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Investigation of a Numerical Approach for Assessing Liquefaction and its Effects on Telecom Systems

S. Bertelli¹, S. Lopez-Querol, T. Rossetto*Department of Civil, Environmental and Geomatic Engineering, University College London, London, UK*

R. Ruiter

Chorus, Christchurch, New Zealand

S. Giovinazzi

Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, NZ

ABSTRACT

Soil liquefaction has caused substantial infrastructure damage in recent earthquakes. During the 2010-2011 Canterbury Earthquake Sequence (CES), in New Zealand, liquefaction-induced damage to buried cables resulted in service interruption of the telecommunication network. This paper is part of a broader study on the seismic risk of buried infrastructure. It aims to compare numerical and empirical prediction methodologies with the observation maps produced in the aftermath of the Christchurch event. Starting from a description of the New Zealand telecommunication infrastructure and pipelines, the research first explores the vast amount of data and in-situ geotechnical inspections collected after the CES. These data are employed to test several liquefaction-triggering models available in the literature and results are provided through an exploratory spatial analysis. Then, a numerical simulation of a soil profile with and without pipelines from the suburb of Avondale, which was one of the locations most impacted by liquefaction damages, is carried out adopting the Byrne's formulation for the classic Martin and Finn's constitutive model in a full dynamic analysis in FLAC-2D. The obtained results from the numerical model are finally cross-checked with the empirical analyses, the existing liquefaction investigation maps, and field observations collected in the aftermath of the event.

Keywords: telecommunications, liquefaction, numerical analyses, buried cables.

1. INTRODUCTION

Telecommunications are one of the essential services provided within the utility systems. When an issue arises, or in the event of a full-blown outage, they can cause much disruption. Their operational robustness becomes even more critical in a post-disaster scenario when these services are adopted for civil protection and emergency plans, as well as the restoration of other critical infrastructures.

Despite the relevance of loss of functionality of telecommunication networks, few attempts of risk assessment exist in the earthquake engineering community (e.g., Leelardcharoen *et al.*, 2011; Esposito *et al.*, 2018) compared to other utility distribution systems such as the electricity, water, and gas (i.e. Kongar *et al.*, 2017;

¹ Corresponding Author: S. Bertelli, *University College London, silvia.bertelli@ucl.ac.uk*

Liu *et al.*, 2015; Esposito *et al.*, 2015). These studies aim to broaden the methodology initially developed for buildings, or point-like structures, to spatially distributed networks. In this regard, several steps are necessary ranging from a classification of the 'system' - stock, description of damages and performances, a definition of appropriate hazard parameters to the generation of hazard scenarios and estimation of functional impacts with correlated direct economic losses (Giovinazzi *et al.*, 2014). Nonetheless, each of these steps presents scientific and practical challenges when dealing with distributed infrastructures systems.

In particular, a critical issue in the seismic risk assessment for spatially distributed networks is the definition of hazard parameters to evaluate the damage on each element. Since the telecom infrastructure is based mainly on buried cables, this system is exposed to a high likelihood of sustaining physical damage caused by both seismic wave propagations and permanent ground deformations (PGD). Indeed, as the network covers a large geographical area, telecommunication cables and pipelines can be subjected to a variety of geotechnical hazards; irreversible ground surface settlements, landslides, liquefaction and associated lateral spreading could permanently buckle pipes, and thus endanger the functionality of the entire system (O'Rourke *et al.*, 1999).

The current paper is part of a broader research project which attempts to analyse the seismic hazard on telecommunication networks. Specifically, this study seeks to investigate a well-known state-of-the-art numerical approach to assess the liquefaction-induced damage on telecommunication piping which could potentially be applied for urban scale analysis. The results are then compared with state-of-practice liquefaction triggering models and observations. Both empirical and numerical methods are adopted, with geotechnical data gathered from the New Zealand Geotechnical Database (NZGD), an online repository of information developed after the 2010-2011 Canterbury Earthquake Sequence (CES).

In the following, Christchurch and its Mw=6.2 earthquake, the event within the CES which caused the most severe and widespread damage to the telecommunication system, are taken as key-study. In particular, the study focusses on the suburb of Avondale which was deeply damaged and declared by the Government as residential 'red zone' due to soil liquefaction. The remaining part of the paper illustrates the data and methodology adopted for the analyses, including a brief description of the New Zealand telecommunication network and liquefaction observations collected after the event. The paper concludes with a summary of the results obtained from several empirical prediction models and comparison with numerical simulations carried out using the Bryne's formulation for the Martin and Finn's constitutive model implemented in FLAC-2D (Itasca Consulting Group, 2011) of an Avondale's soil profile with and without a telecommunication pipe.

