Interaction of geotechnical and structural engineering in the seismic assessment of existing buildings

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ABSTRACT: The performance of buildings in earthquake shaking is influenced by both the performance of the structure and that of the supporting ground in a process known as soil-foundation-structure interaction (SFSI). While strategies to assess the probable structural performance are becoming well established in engineering practice, a similar approach in the assessment of the supporting ground and SFSI is not so widely established. This paper explores the necessary interaction between geotechnical and structural engineering disciplines in the seismic assessment of buildings, considering the potential for an abrupt 'step change' in geotechnical behaviour, or in the absence of this, the potential beneficial influence of SFSI to the building life-safety performance. An integrated seismic assessment framework is proposed, relying on early interaction between the disciplines and risk screening to determine the anticipated seismic response. Future areas for research and improvement are suggested.

1 INTRODUCTION

The performance of a building during an earthquake is, in varying proportions, influenced by both its structure, foundation and the supporting ground. However, strategies which consider both aspects in the assessment of buildings for life safety are not very well-established in New Zealand practice, with many assessments undertaken without consideration of the underlying soils, beyond the assessment of the site soil class. This approach has the potential to result in outcomes that range from overly conservative to potentially unsafe (e.g. Cubrinovski and McCahon, 2011; Mylonakis and Gazetas, 2000).

This paper explores the necessary interaction between geotechnical and structural engineering disciplines in the seismic assessment of buildings, encompassing the identification of potential 'step changes' in geotechnical behaviour and considering the beneficial influence of soil-foundation-structure interaction (SFSI) on the building life-safety performance.

An integrated seismic assessment approach, relying on some early interaction of the structural and geotechnical disciplines is proposed. A number of geotechnical/foundation "failure" mechanisms are discussed, in particular those that have the potential to result in a behavioural step-change and the consequences for building seismic performance. The paper draws from recent learning from Christchurch and the authors' experiences in seismic assessment of various structures. Future areas for research and improvement are suggested.

2 HISTORICAL APPROACH

The seismic assessment of buildings has historically been undertaken by structural engineers with input from geotechnical engineers typically limited to the provision of site seismic class and in some cases assessment of bearing capacity. We believe that a more effective process would involve an early, collaborative approach between the structural and geotechnical engineer. However there are currently a number of barriers to this approach.

2.1 Budget Limitations

We recognise that there is often a constrained budget available for seismic assessment of existing buildings and consulting a geotechnical engineer will generally increase overall costs. However, savings on the initial phases of a seismic assessment can be a false economy as early spending on a collaborative assessment may lead to a reduction in rework and a more accurate and reliable outcome.

2.2 Differences in Perception

Due to the differences in responsibilities and perceptions between structural and geotechnical engineers, there can be disconnections in terms of the technical assessment and the treatment of uncertainties (CERC Vol. 1, 2011). It is also acknowledged that structural and geotechnical engineers may possibly have a different perspective of the same physical reality as illustrated in **Figure 1**.

Even the definition of "failure" differs between geotechnical and structural engineers. For example, foundation "failure" in a liquefiable ground does not necessarily imply collapse, but will invariably indicate excessive permanent displacements of ground or foundations resulting in significant repair or demolition of the building.

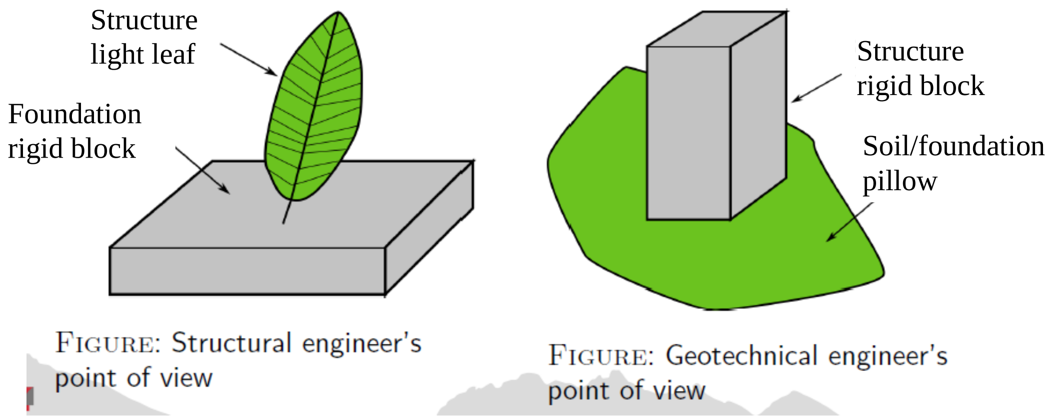

Figure 1. Different perspective of the same physical reality (Amended from S. Grange, 2013).

The above has resulted in the general perception, in the eyes of the structural engineer undertaking the assessment that the building performance will always be dominated by the structural behaviour.

2.3 Interaction between structural and geotechnical engineers

In the historic / conventional approach, geotechnical engineers would be provided a brief or scope of work, as outlined by the structural engineer and the client, for their involvement with the seismic assessment project. In this status quo arrangement, geotechnical engineers may be comissioned to provide input parameters to the structural engineers' assessment, or omitted from the assessment process altogether

In the proposed assessment approach, it is expected that the level of interaction, collaboration and communication between the structural and geotechnical engineers would significantly increase compared to the status quo. To be most effective, this interaction is required at the early stages of the assessment where most value can be realised. This may not necessarily lead to extra geoetchnical assessment costs as the need of physical testing and modelling will be more targeted. In some cases, the extent of the geotechnical investigation may actually reduce.

As part of this process geotechnical engineers are expected to be able to assist in the understanding of the overall soil-structure system behaviour, including exercising some judgment as to the "likely" geotechnical behaviour based on limited information, rather than simply providing design parameters.

Some existing guidelines such as ASCE-41 (2006) and the EAG's Detailed Engineering Evaluation (DEE, 2011) guidelines provide guidance to geotechnical/structural engineers to help them make those engineering judgments when dealing with existing structures.

3 GEOTECHNICAL PERFORMANCE OBJECTIVES

It is important to recognise that existing buildings may require a different set of acceptable performance criteria and objectives compared to newly designed buildings if the key outcome sought is life-safety risk reduction. The current geotechnical performance criteria for new building design may not be appropriate when applied to the seismic assessment of existing buildings. This is particularly true for scenarios where settlement under SLS loading governs the foundation design.

