Research-informed Advancements in Guidelines and Standards of Engineering Practice for Natural Hazards

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Organisation: University of Canterbury

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Executive Summary

Guidelines and Standards of Engineering Practice ("standards" herein) underpin the implementation of new knowledge into engineering practice. They provide the principal mechanism for improving the performance of the built environment against natural hazards, and are also fundamental to achieving long-term resilience of New Zealand society. Despite the recognition of the important long-term role of standards, their foremost role is to summarise the current-state-of-practice and aid engineers in assessment and design. For the purpose of design and evaluation, all standards must address aspects of infrastructure performance which are not fully understood, necessitating judgements based on limited empirical knowledge. All engineering standards related to performance evaluation of land and the built environment under natural hazards, with no exception, encounter this challenge and have a range of issues that need immediate attention.

Recognizing the above need, the presented comprehensive study in this report focussed on using NHRP-supported research to address key issues in NZ standards and guidelines that require research-based resolution and advancement. In doing so, this research programme aimed to employ research as an essential tool for advancement of standards and subject design procedures. The intention was also to provide a mechanism for systematic implementation of research into engineering standards and practices, with the ultimate goal to reduce the gap between state-of-the-art and state-of-practice.

The presented research consists of six objectives, which while inter-related, reflect the main thematic areas associated with assessment and design of the built environment. These six Objectives are:

(1) Hazard demands and performance objectives
(2) Geotechnical hazards and infrastructure
(3) Seismic assessment and improvement of existing buildings
(4) New buildings: system-level interactions
(5) Transportation infrastructure
(6) Tsunami hazards and impacts on coastal infrastructure

The research priorities within each objective were developed on the basis of deep collaboration with regional and national stakeholders and end-users, as well as over-arching documents such as the Canterbury Earthquakes Royal Commission reports and recommendations. This project does have a large emphasis on earthquake-related hazards, given that seismic demands commonly control the design and evaluation of large-scale infrastructure in NZ. However, several objectives also pertain to research which addresses other natural hazards.

The report is organized in six chapters, one for each objective. Several research tasks (topics) were considered within each objective, and each topic targeted a specific problem of interest, and aimed at either development or an update of a specific guideline or standard of engineering practice.

This report provides a brief summary of the research studies and highlights at a high-level key research aspects and outputs, but excludes technical details. A comprehensive summary of the research methodology and outputs is provided in the journal papers listed in the Outputs and Dissemination sections for each topic.

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Objective 1: Seismic Loading and Performance Objectives

Task 1: Seismic Design Spectra and Ground Motion Records

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ABSTRACT
Task 1 of Objective 1 focused on two primary aspects: (i) the advancement and translation of leading ground motion selection methods used in research into practice; and (ii) improved representation of ground motion prediction through examination of case history data and development of modification factors. The Generalized Conditional Intensity Measure (GCIM) method (developed by the PI in 2010) was advanced through the consideration of specific scenario earthquakes, causal parameter bounds, near-fault effects, and epistemic uncertainties – now completing all features necessary to enable it to be routinely adopted in practice; and the method has been used in four major consulting projects over the course of this funding. Observed ground motions from the 2016 Kaikoura earthquake have been extensively examined to understanding the salient phenomena responsible for the anecdotally larger-than-anticipated ground motion amplitudes in Wellington city that caused large damage to structures. On the basis of several technical papers, the systematic nature of the effects have been translated into recommended wellington-specific amplification factors for use in NZS1170.5: 2004 Amendment 2.

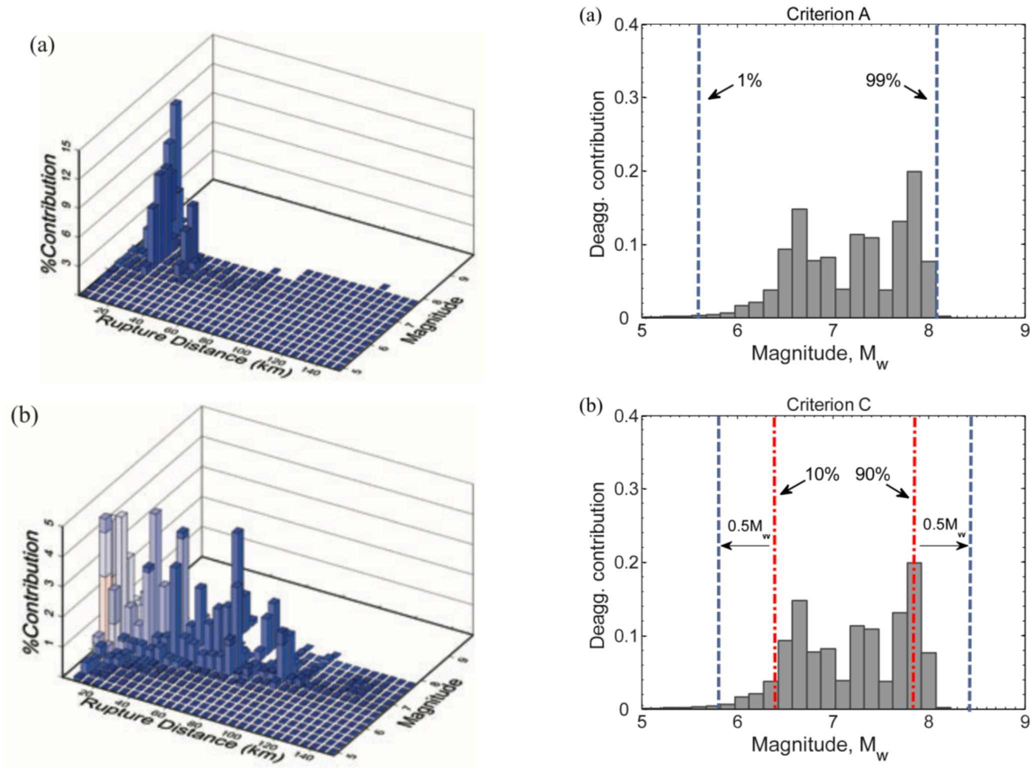
Research Objective No. 1: Ground motion selection

Ground motion selection is the critical link between seismic hazard analysis at a site, and the subsequent seismic response of numerical models of structures. NZS1170.5:2004 prescriptions for ground motion are based on outdated thinking and the consulting industry has increasingly moved away from their direct use in favour of approaches that are increasingly adopted overseas. The most common advancement is the use of the so-called 'conditional mean spectrum' (CMS), which generally leads to seismic response estimates that are less conservative than the use of ground motion records scaled to the uniform hazard spectra. The PI and colleagues have previously developed a further advancement to the CMS, which is referred to as the Generalized Conditional Intensity Measure (GCIM) approach. In this objective the intention was to undertake research required to comprehensively document the capabilities of this method, and provide analysis guidance, that would make the method ready for 'prime time' use in consulting practice.

Causal Parameter Bounds

The effect of causal parameter bounds (e.g. magnitude, source-to-site distance, and site condition) on ground motion selection, based on probabilistic seismic hazard analysis (PSHA) results, was investigated (Tarbali and Bradley 2016). Despite the prevalent application of causal parameter bounds in ground motion selection, present literature on the topic is cast in the context of a scenario earthquake of interest, and thus specific bounds for use in ground motion selection based on PSHA, and the implications of such bounds, is yet to be examined. Thirty-six PSHA cases, which cover a wide range of causal rupture disaggregation distributions and site conditions, were considered to empirically investigate the effects of various causal parameter bounds on the characteristics of selected ground motions based on the generalized

conditional intensity measure (GCIM) approach. It was demonstrated that the application of relatively 'wide' bounds on causal parameters effectively removes ground motions with drastically different characteristics with respect to the target seismic hazard and results in an improved representation of the target causal parameters. In contrast, the use of excessively 'narrow' bounds can lead to ground motion ensembles with a poor representation of the target intensity measure distributions, typically as a result of an insufficient number of prospective ground motions. Quantitative criteria for specifying bounds for general PSHA cases were provided, which are expected to be sufficient in the majority of problems encountered in ground motion selection for seismic demand analyses. Figure 1 provides an illustrate example.



(Left) Examples of seismic hazard disaggregation *(Right) Causal parameter bound criteria*

Figure 1. Illustrative figures of causal parameter bound criteria developed.

<u>Epistemic Uncertainties in Ground motion Selection</u>

Various approaches to propagate the effect of epistemic uncertainty in seismic hazard and ground motion selection to seismic performance metrics were investigated (Tarbali et al. 2018). Specifically, three approaches with different levels of rigor were presented for establishing the conditional distribution of intensity measures considered for ground motion selection, selecting ground motion ensembles, and performing nonlinear response history analyses (RHAs) to probabilistically characterize seismic response. The mean and distribution of the seismic demand hazard were used as the principal means to compare the various results. An example application illustrates that, for seismic demand levels significantly below the collapse limit, epistemic uncertainty in seismic response resulting from ground motion selection can generally be considered as small relative to the uncertainty in the seismic hazard itself. In contrast, uncertainty resulting from ground motion selection appreciably increases the uncertainty in the seismic demand hazard for near-collapse demand levels. Table 1 provides a depiction of the different steps considered, as well as the use of three approaches to understand how the exact analysis (which is computationally prohibitive) can be approximated in a practical manner. Figure 2 provides an illustration of the accuracy of these

approximate methods. The implication of this work is that the mean hazard can be used for the purpose of ground motion selection, and individual branches representing epistemic uncertainties do not, generally speaking, require separate consideration.

Table 1. Comparison of three approaches to propagate the effect of epistemic uncertainties in seismic hazard analysis and ground motion selection to demand-based seismic performance measures

Step	Approach 1: Exact	Approach 2: Approximate full distribution	Approach 3: Approximate mean
1. Seismic hazard analysis	Complete seismic hazard distribution (all logic tree branches)		Mean hazard
2. Ground motion selection	A different GM set for every logic tree branch	One GM set corresponding to the mean hazard	
3. Seismic response analysis	Different seismic response analyses for each GM set	One set of seismic response analyses corresponding to the one GM set	
4. Seismic demand hazard	Exact distribution of the seismic demand hazard	Approximate distribution of the seismic demand hazard	Approximate mean seismic demand hazard

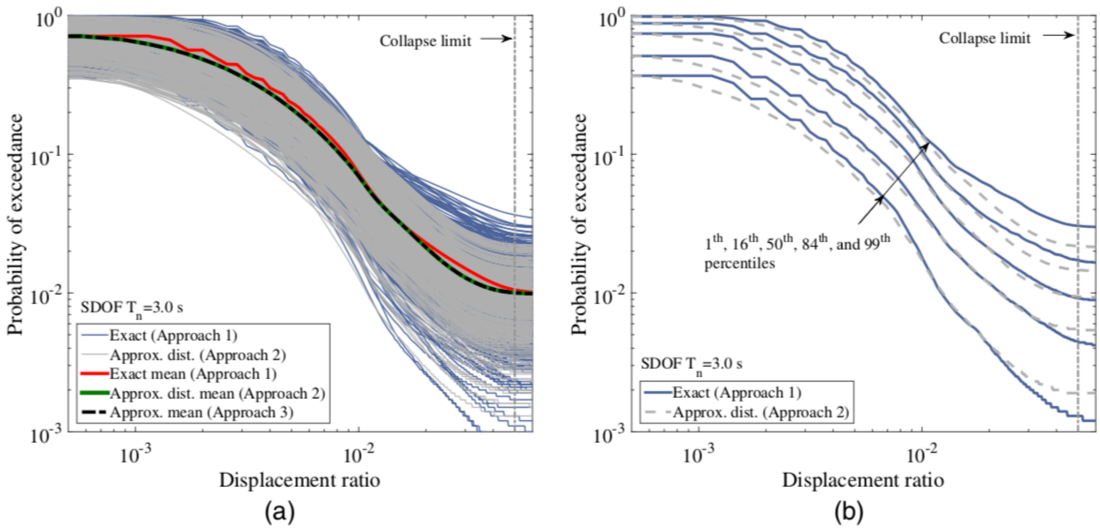


Figure 2: (a) Demand hazard curves from the three presented methodologies; (b) percentiles of the demand hazard distributions from the exact (i.e., Approach 1) and approximate distribution (i.e., Approach 2) methods.

<u>Near-fault directivity</u>

This work (Tarbali et al. 2019) focused on the selection of ground motions for seismic response analysis in the near-fault region, where directivity effects are significant. An approach was presented to consider forward directivity velocity pulse effects in seismic hazard analysis without separate hazard calculations for 'pulse-like' and 'non-pulse-like' ground motions, resulting in a single target hazard (at the site of interest) for ground motion selection. The ability of ground motion selection methods to appropriately select records that exhibit pulse-like ground motions in the near-fault region is then examined. Applications for scenario and probabilistic seismic hazard analysis cases were examined through the computation of conditional seismic demand distributions and the seismic demand hazard. It was shown that

ground motion selection based on an appropriate set of intensity measures (IMs) will lead to ground motion ensembles with an appropriate representation of the directivity-included target hazard in terms of IMs, which are themselves affected by directivity pulse effects. This alleviates the need to specify the proportion of pulse-like motions and their pulse periods a priori as strict criteria for ground motion selection. Figure 3 provides an illustration of the results. The implication of the results is that the method to undertake ground motion selection in the near-fault region can be the same as for 'far-field' sites –important for practical applications.

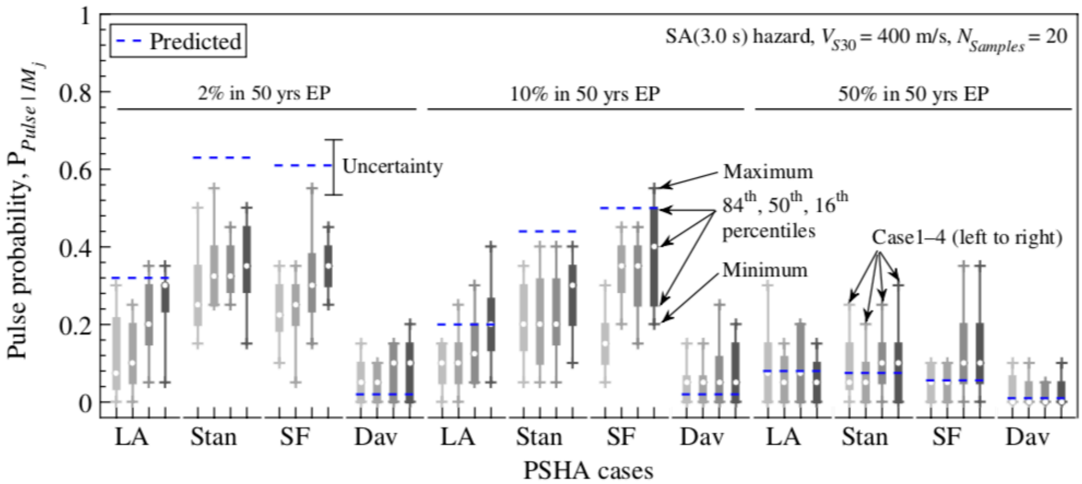


Figure 3: Ground motion selection for several regions, illustrating that ground motion ensembles have the right distribution of pulse-like records even though pulse occurrence is not specifically a variable used in the ground motion selection.

Research Objective No. 2: Ground motion prediction

This objective examined ground motion and site effect observations in the greater Wellington region from the 14 November 2016 M_W7.8 Kaikoura earthquake (Bradley et al. 2017). The region was the principal urban area to be affected by the earthquake-induced ground motions from this event. Despite being approximately 60km from the northern extent of the causative earthquake rupture, the ground motions in Wellington exhibited long period (specifically T = 1 − 3s) ground motion amplitudes that were similar to, and in some locations exceeded, the current 500 year return period design ground motion levels. Several ground motion observations on rock provided significant constraint to understand the role of surficial site effects in the recorded ground motions. The largest long period ground motions were observed in the Thorndon and Te Aro basins in Wellington City, inferred as a result of 1D impedance contrasts and also basin-edge-generated waves. Observed site amplifications, based on response spectral ratios with reference rock sites, are seen to significantly exceed the site class factors in NZS1170.5:2004 for site class C, D, and E sites at approximately T=0.3-3.0s. The 5-95% Significant Duration, D_{S595}, of ground motions was on the order of 30 seconds, consistent with empirical models for this earthquake magnitude and source-to-site distance. Such durations are slightly longer than the corresponding D_{S595} = 10s and 25s in central Christchurch during the 22 February 2011 M_W6.2 and 4 September 2010 M_W7.1 earthquakes, but significantly shorter than what might be expected for large subduction zone earthquakes that pose a hazard to the region. In summary, the observations highlight the need to better understand and quantify basin and near-surface site response effects through more comprehensive models, and better account for such effects through site amplification factors in design standards. Figure 4 provides an illustration of the ground motions observed in Wellington city.

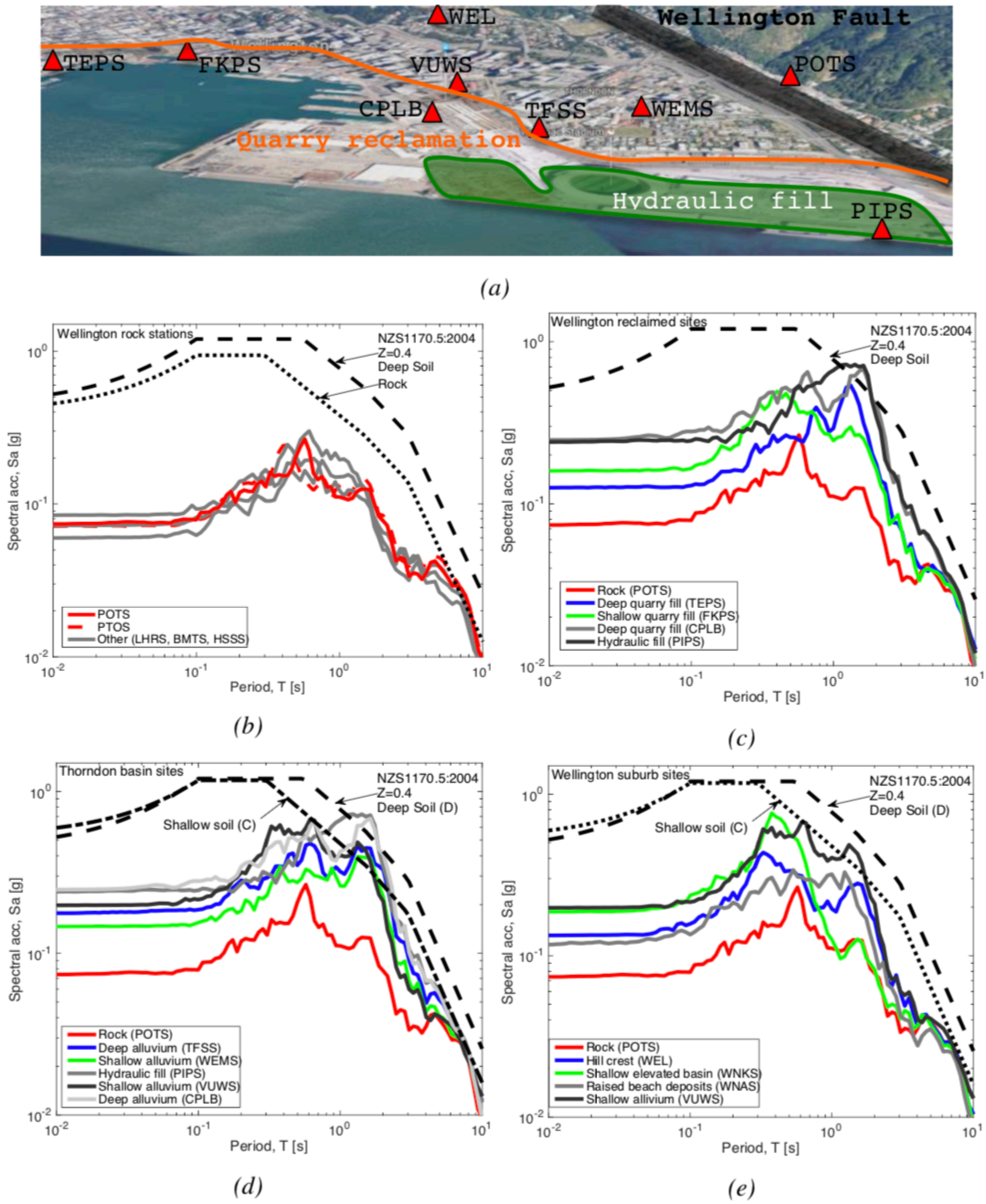


Figure 4: Observed ground motions in Wellington during the 2016 Kaikoura earthquake.

<u>Development of Wellington-specific 'Basin Factors' in NZS1170.5 Amendment 2</u>

On the basis of work partially-funded by this project, the lead PI, among others, sat on the NZS1170.5 Amendment 2 committee tasked with considering how the observed ground motions should be reflected in revised ground motion design guidelines. This work led to the development of a "basin-edge factor' to be used in the Wellington CBD region. These prescriptions are currently under internal review by MBIE, and will subsequently go out for public comment. As a result, it is not possible to document the specific nature of these features here. Dr. Rob Jury is the Amendment 2 chair, and is available for contact on this matter.

IMPLEMENTATION TO PRACTICE

The research conducted in Objective No. 1 has enabled 'unresolved questions' that prohibit the application of state-of-the-art ground motion selection methods to be overcome, and as a result, made performing ground motion selection using the GCIM method now practical in consulting projects. Over the course of the project, the Lead PI had been involved in four commercial projects (undertaken by 3 different consulting companies) within with the method has been adopted. Furthermore, the latest Base Isolation Guidelines now contain explicit mention of the GCIM method as a possible method to use for ground motion selection (there hasn't been any revision to NZS1170.5 which has considered ground motion selection, but the inclusion in the base isolation guidelines now paves the way for a similar adoption into NZS1170.5 next time such clauses are considered for revision).
Research completed in Objective 2 has led to important case histories of ground motion observation and site response effects. In addition, the development of the Wellington basin-factors for use in NZS1170.5 is expected to have a widespread impact on NZ seismic design.

ACKNOWLEDGEMENTS

Additional aligned funding was provided by a UC PhD scholarship, Royal Society Rutherford Discovery Fellowship funding, and the NZS1170.5 Amendment 2 committee.

REFERENCES

Seismic Isolation Guidelines: https://www.nzsee.org.nz/library/guidelines/seismic-isolation-guidelines/
NZS1170.5. Structural design actions, Part 5: Earthquake actions - New Zealand 2004:82.

OUTPUTS & DISSEMINATION

PUBLICATIONS:

Journal papers (published):

Tarbali K, Bradley BA. The effect of causal parameter bounds in PSHA-based ground motion selection. Earthquake Engineering & Structural Dynamics 2016;45:1515–35. https://doi.org/10.1002/eqe.2721.

Tarbali K, Bradley BA, Baker JW. Consideration and Propagation of Ground Motion Selection Epistemic Uncertainties to Seismic Performance Metrics. Earthquake Spectra 2018;34:587–610. https://doi.org/10.1193/061317EQS114M.

Tarbali K, Bradley BA, Baker JW. Ground Motion Selection in the Near-Fault Region Considering Directivity-Induced Pulse Effects. Earthquake Spectra 2019;35:759–86. https://doi.org/10.1193/102517EQS223M.

Bradley BA, Wotherspoon LM, Kaiser AE. Ground motion and site effect observations in the Wellington region from the 2016 Mw7.8 Kaikoura, New Zealand earthquake. Bulletin of the New Zealand Society for Earthquake Engineering 2017;50:94–105.

Bradley BA, Wotherspoon LM, Kaiser AE, Cox BR, Jeong S. Influence of Site Effects on Observed Ground Motions in the Wellington Region from the Mw 7.8 Kaikōura, New Zealand, Earthquake. Bulletin of the Seismological Society of America 2018;108:1722–35. https://doi.org/10.1785/0120170286.

LIST OF KEY END-USERS

- Engineering Consultants (Holmes Consulting LP, Golder Associates, WSP Opus
- Ministry of Business, Innovation and Employment (MBIE)
- Standards New Zealand (NZS 1170.5 committee)

Task 2: Performance Objectives

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ABSTRACT
Task 2 of Objective 1 focused on the subject of performance objectives for seismic design. In the aftermath of the Canterbury earthquakes and more recent Kaikoura earthquake, it is increasingly evident that the traditional performance objective of life-safety is no longer sufficient. The economic impact of the Canterbury earthquakes has been estimated at more than $40billion and the indirect losses, including social issues, are widely considered as being unacceptable. This research has consequently examined what changes could be made to our performance objectives and design process that could lead to better outcomes in future earthquakes? Progress on this topic has been made by quantifying the expected annual losses for a number of typical code-compliant buildings and then considering how the loss would be reduced if alternative, low-damage performance objectives were adopted. In addition, recognising that the quantification of seismic performance requires consideration of a large number of uncertainties, the research has advanced practice-oriented tools for the quantification of the annual probability of exceeding key limit states and losses.

Research Objective No. 1: Losses of code-compliant NZ buildings

Recognising that a useful means of describing performance could be to quantify the expected monetary losses due to repair works, not just in a single scenario earthquake but rather on an average annual basis considering a large range of possible intensity levels, one of the research objectives has been to quantify the losses of code-compliant NZ buildings. Whilst the process of loss assessment is not new and a large number of fragility and consequence functions (required for loss assessment) exist in the literature, they were not all considered directly applicable to typical New Zealand buildings. As such, research was required to establish the information required for loss assessment. Subsequently, a number of case study buildings could be designed using the NZ standards and the expected annual losses due to repair quantified.

<u>Fragility functions typical of New Zealand building components</u>

One barrier to adopting seismic loss estimation frameworks in New Zealand engineering practice is the lack of relevant fragility functions which provide probabilities of exceeding certain levels of damage (e.g. cracking of gypsum wallboards) for a given demand (e.g. interstorey drifts). This research, in collaboration with QuakeCoRE, sought to address this need for four different building components; interior full-height steel-framed plasterboard partition walls, unbraced suspended ceilings, precast concrete cladding, and steel beam-column joints with extended bolted end-plate connections. Fragility functions were sourced from literature, and their potential for use in New Zealand evaluated considering similarities in component detailing with local practices. Modifications to a number of fragility functions, including generalizations for easier adoption in practice, were proposed. By undertaking loss estimation of a 4-storey steel moment-resisting frame building, it was shown that the selection of fragility functions and the definition of damage states can have a noticeable influence on the assessment of incurred repair cost of individual building components. This indicated that fragility functions should be carefully selected, particularly if the performance evaluation of each individual component is key. However, the observed difference in expected

annual repair cost of the entire building was small, indicating that in cases where fragility functions are not readily applicable for use in New Zealand, other fragility functions may be used as placeholders without drastically altering the outcome of loss analysis for the entire building. For details, refer Yeow et al. (2018a).

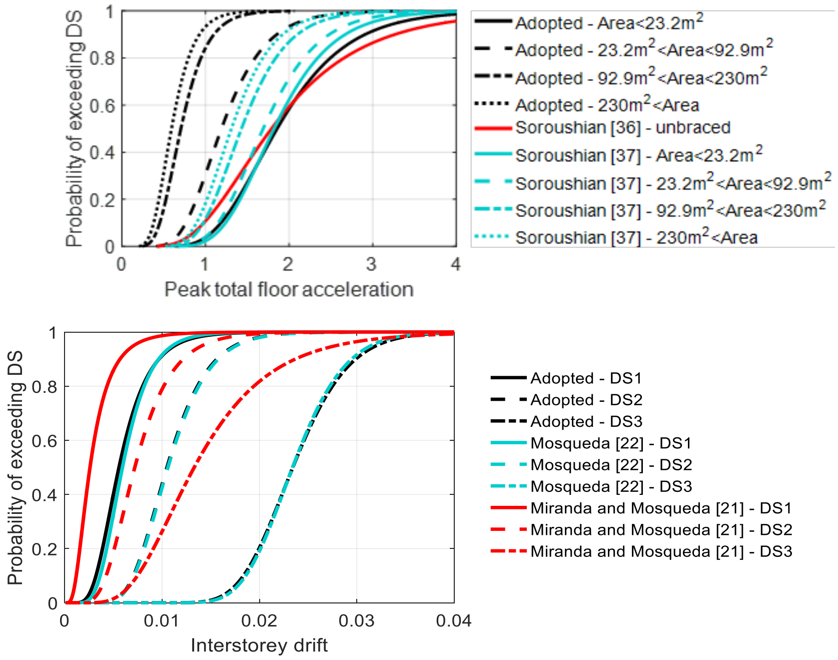

Figure 1. Illustrative figures of fragility functions developed for ceilings (top) and partition walls (bottom) from Yeow et al. (2018a).

<u>Seismic performance of NZ buildings considering direct-repair costs</u>

With fragility and consequence functions established, the research could proceed to evaluate average annual monetary losses due to repairs for typical New Zealand buildings. Furthermore, by collaborating with researchers in QuakeCoRE, it was possible to examine potential benefits of low-damage structural systems. Yeow et al. (2018b) report on initial construction costs and direct-repair costs for New Zealand steel moment-resisting frame buildings with friction connections and those with extended bolted-end-plate connections. A total of 12 buildings were designed and analysed considering both connection types, two building heights (4-storey and 12-storey), and three locations around New Zealand (Auckland, Christchurch, and Wellington). It was found that buildings with friction connections required design to a higher design ductility, yet are generally stiffer due to larger beams being required to satisfy higher connection overstrength requirements. This resulted in the frames with friction connections experiencing lower interstorey drifts on most floors but similar peak total floor accelerations, and subsequently incurring lower drift-related seismic repair losses. Frames with friction connections tended to have lower expected net-present-costs within 50 years of the building being in service for shorter buildings and/or if located in regions of high seismicity. None of the frames with friction connections in Auckland showed significant benefits due to the low seismicity of the region. At a high level, this study is therefore suggesting that loss assessment could be a valuable means of highlighting the relative performance benefits of one structural system versus another.

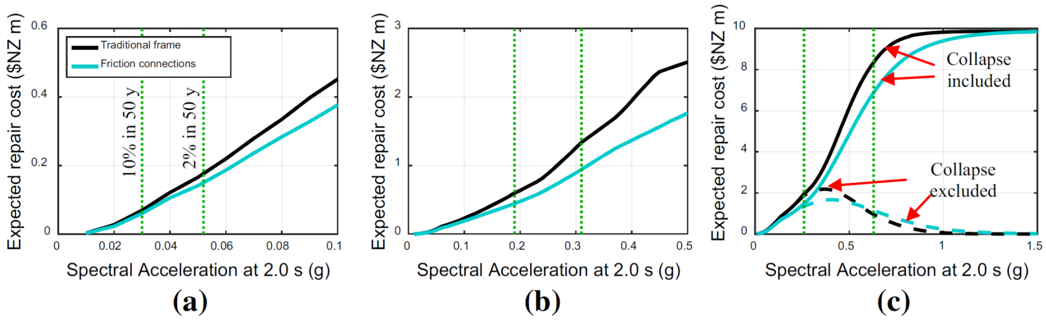

Figure 2. Comparison of expected repair costs, as a function of ground shaking intensity, for a 4-storey steel framed building in (a) Auckland, (b) Christchurch, and (c) Wellington.

Research Objective No. 2: Practice-oriented tools for the quantification of the annual probability of exceeding key limit states and losses

Extending the SAC/FMA approach to account for different possible failure mechanisms

The SAC/FEMA approach is a probabilistic framework in which the seismic risk is quantified analytically but in a simplified manner. Means of improving the accuracy of this approach can be found in the literature and further improvements have been made as part of this research. Firstly, improved intensity–inelastic displacement relationships were established by examining a large data set of non-linear dynamic analysis results. Secondly, a closed form expression has been developed for systems susceptible to different mechanisms controlling performance at the key limit state. The benefits of the new intensity–inelastic displacement relationships have been illustrated through seismic assessment of four single storey case study buildings. Results show that the intensity associated with a specific displacement demand can be predicted reliably. Furthermore, the validity of the new expression to account for different failure mechanisms was demonstrated by assessing the performance of a case study bridge structure against shear failure mechanisms in short and tall piers. The results obtained from simplified and rigorous methods indicated that the simplified method performs well. Limitations with the new approach were discussed and future research needs are identified.

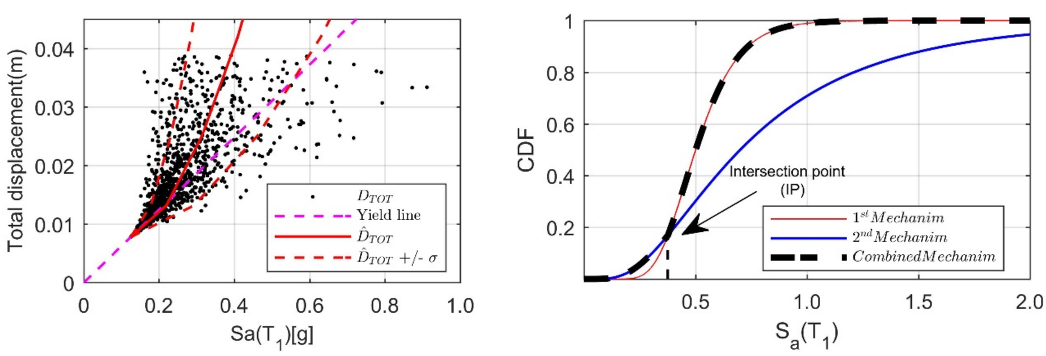

Figure 3. Illustrating the examination of intensity-displacement demand relationships (left) and influence of different mechanisms on limit state failure (right) from Orumiyehei and Sullivan (2019).

IMPLEMENTATION TO PRACTICE

By providing new fragility functions and guidance on the loss-assessment process, the research conducted in Task 2 has enabled more accurate loss assessment to be conducted for

New Zealand buildings. Furthermore, the simplified evaluation of the likelihood of exceeding a limit state via the SAC/FEMA method can be integrated within a simplified procedure to estimate expected annual losses and thus this will help inform emerging low-damage design guidelines.

ACKNOWLEDGEMENTS

Additional aligned funding was provided by QuakeCoRE Flagship 4 funding in support of a post-doc conducted by Trevor Yeow.

OUTPUTS & DISSEMINATION

PUBLICATIONS

Journal papers (published or under review):

Orumiyehei, A., Sullivan, T.J. (2019) “Extending the SAC/FMA approach to account for different possible failure mechanisms” Journal of Earthquake Engineering, Taylor & Francis, *under review*.

Yeow TZ., Sullivan TJ., and Elwood KJ. (2018) “Evaluation of fragility functions with potential relevance for use in New Zealand” Bulletin of the New Zealand Society for Earthquake Engineering, Vol.51, No. 3.

Yeow TZ., Orumiyehei A., Sullivan TJ., MacRae GA., Clifton GC. and Elwood KJ. (2018) “Seismic performance of steel friction connections considering direct-repair costs” Bulletin of Earthquake Engineering http://dx.doi.org/10.1007/s10518-018-0421-x.

LIST OF KEY END-USERS

- Engineering Consultants (Beca, Aurecon, WSP-Opus)
- Ministry of Business, Innovation and Employment (MBIE)
- Engineering New Zealand
- EQC

Objective 2: Geotechnical Hazards and Impacts on Infrastructure

Topic 1: Liquefaction-Induced Lateral Spreading

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ABSTRACT

In the 2010-2011 Canterbury earthquakes and 2016 Kaikoura earthquake widespread liquefaction occurred in urban areas of Christchurch and at the port of Wellington. The severe liquefaction and especially lateral spreading, a phenomenon associated with soil liquefaction, caused major damage to buildings and horizontal infrastructure, and created long-term impacts for two of the largest communities in New Zealand. This study focusses on lateral spreading and aims at developing state-of-the-art methodology for engineering assessment of liquefaction-induced lateral spreading. It builds on well-documented observations from recent NZ earthquakes to interpret complex features of spreading and develop practical approach that will support engineering assessment and design of structures in liquefiable soils with potential for lateral spreading. An overview of key elements and considerations of the research study is presented herein.

INTRODUCTION

The strong ground shaking generated by the 2010-2011 Canterbury earthquakes triggered severe soil liquefaction in nearly half of the urban area of Christchurch. The liquefaction caused extensive damage to buildings, bridges, roads and pipe networks in central and eastern Christchurch. Rough estimates indicate that the economic loss due to soil liquefaction was about 15 billion NZD or approximately 40% of the total economic loss caused by the earthquakes. Liquefaction-induced damage also caused some of the most challenging long-term impacts on the community, as nearly 8,000 residential properties were abandoned in the Red Zone along the Avon River.

The most severe damage occurred in areas where soil liquefaction was accompanied by lateral spreading, which is a phenomenon associated with one of the most damaging forms and effects of liquefaction. Spreading of liquefied soils typically occurs near waterways (along river banks and revetment lines), and involves large movements of liquefied soils which often result in permanent ground displacements of up to several metres. Figure 1 schematically illustrates characteristic ground movements and ground distortion due to lateral spreading of liquefied soils, whereas Figure 2 shows typical spreading-induced damage to buildings and infrastructure observed after the 2010-2011 Canterbury earthquakes.

In the 2016 M_w7.8 Kaikoura earthquake, the port of Wellington (CentrePort) experienced significant liquefaction of reclaimed land that led to wharf and building damage, and consequent temporary loss of operations at CentrePort. The liquefaction caused large settlement of thick fill deposits and spreading-induced lateral movement of the reclamations

towards the sea in the range between 0.5 and 1.5 metres (see Figure 3). Again, the heaviest damage to land and structures was due to lateral spreading.

These recent earthquakes emphasized the high liquefaction vulnerability of native soils and reclaimed land in major urban centres of New Zealand, and highlighted the need for an improved seismic assessment and design of land and structures in such soils. As current state-of-the-practice approaches for engineering evaluation of lateral spreading are highly empirical and have relatively poor predictive capacity, the principal objective of this study was to make use of observations from the recent earthquakes and state-of-the-art methodology to develop alternative procedures for assessment of lateral spreading.

This report provides a very brief summary of the research study and highlights at a high level some of the research aspects and outputs, but excludes technical details. A comprehensive summary of the research methodology and outputs is provided in the journal papers listed in the Outputs and Dissemination Section.

