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# Seismic response of a typical New Zealand pile-supported wharf configuration

B. Raged, L.W. Wotherspoon & J.M. Ingham

*Department of Civil and Environmental Engineering, University of Auckland, Auckland.*



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**ABSTRACT:** Ports are an important part of a country's infrastructure, both in terms of facilitating trade and aiding recovery immediately following an earthquake. In New Zealand ports facilitate the transfer of up to 99% of all exports and imports by volume and thus are important to the success of the country's economy. Past earthquakes have demonstrated that port facilities suffer extensive damage due to poor foundations and backfill soils that are common in waterfront environments. In collaboration with New Zealand port authorities, generic wharf configurations representative of New Zealand structural and geotechnical characteristics have been developed. This paper presents the modelling approach and preliminary results for two common wharf configurations founded in three non-liquefiable soil profiles. The numerical models were created using OpenSees, a non-linear finite element analysis program and subjected to static nonlinear pushover analyses and dynamic time-history analyses. The wharf and the soil-pile interface have been modelled in order to account for the effects of nonlinear behaviour of pile elements and their connections to the wharf deck, and effects of nonlinear dynamic pile-soil interaction. The models were then used to develop fragility curves that are used to predict the probability of a model reaching a defined damage state given a PGA. For low intensity earthquakes there was limited variability in performance between the different wharf models. However as earthquake intensity increased there was a pronounced difference between models with a raked-pile configuration and ones with a tie-back configuration, with the tie-back configuration having lower probabilities of damage. There appears to be no clear pattern with regards to the raked-pile configuration located in different soil profiles.

## 1 INTRODUCTION

Ports are complex commercial and physical entities at the interface between sea and land transport. They have strategic significance to New Zealand's economy, facilitating the transfer of up to 99% of all exports and imports by volume (New Zealand Trade and Enterprise, 2010). Further to the economic importance, the Civil Defence Emergency Management Act (2002) has identified ports as lifelines that need to be in operation following a natural hazard event. Ports have a vital role in delivering aid, emergency water supplies, construction materials, heavy equipment and other goods needed for facilitating a rapid recovery of the local region. The damage to ports as a result of natural hazards can result in significant short and long term losses. The effect of natural hazards on ports was evident in the 1995 Great Hanshin earthquake, Japan, where damage to the port in Kobe was estimated at 1 trillion yen (NZD\$15 billion) and took almost 2 years to repair. The disruption caused by the closure of the port was valued at 30 billion yen (NZD\$453 million) per month due to the loss of port related industries and trade (Chang, 2000). Likewise, the 2010 Darfield earthquake in the Canterbury region caused \$50 million worth of damage to Lyttelton Port (TVNZ, 2010). The port was further damaged in the 2011 Christchurch earthquake resulting in significant loss of operational capability due to wharf movements reaching 0.5 m vertically and 1 m horizontally (Lyttelton Port of Christchurch, 2011). Damage is predicted to cost in excess of \$500 million to repair (Wood, 2013).

The study reported in this paper is part of a larger research project at the University of Auckland aimed at determining the resilience of New Zealand port infrastructure to natural hazards. This paper focuses on wharves, the infrastructure used for mooring vessels and providing a level area to transfer

cargo to and from the vessel. Numerical models for two wharf configuration representative of a typical pile-supported wharf found in New Zealand were developed in OpenSees, a nonlinear finite element analysis software (McKenna, et al. 2000), and subjected to pushover analysis and time-history analyses using a suite of ground motions scaled to multiple levels of PGA. The models were placed in three soil profiles representative of ground conditions found in New Zealand ports. The analysis results were then used to develop fragility curves used for predicting the expected damage at New Zealand wharves.

## 2 WHARF CONFIGURATIONS

In this paper total of 6 numerical models were created to capture the variability of wharves and surrounding ground conditions found in New Zealand. The six models were developed using two structural configurations placed in three soil profiles as shown in Table 1.