In new building design, it is appropriate to adopt a conservative interpretation of geotechnical parameters due to the inevitable uncertainties. However, a "probable behaviour" mind set may be more appropriate in seismic assessment.

While non-linearity in foundation behaviour may not be desirable in new structure design where it is appropriate to limit the risk of damage, some non-linearity in the foundation sub-structure may be acceptable from a life-safety preservation perspective and may be an acceptable mechanism to achieve energy dissipation in an existing building.

Therefore, the commonly applied geotechnical performance objectives will need reconsidering in the context of a seismic assessment, including permitting some non-linearity where appropriate.

We consider the approach adopted in ASCE-41 (2006) Chapter 4 to be a way forward where the acceptable performance for geotechnical behaviour is a function of the consequence of the geotechnical induced deformation / loads on the superstructure's performance. The concept of geotechnical "failure" being demand exceeds capacity, may need to be recalibrated to a displacement-based approach.

4 STEP-CHANGE BEHAVIOUR

4.1 Geotechnical 'Step Change'

The term 'Step Change' is commonly defined as "a sudden, discontinuous change". In the context of geotechnical seismic performance we propose to adopt this term to describe behaviour which results in a disproportionate increase in deflection or load bearing capacity with an increase in seismic shaking. This concept is important in seismic assessment as a step change in **geotechnical** behaviour may readily result in a step change in the **soil-structural system** behaviour i.e. a brittle rather than ductile response.

Consider two example structures supported on shallow foundations, both structures exceed their calculated bearing capacity at around 50% ULS seismic acceleration:

- **Building A** is founded on a soil which exhibits (in structural terms) **ductile** behaviour. While foundation bearing capacity 'failure' may occur, controlled yielding, damping and tolerable displacements allow the superstructure to perform adequately at shaking levels up to ULS.
- **Building B** is founded on a sensitive soil which exhibits (in structural terms) **brittle** behaviour. When foundation bearing capacity 'failure' occurs, the soil losses significant strength leading to uncontrolled / intolerable vertical settlement and lateral movement of the building. This may lead to a brittle failure of the superstructure and hence collapse of the building.

4.2 Soil-Structure System 'Step Change'

The overall soil-structure system behaviour is a function of geotechnical / soil behaviour, and superstructure behaviour. To assess the potential for step-change behaviour in the soil-structure system, one would need to assess the consequence of the geotechnical step-change behaviour to the structural integrity and stability of the building.

One key assumption is that a step change in soil behaviour does not automatically lead to a soil-structure system brittle behaviour. Conversely, if there is a step-change in soil behaviour leading to a brittle response in the structural performance, then it is crucial that the soil-structure interaction is adequately assessed as part of the seismic assessment. **Figure 2** presents a conceptual example of low-rise building on pile foundation to illustrate the relationship between the geotechnical step-change behaviour, the consequence of the step-change behaviour, and the soil-structure system behaviour.

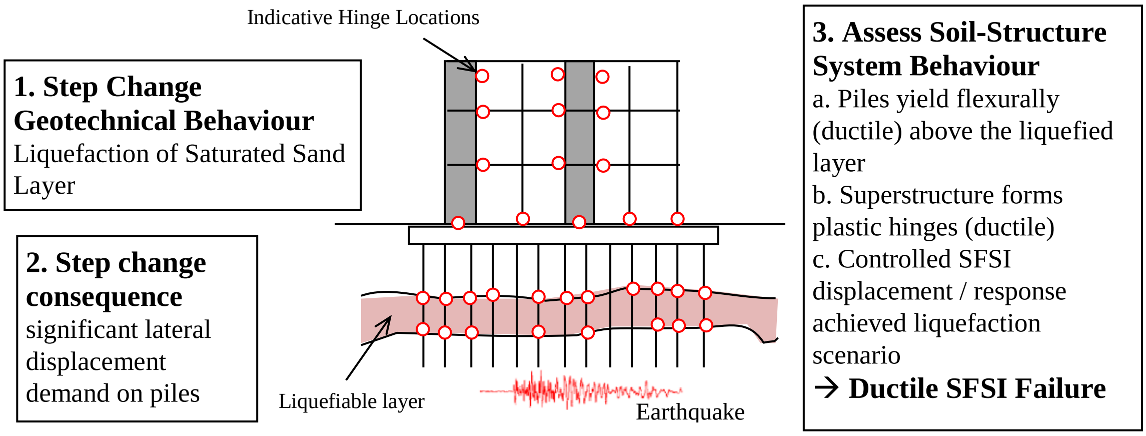

Figure 2. Relationship between step-change geotechnical behaviour and soil-structure system behaviour

In a simplistic but rational manner, the soil-structure system response can be categorised into two broad categories in terms of behaviour under seismic shaking:

4.2.1 *"Ductile" SFSI Response*

This is characterised by a ground or foundation "failure" mechanism that is relatively ductile where the lateral load carrying capacity of the overall soil-structure system is maintained while undergoing substantial lateral / plastic deformation. This behaviour is illustrated in the foundation system load-displacement capacity curves shown in Figure 3a. The plastic mechanism can either be within the substructure, the supporting ground or the interface mechanism e.g. rocking.

It is noted that 'ductile' soil-structural integrated systems have been long considered as part of the performance and displacement-based seismic design of new foundations (e.g. Wotherspoon *et al*, 2004; Blandon et.al., 2012).

ASCE-41 (2006) standards assume the foundation soils are generally not susceptible to significant strength loss due to earthquake loading, unless they degrade significantly in stiffness and strength under cyclic loading. An elasto-plastic load-displacement capacity curve is defined for most ground conditions.

In high rise construction, or in highly irregular structures where the dynamic response is likely to be non-linear, care is required as a SFSI response may not necessarily be beneficial.

4.2.2 *"Brittle" SFSI Response*

This is characterised by a ground or foundation "failure" mechanism that is relatively brittle where the load carrying capacity of the overall soil-structure system is 'lost' at relatively low lateral / plastic deformation. The brittle SFSI mechanism may be due to geotechnical mechanisms such as slope instability leading to significant deflection / force demand to the system, resulting in brittle structural response.

It can be expected that a brittle behaviour may lead to significant displacement/rotation of the soil-structure system, leading to overall instability or structural collapse. In the example shown in Figure 3b, although the anchor foundation system was observed to have a degree of residual tensile capacity at high displacement, a sudden loss in stiffness/strength will likely result in unreliable behaviour and excessive displacement demands on the superstructure.