2. DATA

This section gives a brief overview of the data used for the subsequent analyses. First, a concise explanation of the structure of a telecommunication network and technical specification of its pipelines are provided. This is followed by a report about the 2011 Christchurch event with an indication of where ground motion data are gathered. Then, it continues with a review of the liquefaction susceptibility of the area, and observations' data collected after the event. The section concludes with a description of geotechnical investigations available and specifications about the chosen Avondale site.

2.1 Telecom infrastructure and pipeline details

As far as the telecommunication infrastructure is concerned, networks can be classified into wireless service or landline and broadband data service (Giovinazzi *et al.*, 2017). These two networks are interconnected by a complex hierarchical structure, which can be schematised as point-like components (e.g. major and local exchanges, roadside cabinets, access pits, poles, and cellular towers) joined together by distributed links (i.e. buried or overhead lines of either fibre optics or copper cables – Esposito *et al.*, 2018).

In the particular case of New Zealand, the distributed elements are generally buried fibre optics. The copper cables are still in place but have been progressively replaced due to the "New Zealand government ultra-fast fibre broadband network programme" (Giovinazzi *et al.*, 2017). However, as the system covers a large geographical area and cables tend to be buried, this network is more sensitive to permanent ground deformations induced by liquefaction and lateral spreading (Esposito *et al.*, 2018).

Regarding the distributed networks installation, several construction techniques and piping materials have been employed over the years. Cables are usually pulled into mini-ducts which are in turn wrapped by another protective tube. Successively, this pipe can be organised into formations of multiple pipes running together (i.e. groups of 4, 6, 8 or even more), which are laid into trenches and covered with backfill material or directly placed into the ground with a no-dig procedure. Material and size of the pipes may vary as well, ranging from earthenware, asbestos through the newer (1970's and later) plastic ducts. In addition to the pipe material, several other design factors may have a repercussion on pipe damage during earthquakes, pipe diameter, year laid, pipe type (e.g., primary or trunk), depth laid, and trench backfill type. Thus, for the current case-study, a single High-Density Polyethylene (HDPE) pipe is assumed with a diameter of either of 10 cm or 15 cm, which are explicitly designed for wrapping mini-ducts of fibre-optic cables and adopted for no-dig excavations (see Table 1 for technical specifications).

Table 1. Telecommunication ducts: HDPE pipe material properties and dimensions.

Pipe Properties		HDPE -10 cm	HDPE -15 cm	
<i>Geometrical Configuration</i>	Diameter	<i>mm</i>	100	150
	Area	<i>m²</i>	9.142e-4	1.385e-3
	Moment of Inertia	<i>m⁴</i>	1.076e-6	3.744e-6
	Thickness	<i>mm</i>	3	3
	Length	<i>m</i>	300	300
<i>Material properties</i>	Density	<i>Kg/m³</i>	950- 965	950-965
	Tensile Modulus (short term) E_s	<i>MPa</i>	1000	1000
	Tensile Modulus (long term) E_l	<i>MPa</i>	160	160
	Tensile stress at yield	<i>N/mm²</i>	21	21
	Tensile strain at yield	<i>N/mm²</i>	12	12
	Poisson's Ratio ν	-	0.39	0.39

2.2 The Christchurch event and ground motion information

Among the CES, the 22nd February Christchurch Event had the most significant impact on the performance of the telecommunication infrastructure due to liquefaction manifestations and associated ground deformations (Giovinazzi *et al.*, 2011, Fenwick *et al.*, 2012). The network performed relatively well compared to other infrastructure lifelines, but there were lots of cable faults especially in the liquefied areas (Tang *et al.*, 2014). For instance, the Telecom NZ investigations report (2011) describes utility holes partially floated out of the ground or filled with water in areas where there was severe liquefaction.

The Mw=6.2 earthquake itself was induced by a strike-slip rupture on a formerly unrecognised fault and was centred 10 km to southeast from the city centre at 5-6 km depth. Due to its shallow depth and proximity to the city centre, very high ground motions were registered by the 33 strong motion stations placed around Christchurch City Council area. As indicated by the recordings obtained from PEER Ground motion Database (Ancheta *et al.*, 2013), the highest PGA recorded was 1.41g at Heathcote Valley Primary School and 0.22g at Hulverstone Drive Pumping Station (-43.5015, 172.021), the closest recording station from Avondale.