**Table 1. Models used in this study**

<b>Model</b>	<b>Structural Configuration</b>	<b>Soil Profile</b>
Model 1	Configuration 1 – Raked Pile	Profile A – Soft Clay
Model 2	Configuration 1 – Raked Pile	Profile B – Stiff Clay
Model 3	Configuration 1 – Raked Pile	Profile C – Dense Sand
Model 4	Configuration 2 – Tie-back	Profile A – Soft Clay
Model 5	Configuration 2 – Tie-back	Profile B – Stiff Clay
Model 6	Configuration 2 – Tie-back	Profile C – Dense Sand

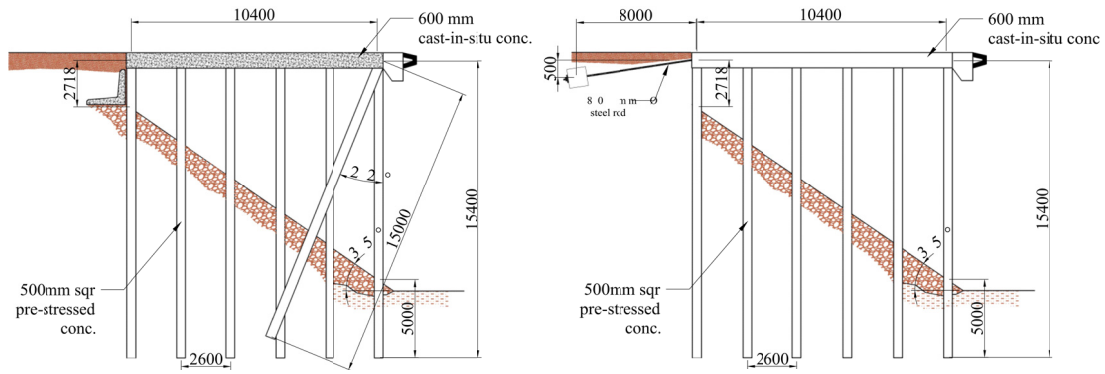
### 2.1 Structural

The deck in both configurations was a cast-in-situ reinforced concrete slab with a thickness of 600 mm, a width of 10.4 m and a concrete compressive strength of 38 MPa. The reinforced concrete piles have 500 mm square sections, constant lengths, 38 MPa compressive strength concrete and 300 MPa reinforcing. The properties adopted in this study were extracted from wharf drawings constructed between 1960 and 1980. The transverse sections were assumed to repeat every 5 m in the longitudinal direction. The lateral loads are resisted by the moment resisting pile-deck connection and raked pile in Configuration 1 and the tie-back in Configuration 2.

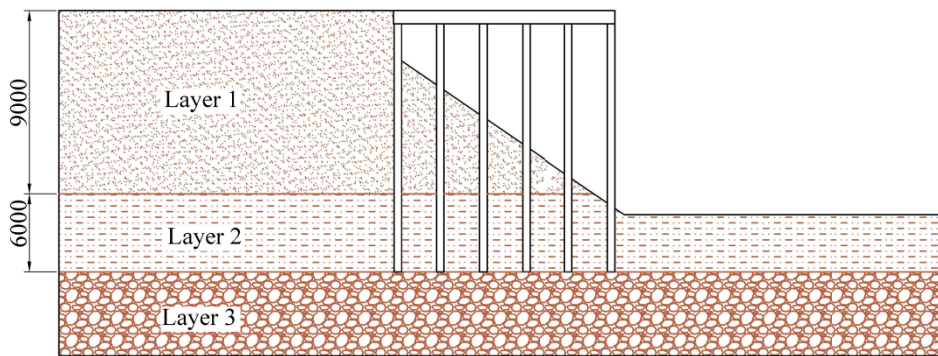
### 2.2 Geotechnical

The generic slope profile adopted in this study, shown in Figure 2, was a simplified representation of typical profiles found surrounding New Zealand wharves. A typical slope consists of several soil layers varying both in thickness and composition but for the purpose of this study the slope has been simplified into three layers representing three distinct composition types. The top layer was assumed to be a fill layer 9 m in thickness, underlain by a shallow layer composed of the local soil deposit and 6 m in thickness. This in turn was assumed to be underlain by a deep stiff layer.

Three soil profiles were fitted to this generic slope as shown in Table 2. Profile A is common for older type wharves in which dredged material was used to reclaim land on which wharves were built. This construction technique results in the fill layer having the same properties as the local shallow layer (either loose sand or soft clay). Profile B and C represent modern construction in which the fill layer is engineered to ensure good quality material is used. In each profile, layer 3 was assumed to be very stiff resulting in high pile tip bearing forces. These generic soil profiles were developed based on an analysis of the available geotechnical data relating to wharves in New Zealand, both collected from ports and supplemented by geological studies found in the literature.



