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LIQUEFACTION EFFECTS ON THE NORTHERN CORONEL PIER DURING THE 2010 MAULE CHILE EARTHQUAKE

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Abstract

During the February 27, 2010 M_w8.8 Maule earthquake, soil liquefaction occurred at many sites; often leading to ground failure and lateral spreading. As a consequence, a variety of structures were severely damaged. Of special relevance were the effects of liquefaction on waterfront structures, which adversely affected the operation of some of Chile's key port facilities. In this sense, lateral spreading and ground settlement damaged quay walls and pile-supported wharfs in older sections of both Valparaíso and San Antonio ports. Nevertheless, the most severe liquefaction-induced damage was found at Concepción and Coronel ports, causing important economic losses and downtimes to these facilities, and with direct costs associated to damage repairs of about US$ 300 million for the main ports in southern Chile. This paper focuses on the observed damage at the Northern Coronel Pier during the 2010 Maule Earthquake, which was strongly related to the soil characteristics at the port site. Available geotechnical information from both SPT and CPTu data, obtained before and after the earthquake, revealed the presence of medium dense to loose saturated sandy layers, at different depths, with a high liquefaction potential. In addition, low-to-negligible resistance mud sediments overlying the bedrock were identified, which could have worsen the effects of lateral spreading. The overall behavior for this case history is assessed using both a modified version of the simplified design procedure proposed in the MCEER/ATC-49-1 report and the design method for pile foundations in laterally spreading ground described in the 2011/04-PEER report. The residual lateral displacement is estimated by means of intersecting two curves: one that represents the lateral force-displacement curve of the structure stabilizing the sliding soils mass, and one that corresponds to the estimated residual displacements of the soil mass for different levels of restraining force. This approach gives permanent lateral displacements in the range of 50 to 120 cm, which is in good agreement with the observed behavior. From these results, design aspects such as the fixity conditions between pile heads and superstructure, the embedment depth of piles within the non-liquefied zone, and the number of piles along the longitudinal and transverse directions of the pier are discussed, and some preliminary conclusions and recommendations are elaborated.

Keywords: Liquefaction; Lateral spreading; Ports; Slope stability; Earthquakes.

1. Introduction

In Chile, the port industry plays a strategic role regarding importation and exportation of goods, by transferring more than 90% of the international trade as well as by connecting local economic activities with worldwide markets (CAMPORT, 2015). The most important commercial ports, Valparaíso and San Antonio, concentrate about 70% of the total load transported from and to Chile; whereas other relevant ports are Quintero, Huasco, Talcahuano, Concepción, and Coronel (MOP, 2005). For the next years, it is expected that the investment requirements for this sector will keep rising, which will imply significant development challenges for the country.

During the February 27, 2010 M_w8.8 Maule earthquake, soil liquefaction occurred at many sites; often leading to ground failure and lateral spreading. As a consequence, a variety of structures were severely damaged. Of special relevance were the effects of liquefaction on waterfront structures, which adversely affected the operation of some of Chile's key port facilities. In this sense, lateral spreading and ground settlement damaged quay walls and pile-supported wharfs in older sections of both Valparaíso and San Antonio ports. Nevertheless, the most severe liquefaction-induced damage was found at Concepción and Coronel ports, causing important economic losses and downtimes to these facilities (Bray et al., 2012). Direct costs in damage for the main ports in southern Chile have been estimated in slightly less than US$300 million (Brunet et al., 2012).

This paper presents the case history of the Northern Coronel Pier, investigated by the Geotechnical Extreme Events Reconnaissance (GEER) team after the 2010 M_w8.8 Maule earthquake, and it focuses on the observed damage due to liquefaction and lateral spreading at the site. The overall behavior is assessed using modified versions of both the simplified design procedure proposed in the MCEER/ATC-49-1 report and the design method for pile foundations in laterally spreading ground described in the 2011/04-PEER report (Ashford et al., 2011). The observations provided herein are based on the GEER edited report (Arduino et al., 2010), and on González & Verdugo (2012).

1.1 Liquefaction susceptibility and effects

Liquefaction susceptibility was evaluated at the pier site using Standard Penetration Test (SPT) and Piezocone Test (CPTu) results obtained before and after the earthquake. The sand liquefaction triggering procedure presented in Youd et al. (2001) was used to define the normalized SPT and CPT threshold values for the occurrence of liquefaction. The 2010 Maule earthquake had a moment magnitude of M_w=8.8, and the ground motion had a peak ground acceleration of PGA≈0.4g in areas close to Coronel (Boroschek et al., 2010).