Besides the recording stations' data (Ancheta *et al.*, 2013), an observed maximum horizontal PGA isoseismal map is available on the NZGD (2015). This map has been developed by combining the prediction from an empirical ground motion model of the fault rupture with the PGA recorded at any adjacent strong motion stations, following the Bradley (2013) procedure. In particular, it shows isoseismal of median PGA at 0.01g interval and corresponding standard deviation bands for the entire city council area. Thus, it can be adopted to simplify the estimation of the liquefaction potential at each site.

2.3 Liquefaction susceptibility and observations

Liquefaction is a long-established risk for Christchurch due to its geomorphological setting (Brackley, 2012). Being placed on the Pacific coast and among three rivers, the city is characterised by a shallow GWT (0-2 m in the eastern suburbs and 2-3 m in the western suburbs) and loose, poorly consolidated, alluvial deposit ground, which results in very high liquefaction susceptibilities (Maurer *et al.*, 2014). Before the CES, localised liquefaction had been observed at the estuary of the Avon and Heathcote rivers in 1869 and coastal areas from Kaiapoi northwards during the Cheviot earthquake in 1901 and Motunau earthquake in 1922 (Brackley, 2012).

Since the 1990s, soil profile characterisation studies have been carried out to assess the liquefaction susceptibility of the council area. Several maps have been produced based on a combination of geology, topology, hydrology and geotechnical investigation data (e.g. Elder *et al.*, 1991; Brown and Weeber, 1992, Beca, 2002-2005, Beca, 2012 in Brackley, 2012). The most up-to-date map (Brackley, 2012), which adopts a “high”, “moderate”, “low”, and “none” liquefaction risk classification, clearly delineates how all the Canterbury plain underlies potentially liquefiable geological materials.

After the Christchurch Earthquake, unprecedented levels of liquefaction were surveyed across a wide area in the suburbs north to south of the city, and northeast along the River Avon as shown in NZGD (2012, 2013). The observed lateral spreading, sand boils, settlements, silt mud ejections and water ponding on the ground surface are consistent with the geology of the area as mentioned earlier (Giovinazzi *et al.*, 2011).

For the current research, two primary sources of liquefaction observations data are available: a regional-scale map displaying the extent of ejected liquefied material interpreted from aerial photography (NZGD, 2012); and property and road observation severity maps developed through on-foot surveys (NZGD, 2013). This second source provides more reliable information, as it classified the observations as none, minor, moderate, severe, moderate-to-severe, very severe based on the evidence and quantity of ejected material as well as the lateral displacement which was visible at the surface. Thus, given the accuracy of site inspections, the NZGD (2013) maps are taken as reference.

2.4 Geotechnical information for model development

The primary method of geotechnical subsurface investigation in Christchurch is the CPT. The NZGD makes publically available more than 30,000 CPT tests performed in the aftermath of the CES for insurance, research and new developments purposes. Through this dataset, 58 high-quality records located across the entire municipal territory are selected for comparison of different liquefaction potential methodologies. These CPT soundings are chosen based on the assessment of their location as susceptible to liquefaction, termination depth over 10 m, proximity to ground motions recording stations, and availability of piezometer readings.

Laboratory tests can provide further insights into the selected sites. For six of these locations, experimental tests have been carried out and reported by Beyzaei *et al.* (2018). Amongst them, a soil column corresponding to the Avondale EQC-4 Site (-43.5014, 172.6857) is chosen for investigating the telecommunications’ pipe performance through additional numerical analyses. Before the CES, this site was occupied by a single-family residential building, and so it can be assumed as a level-ground free-field ground. This location has the potential for both liquefaction and lateral spreading, being placed roughly 80 m from the Avon River. In the 10 m below ground surface, the column consists of a silty sand layer extending from 0-3 meters underlain by cleaner sands (Beyzaei *et al.*, 2018). After the Christchurch event, it was subject to moderate to severe liquefaction manifestations including lateral spreading and ejecta (NZGD, 2013).