**Figure 1. Wharf structural configurations. (a) Configuration 1 - Raked pile, (b) Configuration 2 - Tie-back**



**Figure 2. Generic slope profile used in this study**

**Table 2. Soil profile properties**

Layer	Property	Profile A	Profile B	Profile C
Layer 1	Soil type	Soft clay	Stiff clay	Dense sand
	Unit weight	17 kN/m <sup>3</sup>	22 kN/m <sup>3</sup>	22 kN/m <sup>3</sup>
	Friction angle	NA	NA	35
	Undrained shear strength	50 kPa	100 kPa	NA
Layer 2	Soil type	Soft clay	Soft clay	Dense sand
	Unit weight	17 kN/m <sup>3</sup>	22 kN/m <sup>3</sup>	22 kN/m <sup>3</sup>
	Friction angle	NA	NA	35
	Undrained shear strength	50 kPa	100 kPa	NA

### 3 MODELLING APPROACH

#### 3.1 Deck

The deck has a high stiffness relative to the remainder of the structure and accordingly was expected to remain elastic throughout the analysis. It was modelled using elastic beam column elements with an associated cross-sectional area, elasticity and moment of rotation. A lumped mass model was used in applying the mass of the deck in the horizontal direction. The mass at the deck nodes was based on the tributary area of deck surrounding each node.

### 3.2 Piles

In practice both the section properties and pile length vary both along the pile and from one pile to another, however for the purpose of this generic study all the piles were assumed to be of the same length and have the same section, shown in Figure 3a. The piles were modelled using displacement-based beam-column elements with a defined nonlinear fiber cross-section, shown in Figure 3b. The fiber cross-section contained three discretised sub-regions: a cover layer of unconfined concrete, an inner core region containing confined concrete and a square layer of reinforcement bars. The concrete was modelled using the *concrete02* material model and *steel02* was used to model the reinforcement bars.

### 3.3 Soil-Structure Interface

A Beam on a Non-linear Winkler Foundation approach was used to model the soil-pile interaction effects, shown in Figure 4a. In this approach the interface is modelled as a set of discrete spring elements with an associated force-deformation response. P-y springs capture the lateral response, t-z springs capture the axial response and a q-z spring for pile tip bearing. These interface elements are modelled as a set of *zeroLength* elements with associated *pySimple1*, *tzSimple1* and *qzSimple1* OpenSees material models. These models, developed by Boulanger, et al. (2000) capture elastic response, plastic response, gap formation and radiation damping. The spring force-deformation response is developed based on the geotechnical properties of the soil profile, pile size and element tributary area. Figure 4b shows a set of p-y curves for a pile embedded in Profile A.

### 3.4 Lateral Response Mechanism

A raked pile or tie-back were used in each configuration to resist the lateral loads imposed on the structure. The raked pile was modelled similarly to the vertical piles using displacement-based beam-column elements and p-y, t-z and q-z pile-soil interface elements all angled at 22 degrees from the vertical. The tie-back was composed of 3 components representing the anchor, tie-rod and abutment as shown in Figure 5a. The *steel01* material model was used to capture the response of the tie-rod and the *hyperbolicgapmaterial* model was used to capture the soil response at the anchor and the abutment, as shown in Figure 5b.

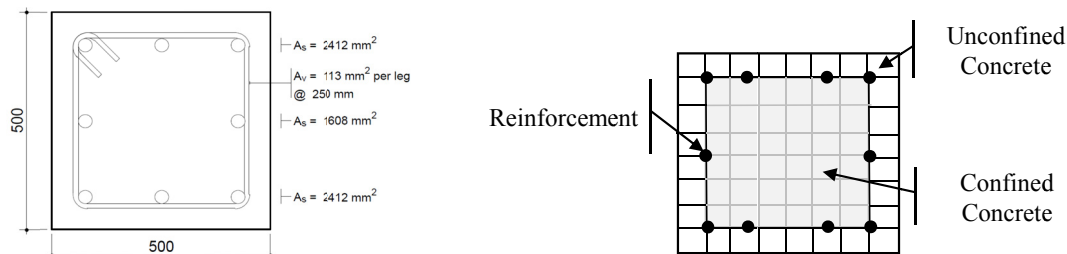


Figure 3. Pile section properties. (a) Pile cross-section, (b) Fiber section discretisation