From its part, effects of liquefaction-induced lateral spreading were evaluated based on both a modified version of the simplified design procedure proposed in the MCEER/ATC-49-1 report and the design recommendations for pile foundations in laterally spreading ground described in the 2011/04-PEER report (Ashford et al., 2011). Some of the main steps involved in these design procedures are:

• Identify the soil layers that are likely to liquefy. The information from available borings was selected and evaluated according to the sand liquefaction triggering procedure presented in Youd et al. (2001).

• Assign undrained residual strengths S_{ur} to the layers that liquefy. The post-liquefaction S_{ur} strength was evaluated using the expression recommended in Ledezma and Bray (2010).

• Perform pseudo-static seismic stability analysis to calculate the yield coefficient (k_y) for the critical potential sliding mass. For all the cases, the horizontal force F required to reach a factor of safety (FS) of 1.0 was calculated for horizontal accelerations k_h of 0.05, 0.10, 0.15, 0.20, 0.25, 0.30, and 0.35. Therefore, $k_y = k_h$ since FS=1.0.

• Estimate the maximum lateral ground displacement. The Bray and Travasarou relationship (Bray and Travasarou, 2007) was used to assess the lateral displacement associated with each critical horizontal acceleration. The initial fundamental period of the sliding mass (T_s) was calculated using the expression: T_s = 4H/V_s, where H = the average height of the potential sliding mass, and V_s is the average shear wave velocity of the sliding mass. Shear wave velocities were obtained using the V_s versus N-SPT correlations proposed by

Brandenberg et al. (2010). The spectral acceleration at a period equal to 1.5 times the initial fundamental period of the sliding mass, $S_a(1.5T_s)$ was estimated in about 1.0g. For this purpose, the elastic response spectra from a series of recorded motions from the Maule earthquake and the elastic design response spectra from the current Chilean building codes were used. A moment magnitude of M_w=8.8 was used in the analyses.

• Identify the plastic mechanism that is likely to develop as the ground displaces laterally. A simple elasto-plastic model is used to reproduce the pile behavior.

• Estimate the expected restraining forces from the pile foundation and pier superstructure for a range of possible liquefaction-induced displacement fields. To properly capture the restraining forces as the ground displacements increase, an "equivalent constant restraining force" from the piles and superstructure was obtained as the running average resistance that develops as the ground displacement increased from zero to its final value (Boulanger et al., 2006). A pseudo-static pile pushover analysis for each case was performed.

1.2. Considerations for Slope Stability and Pile Response Analyses

A series of slope stability analyses were performed using the computer program Slide 6.0 (Rocscience Inc.) to calculate the yield coefficient (k_y) for the critical potential sliding mass. In all the cases, the Bishop's Simplified and Janbu's Simplified methods were utilized, and the restraining forces for each one of the given horizontal accelerations k_h were obtained.

Pile response analyses were modeled in LPile v2012 (ENSOFT). All models considered a single-pile geometry and the same soil profile properties assumed in the slope stability model. Different assumptions and modeling options were made to establish a range of possible solutions for the compatible force-displacement state from the pushover analysis and the slope lateral deformation. In relation with the load-deformation behavior of piles, the p-y curve recommended in Reese et al. (1974) were used for the non-liquefiable sand layers. In the case of fine-grained soils, the p-y curves for soft clays from Matlock (1970) were utilized. Finally, the p-y curves presented in Rollins et al. (2005) were used in the liquefied layers.

2. Case Study – Northern Coronel Pier (NCP)

2.1 Port of Coronel

The Port of Coronel, located 30 km south of the City of Concepción and 545 km from Santiago, is one of the most important port facilities in Chile. It covers a total area of 0.86 km^2 (including warehouses, open-storage areas and access roads) and it is used as a multipurpose terminal for satisfying the requirements of forest, fishing, and agricultural industries, among others. Three different piers are currently used for transferring cargo to ships (Fig. 1).

Fig. 1 – Main piers of the Coronel Port (Photo from Google Earth^{TM}).