3 LIQUEFACTION EVALUATION METHODOLOGY

The next sections describe the procedures and methods used in this investigation. In particular, the first one summarises the state-of-practice semi-empirical approach adopted for assessing the potential of liquefaction manifestations in Christchurch, extending the work presented in Bertelli *et al.* (2019). The second and last paragraph illustrates the numerical simulations’ methodology carried out with FLAC-2D (Itasca Consulting Group, 2011), in which the sandy soil is modelled with Byrne (1991) formulation on a soil profile from the suburb of Avondale.

3.1 Semi-empirical models for the liquefaction triggering assessment

Advancement of the simplified procedure initially proposed by Seed & Idriss (1971) is herein considered to estimate the correspondent Factor of Safety (FS) against liquefaction for all the selected CPT soundings as described in Bertelli *et al.* (2019). Table 2 reports the seven different liquefaction prediction models adopted, based on the selected relationships reported by Youd & Idriss (2001), Moss *et al.* (2006), Idriss & Boulanger (2008), Boulanger & Idriss (2014). For the application of these relationships, soil unit weights are presumed to be 17 kN/m³ above the GWT, and 19.5 kN/m³ below the GWT as suggested by Wotherspoon *et al.* (2014). The PGA at each CPT site is extrapolated from the PGA isoseismal map (NZGD, 2015). For the estimation of Liquefaction Potential Index (LPI) values, layers are considered to be potentially liquefiable if the soil behaviour type index (I_c) is less than 2.6 (Robertson & Wride, 1998).

Table 2. Liquefaction prediction model tested, with the reduction coefficients (r_d) and magnitude scaling factors (MSF) combination adopted (Youd & Idriss, 2001; Moss *et al.*, 2006, Idriss & Boulanger, 2008, Boulanger & Idriss, 2014).

Model ID	Model	r_d	MSF
YEA01a	Youd & Idriss (2001)	Blake (1996, cited in Youd & Idriss, 2001)	Idriss (1995, cited in Youd & Idriss, 2001)
YEA01b	Youd & Idriss (2001)	Blake (1996, cited in Youd & Idriss, 2001)	Andrus & Stokoe (1997, cited in Youd & Idriss, 2001)
YEA01c	Youd & Idriss (2001)	Idriss (1999, cited in Youd & Idriss, 2001)	Idriss (1995 cited in Youd & Idriss, 2001))
YEA01d	Youd & Idriss (2001)	Idriss (1999, cited in Youd & Idriss, 2001)	Andrus & Stokoe (1997 cited in Youd & Idriss, 2001)
MEA06	Moss <i>et al.</i> (2006)	-	-
I&B08	Idriss & Boulanger (2008)	-	-
B&I14	Boulanger & Idriss (2014)	-	-

To assess the efficacy of the selected liquefaction prediction models, LPI values are estimated at each CPT site location according to the Iwasaki *et al.* (1978) methodology. A correlation is then established between the calculated LPI values and the observed liquefaction manifestations reported in the observation maps previously mentioned. In particular, the prediction of liquefaction occurrence is reduce to a binary system according to the Iwasaki Criterion (i.e. if $LPI \geq 5$, liquefaction manifestations are expected at the investigated site). Likewise, the observations maps are reinterpreted by classifying each site as “No Liquefaction” or “Liquefaction”; “none” and “marginal” classes are mapped as negative results of occurrence, whereas the other classes as positive. Thus, each observation-calculated combination case is arranged according to a confusion matrix approach as True-Positive (TP), True-Negatives (TN), False-Positive (FP), and False-Negative (FN), as represented in Table 3.

Table 3. Confusion Matrix: comparison between liquefaction observations and liquefaction predictions

		Liquefaction predictions	
		NO	YES
Liquefaction observations	NO	True-Negative (TN)	False-Positive (FP)
	YES	False-Negative (FN)	True-Positive (TP)

Based on the confusion matrix classification, exploratory spatial analysis is carried out to evaluate the overall LPI performance of the tested methods, and several statistical performance parameters are examined to assess the quality of each semi-empirical method as described in Bertelli *et al.* (2019).