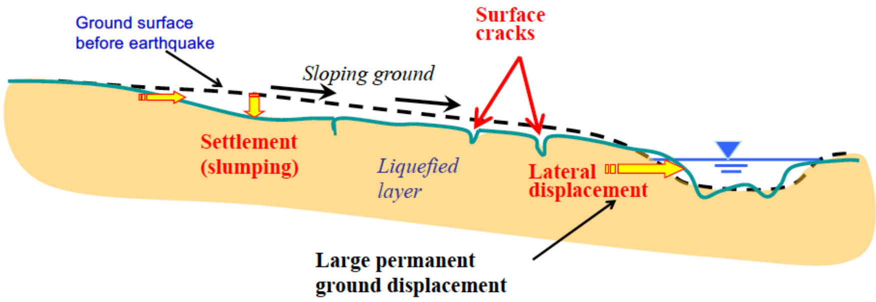


Figure 1. Schematic illustration of ground movements and distortion due to lateral spreading of liquefied soils involving large ground displacements towards a river, settlement (loss of elevation), and substantial ground surface distortion including cracks, fissures and vertical offsets

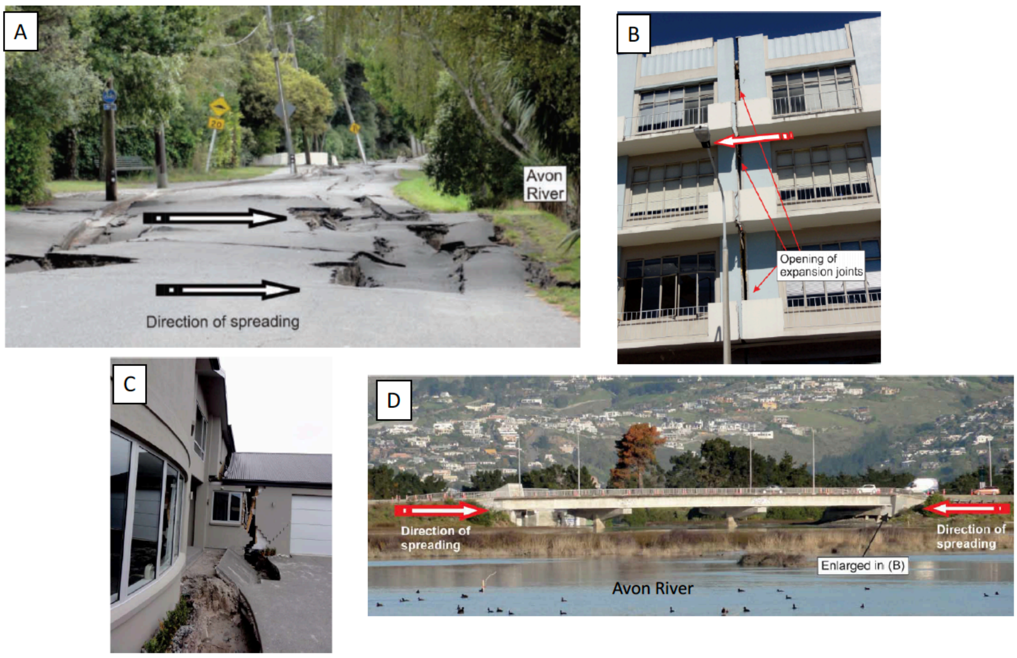


Figure 2. Spreading-induced damage to roads, CBD buildings, residential buildings and bridges observed in the 2010-2011 Canterbury earthquakes

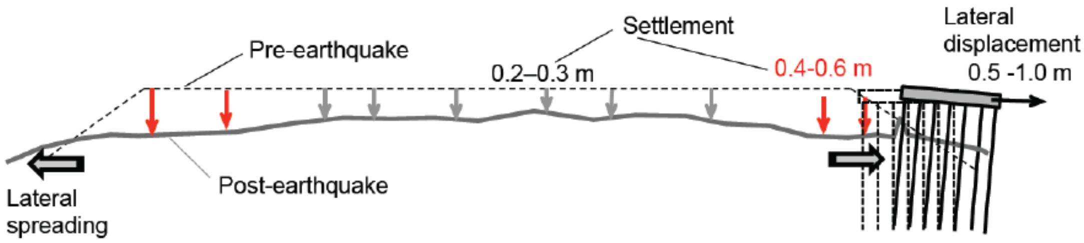


Figure 3. Schematic illustration of liquefaction-induced settlement and permanent lateral ground displacements due to spreading of reclamations at CentrePort in the 2016 Kaikoura earthquake (Cubrinovski et al., 2018)

Research Objective No. 1: Characteristics of Lateral Spreading

Following the Canterbury earthquakes, comprehensive field inspections were performed to document lateral spreads predominantly along the Avon River, but also along the Heathcote River and in Kaiapoi. Data from such case history investigations were the principal source for scrutiny of characteristics of lateral spreads. Lateral spreading always involve a complex interplay of various factors that influence ground response during earthquakes including ground conditions, soil characteristics, river geometry and geomorphology, and characteristics of ground motion (loading) imposed by the earthquake.

Typical features of lateral spreading are illustrated in Figure 4, on a case study at the estuary of Avon River:

(a) The direction of lateral spreading (i.e. the direction of large ground movements) is towards the river (waterway) or/and in the downslope directions
(b) ground displacements are the largest at the river banks and decrease with inland distance from the river
(c) spreading causes extensional ground deformation in the direction of spreading which leads to opening of cracks and fissures at the ground surface which typically run parallel to the river or perpendicular to the spreading direction
(d) engineering structures residing within the spreading zone are subjected to large lateral loads and deformations including a stretch of their foundations due to differential ground movements

There are two principal mechanisms driving lateral spreading displacements: (a) earthquake-shaking, and (b) gravity-induced loads. The latter are the key reason for biased movements in a 'preferred' direction, i.e. in the downslope direction of sloping ground and towards a "free face" (e.g. river channel or waterfront retaining structure), as gravity forces produce shear stresses that act in the downslope direction and towards the free face (Figure 5). The dynamic liquefaction-spreading process could be broken down into several key phases: (i) In the first phase, the earthquake shaking will generate pore pressures in the groundwater and will trigger liquefaction in the soil deposit; in this phase, the soil will lose its strength and stiffness, and will turn into an easily deformable material (liquefied soil); (ii) In the second phase, post liquefaction triggering, both gravity-loads and earthquake loads will produce large ground displacements in the direction of spreading (due to the low resistance to deformation of liquefied soil); (iii) In the final third phase, gravity-induced spreading movements may continue even after the end of shaking due to the low residual strength of liquefied soils.

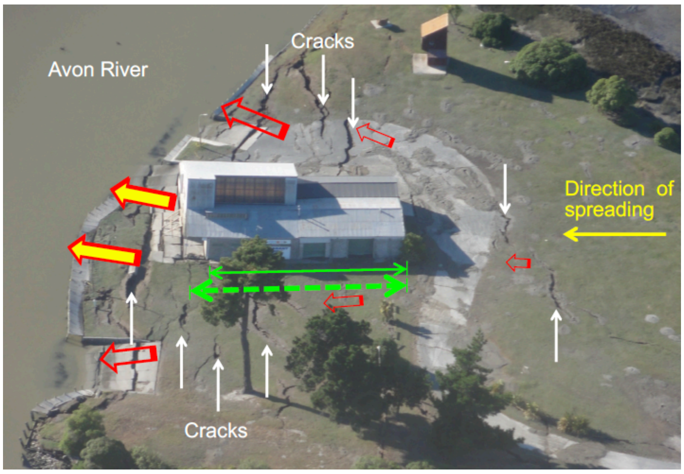


Figure 4. Characteristics features of lateral spreading illustrated on a well-documented case history: direction of spreading; magnitudes of lateral ground displacements (indicated by the size of red arrows; largest ground displacements at the river banks (yellow arrows); ground cracks run perpendicular to the spreading direction, and extensional deformation in the direction of spreading causing stretch of building foundations (Cubrinovski, 2019)

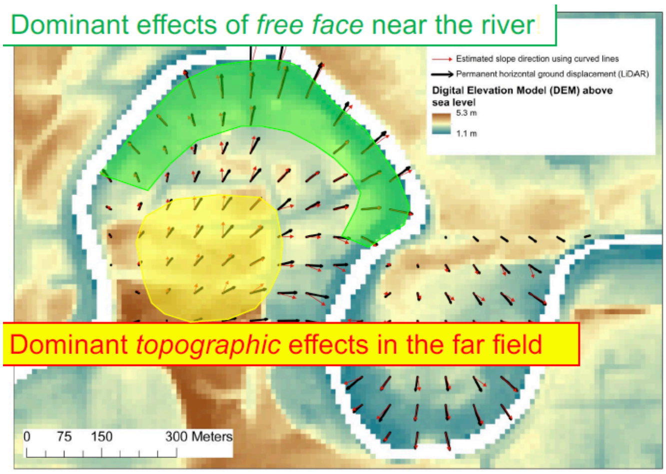


Figure 5. Effects of free face (river geometry) in the near field and topography (ground slope) in the far field on lateral spreading displacements (black vectors) (Cubrinovski et al., 2016; Cubrinovski, 2019)

Research Objective No. 2: Engineering assessment of lateral spreading

Three principal objectives in the engineering assessment of lateral spreading have been identified in Figure 6. They involve estimation of: (1) Magnitude of maximum lateral ground displacement (at river bank or revetment line); (2) Zone affected by lateral spreading (or distance from the waterway within which spreading will produce permanent ground displacements), and (3) Spatial distribution of ground displacements within the zone affected by spreading. Meeting these assessment objectives will allow engineers to quantify seismic loads and deformation demands on structures, and hence to design structures for lateral spreading effects.

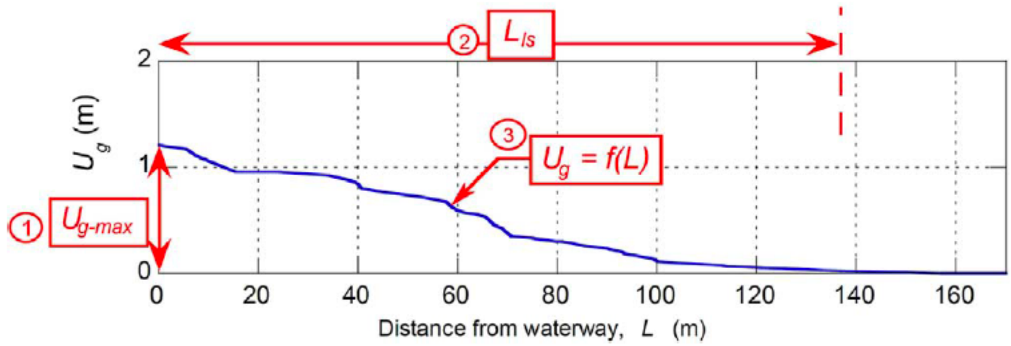


Figure 6. Principal objectives in the assessment of lateral spreading: (1) Maximum magnitude of ground displacement ($U_{g\text{-}max}$); (2) Zone affected by spreading (L_{ls}); (3) Distribution of ground displacements within the spreading zone, $U_g = f(L)$ (Cubrinovski and Robinson, 2016; Cubrinovski, 2019)

To achieve these objectives, it is important to systematically address the following aspects in the evaluation: (i) geomorphology and geologic characteristics of the subject area; (ii) geotechnical characterization and liquefaction resistance of representative soil deposits; (iii) ground motion or earthquake loading characteristics; (iv) dynamic processes of soil liquefaction, consequent deformation and spreading displacements, and (v) important mechanisms and interactions affecting the ground response.

Benefits of appropriate classification of lateral spreads are illustrated in Figure 7 where lateral spreads observed along the Avon River are classified into four groups showing distinct characteristics of their lateral spreading displacements: (a) large maximum displacements from 0.5 m to 1.5 m; (b) moderate displacements of approximately 0.5 m; (c) negligible movements from 0 cm to 20 cm, and (d) large displacements, but associated with very narrow zone of spreading of 20 m to 40 m. As indicated in the figure, key differences between these classes of spreads are in the thickness of their critical layer (weakest layer close to the ground surface), and vertical and lateral continuity of such layers.

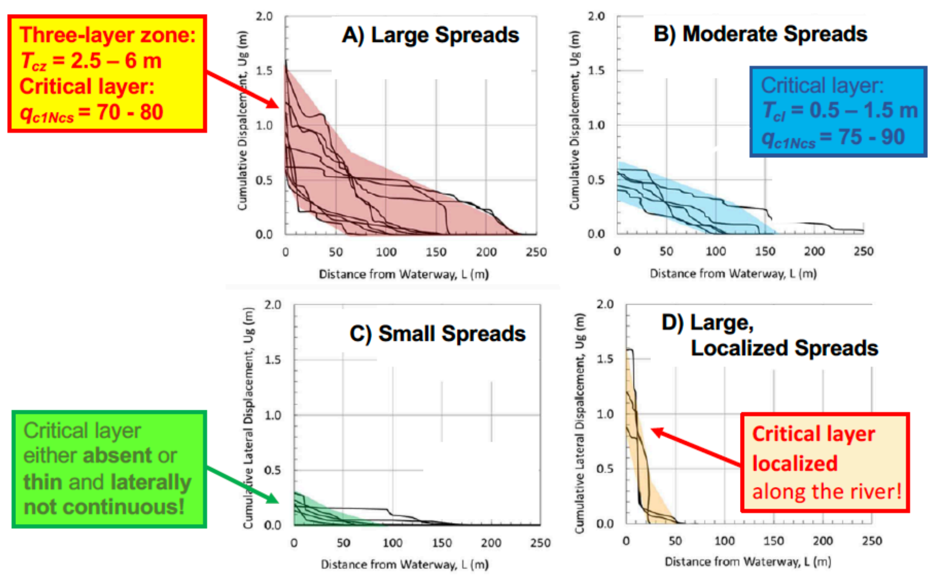


Figure 7. Interpretation and classification of lateral spreads including key factors contributing to differences in ground displacements (Cubrinovski and Robinson, 2016; Cubrinovski, 2019)

Research Objective No. 3: System response of liquefiable soils

Comprehensive investigations revealed that critical liquefiable layers cannot explain alone the observed field performance of different types of liquefiable deposits. Instead, overall deposit characteristics may often have dominant influence on the severity of liquefaction manifestation and consequent damage to near-surface structures. For example, it was consistently found throughout Christchurch that soil deposits composed of liquefiable soils in the top ten metres provided vertical continuity and interactions between layers that generated severe liquefaction manifestation. Contrary to this, highly interbedded deposits consisting of liquefiable and non-liquefiable layers produced different types of cross-layer interactions that mitigated effects of liquefaction, and hence prevented damage to residential buildings on shallow foundations. Both types of deposits, i.e. vertically continuous liquefiable soils and highly interbedded liquefiable and non-liquefiable layers, had identical critical layers at shallow depts of about 2-3 m, but showed extreme difference in the effects of liquefaction, i.e. severe liquefaction and no liquefaction manifestation respectively.

Figure 8 shows a characteristic soil profile for sites with continuous liquefiable soils that manifested sever liquefaction effects (Fig. 8a). The sketch in Figure 8b schematically illustrates the key mechanisms that lead to severe liquefaction manifestation at the ground surface of such sites. They involve: (1) rapid liquefaction of the shallow critical layer; (2) additional disturbance of the liquefied critical layer due to inflow of water from the underlying layers that didn't liquefy but generated high excess pore water pressures; and, (3) seepage-induced liquefaction in shallow soils above the water table. These system response effects result in a strong and damaging discharge of excess pore water pressures in which liquefiable soils from the entire deposit contribute to and intensify the severity of liquefaction manifestation. This explains the severe liquefaction manifestation observed at such sites with vertically continuous liquefiable soils.

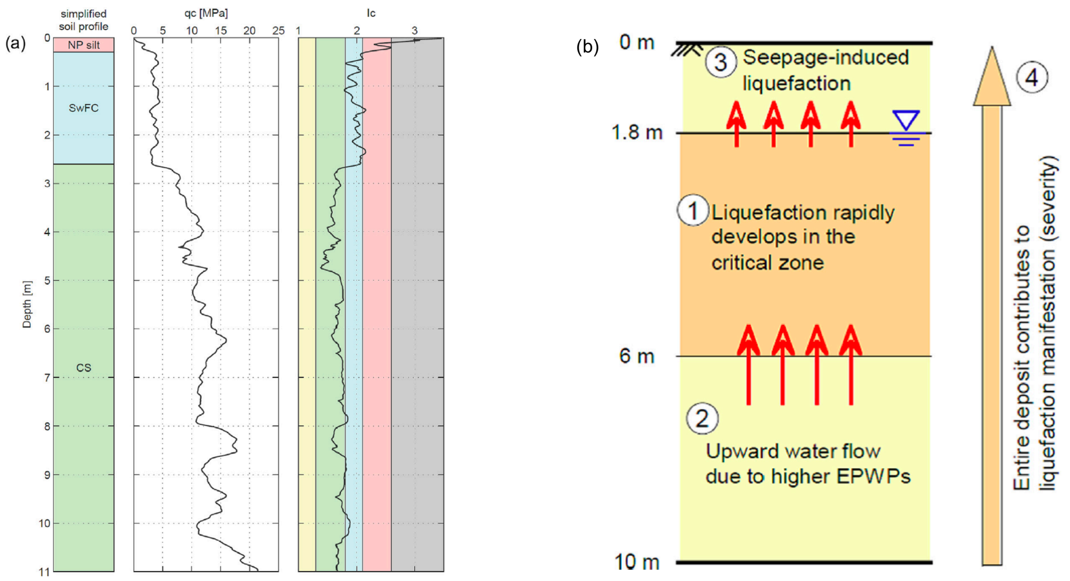


Figure 8. Soil deposits comprising vertically continuous liquefiable sands in the top ten metres that exhibited severe liquefaction manifestation in the Canterbury earthquakes; (b) schematic illustration of system-response mechanisms that intensify liquefaction manifestation and damage to structures at the ground surface (Cubrinovski et al., 2019)

Research Objective No. 4: Combined earthquake loading and gravity-induced effects

One of the complexities in the evaluation of lateral spreading displacements is that both dynamic loads due to earthquake shaking and static loads due to gravity contribute to permanent spreading movements. Hence, both types of loads and their effects on movements need to be quantified.

Figure 9 comparatively illustrates characteristics of earthquake loads (presented in a simplified form) generated by the 2010 Darfield earthquake and 2011 Christchurch earthquake along the Avon River, where a large number of lateral spreads were investigated. By and large, the seismic demand specific to liquefaction was approximately twice larger in the 2011 Christchurch earthquake.

To combine the effects of earthquake and gravity loads, simplified models for lateral spreading sites have been developed as illustrated with the sketch in Figure 10. These models include all the important factors that influence lateral spreading and consequent ground displacements including behavioural characteristics and location of liquefied layer, free face geometry and topographic features of soils adjacent to the river, presence of non-liquefiable crust, possible interactions between different elements of the model, and characteristics of ground motion (earthquake loading).

Effects of various factors on magnitude and spatial distribution of lateral spreading were then investigated using advanced effective stress numerical analyses. In these dynamic analyses, liquefaction processes are rigorously modelled including build-up of pore pressures in the groundwater, flow of water, and cross-layer interactions or system response effects. An example of analysis results from such numerical analyses are shown in Figure 11. Parametric numerical analyses are used to quantify effects of key variables in the model and then to develop simplified models for evaluation of lateral spreading that can support engineering practice.

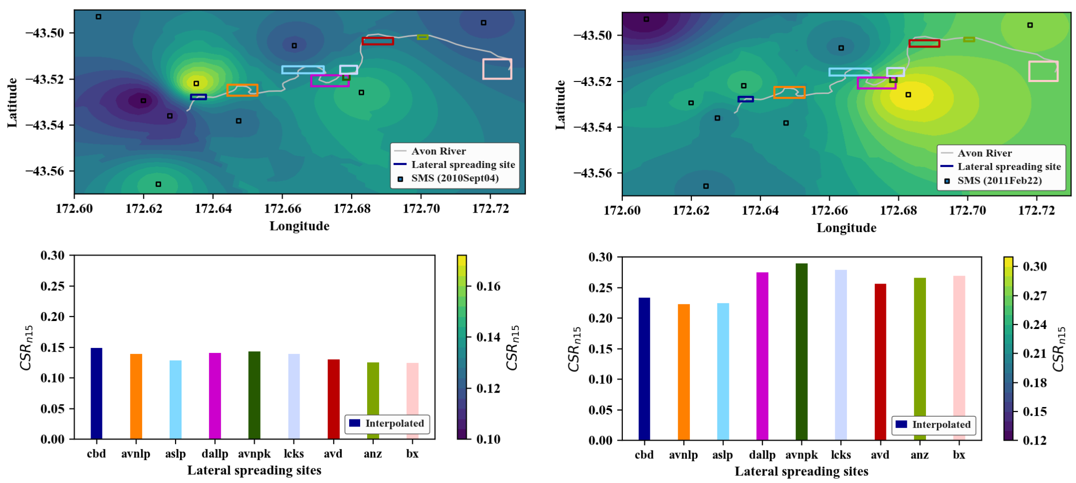


Figure 9. Seismic demand in terms of normalized cyclic stress amplitude for the September 2010 earthquake (left), and February 2011 earthquake (right); seismic demand at lateral spreading sites along the Avon river (from the CBD to the estuary) is shown in the bottom part of the figure

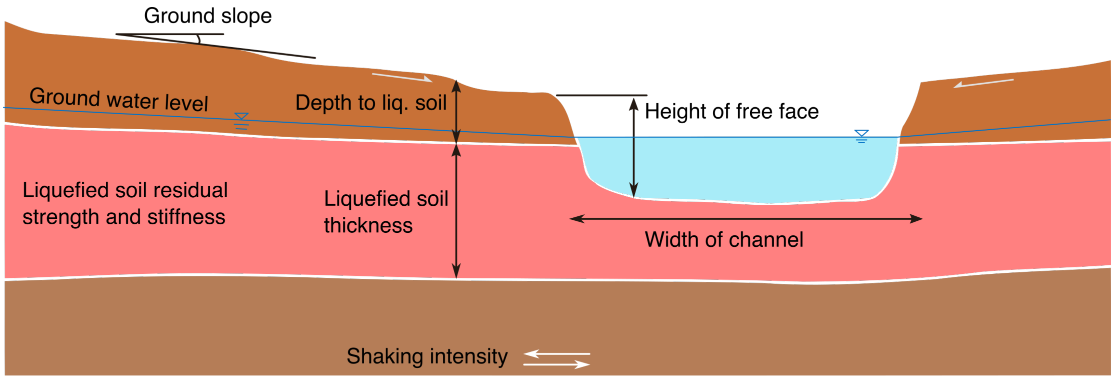


Figure 10. Simplified ground profile illustrating key elements in the evaluation of lateral spreading including: (i) thickness and deformational characteristics of liquefied soil; (ii) ground surface slope adjacent to the river; (iii) height of the free face and width or river channel; (iv) location/position of the liquefied layer relative to the free face; (v) thickness of crust (non-liquefiable surface layer); (vi) intensity of ground motion (earthquake load)

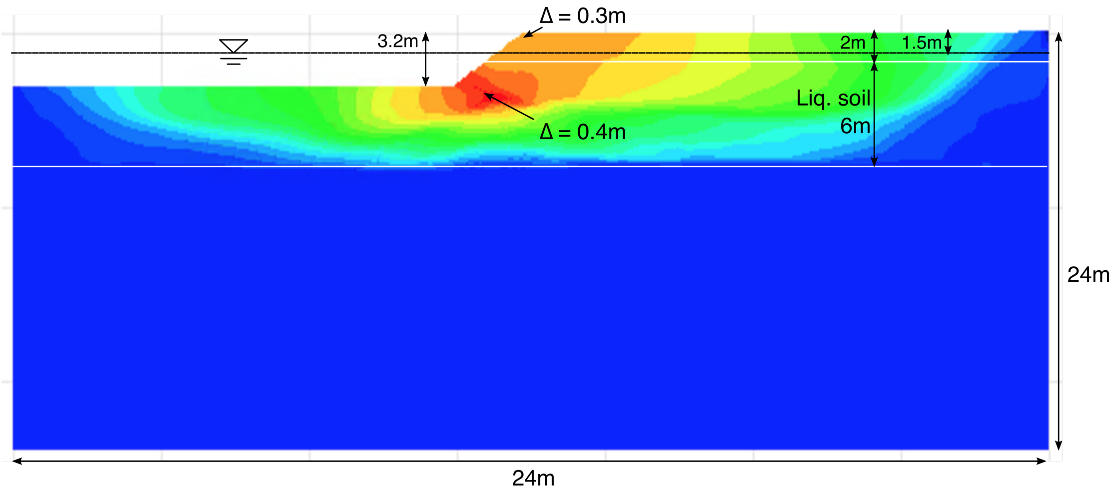


Figure 11. Computed permanent lateral ground displacements in a numerical analysis of lateral spreading for a characteristic Avon River site

CONCLUSIONS AND KEY FINDINGS

Key findings resulting from this research are summarised below:

1) Lateral spreading is a complex phenomenon associated with soil liquefaction that results in a severe damage to land and structures. Current empirical procedures for evaluation of lateral spreading do not provide satisfactory support of engineering assessment and design.
2) Native deposits and reclaimed land in New Zealand have high liquefaction and spreading potential resulting in a high risk for spreading-induced damage to structures and consequent impacts on communities.
3) The principal objectives in the engineering assessment of lateral spreading should be to estimate: (1) magnitude of maximum lateral ground displacement; (2) zone affected by lateral spreading, and (3) spatial distribution of ground displacements within the zone affected by spreading.

4) In the engineering evaluation, it is important to directly consider: (i) geomorphology and geologic features of the area; (ii) soil profile and liquefaction resistance of soils; (iii) ground motion characteristics; (iv) dynamic processes of soil liquefaction and lateral spreading, and (v) system response effects of liquefying deposits.
5) Vertical continuity of liquefiable soils or lack of it, and consequent system response effects significantly influence and even govern severity of liquefaction manifestation through a set of cascading interacting mechanisms. These effects also have direct influence on the magnitude of lateral spreading displacements.
6) Lateral continuity of critical liquefiable layers or lack of it, directly influence the extent of the lateral spreading zone away from the waterway, and hence whether structures will be subjected to spreading loads or not.
7) Lateral spreading assessment should systematically classify spreads and identify key factors that govern the behaviour within a given class. Such models should be developed for a particular ground (geologic) environment and specific soil deposit characteristics.
8) Important factors that influence lateral spreading and consequent ground displacements include: density and location of liquefied layer relative to free face and ground surface, free face height, ground surface slope for soils adjacent to the river, presence and thickness of a non-liquefiable crust, deposit characteristics (layering and anticipated mechanisms of interaction), possible interactions between different factors influencing spreading, and characteristics of ground motion (earthquake loading).

IMPLEMENTATION TO PRACTICE

Key findings from the research study will be incorporated in Module 3 of the MBIE-NZGS Guidelines for the Earthquake Geotechnical Engineering practice in New Zealand. Currently Module 3 of the guidelines is under revision led by an MBIE and Engineering New Zealand panel chaired by Mike Stannard. The principal investigator of this research study is a key panel member and principal author of Module 3. The revised Module 3 document will be completed and opened for public comments in June 2020.

ACKNOWLEDGEMENTS

The authors would like to acknowledge numerous contributions of PhD students and postdoctoral fellows from the University of Canterbury and QuakeCoRE, international collaborators, and industry partners from New Zealand on our Canterbury and Kaikoura earthquakes research studies. We are grateful for the continuous collaboration provided by CentrePort Ltd. on the CentrePort case study.

OUTPUTS & DISSEMINATION

PUBLICATIONS

Journal papers:

Cubrinovski, M., & Robinson, K. (2016). Lateral spreading: Evidence and interpretation from the 2010–2011 Christchurch earthquakes. *Soil Dynamics and Earthquake Engineering*, *91*, 187-201. doi:10.1016/j.soildyn.2016.09.045

Cubrinovski M., Bray J.D., de la Torre C., Olsen, M.J., Bradley B.A., Chairo G., Stocks, E., Wotherspoon, L.M. (2017). Liquefaction effects and associated damages observed at the Wellington CentrePort

from the 2016 Kaikoura Earthquake. Special Issue on the 2016 Kaikoura Earthquake. *Bulletin of the New Zealand Society for Earthquake Engineering*, 50(2): 152-173.

Cubrinovski, M., Bray, J., de la Torre, C., Olsen, M., Bradley, B., Chiaro, G., Stocks, E., Wotherspoon, L., and Krall, T. (2018). Liquefaction-Induced Damage and CPT Characterization of the Reclamations at CentrePort, Wellington", *Bulletin of the Seismological Sociary of America (BSSA),* under review.

Beyzaei CZ, Bray JD, van Ballegooy S, Cubrinovski M, Bastin S (2018). Depositional environment effects on observed liquefaction performance in silt swamps during the Canterbury earthquake sequence, *Soil Dynamics and Earthquake Engineering* 107:303-321.

Cubrinovski*,* M., Rhodes, A., Ntritsos, N., Van Ballegooy S. (2019). System response of liquefiable deposits, *Soil Dynamics and Earthquake Engineering,* 124: 212-229.

Ntritsos, N. and Cubrinovski, M. (2019) A CPT-based effective stress analysis procedure for liquefaction assessment, *Soil Dynamics and Earthquake Engineering* (submitted).

Dhakal, R., Cubrinovski, M., Bray, J. and de la Torre C. (2019). Liquefaction assessment of reclaimed land at CentrePort, Wellington, *Bulletin of the New Zealand Society for Earthquake Engineering*, (submitted).

Conference papers:
Numerous conferences papers in proceedings of major national and international conferences.

KEYNOTE/INVITED LECTURES:

Key findings and outcomes from the research study were presented at 42 keynote and invited lectures at major international conferences in Australia, Canada, Greece, Italy, Macedonia, Portugal, Turkey and USA, and seminars throughout New Zealand.

LIST OF KEY END-USERS

- Ministry of Business, Innovation and Employment (MBIE)
- New Zealand Geotechnical Society (NZGS) and professional engineers
- CentrePort Ltd.
- Tonkin+Taylor
- WSP Opus
- Wellington City Council (WCC)

Topic 2: Geotechnical characterisation of volcanic (pumiceous) deposits

Research Team:
A/Prof. Rolando Orense, University of Auckland, Task Leader
Prof Michael Pender, University of Auckland, Key Researcher
Dr. Mark Stringer, University of Canterbury, Key Researcher
Baqer Asadi, University of Auckland, PhD student
Sadeq Asadi, University of Auckland, PhD student

ABSTRACT

Pumice-rich soils originating from volcanic eruptions are deposited in various parts of the world, such as in the central part of North Island, NZ. Since they are often encountered in engineering projects, their geotechnical characterisation is very important. Pumice sands are highly crushable, compressible and lightweight due to the vesicular nature of the particles. Because of these distinctive properties, there are concerns on the applicability of available empirical correlations, derived primarily from hard-grained sands, to pumice-rich soils. To understand their dynamic and liquefaction characteristics, samples were obtained from various pumice-rich sites in the central part of North Island using diverse sampling techniques. The soil samples were tested in the laboratory using cyclic triaxial apparatus and bender elements. At the same time, various field tests, such as CPT, V_s profiling and SDS tests, were conducted adjacent to the same sampling sites. A comparison of the laboratory and field-based results clearly showed that crushable volcanic soils do not fit existing frameworks in terms of dynamic characterisation and liquefaction assessment and alternate methods are therefore necessary. A key parameter that affected their response was the amount of pumice particles in the soil; to quantify pumice content, a method that can be easily performed in any geotechnical laboratory was proposed.

INTRODUCTION

Soil liquefaction, induced by earthquake shaking, has been one of the main causes of devastating damages to infrastructure and financial losses worldwide. For example, the 2010-2011 earthquakes in Christchurch have demonstrated the impact of soil liquefaction to the built environment (e.g., Cubrinovski and Orense, 2010; Orense et al. 2011; Cubrinovski et al. 2012; Orense et al. 2012a). Consequently, understanding the geotechnical characteristics and seismic behaviour of various local soils is important in designing earthquake-resistant structures.

Volcanic soils, including pumiceous sands, are found in several areas in the central part of North Island. They originated from a series of volcanic eruptions centred in the Taupo and Rotorua region, called the Taupo Volcanic Zone (Pender et al. 2006). By the power of explosion and airborne transport, which were followed by erosion and river transport, the pumice materials were distributed and mixed with other materials in the Waikato Basin. As a consequence of infrastructure development in the region, many engineering projects frequently encounter pumice-rich deposits, so there is a need to understand their dynamic properties.

Pumice is characterised by a number of distinctive properties. Pumice particles are highly crushable, compressible and lightweight as a result of their vesicular nature due to the presence of internal voids (Wesley et al. 1999). Recent studies (e.g. Orense et al. 2012b; Orense and Pender 2013) showed that available empirical correlations to determine various engineering properties for design, which were derived from hard-grained soils, are not

applicable to pumice deposits. Such dilemma raises many concerns from geotechnical practitioners posing questions on how to treat such materials when encountered in the field.

This report provides a very brief summary of the NHRP research study conducted to characterise systematically the dynamic and liquefaction characteristics of natural pumice deposits found in the region through field testing, soil sampling and laboratory testing. The report highlights at a very high level some of the research aspects and outputs, and excludes many technical details. A comprehensive summary of the research methodology and outputs is provided in the journal and conference papers listed in the Outputs and Dissemination Section.

A list of Key End Users is provided at the end of this report. We would like to highlight the exceptional collaboration of researchers of this project and the various regional/city councils and engineering consulting firms who have provided key information in terms of locations and supplementary information on pumice-rich deposits present in their respective areas of interest.

Research Objective No. 1: High-quality undisturbed soil sampling and field testing at pumice-rich sites

Target sites

Pumice sands have been deposited in the Waikato Basin as a consequence of volcanic eruptions within the Taupo Volcanic Zone (TVZ). The air-borne waves of red-hot volcanic debris from these eruptions, which included pumice particles, were lifted into the atmosphere and reached beyond Hamilton. In addition, the pumice-rich pyroclastic flows generated enormous dust clouds that covered a large area with fine pumice particles. After the eruptions, thick layers of pumice, together with other sediments, formed natural dams, which blocked the outlet of Lake Taupo and parts of the valley of the Waikato River. Subsequently, the dams were overtopped and gave way, washing away all the debris on the choked beds and eroded the riverbanks (Manville et al. 2007). The floodwaters covered large areas with a few meters of mudflow (i.e. the debris of the eruption which included pumice particles). After the flood subsided, the pumice-rich debris were left in the river valley and when the river cut new channels through the muddy, pumice-rich debris, layers of pumice-rich deposits were left on the low terrace of the Waikato River (McCraw 2011). These events blanketed the region with air-fall and water-rafted pumice deposits that now characterise the geology of the Waikato and Bay of Plenty regions.

The target sites where both field testing and soil sampling were conducted in this research are shown in Figure 1. Within the Waikato Basin, four sites were in Hamilton (HAM), and one each near the towns of Huntly (HUN) and Rangiriri (RAN); while in the Bay of Plenty region, one site each was chosen in Tauranga (TAU), Whakatane (WHA) and Edgecumbe (EDG).

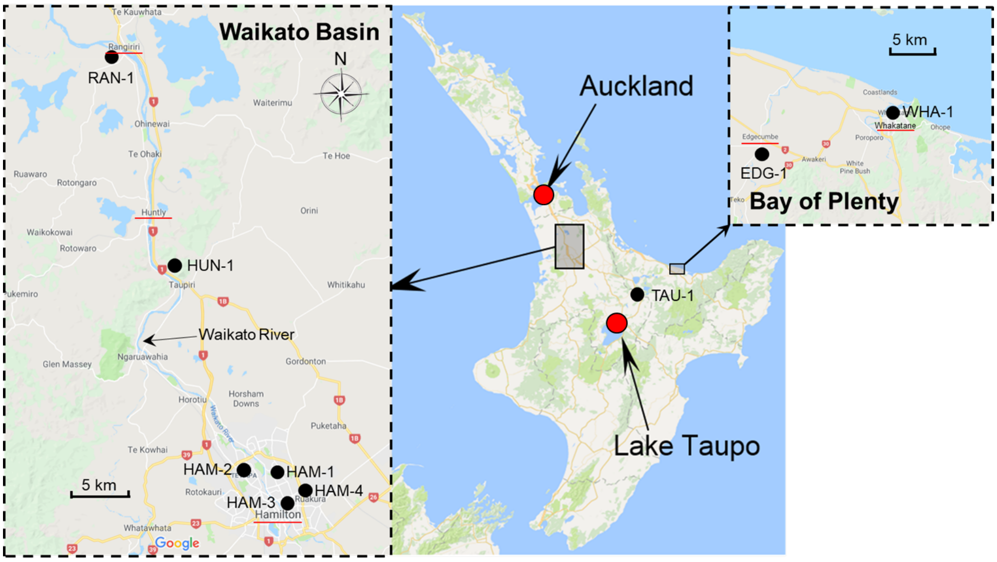


Figure 1. Locations of target sites.

Soil sampling

At these sites, both undisturbed and reconstituted samples were obtained. The undisturbed samples were obtained by either Gel-push sampler (GP-S and GP-Tr), Dames and Moore (DM) sampler, conventional push tubes (PT) or block sampling (see Figure 2). Depths of sampling were chosen based on layer descriptions in the borehole logs. Some of the materials (HAM-1, HAM-2, HUN-1 and RAN-1 sites) were taken from road construction sites where excavations were on-going at the time of sampling.

(a) (b) (c) (d)

Figure 2. Undisturbed sampling at WHA-1 site: (a) using Gel-push sampler; (b) using Dames & Moore sampler; (c) using push tube; (d) condition of one of the samples.

The quality of the undisturbed soil samples was evaluated either through visual inspection or through comparison of shear wave velocities obtained in the field and through the laboratory. Both Gel-push and Dames & Moore sampling methods appeared to indicate generally good quality of undisturbed soil samples.

In addition, for the purpose of comparison, two other materials were used in the research. One was the commercially-available pure pumice sand. Pumice-rich deposits were quarried at sites in Mercer (along the Waikato River) and the pumice particles were centrifugally separated from other constituents so that the samples consisted of essentially pumice particles. The other material was Toyoura sand, which is known as a hard-grained, sub-angular material and commonly used in many geotechnical testing in Japan.

Field testing

Various types of field tests were conducted at the target sites, generally within 5 m from where the samples were obtained. These field tests included cone penetration tests (CPT), shear wave velocity profiling (either through sCPT or sDMT), and screw driving sounding (SDS), a new in-situ method that has recently been developed in Japan, where a rod is drilled into the ground at several loading steps at the same time as the rod is being continuously rotated (Orense et al. 2013b, 2019). Photos showing some of the field tests are depicted in Figure 3.

