used to complete iterative modelling under varying design events (SLS1, SLS2 and ULS) with the project Structural Engineer, incorporating their foundation layout (grillage of at-grade ground beams typically 1 m wide) and pressures at multiple design stages to refine the soil structure response and achieve convergence between our models (i.e. where the foundation deflection and tilt from the structural model aligns with total and differential settlement in the geotechnical model). For the liquefied cases (SLS2 and ULS), structural foundation loading has been applied in the structural 'Y' direction (loading applied parallel to short axis), simulating an 'aftershock' or ground acceleration following the 'peak', an approach often requested by TCC and MoE.

Our iteration resulted in the following two co-seismic models:

- An SLS2 analogue where the geotechnical / Settle3 model includes DS1 liquefying from 2.5 to 4.5 m depth with Es of 900 kPa and DS3 liquefying from approximately 6.0 m depth with Es of 1,200 kPa. This correlated to a 1000 kN/m (1 kPa/mm) spring stiffness applied to the structural model to bring building deflection and ground settlement into alignment (~35 to 76 mm).
- A ULS analogue where the geotechnical / Settle3 model includes DS1 liquefying from 2.5 to 4.5 m depth with Es of 500 kPa and DS3 liquefying from approximately 6.0 m depth with Es of 750 kPa (accounting for the triggering of additional bands in both shallow and deeper liquefied sand layers on top of those already triggered at SLS2). This correlated to a 500 kN/m (0.5 kPa/mm) spring stiffness applied to the structural model to bring building deflection and ground settlement into alignment (~50 to 136 mm).

We consider the modelling completed to incorporate a more probable SLS2 response, where the structure would be expected to tolerate more than one event and be readily repairable in keeping with MoE design criteria, while the reduced stiffness ULS model captures a 'worst case' where the foundation system should not become the weak link in the structure. A stiffer model accounting for reduced triggering in shallow soil layers was not analysed, given it would correlate with an intermediary event more like a 1 in 150 year recurrence, sitting outside of MoE design criteria, and our triggering assessment indicates almost all the liquefaction predicted at PGA <0.3 g occurs below 6 m depth.

3.3 Punching Shear

Based on the supplied bearing pressures, foundations dimensions and loads, punching shear was considered a risk under SLS2 and ULS due to reduced bearing capacity from liquefaction. Accordingly, a geogrid reinforced raft was assessed to be required, and designed to provide adequate bearing capacity under the applied foundation pressures. The design of the raft, and influence of the geogrid reinforcement to distribute foundation loads, was assessed using the T-value calculation methods (Lees and Matthias, 2019), which adopts non-dimensional relationships between bearing capacity ratio and load transfer efficiency of the granular raft to capture the benefits of the geogrid. A 1.0 m thick compacted GAP65 raft with four layers of TX190 or equivalent geogrid was considered necessary to provide mitigation against punching failure.

The geogrid raft also offers a suitable subgrade for releveling and repairs after SLS2 and ULS events (satisfying SGR requirements), while also managing total and differential settlements across the foundation from both building induced and free-field settlement.

3.4 Results Summary

Table 3 presents a summary of the liquefaction design assessments. The results of the liquefaction and SSI analyses enabled the delivery of an SGR compliant design, preventing punching shear and enabling post SLS2 relevelling with differential settlement across the foundation system to be limited to <100mm. Note that the anticipated tilt / differential settlement was assessed across the building width, by taking the full differential settlement calculated from building loads and incorporating up to 50% of the free-field settlement expressing differentially across the foundation width for the upper end of the range.

Table 3: Summary of liquefaction assessments incorporated in design

	SLS (1 in 25 yr)	SLS2 (1 in 250 yr)	ULS (1 in 1000 yr)
Free-Field Settlement	Negligible	70 to 100 mm	85 to 115 mm
Building Induced Settlement	Negligible	35 to 76 mm	50 to 136 mm
Combined Free-Field and Building Induced Settlement	Negligible	~105 to 175 mm	~135 to 250 mm
Anticipated Tilt / Differential Settlement Across Building Width	Negligible	45 to 90 mm	70 to 140 mm
Liquefaction Severity Number (LSN)	0 (no surface expression)	8 to 13 (little to minor surface expression)	10 to 16 (minor surface expression)

4 DISCUSSION

4.1 Project Outcome

Given SLS2 under the NSHM is essentially comparable to Module 1 ULS, a robust design outcome was achieved for the project and both the assessment philosophy and outcome were supported by MoE. In our view this was a more onerous design requirement than Council typically processes. No queries were raised by Council during consent processing indicating that Council accept a more conservative stance being adopted.

Ultimately, the ground improvement solution allows for the structure to be readily repairable following the equivalent of a Module 1 ULS event and provides added resilience meaning it could even tolerate more than one ULS event in its lifetime. This provides futureproofing against some modifications to seismic code criteria.