The northern pier (Fig. 2a) was built between 1995 and 1996, with two mooring sites 170m long. Subsequently, to increase the terminal capacity, two additional mooring stretches extending about 500m were added. This pier is used mainly for general and bulk cargo, and its supporting structure consists on a reinforced concrete deck, resting on longitudinal and transverse steel beams, supported by conventional driven steel pipe piles (vertical and battered) (Port of Coronel, Personal Communication). The southern pier, on the other hand, is 800m long, and it is used for container cargo. It was built between 2006 y 2007 using a base-isolation system (Fig. 2b) that combines vertical piles in parallel with elastomeric isolators placed on top of groups of four interconnected battered piles (Brunet et al., 2012). Finally, the Bulk Carrier terminal (Fig. 2c), with a total length of 1200m, is mainly used for bulk cargo. This pier was under construction when the 2010 Maule Earthquake took place.

(a) (b) (c)

Fig. 2 – Port of Coronel: (a) Northern Pier (Photo: www.belfi.cl), (b) Southern Pier base-isolation system (Photo: www.sirve.cl), and (c) Bulk Carrier terminal (Photo: www.puertodecoronel.cl).

In this paper, NCP is selected as the case study, as it represents the structural and geotechnical configurations of a number of piers in Chile and elsewhere. In addition, this is a well-documented case in which liquefaction phenomenon caused severe damage, including cracks and ground settlements, sinkholes, and also rotations and displacements of the pile foundations (see figures 3a, 3b, and 3c).

(a) (b) (c)

Fig. 3 – Damages observed in NCP after the 2010 Maule Earthquake. (a) Pavement cracks and ground settlements, (b) Sinkholes, and (c) Rotation of pile foundations (Photos: courtesy of D. Asimaki–GEER Association Team).

2.2. Geotechnical description of NCP

Before the construction of the pier, geotechnical information at the NCP site consisted on a series of boreholes and SPT test results (Geovenor 1989; 1995; 2003). After the 2010 Maule earthquake, in the context of research projects lead by Prof. Christian Ledezma (PUC) and Prof. Kyle Rollins (BYU), new exploration boreholes and SPT/CPTu tests were performed at the three piers of the Coronel Port in 2014. Fig. 4 shows location of the available boreholes and geotechnical soundings at the NCP site.

Fig. 4 – Location of available boreholes and geotechnical soundings at the NCP site. (Photo from Google Earth™).

The geotechnical information collected indicates that, near the pier access platform, there is a predominant presence of poorly graded sand and silty sand layers; whereas away from shore, low-consistency sandy silt layers (with some clay intercalations) are identified. Of special importance is the presence of a low-to-negligible resistance mud sediments along the longitudinal direction of the pier, 3.5m to 4.0m deep in average. Finally, a sedimentary bedrock, composed by sandstones and soft siltstones, is located approximately 30m below sea level near the pier access platform, which gets deeper in the southern pier direction (Geovenor, 1995; González & Verdugo, 2012). Based on the previously mentioned information, a simplified soil profile for the NCP site was developed (Fig. 5). This geotechnical model was used for all the analyses included in the present paper.

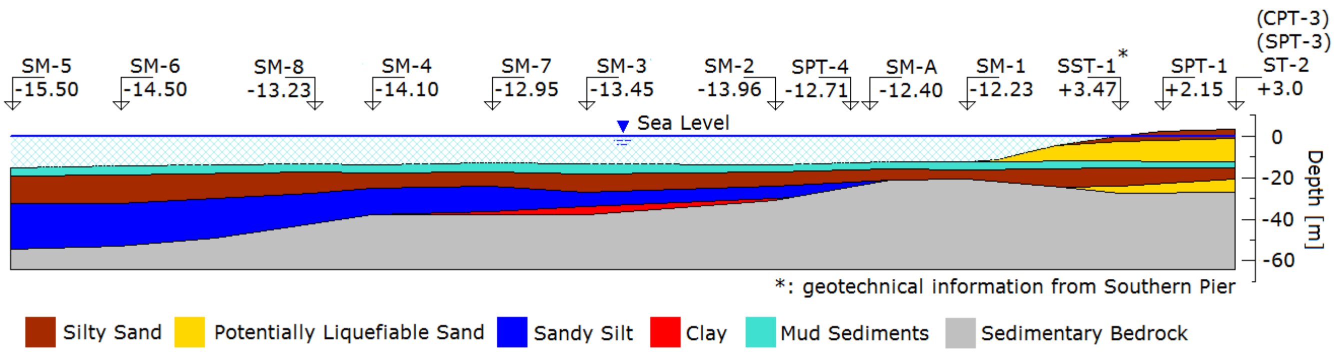

Fig. 5 – Simplified soil profile for the NCP site.