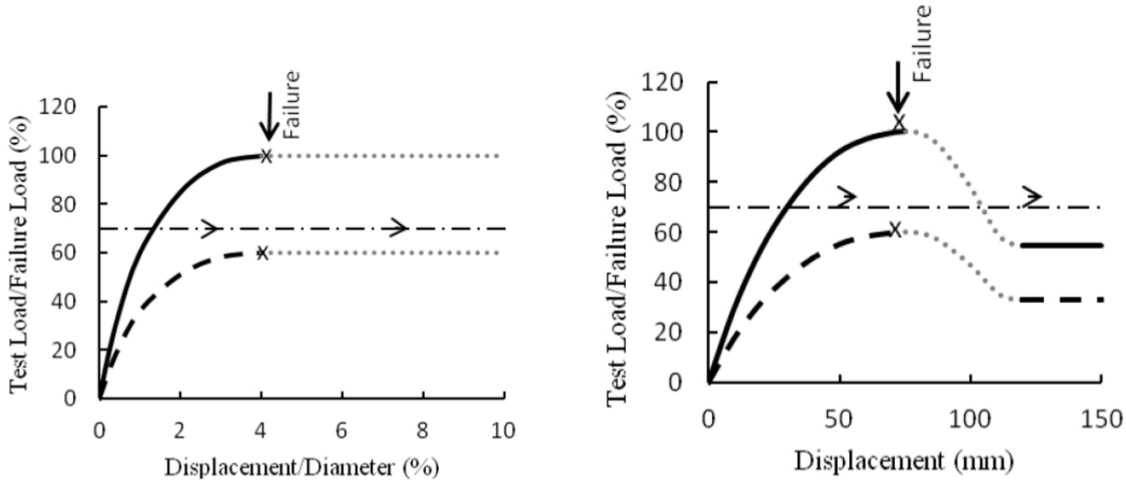

Figure 3. Soil-Structure Response Mechanism - Broad Characterisation: a) "Ductile" geotechnical behaviour – axial-compression load-displacement capacity curve of pile in cohesive soil; b) "Brittle" geotechnical behaviour – axial tension load-displacement capacity curve of anchor in Greywacke rock. (Figure is from Palmer, 2013).

4.3 Ductile and Brittle Soil-Foundation-Structure Interaction (SFSI) System Behaviour

The table below shows several potential examples of "ductile" and "brittle" soil-foundation-structure interaction (SFSI) system behaviour. Further analysis and research is required to provide broad measurable parameters for engineers to screen for "brittle" SFSI behaviour.

Table 1. Potential examples of "Ductile" and "Brittle" Response Mechanism of Soil-Structure System

"Ductile" SFSI system response (assume building is well-tied together)	"Brittle" SFSI system response
Shallow foundation rocking on soil	Global overturning leading to instability (**Figure** 5a)
Piles tension uplift with pile reinforcing yielding	Pile tension uplift with loss of end anchorage and skin friction capacity leading to global instability (**Figure** 5a)
Exceeding the bearing capacity of soil with a good post-yield behaviour e.g. sand (Blandon *et al*, 2012)	Significant differential settlement under poorly tied buildings i.e. unreinforced brick buildings or beams on shallow seatings untied.
Liquefaction-induced differential settlement under well-tied together structure (**Figure** 7a)	Instability of the building platform due to supporting slope instability
Foundation sliding on flat site	

5 PROPOSED APPROACH

5.1 Pre-Assessment Discussion

Fundamental to the proposed integrated approach is a pre-assessment discussion where the structural and geotechnical engineers work together to identify potential geotechnical hazards, the expected SFSI response mechanism and the level of sensitivity of the structure to the foundation behaviour. Screening for the potential for an abrupt behavioural step-change is seen as a key step in the initial assessment. As an outcome, the need for any geotechnical site investigation or testing can be determined.

5.2 Integrated structural and geotechnical seismic assessment

The following flowchart attempts to illustrate a suggested "integrated" structural and geotechnical approach for the seismic assessment of existing buildings. Potential step-change behaviour of the soil-structure system can be identified earlier and potentially beneficial SFSI effects can be more appropriately considered.

The simplistic flowchart below illustrates several possible scenarios:

