ANALYSIS OF PILE FOUNDATIONS AFFECTED BY LIQUEFACTION AND LATERAL SPREADING WITH PINNING EFFECT DURING THE 2010 MAULE CHILE EARTHQUAKE

K. Kato¹, D. Gonzalez², C. Ledezma³ and S. Ashford⁴

ABSTRACT

Earthquake provides unique opportunities for engineering designs to confirm full-scale foundation and structural performances during ground shaking. Pile foundations suffered from liquefaction and lateral spreading in the 2010 Maule Chile earthquake is collected to analyze their performances and compared to the recently developed displacement-based design procedure. Three pile-supported bridges were selected from screened case-history data, and the analysis was performed with several sand models to calibrate the pinning effect against slope displacement. Results show that the prediction of slope displacement is affected by soil modeling and slope stability procedures. Also, in most cases, the pinning effect develops before reaching the maximum restricting force of the piles.

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Earthquake provides unique opportunities for engineering designs to confirm full-scale foundation and structural performances during ground shaking. Pile foundations suffered from liquefaction and lateral spreading in the 2010 Maule Chile earthquake is collected to analyze their performances and compared to the recently developed displacement-based design procedure. Three pile-supported bridges were selected from screened case-history data, and the analysis was performed with several sand models to calibrate the pinning effect against slope displacement. Results show that the prediction of slope displacement is affected by soil modeling and slope stability procedures. Also, in most cases, the pinning effect develops before reaching the maximum restricting force of the piles.

Introduction

Performance evaluation of existing pile foundations in areas subject to liquefaction and lateral spreading is an important process for earthquake design. The 2010 Maule Chile earthquake provides an opportunity to confirm the recently developed design procedure for pile foundations by Ashford et al. (2011) for liquefaction and lateral spreading, with making use of the wealth of data collected during reconnaissance efforts. Bradenberg et al. (2013) analyzed three bridges affected by liquefaction with different performance levels using a beam on nonlinear Winkler foundation model. The well prediction for pile foundations in gently sloped ground are reported when the measured lateral displacements were imposed. Ledezma et al. (2010) concluded that seismic performance of bridges founded by on piles is significantly affected by liquefaction and the pile-pinning effect. However, these analyses were implemented for bridges on gently sloped ground, and they do not provide parameters for piles in slopes. In this paper, three pile-supported bridges in sloping ground condition affected by liquefaction and lateral spreading during the 2010 Maule Chile earthquake are analyzed. The screened case history data documented by several reconnaissance efforts (e.g., Ledezma et al. 2012) are selected to evaluate pile behavior using the design procedure recommended by Ashford et al. (2011) and Shantz (2013). The results are compared to observed pile performances to provide engineering demand parameters.

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Description of Cases

Mataquito Bridge

The Mataquito Bridge is located in Iloca, which is 124 km away from the 2010 earthquake’s epicenter, and was moderately damaged by liquefaction and lateral ground spreading. Peak ground acceleration (PGA) 0.389g (north-south), 0.461g (west-east), and 0.390g (vertical) were recorded in Hualañé -north of Iloca, 12 km away from the bridge (Boroschek et al., 2010).

Although significant liquefaction and lateral ground spreading at the south abutment as observed, no displacement was observed on the abutment structure and nor on the pier adjacent to river. In contrary, the north abutment was slightly displaced (less than 20 mm) due to settlement of the approach fill and lateral ground spreading. The north abutment wall experienced a minor crash with the deck as a result of displacement. The approach fill settled approximately 0.5 m – 1.0 m. Transvers cracks approximately 200 m on the north approach embankment was observed (FHWA 2011).

The bridge consists of eight spans and seven interior piers that consist of 3×1 groups of the same drilled shafts. The piers are capped at the connection to the bridge girders. Two seat-type abutments with wingwalls on both ends of the bridge were supported by 4×2 groups of reinforced concrete drilled shafts, 1.5 m in diameter, and 17 m long. The height of both abutments is 10.0 m, the width is 14.0 m, and the transverse length is 8.0 m (McGann et al. 2012). Liquefiable sand layer, 4 m in thickness, is embedded beneath the north abutment. Subsurface explorations were made adjacent to each abutment. Figure 1 shows that the SPT N-values at the north abutment vary from 9 to 26 blows/foot in the liquefiable sand layers (i.e., some of the sand is very loose and liquefiable, and other deposits are denser). Below the liquefiable sand layer, a 4 m layer of fine, dense sand is embedded. At the south abutment, a 4 meters- thick liquefiable sand layer is embedded. SPT N-values at the south abutment range from 4 to 10 blows/foot within the liquefiable sand layers.