The observed damage of NCP during the 2010 Maule Earthquake is strongly related to the soil characteristics at the port site. The available Standard Penetration Tests (SPT) revealed the presence of medium dense to loose saturated sandy layers at different depths, with overburden-corrected resistances of $N_1 \leq 20$ blows/ft; i.e., soils with a high liquefaction potential (González & Verdugo, 2012). This is consistent with both the black sand ejecta obtained from cracks as reported by Bray et al. (2012) and the liquefaction susceptibility analyses performed by the authors according to the sand liquefaction triggering procedure presented in Youd et al. (2001) (see Fig. 6). In addition, it is believed that the mud sediments overlying the bedrock, as it was also identified by González & Verdugo (2012), could have played an important role on the observed damages, leading to a failure along low-resistance planes and making the effects of lateral spreading even worse.

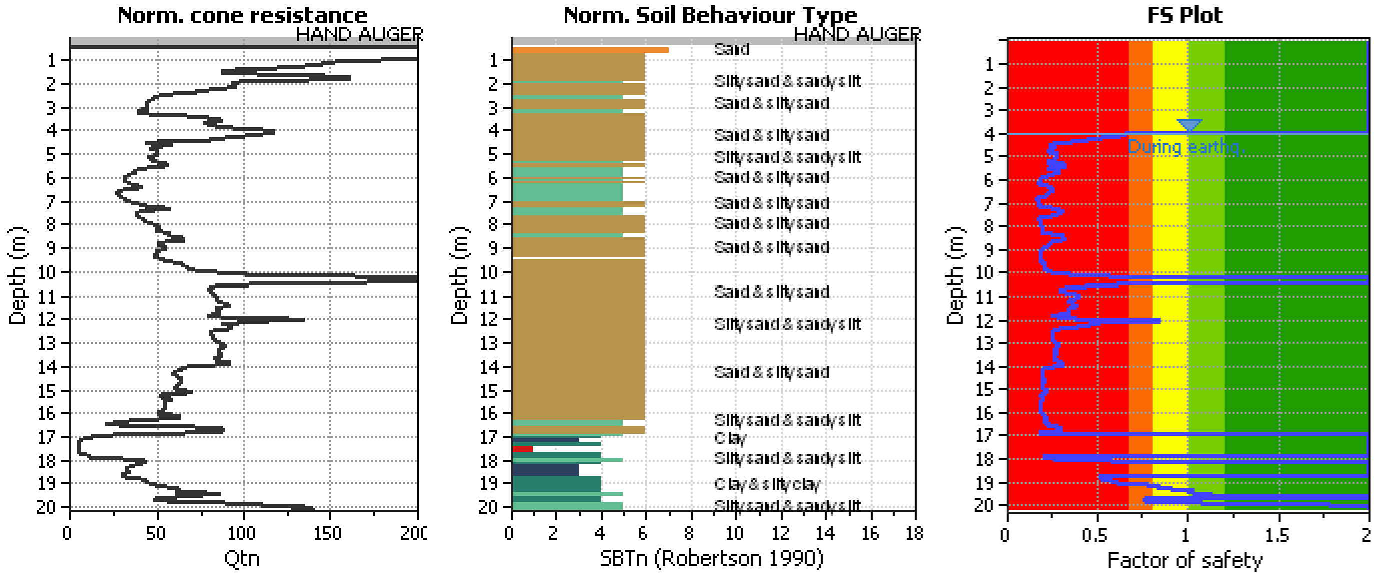

Fig. 6 – Results obtained from data from the CPT-3 sounding. (a) Normalized cone resistance, (b) SBTn Index and (c) Factor of Safety against liquefaction (M_w=8.8; a_{max}=0.4g).

2.3 Structural description of NCP

The supporting structure of NCP consists on a reinforced concrete deck, resting on longitudinal and transverse steel beams, supported by either vertical and/or inclined steel pipe piles. These piles are welded to plates, which are in turn welded to the resisting steel beams.

Standard configurations of pile foundations at NCP consist on either simple vertical structures or double-pile structures (as shown in Fig. 7); with a typical spacing of about 5.0m and 11m in the transverse and longitudinal direction of the pier, respectively (Port of Coronel, Personal Communication).