3.2 Numerical simulation

To further assess the implication of liquefaction on the performance of the telecommunication networks, a numerical simulation of a soil column representing Avondale EQC-4 location is carried out utilising the

commercial platform FLAC-2D (Itasca Consulting Group, 2011). This program is chosen as it is a widely popular numerical analysis tool for modelling soil liquefaction as well as for its simplicity for performing full non-linear dynamic analysis. Indeed, the software has built-in the FINN model-Byrne (1991) formulation, which has been implemented by Itasca as described in FLAC User's Manual (Itasca Consulting Group, 2011b). This model includes the relationship between irrecoverable volume change and the cyclic shear-strain amplitude into the Mohr-Coulomb model and takes into account the buildup of the pore water pressure. It has been extensively applied in recent researches as for example those reported by Tang & Orense (2014), Vargas *et al.* (2015), and Beaty & Perlea (2011). As illustrated in the aforementioned literature, the strength of this model is the relatively restricted parameters needed for its calibration in comparison to more advanced constitutive models, which makes it potentially suitable for regional-scale analysis.

For the current research, three different models are designed; a first model represents just the 7 m deep and 1 m width soil column, whereas the second and third models integrate into the soil column a single 100 mm HDPE or 150 mm HDPE pipe, respectively. Based on the experimental information provided by Beyzaei *et al.* (2018), all the soil column is assumed to be liquefiable sandy material, and the GWT is presumed to be immediately above the pipe (0.95 m). Free field boundary conditions are also implemented in the models. Regarding the geometry of the problem, the 7 m soil column is modelled as a 20 x 160 mesh composed by 3200 rectangular elements, and with an additional meter at the bottom to avoid localised instabilities of the software where the ground motion is applied. The HDPE pipe itself is configured creating a circular cross-section of 'liners' as structural elements 'glued' to the surrounding soil at 1 m depth and 0.5 m width, considering the technical specifications in Table 1. Control points are added at depth intervals of 0.7m as reported in Figure 1.