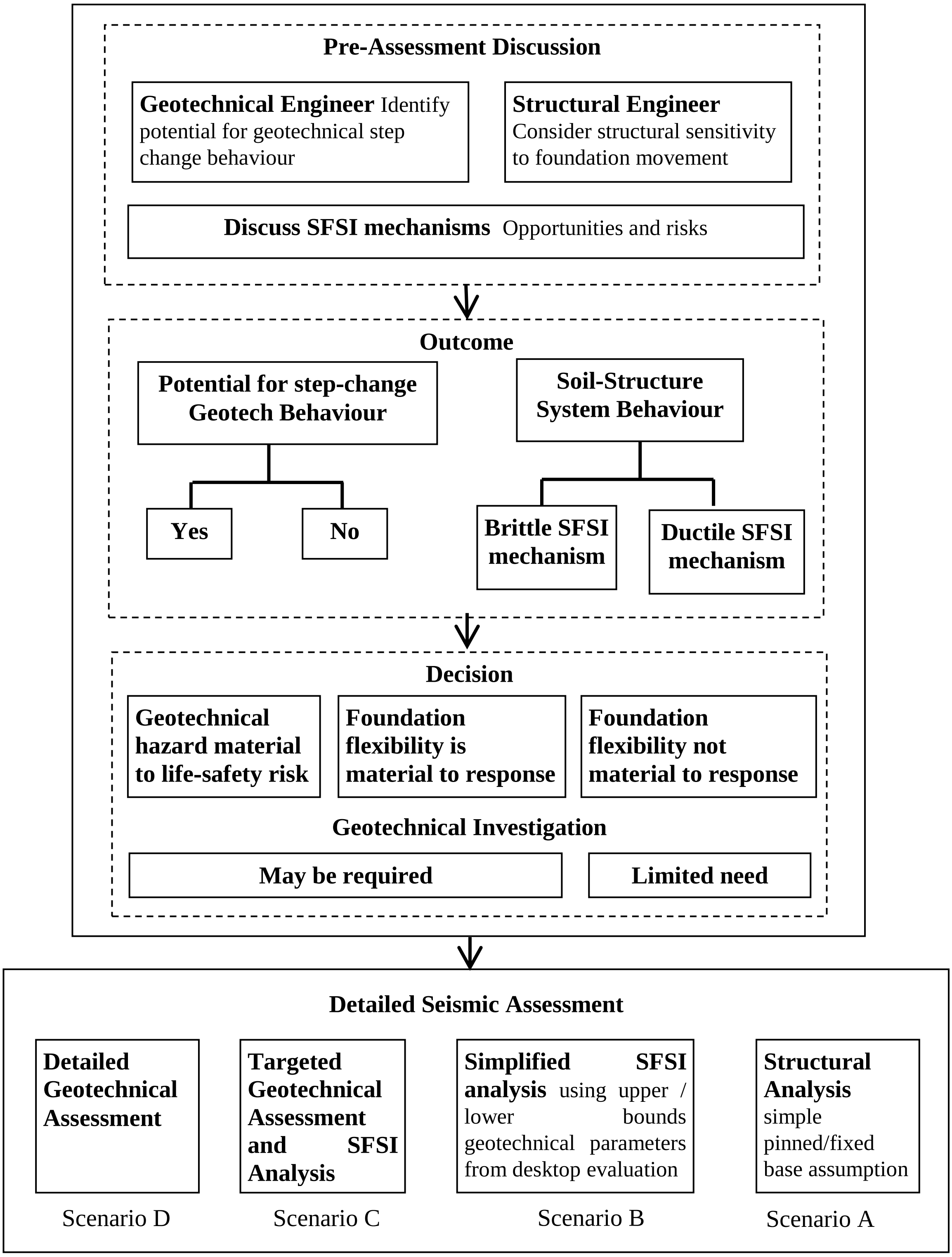

Figure 4. Integrated structural and geotechnical seismic assessment flowchart.

Scenario A: A ductile mechanism was identified for the SFSI system, and foundation flexibilty was considered not material to superstructure performance. Limited geotechnical input is required after the initial assessment. **E.g. Low-rise well-tied buildings on shallow foundation on "good ground".**

Scenario B: A ductile mechanism was identified for the SFSI system and foundation non-linearity was considered material to superstructure performance. A simplified SFSI analysis was completed with upper and lower bound geotechnical parameters (e.g. spring stiffness, p-y curves). **E.g. reinforced concrete wall building on shallow strip footings with potential for rocking or sliding.**

Scenario C: A brittle mechanism was identified for the SFSI system material to the superstructure performance.Site-specific geotechnical investigation / assessment was completed to confirm geotechnical parameters **E.g. Loss of pile capacity and over-turning to tall slender shear wall.**

Scenario D: A geotechnical step-change mechanism was identified that was considered a life-safety risk irrespective of the structural response. Detailed geotechnical assessment, including site-specific geotechnical investigation was completed. **E.g. Slope instability that would result in complete loss of the building platform.**

Section 6 and 9 provide examples from earthquake observations and recent seismic assessment projects to illustrate the proposed approach.

6 GEOTECHNICAL "FAILURE" AND BUILDING PERFORMANCE

6.1 Lessons from Past Earthquakes

Reconnaissance reports of past earthquakes confirm that the seismic performance of building can be significantly influenced by the geotechnical performance of the supporting ground. Buildings have collapsed or been significantly damaged due to either foundation (shallow or deep) "failure" and/or liquefaction-induced settlements. Similarly, there are buildings that could have collapsed but have not due to the beneficial effect of the SFSI.

Figure 5 shows overseas examples of a) building collapse b) brittle pile shear failure, both as consequences of ground liquefaction and foundation failure from the 1964 Niigata earthquake. Both mechanisms would not have been identified by an engineer undertaking a simple pinned/fixed-based structural analysis. It is noted the level of understanding of liquefaction risk was minimal at the time of 1964 Niigata earthquake.

The building in **Figure 5**b remained in service for 20 years after the earthquake despite the hidden shear failure of the piles, illustrating the difficulty in predicting foundation performance and identifying foundation damage post-earthquake (Yoshida and Hamada, 1991).

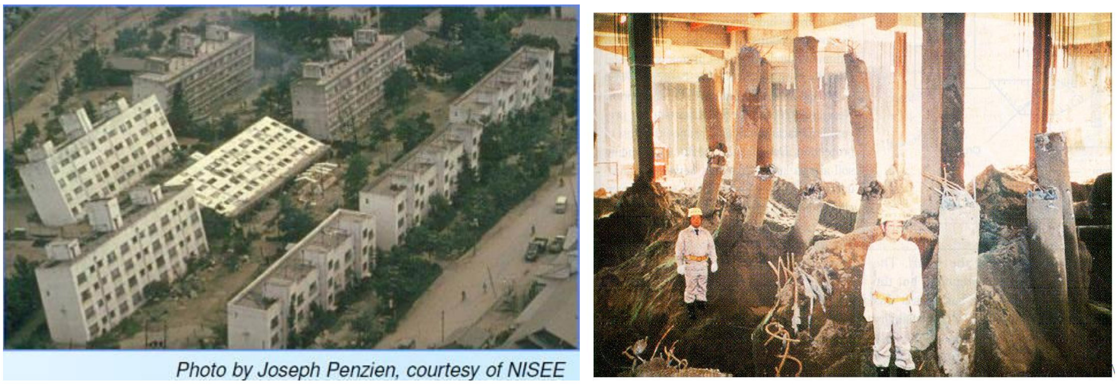

Figure 5. Significant building damage and collapse due to ground failure: a) Niigata 1964; b) Pile shear failure, observed in an excavation 20 years after the 1964 earthquake (image from Yoshida and Hamada, 1990).

There are several notable examples where the geotechnical foundation system step-change behaviour had led to a brittle failure mode in the sub-structure and super-structure. **Figure 6** illustrates a 5-storey building example from the Christchurch earthquake (Kam *et al*, 2012). The site (Madras St) showed evidence of moderate liquefaction surface manisfestation.

The foundation of the core wall on the southern elevation has lost its bearing capacity, possibly during or after the earthquake event, and the wall had settled about 450mm vertically. The settled core wall appeared to have pulled the floor slab and the rest of building towards it. The external ground beam connected to the wall, and a number of frame beam-column joints had failed in a brittle shear mechanism (c), likely to be a consequence of both seismic shaking and induced vertical displacement demand from the wall's foundation failure. The building's lateral load system was severely comprised due to the foundation-wall system failure and it partially collapsed in a subsequent aftershock.