Fig. 7 – Typical configurations of pile foundations at NCP (Photo: courtesy of D. Asimaki – GEER Association Team).

Near the pier access platform, where the liquefaction-induced damage was more extensive, piles are approximately 50 cm in diameter and 20 m deep in average. Therefore, these piles did not reach the bedrock level (González & Verdugo, 2012). It is believed that this fact, in addition to the fixity conditions between the piles' heads and the superstructure, may have played a key role in the observed failure mode of pile foundations. In this sense, one of the most important effects of earthquake-induced lateral spreading on the pier structure where pile rotations, which caused buckling of the stiffeners on the compression side and yielding of the stiffeners on the tension side of the supporting beams (Arduino et al., 2010).

3. Results

3.1 Liquefaction evaluation

Geotechnical information from borings ST-2, SPT-1, SPT-3, and SST-1, from CPT-3 were used for the liquefaction evaluation at the pier access platform zone (see Fig. 5). These analyses show the presence of two potentially liquefiable layers: (i) a shallow one, located close to the ground surface and extending to a depth of about 15 m below sea level, and (ii) a deeper one, 3.0m thick in average, overlying the bedrock. Average $(N_1)_{60cs}$ values of ≈16 blows/ft and ≈5 blows/ft were estimated, respectively, for these two sand layers. The S_{ur}/σ'_v ratios for the slope stability model were estimated, using Ledezma & Bray (2010), as 0.25 and 0.06, respectively.

3.2 Slope stability analysis

Based on the available geotechnical data, a slope stability model of NCP was created (Fig. 8). Information from geotechnical reports were used to define the properties of the non-liquefiable layers (Geovenor 1989; 1995; 2003). For the silty-sand material, properties of γ=18 kN/m^3, ϕ'=35º and c′=0 kPa were considered; whereas for the sandy-silt layer the properties were γ=18 kN/m^3, S_u=65 kPa. For its part, the unit weight of the mud sediments was γ=18 kN/m^3 and the undrained shear resistance was taken as 8 kPa, based on the low N-SPT values obtained from tests. Finally, for the clay material the properties were γ=18 kN/m^3, S_u=50 kPa; while the bedrock properties were assumed to be γ=22 kN/m^3 and S_u=1500 kPa.

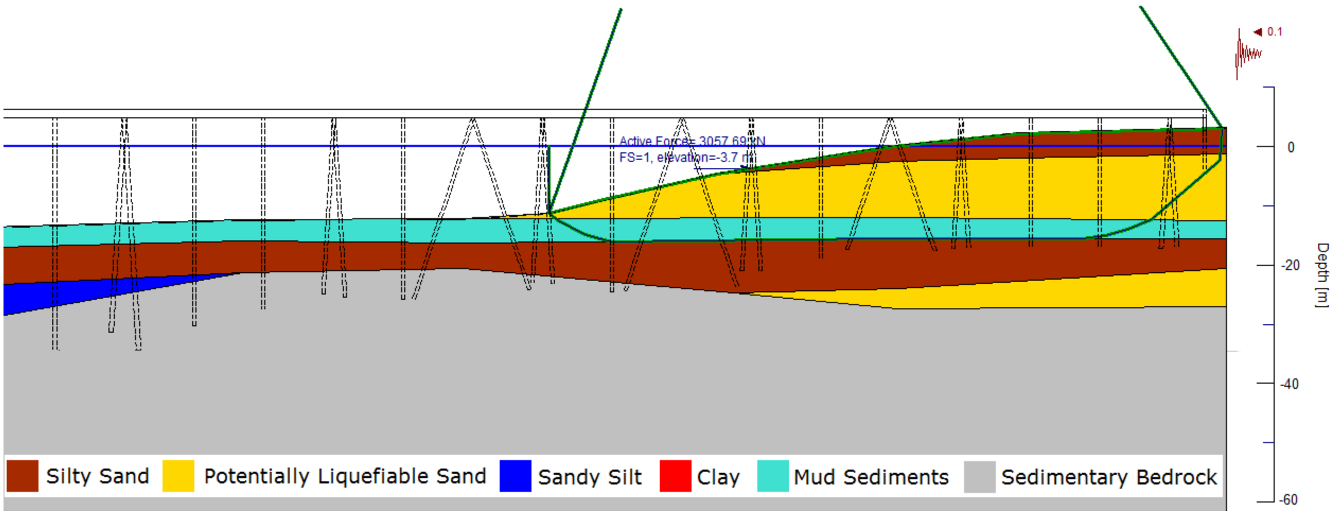

Fig. 8 – Post-liquefaction slope stability model generated for NCP.