(a)

(b)

Figure 3. Field testing conducted in WHA-1 site: (a) CPT and sCPT; and (b) SDS test.

Research Objective No. 2: Laboratory testing on pumice-rich soils

Mineralogy of pumice-rich sands

To understand the mineralogy of the various components of the pumice-rich sands from different locations, several series of environmental scanning electron microscope (ESEM) imaging accompanied by energy dispersive spectroscopy (EDS) analysis were performed on the materials. Figure 4 illustrates the ESEM images and the EDS analysis results for pure pumice sand and Toyoura sand, as well as the hard-grained components and pumice particle components of the pumice-rich sands. From the figures, it is clear that the pumice particles have very irregular and angular surface texture, quite different from normal sands. Moreover, the hard-grained particles of pumice-rich sand have similar chemical composition to Toyoura sand while its pumice particles have similar composition to pure pumice sand. These results confirmed that the pumice particles in the pumice-rich sands of the Basin are quite similar; i.e. they have similar mineralogy to the pure pumice sand, also sourced from nearby locality.

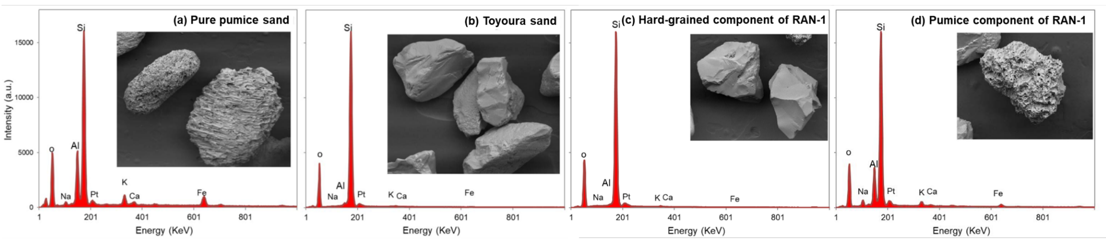


Figure 4. ESEM image and EDS analysis of: (a) pure pumice sand; (b) Toyoura sand; (c) hard-grained component of RAN-1 sand; and (d) pumice component of RAN-1 sand.

Pumice content quantification

A key component that differentiates the behaviour of pumice-rich soils to other normal (hard-grained) sands is the presence of crushable pumice sands within the soil matrix. Currently, other than visual inspection, there is no well-accepted method to quantify the pumice content (*PC*). In this research, attempts were made to quantify *PC* through the crushability feature of pumice sand. For this purpose, laboratory tests were performed on

known mixtures of pure pumice sand and hard-grained sand. The soil mixtures (with known *PC*) were compacted using

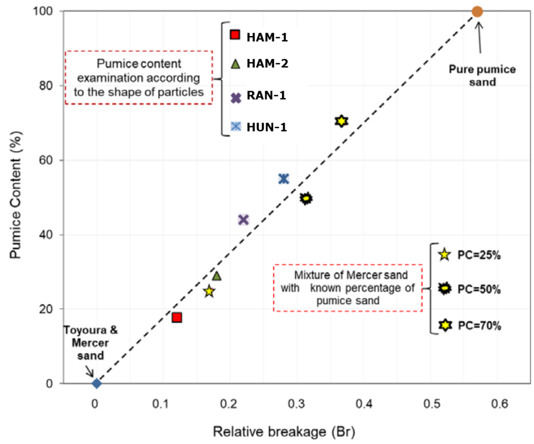
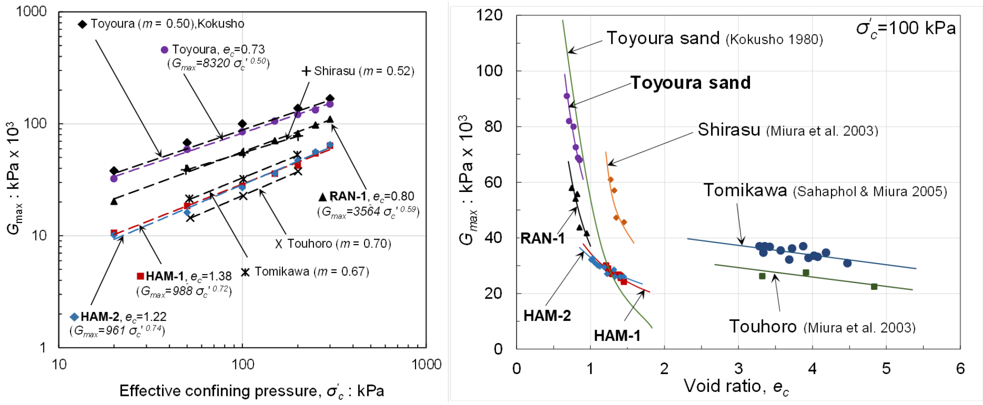


Figure 5. Estimation of pumice content according to the relative breakage.

a modified version of the maximum dry density test on sand. The particle size distribution curves before and after compaction were determined, and the degree of particle crushing was quantified using relative breakage, B_r (Hardin 1985), which was found to correlate well with *PC*. The developed procedure was validated by quantifying the *PC* of several pumice-rich samples sourced from the target sites by visually counting the pumice-looking particles in SEM images taken; the results agreed quite well with those estimated using the proposed method. The chart used to quantify the pumice content of soil deposit is shown in Figure 5.

Bender element testing

The maximum shear modulus (G_{max}) of the pumice-rich samples, both in undisturbed and reconstituted states, were measured using bender elements. In one stage of the study, the effects of effective confining pressure (σ'_c) and void ratio (e) on the G_{max} of the reconstituted soils were assessed. For comparison purposes, similar tests were also conducted on Toyoura sand. Figure 6 shows the comparison of the G_{max} dependency of several pumice-rich sands on σ'_c and e with those obtained for Toyoura sand and other volcanic soils. It can be seen that the G_{max} of pumice-rich soil is considerably lower than that of hard-grained Toyoura sand at all investigated levels of σ'_c under a similar void ratio. Furthermore, the $G_{max} - \sigma'_c$ dependency of pumice-rich soil is more pronounced as compared to Toyoura sand; in contrast, the $G_{max} - e$ relation shows a lower dependency for pumiceous soils. Such behaviour can be explained in terms of particle shape characteristics and crushability of the pumice sand components.



(a) *(b)*

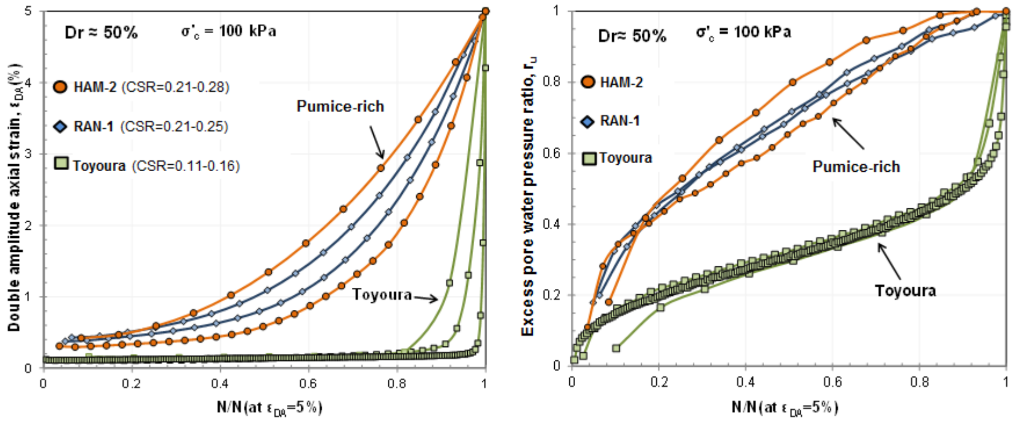
Figure 6. Dependency of G_{max} on: (a) effective confining pressure; and (b) void ratio for several pumice-rich sands and Toyoura sand. Also shown are some available relationships for other volcanic soils.

Undrained cyclic triaxial testing

Several series of undrained cyclic triaxial tests were performed on both reconstituted and undisturbed pumice-rich specimens obtained at the target sites, as well as on pure pumice sand and Toyoura sand. The following sections highlight the difference in the undrained response between pumice-rich sands and hard-grained (quartz) sands as well as between undisturbed and reconstituted pumice-rich sands.

<u>Reconstituted pumice-rich sand and Toyoura sand</u>

In order to highlight the difference in the undrained cyclic response of sand containing crushable pumice components with that of hard-grained sand, cyclic triaxial tests were performed on reconstituted specimens to investigate the effect of particle characteristics and crushability on the undrained cyclic behaviour of pumice-rich sandy materials. Typical results for medium dense (D_r=50%) pumice-rich sands, in terms of the development of double amplitude axial strain, ε_{DA}, and excess pore water pressure ratio, r_u, with the number of cycles, N, normalised by the number of cycles required to obtain ε_{DA}=5%, for different cyclic stress ratios ($CSR= \sigma'_d/2\sigma'_c$), are shown in Figure 7. Here, σ'_d is the deviator stress and σ'_c is the initial effective confining pressure. From the figure, Toyoura sand shows gradual build-up of excess pore water pressure during the early stage of cyclic loading accompanied by negligible strain development; however, the rate of strain development increases dramatically as soon as the specimen reaches pore water pressure ratio, r_u ≈ 0.5-0.6. Conversely, the response observed for pumice-rich sand is different; pumice-rich sand specimens undergo significant deformation from the start of cyclic loading accompanied by high r_u development. In the subsequent loading cycles, the axial strain continues to increase at almost linear trend to reach ε_{DA}=5%, while the rate of change in r_u decreases. Essentially similar trend was observed for dense (D_r=80%) specimens. Pumice-rich specimens show very contractive behaviour during the initial cycle of loading because of the occurrence of particle crushing. However, under high r_u, the behaviour becomes very dilative, conceivably because the initial particle crushing and subsequent particle rearrangement, as a result of the application of the next cycles of loading, lead to gradual formation of more stable soil skeleton inside the specimen.



(a) (b)

Figure 7. Comparison of undrained cyclic behaviour of medium-dense (D_r≈50%) pumice-rich sands and Toyoura sand under different levels of CSR in terms of: (a) double amplitude axial strain, ε_{DA}; and (b) excess pore water pressure ratio, r_u.

The liquefaction resistance curves for some of the materials tested are shown in Figure 8, relating the number of cycles required to attain ε_{DA}=5% for the specified *CSR*. As depicted in the figure, the liquefaction resistance of reconstituted pumice-rich sands increases with increasing relative density, D_r. It is noteworthy that pumice-rich sands are more resistant to liquefaction compared to Toyoura sand, partly due to the high angularity of the pumice sand components. Since pumice particles have very irregular and angular surface texture, particle crushing causes the contact surface of the particles to increase. While the soil particles crush and rearrange under the application of initial cyclic loading, the fine crushed materials develop bonds between the particles, which provide higher interlocking effect on the soil samples.

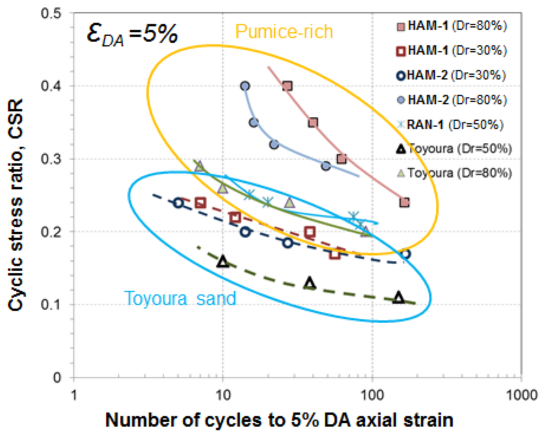
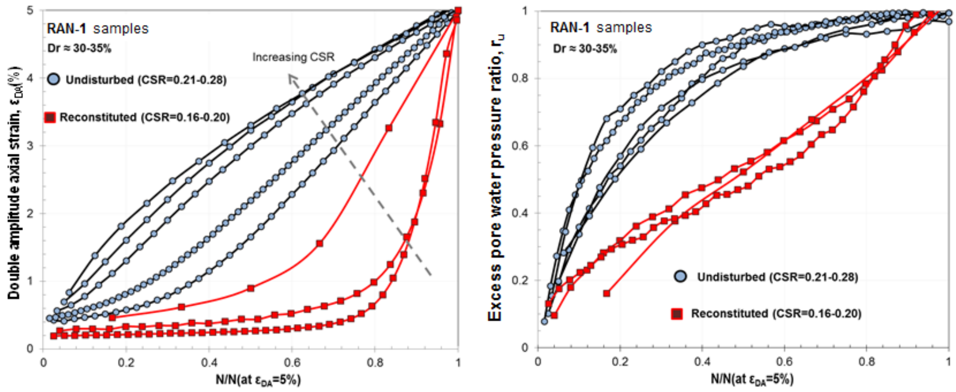


Figure 8. Liquefaction resistance curves of reconstituted pumice-rich sands and Toyoura sand.

<u>Undisturbed vs reconstituted pumice-rich samples</u>

Similarly, several series of undrained cyclic triaxial tests were performed on undisturbed pumice-rich sands. To illustrate the influence of soil fabric and structure, the test results on undisturbed pumice-rich sand were compared with those of similar materials reconstituted to the same relative density. Note that due to the different liquefaction resistances of the undisturbed and reconstituted samples, the range of *CSR* applied to the specimens was different, i.e., it was not possible to compare their behaviour under the same *CSR*. The

results shown in Figure 9 are for RAN-1 specimens that underwent liquefaction (reached ε_{DA} = 5%) after similar number of cycles.



(a) *(b)*

Figure 9. Comparison of cyclic response of undisturbed and reconstituted RAN-1 sand under different levels of CSR: (a) double-amplitude axial strain; (b) excess pore water pressure ratio plotted against normalised number of cycles.

From the figure, it is observed that, with the increase in the *CSR*, both sets of specimens undergo greater initial deformation. The r_u responses of undisturbed and reconstituted specimens are significantly different. For instance, during the first quarter to the first third of cyclic loading ($N/N_{\varepsilon DA}$=5% ≈ 0.25–0.35), the r_u of undisturbed specimens increases significantly, reaching up to 0.8; this is followed by a gradual increase during the subsequent cycles to an r_u > 0.95. In contrast, for the reconstituted specimens, r_u = 0.1–0.15 is generated inside the specimens as a result of the first cycle of loading, followed by a gradual increase in r_u. The specimens reach r_u =0.80 when the normalised number of cycles is almost 0.8, and initial liquefaction (r_u =1) occurs just before ε_{DA} reaches 5%. For undisturbed specimens, the rate of increase in strain is almost unaffected by changes in r_u and under r_u > 0.8, the specimens are capable of being subjected to significant cyclic loading without undergoing large deformation. In contrast, the reconstituted specimens start to deform faster when r_u = 0.8 is reached.

An important feature of the undrained cyclic behaviour of the undisturbed soil samples is that, despite the higher initial compressibility, the specimens show stable behaviour under high excess pore water pressure. That is, at low mean effective stress (near liquefaction state), a very steady deformation occurs with cyclic loading. This behaviour implies that contact is maintained within the particle network, evidence of a stable granular structure.

Figure 10 compares the liquefaction resistance of undisturbed and reconstituted pumice-rich sand specimens from HUN-1 site (D_r=30–35%). It is seen that the undisturbed specimen has higher liquefaction resistance than the reconstituted one; e.g., the liquefaction resistance of undisturbed sand is about 1.6 times higher than those of reconstituted sand. Moreover, the undisturbed specimens exhibit steeper liquefaction resistance curve. These differences highlight the contribution of the soil fabric, structure, stress history, etc. on the liquefaction resistance since the effects of these parameters are erased during the preparation of reconstituted specimens.

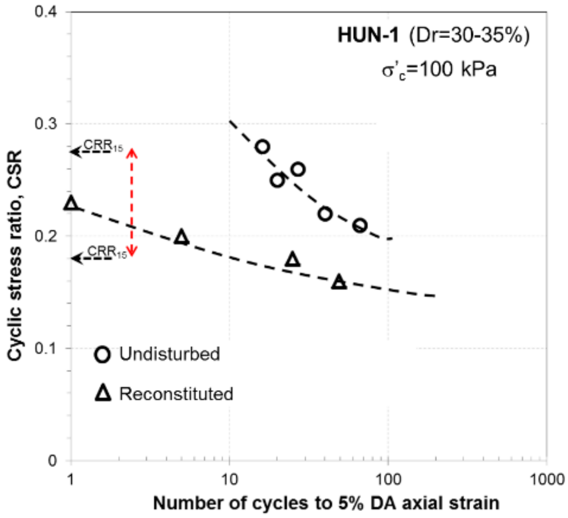


Figure 10. Comparison between liquefaction resistance curves of undisturbed and reconstituted pumice-rich HUN-1 sands.

Research Objective No. 3: Liquefaction assessment of pumice-rich deposits

Application of available simplified procedures for liquefaction triggering

The 1987 Edgecumbe earthquake showed evidence of localised liquefaction of sands of volcanic origin (Pender & Robertson 1987). Therefore, an attempt was made to investigate whether existing empirical field-based methods to evaluate the liquefaction potential of sands, which were originally developed for hard-grained soils, are applicable to crushable pumice-rich deposits. For this purpose, two sites, one in Whakatane (WHA-1) and another in Edgecumbe (EDG-1), were selected where the occurrence of liquefaction were reported following the Edgecumbe earthquake. Manifestations of soil liquefaction, such as sand boils and ejected materials, have been reported at both sites. Field tests, including CPT, V_s-profiling, and SDS tests were performed at the sites. Then, considering estimated peak ground accelerations (PGAs) at the sites based on recorded motions and possible range of ground water table locations, liquefaction analysis was conducted at the sites using the following semi-empirical approaches: three CPT-based methods (Boulanger and Idriss, 2014; Robertson and Cabal 2012; and Moss et al. 2006); two Vs-based methods (Andrus and Stokoe 2000; Kayen et al. 2013); and an SDS-based method (Mirjafari et al. 2016).

Based on the results, all the empirical methods considered predicted liquefaction of the pumice-rich deposits at both sites (at depths ranging from the water table location up to 7 m depth). However, from the results of the analyses, it was observed that for both sites, the calculated factor of safety, $FoS \approx 0.3–0.5$; expressed in terms of indices of liquefaction-induced damage, such as the post liquefaction settlement (Δs), Liquefaction Potential Index, *LPI* (Iwasaki et al. 1978) or Liquefaction Severity Number, *LSN* (Tonkin + Taylor 2013), the low *FoS* would have resulted in severe manifestations of liquefaction, such as extensive sand boils, ground cracks and surface settlement (e.g., for WHA-1 site, *LPI* = 28–32, Δs = 35–40 cm; for EDG-1 site, *LPI* = 15–20, Δs = 10–16 cm). However, per the reconnaissance reports, only minor to moderate damages, in the form of localised sand boils, were observed at both sites.

The discrepancy between the estimated damage and observed manifestation of liquefaction may be due to the following factors: (1) the *CSR* induced by the earthquake, manifested by the *PGA* used, may be quite high; and (2) the *CRR*, estimated using field-based parameters may have significantly underestimated the actual liquefaction resistance of the pumice-rich soils. Considering that the *PGA* values used are consistent with the overall analysis reported by Mellsop (2017), it is highly likely that the *CRR* used is substantially underestimated.

Comparison between laboratory-obtained and field-based liquefaction resistance

The results of the undrained cyclic triaxial tests on the high-quality undisturbed specimens were summarised in the form of liquefaction resistance curves. It was noted from the plots (not shown here) that there was scatter in the data points, indicating that within the samples obtained over a certain depth (general depth interval of about 0.5 m) or within the 126 mm-high specimen itself, the samples were non-homogenous. Bands consisting of high pumice contents appear in-between layers of low pumice contents; such inhomogeneity of samples can result in notably different response when compared to samples from adjacent depth.

In view of the typical number of significant cycles present in many time histories of accelerations recorded during past earthquakes, it is customary to consider 15 cycles of loading, representing M_w=7.5 earthquake, to estimate the liquefaction resistance of the soil; herein, this is referred to as $(CRR)_{lab} = (\sigma'_d/2\sigma'_c)$. However, the conditions the laboratory specimens were subjected to are different from those in-situ. Hence, in order to estimate the in-situ liquefaction resistance, $(CRR)_{field}=\tau_{cyclic}/\sigma'_v$, corrections need to be applied to the laboratory-obtained values; these corrections are discussed elsewhere (Ishihara 1985; Towhata 2008). Thus, the $(CRR)_{lab}$ corresponding to 15 cycles from all the undrained cyclic triaxial tests conducted to date (with effective confining pressure σ'_c=100 kPa) were collated and correlated to the field parameter (CPT, V_s or E_s of SDS) measured at the specified depth where the samples were obtained. These were then plotted in the empirical charts, as shown in Figure 11. In the figure, the CPT-based chart is that proposed by Boulanger and Idriss (2014), while the V_s-based chart and SDS-based chart are from the procedure proposed by Kayen et al. (2013) and Mirjafari et al. (2016). Note that all the charts are for clean sands (fines content *FC* < 5%) and correspond to M_w=7.5 earthquake and σ'_v=1 atm (=100 kPa). Also incorporated in the figure are the data points reported by Orense & Pender (2013) based on their previous study; since SDS tests were not conducted then, the penetration energy in SDS ($E_{s,1}$) was estimated from correlation with CPT data, as reported by Orense et al. (2019).

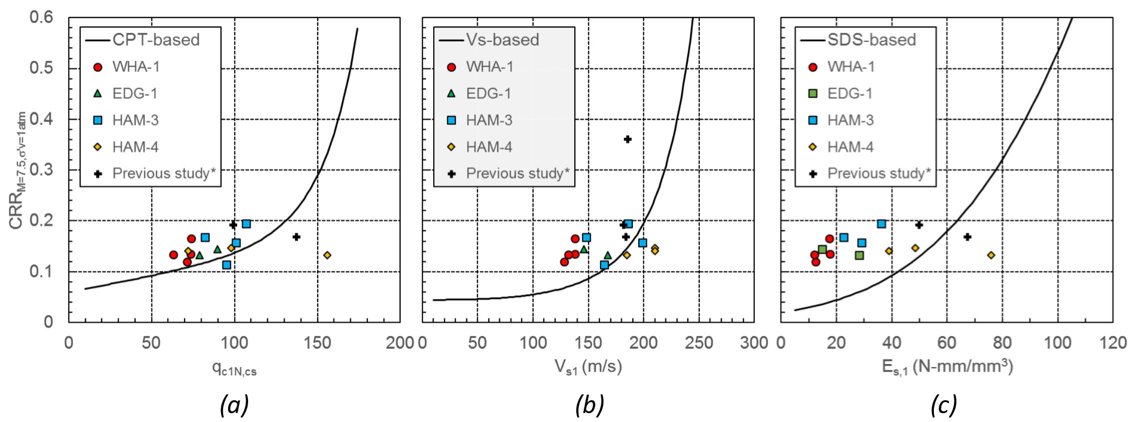


Figure 11. Comparison between lab- and field-based CRR: (a) using CPT; (b) using V_s; (c) using SDS.

It is observed from the figure that all the three field-based methods generally underestimate the liquefaction resistance of the pumice-rich deposits. Although the solid lines indicated are for equivalent clean sands and the actual samples have some amount of fines (up to about *FC*=20%), the appropriate curves may shift a bit to the right; however, the change would not be that significant. While penetration-based methods, such as CPT and SDS, are expected to provide underestimation due to particle crushing when the rods penetrate into the pumice-rich layer, even the V_s-based approach also underestimated the *CRR*.

Proposed procedure to estimate liquefaction resistance of pumice-rich soils

Geotechnical engineers currently prefer the use of field-based methods over laboratory testing; among the field-based methods, the V_s-based approach seems to be more practical as it is not affected by possible crushing of the pumice particle components. Hence, efforts were focused on the development of liquefaction triggering methods based on V_s.

The cyclic triaxial and bender element tests performed on pumice-rich sands showed that under similar D_r and σ'_c, pumice-rich sands have considerably lower V_s when compared to that of Toyoura sand due to the presence of crushable, porous and lightweight pumice particles with irregular surface texture. Moreover, under similar D_r and σ'_c, the liquefaction resistance of pumice-rich sands was higher than that of Toyoura sand due to the complex surface texture and the occurrence of particle crushing during cyclic loading, which resulted in more stable soil structure during cyclic shear application. As a result, the $(CRR)_{field}$ - V_{s1} relations for pumice-rich sands were located considerably to the left side of data for hard-grained sands. The obtained correlations for pumice-rich sands are compared with those derived for Toyoura sand in Figure 12, as well as to other hard-grained sands reported in the literature. Note that the position of the curve is affected by the pumice content, *PC*; as *PC* decreases, the curve shifts toward those of (pure) hard-grained sands.

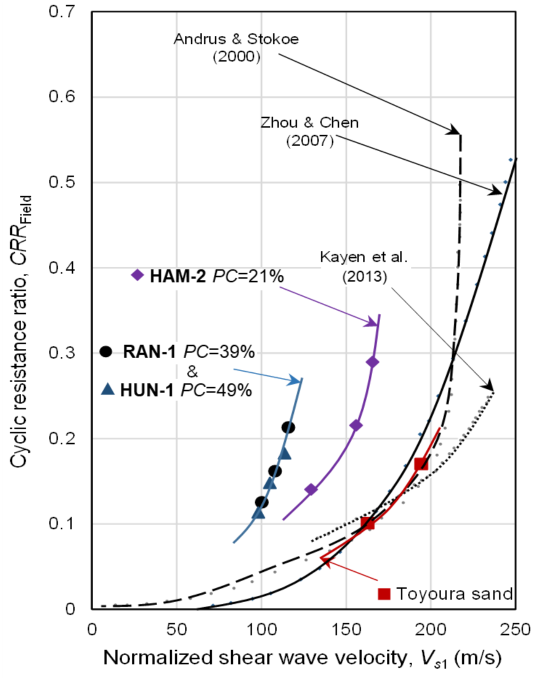


Figure 12. $(CRR)_{field}$–V_{s1} correlations for the tested pumice-rich materials (together with their pumice contents) and comparison with available correlations in literature.

It should be mentioned that the tests were performed on reconstituted sands, and the effect of soil fabric, structure, stress history, etc. on the behaviour of the specimens are not considered. However, when these are taken into account, it is expected that both *CRR* and V_{s1} of pumice-rich sands would increase, and may result in a trend not much different to those of the reconstituted specimen.

Further tests are currently being conducted on both reconstituted and undisturbed specimens of pumice-rich samples obtained from the other target sites to clearly define the $CRR - V_{s1}$ curves for sands containing various pumice contents. Nevertheless, at this stage, the following procedure is proposed to determine the liquefaction resistance of in-situ deposit of pumice-rich soils based on shear wave velocity:

Step (1): perform shear wave velocity profiling at the target site;

Step (2): obtain disturbed samples at target depths and perform the modified maximum density test to estimate the relative breakage, B_r;

Step (3): quantify their pumice contents (PC) using the chart shown in Figure 5; and

Step (4): for the estimated *PC*, use Figure 12 to approximate the $(CRR)_{field}$ at the target depth.

CONCLUSIONS AND KEY FINDINGS

Key findings resulting from this research are summarised below:

9) Image processing and chemical characterisation showed that the pumice sand particles present in pumice-rich soils from the target sites were quite similar, indicating that they came from identical source(s). These pumice sand components

were very crushable and had very irregular surface shape and texture due to their vesicularity.

10) The pumice contents of volcanic soils can be estimated according to the relative breakage the materials would experience during a modified maximum dry density test. The approach was validated by visually quantifying the amount of pumice-looking particles from ESEM images of the materials investigated.
11) High quality undisturbed sampling of pumice-rich sands was possible with techniques currently available in NZ. In particular, based on the experience of the researchers, Gel-push sampler and Dames and Moore sampler provided the best quality of samples.
12) The peculiar features of the pumice sand components made pumice-rich samples behave differently when subjected to undrained cyclic shear leading, as compared to normal sands. Due to particle crushing and subsequent interlocking, the patterns of development of axial strain and excess pore water pressure during cyclic loading were different, leading to their higher liquefaction resistance.
13) Pumice-rich sands have significantly lower G_{max} and V_s than hard-grained sands. The higher the pumice contents, the lower the G_{max} and V_s. Likewise, when compared to hard-grained sand, the G_{max} of pumice-rich sands was more dependent on σ'_c and less on e, due to the high angularity and elongated nature of the pumice particles.
14) Comparison of the liquefaction resistances obtained from cyclic triaxial tests on undisturbed samples with those estimated using field parameters indicated that the latter provided underestimation. Hence, current field-based methods of estimating liquefaction potential derived for normal sands would not work for pumice-rich sands.
15) However, it was possible to estimate the liquefaction resistance of pumice-rich sands from shear wave velocity by considering the effect of pumice content on the *CRR* and V_s of the deposit. Since V_s measurement is a non-intrusive testing technique, the effect of particle crushing during the test can be avoided.

IMPLEMENTATION TO PRACTICE

Some outputs of the research have been used by various end-users in the following ways:

- Adoption of recommendations on the best methodology to obtain undisturbed samples of pumice-rich deposits from the experience obtained in the research.
- Application of proposed laboratory method to quantify the pumice contents of volcanic soil samples, to supplement current approach of visual observations.
- Guidance on evaluation of small strain stiffness and undrained cyclic response of natural pumiceous deposits.
- Field-based liquefaction assessment of pumice-rich soil samples, especially the limitations of available simplified (empirical) methods.

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OUTPUTS & DISSEMINATION

PUBLICATIONS:

Journal papers:

Asadi MS, Asadi MB, Orense RP, Pender MJ. (2018). "Undrained cyclic behavior of reconstituted natural pumiceous sands". *J. Geotech. Geoenviron. Eng.*, ASCE, 144 (8): 04018045, https://doi.org/10.1061/ (ASCE)GT.1943-5606.0001912

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Keynote/Invited lecture:

Orense RP. (2019). "Field and laboratory assessment of liquefaction potential of crushable volcanic soils," Theme Lecture, *7th International Conference on Earthquake Geotechnical Engineering*, Rome, Italy, 17-20 June 2019, 442-460.

Conference papers:

Asadi MS, Orense RP, Asadi MB, Pender MJ. (2019). "Undrained monotonic behaviour of liquefied pumiceous sands". *2019 Pacific Conf. Equake Eng.*, Auckland, 8pp.

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LIST OF KEY END-USERS

- Regional councils (Waikato, Bay of Plenty)
- City/District councils (Hamilton, Tauranga, Whakatane)
- Ministry of Business, Innovation and Employment (MBIE)
- New Zealand Geotechnical Society (NZGS)
- Consulting engineers (Tonkin+Taylor, AECOM, WSP-Opus, CMW Geosciences, etc.)
- Geotechnical contractors (McMillan Drilling, Ground Investigation Ltd., etc.)

Objective 3: Seismic Assessment and Improvement of Existing Buildings

Topic 1: Detailed seismic assessment of undamaged existing buildings

Topic 1a: URM macroblock technique for complex URM buildings

Research Team: Same for topics 1a and 1b.
Prof. Jason Ingham, University of Auckland
Dr. Dmytro Dizhur, University of Auckland
Francisco Galvez, University of Auckland, PhD Candidate

Current New Zealand seismic assessment guidelines provide instructions for how to assess the maximum capacity of a simple URM building via consideration of the local out-of-plane capacity of parapets and vertically spanning walls and the overall in-plane capacity of the building. Emerging international practise is associated with use of the macroblock technique to divide complex problems into smaller, more achievable exercises by applying a displacement based kinematic analysis of rigid bodies. A New Zealand macroblock catalogue was derived from collapse mechanisms observed after past New Zealand earthquakes and the statistics of repeatable observations were studied. Applying the 27 macroblocks identified in the New Zealand Macroblock Catalogue and knowing the typology that corresponds to the building being assessed, every URM building in New Zealand, regardless of the complexity, can be assessed with a %NBS rating and with the potential collapse mechanisms captured. Using displacement based kinematic analysis the vulnerability of each macroblock can be assigned. The suitability of such simplified kinematic models (that are in full agreement with the currently implemented theory in the C8 guidelines) is being correlated and validated with 3D discrete rocking models subjected to a combination of earthquakes via incremental dynamic analysis (IDA). The numerical model has already been validated for the majority of macroblocks. An example of finished work performed on the simplest macroblock (rocking parapet – mechanics O2) is shown in the image below.

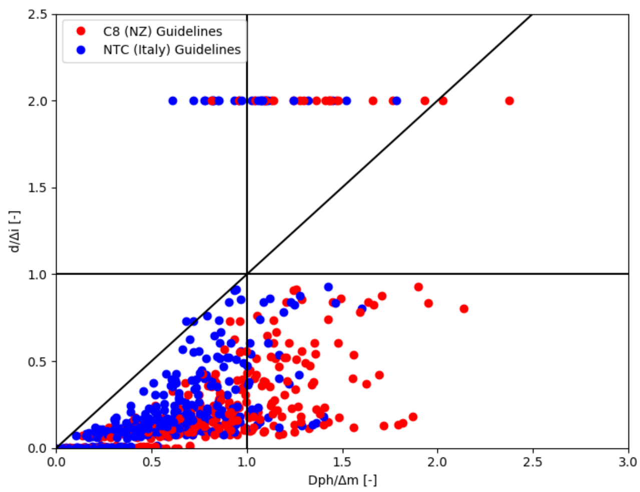


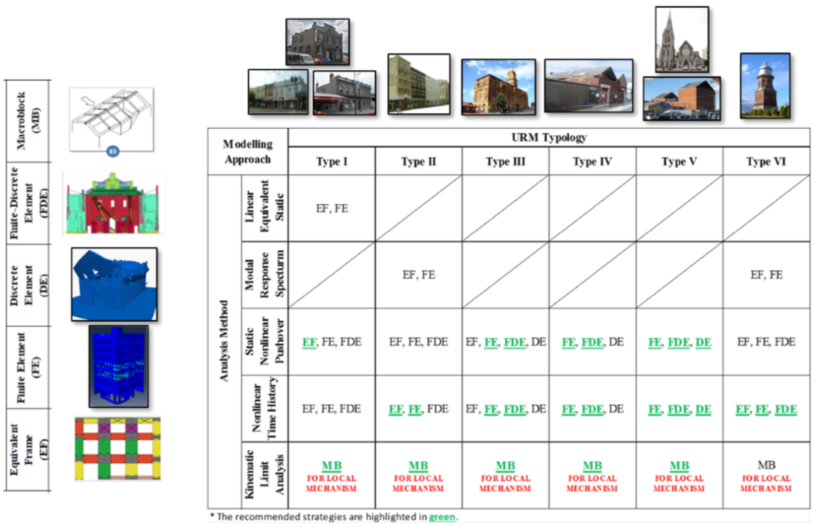
Figure 1: Demand/capacity plots for evaluation of the seismic assessment method versus results of incremental dynamic analyses.

A clear comparison between the macroblock analysis implemented in the NZ Eq-Assess methodology (horizontal axis) and the predicted rocking behaviour of the macroblock incrementally subjected to 30 different earthquakes can be made. The notation adopted in

the image corresponds to the used in the C8 guidelines, where D_{ph} was obtained from the displacement spectra of each earthquake used in the IDA and d is the maximum displacement observed in each time-history analysis. Simulations that collapsed were represented as $d/\Delta i=2$. If safety factors are ignored when calculating seismic risk, in the horizontal axis 0.5, 1, 1.5 and 3 correspond to 200, 100, 66 and 33%NBS respectively.

Topic 1b: URM numerical modelling procedures for complex URM buildings

A literature review of numerical modelling approaches for complex URM buildings was undertaken, with a summary provided here of the practical methods developed in academia and applied in the structural engineering industry. A modelling-driven classification of URM buildings was developed with the focus on the New Zealand URM buildings stock. Six building types were defined based on typical damage mechanism and seismic response of URM buildings, in a range from simple and small buildings to large scale multi-storey buildings including special structures with complex geometry. This proposed taxonomy is easy to match with the suitable modelling strategies, in which the advantages and limitations were discussed in the literature review. The proposed systematic modelling framework breaks down the modelling strategies into fundamental components, including modelling approach and analysis methods, and is then presented in a simple matrix format that contains the recommended modelling options corresponding to the building types (Figure 2). This framework allows practitioners to quickly identify appropriate modelling strategies and hence the best practice can be determined for various purposes of URM modelling.



Modelling Approach		URM Typology					
		Type I	Type II	Type III	Type IV	Type V	Type VI
Analysis Method	Linear Equivalent Static	EF, FE					
	Modal Response Specturm		EF, FE				EF, FE
	Static Nonlinear Pushover	EF, FE, FDE	EF, FE, FDE	EF, FE, FDE, DE	FE, FDE, DE	FE, FDE, DE	EF, FE, FDE
	Nonlinear Time History	EF, FE, FDE	EF, FE, FDE	EF, FE, FDE, DE	FE, FDE, DE	FE, FDE, DE	EF, FE, FDE
	Kinematic Limit Analysis	MB FOR LOCAL MECHANISM	MB FOR LOCAL MECHANISM	MB FOR LOCAL MECHANISM	MB FOR LOCAL MECHANISM	MB FOR LOCAL MECHANISM	MB FOR LOCAL MECHANISM

* The recommended strategies are highlighted in green.

Figure 2: Proposed Systematic Numerical Modelling Framework.

A case study of the Basilica of the Sacred Heart in Timaru, New Zealand was chosen to demonstrate application of the proposed systematic numerical modelling framework for complex URM buildings. The study showed that the proposed framework is a user-friendly guideline that allows practitioners to explore a range of approaches for modelling a URM church building with complex geometry. An overview of the application, including the modelling methodology, geometry modelling, summary of the analytical model and analysis results, are provided as an example to illustrate the finite-discrete element modelling procedures involved in assessment. The macro-block approach adopted in the local damage assessment demonstrated the good practice and importance for verification of complex URM building modelling. The information in this case study serves as a reference for practitioners to evaluate the most cost-effective modelling approaches for similar URM buildings.

The proposed framework represents the best practices available in the engineering industry and can be further developed when applied in other projects with different typologies. The future development of this framework will provide additional information and guidance to quantify the associated time and cost for projects at the preliminary stage.