(a) (b) (c)

Figure 6. 5-storey building with shallow foundation failure beneath core walls: a) plan; b) south-east elevation; c) shear failure of ground beam connected to shear wall (Adopted from Kam *et al*, 2012)

However, liquefaction-induced ground failure did not resulted in any direct fatalities despite the widespread damage to residential and commercial buildings in the Central Business District (CBD) in the Christchurch earthquake (Cubrinovski and McCahon, 2012; Murahidy *et al*., 2012). Rock fall and landslides at the fringe of the city however has resulted in 5 fatalities (Dellow et al., 2011).

A similar conclusion can be drawn from the 14 representative buildings studied by the Canterbury Earthquakes Royal Commission (CERC Vol 2, 2012). While ground failure (e.g. liquefaction) and foundation damage were observed at a number of sites (e.g. Townhall, Police HQ, and 100 Armagh St Apartments), these buildings have generally satisfied the life-safety performance required by the New Zealand Building Code. However, the economical repairability of these buildings is an on-going debate.

Figure 7 presents several examples of significant building residual deformations due to foundation "failure" observed in Christchurch CBD (Kam *et al*, 2012). As a general observation of building performance in Christchurch, if the superstructure was robust (well-tied together), integral and responding in a ductile manner, foundation failure would excacerbate the inelastic demand on the superstructure's plastic hinges such as those shown in **Figure 7** but may not necessarily result in a uncontrolled displacement response.

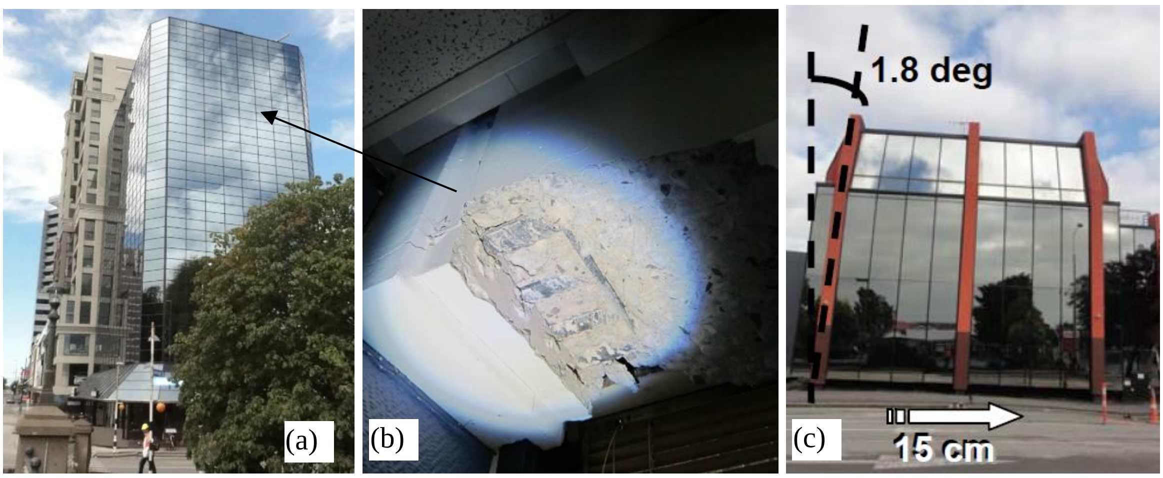

Figure 7. Building Foundation "Failure": a-b) 1980s high rise on basement and raft foundation; with beam plastic hinges observed throughout the building ; c) 1980s low rise on shallow foundation with significant differential settlement and sliding movement (photo from Cubrinovski and McCahon, 2012).

7 INITIAL SOILFOUNDATION-STRUCTURE INTERACTION (SFSI) ASSESSMENT

As discussed in Section 5.1, it is anticipated that any initial SSI assessment is undertaken jointly by suitably qualified structural and geotechnical engineers for most building types, except for low risk structures where an experienced structural engineer may complete the initial SFSI assessment (e.g. importance level 1 buildings, low-rise modern buildings with existing geotechnical information).

Without undertaking an integrated review of the potential geotechnical risks and SFSI mechanisms, the consequence of the potential step change behaviour may not be identified or its consequences correctly assessed.

A preliminary triage checklist of potential geotechnical step change behaviour is provided in the table below:

Table 2. Step-Change Geotechnical Behaviour Check List [Work in Progress]

Potential Step-Change Geotechnical Behaviour	Implication to the Soil-Structure System	Preliminary Recommendations as part of the Initial Geotechnical Assessment
Slope: Seismically-Induced Slope Instability (excludes liquefaction effects)		
Underslip	Instability within the building platform may reduce or remove vertical support to the structure or apply lateral load/impose displacements on foundations	a) **Desktop based assessment** utilising information on existing geology and geotechnical data, rules-of-thumb for behaviour of local soils, and where available historical records such as cut and fill records and historic stereo aerial photographic pairs. b) **Site visit** including visual assessment of evidence of incipient instability (e.g. cracking or unevenness in structure / roads / ground surface). c) **Preliminary stability assessment** (e.g. such as the use of Hook and Bray stability charts) considering back analysis where appropriate for calibration purposes. If the approaches above do not demonstrate that slope instability is unlikely, then a refined geotechnical assessment is warranted. For overslip - rock fall scenario, the desktop assessment would include looking for evidence of historic rock fall e.g. boulders at the toe of a slope on historic aerial photographs. The site visit and the preliminary assessment would include collecting data on rock type, defect persistence and orientation, and block size.
Overslip –Soil / Rock (Rock fall)	Instability within cut or natural slopes above the building platform may apply lateral load/impact on/inundate the building. In this context rock fall is also considered an overslip. A fundamental consideration for overslip failure is the potential for a large rock or volume of material with sufficient inertia to impact and cause structural collapse.	
Ground: Liquefaction/Cyclic Softening/Lateral Spreading		
Liquefaction	May result in loss of bearing / skin-friction capacity, excessive settlements, loss of lateral restraint, large cyclic ground movements, loss of confinement and buoyancy of light	**a) Preliminary assessment** utilising: i) Records of historical performance and any historic records such as nature of reclamation filling, geologic maps, nearby existing investigations (soil strength and groundwater

<table>
<tr><td></td><td>or deep structures.