The horizontal force F required to reach a factor of safety (FS) of 1.0 for different horizontal accelerations was located on the slope face at a depth corresponding to the center of gravity of the potentially unstable soil mass. Then, the Bray and Travasarou relationship (Bray & Travasarou, 2007) was used to obtain the lateral force versus displacement curves.

3.3 Pile response analysis

A simplified pushover analysis was performed considering an equivalent single-pile geometry and the aforementioned soil properties. ASTM A252 Grade 3 steel pipes, 50 cm in diameter and with a wall thickness of 9.5 mm, were modeled using the software LPile v2012 (ENSOFT). Different pile-head fixity conditions and embedment depths were analyzed. The pattern of imposed displacements followed the form indicated in Fig. 9.

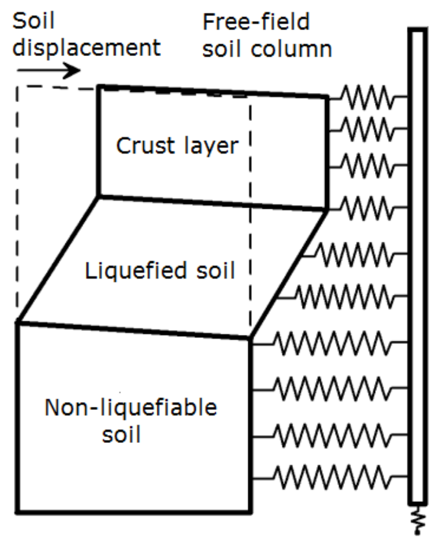

Fig. 9 – Pattern of imposed free-field soil displacements for pile pushover analyses. Modified from Boulanger et al. (2003).

Given that each group of foundation piles consist on two rows of elements along the transverse direction of the pier and considering a spacing of S=5 m between piles, the equivalent per-unit-width force R was estimated as R=2nV/S. In this expression, V corresponds to the "equivalent constant restraining force" from the piles and superstructure, obtained as the running average resistance that develops as the ground displacement increased from zero to its final value (Boulanger, 2006), while n is the number of piles along the longitudinal direction of the pier within the failure zone determined from slope stability analyses.

3.4 Compatibility of ground and pile displacements

The estimated residual lateral displacement can be found at the intersection of two curves: the lateral force-displacement curve of the structure stabilizing the sliding soils mass, and one that corresponds to the estimated residual displacements of the soil mass for different levels of restraining force. The result of these analyses is shown in Fig.10. As it can be noted, the resulting curves corresponding to the residual displacements of the soil mass depend on the slope stability procedure that is used. In addition, this figure includes the 16% and 84% percentiles from the Bray and Travasarou relationship.

The base model for the pushover analyses, named Model 1, is the one that corresponds to the estimated "actual" fixity and embedment conditions for piles of NCP when the earthquake took place. It considers partial fixity conditions at the pile head (i.e., the pile head is free to rotate under lateral loading), and an embedded depth of 1 m in the silty sand layer below mud sediments.

Additionally, to investigate the influence that both pile-head fixity conditions and embedment depth of the piles might have had on the magnitude of the expected liquefaction-induced lateral displacements for this case history, three additional models were developed:

• Model 2: considers a fixed-head condition at the pile top and an embedded depth of 1 m in the silty sand layer below mud sediments.

• Model 3: considers a partial fixity condition at the pile head and an embedded depth of 1 m below bedrock level.

• Model 4: considers a fixed-head condition at the pile top and an embedded depth of 1 m below bedrock level.