It could be argued that importation of aggregate to the site was a less sustainable solution, however there was opportunity for the surplus fill (and fill associated with the platform undercutting) to be maintained on site and used for amenity (landscape filling) in a flood prone area around the perimeter of the new buildings. In addition, sit won sands were reused to backfill between ground beams to support the floor slab, reducing the export of soils offsite. During design iteration the structural beams were reduced in thickness, saving on concrete and steel costs.

While multiple changes in seismicity between Preliminary and Detailed design did cause project delays, MoE directed these hold points and key project delivery stages were adjusted accordingly. Local geotechnical consultants were already revising several projects previously assessed using the Bradley Study and thankfully most of the project teams were understanding of changing industry design guidance. Adding the requirements to review and apply the updated NSHM to this project was an appropriate decision and ultimately resulted in a desirable outcome for the project.

4.2 Implications of TS1170.5

A draft of TS1170.5 was released for comment in 2024. A large amount of work was carried out by the Seismic Risk Working Group and Standards New Zealand committee to determine how to integrate the NSHM into design practice, however the document is not intended to replace NZS1170.5:2004 and associated amendments but instead be considered an ‘alternative solution’ under B1/VM1. While the document does add more to support geotechnical considerations, the intention is to have geotechnical practitioners continue using Modules 1 through 3, and that these modules will be updated after TS1170.5 is published in final.

Both NZS1170.5 and TS1170.5 do not address how the performance of the ground and the interaction of the ground with foundations are to be considered in seismic design. As outlined in Section 10.1 of MBIE Module 3, there is no code requirement to consider intermediate limit states or to understand how damage is going to evolve between SLS and ULS. MBIE provide a brief outline of “holistic approaches” to evaluate seismic performance including step change behaviour however there remains to be no code requirements.

TS1170.5 currently leaves it up to industry to define how this should be done (a continuation of the current situation), which increases the value of this case study and others, which present a pathway to assess soil-structure interactions.

4.3 Lessons from MoE SGR

After working on several MoE projects across multiple regions in New Zealand It is the author’s opinion that the SGR provides useful and appropriate design guidance for geotechnical professionals assessing liquefaction step change behaviour and associated building performance and design resilience. The MoE DRP staged design review approach also assists with identifying and rectifying perceived non-conformances relatively early in the project life.

The guidance itself, which forces consideration of SLS2 effects, has a positive effect on building resilience / robustness which is not required by current codes and guidelines. This is especially apparent in lower-seismicity areas where SLS settlements or triggering may be minimal or expected to not occur at all, but these effects can be pronounced at SLS2.

There have been comments and uncertainties around whether SLS2 performance criteria will be added to the NZ Building Code in future revisions, and while the building code does request ‘robustness’ in solutions, it remains unspecific in how that should be assessed or applied in projects. This leaves a gap where specific design guidance such as the SGR, or local body Development Codes, may feel the need to step in and issue design guidance covering intermediate events. Alternatively, the use of intermediate cases or parametric triggering assessments could be covered more specifically in Modules 1 through 3, ideally also including guidance on soil-structure interaction.

4.4 Consideration for services in liquefiable soils

The authors are seeing increasing regulatory requirements being introduced and enforced by Territorial Authorities regarding land and building development. Much of the focus however remains to be associated with private buildings (residential, commercial and industrial) and critical infrastructure (pump stations, bridges etc.) however the authors recognise considerable inconsistencies with limited application to the design of non-critical public assets including lineal infrastructure (three waters utilities) or with regards to outbuildings.

In the author's opinion, linear infrastructure and external buildings providing critical services (wastewater, water supply, pump stations, fire suppression services) to a main building and/or for the wider land development should be treated as having the same or higher importance level as the main building to provide equivalent levels of resilience and post disaster serviceability and function. It is the geo-professional's role to assess and communicate project risk. This should include discussing resilience and robustness with building utilities designers and civil designers. These assessments and discussions would be aided by Territorial Authorities implementing learnings from the Christchurch earthquake sequence and subsequent rebuild in their development codes and assessment procedures.

5 CONCLUSIONS

A case study has been presented, adopting the MoE SGR approach, resulting in a robust and resilient school classroom development. Design challenges were introduced by changes in seismicity lead both by TCC and by MoE.

This case study, plus our work on multiple other MoE and private projects has highlighted the lack of clear industry guidance on how to approach seismic resilience in design, especially where there is a pronounced step-change in behaviour prior to ULS ground shaking. The SGR addresses this by setting SLS2 performance criteria.

It is increasingly clear that either an intermediate / SLS2 case with parametric triggering assessments should be undertaken on all projects to facilitate a clear discussion on resilience or robustness of design solutions. This would also allow for understanding of not only if a step change in behaviour could occur, but where that change in behaviour may occur, and therefore provide opportunity for consideration to design resilience and potential costs associated with achieving an increased resilience beyond the traditional and nominal SLS1 and ULS.

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