Juan Pablo II Bridge

The Juan Pablo II Bridge crosses the Biobío River in Concepción, and it was constructed in 1974. PGAs of 0.402g (north-south), 0.284g (east-west), and 0.398g (vertical) were recorded in
Concepción (Boroschek et al. 2010). Liquefaction and lateral spreading caused severe damages on the bridge structures and on the approach road. The north approach road settled significantly and it was not in service after the earthquake. Vertical settlements, ranging from 0.4 m to 1.5 m at some piers which were supported by short piles, were observed. Piers #45 and #60 located in the middle of the bridge settled about 0.6 m and 0.8 m, respectively. The settlement of the piers close to the approaching road in Concepción was larger than that of the piers close to San Pedro. Sand and water ejecta on the vicinity of the piers were observed. The bridge consists of 70 spans (each 33 m long and 21.9 m wide) which are composed of seven reinforced concrete girders and a concrete deck supported by two drilled concrete piers and rather short piles (16 m long) (Ledezma et al. 2012). Boring explorations along the bridge were made on June 2010. SPT values at the S-14 site obtained before the earthquake shows that a liquefiable fine sand layer from -2.65 m to -8.5 m is embedded (N-values range from 6 to 20 blows/foot). Non-plastic silt and clay is embedded below the liquefiable layers.

*Figure 2* Boring data and soil profiles near the north abutment, the pile section of Juan Pablo II Bridge

**Llacolén Bridge**

The Llacolén Bridge, which has four vehicular lanes and pedestrian access, crosses the Biobío River and it was constructed in Concepción in 2000. The span at the north approach unseated due to lateral ground movement. Cracks on the columns supporting the cap beam at the level of the rock rip rap on the embankment below the bridge were observed, likely developed due to the lateral ground movement. FHWA (2011) reported that although no damages on the bent or columns at the opposite end of the unseated span were observed, the ground settled about 0.4 m and resulted in a separation of about 0.25 m between the columns and the surrounding ground. Several locations with liquefactation-induced ground settlements near the bridge bents at the west end of the Llacolén Bridge were observed. The total length of the bridge is 2,160 m. Each span consisted of a deck slab and six precast and prestressed girders supported by six column bents. The columns are supported by 20 m long reinforced concrete piles (1.5 m diameter). Although there are no soil data at north abutment, SPT data indicates that weak soil layers, N-values ranging from 1 to 13 blows/ft, are embedded near the pier #45.
Ashford et al. (2011) and Shantz (2013) recommended a design procedure for pile foundations in areas subjected to liquefaction and lateral spreading. The following steps describe the material modeling and the evaluation of the soil-pile interaction to estimate pile-group performance in laterally spreading ground. Details of the procedures are described in Shantz (2013).

**Pile modeling**

A pile group is modeled as an equivalent non-linear single pile. The bending stiffness of the rigid abutment is calculated as one hundred times the bending stiffness of the original pile by the number of piles in the group. The abutment configurations are modeled as a single pile that has the same diameter as the original pile. The bending stiffness of the equivalent single pile is calculated by multiplying the bending stiffness of the original pile by the number of piles in the group. The diameter of the equivalent single pile is the same as the original pile. Although the bending stiffness of the piles in liquefiable soil during ground shaking is affected by the confining pressure of the surrounding soils, no changes are considered in the soil-pile interaction in this model.

**Developing p-y curve for the crust layer**

Two possible failure modes are considered for estimating the passive pressure against the abutment and pile due to the crust layer movement. For the first case, a log-spiral based passive pressure is applied to an abutment. This passive pressure is combined with the lateral resistance provided by the portion of pile length that extends through the crust layers. A side force on the pile cap is also added to the passive resistance. For the second case, the abutment, soil crust beneath the abutment, and piles within the crust layer are assumed to behave as a composite block. Rankine based passive pressure is applied to this composite block. The side force is developed over the full height of the block. Rankine passive pressure is utilized in second case because the loads from the liquefied layer is small and does not transmit enough to develop the deeper log-spiral failure surface that is generated by wall face friction. And, if the possible failure surface of the log spiral based passive pressure directly reaches beneath the composite block, such a condition is not appropriate to
estimate passive pressure acting on an abutment. The ultimate load due to ground laterally spreading ground is computed using equation (1).

\[ F_{ULT} = F_{PASSIVE} + F_{PILES} + F_{SIDES} \]  
(For Rankine pressure, \( F_{PILES} \) equals to zero)  
(1)

Brandenberg et al. (2007) suggests that mobilization of the full passive force requires relative displacements much larger than 5% of wall height for the case of a crust overlying a liquefied layer. The maximum displacement for the p-y curve of the crust layer is estimated using equation (2).

\[ \Delta_{MAX} = (T) \cdot \left( 0.05 + 0.45 \cdot f_{depth} \cdot f_{width} \right) \]  
(2)