The consequence for the structure of liquefaction must be considered, specifically considering the potential for brittle structural behaviour e.g. loss of seating for precast floors or URM wall collapse.</td><td rowspan="3">levels) and past reclamation. And ii) local rules of thumb for soil behaviour, in particular the potential for liquefaction or sensitive soils that may be subject to cyclic softening. Refer to NZGS Module 1 and ASCE-41 (2006) Table 4-1.

If warranted following site inspection, and where sufficient information is available, undertake limited calculation to review the potential of the crust to “raft” above the potentially liquefiable layer.

Other strategies for assessing the potential impact of liquefiable ground, with and without lateral spreading, are provided in NZGS Module 1, ASCE-41; Cubrinovski, 2006; and ATC-83 guidelines.</td></tr>
<tr><td>Lateral spreading due to liquefaction</td><td>May result in excessive ground displacements caused by spreading of liquefied soils in sloping ground and in waterfront areas. Lateral spreading may also occur in seemingly ‘flat’ ground up to hundreds of meters away from a slope or free face.</td></tr>
<tr><td>Cyclic softening of weak and sensitive clay-like soil</td><td>Cyclic strain softening has the potential to result in similar mechanisms to liquefaction, resulting in significant ground deformation and foundation failure (Boulanger and Idrisss, 2007).</td></tr>
<tr><td>Very soft subsoil e.g. marine clay, reclaimed land</td><td>Dynamic amplification and soil-structure interaction may occur, resulting in unexpected soil-structure system behaviour.</td><td>Preliminary soil-structural analysis is undertaken with assumed upper and lower bound geotechnical parameters based on a desktop assessment. This will indicate whether a more refined geotechnical investigation is warranted.</td></tr>
<tr><td colspan="3">Site Geological Hazard: Fault rupture (other risks e.g. tsunami & inundation to be included)</td></tr>
<tr><td>Fault Rupture</td><td>May impose large displacement and acceleration demands on structures in the immediate vicinity of the fault rupture zone.</td><td>Refer to published fault study for the region, including geologic maps and GNS Active Fault database.</td></tr>
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8 GUIDELINES FOR GEOTECHNICAL SEISMIC ASSESSMENT

The **New Zealand Society for Earthquake Engineering (NZSEE)’s seismic assessment guidelines (2006).** No specific recommendation for geotechnical seismic assessment is provided except that confirmation should be sought that the foundations are capable of developing the strength of the superstructure. It is understood within the next revision of the NZSEE guidelines, it is proposed to provide further guidance on how geotechnical seismic assessment can be integrated with the structural assessment.

ASCE-41 (2006), the American Standard for seismic evaluation of existing buildings has a chapter on “Foundation and Geologic Site Hazards”. ASCE-41 recognises that the acceptability of the behaviour of the soil and foundation system depends primarily on the effect of the deformation on the structure, which in turn depends on the desired Structural Performance Level. As such, the failure of the ground may not necessary be governing.

ASCE-41 also includes a number of useful recommendations to determine foundation load-deformation characteristics (soil strength and stiffness), post-liquefaction assessment and how to consider soil-foundation-structure interaction (SFSI) effects. While the parameters set out in ASCE-41 are potentially useful for performance-based seismic assessment, we consider some of the requirements to be very stringent and the exercise may be simplified for most common buildings in New Zealand.

Engineering Advisory Group (EAG) Detailed Engineering Evaluation (DEE) Guidelines (applicable to Canterbury): Table 4-2 sets out soil and foundation damage assessment criteria and when to "trigger" geotechnical inputs. The DEE guidelines also provide recommendations for a minimum level of site-specific ground investigation. In the absence of site-specific geotechnical data, it is recommended the structural assessment is heavily qualified as the sub-soil profile across the Canterbury plains can be highly variable.

The DEE guidelines appear to be focussed on assessing and mitigating liquefaction and ground damage for both SLS and ULS. Some limited guidance to undertake quantitative geotechnical assessment is also provided.

The **New Zealand Geotechnical Society (NZGS)** *"Geotechnical earthquake engineering practice – Module 1"* provides guidance on geotechnical assessment of liquefaction hazard and ground motion parameters, primarily for new building design. There is no allowance given for geotechnical assessment of existing structures at this stage.

The **Canterbury Earthquakes Royal Commission (CERC)** investigation has recognised the lack of guidance on acceptable foundation settlement for ULS and SLS (CERC Volume 1, 2011). CERC recognises that acceptable deformations of the foundations should be a function of the consequential structural ductility demand and 'desired' structural response.

NZBC B1/VM4: Foundation: NZBC B1/VM4 outlines performance criteria for foundation design and excludes a number of "brittle SFSI" scenarios including foundation on loose sand, saturated dense sand and on cohesive soil with sensitivity greater than 4. NZBC B1/VM4 is written primarily for new structure design.

9 CASE STUDIES / EXAMPLES

9.1 Low-rise reinforced concrete walls on basement foundation on liquefiable flat ground

Figure 8a shows a twostorey plus mezzanine and a one-level basement building. It is a pre-1935 reinforced concrete walls structure with cast-in-situ floor and roof slabs. The structure in isolation was assessed to be an earthquake-risk building in terms of expected seismic performance for life safety and avoidance of collapse in a 1 in 500-year design level earthquake. However, the building is susceptible to liquefaction-induced settlement and lateral spreading which is a geotechnical step-change behaviour. So this was assessed to determine whether it would lead to a brittle SFSI mechanism.