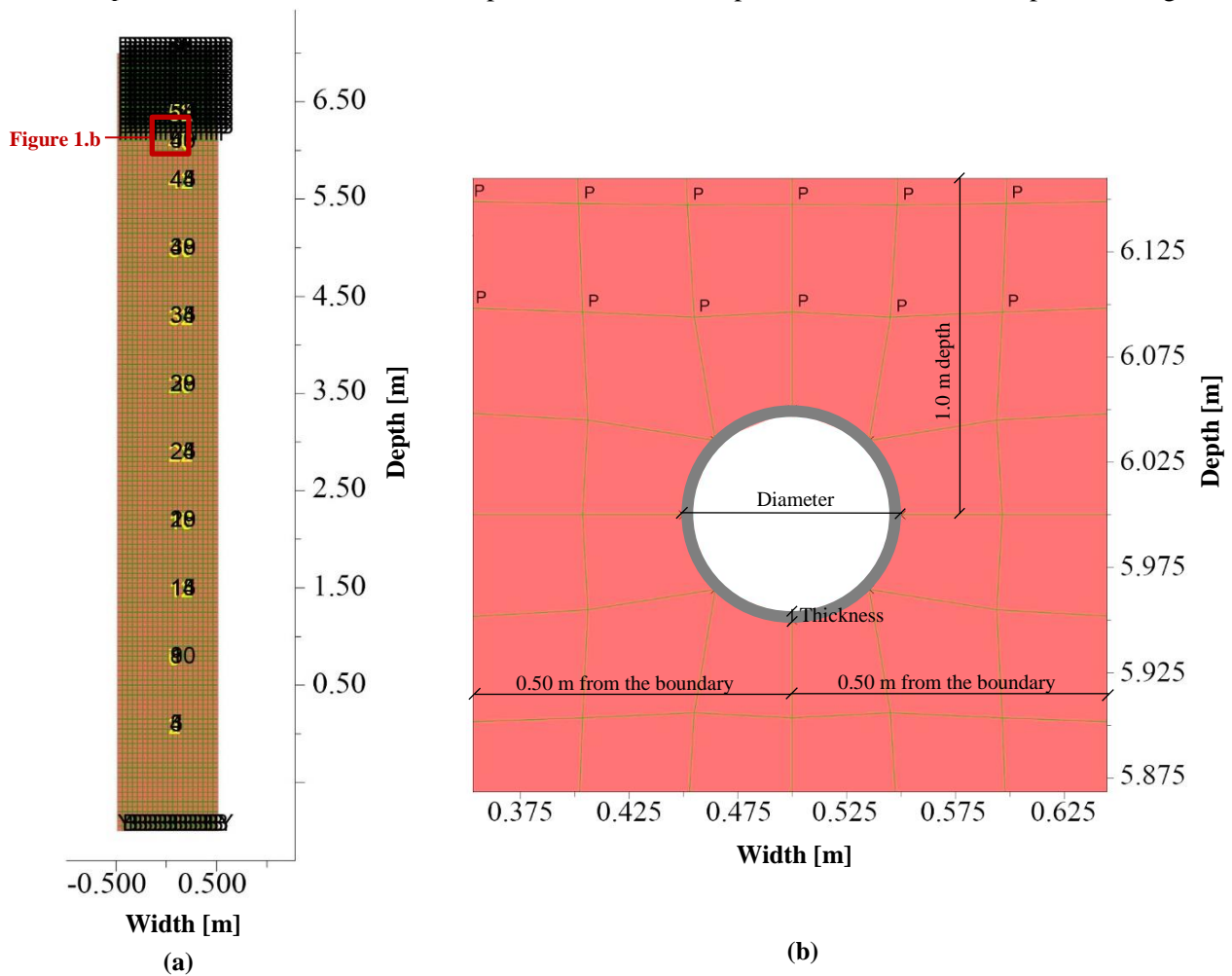


Figure 1. a) FLAC-2D model of the entire soil column profile with control points; b) Detail of the soil column model at 1m depth and 0.50 m width from the boundary to capture the geometry of the HDPE pipe. The symbol 'P' at the upper nodes indicates that these nodes have been prescribed to have null water pressure.

After creating the geometry, a two-steps calculation is considered. Firstly, a static equilibrium calculation for the site which includes the steady-state groundwater conditions. Then, the dynamic analyses applying as input loading at the base of the model the history of Peak Ground Velocities (PGV) recorded at the at Hulverstone Drive Pumping Station, the closer recording station from the Avondale EQC-4 site according to PEER database (Ancheta *et al.*, 2013). As previously mentioned, the nonlinear stress-strain behaviour at the first stage of the calculation is represented by an elasto-plastic Mohr-Coulomb model, whereas the liquefiable material is simulated according to the formulation proposed by Byrne (1991). In particular, the FINN model-Byrne formulation in FLAC-2D (Itasca Consulting Group, 2011) involves the setting up of two constant C_1 and C_2 following the expressions:

$$C_1 = 8.7(N_1)_{60}^{-1.25}$$

and:

$$C_2 = 0.4/C_1$$

where $(N_1)_{60}$ is the normalised standard penetration test value which is set to correspond to SPT measurements. Therefore, the main parameters adopted for this sandy material are $E=31\text{MPa}$, $\nu=0.2$ a $\phi'=37\text{degree}$, and $C_1=0.30$, $C_2=1.36$, which are estimating assuming a $(N_1)_{60} = 8.5$ trough an evaluation of an SPT sounding effectuuated before the CES in the proximity of the EQC-4 site and reported in the NZGD.

4. RESULTS AND DISCUSSION

The section delineates the results of both the semi-empirical methods and numerical analyses. A comparison of the previously mentioned methodologies is presented from a geographical point of view and discussed, with considerations on the adequacy of these models for assessing the potential of liquefaction manifestations in Christchurch. Then, the numerical simulation of the simplified soil profiles from Avondale subjected to the Christchurch earthquake, with and without a telecommunication pipe, are compared to the semi-empirical analyses and site observations.

4.1 Spatial analysis and test diagnosis results of the liquefaction prediction models

As can be seen in Figure 2, the exploratory spatial analysis results in a general over-prediction of the semi-empirical models. The pie-charts adopted for symbolising the cumulative results from the seven different methodologies at each CPT-location are predominantly “yellow” (False-positive) in the westerns suburbs of Christchurch. This inconsistency between the predictions and observations is might due to the geomorphological features of this area. The increasing mix of sand, silt and gravel in these soil profiles would have misled the calculation of I_c factors, which resulted in higher LPI values and lead to an overprediction of liquefaction manifestations.