Topic 1c: Assessment procedures for corroded concrete structures

Research Team:
Prof. Jason Ingham, University of Auckland
Dr. Lucas Hogan, University of Auckland
Assoc. Prof. Alan Scott, University of Canterbury
Dr. Dmytro Dizhur, University of Auckland
Sunil Nataraj, University of Auckland, PhD Candidate

The durability of older reinforced concrete (RC) structures is significantly affected by corrosion of the reinforcement and is a critical issue for New Zealand RC buildings because of the close proximity of most urban centres to a marine environment. In addition, sustained degradation without maintenance can significantly reduce the base shear and displacement capacity of a structure during an earthquake. Currently, the seismic assessment of such buildings is a challenging task because there is limited and dispersed guidance for how to assess the residual seismic capacity of corroded RC members and determine the likely overall seismic behaviour of an RC building exhibiting corrosion. A seismic assessment methodology has been developed to account for the effect of corrosion on the residual strength and deformation capacity of corroded RC members. Analytical models to predict material and member properties have been considered and modified to account for pitting corrosion. The modified corrosion-dependent models proposed have been validated against a large database of experimental results from the available literature. The Simplified Lateral Mechanism Analysis (SLaMA) procedure detailed in the EQ-Assess Guidelines can now be implemented with the proposed corrosion dependent modifications.

Further, an existing corroded RC building in Auckland, New Zealand constructed in the year 1928 was found to exhibit severe pitting corrosion. Hence the same building has been considered as a case study and assessed using the proposed methodology to investigate the reduction in seismic capacity due to corrosion. While no change in failure mechanism was found, the overall displacement capacity of the building was reduced by 25% compared to the assessed uncorroded condition. The result shows that the column sidesway mechanism is concentrated at the fourth storey due to weak column-strong beam behaviour and possess a 27%NBS rating, resulting in the building being assessed as earthquake-prone. Finally, to estimate the changes in capacity and failure mode with respect to exposure time in a corrosive environment, the long-term corrosion effect was studied for the case study building assuming no remediation of the corroded reinforcement. After 30 years of continued deterioration, the base shear capacity reduced to almost 15%, and more significantly, the ultimate displacement capacity was reduced by more than 40%, see Figure 3. Hence, a severely corroded RC building would experience a significant reduction in displacement capacity and will require timely assessment to understand the severity of such buildings with continued deterioration and the criticality of future remediation.

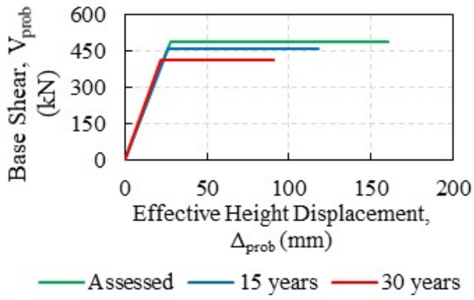


Figure 3: Probable lateral capacity curves for the case study frame with continued corrosion deterioration

CONCLUSIONS AND KEY FINDINGS

1. The combination of the 27 identified macroblocks were sufficient to assess complex URM buildings.
2. DEM has been shown to be capable to reproduce laboratory testing of various macroblocks subjected to earthquakes.
3. Comparison between DEM simulations and seismic assessment guidelines have shown that New Zealand methodologies are capable to capture the behaviour of the simplest macroblock (parapet overturning). The rest of the macroblocks are being currently modelled to be compared with simplified kinematic models to be implemented in future guidelines.
4. Corrosion in RC structure can reduce base shear capacity and more significantly displacement capacity. Hence, there is a significant need for seismic assessment of corroded RC buildings.
5. The capacity of corroded RC buildings is time dependent, henceforth, timely assessment and maintenance of such buildings is required.

IMPLEMENTATION TO PRACTICE

Topic 1a and 1b:

During February and March 2019 the first round of SESOC seminar took place. The macroblock method was presented to the New Zealand engineering community as a workshop on detailed seismic assessment of complex unreinforced masonry buildings. Attendees were able to identify potential macroblocks in the building to be assessed and derive a %NBS for each of them. A simplified procedure will be developed based on the findings of Topic 1 and implemented in future seismic assessment guidelines.

Topic 1c:

The developed seismic assessment methodology has been presented in an international conference in Taipei, Taiwan during the month of September (International Conference in Commemoration of 20th Anniversary of the 1999 Chi-Chi Earthquake, Taipei, Taiwan, 15-19 September). A detailed seismic assessment guideline and journal paper will be published soon.

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OUTPUTS AND DISSEMINATION

Topic 1a and 1b:

- COMPUTATIONAL CIVIL ENGINEERING - May 30th-31st 2019, Iasi, Romania
- SESOC Seminar Series 2019: Macroblock Method. Detailed Seismic Assessment of complex masonry buildings.
- ECCOMAS - 2nd International Conference on Recent Advances in Nonlinear Models – Design and Rehabilitation of Structures

Topic 1c:

- Presentation at an international conference, International Conference in Commemoration of 20th Anniversary of the 1999 Chi-Chi Earthquake, Taipei, Taiwan, 15-19 September.

LIST OF KEY END USERS

- Ministry of Business, Innovation and Employment (MBIE)
- Consulting engineers undertaking seismic assessment of URM and corroded RC buildings.

Topic 2: Residual capacity of earthquake damaged multi-storey buildings

Topic 2a: Steel Buildings: Update to assessment guidelines for yielded links in eccentrically braced frame buildings

Research Team:
AP Charles Clifton, AI - 2, University of Auckland
Prof Emeritus George Ferguson, University of Auckland
Dr Michael Hodgson, University of Auckland
Mr. Robert Currie, University of Auckland, ME Candidate, completed 2017

ABSTRACT
Eccentrically Braced Frames (EBFs) are a form of braced frame steel framed seismic resisting system where the braces do not meet at a point on the collector beam, leaving a length of collector beam between the braces in which the web will deform inelastically in shear under severe earthquake action. This is called the Active Link. Shear deformation in a steel plate is very ductile and the EBFs combine the elastic strength and stiffness of a braced frame with the dependable, ductility capacity of a moment resisting frame. As such, prior to the 2010/2011 Canterbury earthquake series, they had become the most commonly used seismic resisting system in multistorey buildings in New Zealand.

That earthquake series was the first worldwide to push EBF systems into the inelastic range, with widespread shear yielding of the active links in multi-storey EBF systems in the Christchurch CBD. In almost all buildings, yielding was confined to the active links and the buildings returned to their original position with residual drifts of under 0.15%, meaning the buildings could be repaired and returned to service. That raised the question as to how much inelastic demand could be tolerated within a yielded active link before replacement was required.

A research programme to answer this was undertaken from 2011 to 2014, resulting in guidelines for active link assessment and provisions for replacement being produced and presented to MBIE in 2015, with an update in 2016. However, questions relating to the influence of strain aging on the relationship between hardness and plastic strain and on the future deformation capacity were not conclusively answered in that study. This necessitated a follow-up study to specifically address those issues, that follow-up study was funded by the NHRP under this programme.

That study was undertaken in 2016/2017 and showed that, for the seismic grade of steels required to be used for EBF active links, the cyclic inelastic shear strength of the steel is not affected by strain ageing with regard to maximum plastic capacity and cumulative plastic capacity and the influence on hardness readings is only very slightly conservative, thus the recommendations of the 2016 report don’t need to be amended to account for the influence of strain ageing.

INTRODUCTION

Background

The 2010/2011 Canterbury earthquake series was the first worldwide to push Eccentrically Braced Frames (EBFs) into the inelastic range, with widespread yielding of the Active Links reported [1]. At that time, there were no guidelines available for determining whether these

yielded links could be left in place and, if so, what the influence of the increase in strength would be on the post earthquake residual capacity.

A research project was instigated to answer these questions and provide suitable guidance. This was commenced in 2011 and comprised a Phd project [2], development of engineering guidelines for post-earthquake active link and EBF system assessment [3]. The outcomes were also presented at conferences and in journal papers [4-6].

However, Nashid's study did not specifically address the question of strain ageing of the yielded active links on their subsequent inelastic cyclic behaviour. This was addressed in a 2016/2017 study by Currie, with the outputs presented in a ME thesis [7] and in a paper [8] to the 2017 NZSEE Conference.

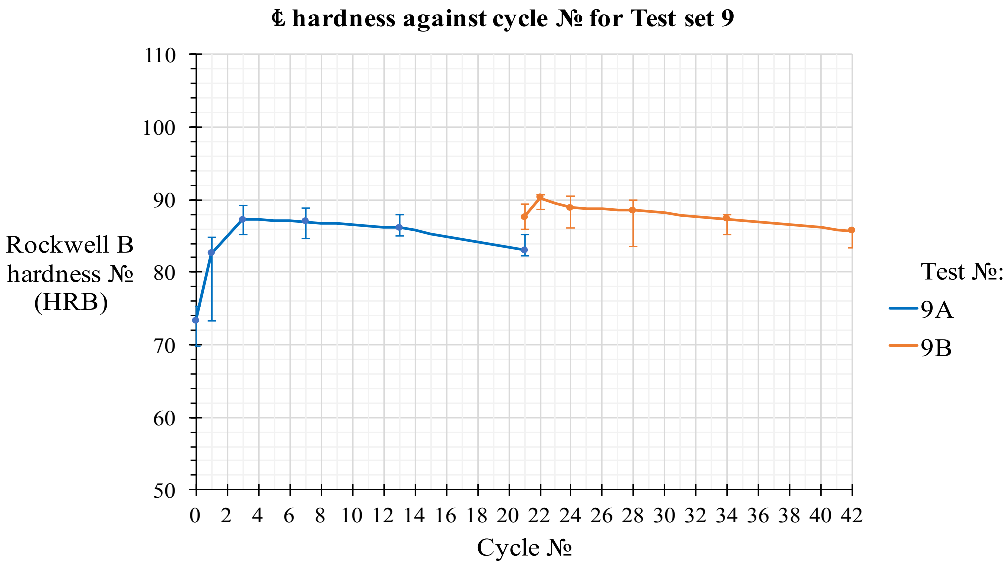

Figure 4 Centreline hardness history for Test Set 9 (from [x])

Figure 4 shows the outputs from one of the test series undertaken. Test series 9A is the first test series undertaken on the unyielded active link, this one comprising a series of inelastic shear cycles starting at 7% plastic shear strain and reducing to 2% plastic shear strain; representative of the impact of a severe near fault earthquake. This is then fully strain aged by suitable heat treatment and subjected to a repeat of the loading history, shown as test series 9B. This was one of 12 active link pairs tested under a range of deformation controlled inelastic loading profiles comprising constant strain cycles, ascending strain cycles descending strain cycles and variable strain cycles.

Topic 2a: Residual Capacity of Earthquake-Damaged Concrete Buildings

Research Team:
Prof Ken Elwood, University of Auckland
AP Charles Clifton, University of Auckland
Dr. Kai Marder, PhD student University of Auckland (completed 2018)
Mehdi Sarrafzadeh, PhD student University of Auckland (PhD expected 2020)
Assistant Professor Chris Motter, Washington State

INTRODUCTION

Reinforced concrete buildings that satisfied modern seismic design criteria generally behaved as expected during the Canterbury and Kaikoura earthquakes in New Zealand, forming plastic hinges in intended locations. "Plastic hinges" are regions in a structure where, by design, nonlinear (plastic) behaviour is intended to occur during an earthquake thereby dissipating some of the energy generated by the earthquake. While this meant that life-safety performance objectives were met, widespread demolition and heavy economic losses took place in the aftermath of the earthquakes. The Christchurch Central Business District was particularly hard hit, with over 60% of the multi-storey reinforced concrete buildings being demolished. A lack of knowledge on the post-earthquake residual capacity of reinforced concrete buildings was a contributing factor to the mass demolition.

As summarised in Elwood et al (2016), many aspects related to the assessment of earthquake-damaged reinforced concrete buildings require further research. This project focussed on improving the state of knowledge on the post-earthquake residual capacity and reparability of moderately damaged plastic hinges, with an emphasis on plastic hinges typical of modern moment frame structures. The formation of plastic hinges, however, constitutes damage to the structural system with cracks, spalling, and yielding of reinforcement. The most common repair method for a moderately damaged plastic hinge is epoxy injection of cracks and patching of spalled concrete. A targeted test program on seventeen nominally identical large-scale ductile reinforced concrete beams, three of which were repaired by epoxy injection following initial damaging loadings, was conducted to support these objectives (Marder et al 2018a). Test variables included the loading protocol, the loading rate, and the level of restraint to axial elongation.

To date the most common metric used to represent the damage to a plastic hinge has been the maximum residual crack widths. This study demonstrates that residual crack widths are dependent on residual deformations, and are not necessarily indicative of the maximum rotation demands or the plastic hinge residual capacity (Marder et al 2019a). Crack width data taken from standard cyclic tests show trends that are artefacts of the loading protocols rather than causal relationships. Given these limitations, residual crack widths may be best used as a qualitative metric. In beams, metrics capturing the distribution of cracking can indicate whether or not sufficient inelastic rotations to induce strain hardening in the longitudinal reinforcement are likely to have occurred. Rather than focusing solely on component damage, peak drift demand should be estimated through analysis of the building system and validated against the distribution of damage observed in the building. This peak drift demand from the model can be compared with the range of peak demand estimates based on the extent of plastic hinge damage. The experimental data is further used to demonstrate that prior earthquake cycles below drifts of 2% of the beam length did not influence the ultimate drift or energy dissipation capacity (Marder et al 2018b); hence, knowledge of the peak drift demands is critical to the assessment of residual capacity and the criticality of earthquake damage.

This study further demonstrates that the residual stiffness of a plastic hinge can be approximated as a function of the estimated maximum deformation demand during the damaging earthquake (Marder et al 2019a). A simple inverse relationship between the residual stiffness and the displacement ductility was found to yield a lower-bound estimate, based on a dataset of reinforced concrete components where the residual stiffness was measured after application of arbitrary earthquake-type loadings.

Experimental data, from the authors and the literature, are used to demonstrate that the strength and deformation capacity of plastic hinges with modern seismic detailing are often unreduced as a result of moderate earthquake-induced damage, albeit with certain exceptions (Marder et al 2019a). The effects of prior yielding of the longitudinal reinforcement, accounting for the low-cycle fatigue and strain ageing phenomena were investigated. A material-level testing program on the low-cycle fatigue behaviour of grade 300E reinforcing steel, supplemented with additional data from the literature, was used to demonstrate that for the number of cycles and strain levels expected in most earthquakes the strain capacity of 300E reinforcement remains large despite reductions due to strain ageing and low cycle fatigue.

In terms of the performance of beam plastic hinges repaired with epoxy injection, when compared with nominally identical (unrepaired) test specimens, repaired specimens were found to exhibit (Marder et al 2019b):

- increased strengths (<10% higher),
- reduced secant stiffness to yield values (15 to 25% lower),
- increased axial elongation due to locked in residual strains from prior loading,
- longer lengths along which inelastic damage was spread, and
- comparable energy dissipation and deformation capacities.

While based on limited data, these results suggest that epoxy injection is a viable repair alternative for moderately damaged plastic hinges in reinforced concrete beams as long as the reduced stiffness, locked in residual strains, and potential for increased flexural strength are considered in the assessment of the repaired building.

The above conclusions are further supported by testing of beams extracted from a building damaged in the Kaikoura Earthquake and subsequently demolished (Sarrafzadeh and Elwood 2019). The tests demonstrated that while the beam reinforcement had clearly experienced strain hardening and ageing due to plastic hinging during the Kaikoura Earthquake, the beams were still able to achieve 6% drift prior to failure. Furthermore, concerns over any embrittlement of the reinforcement were unfounded for these beams as the final failure mode was controlled by shear distress in the plastic hinge rather than fracture of the reinforcement.

CONCLUSIONS AND KEY FINDINGS

The key findings with regard to steel buildings for topic 2a are summarised below:

1. The relationship between measured hardness and cyclic shear strain determined by Nashid is minimally affected by strain ageing.

2. The peak plastic shear strain and the cumulative plastic shear strain limits are minimally affected by strain ageing.

3. The initial recommendation to base the criteria for replacement on reaching up to 50% of the cumulative plastic displacement limits given by Nashid [4] are reasonable; all specimens loaded to within that limit for test series A survived the full test series B without failure and exhibiting very similar load – deflection behaviour.

4. The recommendations to MBIE given in [3] remain appropriate for the post earthquake assessment of EBF systems following a severe earthquake to determine if replacement of the yielded active links is needed.

The key findings from this research with regard to the residual capacity of moderately damaged concrete beams designed to satisfy NZS 3101 ductile detailing provisions can be summarised as follows:

1. Extent of cracking (e.g. number of cracks), rather than maximum residual crack widths, should be used, in combination with results from building model analysis, to estimate the maximum deformation demands experienced by a damaged concrete beam during an earthquake.
2. Cycles below 2% drift typically will not influence the drift capacity of a modern ductile beam.
3. Stiffness of a concrete beam can decrease rapidly due to prior earthquake loading; the reduction in stiffness can be conservatively taken as the inverse of the displacement ductility in the prior earthquake.
4. For the number of cycles and strain levels expected in most earthquakes, the strain capacity of 300E reinforcement in earthquake-damaged beams remains large despite reductions due to strain ageing and low cycle fatigue.
5. Moderately damaged beams repaired with epoxy injection will typically exhibit similar deformation capacity to undamaged beams, increase (~10%) in flexural strength, and a (~20%) decrease in stiffness.

The above conclusions are based on experimental studies by the authors and additional data sourced from the literature from ductile beams and columns experiencing moderate earthquake damage. Further research is required to validate the above conclusions for other design conditions.

IMPLEMENTATION TO PRACTICE

Topic 2a:

Curries was able to apply the procedure to yielded active links from the Customhouse building, Wellington, which had active links between coupled rocking reinforced concrete shear walls. These showed peak plastic strains of over 8% from the 2016 Kaikoura Earthquake and, being over the 7% peak plastic strain limit, the active links were replaced.

Topic 2b:

This research was informed by research needs identified by the MBIE Residual Capacity Working Group and provided guidance for engineers during the assessment of buildings after the Kaikoura Earthquake. The publications have been widely distributed in engineering community and informing the assessment and repair of buildings in Wellington.

Recognising New Zealand's leadership and the importance of the topic of residual capacity and repair, Ken Elwood was invited in 2018 to lead a multi-year US government-funded project to identify when and how earthquake-damaged buildings can be repaired (ATC 2019). The project includes researchers from four US universities and practicing engineers from US and NZ (with US funding from FEMA and supplemented by NZ funding from EQC and QuakeCoRE).

ACKNOWLEDGEMENTS

Topic 2a:

The authors would like to acknowledge the NHRP for two lots of funding for this project. The first is from Objective 3; the second is unspent money from the PhD contract with Nashid. It was the combination of this funding that covered the experimental testing and the student stipend. They would also like to acknowledge the excellent support from the University of Auckland Structures Test Hall staff especially Lucas Hogan and Jay Naidoo.

Topic 2b:

The research presented here was primarily funded by NHRP with supplemental funding from QuakeCoRE and MBIE Building Performance. This funding is gratefully acknowledged. The authors would also like to acknowledge the excellent support from the University of Auckland Structures Test Hall staff especially Lab Director Lucas Hogan.

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Topic 2b:

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OUTPUTS AND DISSEMINATION

Topic 2a: References [7, 8] are from this project.

Topic 2b: References above are from this project.

LIST OF KEY END-USERS:

Topic 2a:

- Ministry of Business, Innovation and Employment (MBIE)
- New Zealand HERA
- Steel Construction New Zealand
- Consulting engineers undertaking post earthquake building assessment of EBFs with yielded active links.

Topic 2b:

- Ministry of Business, Innovation and Employment (MBIE)
- Concrete New Zealand
- Applied Technology Council (ATC), California
- Consulting engineers undertaking post earthquake building assessments.

Topic 3: Seismic improvement of existing buildings

Research Team:
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Prof. Milo Kral, University of Canterbury
Prof. Misko Cubrinovski, University of Canterbury
Assoc. Prof. Alan Scott, University of Canterbury
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Dr. Veronica Ligabue, University of Bologna, (Visiting Researcher at UC)

ABSTRACT
The NZ Society of Earthquake Engineering (NZSEE) 2006 AISPBE guidelines provide (only) high level qualitative informative on methods for improving the performance of existing buildings. Research data related to the seismic improvement of existing buildings is available from past New Zealand research (most notably the 2004-2010 Retrofit Solutions project) and from international sources but has not yet been prepared in the form of a comprehensive performance-based framework. Furthermore, recent legislation has underscored the need for more detailed and accurate guidance on seismic improvement options to enable appropriate cost-effective decisions regarding seismic improvement options, and the Canterbury earthquakes have revealed the need for better guidance on repair interventions.

In response to these needs the present research has focused on:

- Developing a framework for performance-based seismic improvement that incorporates both foundation and superstructure interventions and correlates component level interventions with global structural response
- Developing a framework that allows benefit-cost analysis (and/or other decision support tools) to inform structural intervention investment decisions
- Developing a better understanding on feasibility and costs of repair interventions, linking with topic 2 above, as distinct from interventions appropriate for installation in an undamaged building.

INTRODUCTION
The crucial need to develop and implement simple and cost-effective (repair and) retrofit strategies and solutions for existing structures has been once again emphasized, if at all needed, by the recent catastrophic earthquake events.

The significant socio-economic impact of the Canterbury earthquakes sequence in the 2010-2011 has triggered a step-change in the high-level approach towards the implementation of

seismic risk reduction. The typical passive or volunteer-based approach of local authorities or building owners has been replaced by the New Zealand most systematic by-law enforcement of a national-wide mandatory long-term program to assess the seismic vulnerability/capacity of the whole (non-dwelling) building stock. If below a minimum level of capacity/demand ratio (expressed in terms of %NBS, or % New Building Standard), seismic upgrading/retrofit beyond that threshold must be carried out at the owner's expenses within a given risk-based timeframe.

As part of this government-driven program, new guidelines for the "Seismic Assessment of Existing Buildings" (NZSEE2017) have been prepared and are now publicly available, through the joint effort of the key engineering learning societies (NZSEE, SESOC, NZGS) under the coordination and endorsement of MBIE (Minister of Business Innovation and Employment) and EQC (Earthquake Commission). A comprehensive dissemination program has also been initiated to the technical and non-technical community.

This NHRP project has been functional to the development and further refinement of key aspects of the aforementioned Seismic Assessment Guidelines, focusing on the practical aspects faced by practicing engineers, whilst, in parallel, developing a performance-based retrofit framework to support the selection of alternative retrofit options as well as the delicate decision making of repair vs. demolition.

This report provides a very brief summary of the NHRP research study. It highlights at a very high level some of the research aspects and outputs, without entering into complex technical details. A comprehensive summary of the research methodology and outputs is provided in the journal papers, conference papers and technical reports listed in the Outputs and Dissemination Section.

The research programme has been carried out with a very fruitful collaboration with international partners and exchange of visiting researchers.

Research Objective No. 1: Framework for Performance-Based Seismic Improvement that incorporate Superstructure and Foundation system

Accounting for the Effects of Soil Structure Interaction

The complex failure mechanisms that can occur in existing buildings often makes seismic analysis through complex numerical modelling (e.g. non-linear time history analyses) very difficult.

The effects of interaction between soil-foundation and the superstructure (e.g. building, bridge), referred to as SFSI interaction, can modify the dynamic response of the building and can completely change the deformation/failure mechanisms in the structure. These effects are especially important to consider when considering retrofit options for a structure, which may result in excessive loads being applied to the foundation.

The assessment of our ageing building stock is necessary to understand our seismic risk and the consideration of SFSI in this assessment is paramount.

In this research project focus has been given on the development of a analytical (rather than numerical or computer-based) procedure to assess existing buildings and rationally consider the effects of SFSI.

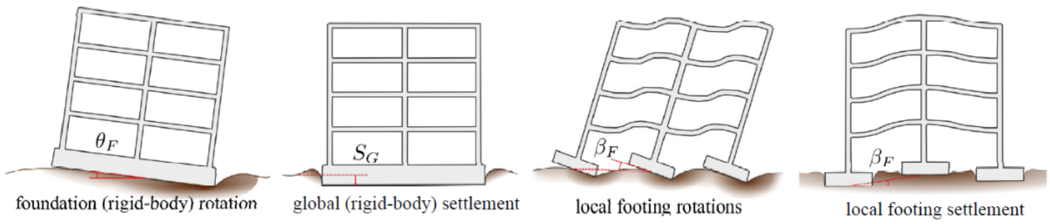


Figure 1.Building-Foundation combined Failure Mechanisms (Millen et al., 2019)

Integrated Performance-based Framework

The current framework for performance-based design sets out performance criteria that consider the performance of the superstructure at discrete design levels with no explicit consideration for foundation performance. However, recent events, such as the Christchurch earthquake series (2010-2011) resulted in buildings being deemed irreparable due to foundation damage (Cubrinovski et al., 2011). Numerous numerical studies have also demonstrated that foundation-performance can adversely increase the peak and residual displacements of a building (e.g. Moghaddasi et al., 2011). It is therefore necessary to explicitly consider foundation deformations within the building design process to comprehensively assess the performance.

This research programme has developed a framework for the inclusion of transient and residual foundation deformations within performance-based design.

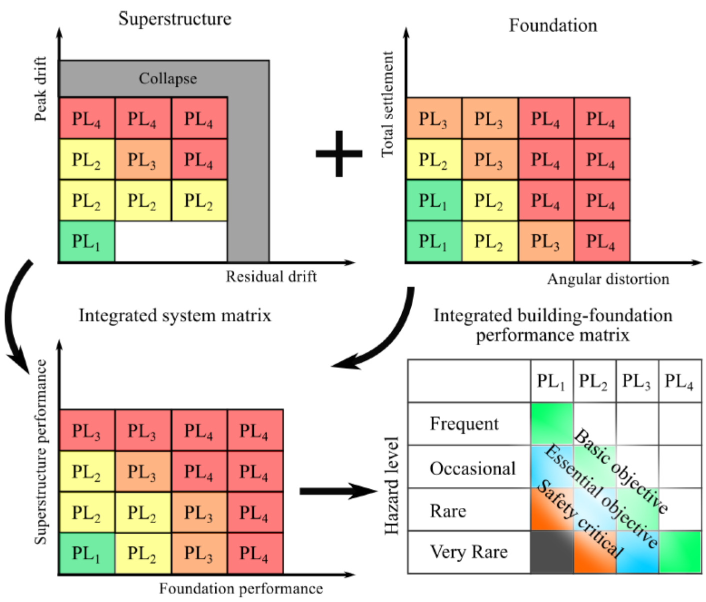


Figure 2. Integrated Building-Foundation Performance Matrix (Millen et al., 2019)

The framework has been complemented by a simplified displacement-based design procedure to design building-foundation systems. Empirical equations have been developed based on numerical time history analyses that provide an estimation of foundation residual deformations based on the foundation peak rotation. A design example has been developed and presented for a six-storey concrete wall building and analysed by time-history analysis to demonstrate the simplicity and accuracy of the design procedure.

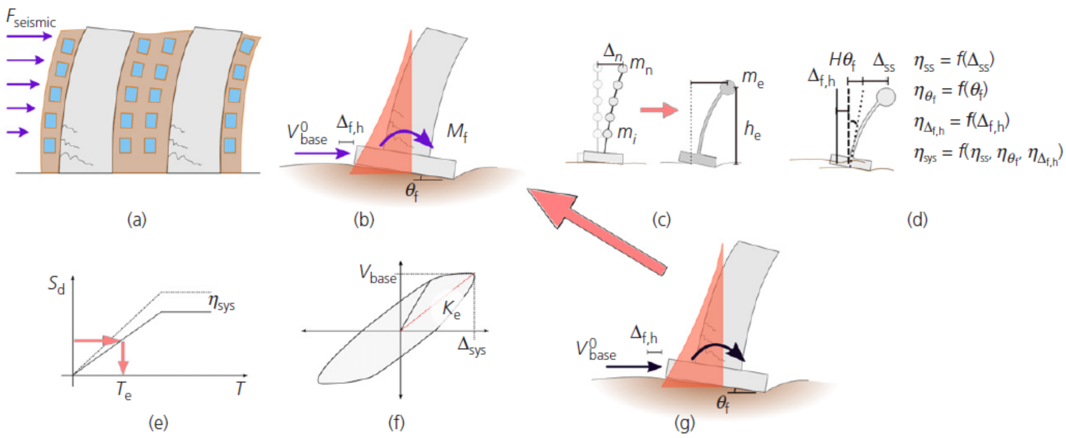


Figure 3. Schematic illustration of the principal steps of a Displacement-Based Design procedure considering non-linear SFSI effects (Millen et al., 2018)

Research Objective No. 2: Cost-Benefit Evaluation of Alternative Retrofit Solutions

Development and further refinement of the SLAMA procedure

Prior, and in order, to define a proper seismic retrofit solution it is crucial to develop an accurate and reliable assessment for the vulnerability of the existing building.

Using an analogy with the field of medicine, it is necessary, at first, to improve and standardize the tools and procedures ('protocols') for the 'diagnosis' and 'prognosis' of the seismic vulnerability and of the expected performance of existing buildings, based on state-of-the-art but simplified methodologies (analytical rather than numerical approaches) that could highlight the structural weaknesses of the building system while ensuring consistency of results and proper level of independently from the operators. Similarly, suitable "therapeutic pathways" or appropriate retrofit strategies can be defined by comparing alternative options through a cost-benefit approach (Pampanin, 2017)

As part of this NHRP research project, a significant effort has thus been dedicated to an extensive validation and further refinement of the SLAMA (Simple Lateral Analysis) procedure, core engine of the NZSEE Seismic Assessment Guidelines. A comprehensive numerical investigation has been carried out to investigate the accuracy of the analytical approach via comparison with numerical 2D- pushover on 40 RC frames. The SLaMA procedure has proven to be effective in capturing the plastic mechanism of the frames, including global or soft-story mechanisms. Further-yet-simple refinements of the procedure for bare frames have been suggested along with significant and novel integrations for masonry infilled frames and dual (wall-frame) systems.

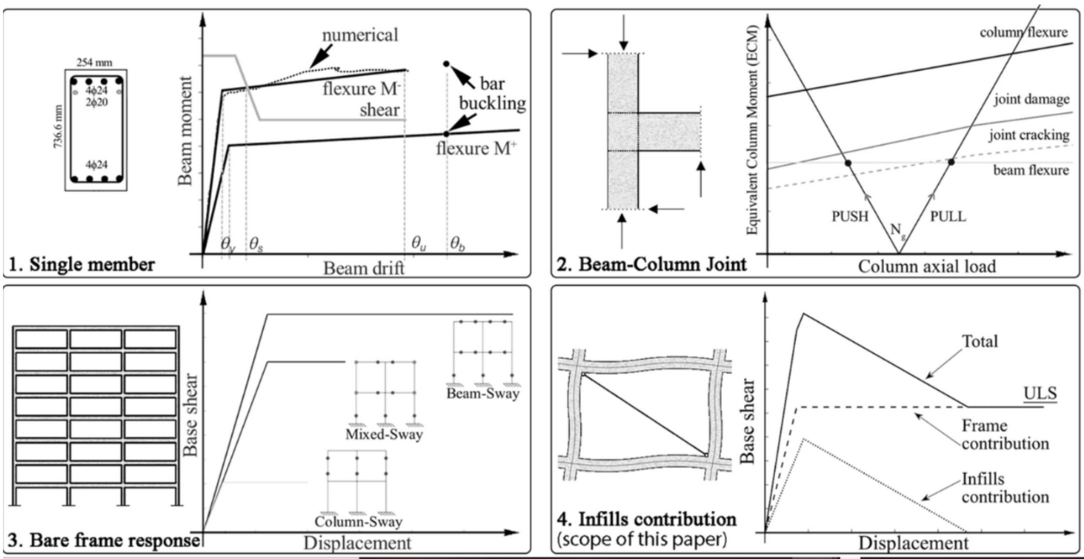


Figure 4. Overall Scheme of the SLaMA method for bare and infilled frames

Selection of Alternative Retrofit Strategies

The SLaMa method can be used for a relatively quick (when compared to what is required by detailed non-linear pushover end time-history analyses) estimation of the expected behaviour and performance of the building (or a class or buildings) before and after a retrofit/strengthening intervention, thus becoming a fundamental supporting tool for the implementation of a medium-long term strategy of seismic-risk reduction at national scale.

In fact, as part of the analytical evaluation of the force-displacement capacity curve of the system, the sequence of local and global mechanisms can be captured, (i.e. what happens at what stage).

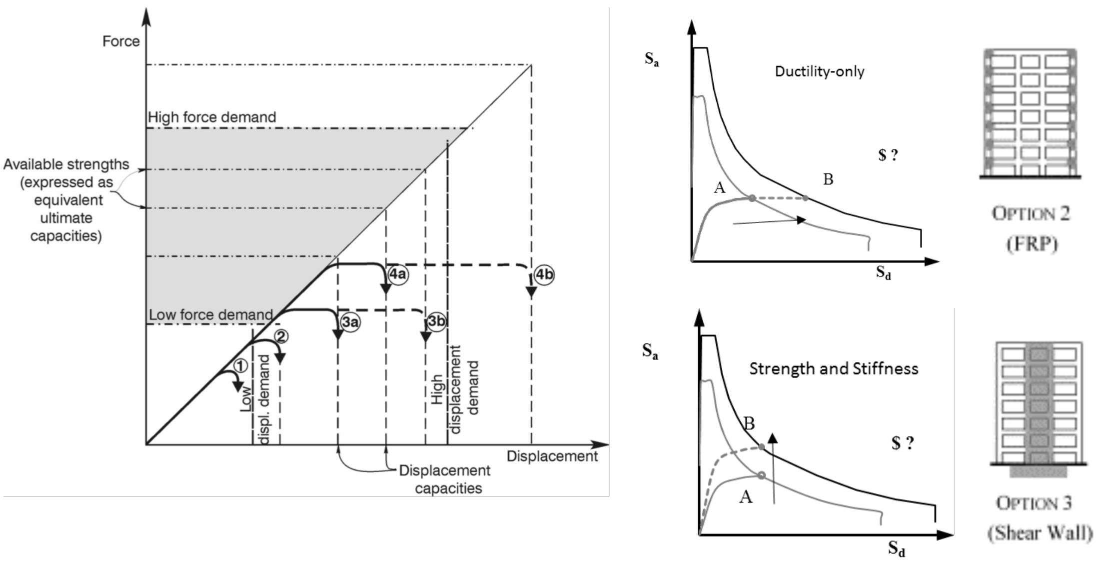


Figure 5. Left: use of SLaMA method to capture the sequence of events/mechanisms within an analytically-derived pushover capacity curve. Right: selection of alternative retrofit strategies and techniques to achieve the targeted performance (Pampanin, 2017).

The feasibility and efficiency, in the contest of a performance-based retrofit approach, of adopting and/or combining different solutions such as Fiber Reinforced Polymers, low-invasive low-cost metallic diagonal haunches and different dissipative bracing systems, post-tensioning wall systems or selective weakening techniques, can be and has been analysed, based on numerical and experimental evidences and within the framework of a performance-based retrofit philosophy.

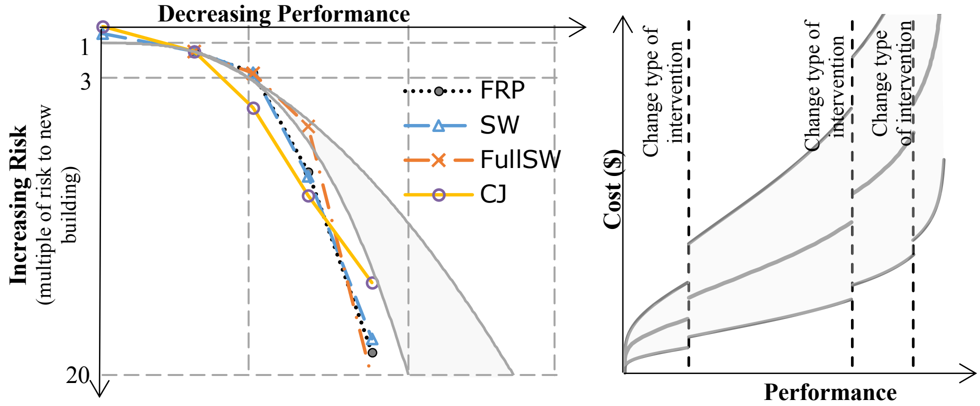


Figure 6: (a) Correlation between performance and risk (reproduced from the NZSEE2006 Guidelines); (b) expected cost of retrofit intervention as a function of performance; (c) alternative strategies for the achievement of 100%NBS (adapted from Ligabue et al., Pampanin, 2017)

Research Objective No. 3: Residual Capacity and Repair options for Damaged Buildings

Evaluation of Post-earthquake Capacity of Real Reinforced Concrete Buildings

One of the most controversial issues highlighted by the 2010-2011 Christchurch earthquakes sequence (CES) has been the lack of a) comprehensive and robust guidelines to assess the residual capacity of damaged modern buildings, as well as b) in depth and evidence-based knowledge for selection and implementation of a reliable repairing technique capable of bringing (either totally or partially) the structure back to its pre-earthquake condition.

Arguably, partly (although not exclusively) as a result of such lack of knowledge and guidelines, many modern buildings, in a number exceeding typical expectations from past experience at international level, have ended up being demolished.

As part of the research project, an experimental and numerical investigation was carried out on three real beam-column joint subassemblies extracted from a 22-storey reinforced concrete frame building constructed in late 1980s at the Christchurch's Central Business District (CBD) area, damaged and demolished after the 2010-2011 Canterbury earthquakes sequence (CES).

Figure 7 – Elevation of the PWC building during the deconstruction process (left); extraction of one of the "H frames" (upper-right); typical floor plan view at the upper levels (lower-right), red lines indicate locations where the specimens were taken out of the 16th floor level.

The specimens were tested at the Structural Laboratory of the University of Canterbury under quasi-static cyclic displacement controlled lateral loading (simulated earthquake loading). One of the specimens, showing no visible residual cracks was cyclically tested in its as-is condition. The other two specimens which showed residual cracks varying between hairline and 1.0mm in width, were subjected to cyclic loading to simulate cracking patterns consistent with what can be considered moderate damage. The cracked specimens were then repaired with an epoxy injection technique and subsequently retested until reaching failure. The epoxy injection techniques demonstrated to be quite efficient in partly, although not fully, restoring the energy dissipation capacities of the damaged specimens at all beam rotation levels. The stiffness was partly restored within the elastic range and almost fully restored after the onset of nonlinear behaviour.