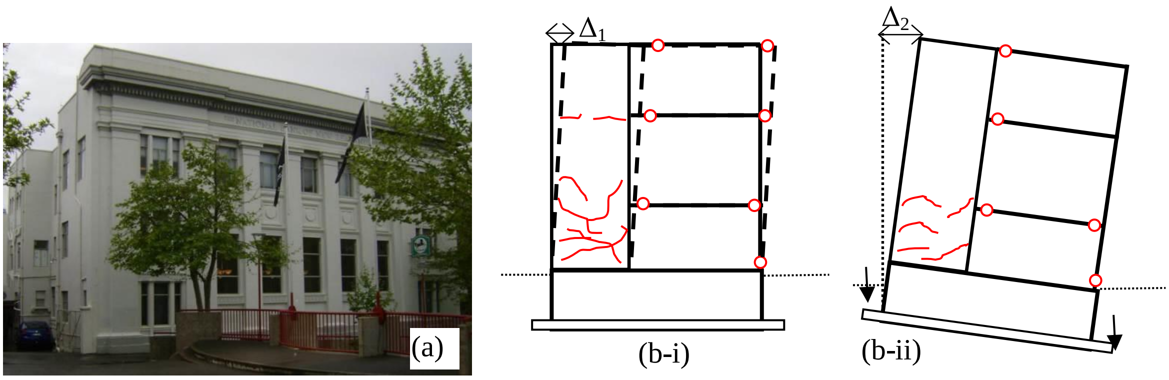

Figure 8. a) Low-rise reinforced concrete walls building on liquefiable flat ground; b) Dual wall-frame structure with rigid basement wall: i) Fixed based assumption ii) SFSI likely behaviour

The life-safety risk from liquefaction-induced ground failure (differential settlement and lateral spreading) was considered to be limited as the building superstructure and foundation structure are appropriately tied together to be able to accommodate the potential ground deformation. Additional distress on the structural members is expected to be limited as the building is more likely to settle or rotate as a "rigid block" (Figure 8b). Global overturning collapse is unlikely due to the squat nature of the building.

A simplified SFSI analysis was undertaken with upper and lower bound geotechnical parameters (e.g.

to determine most adverse consequences from probable range of differential settlements due to liquefaction) and step-change scenarios (liquefaction settlement occurs or not). A desktop-based geotechnical assessment was sufficient.

It was noted that any ground remedial work in order to mitigate the effect of liquefaction-induced settlement and lateral spreading was anticipated to be either very costly or ineffective unless a large scale area-wide intervention is considered. Thus, the recommendation to minimise life-safety risk was to strengthen the building structure in order to improve its ability to plastically deform. The client was advised and accepted that in the event of a "large" earthquake, the building may be inaccessible, subject to loss of serviceability and may not be economical to repair due to the liquefaction-induced damage.

9.2 **Wharf on piles foundation**

The wharf is supported on piles penetrating marine clay and founded on rock at a depth of 15-25m. The superstructure comprises a reinforced concrete deck on transverse secondary beams supported on primary girders that span between precast concrete driven piles. The wharf derives lateral stiffness from the piles and frame action. The piles essentially cantilever out of the seabed and are semi-rigidly fixed to the wharf at the top.

In the initial assessment, a number of potential geotechnical step-change behaviours were identified and several potentially brittle modes in the structure. Amongst the mechanisms considered:

- Instability of the marine clay resulting in significant slip and lateral deformation.
- Abutment and sea-wall slipping towards the sea
- Pile shear failure
- A loss in stiffness within the reinforced concrete frame structure of the wharf, including pile-deck joint softening, pull-out of plain round bars, anchorage failure of the reinforcing in waler and diagonal bracing members.

A decision was made in the initial assessment to undertake site-specific ground investigations and model the SFSI as part of the seismic assessment.

A destructive investigation of the existing wharf structure was completed. Some geotechnical investigation was undertaken, in addition to depth soundings to map the seabed profile. A 3D non-linear computer model of the wharf, including the various piles, braces, and the soil profile, was assembled for the analysis (Figure 9).

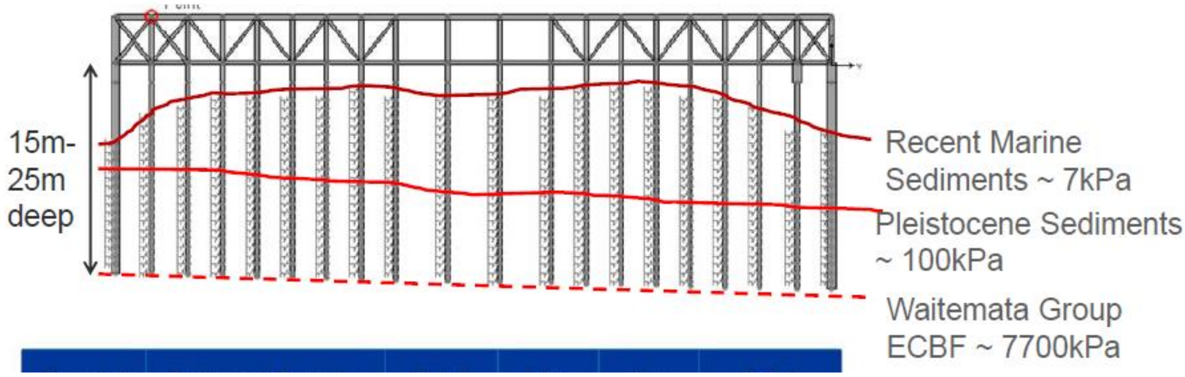

Figure 9. Transverse cross-section of the soil-structure interaction model of the wharf.

Non-linear static pushover and elastic dynamic analyses, with a sensitivity test of the critical input parameters was undertaken. The bounded SFSI analysis allowed a more thorough consideration of the influence of the soft marine clay and the varying seabed profile on the pile curvature demand. The consideration of non-linearity in the marine clay and selected structural members allowed a more reliable assessment of the overall system displacement capacity of the wharf.

10 FUTURE RESEARCH

10.1 Reliable load-displacement monotonic and cyclic "hysteresis" curves

There are a lot of uncertainties associated with understanding the plastic and cyclic behaviour of various soil-structure systems, in particular where geotechnical step-change behaviour may be occur. Even where good subsoil geotechnical data is available, geotechnical engineers would still expect a high level of uncertainty in the predicted load-displacement relationships, especially for loads beyond traditional design failure / ULS (e.g. Palmer, 2013; McManus and McCahon, 2011).

Similarly, there is significant challenge to be able to characterise the damping contribution in the soil-foundation system. A number of experimental research is underway at the University of Auckland, including field testing of actual bridge piers.

The challenge is therefore for the research fraternity to progress our ability to determine appropriately bounded load-displacement monotonic and cyclic "hysteresis" curves in seismic loading conditions for various soil and foundation systems. There is a large body of research presently underway to understand the seismic displacement capacity of shallow (e.g. Wotherspoon *et al.*, 2004; Blandon *et al*, 2012) and deep (e.g. Shirato *et al*, 2009) foundations. This research could lead to further development in displacement-based seismic assessment and design.