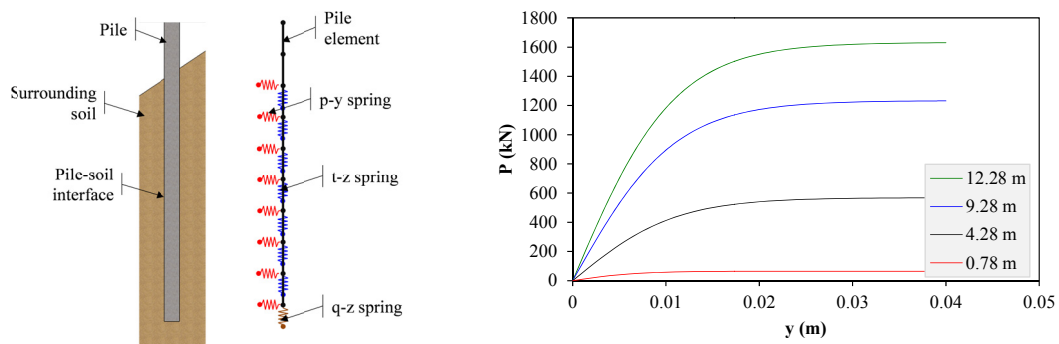
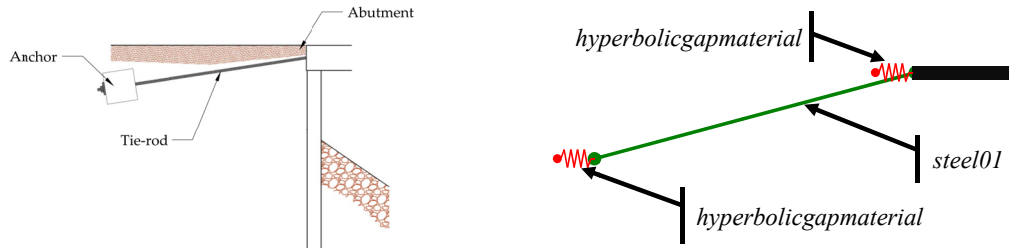


Figure 4. Soil-pile interface modelling. (a) Schematic of soil-pile system, (b) Representative p-y curves at increasing depth



**Figure 5. Tie-back mechanism. (a) Schematic of tie-back system, (b) schematic of numerical model**

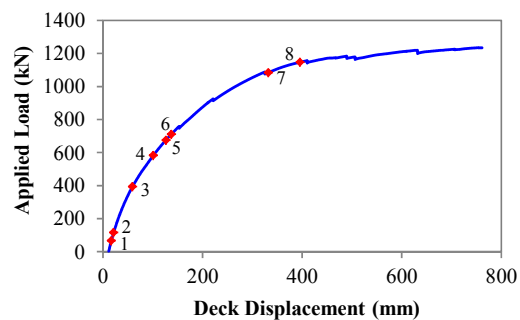
#### 4 FRAGILITY MODELLING

A fragility curve describes the probability that the structure reaches a prescribed damage state conditioned on a seismic intensity measure representative of the seismic loading. These curves are a key component in predicting the damage of a structural system when subjected to an earthquake. In this study peak ground acceleration (PGA) was used to represent seismic intensity and deck displacement as the fundamental response parameter. The use of other intensity and response parameters will be examined in further studies. The fragility modelling process consisted of the following steps:

1. Determine a relationship between basic engineering demand parameters (EDP) and deck displacement for each wharf model.
2. Subject each model to a suite of ground motions scaled to a range of PGAs.
3. Determine bound limits for each damage state.
4. Determine the probability of each damage state at each PGA level.
5. Fit the model response data to a lognormal cumulative distribution function.

##### 4.1 Deck displacement and EDP relationship

A pushover analysis was conducted for each wharf model to determine a relationship between the deck displacement at stresses and strains in the pile sections. For each analysis, stresses and strains in the extreme concrete and reinforcement fiber were recorded to determine the onset of cracking, yield and ultimate capacity. The pushover analysis highlighted the effect of increasing unsupported pile length from landward to seaward piles. The resulting decrease in pile stiffness results in the most landward pile attracting the highest stresses and strains, thus the earliest onset of damage assuming all pile-sections are the same. In Figure 6, an example output from the pushover analysis is shown indicating the locations along the forced-displacement curve in which the concrete and reinforcement first reach cracking, yield and ultimate capacity. A distinction was made between a pile section reaching a limit below or above the ground. This distinction was to account for the difference in which each type of damage will be detected and remedied.