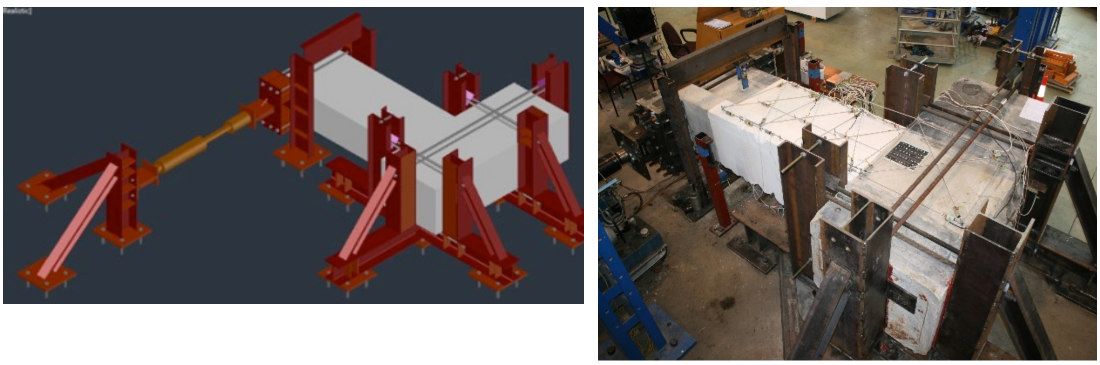


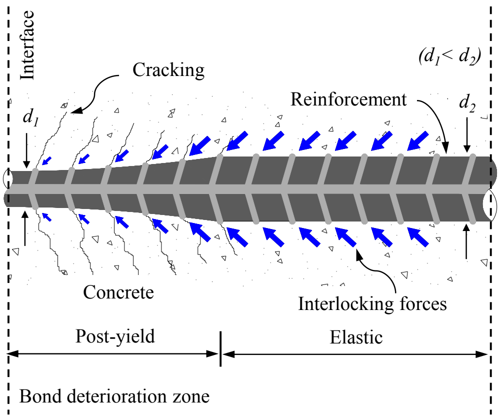
Figure 8 – Experimental campaign on the extracted superassemblages (portions of the PWC building): test set-up, epoxy repairing intervention

Post-yield Bond Deterioration – monitoring using Fiber Optics

Postyield steel–concrete bond has been the subject of considerable investigations using pull-out or direct tensile tests; however, the degradation in bond due to reinforcement yielding in RC beams subjected to lateral loading has not been fully scrutinized. Conventional measuring instruments (strain gauges or LVDTs) cannot precisely measure strain distribution along the yielded reinforcing bar with a minimum level of interference to the bond. Therefore, in this project the postyield behaviour of bond in RC cantilever beams subjected to monotonic lateral loading monitored using a distributed fiber-optic strain sensing system (DFOSSS).

The DFOSSS enabled accurate monitoring of deformations of the embedded reinforcing bars and the strains on the concrete surface. This allowed slip, steel stress, bond stress, bond deterioration length, and the locations of cracks to be determined. Using the new values for maximum bond stress, a model was proposed to predict pre- and postyield bond behaviour, including steel strain effect.

Finally, the mean bond stress values were presented for the simple assessment of bond strength in both pre- and postyield regions.



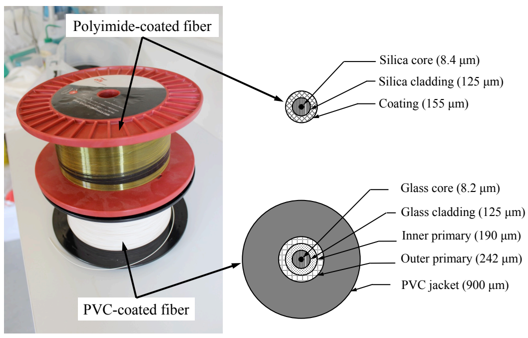


Figure 8 – Bond deterioration concept (left); schematic of fiber optic components (right) (Malek et al., 2019)

CONCLUSIONS AND KEY FINDINGS

Key findings resulting from this research are summarised below:

1. A simplified analytical (rather than numerical or computer-based) procedure to assess existing buildings and rationally consider the effects of Soil-Foundation-Structure-Interaction has been developed
2. A framework for an integrated performance-based assessment, retrofit and design of building-foundation systems has been proposed. Combined/jointed performance criteria and objective have been developed to reflect the observation from the aftermath of the NZ Canterbury Earthquake
3. A comprehensive validation of the NZSEE Seismic Assessment Guidelines (2016/2017) has been carried out, comparing the simplified SLAMA procedure and numerical modelling on numerous case study frame buildings. The procedure has also been extended to infills frames and dual (wall-frame system)
4. A procedure/framework for a performance-based retrofit of an existing building and a cost-benefit analysis of alternative retrofit solutions has been developed. Correlation between safety index (%NBS, or % New Building Standard) and the Risk of collapse as well as the economic losses has been developed. Apparently similar retrofit solutions in terms of targeted safety can lead to very different outcomes when considering the life-cycle economic losses in the aftermath of an earthquake.
5. Superassemblages extracted from real building prior to demolition (PWC Building) have been tested to assess their residual capacity and evaluate the feasibility and efficiency of repairing techniques based on epoxy injections. The tests and associated numerical models confirmed the complexity of the topic, which require further investigation, highlighting that epoxy injection can only partly (not fully) recover the initial stiffness. This should be accounted for in the evaluation of future performance of existing buildings.
6. Overall the project has developed and already disseminated key new knowledge and understanding in terms of assessment and retrofit in the form of procedure and tools that can be used on a daily basis by practicing engineers.

IMPLEMENTATION TO PRACTICE

NZ practicing engineers have been adopting (as per the Building Act) the NZSEE2017 guidelines for their seismic assessment evaluation of existing buildings. Also many of the retrofit solutions developed/refined by the authors have been implemented in real practice.

The new findings of the projects have been promptly disseminated via publications, seminars and courses in NZ and overseas. For example, the full design example on the implementation of the SLaMA procedure to a NZ case Study Building published through a UC report as well as the companion papers developing the procedure further for infill frames and dual system have been passed on directly to the practicing engineers attending the courses as well as made available on Research Gate or similar portals.

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Pampanin, S., One day Seminars/Courses on Seismic Assessment and Retrofit to practicing engineers in numerous cities: Milan, Rome, Verona, Naples, Parma, Bologna, Bergamo, Cassino, Catania, Cosenza, Bari etc.

OUTPUTS AND DISSEMINATION

References above are from this project.

LIST OF KEY END-USERS:

- Ministry of Business, Innovation and Employment (MBIE)
- New Zealand Society for Earthquake Engineering
- NZ Society for Earthquake Engineering (NZSEE)
- NZ Structural Engineering Society (SESOC)
- *fib*, International Federation of Concrete
- Italian Civil Protection
- Practicing Engineers
- Researchers and Postgraduate Students

Topic 4: Protection devices and systems

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ABSTRACT

Energy dissipation devices are an essential part of reducing seismic response and protecting structures. Such devices are equally applicable to both the retrofit of existing buildings and the design of new buildings. This project has developed several methods of providing low-damage energy dissipation devices that can be used as part of the retrofit of existing structures. Devices based upon metallic extrusion, yielding steel, friction, and viscous fluid motion have all been tested and characterised as part of the research within this task. Furthermore, hybrid damping devices, which incorporate two (or more) reaction mechanisms within the same physical damping device have also been developed and tested.

The research programme has included individual component testing of the damping devices, as well as their incorporation into three large-scale dynamic tests. At the date of writing this final report, only one of those three large-scale tests has been completed, within final planning for the other two tests being completed. It is worth noting that these tests are additional to the original research plan, due to opportunities that have arisen during the research programme.

Key outcomes from this research include the development of modelling strategies to capture device mechanics at a component level, as well as structure-level modelling and simplified design equations to understand the influence of the devices on system-level structural response. A particular focus has been on the implementation detailing and end conditions when implementing energy dissipation devices into the retrofit of existing buildings. These end condition/mounting details are essential for the successful performance of the devices within real structures in the field, so further guidance is being developed for practitioners on this specific issue.

INTRODUCTION

While many different energy dissipation devices exist, their incorporation into structural design and retrofit is often quite difficult. The energy dissipation response mechanics often differ from inelastic performance of structural members, so the devices do not easily integrate into simplified structural design methods. This research has investigated the incorporation of energy dissipation device mechanics into simplified design methods, where detailed response simulations are characterised in an aggregate sense across a range of ground motions to enable a simple equation to predict the likely structural response reductions achieved through augmenting a structure with supplemental energy dissipation mechanisms.

In addition to these high-level design methods to enable sizing and specification of a particular damping device method, the specific details of the implementation and device connections are of particular importance. If these end conditions are not appropriately detailed, there is a risk of adverse performance such as local buckling or the energy dissipation device

unintentionally becoming a lateral shear transfer mechanism. These details are especially important as most energy device testing is undertaken in uniaxial test machine, where the full "in-service" loads are not accurately represented.

Extensive component-level testing has been undertaken on a range of energy dissipation device based upon metallic extrusion, yielding steel, friction, and viscous fluid motion have all been tested and characterised as part of the research. Hybrid damping devices, which incorporate more than one reaction mechanisms within the same physical damping device have also been developed and tested. Comparative performance and design recommendations are being finalised based upon the information learnt from these tests.

Figure 5: Incorporation of non-linear viscous fluid dampers within a full-scale shake table test to investigate both end conditions and structural response against simplified design methods.

The research undertaken within this Task was also linked to the QuakeCoRE-ILEE full-scale building test undertaken in collaboration with Objective 4 leader Rick Henry and Yiqiu Lu. This project provided the opportunity to incorporate three different energy dissipation device mechanisms (yielding steel, metallic extrusion, and viscous fluid) into the full-scale system-level testing at the International Joint Research Laboratory for Earthquake Engineering (ILEE) test facility at Tongji University in Shanghai, China. This test enabled assessment of relative performance of the different energy dissipation methods, assess performance against expected structural performance from simplified design methods, and assess the performance of different dissipater mounting/end conditions. An example of the non-linear viscous fluid dampers within the structure is shown in Figure 1. That project was presented at an invited plenary talk at the 2019 Pacific Conference on Earthquake Engineering, and was recently awarded a 2019 Technology Award commendation from the Concrete NZ Learned Society, along with a commendation for the Sandy Cormack Award for Concrete NZ. Due to the relatively recent completion of this major test, full data analysis is still being completed and research outputs will extend beyond the end of the current contract.

CONCLUSIONS AND KEY FINDINGS

Some of the key findings from this research are:

1. Connection of energy dissipation devices into a structure through the use of spherical bearings is the best option as they eliminate unintended shear and moment transfer into the energy dissipation devices, eliminating prying effects and unintended load paths.
2. Even using spherical bearings, care must be taken to ensure sufficient clearances are provided to allow adequate in-plane and out-of-plane rotation of the energy dissipation devices to ensure that the spherical bearing connections can operate as intended.
3. Simplified design methods can be developed that are generally relatively accurate and can approximate the likely structural response reductions without the need for detailed non-linear time history analysis.

Key Achievements/Awards/Recognition:

Task Leader (Rodgers) was awarded the **2017 Royal Society Te Apārangi Early Career Research Excellence (Cooper) Award for Technology, Applied Sciences and Engineering** (October 2017) and the **2017 Kiwi Innovation Network (KiwiNet) Norman F. B. Barry Foundation Emerging Innovator Award** for his work in the development of low-damage design and energy dissipation devices.

PhD students where NHRP co-funding enhanced their PhD research: Jamaledin Borzouie, Jarrod Cook, Nikoo Hazaveh, Farzin Golzar.

Ongoing research partially enabled through NHRP funding: Ongoing collaborative research is being undertaken with the Earthquake Engineering Research Facility (EERF) at the University of British Columbia on hybrid damping devices and joint experimental-computational testing. This project includes a large test of a one-fifth scale high-rise core-wall structure on Tongji University's multi-function shake table array. This project is funded by a Royal Society Catalyst Seeding grant, awarded June 2019.

IMPLEMENTATION TO PRACTICE

Task leader (Rodgers) worked with Lewis Bradford Consulting Engineers and Southbase Construction to develop the damping devices used in the new Tūranga (Christchurch Central Public Library) building. This anchor project within the Christchurch rebuild incorporates the damping devices developed by Rodgers and the low-damage column base connections developed by Jamaledin Borzouie. This project has received the following notable industry body awards:

- 2019 Seismic Resilience Award for Design to Achieve Low Damage from the New Zealand Society of Earthquake Engineering (NZSEE)
- 2019 Supreme Award for structural engineering excellence from Structural Engineering Society New Zealand (SESOC)
- 2019 Silver award from The Association of Consulting Engineers New Zealand (ACENZ)
- 2019 Sandy Cormack Award at the Concrete Society Annual Conference (October 2019)
- Shortlisted (top 6 in the world) for an 'Extreme Conditions' award, 2019 from Institution of Structural Engineers in UK. The only building project from the Southern Hemisphere shortlisted in any category.

Task leader (Rodgers) also worked with Mar Structural Design in Berkley, California on community housing project Casa Adelant, which received a US Resiliency Council Gold Rating for seismic resilience. The project was described by Even Reis (Executive Director of USRC) as

representing a milestone in resilient design. These projects demonstrate national and international uptake of the research outcomes within the contract period, in addition to ongoing dissemination which will likely lead to future industry adoption of the techniques developed.

ACKNOWLEDGEMENTS

The researchers would like to acknowledge the NHRP for funding for this project. This funding was used to complement other funding avenues including from Te Hiranga Rū QuakeCoRE, Rutherford Discovery Fellowship, and MBIE Building System Performance Branch. This co-funding has enable the research to be applied to large-scale tests which would not otherwise have been possible.

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***Note**: there may be some overlap with other tasks/objectives as some of this research has been undertaken collaboratively across the different tasks and objectives, rather than operating in isolation.*

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2. Hazaveh, N.K., Rad, A.A., Rodgers, G.W., J.G. Chase, Pampanin, S., Ma, Q.T. (2017) "Shake table test a structure retrofitted using 2-4 Direction Displacement Dependent (D3) viscous dampers" New Zealand Society for Earthquake Engineering (NZSEE) Annual Technical Conference, Wellington, New Zealand, April 27-29 (8 pages) - **Awarded joint best poster paper award at the NZSEE conference**
3. Hazaveh, N, Rodgers, G.W., Pampanin, S., Chase, J.G. (2016) Design and experimental test of a Direction Dependent Dissipation (D3) device with off-diagonal (2-4) damping behaviour. Poster Presentation at the 2016 QuakeCoRE Annual Meeting, August 31-September 2, Wairakei, New Zealand.
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OUTPUTS AND DISSEMINATION

References above are from this project.

Other Dissemination:

1. Good vibrations – bolder building solutions, New Zealand Construction News, August-September 2016, page 13

2. The BBC (radio broadcast and on the BBC website) Innovative Christchurch buildings designed to beat quakes - By Phil Mercer, BBC News, 7 September 2016 (http://www.bbc.com/news/business-37247536)

3. "Building resilience: a challenge for all" By Geoffrey Rodgers – 1 November 2016, BRANZ Build Magazine, Number 156 (Invited Opinion Piece).

4. Task Leader (Rodgers) has given two invited presentations to the New Zealand Institute of Artchitects Auckland Chapter on developments of low-damage design and energy dissipation/damping systems (June 2018 and October 2019).

LIST OF KEY END-USERS:

- Lewis Bradford Consulting Engineers/Southbase
- Mar Structural Design, Berkeley, California.
- Consulting engineers undertaking retrofit of existing buildings to improve seismic performance

Objective 4: New Buildings: System-Level Interactions

Topic 1: Concrete Buildings

Research Team:
Dr. Rick Henry, University of Auckland, Lead PI
Assoc. Prof. Allan Scott, University of Canterbury, AI
Prof. Des Bull, University of Canterbury, AI
Dr. Lucas Hogan, University of Auckland, AI
Dr. Yiqiu Lu, University of Auckland, PhD student, Research Fellow
Arash Pir, University of Auckland, PhD student
Ericson Encina, University of Auckland, PhD student
Qi Wang, University of Auckland, PhD student

ABSTRACT
The Concrete Buildings Task of Objective 4 focused on concrete wall design and provisions in the Concrete Structures Standard (NZS 3101) to account for whole-of-building response. The influence of wall-to-floor interaction has been assessed using detailed numerical models. Modelling results have shown that the inclusion of the floor stiffness and strength can significantly increase the axial and shear demands imposed on walls and columns compared to the demands estimated from simplified models used in design. In addition, modelling of coupled wall systems has quantified the over-strength that occurs due to axial restraint from floor slabs adjacent to coupling beams. Lastly, the influence of load rate on reinforced concrete walls has been investigated with a series of tests on axially loaded prisms. The prism test results were used to verify the dynamic load factors included in the NZS 3101 previsions for wall minimum vertical reinforcement. Lessons from research into wall-to-floor interaction were incorporated in the ILEE-QuakeCoRE low-damage concrete wall building recently tested on the shake-table at Tongji University.

INTRODUCTION
Trends in building construction are constantly evolving as engineers adopt new systems and incorporate lessons from past experience. The Christchurch rebuild is a great example of this evolution, where the previously popular reinforced concrete frame buildings with precast floors are being replaced with buildings that use stiffer braced steel frames or concrete walls, composite floors, and a range of new low-damage technologies. It is critical the research into structural systems keeps up with trends in current building practice and that design standards and guidelines are updated accordingly.

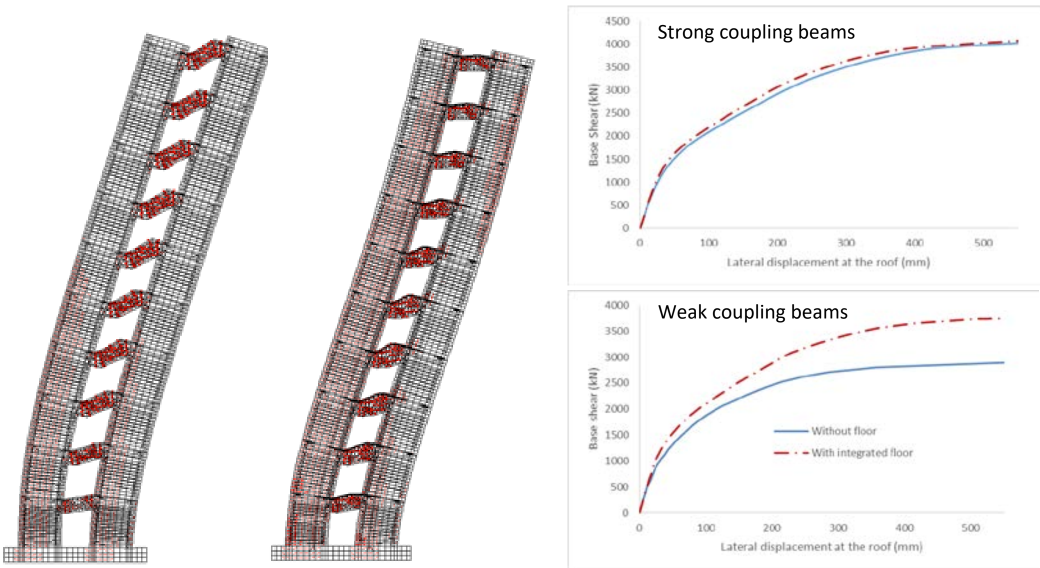
Despite having extensive understanding of component behaviour and design, system-level interactions can significantly alter the demands on individual components and influence the seismic response of buildings. Furthermore, full consideration of system interaction during seismic response may lead to lower drift demands and, in some cases, explain the generally better performance observed in the field compared to conventional predictions. In response to this potential gap in understanding, the Concrete Buildings Task focused on investigating whole-of-building response, including specific consideration of coupled wall systems, wall-to-floor interaction, and earthquake load rate effects.

Research Objective No. 1: Coupled Wall Systems

Coupled walls are a common structural form that consist of individual concrete wall piers joined by coupling beams. As with most structural systems, the development of coupled walls systems (and the diagonal reinforcement detailing) considered the wall system in isolation.

More recent understanding of elongation demands in concrete members has raised concerns over the behaviour of coupled wall systems where the floors and wall piers may provide significant axial restraint to the coupling beams.

A numerical modelling technique was developed using VecTor2 software that could accurately capture the response of coupled wall systems. Parametric analyses were conducted varying a range of key design variables related to the wall piers, coupling beams, and adjacent floor systems and restraint capacity, as shown by the examples in Figure 1. The modelling results confirmed that axial restraint from floors could significantly increase the capacity of the coupling beams (up to 300%), altering the axial and shear demands in the wall piers from the assumed design actions and in extreme cases changing the inelastic mechanism of the coupled wall system. Existing methods to calculate the coupling beam over-strength due to floor restraint were shown to be an upper bound estimate, with the actual over-strength dependent on the wall system design. Criteria and design limits for coupled wall systems were proposed that would ensure the intended inelastic mechanism formed and to identify when additional over-strength from floor and wall pier axial restraint needs to be considered during the design. Lastly, the behaviour of wall piers subjected to tension demands (as in coupled wall systems) was investigated as part of a collaborative research project with Tsinghua University.



a) Coupled wall models *b) Pushover response for coupled wall models*

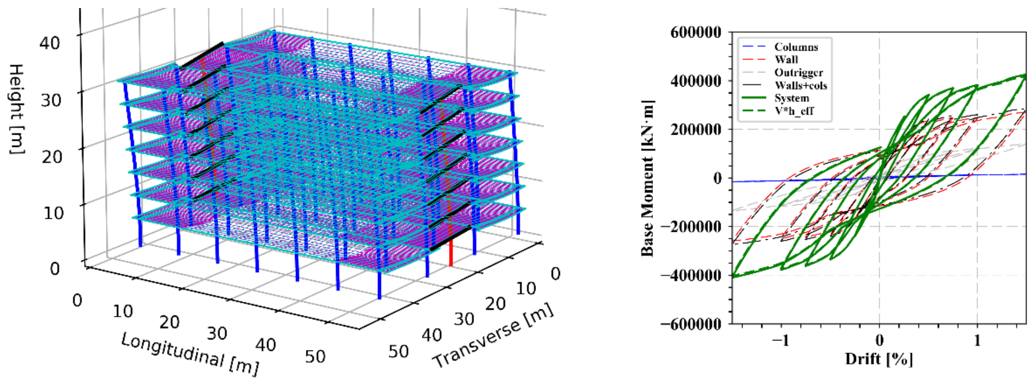
Figure 1. Coupled wall system modelling

Research Objective No. 2: Wall-to-Floor Interaction

During building design, the design actions on components are generally calculated using simplified models that do not account for interactions between components. In the case of concrete wall buildings, which have become increasingly popular in recent construction, the deformations imposed on the floors as the walls deform and the resulting demands that the floor strength and stiffness creates can fundamentally alter the component loads. Accounting for these system interactions can be challenging and requires complex 3D numerical models.

A systematic study of wall-to-floor interaction in a typical 7-storey concrete building was conducted in three phases. 1) Elongation of concrete walls was quantified and fibre element models were validated that could accurately capture this axial elongation. 2) A 3D model of a prototype building was developed using similar modelling techniques to that developed and validated for a 5-storey post-tensioned wall building tested in Japan. 3) Parametric analyses were used to quantify the system response and demands on local components when key

design parameters were varied, including wall dimensions and design, column spacing, floor type and orientation. The results of an example analysis are shown in Figure 2. The detailed 3D models have been used to develop simplified mechanism-based models that could be implemented to estimate the wall and column over-strength demands (esp. axial and shear demands) following capacity design principles.



a) Model subjected to lateral demands *b) Moment-drift response*

Figure 2. Building model used to study wall-to-floor interaction

Research Objective No. 3: Load Rate Effects

Another important aspect of building system response is the effect of loading rate and history. It is well known that dynamic loading rates affect the strength and ductility of materials, but the combination of these altered member properties on the response of concrete components and systems is not as well understood. Prior research conducted within the NHRP investigated the effect of reinforcing content on the seismic performance of concrete walls following concerns raised during the Canterbury earthquakes. Proposed minimum reinforcement requirements had included provision for increased material strengths due to dynamic loading, but these were not thoroughly investigated at the time. A series of concrete prism tests were conducted that represented the end region of concrete walls, as shown in Figure 3. A total of 15 prisms prism were tested with variations in reinforcement content, loading history and loading speed. The results showed that the loading rate did affect the prism strength, but had only a minor influence on the cracking behaviour and ductility for prisms with reinforcement contents that exceeded current requirements in the New Zealand Concrete Structures Standard (NZS 3101). Recommendations were made regarding the concrete wall designs that may be influenced by loading rates, but it was ultimately determined that no immediate changes to NZS 3101 are required.

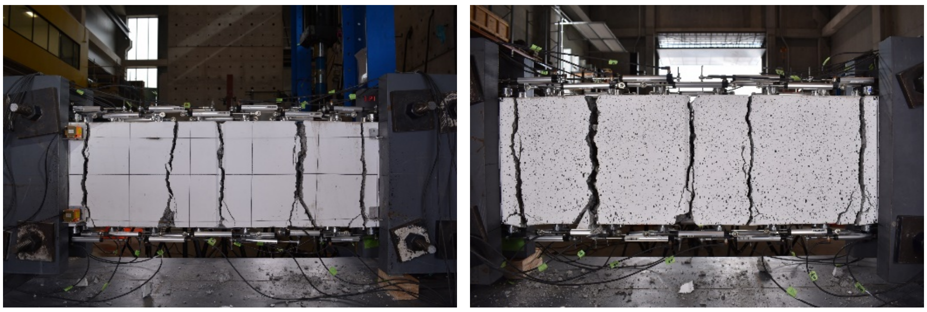


Figure 3. Concrete prism tests

Aligned Research Project: Low-damage Building Test

The research conducted within the Concrete Buildings task was used to inform the design of a two-storey low-damage concrete building that was tested In China as part of a collaborative

ILEE-QuakeCoRE project. In particular, the test building implemented specific detailing to alleviate adverse effects of wall-to-floor interaction and was subjected to dynamic loading by a pair of large shake-tables that replicated earthquake ground motions. The test provided valuable experimental data to validate current design practice and numerical models developed to investigate whole-of-building response. Specific guidelines for low-damage building design are currently under development that draw on the test results and other research, such as that conducted within the NHRP Concrete Buildings Task.

Figure 4. ILEE-QuakeCoRE low-damage building test

IMPLEMENTATION TO PRACTICE

The research conducted within the concrete buildings task will primarily be implemented within the New Zealand Concrete Structures Standard (NZS 3101:2006). Despite recent efforts to initiate a revision to NZS 3101, there is currently no active technical committee. As such, research findings will be summarised and published in local and international journals and presented to the NZS 3101 committee when formed. Key recommendations include:

- Limitations on coupled wall design parameters (e.g. degree of coupling) to ensure that the intended inelastic mechanism forms.
- Proposed revisions to over-strength estimates for coupling beam to accurately reflect the contributions from floor and wall pier restraint and include criteria for when axial restrain is not critical.
- Criteria recommended for buildings configurations where wall-to-floor interaction will significantly alter component demands and for which detailed analysis needs to be conducted during design.

In addition to New Zealand design standards, provisions for minimum longitudinal reinforcement requirements for concrete walls were proposed and implemented in the US concrete design standards (ACI 318-19). These provisions included explicit allowance for earthquake loading rate effects on the material strength and resulting reinforcing contents required to achieve sufficient crack distributed.

ACKNOWLEDGEMENTS

Additional aligned funding was secured for the ILEE-QuakeCoRE building test from the Building Systems performance branch of Ministry of Business, Innovation and Employment (MBIE), QuakeCoRE, and ILEE at Tongji University. In addition, the contributions of international

collaborators at Tongji (Profs. Zhou and Yang), Tsinghua (Prof. Ji), and Nagoya (Prof. Nagae) are gratefully acknowledged.

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OUTPUTS & DISSEMINATION

PUBLICATIONS:

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Watkins, J., Sritharan, S., Nagae, T., Henry, R. S. (2017). 'Computational modelling of a 4-storey post-tensioned concrete building subjected to shake table testing', Bulletin of the New Zealand Society for Earthquake Engineering, 50(4), 595-607.

Encina, E., Lu, Y., Henry, R. S. (2016). 'Axial elongation in ductile reinforced concrete walls', Bulletin of the New Zealand Society for Earthquake Engineering, 49 (4).

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Encina, E., Henry, R. S. (2017). 'Wall-to-floor interaction in RC buildings: Modelling case study', 16th World Conference on Earthquake (16WCEE), Santiago, Chile, 9-13 January.

Pir, A., Henry, R. S., Ingham, J. M. (2017) 'Nonlinear parametric analysis of reinforced concrete coupled walls', Proceedings of the 2017 NZSEE Annual Conference, Wellington, April 27-29.

KEYNOTE/INVITED LECTURES:

Henry, R. S., Rodgers. G. W. (2019). 'ILEE-QuakeCoRE collaboration: Low-damage concrete wall building test', 2019 Pacific Conference in Earthquake Engineering (PCEE), Auckland, 6 April.

Henry, R. S. (2018). 'Implementation of low-damage design – how far have we come?', QuakeCoRE Annual meeting, Wairakei, 6 Sept.

AWARDS:

New Zealand Concrete Society (NZCS) Sandy Cormack Commendation for runner-up best conference paper (2018).

Ivan Skinner Award for the advancement of earthquake engineering, Earthquake Commission (EQC) and New Zealand Society for Earthquake Engineering (2018).

LIST OF KEY END-USERS

- Concrete NZ
- Ministry of Business, Innovation and Employment (MBIE)
- Standards New Zealand (NZS 3101 committee)

Topic 2: Steel Buildings

Topic 2a: Design of steel-concrete composite structural systems for diaphragm action and interface performance

Research Team:
AP Charles Clifton, PI, University of Auckland
AP James Lim, University of Auckland
Mr. Hooman Rezaeian, University of Auckland, Ph.D. Candidate, commenced 2016 due for completion 2019
AP Greg MacRae, University of Canterbury

ABSTRACT

This composite floor slab topic (Topic 2a) of Objective 4 involves determination of the demand on composite floor slab diaphragms and determination of the capacity of the diaphragm interfaces between the floor slab and the seismic resisting systems.

Floor diaphragms play a key role in the dependable performance of a multi-storey building under earthquake actions. They act as the rigid link holding the gravity resisting and the seismic resisting system components of the building together at each floor level and allowing the seismic resisting systems to undergo any expected inelastic demand. Earthquakes induce in-plane actions in the diaphragms and they must be able to resist these actions in an elastic manner and transfer them to/from the seismic resisting systems into which it is connected also while remaining effectively elastic.

Composite floors comprising lightly reinforced concrete slabs cast onto steel decking and supported on a network of steel beams are commonly used in multi-storey buildings in New Zealand and other high seismic zones. In the damaging 2010/2011 Canterbury earthquake series, they delivered excellent performance, despite not being designed and detailed to the current recommendations for determining the demand on diaphragms and the capacity of the diaphragm interfaces. The purpose of this project has been to investigate the adequacy of the current procedures for determining the demand on diaphragms through numerical modelling of a representative building system and to determine the capacity of the diaphragm interface in the elastic and the inelastic range through experimental testing.

The numerical modelling has shown that the procedure for determining the demand is reasonable, predicting demands at between 75% and 80% of the spread of actions determined from numerical modelling.

The experimental testing has shown that the current method for determining the diaphragm interface capacity at the end of elastic behaviour under-predicts this by a factor of nearly 2 and proposes a new procedure which is less conservative. The combination of these two findings is consistent with the observations from Christchurch and Wellington, where diaphragms performed better than expected.

INTRODUCTION

Background

Neither the current edition of NZS 3404 [1], nor the current seismic design procedures for steel structures [2], present procedures for determining the demand on composite floor diaphragms. The most comprehensive design procedure is contained in section 8.4 of [3], which determines the design actions arising from inertial forces, displacement compatibility

between adjacent seismic-resisting systems and transfer diaphragm actions. The provisions for determining the capacity of the diaphragm are given in [4]. However, when combining the demand from [3] with the capacity from [4] in seismic resisting systems which are in isolated bays, such as Eccentrically Braced Frames (EBF), often the beams in adjacent bays of the gravity system must act as diaphragm drag beams to develop sufficient diaphragm capacity to meet the demand.
None of the diaphragms in buildings in Christchurch subjected to the damaging earthquakes of 2010/2011 had these diaphragm drag beams, however all diaphragm interfaces delivered excellent performance [5] in an earthquake series that was at Maximum Considered Event level.
The first part of this project determined the demand on the diaphragm interfaces taking into account the inertial forces and the additional forces developed by deformation incompatibility between adjacent seismic resisting systems. Details are presented in [6]
The second part is being presented in Rezaeian's PhD thesis, which is due for submission before end 2019 and in four journal papers, one published, two accepted for publication and one currently under review, details of which are given in the listed outputs below.

Determination of diaphragm demand through numerical modelling
Figure 1 shows the details of the building model used in the numerical integration time history analyses used to determine the spread of diaphragm demand between the seismic resisting systems and the floor diaphragms.

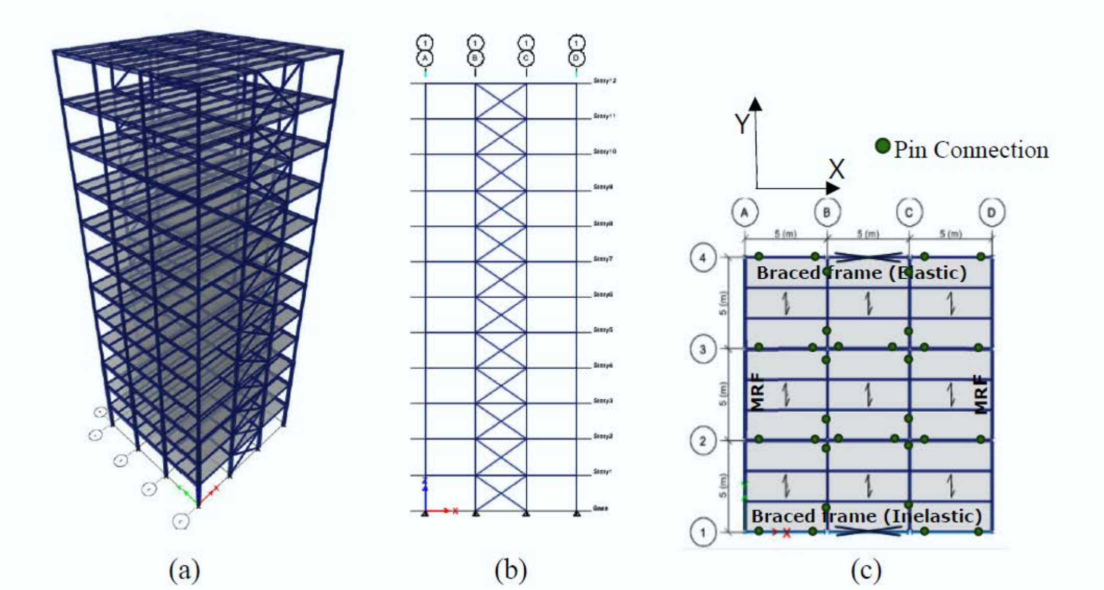
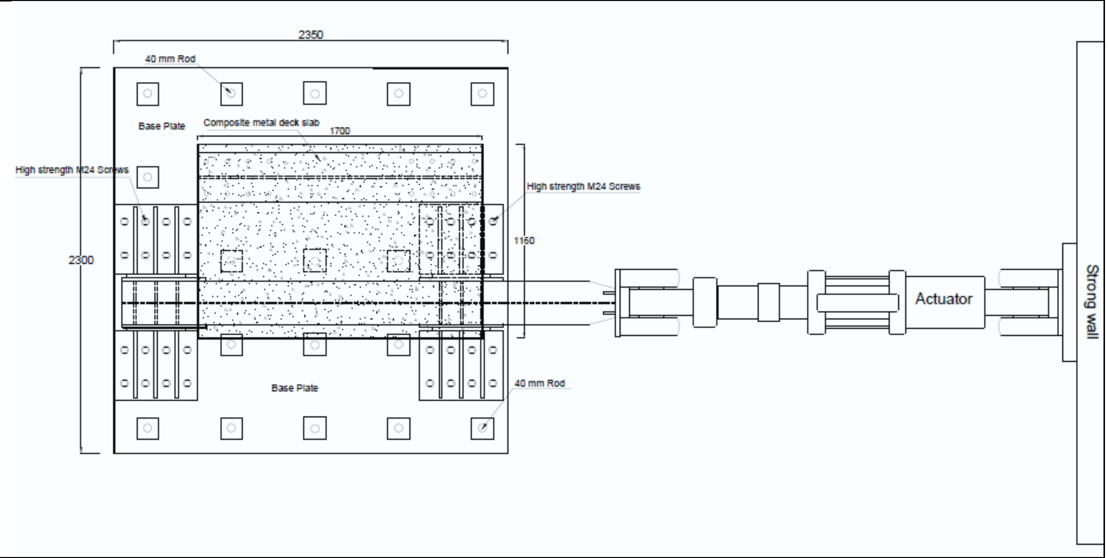


Figure 1 Building model: a) 3D View, b) Elevation view and c) Plan view

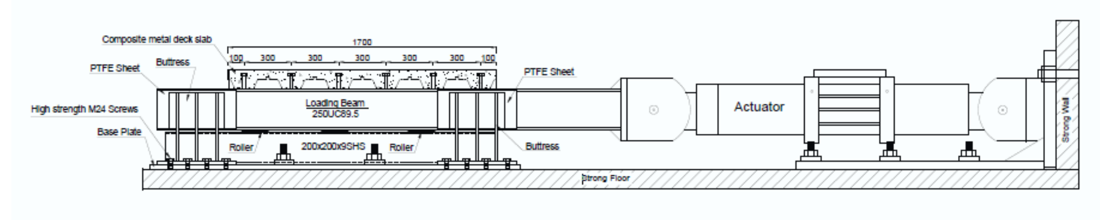
Three strong ground motions were used in the studies. Inertial forces were determined through modelling the seismic resisting systems as built. Displacement compatibility forces were determined by allowing the EBF system on one side to yield while keeping the other system elastic.

The results show that the inertial forces given by the method in section 8.4 of [3] generate answers that are between 75% and 80% of the maximum inertial forces generated by the nonlinear inelastic time history analysis results [4] and that the compatibility forces increase these by a factor of up to 1.2 [6] for two adjacent seismic resisting systems of the same type, which is consistent with the design procedure presented in [3].