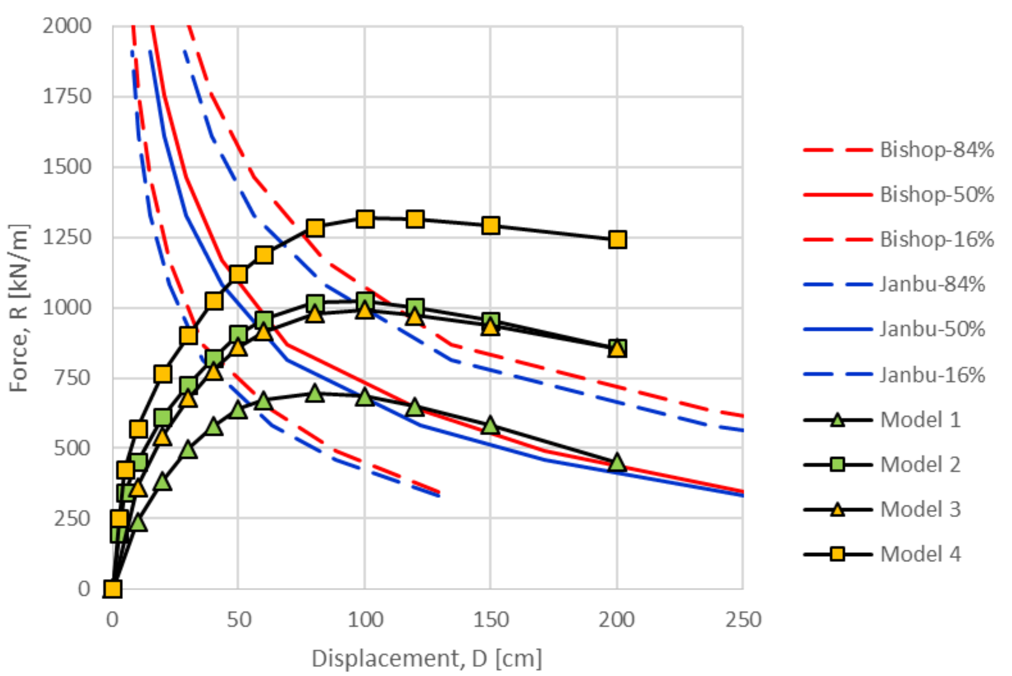

Fig. 10 – Expected lateral displacement D for different values of resisting force R for the pile foundations of NCP.

For Model 1, this approach gives permanent lateral displacements in the range of 50 to 120 cm. This is consistent with the lateral spreading displacements of 1.0 to 1.2 m measured by the GEER Association Team members at the Port of Coronel (Arduino et al., 2010), which produced excessive rotation of piles (of about 14º) and failure on the welded connections between piles and supporting steel beams (González & Verdugo, 2012).

Figure 10 shows that, in this case, the effect of providing a full fixity condition at the pile heads or increasing the embedded length of piles (Models 2 and 3) produce very similar results, and they would help on reducing the mean expected displacements to about 50% to 60% of its initial values. When both full-fixity conditions and increased embedment depths are considered (Model 4), displacements are reduced even more. However, these reductions are still not sufficient to ensure an appropriate pile foundation performance under lateral loading. The results obtained suggest that the number of piles supporting the sliding mass was not sufficient to carry the lateral loads induced by the occurrence of lateral spreading.

The seismic design of port structures is quite complex. On one hand, it deals with multiple factors and uncertainty sources such as large loads, soft ground zones, and exposure to natural hazards, among others. On the other hand, the marine environment in which these facilities are built makes some soil layers particularly susceptible to liquefaction occurrence during earthquakes, which may induce large displacements and settlements on both ground and supporting structures. For these reasons, one of the aspects that has to be considered in the seismic design of foundations for port structures is the horizontal loads that laterally spreading ground may induce on the supporting piles, in addition to the loads imposed on the piles from other sources.

4. Conclusions

In this paper, the case history of the Northern Coronel Pier, affected by liquefaction and lateral spreading during the 2010 M_w8.8 Maule earthquake, was presented. The seismic performance of this pier was evaluated using a simplified procedure that considers the intersection of the lateral force-displacement curve of the structure stabilizing the sliding soils mass, and the one that corresponds to the estimated residual displacements of the soil mass for different levels of restraining force. Assessment of liquefaction susceptibility and liquefaction effects, in terms of induced lateral spreading, was determined from available SPT and CPT profiles at the pier site.

The results obtained indicate that simple procedures correlate well with the observed pier damage and occurrence of liquefaction at the site. Although it is clear that more sophisticated models are required to get a better and more detailed understanding of the reasons behind the observed seismic behavior of NCP, the simplified procedures presented throughout this paper may be considered as a useful tool for preliminary design purposes. They provide a reasonable estimate of the permanent displacements due to lateral spreading that could be expected in a pile foundation system for a given design-earthquake. In this way, an initial estimation of the required number of piles and spacing, their structural properties, and the corresponding fixity and embedment conditions can be made in a relatively simple and straightforward manner.

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