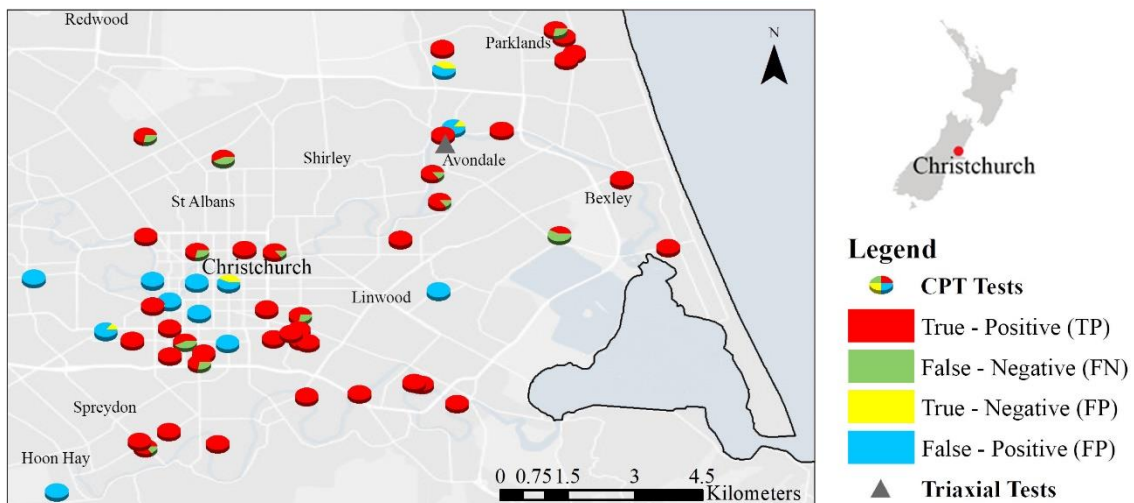


Figure 2. Comparison of observation liquefaction data with 7 LPI prediction models: True-Positive (TP), True-Negative (TN), False-Positive (FP), False-Negative (FN).

The model which seems to perform better in this particular case is the Idriss & Boulanger (2008) as it is the one with the highest accuracy rate. By way of illustration, the LPI forecast for Avondale EQC-4 site is a severe liquefaction prediction which corresponds to the manifestation observed in the aftermath of the Christchurch event. However, the MCC (Matthews Correlation Coefficient) calculated from the Idriss & Boulanger (2008) method's LPI values is close to zero, indicating that the model cannot adequately predict the non-liquefaction occurrence and the correlation is rather casual. Similar parameters are indeed observed among all the selected methodologies, confirming the overall trend of the semi-empirical models to overpredict liquefaction manifestations.

4.2 Results of the numerical simulation of the column profile

The numerical simulations yield evident liquefaction in Avondale EQC-4 site, in agreement with the observations reported after the Christchurch event. The liquefaction occurs mainly between 1-3.5 m depth as shown in the isochrone plots reported in Figure 3.a, 3.b and 3.c, respectively, for the soil column without, with the 10 cm, and 15 cm HDPE pipe. In these figures, liquefaction is predicted when the increment of pore water pressure touches the line representing the initial effective stress. Interestingly, liquefaction is predicted in a broader range of depths in the profile without the pipeline as the pore pressure reaches the initial effective stress. The presence of the pipe results in lower pore pressure values for all the different periods, as if pipelines work as a reinforcement of the soil, making it stronger against liquefaction.