1. Concrete cover crack above ground
2. Concrete cover crack below ground
3. Reinforcement yield above ground
4. Concrete yield above ground
5. Concrete ultimate strain above ground
6. Reinforcement yield below ground
7. Concrete yield below ground
8. Concrete ultimate strain below ground

**Figure 6. Force-displacement curve for Model 1 and the associated limit points**

## 4.2 Ground Motions

Each wharf model was subjected to two suites of ground motions consisting of 14 motions extracted from the suites of motions recommended by Oyarzo-Vera et al. (2012) for time-history analysis for structures in the North Island. The recommended ground motions were developed based on a seismic zonation study that divided the North Island into 5 zones with similar seismic hazard. For the purpose of this study 14 ground motions (7 earthquake events using both horizontal motions) were used to analyse wharves assumed to be located in the Auckland region (Zone A) and the other 14 used to analyse wharves assumed to be located in the Wellington region (Zone NF). Table 3 shows the motions chosen for each suite. Each ground motion was scaled to a range of PGAs from 0.2 to 0.9g. Each model was therefore subjected to 224 simulations resulting in a total of 1344 simulations. The NeSI Pan Cluster (High Performance Computer) at the University of Auckland was utilised to complete these simulations.

**Table 3. Earthquake events used for the time-history analysis**

Zone A	Zone NF
El Centro, Imperial Valley, USA	El Centro, Imperial Valley, USA
Delta, Imperial Valley, USA	El Centro #6, Imperial Valley
Chihuahua, Victoria, Mexico	Caleta de Campos, Mexico
Corinthos, Greece	Yarimka YPT, Kocaeli, Turkey
Kalamata, Greece	TCU 051, Chi-Chi, Taiwan
Westmorland, Superstition Hill, USA	Arcelik, Kocaeli, Turkey
CHY101, Chi-Chi, Taiwan	La Union, Mexico

## 4.3 Damage States

The International Navigation Association (PIANC) proposed qualitative criteria for judging the degree of damage to a pile-supported wharf based on the peak responses of the piles. Four damage states were determined corresponding to serviceable, repairable, near collapse and collapse levels of a wharf structure. In contrast, RiskScape, a multi-hazard loss assessment tool developed for New Zealand (King & Bell, 2006) defines 5 damage states as shown in Table 4. In this study the RiskScape damage state definitions have been adopted. The insignificant damage state was defined as the point at which the concrete cover below ground first cracks. Damage prior to this point will usually be very minor. The light damage state was defined as the point at which the reinforcement in the pile first yields above ground. Prior to this point the structure will not have suffered any structural damage with only some concrete cover cracking. The limit of the moderate damage state was defined as the point in which the reinforcement first yields below ground. Yielding below ground is significantly harder to detect and remedy, therefore this sort of damage should be considered severe. The limit for the severe damage state was defined as the point in which the concrete reaches its ultimate strain indicating crushing of the concrete and loss of structural integrity. Any model subjected to a greater demand was assumed to have reached the collapse damage state. It is important to note that actual collapse of a structure will be dependent on the overall redundancy of the system however for the purpose of this study the point at which one pile reaches the ultimate concrete capacity was assumed to be the start of the collapse damage state.

## 4.4 Probability Analysis

The response of the models was assumed to be a lognormal distribution with a probability density function as follows:

$$f_X(x) = \frac{1}{\sqrt{2\pi}\zeta x} \exp\left[-\frac{1}{2}\left(\frac{\ln(x)-\lambda}{\zeta}\right)^2\right] \quad (1)$$

where  $\zeta$  and  $\lambda$  are the two parameters of the lognormal distribution of the random displacement

variable  $X$ . Accordingly the fragility curve for each damage state is the conditional probability that the wharf has a state of damage exceeding the damage state,  $s_i$  at a specific PGA level, as shown below:

$$P[S > s_i | PGA] = P[X > x_i | PGA] = 1 - \Phi \left[ \frac{\ln(x_i - \lambda)}{c} \right] \quad (2)$$

where  $\Phi(\cdot)$  is the standard normal cumulative distribution function and  $x_i$  is the upper bound limit for  $s_i$ . The calculated probabilities at each PGA and for each damage state are then plotted to form a fragility curve. The curves are then further simplified by fitting them to a lognormal cumulative distribution function defined by two parameters calculated using regression analysis on the lognormal of the PGA values and the standard normal variable (Chiou et al., 2011).