10.2 Acceptable performance criteria

A consensus document that provides probable behaviour performance criteria for geotechnical seismic assessment is required. As discussed in Section 3, modified performance objectives are required that may permit a degree of foundation damage and non-linearity as part of a "ductile" SFSI mechanism. Furthermore, the guidance document may need to establish a global displacement limit state and seismic assessment philosophy for use in integrated SFSI assessment.

11 CONCLUSIONS

We have improved our understanding of the interaction of geotechnical behaviour and seismic performance of buildings from recent research and post-earthquakes observation. We propose an integrated approach to seismic assessment of existing buildings that brings together the geotechnical and structural engineering disciplines.

In contrast to new building design, we consider the seismic assessment of existing buildings requires a different set of geotechnical performance objectives if the key outcome sought is a life-safety risk reduction. In particular, the geotechnical design parameters would need to be displacement-based and to accommodate for non-linearity in the foundation system or in the supporting soil.

Fundamental to the proposed integrated approach is a pre-assessment discussion where the structural and geotechnical engineers work together to identify potential step-change geotechnical behaviour and the expected SFSI response mechanism, and potential beneficial or adverse scenarios.

We recognise the importance of identifying any step-change geotechnical behaviour and the nature of the soil-structure system behaviour at the start of the process in order to appropriately determine the level of geotechnical investigation and SFSI analysis required. A working draft framework to identify and assess step-change geotechnical behaviour is provided.

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REFERENCES

ASCE-SEI-41-06. 2007. *Seismic rehabilitation of existing buildings. ASCE standard ASCE/SEI 41-06.* American Society of Civil Engineers (ASCE), Reston, VA

ATC-83. 2012. *Soil-Structure Interaction for Building Structures. NIST Report GCR 12-917-21.* National

Institute of Standards and Technology (NIST), Gaithersburg, MD.

Blandon C.A., Rave-Arango J.F., Ortiz A.R., Smith-Pardo J.P. 2012. Seismic Displacement Capacity of Vulnerable Shallow Foundations. In *Proc. Of the 15th World Conference on Earthquake Engineering*. 24-28 September, 2012. Lisbon. Paper No. 649

Boulanger R.W. and Idriss I.M. 2007. Evaluation of cyclic softening in silts and clays. *Journal of Geotechnical and Geoenvironmental Engineering.* Vol **133** (6): 641-652.

CERC. CERC Final report - Volume 1: Seismicity, soils and the seismic design of buildings. Canterbury Earthquakes Royal Commission (CERC). Aug 2012. 672 pages.

CERC. CERC Final report - Volume 2: The performance of Christchurch CBD Buildings. Canterbury Earthquakes Royal Commission (CERC). Aug 2012. 236 pages.

Cubrinovski, M. 2006. Pile response to lateral spreading of liquefied soils. *Proc. of the New Zealand Geotechnical Society (NZGS) Biennial Symposium: Earthquakes and Urban Development*. Nelson, New Zealand Vol **31**(1), 127-142.

Cubrinovski M., McCahon I. 2012. *CBD Foundation Damage - a report for the Natural Hazards Research Platform.* Feb 2012, Christchurch. University of Canterbury. 32 pages.

Cubrinovski, M. and McCahon, I. (August 2011). *Foundations on Deep Alluvial Soils*. Christchurch, New Zealand: Canterbury Earthquakes Royal Commission

Dellow G. et al. 2011. Landslides caused by the 22 February 2011 Christchurch earthquake and management of landslide risk in the intermediate aftermath. *Bulletin of the New Zealand Society of Earthquake Engineering, Special Issue* **44**(4): 227-238.

FEMA-440 *Improvement of inelastic seismic analysis procedures, FEMA 440*, Federal Emergency Management Agency, Washington, D.C., 2004

Grange S. 2013. Simplified modelling strategies for soil-structure interaction problems: The macro-element concept. Presentation to the ALERT Doctoral School. 5 October 2013, Aussois, France. Available online at: http://alertgeomaterials.eu/data/school/2013/session3/ALERT2013_Doctoral%20School_Grange.pdf

Kam, W.Y., Pampanin, S. and Elwood, K. 2011. Seismic Performance of Reinforced Concrete Buildings in the 22 February Christchurch (Lyttleton) Earthquake. *Bulletin of the New Zealand Society of Earthquake Engineering, Special Issue* **44**(4): 239-279.

Mylonakis, G., Gazetas, G. 2000. Seismic soil-structure interaction: beneficial or detrimental? *Journal of Earthquake Engineering*, Vol **4**(3): 277 — 301

Murahidy, K.M., Sleight A.F, Sinclair T.J.E. 2012. Christchurch CBD: Lessons learnt and strategies for foundation remediation – 22 February 2011 Christchurch, New Zealand, Earthquake. In *Proc. Of the 15th World Conference on Earthquake Engineering*. 24-28 September, 2012. Lisbon. Paper No. 4666

NZGS. 2011. *Geotechnical earthquake engineering practice. Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards.* New Zealand Geotechnical Society (NZGS). 28 pages.

NZSEE. 2006. *Assessment and improvement of the structural performance of buildings in earthquakes*. New Zealand Soc. for Earthquake Eng. (NZSEE), Wellington, New Zealand.

Palmer S. 2013. Strength reduction factors for foundations and earthquake load combinations including overstrength factors. In *Proc. Of the 2013 NZSEE Conference*. Wellington. Paper No. 1

Sancio, R.B., Bray, J.D. Durgunoglu T., & Onalp A. 2004. Performance of buildings over liquefiable ground in Adapazari, Turkey. In *Proc. Of the 13th World Conference on Earthquake Engineering*. 1-6 August, 2004. Vancouver, Canada. Paper No. 935

Shirato M., Kohno T. and Nakatani S. 2009. Geotechnical criteria for serviceability limit state of horizontally loaded deep foundations. *Geotechnical Risk and Safety – Honjo et al. (eds)*. Taylor & Francis Group, London.

Wotherspoon L., Pender, M., Ingham J. 2004. Combined modelling of structural and foundation systems. In *Proc. Of the 13th World Conference on Earthquake Engineering*. 1-6 August, 2004. Vancouver, Canada. Paper No. 411

Yoshida, N. & Hamada, M. 1990: Damage to foundation piles and deformation pattern of ground due to liquefaction-induced permanent ground deformations, *Proc., 3rd Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction*, San Francisco, CA, pp. 147-161