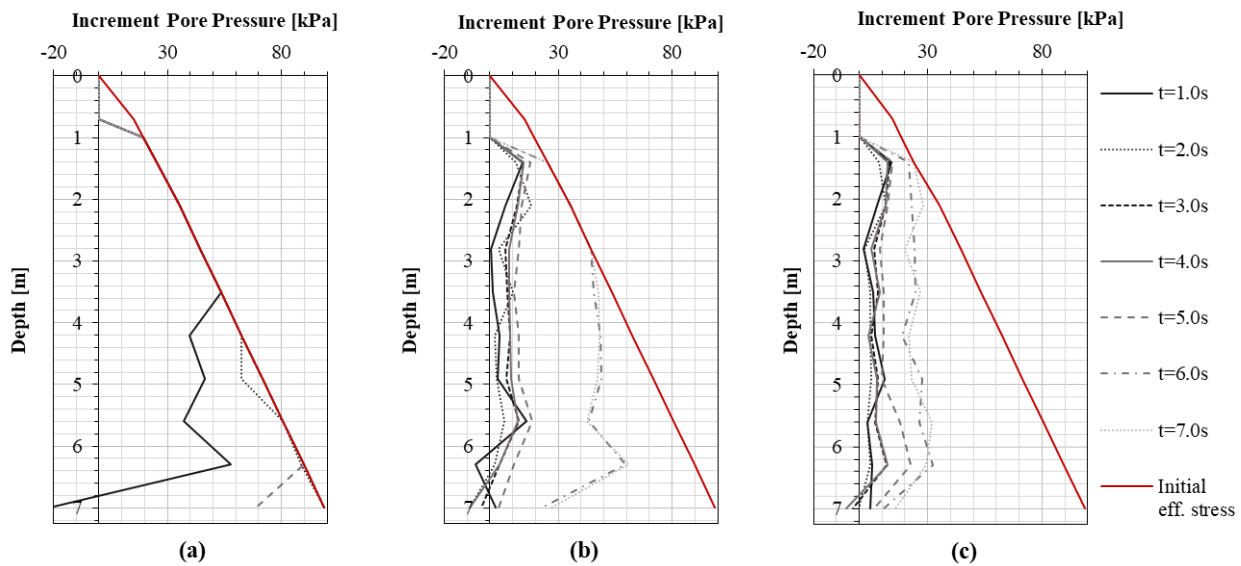


Figure 3. Isochrone Plots: a) Soil column b) soil column with the 10 cm pipe c) soil column with the 15 cm pipe.

A comparison between the isochrone plots indicates that liquefaction manifestations occur almost immediately in the soil column without pipes. In particular, it happens up to a roughly 3.5 m depth in the first second of the event, and it broadens to the entire profile for the following periods. Instead, liquefaction is predicted starting from 6 seconds and between 1.5-3 m depth for the 10cm pipe, and it is almost null for the 15 cm pipe whose isochrones of 6.00s and 7.00s just touch the effective stress at 1.4 m depth. This effect would be somewhat different for smaller cross-sections of pipelines and with different backfill materials. The correspondent representations of PGD are not reported as they cannot be considered a good approximation. The maximum value reached is roughly 0.3 mm and is in contrast with the observations of lateral spreading at the site after the event. This inconsistency might be due to inaccuracy in the numerical simulation once liquefaction has taken place, due to high strain level which causes numerical instabilities. Hence, these results need to be instead interpreted qualitatively.

Therefore, the results of these numerical simulations have significant implications for the development of seismic risk assessment of buried pipelines. Comparing with the semi-empirical methods, the numerical models offer more reliable predictions of liquefaction as demonstrated by these findings which estimate correctly both where and when liquefaction might have happened. However, models like the one employed in this research do not offer an accurate prediction of PGD which is one of the leading cause of damage to buried pipelines. Lagrangian meshfree models seem to be a more suitable option for this kind of problems. In addition,

these numerical tools require more detailed site-specific geotechnical data and laboratory tests to be calibrated and as such are less appropriate for regional-scale analysis (López-Querol & Blázquez, 2006). Nonetheless, the present research also raises the possibility of modelling the interaction of the soil around the buried pipeline. Much research is still needed to understand the effect of liquefaction on buried pipelines, but this numerical model might offer a simple solution.

5. CONCLUSIONS

As a first step into the seismic risk assessment of the telecommunication networks, this paper has tried to investigate a numerical approach for examining the liquefaction-induced damage on buried telecommunication piping. Firstly, adopting in-situ geotechnical tests and liquefaction observation data collected in the aftermath of the 2011 Christchurch earthquake, seven different semi-empirical methodologies which constitute the state-of-practice for assessing liquefaction manifestations have been applied to several locations spread around the council area. Generally, outcomes show an over-prediction of liquefaction occurrence one among the selected approaches as the results they provide are non-dissimilar. Even though their ability to predict liquefaction occurrence is quite poor, the LPI method proposed by Idriss & Boulanger (2008) is the best performing one, especially in the case of Avondale EQC-4. Regarding the numerical simulations, the findings show more precise estimations of the occurrence of liquefaction and the possibility to model the soil interaction with the buried pipeline.

Taken together, the semi-empirical models are fast and straightforward to apply, but the provided liquefaction occurrence predictions are rather casual and, hence, they would not be taken for granted for carrying out further vulnerability assessment studies. Instead, the numerical simulations like the ones presented in this research provide results in agreement with liquefaction observations but require more accurate site-specific geotechnical data, and they do not offer much more information regarding PGD measures. In addition, these analyses might result in being excessively time-consuming if the telecommunication network is modelled assuming a smaller mesh for the soil column characterisation, and, hence, less appropriate for regional-scale studies. Nonetheless, in future investigations, it might be worthwhile applying this the numerical approach for analysing different telecommunication pipeline's materials, sizes, depths and their interdependencies with electricity, water, gas infrastructure systems as well as exploring more advanced constitutive models available in the literature..

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