**Table 4. Damage state definitions and associated limits**

Damage State	RiskScape	PIANC	Associated Limit
0 – Insignificant	No damage, or minor non-structural damage		2 – Concrete cover crack below ground
1 – Light	Non-structural damage only	Essentially elastic response with minor or no residual displacement	3 – Reinforcement yield above ground
2 – Moderate	Reparable structural damage	Controlled limited inelastic ductile response and residual deformation intending to keep the structure repairable	6 – Reinforcement yield below ground
3 – Severe	Irreparable structural damage	Ductile response near collapse (double plastic hinges may occur at one or limited number of piles)	8 – Concrete ultimate strain below ground
4 - Collapse	Structural integrity fails	Beyond the severe state	

#### 4.5 Fragility Curves

The final output of the fragility modelling process was a set of curves for each model, as shown in Figure 7. The plotted dots are the raw fragility probabilities and the dotted lines are the fitted lognormal cumulative distribution curves. There appears to be a satisfactory fit that becomes more varied with increasing earthquake intensity. In future research a shifted cumulative distribution function will be used to improve the fit to data.

The set of curves in Figure 7a show the fragility of a pile-supported wharf with a raked pile for lateral resistance, situated on soft clays and located in a high seismic region such as Wellington. In the event of a 0.6 PGA earthquake it is predicted that a similar wharf will be 100% likely to suffer some minor damage, 95% likely to suffer moderate damage, 85% likely to suffer severe damage and 15% likely to suffer collapse. In Figure 7b, a set of fragility curves for a pile-supported wharf with a tie-back for lateral resistance also situated on soft clays is shown. In the event of a 0.6 PGA earthquake it is predicted that this type of wharf will have a 97%, 83%, 30% and 0.1% probability of suffering light, moderate, severe and collapse damage, respectively. It is clear that the for the same soil profile a tie-back structural configuration would be expected to undergo significantly less displacements and attract less damage in comparison to a raked-pile configuration.

The plots in Figure 8 show fragility curves for the light, moderate, severe and collapse damage states for all models in a high seismic region. For the light damage state, shown in Figure 8a, there is limited variability between the wharf models, except Model 4 (tie-back in soft clay soil profile) which has a significantly lower probability of reaching the light damage state. Model 1 (raked-pile in soft clay soil profile) has the highest probability of being susceptible to light damage. The variability decreases further for the moderate damage state, shown in Figure 8b. Model 4 still has the lowest probability of reaching the moderate damage state. In contrast, the probability of Model 1 reaching the moderate damage state is lower than other models. This trend indicates that Model 1 performs more favourably



in moderate sized earthquakes. The plot in Figure 8c indicates there is a wider variability in predicted damage for the severe damage state between different wharf models. Based on that figure, there is a general observation that models with tie-back configurations (Models 4-6) were performing better than raked-pile configurations (Models 1-3). This observation was more pronounced for the collapse damage state (shown in Figure 8d). Further analysis on the sensitivity of these results to input parameters needs to be completed before a more definitive conclusion could be reached.

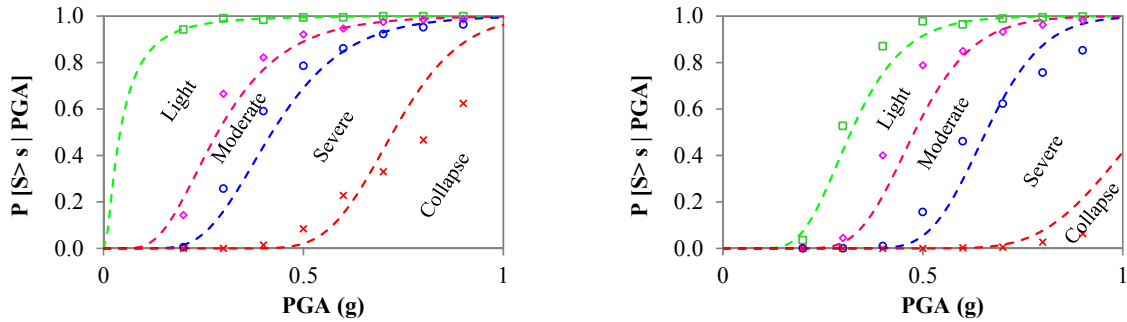


Figure 7. Fragility curves. (a) Model 1 – Raked pile, soft clay soil profile, (b) Model 4 – Tie-back, soft clay soil profile.