**Evaluating the group reduction factor for non-liquefied and liquefied soil layers**

Piles in group tend to reduce resisting lateral load per pile. This effect for subgrade reactions is simply described by using p-multipliers \( m_p \). Mokwa and Duncan (2000) recommended p-multipliers as a function of pile spacing and transverse oriented row. To apply this to an equivalent single pile, the p-multiplier is averaged. The subgrade reaction in liquefied soil is less efficient in resisting lateral load due to the loss of shear strength. The scale down p-multipliers to modify the p-y curve of liquefied soil layers is used. The back calculated p-multipliers from a number of studies for the liquefied soil layers are utilized in this analysis (Ashford et al., 2008). A recommended equation for \( m_{p-\text{liq}} \) is given in equation (2). In this equation, \( N \) refers to the clean-sand equivalent corrected blow count \((N_{1})_{60CS}\). A clean sand correction is provided by Idriss and Boulanger (2008).

\[ m_{p-\text{liq}} = 0.0031\left( N_{1} \right)_{60,CS} + 0.00034\left( N_{1} \right)_{60,CS}^{2} \]  
(3)

The occurrence of liquefaction affects subgrade reactions near the boundary of the non-liquefied soil layers because non-liquefied layers above or below liquefied layers are pushed into the liquefied layers. To consider this effect for the soil-foundation interaction, p-multiplier is adjusted to appropriate values for soil layers close to liquefied ones. The modified p-multiplier for near liquefied soil layers is estimated using equation (4). In this equation, \( z \) is the depth from the ground surface, \( D \) is the diameter of pile, \( s_b \) is the modification factor \((s_b \text{ is range from 1 to 2})\), and \( p_{\text{u-L}} \) and \( p_{\text{u-NL}} \) are the ultimate subgrade reactions in the adjoining liquefied and non-liquefiable layers respectively.

\[ m_{p-\text{near}} = \frac{p_{\text{u-L}}}{p_{\text{u-NL}}} + \left( 1 - \frac{p_{\text{u-L}}}{p_{\text{u-NL}}} \right) \left( \frac{z}{N_{1}D} \right) \]  
(4)

**Estimation of the crust displacement due to liquefaction with pinning effect**

When piles are located in a slope which is potentially moving downwards, pile’s lateral restricting loads restrain this movement by the pinning effect. To consider this restricting effect, the shear resistance force in the pile should correspond to the driving force against the piles at the bottom of the slope failure surface. The pushover analysis for the foundation model, for a series of increasing soil displacement profiles, is developed using the LPile 2012 software. Since the driving force of piles in the equivalent single pile at the bottom of the slope failure surface is obtained, the relationship between the imposed soil displacement and the shear force can be plotted. Next is the
calculation of the resistance force $R$ that restricts the crust displacement at the bottom of the slope failure surface. This resistance force $R$ derives from the pinning effect. If the bridge deck is considered to behave as a longitudinal resistance to an abutment movement, the calculation of the passive force is required for the full mobilization of this resistance against the crust movement. The slope displacement is calculated using Bray and Travasarou (2007). The point corresponding to the intersection the pushover analysis and the slope displacement represents the expected crust displacement demanded on the foundation.

**Results and Discussions**

**Mataquito Bridge**

Figure 4 shows the relationship between the resistance force and the displacement using several analysis of the south abutment foundation of the Mataquito Bridge. The simplified Bishop and Janbu methods for slope stability analysis were conducted in this analyses. The estimated displacement using an equivalent single pile method is 4 cm. McGann et al. (2012) analyzed the south abutment pile foundation using an OpenSees 3D finite-element model. The estimated slope displacement was 24 cm, relatively consistent with field observations. Gonzalez and Ledezma (2013) also analyzed the abutment foundation using the LPile 2012 software with modified $m_p$ sand models, and the estimated slope displacement was 11 cm. The result of the pushover analysis is closer to the one by Gonzalez and Ledezma (2013) and the difference is approximately 18%. In contrast, the comparison against the 3D FEM analysis indicates that 3D geometry clearly affects the pile response in liquefiable sand layer. Figure 5 shows the pile deflection, bending moment, shear force, and subgrade reaction profiles for the pile foundations with 4.3 cm lateral ground displacement. The fixed condition at the top of the abutment is applied. Although this prediction is adequately corresponding to the observation, the estimated ground displacement is still high because negligible displacement of the south abutment was observed.

![Figure 4](image-url)  
**Figure 4** Estimated lateral displacement and resisting force for the piles in the south abutment of the Mataquito Bridge.
Analysis of the Mataquito Bridge abutment foundation using the equivalent single-pile method, 4.3 cm of slope displacement were applied.