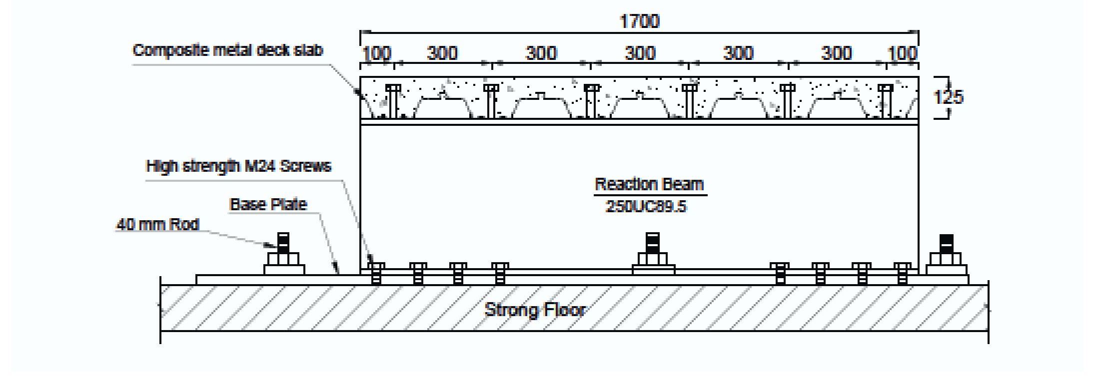
Determination of diaphragm interface behaviour through experimental testing.
Figure 2 and Figure 3, taken from [7], show details of the experimental test setup and specimens.



a) Top view



b) Long side view



c) Short side view

Figure 2 Test Rig

a) Sp1 b) Sp 2 (New Zealand detail) c) Sp 3 (European detail)

Figure 3 Photograph of the three specimens

Three nominally identical specimens of each type were made. Because this was a world first test setup, we had no precedent results to give indicative performance. Because of this, the first test of each specimen type was undertaken monotonically to establish the force – deflection curve and key force levels in the elastic range. The subsequent two specimens were tested cyclically, with three cycles of loading up to the 500 kN level of load under force control, then with subsequent cycles under displacement control.

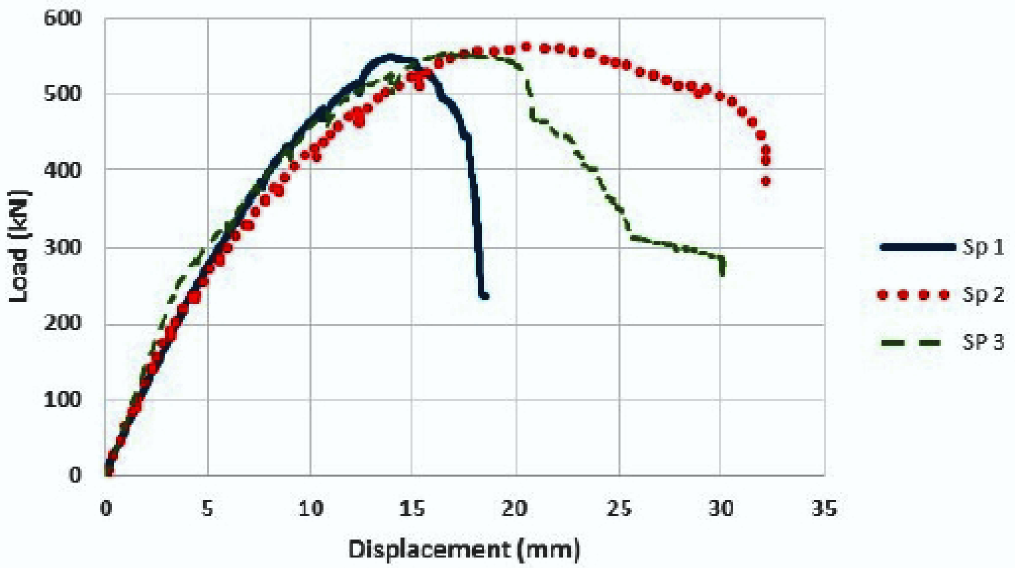


Figure 4 Load - displacement curves from the three specimen types monotonic tests

Figure 4 shows the load displacement curves from the monotonic tests on each specimen type. The performance in the elastic range was very similar for the three specimen types and the peak load reached was similar. The ductility in the post elastic range was different, with the primary beam interface configuration showing the lowest ductility and the secondary beam interface with the recommended New Zealand edge detail showing the greatest ductility. However, in all cases the design procedure that has been developed will keep the diaphragms dependably within the elastic range of the ascending force-deflection curve, so the behaviour following the peak load is of research interest only.

CONCLUSIONS AND KEY FINDINGS

The key findings from this research are that the current method for determining diaphragm demand predicts results above the 75% of the range developed in practice, while the currently predicted capacity is not more than 50% of the elastic threshold of diaphragm behaviour.

IMPLEMENTATION TO PRACTICE

It is too early for these provisions to be used in practice, however, given the key results shown above, there is not an urgent need to change the current practice. Once the journal papers under review are published and the PhD thesis is completed and under examination, a paper presenting the final design and detailing requirements will be published in the SESOC Journal for New Zealand practitioners and also made available through SCNZ.

ACKNOWLEDGEMENTS

In addition to the support from MBIE through the NHRP, for which the researcher is very grateful, the authors would also like to thank the EQC, for funding of the experimental testing, conference attendance in advance to the WCEE, 2020 and a final year PhD stipend and fees. They would also like to acknowledge the excellent support from the University of Auckland Structures Test Hall staff especially Lucas Hogan and Jay Naidoo.

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OUTPUTS & DISSEMINATION

PUBLICATIONS:

Journal papers:

- Rezaeian, H., Clifton, G.C., Lim, J.B.P. (2019) Failure modes for composite steel deck diaphragms subject to in-plane shear forces – a review. Engineering Failure Analysis. (Accepted for publication)

- Rezaeian, H., Clifton, G.C., MacRae, G.A., Lim, J.B.P. (2019) In-plane cyclic behaviour of composite floor slab diaphragm interfaces under high shear demand. Journal of Constructional Steel Research. (Accepted for publication)
- Rezaeian, H., Clifton, G.C., Lim, J.B.P. (2019) Experimental study on in-plane shear behaviour of composite floor slab diaphragm interfaces under high shear demand. Journal of Constructional Steel Research.(Under review)
- Rezaeian, H., Clifton, G. C., & Lim, J. B. P. (2018). Compatibility Forces in Floor Diaphragms of Steel Braced Multi-Story Buildings. *Key Engineering Materials*, *763*, 310-319. doi:10.4028/www.scientific.net/KEM.763.310

LIST OF KEY END-USERS:

- Ministry of Business, Innovation and Employment (MBIE)
- New Zealand HERA
- Steel Construction New Zealand

Topic 2b: Buckling restrained brace frame (BRBF) performance of systems using concrete filled BRBs

Research Team:
Senior Lecturer, Chin-Long Lee, University of Canterbury
Associate Professor, Gregory MacRae, University of Canterbury
PhD Candidate, Jian Cui, University of Canterbury

ABSTRACTS

Buckling restrained braced frames (BRBFs) have become widely used in New Zealand (NZ) since the 2011 Christchurch earthquakes, but a proper design code for them is not yet available in NZ. Local engineers have to adopt relevant design standards from other countries. However, existing design codes and recommendations adopt different design approaches in computing the capacity of BRBs, and do not necessarily consider all the potential factors affecting the stability performance of BRBs. In this research, important factors and failure modes, such as global buckling, local bulging and restrainer end failure, are considered. Methods include experimental testing of a scaled BRBF subjected to bi-directional loading, and numerical modelling with a parametric study. Relevant design equations for these factors are derived, particularly with the out-of-plane loading effect considered. A practical design procedure including two simple methods for BRBF stability design considering the effect of out-of-plane loading is also proposed.

INTRODUCTION

Since the 2011 M6.3 earthquake in Christchurch, owners and structural engineers realized the importance of earthquake resistance technologies in structural design. Among a variety of new energy dissipation and base isolation technologies, buckling restrained braces (BRBs) and BRB frames (BRBFs) have become widely used in the Christchurch rebuild (MacRae & Clifton, 2015).

In New Zealand, there is no design standard for BRBs and BRBFs. Engineers would adopt foreign design standards. Among these standards, AISC seismic provisions (AISC341, 2016) are often used with a New Zealand code-compliance framework (Kam, Built, & Gardiner, 2017). In particular, these seismic provisions would require experimental testing of BRBs before they are deployed in buildings. However, in practice, most BRBs installed in NZ were deployed without lab tests and, therefore, their performance during earthquake events is not fully understood.

With the aim to help NZ engineers to properly design BRBs ensuring their good performance during earthquake events, our research team sets out to investigate the factors and failure modes affecting the BRB performance. These factors and failure modes include out-of-plane loading, global buckling, local bulging, and restrainer end failure.

In this NHRP research study, the team conducted a critical review of existing design standards, and carried out experimental testing and numerical studies. The experiment includes testing of a scaled BRBF (one-bay one-storey) subjected to in-plane and bi-directional (combined in-plane and out-of-plane) loadings. The numerical study included detailed 3D modelling of a BRB and a BRBF, validated with experimental data, and a parametric study quantifying the effects of key factors affecting the performance of BRBs. In addition, the research also investigates the factors that affect the BRB system stability in a frame, and proposes design methods.

At the end of this study, the research team will present recommendations in journal and conference papers for design of BRBs and BRBFs, considering the abovementioned factors and failure modes, and present these recommendations to the standards committee.

This report provides a very brief summary of the NHRP research study. It highlights at a very high level some of the research aspects and outputs, and excludes technical details.

Research Objective No. 1

BRB and BRBF design

The research team focuses on reviewing common failure modes and typical design procedures for BRBs and BRBFs.

Failure modes of BRBs

The research team has identified the following common failure modes found in BRBs, despite being designed according to existing standards:

1. Global buckling
2. Local buckling including local bulging
3. Restrainer end failure

In addition, for a steel-mortar BRB, the presence of friction during compressive loading cycles may cause an unbalance between compressive and tensile strengths. It is noted that a proper gap and good debonding mechanism can reduce and even eliminate this influence, and with them a balanced and stable hysteresis loop can be obtained, as also demonstrated in the experiments conducted in this study.

The failure modes discussed above are yet to be communicated with the engineering communities, but will soon be discussed in detail, together with design recommendations, in Jian's PhD thesis, conference presentations and journal publications by the research group in a couple of years.

BRB design

In the design of BRB, two common factors: strength and stiffness, would dictate the modes of failure. If the strength of a BRB is too high while the stiffness is too low, global buckling would occur. Conversely, local bulging would occur (Nagao & Takahashi, 1990). It is, therefore, important for engineers to find a balance between the strength and stiffness requirement in order to avoid both global buckling and local bulging.

In New Zealand, most gusset plates connecting BRBs to beam-column joints do not have full-depth stiffeners, resulting in having a weak connection. This could lead to a restrainer end failure that has a plastic hinge forming on the steel core inside the restrainer within the end zone. To prevent this failure, the team is trying to find a proper inserted length of the steel core inside the restrainer.

BRB-GP system stability

One of the major concerns that affect the performance of BRBs in a system is the system stability. To prevent a stability issue with the gusset plates under large in-plane compressive forces, a stiff/strong gusset plate and end connection is required. However, during out-of-plane frame deformations/drifts a stiff connection attracts load and may yield. Plastic deformation at the end region may compromise the ability of the BRB to perform well under any subsequent in-plane displacements. Therefore, both low end stiffness and high end stiffness may be problematic in terms of performance. This issue, which has not been solved

by BRB manufacturing companies or by previous studies. Also, complicated, arbitrary, complex, force-based, or lack of comprehensibility of proposals made for BRB system design have contributed to the delay of the development of national design recommendations for BRB systems, and also to a reduction in their use. One smart method to mitigate the out-of-plane deformation effects on a BRB itself has been to place a perfect pin at the end of the BRB, but no explicit design considerations are also available regarding the performance of a system containing such pins.

Typical BRBF design procedure

A typical BRBF design process incorporates coordination of a BRB supplier and flow of information between structural engineers and brace suppliers (Robinson, 2014). Three key critical design items all shown in the Figure 1.

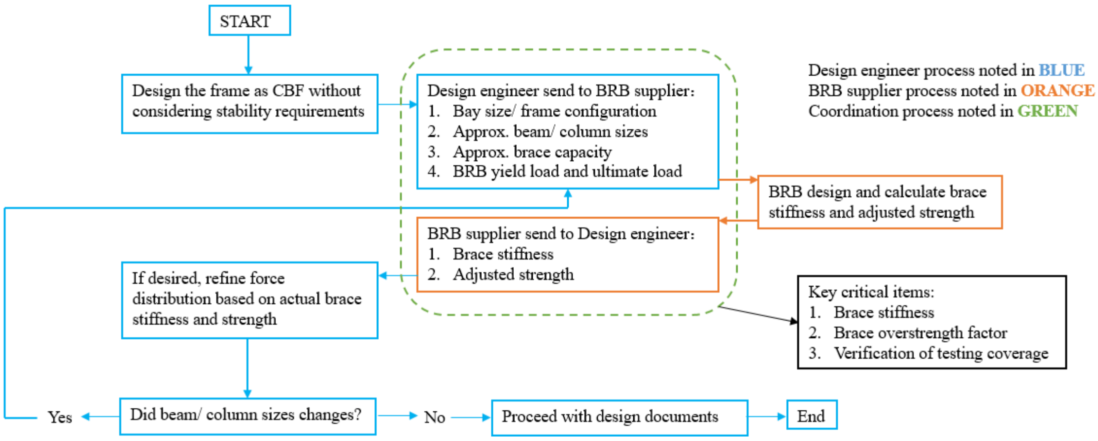


Fig. 5 – A typical BRBF design process (adopted from Robinson, 2014)

Research Objective No. 2

Lab test

A buckling restrained braced frame (BRBF) with several BRB specimens was tested in the Wing Lab of University of Canterbury. The BRBF was designed according to the procedure shown in Figure 1. Two batches of BRBs were tested, batch 1 includes five BRBs of poor quality, while batch 2 includes six BRBs that were subjected to very strict quality control.

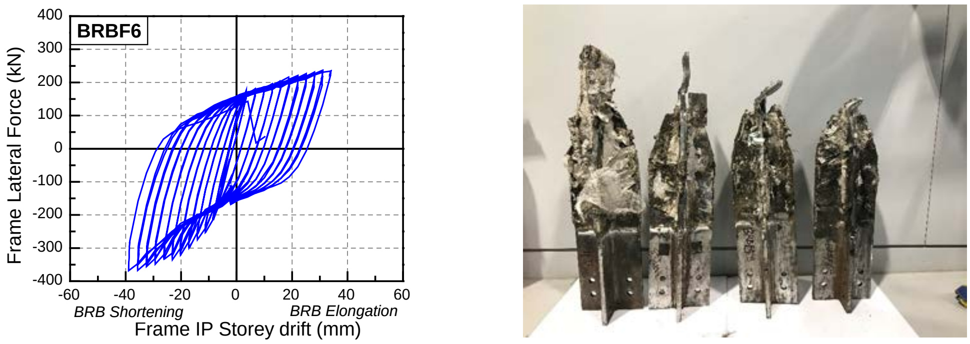
This BRBF was subjected to both in-plane loading and bi-directional loading (combined in-plane and out-of-plane). The loading was applied with displacements being controlled. For the bi-directional loading, the controlled displacements follow a clover-leaf pattern. In terms of amplitude, two cycles were applied at displacement ductility of 2 to 18 at a step of 2, and followed by ten repeated cycles at displacement ductility of 20.

Batch 1 results

In general, most BRBs exhibited good hysteresis loops and their cumulative plastic deformation (CPD) values greatly exceed the required value. However, they exhibited an unbalance ratio β of 1.7 between compression and tension strength. This exceeds the acceptance criterion of 1.5 stated in AISC341.

Moreover, localized buckling was found in the transition zone where the debonding mechanism was not fully achieved. Since the clearance in the transition zone was insufficient,

this localized buckling has led to the increase in friction between the steel core and the concrete.

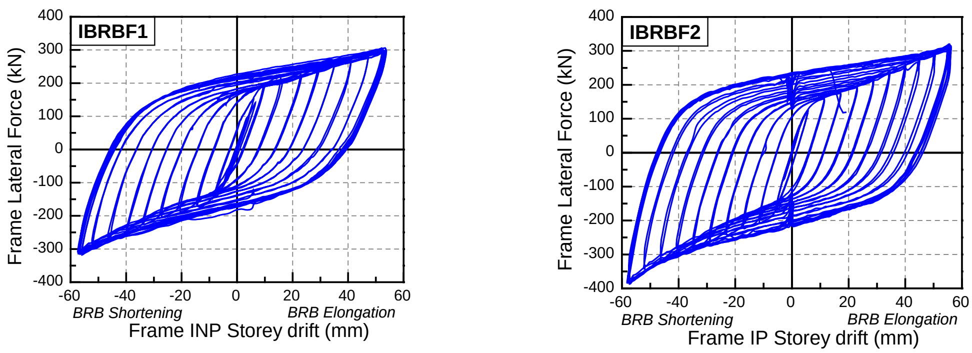


(a) Hysteresis loop *(b) Localized buckling*

Fig. 6 – Batch 1 results example

Batch 2 results

Compared to the results from Batch 1, these BRBs exhibited a balanced and stable hysteresis loop and higher CPD (cumulative plastic deformation) values. This indicates that the quality control during manufacturing process is important in ensuring a good performance.



(a) Under in-plane loading *(b) Under bi-directional loading*

Fig. 7 – Batch 2 results example

Research Objective No. 3

Numerical analysis

Two numerical models were created: connector model and 3D solid model.

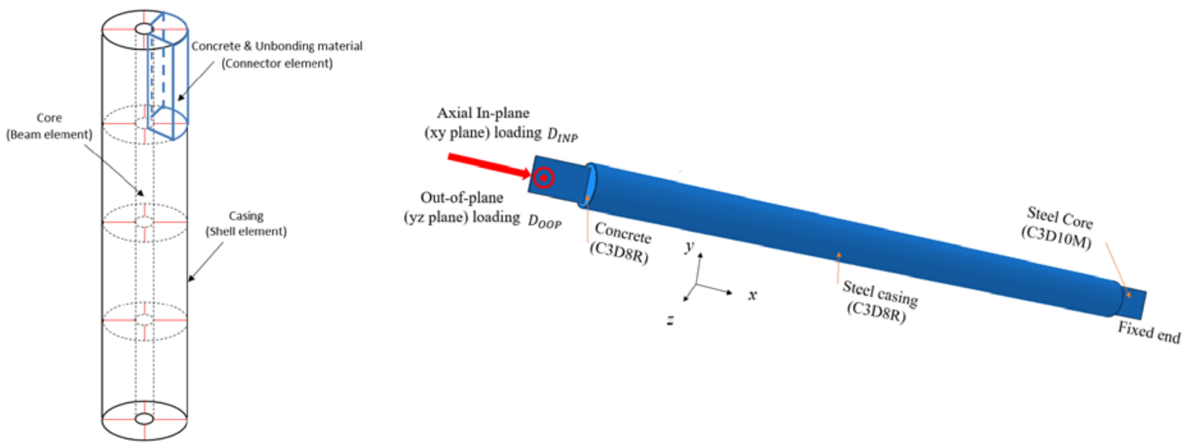
Connector model

In this model, connector elements were used to simulate the behaviour of concrete and unbonding material to transfer the radial stress from the core to the casing. This model has good computational efficiency because contact surfaces between the core and the concrete were not explicitly modelled. It also enables non-linear response characteristics of the unbonding material to be incorporated (Cui, Lee, & MacRae, 2018).

Based on this method, a finite element model was able to capture BRB behaviour under a combined in-plane and out-of-plane (OOP) loading. The numerical results showed that OOP loading would decrease the capacity of BRBs.

3D solid model

It is found from the experiments that the compressive overstrength of BRB could be affected by the friction between the steel core and the concrete. Connector elements (compression only) in the connector model would not be able to capture the friction behaviour correctly. Therefore, a 3D solid model was developed where surface contact was used to explore two conditions: manufacturing imperfection (for Batch 1 BRBs) and out-of-plane loading (for Batch 2 BRBs), and how the β ratio would be affected by these two conditions. At this stage, the numerical models are still being calibrated against experimental data. Once it is done, a parametric study will follow to quantify the effects of out-of-plane loading and manufacturing imperfections on the performance of BRBs and BRBFs.



(a) Connector model *(b) 3D solid model*

Fig. 8 – Numerical models

Research Objective No. 4

BRB system design for stability

This research describes the likely mechanisms of failure of a BRB as part of a global system considering frame stability, and the requirements for reliable force resistance. A new, and economical, concept is proposed which solves the end stiffness issue described previously. A method to estimate the deformation capacity for such a system is developed as a function of out-of-plane drift. Furthermore, to provide the desired performance, detailing suggestions and recommendations are made to restrict the deformation of the boundary elements at the brace end. The approach encourages the system, rather than just the brace, to perform well.

The method proposed prevents three moment-releases (hinges) from forming as this causes a mechanism. Also, for the example shown using the new concept developed it is possible to achieve out-of-plane interstorey drifts of 3% while limiting strains to less than twice the yield strain while the zone axial force ratio is 0.50. The boundary element restraint is relatively easy to provide when the gusset plate has edge stiffeners and a reinforced concrete floor slab wraps around the column. Finally, it is emphasised that if designers cannot guarantee the performance of a BRB system under bidirectional loading, they should not be recommending this particular system.

CONCLUSIONS AND KEY FINDINGS

1) At present, there is no proper code available in NZ for designing BRBFs. Lab testing is always required to ensure the BRBs being deployed could meet the seismic performance requirements. However, because of lack of adequate lab facilities and

high testing cost, few engineers in practice have done the testing before deploying the BRBs. Using experimental testing, this study showed that, a good quality control during manufacturing of BRBs could ensure their good performance (e.g. a balanced and stable hysteresis loop and high cumulative plastic deformation value), thereby eliminating the need for conducting a test.

2) Previous research and design recommendations do not always consider the out-of-plane effect on the strength and capacity of BRBs. Some previous tests showed that this effect would lead to the failure at the restrainer end of the BRB. To prevent this type of failure, the research team investigated the relationship between the moment capacity of the restrainer end and the inserted length of the transition zone. Based on the results from a parametric study, a proper inserted length and a wider gap at the end of the gusset plate in the compressive material end are proposed.
3) The experiment showed that the BRB overstrength values could increase by 30% due to the out-of-plane loading. This was also supported by a numerical analysis on a 3D solid model with a proper friction value between the steel core and the concrete. Note that a simpler connector model would not be able to capture this overstrength effect. This study proposed to modify the overstrength coefficient β to account for this observation. Different unbonding material and magnitude of out-of-plane movement are also incorporated in the proposed modification.
4) No local bulging was found in the experiment, because it is believed that, a BRB with a circular cross-section, compared to a rectangular cross-section, would have higher strength to resist this failure mode.
5) Two simple methods were proposed for BRBF design considering the out-of-plane effect: (a) A simple method to allow frame out-of-plane deformations without compromising the brace capacity; (b) A simple method to provide boundary element restraint. Design examples were also provided.

IMPLEMENTATION TO PRACTICE

The outcomes of this research will be disseminated to practicing engineers in the form of journal and conference papers. Two conference papers have been presented in the STESSA18 and PCEE19. Journal papers are currently drafted, and will be submitted in the next 6 months. The design recommendations discussed in these papers would also be presented to the Standards committee for NZS3404, of which Greg MacRae is a member.

ACKNOWLEDGEMENTS

The authors appreciate the MBIE Natural Hazards Platform (Research Objective 4, New Buildings) for the financial support provided for this study. The authors also acknowledge the funding from HERA and CNRE department of the University of Canterbury for the experimental study.

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Cui, J., Lee, C. L., & MacRae, G. A. (2018). *Finite Element Modelling of Buckling Restrained Braces under Combined In-Plane and Out-of-Plane Loading.* Paper presented at the Key Engineering Materials.

Kam, W., Built, R., & Gardiner, R. (2017). Seismic Design of the New Zealand International Convention Centre (NZICC).

MacRae, G. A., & Clifton, G. C. (2015). *Research On Seismic Performance Of Steel Structures.* Paper presented at the Steel Innovations 2015 Conference, Steel Construction New Zealand, Aotea Centre, Auckland, Christchurch.

Nagao, N., & Takahashi, S. (1990). A study on the elasto-plastic behavior of unbonded composite bracing (Part 1: experiments on isolated members under cyclic loading). *Journal of Structural Engineering, 415*, 105-115.

Robinson, K. (2014). *Advances in design requirements for buckling restrained braced frames.* Paper presented at the New Zealand Society of Earthquake Engineering Conference.

OUTPUTS & DISSEMINATION

PUBLICATIONS:

Conference papers:

Cui, J., C. L. Lee, and G. MacRae. "Strong-axis and weak-axis buckling of Buckling Restrained Braces under bidirectional loading." *11th Pacific Conference on Earthquake Engineering (PCEE)*. 2019.

Cui, J., C. L. Lee, and G. MacRae. "Finite Element Modelling of Buckling Restrained Braces under Combined In-Plane and Out-of-Plane Loading." *Key Engineering Materials*. Vol. 763. Trans Tech Publications, 2018.

KEYNOTE/INVITED LECTURES:

- MacRae, G. “New Zealand Research Applications of, and Developments in Low Damage Technology for Steel Structures”, the 9th International Conference on Behaviour of Steel Structures in Seismic Areas (STESSA 2018), Christchurch, New Zealand, February 14-17, 2018
- The research team is leading the BRB sessions at the 9th International Conference on Behaviour of Steel Structures in Seismic Areas (STESSA 2018), Christchurch, New Zealand, February 14-17, 2018
- MacRae, G. “New Zealand Seismic Design of Buildings and Some Research Studies”, Invited Lecture (together with Liangjiu Jia, professor of Tongji University), Taiyuan University of Technology, May 31, 2019
- MacRae, G. “Recovery after the Canterbury Earthquakes”, the 12th Canadian Conference on Earthquake Engineering (CCEE2019), Quebec, Canada, June 17-20, 2019
- MacRae, G. keynotes and invited lectures, the 8th International Conference on Seismology & Earthquake Engineering (SEE 8), Tehran, Iran, 11-13 Nov, 2019 (to be delivered)
- The research team is leading the BRB sessions at the 17th World Conference on Earthquake Engineering (17WCEE), together with Prof. Takeuchi from Tokyo Institute of Technology, Sendai, Japan, September 13-18, 2020

LIST OF KEY END-USERS:

- NZS3404
- Geoff Bird, BECA Auckland (together with David Whittaker - Christchurch and Sean Gledhill – Tauranga)
- Tim Shannon, Lewis Bradford
- Stephen Hogg, Aurecon
- Kevin Cowie, SCNZ

Topic 2c: Stability of Buckling Restrained Braced Frame (BRBF) Connections Using a Simplified Notional Yield Line Method

Research Team:
Associate Professor G Charles Clifton, University of Auckland
PhD Candidate: Behnam Zaboli, University of Auckland
Steel Construction New Zealand, Senior Structural Engineer (Technical Development): Kevin Cowie

BACKGROUND AND OVERVIEW

This research project has been funded by the NHRP in a different funding stream (NHRP Contest 2017) and is reported on separately in the following report:

Zaboli, B and Clifton, G.C., (2019) *Stability of Buckling-Restrained Braced Frame (BRBF) Connections Using a Simplified Notional Load Yield Line Method*, Final Report to the NHRP Contest 2017, University of Auckland.

The driver for this project was concerns expressed by the consulting profession in early 2016 that the design methods for gusset plate connections of BRBs into the braced frame seismic resisting system using these novel braces may not be adequate to ensure stability of the BRBF gusset plate system (referred to as the BRBF system) against out of plane buckling. This concern was fuelled by failures of BRBF systems in experimental testing in Taiwan and Japan. The consulting engineering profession approached Steel Construction New Zealand for advice on this and Charles Clifton became involved in providing interim design advice. He based this design advice on achieving BRBF gusset plate stability through the application of a notional load, following the concept specified in NZS 3404 for stability of compression members. However, it was quickly realised that the simple application of the notional load in the manner proposed may not always be conservative for BRBF systems, as compared with conventional CBF systems and this led to the establishment of a PhD project, undertaken by Behnam Zaboli, focussing on the buckling behaviour and design of gusset plates in braced steel structures, which commenced in September 2016, with initial funding from SCNZ. Zaboli developed an interim design procedure based on a Notional Yield Line Method which was peer reviewed by Clifton and Cowie with the procedure first presented at a BRBF special session of the 16th World Conference on Earthquake Engineering, Santiago, Chile, January 2017. This method covers both BRBF and CBF systems.

Based on feedback from that Conference, the procedure was modified and presented at the 2017 NZSEE Conference:
Zaboli, B., Clifton G.C., Cowie, K, (2018) *Out of plane stability of gusset plates using a simplified notional yield line method*, Proceedings of the 2017 Technical Conference of the NZSEE, Wellington, New Zealand.

Ongoing development of the procedure and peer review led by SCNZ resulted in the current published design procedure in the SESOC Journal, April 2018:
Zaboli, B., Clifton, C., & Cowie, K. (2018). *BRBF and CBF Gusset Plates: Out-of-plane Stability Design Using a Simplified Notional Load Yield Line (NLYL) Method*. Journal of the Structural Engineering Society of New Zealand (SESOC) Vol31 No1 APR 2018

This procedure was validated as best possible against many BRBF tests and some CBF tests undertaken overseas, which are referenced from the SESOC paper.

When applying the procedure for CBF braces, all key variables could be determined from the geometry of the gusset plate and the brace, so no further experimental testing was required. However, application to BRBs was not so straightforward, as key variables such as the strength and stiffness of the transition zone between the yielding core and the brace end connection into the gusset plate, the length of embedment of this transition region into the BRB restraining jacket, the effective thickness of the slip plane between the yielding core and the buckling restraining system and the depth of bearing at the end supports were difficult or impossible to accurately determine from proprietary manufactured braces. Therefore Zaboli and Clifton sought to validate the procedure against custom made BRBs in which these variables could be carefully controlled. An all steel BRB was developed for this purpose and funding was obtained from the NHRP Contest 2017 fund for 8 experimental tests, subsequently extended to 12 tests.

These tests have been successfully completed and have shown that the design procedure currently published in the SESOC Journal is satisfactory. Using an all steel BRB has enabled key information on the behaviour of the core when being plastically squashed in compression and forming a short wavelength buckled shape against the restrainer to be determined, as well as the influence of the plastic stiffness of the transition region, the depth of embedment of this region, the thickness of the slip plane on the overall behaviour of the BRB gusset plate system. This information is currently being fully processed and will be included in Zaboli's PhD thesis due for submission in February 2020 and in two journal papers currently in preparation.

RELATIONSHIP BETWEEN THE PROJECTS 2b and 2c AND FUTURE OUTCOMES

All steel research undertaken in New Zealand comes under the auspices of the HERA Steel Research Panel, which comprises senior representatives from academia, the consulting profession, the fabrication industry, the construction industry, HERA and SCNZ. This include the two projects, which are complementary, with the focus on 2b being developing a detailed understanding of the BRBF system stability using BRBs with a concrete filled steel tube as the restrainer. The focus on 2c has been on providing an interim design procedure for gusset plate stability of both BRBF and CBF systems using a notional yieldline method as quickly as possible and then hopefully (and successfully) validating that procedure with experimental testing of an all steel BRB with carefully controlled key internal parameters.

At present both projects are in the stage of writing up the experimental data and outcomes from their research. Project 2b has focussed on concrete filled BRBs and the importance of manufacturing quality on the behaviour of the brace and BRBF system. Project 2c has focussed on the system performance of all steel BRBF systems and the magnitude and type of internal actions developed between the yielding core and the restrainer jacket. The information gained from 2b will be important in quantifying the key internal parameters of well built and poorly built BRBs which will feed into the NLYL method of 2c. The internal actions developed within the all steel BRBs will be important in explaining observed behaviour of the concrete filled BRBs from 2b. This interchange of results from the two projects will lead to a better quantified final outcome than either project could achieve on its own and will take place when the proposed design procedures from 2b are completed, under the direction of the HERA Steel Research Panel.

Topic 3: Timber Buildings

Research Team:
Dr. Minghao Li, University of Canterbury, PI
Dr. Lisa-Mareike Ottenhaus, University of Canterbury, former Ph.D. Candidate
Mr. Wenchen Dong, University of Canterbury, Ph.D. Candidate

ABSTRACTS

This Timber Building topic (Topic 3) of Objective 4 involves study of the seismic performance of high capacity dowelled hold-down connections in cross-laminated timber (CLT) shear walls and development of a type of innovative timber-steel hybrid structure consisting of glulam frames braced by buckling restrained braces (BRBs). The overall objective is to achieve robust seismic design for multi-storey mass timber buildings in NZ. Based on the experimental data, design recommendations to improve the design of dowelled connections have been provided to the new timber design standard (NZS/AS 1720.1). Full-scale testing on the hybrid structures also demonstrated enhanced seismic performance in terms of ductility, energy dissipation and reparability.

INTRODUCTION

Public awareness on reducing carbon footprint in building industry has dramatically increased the demand for sustainable construction globally and has initiated the resurgence of using wood in mid-rise and high-rise buildings in the 21st century. NZ has 1.79 million hectares of sustainably managed plantation forests that are able to supply high quality building products, such as glulam, LVL and CLT. Timber materials are relatively light but strong. With proper design, timber buildings generally perform well in earthquakes. In the 2011 Canterbury earthquakes, most timber buildings have proved to have adequate seismic capacity and meet the life safety criteria in the building code. Thus, there are significant economic and societal benefits to build more timber buildings in NZ.

The current timber design standard NZS 3603, and the upcoming new standard NZS/AS1720.1, do not include the most updated seismic design provisions related to mass timber building design. Therefore, the overall objective of the timber topic is to provide robust seismic design solutions for multi-storey mass timber buildings to the engineering community. In particular, this is undertaken by addressing some seismic design challenges caused by high seismic load demand. This topic consists of two research objectives: the first one is to quantify the ductility and overstrength of high-capacity dowelled hold-down connections in multi-storey CLT shear walls; and the second one is to develop innovative timber-steel hybrid structures consisting of glulam frames braced by buckling restrained braces (BRBs).

Research Objective 1
Investigating ductility and overstrength properties of high-capacity dowelled hold-down connection in CLT shear walls

Background

CLT is an engineered wood panel product which has rapidly gained popularity in mid-rise and high-rise building construction over the last decade. CLT panels are used for roofs and floors to withstand gravity loads, and for shear walls that form the lateral load resisting system to withstand wind and seismic loads. Dowelled connections with inserted steel plates are commonly used in mass timber construction and can be used as hold-downs, tie-downs and

splices in CLT shear walls, and provide a key source of ductility and energy dissipation under seismic loading. With the timber itself being prone to brittle failure, seismic energy dissipation relies on ductile elements such as the connections made of inserted steel plates and fasteners. The steel fasteners' ability to be repeatedly loaded beyond their yield limit without significant loss of strength or hysteretic damping capacity is defined as connection ductility (see Figure 1). Capacity design ensures that the system responds in a ductile manner by designing the ductile connection as the weakest link in the strength hierarchy and protecting all brittle components from the connection's overstrength (Jorissen & Fragiacomo, 2011). Therefore, connection overstrength and ductility need to be well understood.

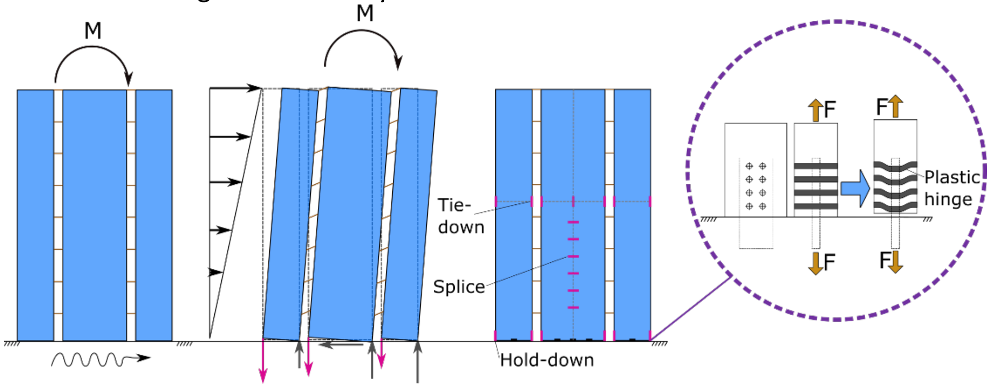

Figure 1. Shear wall structures with ductile connection systems.

Experimental testing

Twelve large-scale dowelled connections in Radiata pine CLT and 39 small-scale dowelled connections in Douglas-fir CLT were tested under monotonic and cyclic loading. The connection design was inspired by the hold-downs in CLT shear wall systems (Figure 1). The specimens were loaded uni-directionally to assess the accuracy of the strength prediction equations following the European Yield Model in Eurocode 5 (2008) and to evaluate the connection performance. Different connection layouts and CLT panel layups were also evaluated to study the influence of dowel spacing and lamination materials on the connection behaviour. Figure 2 and Figure 3 show the test photos of the small-scale connection tests and the large-scale connection tests, respectively. Based on the load-slip curves of the connections, critical connection properties such as strength, stiffness, ductility and overstrength were assessed and design recommendations to improve the connection performance were made.

Figure 2: Dowelled CLT connection tests (left: small-scale connection with 4 ø20mm dowels; right: large-scale connections with 16 ø20mm dowels)

Research Objective 2
Seismic performance of glulam frames integrated with buckling restrained braces (BRBs)

Background
In conventionally braced timber frames, ductility and energy dissipation often rely on the timber connections between the braces and the main frame. Under severe earthquakes, these connections may yield and suffer significant damage. It is often very difficult to repair timber connections and damaged timber members. BRBs are gaining popularity in new steel structures in NZ after the 2011 Canterbury earthquakes since they may provide superior seismic performance compared with conventional steel or timber braces. This project is to investigate the seismic performance of a new hybrid structure type consisting of glulam frames integrated with BRBs that act as critical components to resist seismic loads. In this structure, BRBs provide strength, stiffness, ductility and energy dissipation for the structure while timber members and connections are protected from severe damage according to the capacity design approach.

Experimental testing
This experimental testing consisted of two phases. The first one was to evaluate the behaviour of critical timber-BRB interface connections since these connections must be strong and stiff enough to transfer the load between BRBs and the glulam frames without causing significant damage to timber. Two connection options with steel dowels and screws were investigated and tested, as shown in Figure 3. The second one was to evaluate the hybrid system performance under cyclic loading. As shown in Figure 4, two full-scale one-storey one-bay frame specimens (8 m wide and 3.6 m high) were tested with the dowelled and screwed frame connection systems, respectively.