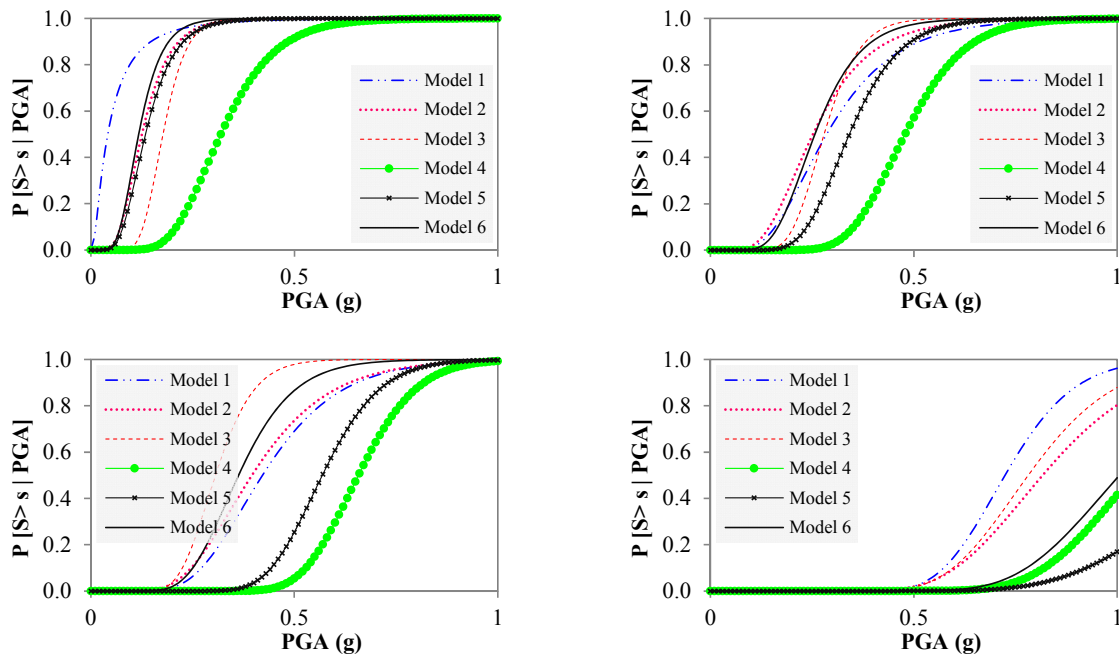


Figure 8. Fragility curves for each damage state. (a) Light, (b) Moderate, (c) Severe, (d) Collapse.

## 5 FUTURE RESEARCH

The next phase in this research is aimed at completing finite element numerical models to represent a two-dimensional soil slope based on the soil profiles developed thus far. Preliminary models have been developed and aimed at capturing site effects and slope displacements resulting from earthquake motions travelling through the soil profile. These soil slope models will be subjected to a set of ground motions at the bottom boundary and the response time-history at the location of foundation elements will then be used as input into the Soil Foundation Structure Interaction (SFSI) models. Fragility curves, similar to the ones developed in this study, will be developed and incorporated into RiskScape.

## 6 CONCLUSIONS

In this study two wharf configurations and three soil profiles were developed to represent typical New Zealand pile-supported wharves. The numerical modelling procedures used to model the wharves with realistic representation of different components were then presented. Fragility curves were developed

to predict the probability of a model reaching a defined damage state given a PGA. The damage states used in the study were adopted from the qualitative definitions provided by RiskScape. For low intensity earthquakes there was limited variability in performance between the different wharf models. However as earthquake intensity increased there was a pronounced difference between models with a raked-pile configuration and ones with a tie-back configuration, with the tie-back configuration having lower probabilities of damage. There appears to be no clear pattern with regards to the raked-pile configuration. Further sensitivity analysis on the numerical models needs to be completed to quantify the uncertainty in the results presented in this study.

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