Juan Pablo II Bridge

Since no structural data about the pile was found, some assumptions were made in this analysis. A pile cross section with a diameter $D=2.50$ m and steel area equivalent to 1.08% of the cross area, 55 φ 35mm rebar, were used. Figure 6 shows that the relationship between the estimated lateral displacement and the resisting force for the pile at the north approach road of the Juan Pablo II Bridge. The simplified Bishop and Janbu methods for slope stability analysis were conducted in this analyses. The top of the pier columns is fixed against displacement and rotation and an inertial load deriving from the pier 6,800 kN, which is estimated by assuming to reach the yield moment of the pier, is imposed at the top of pile cap section. The estimated ground surface displacement with the modified $m_p$ sand models by Gonzalez and Ledezma (2013) is 12.4 cm. Figure 7 shows the pile deflection, bending moment, shear force, and subgrade reaction profiles for the pile foundations with 3.6 cm lateral ground displacement.

Figure 5  Analysis of the Mataquito Bridge abutment foundation using the equivalent single-pile method, 4.3 cm of slope displacement were applied

Figure 6 Estimated lateral displacement and resisting force for the pile at the north approach road of the Juan Pablo II Bridge
Figure 7 Analysis of the Juan Pablo II Bridge foundation at the north approach road using equivalent single-pile method, 3.6 cm slope displacement were applied.

**Llacolén Bridge**

Figure 8 shows the relationship between the estimated lateral displacement and the resisting force for the pile at the north approach road of the Llacolén Bridge. The simplified Bishop and Janbu method for slope stability analysis were conducted in this analyses. This bridge presented small to moderate damage. However, vicinity of the north abutment the bridge deck unseated, forcing the closure of the bridge. An inertial load deriving from the pier 1,660 kN, which is estimated by assuming to reach yield moment capacity of the pier, is imposed at the top of pile cap section. The estimated displacement using an equivalent single pile method is 5.8 cm. Gonzalez and Ledezma (2013) analyzed the slope displacement using several soil models. The estimated displacement ranges from 2.4 cm to 4.1 cm. The pushover analyses indicate that about 90% of the maximum resisting force of the piles in Llacolén Bridge is reached with approximately 10 cm of lateral displacement. In all cases of the Llacolén Bridge, the slope displacement is reached before the maximum restricting force is achieved. Figure 9 shows the pile deflection, bending moment, shear force, and subgrade reaction profiles for the pile foundations using 5.8 cm lateral ground displacement.

Figure 8 Estimated lateral displacement and resisting force for the piles in the north approach road of the Llacolén Bridge
Figure 9 Analysis of the Llacolén Bridge foundation at the north approach road using equivalent single-pile method, 5.8 cm slope displacement were applied

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mataquito Bridge</th>
<th>Juan Pablo II Bridge</th>
<th>Llacolén Bridge</th>
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<tr>
<td></td>
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<td>SPT N-value</td>
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<td>1-21</td>
<td>2-20</td>
</tr>
</tbody>
</table>

Range of predicted slope displacements (cm)
- Mataquito Bridge: 4 - 24
- Juan Pablo II Bridge: 3 - 12
- Llacolén Bridge: 2 - 6

Measured slope displacements (cm)
- Mataquito Bridge: less than 2 cm
- Juan Pablo II Bridge: Settlement and slope movement were observed
- Llacolén Bridge: Cracks on the slope were observed

**Conclusions**

The case history data of three bridges that suffered from liquefaction and lateral spreading in the 2010 Maule Chile earthquake are described to identify pile performances during ground shaking. The Mataquito Bridge experienced negligible abutment displacement although significant liquefaction was observed in the vicinity of the bridge. The pile foundations and piers of Juan Pablo II Bridge and Llacolén Bridge, located in Concepción where large ground displacements occurred due to liquefaction, were damaged due to settlement and cracking. The procedure of an equivalent single-pile method is described and used to evaluate pile behavior against lateral spreading. Finally, observed damages and the analysis are compared to calibrate the pinning effect. Several sand models are used in the pushover analyses to estimate slope displacements.

Analyses of the three bridges considering the pinning effect leads to the following conclusions.

1. 3D geometry clearly affects the laterally loaded pile behavior. For example, when LPile and 3D FEM approaches are used to compute the lateral displacement of the south abutment foundation of the Mataquito Bridge, the estimations range from 4 cm to 24 cm.
Although no specific data about slope displacements of the Juan Pablo II Bridge and the Llacolén Bridge, the predictions tend to underestimate the slope displacements judging from the comparison against the observed level of damage.

In most cases, the pinning effect using the restricting force versus displacement relationship is exhibited before reaching the maximum restricting force of the piles.

References