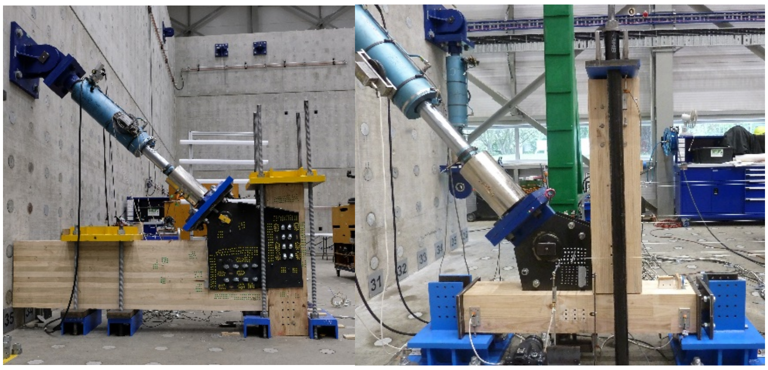

Figure 3. Timber-BRB interface connection tests (left: screwed connection; right: SFS dowelled connections)

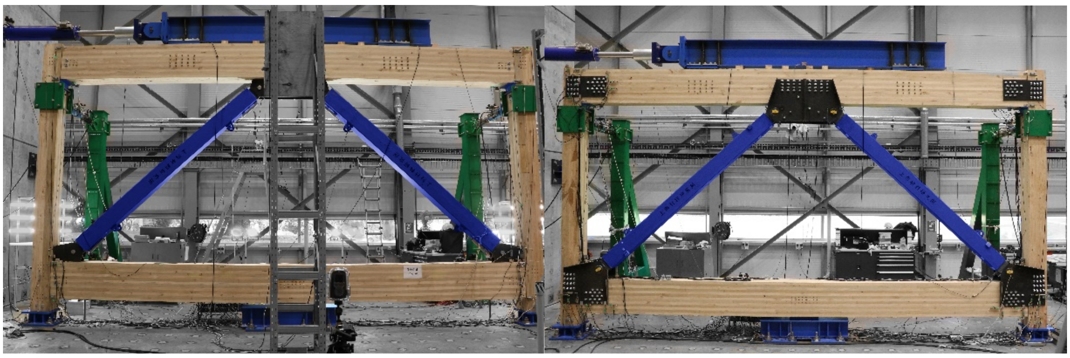

Figure 4 Hybrid system tests (left: with dowelled connection system, right: with screwed connection systems)

CONCLUSIONS AND KEY FINDINGS

Research objective 1 on dowelled CLT connections

- The small-scale dowelled CLT connections generally exhibited high ductility (7.3–14.6) and the large-scale dowelled CLT connections had medium to high ductility (4.8–6.3) without the introduction of an excessive amount of overstrength (1.10–1.63). All the dowelled connections were able to dissipate a significant amount of energy under cyclic loading. Therefore, they can provide robust hold-down solutions for multi-storey CLT shear walls;
- Connection strength can be significantly improved by replacing the outer timber laminations with Laminated Veneer Lumber (LVL) laminations or by using a wider dowel spacing than that specified in current standards;
- It is recommended to use bolts or dowels with threaded ends with nuts and washers in the end row of the connection to avoid opening up of the CLT laminations, thus improving the overall connection behaviour; and
- An analytical method to predict overstrength based on basic material properties of the fasteners and timber materials was developed and verified with experimental data.

Research objective 2 on glulam- BRB hybrid frames

- The hybrid frame specimens had high strength and energy dissipation capability. BRBs yielded significantly and timber members were well protected from damage. Minor damage was observed in the connections. Overall, the capacity design worked well for the hybrid frame system with the BRB overstrength factor $\gamma_o = \omega\beta = 1.5$.
- The minimum system ductility factor of the specimens was 3.0, indicating a ductile system according to NZS1170.5.
- The dowelled connections and screwed connections showed high strength and stiffness. The dowelled connections had slightly lower initial stiffness but became very stiff once all the dowels were fully engaged. The screwed connections provided high initial stiffness but the oversized slot holes on the steel side plates could degrade the stiffness under cyclic loading.
- It was found BRBs could be replaced relatively easily when the residual drift was less than 1%.

IMPLEMENTATION TO PRACTICE

Research Objective 1:

The following recommendations were provided to new timber standard NZS1720.1:

- Use the plastic dowel bending moment equation in the connection strength calculations to achieve better prediction of the connection strength;
- Average overstrength factor for dowelled connections of the sort recommended for NZ was 1.5 for LVL, and 1.7 for CLT members.
- Increase minimum dowel row spacing (7d or larger) for CLT to achieve high connection ductility
- Prying prevention requirement:
  e.g. 1 bolt per 4 dowels, bolts in end row

Research Objective 2:

The innovative timber-BRB system has not yet been applied in practical design. However, design engineers from SESOC, Christchurch Timber Forum and NZ Timber Design Society have shown great interest in this system during a workshop organized in SEL lab on campus in May 2019.

ACKNOWLEDGEMENTS

In addition to the support from MBIE through the NHRP, for which the researchers are very grateful, the authors would also like to thank ARC future timber hub, UC Department of Civil and Natural Resources Engineering, EQC PhD research grant, and QuakeCore Flagship 4 for co-funding the projects. UC structures technical staff Alan Poynter, Russell McConchie and Alan Thirlwell are also acknowledged for providing supports in the experimental testing.

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OUTPUTS & DISSEMINATION

PUBLICATIONS:

Journal papers:

Ottenhaus, L., Li, M. and Smith T. 2019 "Analytical method to estimate overstrength of dowel-type timber connections". ASCE Journal of Structural Engineering (under review)

Ottenhaus, L., Li, M. and Smith T. 2018 "Structural performance of large-scale dowelled CLT connections under monotonic and cyclic loading" Engineering Structures 176:41-48

Ottenhaus, L., Li, M., Smith, T. and Quenneville, P. 2018 "Mode cross-over and ductility of dowelled LVL and CLT connections under monotonic and cyclic loading" ASCE, Journal of Structural Engineering, 144(7): 04018074

Ottenhaus, L. and Li, M. 2018. "Embedment strength of New Zealand cross laminated timber." New Zealand Timber Design Journal 26(1):12-16

Ottenhaus, L., Li, M., Smith, T. and Quenneville, P. 2018 "Overstrength of dowelled CLT connections under monotonic and cyclic loading" Bulletin of Earthquake Engineering. 16(2):753-773.

Conference Papers:

Dong, W., Li, M., Lee, C-L., MacRae, G. and Abu, A. 2019. "Lateral behavior of glulam frames with buckling restrained braces (BRBs)" Proc., 5th Pacific Timber Engineering Conference, Brisbane, Australia

Dong, W. and Li, M. 2019. "A preliminary study on cyclic behaviour of SFS dowelled connections in glulam frames" Proc., 11th Pacific Conference on Earthquake Engineering, Auckland, New Zealand

LIST OF KEY END-USERS:

- Ministry of Business, Innovation and Employment (MBIE)
- New Zealand Timber Design Society and professional engineers
- PTL Consulting, Christchurch
- Enovate Consultants, Auckland

Objective 5: Transportation Infrastructure

Research Team:
Prof. Alessandro Palermo, University of Canterbury, Lead PI
Dr. Royce Liu, University of Canterbury, Post-doctoral Fellow
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Michael Cowan, WSP-OPUS
Mustafa Mashal, Idaho State University, the U.S.

ABSTRACT

Seismic codes do not focus on earthquake resilience, the ability of a system to quickly recover after a large earthquake. Traditionally, the objective of the standards has been only to prevent collapse and ensure life-safety. The Kaikoura earthquake impaired a severe lessons to our bridge community and highlighted the importance to better quantify the post-earthquake residual life of bridges. The extensive structural damage disrupted the functionality of several bridges and became clear that designers should opt for resilient technologies. DCR, dissipative controlled rocking, is a resilient technology that limits the damage to “dissipative fuses” that are easy to replace after an earthquake. The technology was widely tested in the last 8-10 years and despite Wigram-Magdala bridge link, Christchurch – NZ, represented the first real world application, some complexities in the design and construction highlighted the need for the optimization of the design methodology and technology. The outcomes of the last four years programme were promising and an optimised DCR design can be considered almost a cost-neutral solution to a conventional design. NZTA expressed willingness to see DCR implemented more widely and the technology will be properly referred in the new edition of the Bridge Manual.

INTRODUCTION

The Objective 5 – Transportation Infrastructure research programme aimed to optimise damage resistant technologies of bridges built with ABC (Accelerated Bridge Construction) concepts. The Kaikoura earthquake created the opportunity to better investigate the low-cycle fatigue effects of the reinforcement in the proximity of bridge pier plastic hinges (Obj. 1) and then transfer that knowledge into the definition of strain limits of the dissipaters used in DCR (dissipative controlled rocking) Obj. 3. The reparability of a low-damage DCR system is simply limited to the potential replacement of the external dissipaters and therefore this allows to accept more frequent earthquakes at high performance levels (Damage Control Limit State). This design shift allows to design more cost-effective bridges and drastically improve their resilience. One of the lessons learnt from the Wigram-Magdala Bridge link was that the dissipaters needed to be optimised and their connections to the structural elements simplified (Obj. 4). In the following sections each objective is summarised with more details.

Research Objective No. 1: Determining the Residual Capacity of a Plastic Hinge

The 14th November 2016 North Canterbury earthquake caused significant damage to the transportation network. Several bridge structures showed signs of distress, including approach and pier settlement, unseating of spans from bearings, flexural cracking and damage to “knock off” elements and shear keys among other things. Severe plastic hinge damage was only observed in structures close to the fault rupture. However, it was the plastic hinge damage and consequent residual deformations which resulted in costly repairs or full replacement (Wood, 2018).

Capacity design, as it currently stands, allows the formation of a plastic hinge as a means of reducing design forces. However, it is well understood that when plastic hinges form, the residual capacity is limited and repair is necessary to ensure adequate performance in future seismic events. Estimating the residual capacity of plastic hinges is difficult due to various phenomena that occur to longitudinal reinforcement as a result of the loading history. The phenomena include: strain hardening; strain ageing; low cycle fatigue; and buckling, all of which are complicated and difficult to predict. However, the cost benefits of being able to estimate the residual capacity of a plastic hinge are significant, especially when several structures are damaged.

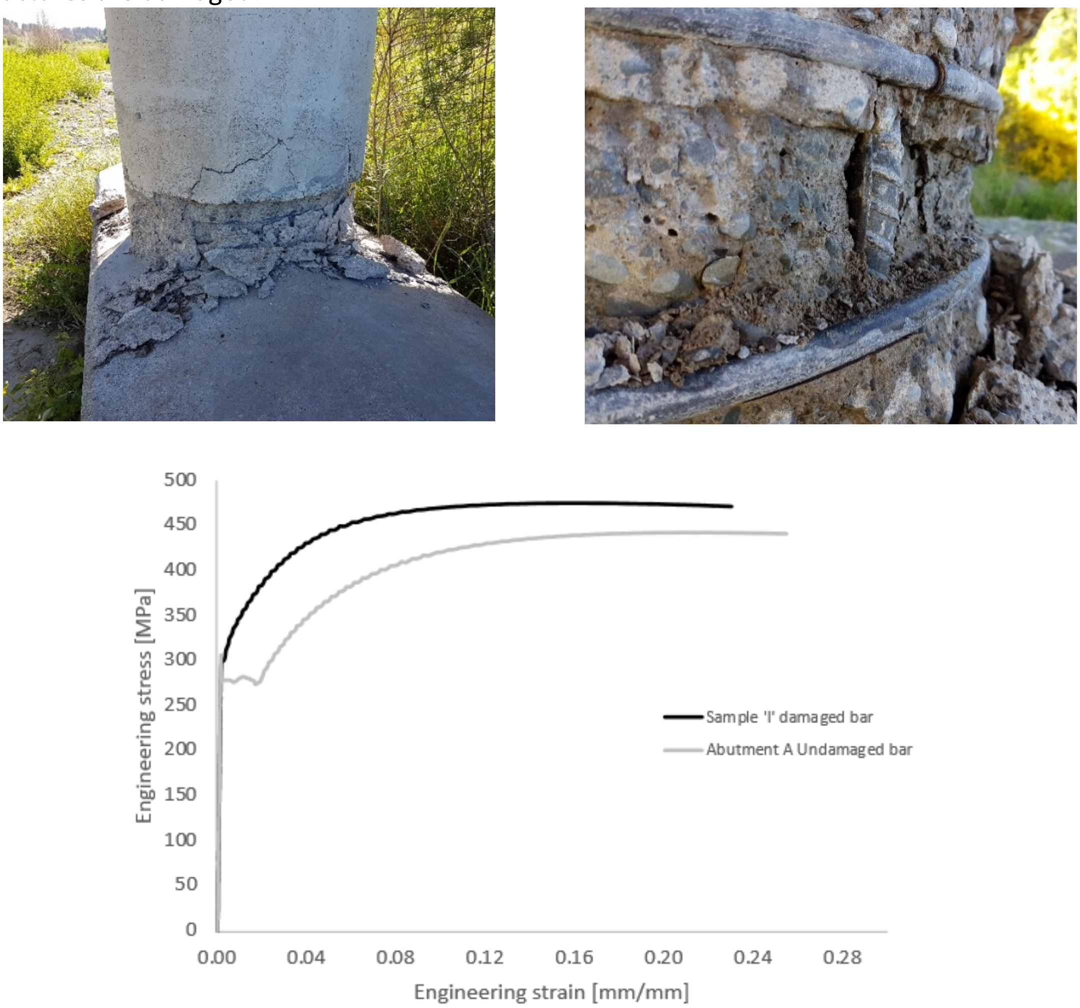

Figure 1: Mason River bridge damage & Stress strain behaviour of damaged and undamaged specimen

The Mason River Bridge was one of the structures that had plastic hinge damage in the Kaikōura earthquake. The damage was severe and repair was difficult given foundations. Therefore, material testing and complex non-linear time history analysis was undertaken in conjunction with WSP-OPUS to try to obtain the residual capacity. Vickers hardness testing and tensile testing were carried out on damaged mild steel samples. Calibration curves developed by (Loporcaro, Pampanin, & Kral, 2018) in conjunction with calibration curves developed specifically for the grade of steel at the Mason River Bride showed the maximum peak strain in the bars was around 4% (Mchaffie, 2018) similar to that obtained through non-linear time history.

The tensile testing of the reinforcement provided further evidence that the ultimate tensile strain assumed during design (12%) had not been reduced. Finally, low cycle fatigue equations were used to predict the low cycle fatigue capacity of the bar. The plastic hinge was found to have sufficient capacity to undergo future DCLS events. Thus, a method for estimating the

residual capacity of a plastic hinge was presented using a combination of non-linear time history and material testing. The cost of analysis was around 5-10% of the estimated repair cost (large cost savings).

Research Objective No. 2: Alternative Design Philosophy

The design of bridges within New Zealand is largely driven by design and construction costs. As a result, the preferred seismic resisting design philosophy is to allow the formation of a plastic hinge to reduce foundation demands. However, as seen in the Kaikōura Earthquake this can result in significant damage and reduce network resilience severely. With the development of Dissipative Controlled Rocking (DCR), a solution that offers improved performance and post-earthquake reparability, a means of accounting for these performance improvements needed to be considered for the design level.

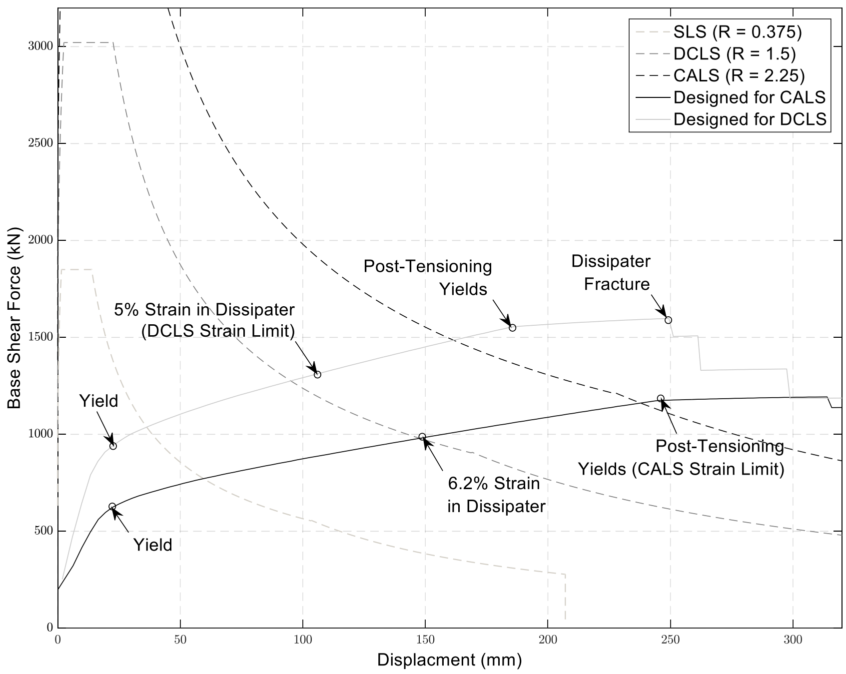

Figure 2: Potential reduction in capacity for DCR connections designed using the alternative design philosophy.

An alternative design philosophy was developed and tested numerically whereby rather than designing to the Damage Control Limit State, as is typically done for plastic hinges, the connection is designed for the Collapse Avoidance Limit State (CALS). The shift in design strategy is based on the fact that the Low-Damage connection is repairable and consequently meets the Damage Control Limit State requirements (ensure reparability) for levels of shaking below the Collapse Avoidance Level. Therefore, the Damage Control Limit State only alters the frequency of repair. This shift in design strategy for low-damage connections allows smaller more efficient connections to be designed (Figure 2). This results in smaller foundations, which has the potential to offset the costs of using the low damage connection potentially helping to facilitate the broader use of the technology.

Research Objective No. 3: Appropriate Strain Limits (CALS)

As a result of the shift in seismic design philosophy, strain limits applicable for dissipative controlled rocking connections needed to be developed for the CALS. A non-linear time history analysis regime using dissipater fatigue models developed by (Liu, 2018) was undertaken. The

fatigue model allows the prediction of the dissipater damage as a result of seismic excitation. By recording the damage for dissipaters designed to various strain levels subjected to ground motions scaled to represent CALS earthquakes an appropriate strain limit was selected.

Table 1: Summary of total dissipater damage for extreme fibre dissipater after a single DCLS earthquake.

	Unbonded Length of Dissipater (mm)					
	400	**500**	**600**	**650**	**700**	**900**
Peak Displacement (mm)	346	348	349	342	343	349
Peak Dissipater Strain (%)	10.3	8.4	7.3	6.4	6	5.1
Peak Damage (%)	49.6	32.4	23	14.5	12.8	10.8

Table 2: Summary of total dissipater damage for extreme fibre dissipater after a single CALS earthquake.

	Unbonded Length of Dissipater (mm)					
	400	**500**	**600**	**650**	**700**	**900**
Peak Displacement (mm)	460	460	459	459	459	459
Peak Dissipater Strain (%)	14.7	11.9	10.5	9.1	8.5	6.1
Peak Damage (%)	100	100	59	50.8	44	26

Preliminary results indicate that a CALS strain limit of 10% would result in adequate performance of the connection at the DCLS and CALS.

Research Objective No. 4: Optimization of dissipative controlled rocking connections – dissipative device development.

While dissipative controlled rocking connections have been well developed over a period of time there are still some aspects which could potentially be improved in order to further optimize the connections. One key area is the dissipative devices which contribute to a significant portion of the additional cost associated with DCR. Two devices were tested as part of a full scale bridge pier. An optimized version of the grooved axial dissipater and the lead extrusion damper where examined. Compared to the approximate base cost of the grooved dissipater the lead extrusion damper was around 30% more expensive and the optimized axial dissipater was around 50% cheaper.

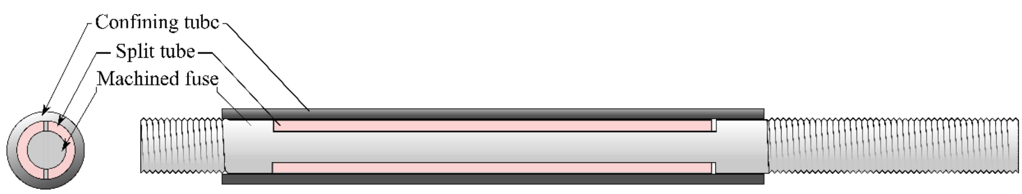

Figure 3: Split tube type dissipater.

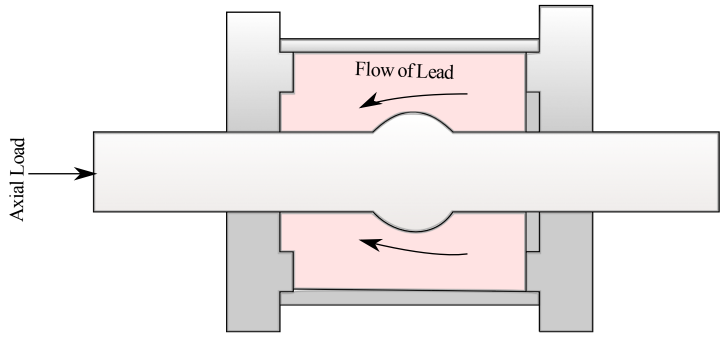

Figure 4: Lead Extrusion damper

The performance of the axial dissipater was found to have a reduced cyclic fatigue life which is due to the geometry of the dissipater itself being less resistant to induced moment demands. The lead extrusion damper (Rodgers, Mander, Chase, & Dhakal, 2016) performed very well with additional displacement capacity and the ability to go through multiple earthquakes without damage.

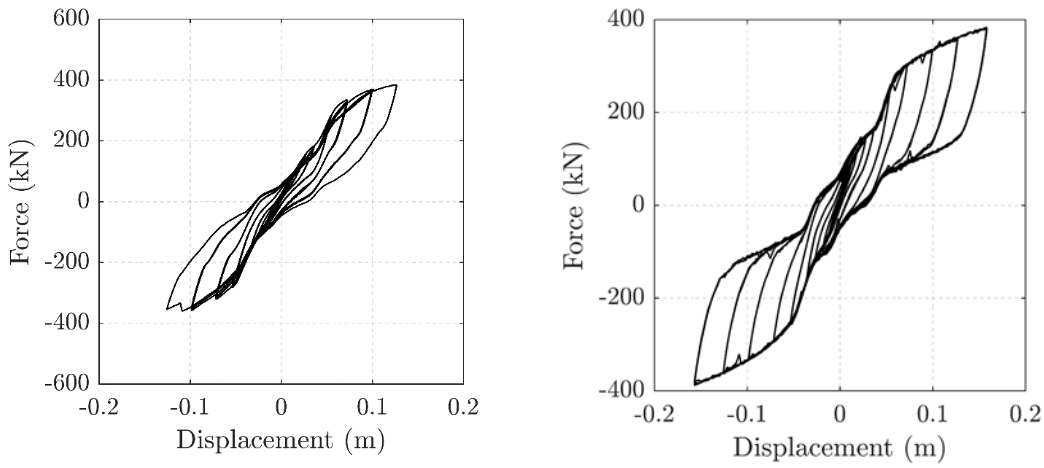

Figure 5: Left – Split tube type dissipater, Right – Lead extrusion damper.

The research has investigated two dissipative devices. The first is cheaper than the four-grooved axial dissipater but has poorer performance with fracture a cycle earlier at 3.5% drift. The drop in performance is only very slight and the dissipater was still found to be suitable for seismic applications. The second is more expensive but has the ability to offer a no damage solution giving the designer the opportunity to bury the dissipaters without having to be concerned with future access. The lead extrusion damper was found to perform to the same level of drifts as the axial dissipater without any damage.

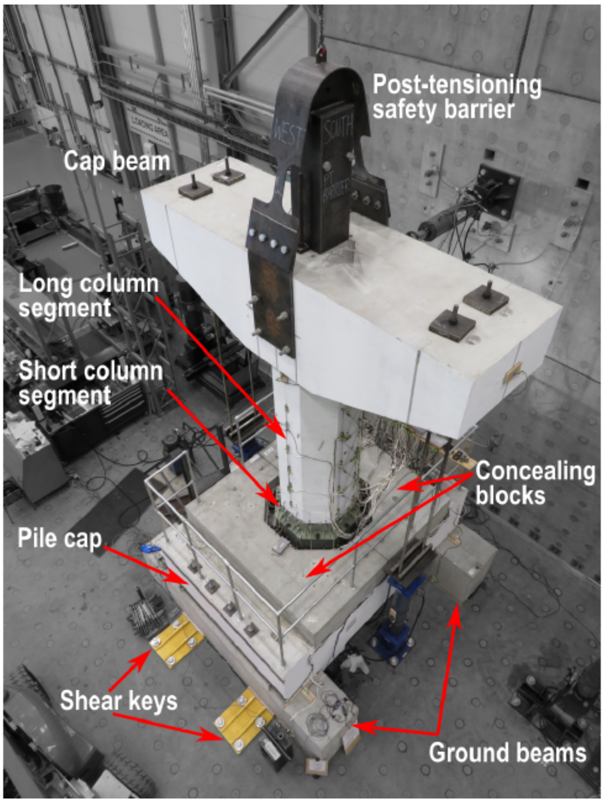

Figure 6: Experimental test setup of a ABC low-damage pier in SEL.

CONCLUSIONS AND KEY FINDINGS

Key findings resulting from this research are summarised below:

1) The bridges damaged by the Kaikoura earthquake highlighted the lack of understanding around the estimation of the residual capacity of the structure. A methodology that combines localised bar testing with advanced numerical analysis was applied to a real case study and allowed to properly predict the residual seismic life of the bridge and therefore saving thousands of dollars to the Agency (Obj. 1).
2) The damage in a DCR (Dissipative Controlled Rocking) bridge pier is similar among the different limit states beyond the yielding and is localised within the dissipaters. As referred in the NZTA Bridge Manual the excellent reparability of DCR can be translated in a higher displacement ductility or a lower return period for a given limit state design. The proposed methodology in Obj. 2 allows to design for most severe limit state scenario, i.e. MCE (Maximum Credible Earthquake) and check the other limit states. This allows huge saving in construction cost since the piers' section can be reduced up to 20%.
3) Low cycle fatigue of conventional bars and dissipaters is crucial for the proper determination of the strain limits at DCLS (Damaged Controlled Limit State) and CALS (Collapse Avoidance Limit State). The experimental and numerical analyses carried out on several prototypes allowed to confirm that a dissipater's strain of 10% at CALS is adequate (Obj. 3).
4) DCR or low damage technology has been widely tested in the laboratory during the first phase of the project (2011-2015). However, a proper optimization of the connection

details was necessary to make the solution more cost-effective. The replaceable dissipaters have been optimised in their size and higher grading (500 MPa) was also considered. The testing confirmed that the dissipater's necking can be limited to 80% of the threaded part and the higher strength (grade500) doesn't compromise the overall ductility and maximum cyclic strain of the device. The led-extrusion dampers also appeared to be a viable alternative although their durability when exposed to harsh environment has not been widely tested yet. (Obj. 4).

IMPLEMENTATION TO PRACTICE

- Real case studies structures: Wigram-Magdala Bridge link, opened in July 2016, clearly expresses the implementation of resilient, repairable and re-centering plastic hinge in bridge piers. It is a reflection of the research carried out at the University of Canterbury. Another novel feature was the use of partially prefabrication of the bridge piers, i.e. partial ABC (Accelerated Bridge Construction) of the bridge substructure. The research around ABC built confidence also within the construction companies. The Taramakau bridge, South Island West Coast opened in July 2018 used full ABC for bridge piers and was completed six months ahead of schedule. Fulton Hogan led the initiative into ABC. NZTA acknowledged the benefits of ABC and low-damage design and, in the Manawatu Tararua Highway, recently incorporated requirements for low damage design. Prof. Palermo is currently working closely with the designers.
- NZTA Bridge Manual: WSP-OPUS and the University of Canterbury submitted a "draft clause" to be inserted in the Chapter 5 – seismic design. The draft reflected the outcomes of a joint WSP-NZTA-UoC workshop organised in 2017 aiming to analyse the lessons learnt from the Wigram-Magdala bridge link. The proposal aimed to properly define low-damage design, its benefits and when it is appropriate to apply it. The next edition of the bridge manual will include a dedicated paragraph aiming to facilitate design departures for special projects.

ACKNOWLEDGEMENTS

The authors are grateful for the continuous collaboration provided by WSP-Opus, NZTA and John Wood Consulting. In particular, John Reynolds, Peter Routledge and Michael Cowan were instrumental in guiding the UoC research team into practical context and applications. Other researchers not directly involved in the project contributed in some testing phases: Prof. Geoff Rodgers provided guidance within the detail design of the led-extruder dampers; Dr. Gabriele Granello and Hayato Auman developed a pseudo-dynamic testing procedure and Adnan Rais supported the PhDs on the project by developing bridge prototypes models. The expertise of the technicians of the University of Canterbury Structural Engineering Laboratory, Gavin Keats and Russell MacConckie, during the experimental phases of the project was instrumental in guiding Dr. Royce Liu, Brendon McHaffie and Mustafa Mashal. The support of the SESOC Bridge Group was also highly appreciated, since it allowed to have most of the research outcomes to be presented to several bridge practitioners.

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Rodgers, G. W., Mander, J. B., Chase, J. G., & Dhakal, R. P. (2016). Beyond Ductility:

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Wood, J. (2018). Five Bridges Located Between Seddon and Kaikoura Report Prepared for NZ Transport Agency. *NZTA*, (May).

OUTPUTS & DISSEMINATION

PUBLICATIONS:

Journal papers:

Liu, R. & Palermo, A. (2019). Using multi-"hinge" hierarchical activation to improve robustness. ACI Structural Journal. (Accepted for publication).

Liu, R. & Palermo, A. (2019). Ten years of experiments on bridges using resilient damage resistant systems and accelerated bridge construction techniques. Structural Engineering International. (Under review).

Liu, R. & Palermo, A. (2019). Characterization of the geometric and low cycle fatigue properties of the grooved type dissipater. ASCE Journal of Structural Engineering. (Accepted for publication).

Liu, R. & Palermo, A. (2019). Using multi-"hinge" hierarchical activation to improve robustness. ACI Structural Journal. (Accepted for publication).

M. Mashal and A. Palermo (2019). Innovative Metallic Dissipaters for Earthquake Protection of Structural and Non-Structural Components, Journal of Soils Dynamics and Earthquake Engineering, Special Issue on Earthquake Resilient Buildings, Vol 116, 31-42.

M. Mashal and A. Palermo (2019). Emulative Seismic Resistant Technology for Accelerated Bridge Construction, Elsevier Journal of Soils Dynamics and Earthquake Engineering, Special Issue on Earthquake Resilient Buildings, Vol 124, 197-211.

M. Mashal, A. Palermo, and G. Keats (2019). Low-Damage Seismic Design for Accelerated Bridge Construction, Special Issue on Accelerated Bridge Construction, ASCE Journal of Bridge Engineering, Vol 24(7).

Andisheh, K.; Liu, R.; Palermo, A.; & Scott, A. (2018). Cyclic behaviour of corroded fuse-type dissipaters for posttensioned rocking bridges. Journal of Bridge Engineering. 23(4).

Palermo, A.; Liu, R.; Rais, A.; McHaffie, B.; Pampanin, S.; Gentile, R.; Nuzzo, I.; Granerio, M.; Loporcaro, G.; McGann, C.; Wotherspoon, L.; & Andisheh, K. (2017). Performance of road bridges during the 14 November 2016 Kaikoura Earthquake. BNZSEE. 50(2).

KEYNOTE/INVITED LECTURES:

Palermo, A.: "Resilient Design of Bridge for Accelerated Bridge Construction", keynote lecture, plenary, *Japanese Prestressed Concrete Institute Symposium*, Nagoya, 6-7 November 2019. 400 participants.

Palermo, A.: "Lessons learnt from the Kaikora earthquake and moving towards resilient design", Invited Speaker, plenary, *8th Kwang-Hua Forum on Innovations and Implementations in Earthquake Engineering Research*, Shanghai, Cina, November. 14-16 December 2018. More than 50 participants (on special invitation).

Palermo, A.: "Lessons learnt from New Zealand Earthquakes and resilient solutions", Invited Speaker special session on - *International Federation for Structural Concrete (fib) - 5th International Congress*, Melbourne, 7-11 October 2018. 550 participants.

Palermo, A.: "*Smart technologies and materials for bridge engineering*", Invited speaker, plenary, NZTA Bridge Summit, Auckland, 6-7 Nov. 2018. 200 participants.

Palermo, A.: "*Resilient design of bridges*", Invited Speaker, plenary, 1st NZ-Japan Joint Symposium on Structural and Geotechnical Earthquake Engineering, Christchurch, 27 Nov. 2018. 100 participants.

Palermo, A.: "Earthquake damage resistant connections for bridge piers" Keynote lecture, plenary, 2nd *Workshop IABEE - International Association of Bridge Earthquake Engineering*, Shanghai, Cina, November. Note: Youngest Key-note presenter among world-renowned researchers of the calibre of Profs. Calvi, Buckle, Kawashima and Saidi. More than 50 participants (on special invitation).

Palermo, A.: "Eight years of developments in seismic low damage bridge design", Invited speaker, plenary, *NZSEE (New Zealand Society for Earthquake Engineering) - ASSISI (Anti-Seismic Systems International Society) joint Conference*, Wellington, New Zealand, April. Special session on low-damage technologies: overview of Accelerated Bridge Construction technologies in seismic areas. 400 participants.

Palermo, A.: "Overview of bridge performance under Kaikoura earthquake", Invited speaker, plenary (two internationals only), *NHERI-UC San Diego Large-Scale Geotechnical Shake Table Test Planning Workshop*, May 31, 2017. More than 40 U.S. participants. Top experts on geotechnical-soil structure interaction (invitation only).

LIST OF KEY END-USERS:

- NZTA - Bridges
- Christchurch City Council.
- Ministry of Business, Innovation and Employment (MBIE)
- WSP – OPUS

Objective 6: Tsunami Hazards and Impacts on Coastal Infrastructure

Research Team:

Prof. Asaad Shamseldin, University of Auckland, Key Researcher
Prof. Bruce Melville, University of Auckland, Science Leader
Dr N. Nandasena, University of Auckland, Science Leader
Dr Liam Wotherspoon, University of Auckland, Science Leader
Ben Popovich, University of Auckland, PhD student
Cheng Chen, University of Auckland, PhD student
Farzad Farvizi, University of Auckland, PhD student
Zhonghou Xu, University of Auckland, PhD student

INTRODUCTION

New Zealand is vulnerable to tsunami due to its long coastline and because 80% of all tsunamis occur in the Pacific Ocean. Ports and bridges on the State Highway network are of significant importance to the NZ economy and are lifeline structures that need to be operational following natural disaster events. An understanding of their performance under tsunami loading is of regional and national importance and will allow for better quantification of the impacts of tsunami events on the wider community. However, there are currently no design guidelines or fragility models for NZ specific construction typologies and conditions.

This objective is focused on: (1) research to support the development of design guidelines for tsunami loading on representative NZ port infrastructure and bridges; (2) characterising the tsunami-loading mitigation effects of representative coastal protection structures and design modifications; (3) development of fragility models for this infrastructure.

Approach taken by the University of Auckland

The university has advanced three threads of research:

- Physical scale modelling to determine likely tsunami loads on wharves and bridges;
- Numerical modelling to estimate the resulting deformation and failure of wharves; and
- Physical scale modelling to assess methods of mitigating those loads by constructing barriers to the tsunami wave, and by design of the structure itself.

The university has undertaken a suite of studies, each of which has had a Ph.D candidate as the primary researcher. Appendix 1 to this report provides a collation of the key findings and formulas that can form the basis for a guideline or code of practice for tsunami design. For bridges, these key findings should be combined with previous research outcomes (Melville et al., 2014, 2015). For wharves, although there is some previous research which should also be taken account of, a new guideline would be needed.

The reporting below on each objective briefly describes the investigation and the outcomes. Appendix 1 is a tabulation of recommended formulas and graphs that could be included in guidelines for the design and appraisal of bridge and wharves. For a full description of the research methodology and the derivation of the resulting formulas and graphs, the reader is referred to the journal papers and conference papers and thesis listed in the Outputs and Dissemination Section

This report's conclusions include an Implementation section that sets out the next steps which could be taken towards a robust approach to the design of wharves and bridges against tsunami. Essentially this entails incorporating the empirical equations from this research into new or existing design guidelines.

Research Objective No. 1: Tsunami forces on bridges

Existing guidelines /information for designing bridges against tsunami

A literature research covering tsunami loads on bridges was carried out by Melville et al. (2014). Subsequent research by the University of Auckland, prior to the present investigations, is described by Melville et al. (2015), who present what is effectively a guideline for design.

Bridge structures modelled in these studies

There are quite a few different bridge types common in New Zealand. Our research has been restricted to more recent types of structure, reasoning that these are the more important to consider because older structures will be replaced over time anyway.

Many New Zealand bridges at the coast cross wide rivers, where it has been economical to build long bridge approaches to force the river flow under a shorter bridge. Our tsunami experiments have therefore included bridge approaches of various lengths.

The modelled bridge layout comprised a 2-span deck (Figure 1 and Figure 2) supported by a single central concrete pile. This layout is common for New Zealand road bridges over smaller rivers, and crossing of wider rivers often differ from Figure 1 only in having more than 1 pier. The essential dimensions of this bridge were obtained as simple averages of the values from five actual New Zealand bridges, all multi-span: Kakanui River (Otago), Kowhai River (Canterbury), Ngaruroro River Diversion (Hawkes Bay), and Waimakariri River (Canterbury).

*Figure **1*** shows generic bridge abutments; wing-wall and spill-through abutments (both commonly built in New Zealand) were both tested. One set of tests was of layouts with the bridge and abutments skewed with respect to the river channel.

Two types of model bridge superstructure have been tested. The first of these is simplified in that it is a “box-section” (*Figure **2***) with a prototype depth of about 1.4 m. Typical bridge designs in New Zealand have a reinforced concrete deck of lesser depth, supported by reinforced concrete or steel beams for which the 1.4 m depth is typical.

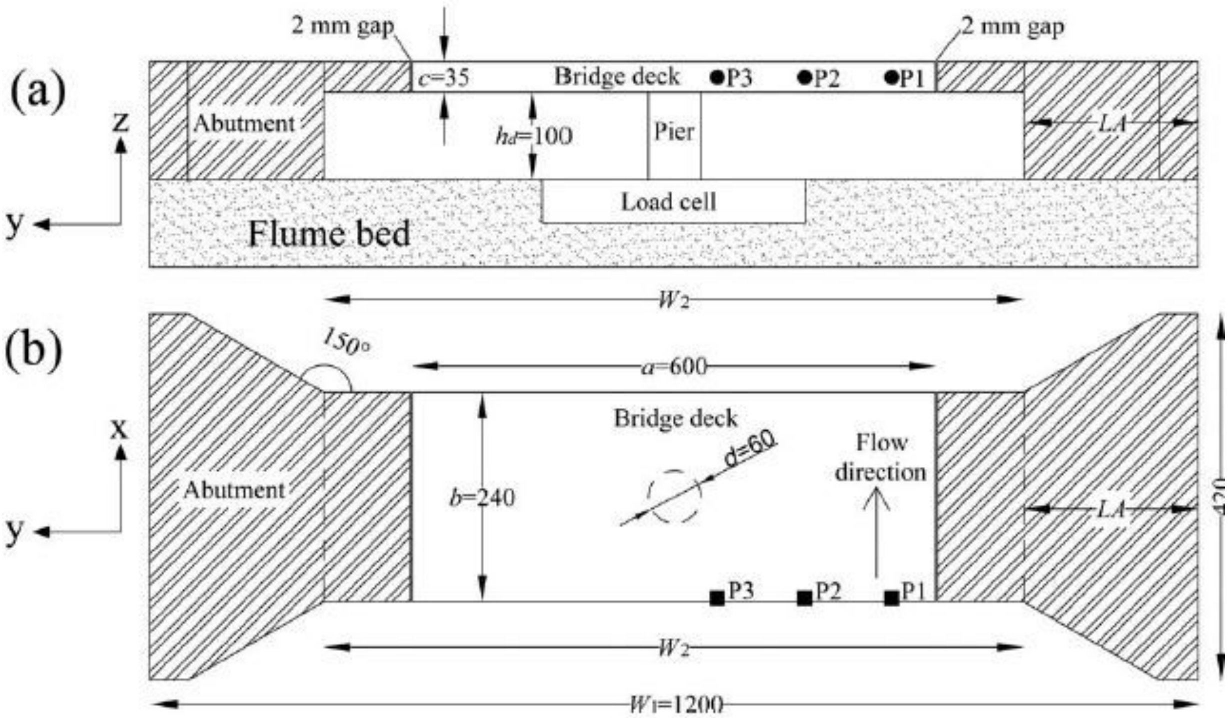

Figure 1 Elevation (a) and plan view (b) of model bridge tested by Cheng et al. (2018) and Farfizi et al. (under review). Dimensions are in mm and the scale factor is 1:40.

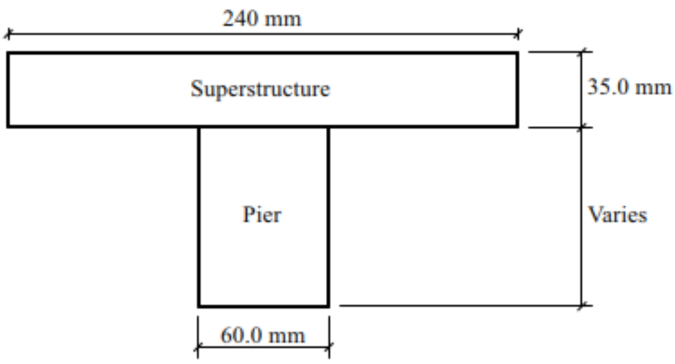

Figure 2 Bridge cross-section, "box-section", model dimensions

The second type of model superstructure tested comprised a thin slab deck supported by girders which are in turn supported by a pile cap (*Figure **3***). This model deck included handrails, which were pervious in some tests and solid in others (both being found in practice).

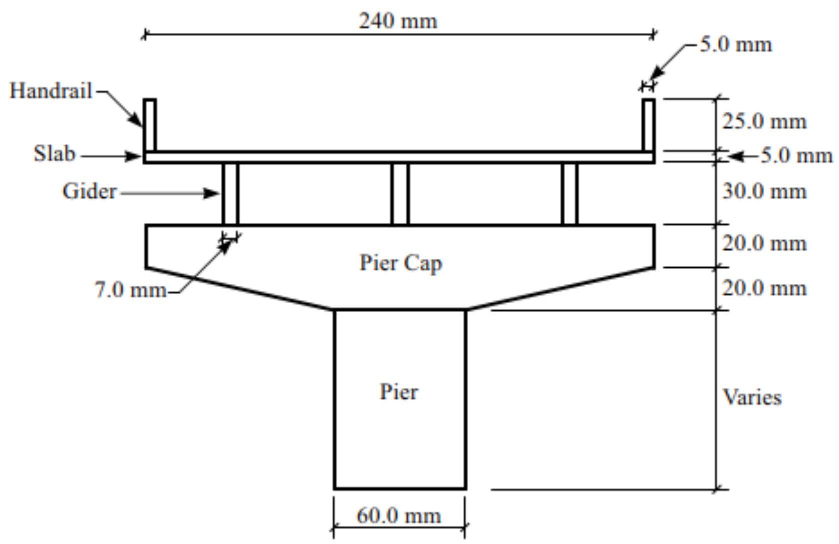

Figure 3 Bridge cross-section, "girder section", model dimensions

Both box-section and girder-section bridge superstructures were tested with the bridge aligned at skew angles of 10˚, 20˚ and 30˚ as well as perpendicular to the flow.

Three different styles of abutment were investigated. A bridge without abutments was tested as well as 3 different lengths of abutment up to the total abutment length equalling that of the bridge.

Testing and outcomes

To cover the range of scenarios detailed above, several sets of laboratory experiments were carried out, each reported in a peer-reviewed paper. Total forces on the bridge superstructure were measured as time series and the peak values identified.

Empirical equations were determined to specify peak horizontal and vertical forces on the bridge superstructure, as functions of the height of the tsunami bore and the height of the superstructure above the channel bed. These include both the peak vertical uplift force (occurring when the tsunami bore arrives) and the generally lesser peak downward force occurring later as the tsunami washes over the structure. Data were also obtained for the peak overturning moment on the superstructure.

These end results of the experiments are tabulated in Appendix 1 and are in a form suitable for marrying with the existing recommendations by Melville et al. (2015) to form a new and more complete guideline.

Published output

- Chen *et al.* (2018)
- Farvizi, F. (under review) Tsunami bore impact force and pressure on bridges. PhD thesis, University of Auckland.
- Farzad Farvizi, , Bruce W. Melville, Asaad Y. Shamseldin, Stuart E. Norris (under review) Experimental investigation of tsunami bore impact force and pressure on a bridge model with different types of abutments

Research Objective No. 2: Tsunami forces on wharves

Type of wharf structure modelled

Many of New Zealand's wharves fall into two structural categories: finger wharves supported on piles, and quays backed by reclaimed land and partly supported on piles. Of the two, the quays are more at risk from tsunami, as a tsunami wave approaching the quay at right angles will be stopped and deflected upwards by the sloping face of the reclamation.

Our research into tsunami effects on wharves has therefore concentrated on the quay structure: a deck supported by piles and by the edge of the reclamation (*Figure **4***)

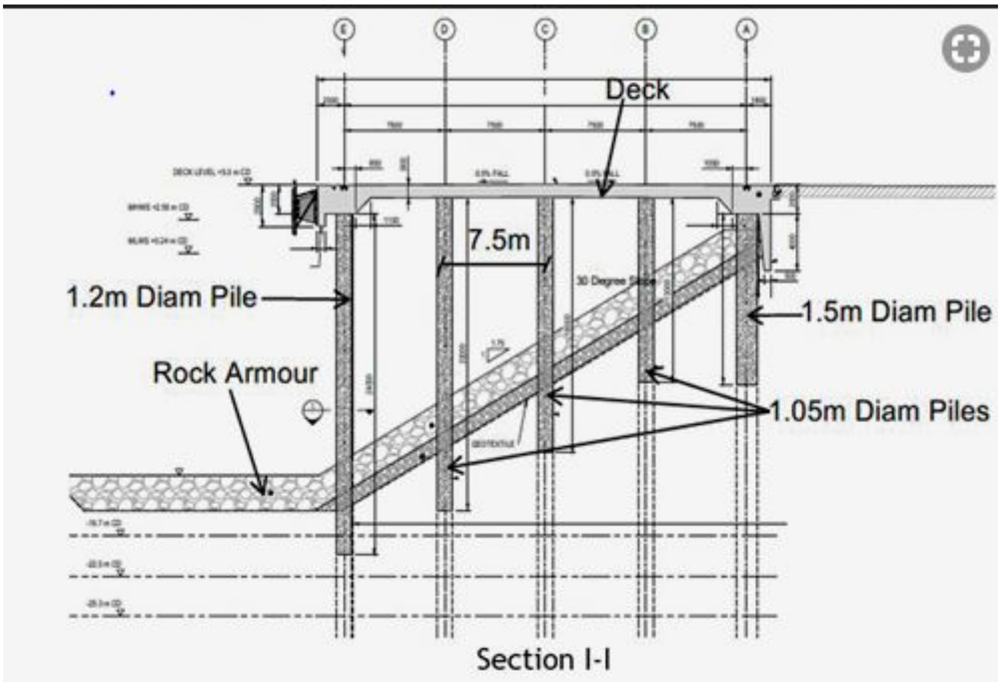

Figure 4 Cross-section of a quay proposed for Lyttelton Port

The exact dimension, the slope of the face of the reclamation, and the arrangement of supporting piles have been varied between individual studies within our investigation. However, the essential physical features have been retained throughout: a reinforced concrete deck supported by piles and a sloping earth face under the deck.

Our physical model studies in laboratory flumes have typically not been of a particular wharf, but of a generic structure representative of those typical in New Zealand. The scale factor for these flume studies is about 100 but will therefore vary a little depending on the prototype wharf being considered.

Testing and outcomes

A series of experiments were carried out on the model quay, with a range of tsunami bore heights. The uplift force on the deck of the quay is the most destructive aspect of tsunami loading, and was measured as a time series with the main interest being in the peak uplift that occurs very shortly after the tsunami bore arrives.

Design equations were formulated for these tsunami-caused uplift loads, as a function of the height of the tsunami bore and the height of the deck above sea bed. The equations cover a wide enough range of conditions to provide the basis for a design guideline.

In some coastal situations, and with some tsunami waves, the wave may arrive at the wharf without breaking to form a bore, and is then a solitary wave or similar. A further set of laboratory experiments were therefore carried out with a solitary wave rather than a tsunami bore, and showed that the uplift force is always less than that from a bore of comparable height.

Existing software for structural analysis was applied to modelling of the deformation and failure of wharf piles under tsunami wave load. This analysis found that when failure occurred it was due to axial tension in the piles, and occurred near the top of a pile and near ground level.

The software can be applied to both existing and proposed wharf pile layouts, for risk assessment and design purposes respectively.

Published papers

Chen *et al.* (2015)

Popovich *et al.* (2015)

Chen & Melville (2015)

Chen *et al.* (2016a)

Chen *et al.* (2018b)

Research Objective No. 3: Tsunami mitigation by protective structures

Laboratory research was undertaken to determine how much mitigation from tsunami forces could be provided for a wharf by a breakwater. Generic scenarios were modelled with a vertical wall breakwater at various distances from the quay it is protecting. In addition, model study for the Port of Napier tested the effectiveness of a "rubble-mound breakwater surfaced by Akmon units.

Vertical wall breakwater

Two sets of laboratory experiments were carried out, each reported in a peer-reviewed paper. One study investigated the force on the deck of a quay, and the other investigated the forces on its piles.

It was found that the distance between the breakwater and the quay had little effect on the mitigation. This is a satisfactory outcome as it means that a purpose-built breakwater can be positioned so as to not to compromise shipping movements.

Empirical equations were determined to describe the forces on the deck and piles, as a function of the tsunami bore height and the breakwater height.

An empirical equation was also found to specify the tsunami force on the breakwater.

Napier Port rubble-mound breakwater

Laboratory experiments were carried out to determine how a rubble-mound breakwater would stand up to tsunami attack and what mitigation it would provide to the quay it protects.

The experiments identified different degrees of damage to the breakwater, and identified empirical formulas that classify the damage as a function of the tsunami height, the breakwater height and the weight of a breakwater element (in this case an Akmon unit).

The breakwater suffered moderate damage in moderate-sized tsunamis, whilst continuing to protect the quay. Only the largest tsunami bores destroyed the breakwater and negated its protective value.

This study was of a particular wharf and breakwater, and more wide-ranging experiments would be needed for generally applicable guidelines. Nevertheless, the findings provide useful and quantitative information which should assist other design situations.

Published papers

Chen *et al.* (2016b)

Research Objective No. 4: Development of fragility models for New Zealand bridge and wharf structures

This objective undertook computational modelling of infrastructure components under tsunami bore impacts. The research advisory group suggested a focus on port components for modelling of damage and performance, given there was more unknowns and further physical testing needed to understand tsunami loading on bridges. As such, the final outputs focus on wharves, and as discussed below, was expanded to specific hazard studies at each port location.

Models for wharf performance

Wharf models were developed using the OpenSees finite element software to represent the structural and geotechnical characteristics of a range of typology wharf models.

A tsunami time series pushover approach was developed, applying both horizontal and uplift loads to the wharf structure. The allowed for assessment of the effect of peak and steady state portions of the time series.

The main output was a set of comprehensive damage thresholds which indicated at which amplitude a certain level of damage would likely occur. Newer wharf structures generally performed better than older structures as a result of more rigorous seismic design standards.

This approach, rather than a fragility based approach allows for rapid implementation into risk assessment software. With the current update of Riskscape in progress, this information is being planned to be incorporated in the next development round.

NZ Port tsunami exposure assessment

Using tsunami propagation models for 13 port sites in New Zealand, the tsunami characteristics at each of New Zealand's largest ports were investigated for both local and distant source scenarios.

In general, the local source scenarios produce greater tsunami at most sites with the exception of those farther south. Distant source tsunami from South America still pose a threat, particularly in the form of damaging tsunami-induced currents.

The modelling was used in conjunction with wharf damage state thresholds to provide a generic assessment of the potential for wharf damage across all ports for a range of events. This is being developed further through co-funded projects, and has been expanded out to other port infrastructure components.

Published papers

Popovich et al. (2019)

Popovich and Wotherspoon (2017)

CONCLUSIONS AND KEY FINDINGS

Conclusions /Summary

1) Physical hydraulic modelling has been carried out of a tsunami bore impacting on representative forms of New Zealand bridges and on a representative New Zealand quay wharf. Further modelling with the quay has tested the effectiveness of a breakwater in mitigating the tsunami bore.

   These experiments have covered a range of tsunami bore height and a range of geometric scenarios. This has allowed empirical equations to be formulated from the results, specifying the forces on the structure as functions of the bore height and structure dimensions including the deck height above the sea/river bed.
2) Existing software for structural analysis was successfully adapted to predict the deformation and failure of wharf piles under tsunami wave load. Pile failure was found to be due to axial tension, and occurred near the top of a pile and near ground level.
3) Physical hydraulic modelling of two forms of defence structure – a vertical wall and a rubble-mound breakwater - showed significant reduction of some tsumami bores. There are likely to be breakwaters that have been built to protect against ocean waves but incidentally provide tsunami protection.

   Empirical equations were obtained specifying the tsunami forces on a vertical wall and the reduction in forces on the wharf it protects.

Recommended future research

- In the experiments described in this report, the Froude Number of the tsunami bore was almost always about 1.6. An investigation of typical tsunami bores is needed to determine whether the Froude Number and other hydraulic properties vary from this laboratory form.
- To determine tsunami loads at a particular location, site-specific work is needed on the likely tsunami bore height resulting from likely tsunami events.
- The experiments described in this report generally included an initially dry bed. Whilst there is often an initial sea level drawdown that leaves the bed dry, further experiments are warranted with a non-zero initial water depth.
- There are many different site geometries and different bridge and wharf structures that could be tested.
- All the physical experiments described in this report have been with clear water. This is an approximation for tsunami bores at some locations, and further experiments could include the high sediment concentration and floating debris that are a feature of many tsunamis as they progress onshore.

IMPLEMENTATION TO PRACTICE

The empirical equations and curves presented in Appendix 1 cover a good range of the more common scenarios in bridge and wharf design, and are in a form suitable for hydraulic design. The equations are therefore suitable for inclusion in a design guideline or manual.

The bridge scenarios cover a range of alignments and approach lengths and two different types of superstructure. The equations should therefore supersede those of Melville et al. (2015b) for these scenarios, whilst the simpler Melville et al. (2015) equations, which were proposed as an upper envelope to the then available data, should still apply to other situations.

There is no New Zealand guideline for the design of quay wharves, nor for the assessment of protecting breakwaters. The empirical relationships found in this series of investigations should be assembled into a guideline, with care taken to circumscribe the conditions to which the equations apply.

Vertical wall breakwaters were tested reasonably thoroughly, but testing of a rubble-mound breakwater was specific to a particular structure. Interpreted with care, the experimental results may provide indicative values for the mitigation provided by other breakwaters. It is perhaps not very likely that a breakwater would be designed and built for tsunami protection. More likely, engineers will want to assess the tsunami protection provided by breakwaters already designed or built for protection from ocean swell.

ACKNOWLEDGEMENTS

We acknowledge the research advisory group for providing guidance during the research programme and key issues for industry. This includes Michel de Vos from Napier Port, Rowan Johnstone from Port of Tauranga, Donald Kirkcaldie as NZ Transport Agency representative, Chennon Chin from Jacobs, and Seyedreza Shafiei from Beca.

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Melville, B., Shamseldin, A., Shafiei, S., & Adams, K. (2015). Outline for Designing Bridges That May Be Subjected to Tsunami Loads; Stage 2: Draft requirements for the consideration of tsunami effects on bridges.

OUTPUTS & DISSEMINATION

PUBLICATIONS:

Journal papers:

Chen, C., Melville, B. W., Nandasena, N.A.K., Shamseldin, A. Y., & Wotherspoon, L. (2016a). Experimental study of uplift loads due to tsunami bore impact on a wharf model. Coastal Engineering, 117, 126-137.

Chen, C., Melville, B.W., Nandasena, N.A.K., Shamseldin, A.Y., & Wotherspoon, L. (2016b). Mitigation Effect of Vertical Walls on a Wharf Model Subjected to Tsunami Bores. Journal of Earthquake and Tsunami. 11

Cheng Chen, Bruce W. Melville, N.A.K. Nandasena, and Farzad Farvizi (2018a) An Experimental Investigation of Tsunami Bore Impacts on a Coastal Bridge Model with Different Contraction Ratios. Journal of Coastal Research: Volume 34, Issue 2: pp. 460 – 469

Chen, Cheng, Melville, Bruce & Nandasena, N.A.K. (2018b). Investigations of Reduction Effect of Vertical Wall on Dam-Break-Simulated Tsunami Surge Exerted on Wharf Piles. Journal of Earthquake and Tsunami. 12.

Chen, C., Melville, B.W., Nandasena, N.A.K (in preparation) Numerical investigations of Dam-break-like tsunami bore impact on a wharf model.

Farzad Farvizi, Bruce W. Melville, Asaad Y. Shamseldin, Stuart E. Norris (under review) Experimental investigation of tsunami bore impact force and pressure on a bridge model with different types of abutments

Popovich, B., Wotherspoon, L.M., Borrero, J. (2019) An assessment of subduction zone generated tsunami hazards in New Zealand Ports, Natural Hazards, in review.

Xu, Z., Melville, B.W., Wotherspoon, L.M., Nandasena, N.A.K. (2019) Stability of composite breakwaters under tsunami attack, Journal of Waterway, Port, Coastal and Ocean Engineering, in review.

Conference papers:

Chen, C., Melville, B.W., Shafiei, S, Popovich, B., Shamseldin, A.Y., Wotherspoon, L. (2015) Quantifying uplift loads on pile supported wharf structures, Part 1: Tsunami bore. Australasian Coasts & Ports Conference 2015. pp. 192-197.

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Popovich, B., Wotherspoon, L., (2017) Performance of New Zealand wharves under Tsunami loading. Australasian Coasts & Ports 2017, 21-23 June, Cairns,Australia.

Reports and theses:

Popovich, B. (2018) Performance of wharves under tsunami loading. PhD thesis, University of Auckland.

Farvizi, F. (under review) Tsunami bore impact force and pressure on bridges. PhD thesis, University of Auckland.

Xu, Z., Melville, B.W., Wotherspoon, L.M. (2019) Stability of composite breakwaters under tsunami attack: An experimental case study (Napier Port Breakwater, New Zealand), University of Auckland Report.

KEYNOTE/INVITED LECTURES:

Popovich, B. Invited lecture PIANC Seminar – Crisis Management and Natural Disaster Response, Australasian Coasts and Ports 2019, September 2019, Hobart, Australia.

Wotherspoon, L. Industry lecture, Lyttelton Port, Response of New Zealand wharves, August 2018, Lyttelton Port.

Wotherspoon, L. Industry lecture, PIANC NZ meeting, Tsunami loading on coastal structures, November 2017, Napier, NZ

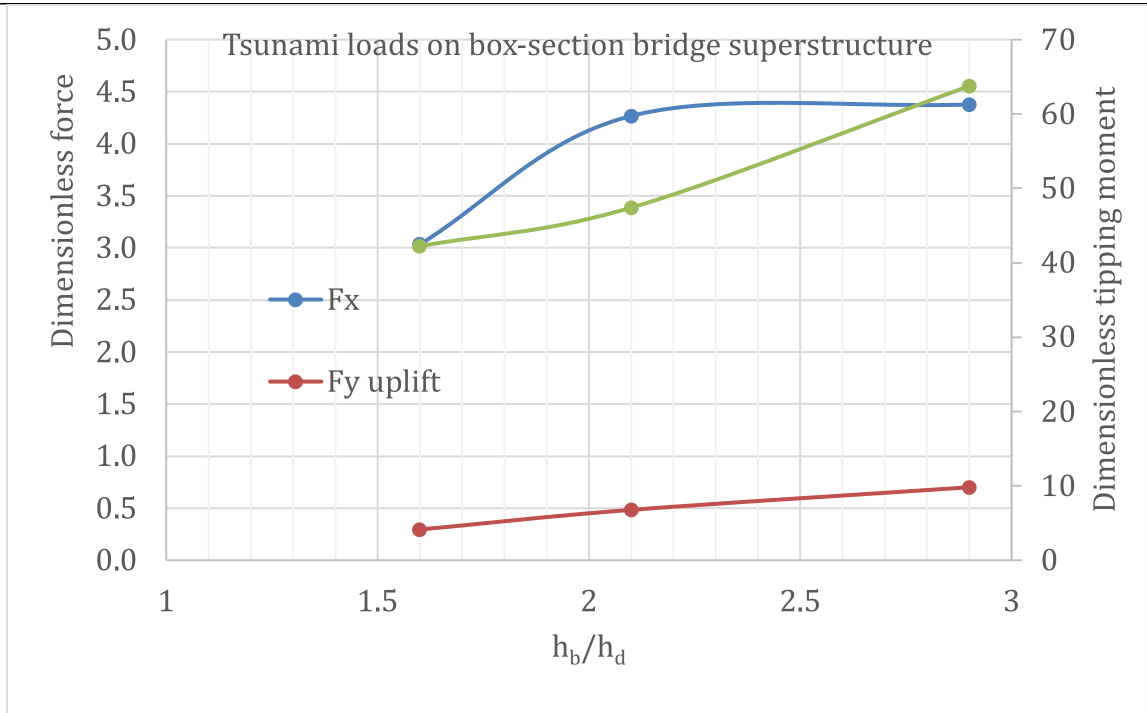
LIST OF KEY END-USERS:

- NZTA
- Port of Napier
- Ministry of Business, Innovation and Employment (MBIE)
- Port of Tauranga
- Engineering consultancies

Appendix 1: Research outcomes: Equations for use in design and assessment of structures

	Quay Wharves
Experimental Study of Uplift Pressures on Wharf Decks Due to Tsunami Bores Chen et al. (2016a)	**Peak upward pressure on the deck of a quay wharf** $$\frac{p_f}{\rho g h_b} = (\cot\theta + C_2) * \ln\left[\frac{h_b}{h_d} * \left(\frac{1}{2}Fr_b^2 + 1 - \frac{1}{2}k_1 Fr_b^2\right)\right]$$ $$for\ 0.5 \leq \frac{h_b}{h_d} \leq 1.4, 20^\circ \leq \theta \leq 90^\circ, Fr_b = 1.6 \qquad (12)$$ where $C_2 = 1.83$ by fitting from the experiment data. and k_1 is an energy loss factor; experiment indicates a value of 0.22 **Caution:** the dependence on Froude Number is not confirmed, as all the experimental bores had comparable Froude Number of about 1.6. For a moderately conservative estimate, Fr = 1.8 is recommended. A comparable formula is also provided in the paper for the longer-lasting quasi-steady upward pressure that follows the above peak.

	Wharves: mitigating wall
Mitigation Effect of Vertical Walls on a Wharf Model Subjected to Tsunami Bores Chen et al. (2017)	**Peak upward pressure on the deck of a quay wharf protected by a vertical wall** $$\frac{p_d}{\rho g h} = \left(c_1 \frac{h_w}{h_d} + c_2\right) * ln\left(\frac{h}{c_3 h_w + c_4 h_d}\right) \quad \text{for}$$ $$0.68 \leq \frac{h}{h_d} \leq 1.12, \quad 0 \leq \frac{h_w}{h_d} \leq 1.2, \quad \text{and} \quad \mathrm{Fr}_b = 1.4.$$ where p_d is deck-averaged uplift pressure, ρ is density of water, g is gravity acceleration, h is tsunami inundation depth, u_b is bore velocity, h_d is deck height, h_w is vertical wall height, and S is the distance between the wall and the wharf model. The coefficients c_1 to c_4 have the values 1.27, 3.71, 0.28, and 0.45. A comparable formula is also provided in the paper for the longer-lasting quasi-steady upward pressure that follows the above peak.
Investigations of Reduction Effect of Vertical Wall on Dam-Break-Simulated Tsunami Surge Exerted on Wharf Piles Chen et al. (2018b)	**Peak pressure on the wharf piles** $$\bar{p}_1 = \rho g h_s \left(0.53 \frac{\bar{h}_w}{\bar{h}_p} + 3.36\right) \ln\left[\frac{h_s}{h_p}\left(1.93 - 0.58 \frac{h_w}{h_p}\right)\right]$$ Within $0.62 \leq \bar{h}_s/\bar{h}_p \leq 1.17$ and $0 \leq \bar{h}_w/\bar{h}_p \leq 1.25$ (1) density of water ρ; (2) gravity acceleration g; (3) tsunami surge height h_s; (4) tsunami surge velocity u_s; (5) vertical wall height h_w; (6) pile height h_p; and (7) the distance between the wall and the pile model S.

	Bridges In these graphs and equations, h_d is the height above bed of the superstructure soffit, h_b is the height of the tsunami bore, and the submergence ration Sr = h_b/h_d
	Box-section bridge (*Figure* 2) spanning complete waterway, no skew
Farvizi (**thesis**)	 **Where:**

	Fx* = Fx/span/superstructure height/$\rho g h_b$ **Fy*** = Fy/span/superstructure width/$\rho g h_b$ **M* = M/**span/(superstructure height)^2/$\rho g h_b$
	Bridge with approaches (restricted waterway) Defining the contraction ratio R = flow width at the bridge/flow width in the approach channel:
An Experimental Investigation of Tsunami Bore Impacts on a Coastal Bridge Model with Different Contraction Ratios Chen et al. (2018a)	It should be noted that Equations (6)–(11) are based on the experimental data for the following ranges of variables: $0.5 \leq R \leq 1$, $1.2 \leq h_b/h_d \leq 2.2$, $1.8 \leq Fr_b \leq 1.9$, and $b/h_d = 2.4$. where b is the superstructure width (i.e. deck width). **Peak horizontal pressure on bridge piles and superstructure** $$\frac{p^*_{x(\max)}}{\rho g h_b} = 0.41 \times R + 0.98 \quad (6)$$ **Peak vertical pressure on bridge superstructure** $$\frac{p^*_{z(\mathrm{up})}}{\rho g h_b} = -1.71 \times R^2 + 2.15 \times R - 0.21 \quad (8)$$ $$\frac{p^*_{z(\mathrm{down})}}{\rho g h_b} = -0.46 \times R + 0.79 \quad (9)$$

	Peak overturning moment on bridge superstructure $$\frac{M^*_{(\max)}}{\rho g h_b} = 4.60 \times R^2 - 8.50 \times R + 10.67 \qquad (10)$$ Where M* = overturning moment/[h_d x area of front face of superstructure]
Experimental investigation of tsunami bore impact force and pressure on a bridge model with different types of abutments Farvizi (**thesis**)	
	Box-section: Wing wall and spill-through abutments
	Peak horizontal force on bridge superstructure $$\frac{F_x}{\rho g h_b A_h} = (e \ln(S_r) + f)R + i \qquad (4.7)$$ where A_h = (span x superstructure height) is the area subject to horizontal pressure. **Peak vertical <u>uplift</u> force on bridge superstructure** $$\frac{F_{z^+}}{\rho g h_b A_v} = (b \ln(S_r) + c)R^2 + (e \ln(S_r) + f)R \qquad (4.8)$$

Peak vertical <u>downwards</u> force on bridge superstructure

$$\frac{F_{z^-}}{\rho g h_b A_v} = \left(aS_r^2 + bS_r + c\right)R^2 + \left(dS_r^2 + eS_r + f\right)R + \left(gS_r^2 + hS_r + i\right) \quad (4.9)$$

where A_v = (span x superstructure width) is the area subject to upwards and downwards vertical pressure.

The coefficients for the above three equations have been determined as follows:

Eq.	Force	Abutment	*a*	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*
(4.7)	F_x	Wing Wall	-	-	-	-	1.4065	-0.7151	-	-	1.75
		Spill-through	-	-	-	-	1.3226	-0.5527	-	-	1.75
(4.8)	F_z^+	Wing Wall	-	-0.758	-0.9296	-	0.9309	0.9452	-	-	-
		Spill-through	-	-0.509	-1.1403	-	0.7106	1.1947	-	-	-
(4.9)	F_z^-	Wing Wall	-1.1144	2.66	-0.6263	1.7169	-3.4772	-0.6236	-0.5773	0.5641	1.8896
		Spill-through	-0.6733	2.2015	-1.0177	1.3532	-4.9535	2.416	-0.6289	2.852	-1.2295

Skewed box-section

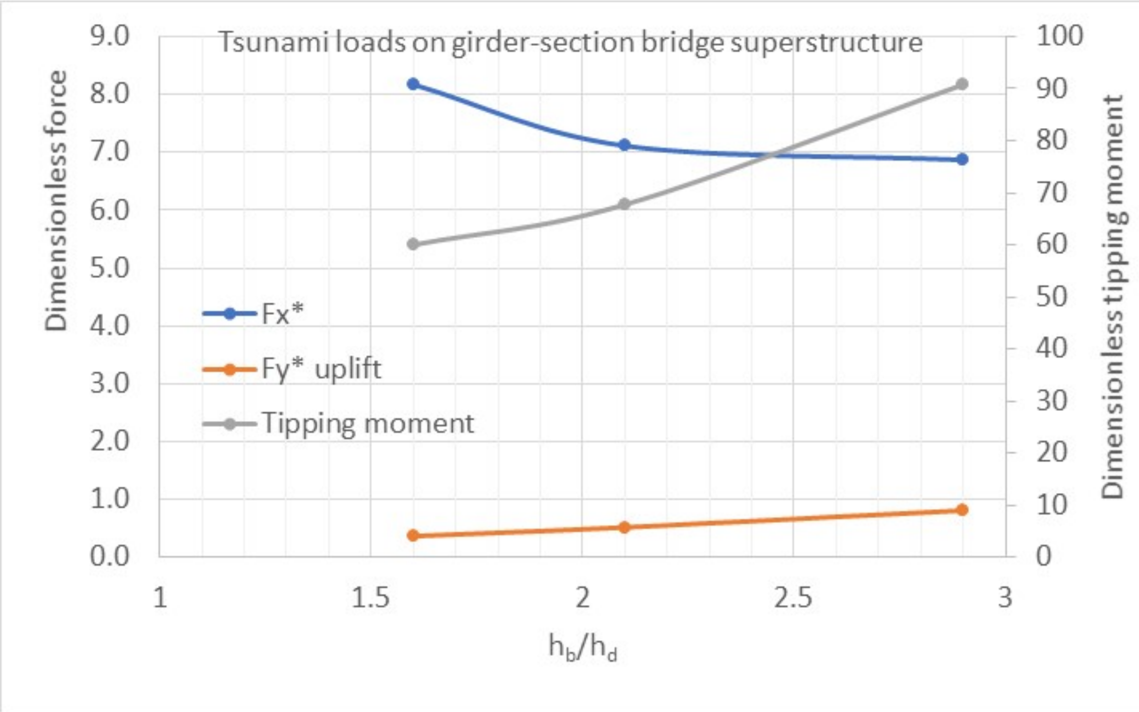
In these equations, θ is the angle of the bridge alignment from perpendicular to the river channel

Peak horizontal streamwise force on bridge superstructure

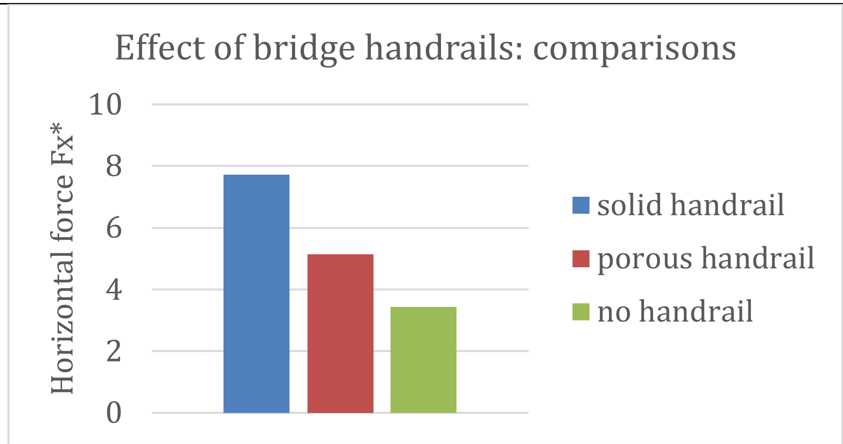
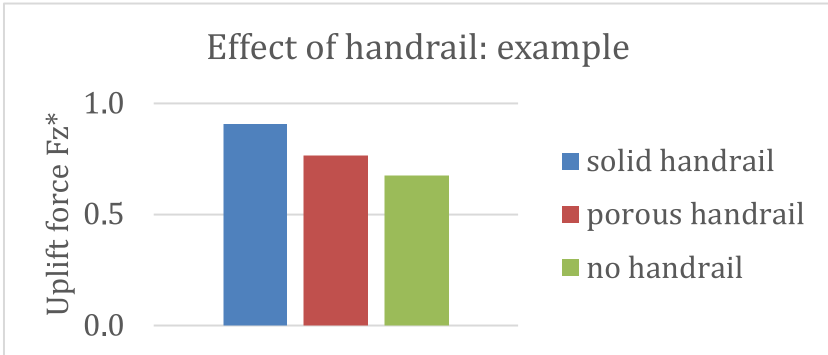
$$\frac{F_x}{\rho g h_b A_h} = \left(-\tan\theta + C_1\right) * \ln\left(2.86 * S_r\right) \quad (4.20)$$

For $1.05 \le S_r \le 2.90$, $Fr = 1.90$ and $0° \le \theta \le 30°$.

	where $C_1 = 2.1$ by fitting from the experimental data. **Peak horizontal force on bridge superstructure perpendicular to the river channel** $\frac{F_y}{\rho g h_b A_h} = (\tan\theta + C_1) * \ln(4 * S_r)$ (4.24) for $1.05 \le S_r \le 2.90$, $Fr = 1.90$ and $0° \le \theta \le 30°$. where $C_1 = 0.08$ by fitting from the experimental data. **Peak vertical <u>uplift</u> force on bridge superstructure** $\frac{F_z}{\rho g h_b A_v} = (-\tan\theta + C_1) * \ln(3 * S_r)$ (4.28) for $1.05 \le S_r \le 2.90$, $Fr = 1.90$. where $0.30 \le C_1 \le 0.85$ for $0° \le \theta \le 30°$ by fitting from the experimental data. **Caution: note uncertainty in C_1.**

	Girder-section (*Figure **3***)
	

	Skewed girder section **In these equations, θ is the angle of the bridge alignment from perpendicular to the river channel**
	Peak horizontal streamwise force on bridge superstructure $\frac{F_x}{\rho g h_b A_h} = \left(-\tan\theta + C_1\right) * \ln\left(2.22 * S_r\right)$ (4.32) For $1.05 \le S_r \le 2.90$, $Fr = 1.90$ and $0° \le \theta \le 30°$. where $C_1 = 3.0$ by fitting from the experimental data. **Peak horizontal force on bridge superstructure perpendicular to the river channel** $\frac{F_y}{\rho g h_b A_h} = \left(\tan\theta + C_1\right) * \ln\left(2.22 * S_r\right)$ (4.36) for $1.05 \le S_r \le 2.90$, $Fr = 1.90$. where $0.2 \le C_1 \le 0.4$ for $0° \le \theta \le 30°$ by fitting from the experimental data. **Caution: note uncertainty in C_1.** **Peak vertical <u>uplift</u> force on bridge superstructure** $\frac{F_z}{\rho g h_b A_v} = \left(-\tan\theta + C_1\right) * \ln\left(2.86 * S_r\right)$ (4.40) for $1.05 \le S_r \le 2.90$, $Fr = 1.90$. where $0.40 \le C_1 \le 0.90$ for $0° \le \theta \le 30°$ by fitting from the experimental data **Caution: note uncertainty in C_1.**

	Effect of bridge handrails: comparisons Horizontal force Fx* 0 2 4 6 8 10 solid handrail porous handrail no handrail Effect of handrail: example Uplift force Fz* 0.0 0.5 1.0 solid handrail porous handrail no handrail