Performance of bridges in regions affected by liquefaction during the 2010-2011 Canterbury earthquake sequence

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SUMMARY:
The Canterbury earthquakes sequence of 2010 and 2011 resulted in widespread liquefaction damage in the eastern suburbs of the city of Christchurch, New Zealand and many bridges were damaged as a result of lateral spreading. Abutments, approaches, and piers suffered varying levels of damage, while little damage to the bridge superstructure was observed. This paper presents a summary of the bridge damage in eastern Christchurch as a result of lateral spreading during the Canterbury earthquake sequence. Using structural and geotechnical site investigation data, simplified computational models of a number of bridges were developed to model the response of the bridge abutments and foundations to lateral spreading. This is presented herein for one case study that was compared against the actual damage and displacements identified in the field, and used to evaluate the expected performance of the foundation system at depth.

Keywords: bridges, lateral spreading, liquefaction, abutment damage, Canterbury earthquakes

1. INTRODUCTION

The bridge stock in the city of Christchurch suffered varying levels of damage as a result of the 2010-2011 Canterbury earthquake sequence. The most affected were those in the central and eastern regions of Christchurch where significant liquefaction resulted in lateral spreading in the regions along the Avon and Heathcote Rivers. Few bridges suffered significant damage on non-liquefied sites during the 22 February 2011 M_w6.2 Christchurch earthquake. Detailed analyses and damage observations for these bridges can be found in Palermo et al. (2012).

The majority of bridges in Christchurch are symmetric, with spans between 15 and 25 m and a wide range of construction dates. More than 50% of bridges are reinforced concrete structures, with half of these cast in place and half with prefabricated prestressed concrete decks. Typical damage as a result of lateral spreading was abutment movement and cracking, approach settlement and spreading, pile hinging, and pier cracking. Very little superstructure damage was noted on any of the bridges and limited to pounding between the deck and the abutments. Further details on this bridge damage is summarised in Palermo et al. (2011).

This paper presents a summary of the lateral spreading induced damage to a selection of bridges in eastern Christchurch as a result of the Canterbury earthquake series, and specifically the performance during the 4 September 2010 M_w7.1 Darfield earthquake and the Christchurch earthquake. Simplified lumped plasticity beam-spring models were then developed to replicate the response of each bridge to lateral spreading displacements. For the sake of brevity, only one case study is reported herein.

2. LOCAL GEOLOGY AND GROUND MOTIONS

The city of Christchurch is located in the central coast of the Canterbury Plains, an approximately 50 km wide and 160 km long region created by the overlapping alluvial fans of rivers flowing east from the Southern Alps. Interbedded marine and terrestrial sediments approximately 100 m deep overlie 300 to 400 m of late Pleistocene sands and gravels (Brown & Weeber 1992). Much of the city was originally swampland, beach dune sand, estuaries, and lagoons, which were drained as part of the settlement and expansion of the city (Brown et al. 1995). A high water table, that in the eastern parts of the city is one to two meters below the ground surface, gradually increases in depth moving across the city to the West (Brown & Weeber 1992). Two rivers, the Avon (shown in Figure 1) and the Heathcote, cut through Christchurch and are both spring fed meandering rivers.

The M_w7.1 Darfield earthquake occured 40 km west of the Christchurch CBD at a focal depth of 10 km (Geonet 2012). The peak ground acceleration (PGA) in eastern and central Christchurch during this event are summarized in Figure 1. The M_w6.2 Christchurch earthquake was centered less than 10 km from the Christchurch CBD along the south-eastern perimeter of the city in the Port Hills. In the city, ground motions were characterized by large vertical accelerations, resulting from the oblique thrust faulting mechanism (Geonet 2012). In the vicinity of the Avon River, horizontal PGAs ranging from 0.19g to 0.63g and vertical PGAs from 0.49g to 1.89g were recorded (Figure 1).

3. SUMMARY OF BRIDGE DAMAGE

In this paper we will focus on the damage to the following bridges in eastern Christchurch highlighted in Figure 1: Gayhurst Rd, Avondale Rd, ANZAC Dr, Pages Rd and South Brighton. During the Darfield earthquake only two of these bridges, Gayhurst Rd and South Brighton, suffered moderate lateral spreading induced damage, with no other bridges damaged to any degree. Gayhurst Rd and Pages Rd bridges are cast-in-place integral bridges and the oldest structures (constructed in 1929 and 1954, respectively), with the remaining bridges all with prefabricated prestressed concrete decks.

During the Christchurch earthquake five bridges suffered severe damage and ten developed moderate damage in central and eastern Christchurch as a result of lateral spreading. A more complete summary of the bridge damage in the region following the Darfield and Christchurch earthquakes can be found in Palermo et al. (2010) and Palermo et al. (2011). The region of severe liquefaction damage following the Christchurch earthquake is shown in Figure 1, with all the bridge locations within or on the edge of this damage zone.

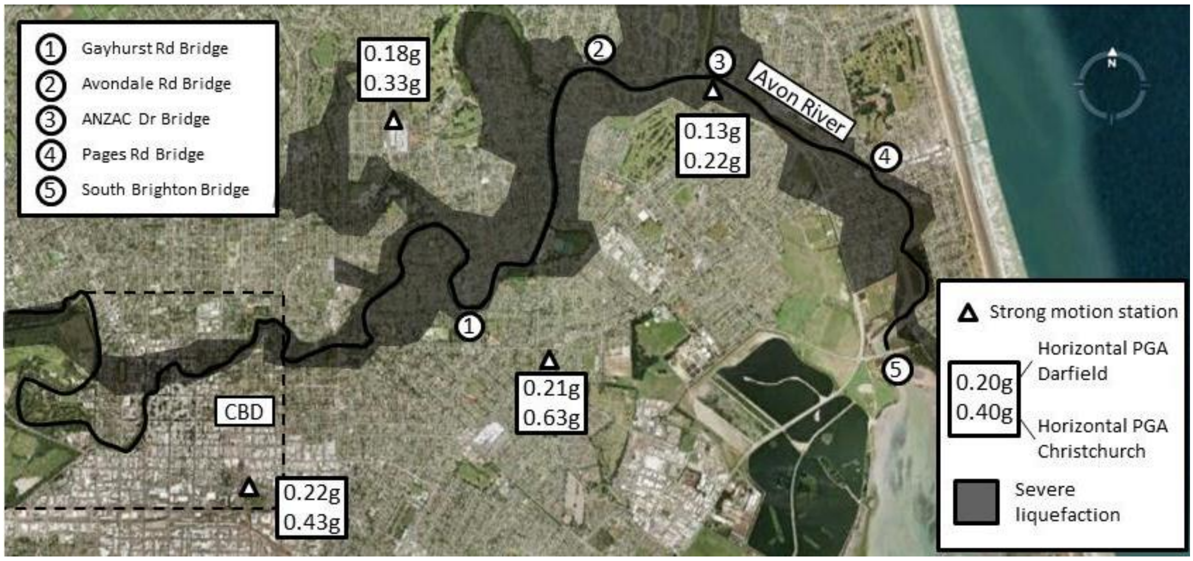

Figure 1. Map of central and eastern Christchurch indicating bridge locations, strong motion station locations and the region of severe liquefaction damage following the Christchurch earthquake

Figure 2. Examples of bridge damage following the Christchurch earthquake a) Gayhurst Road Bridge northern abutment; b) Avondale Road Bridge southern abutment; c) ANZAC Drive Bridge southern abutment; d) South Brighton Bridge western abutment

3.1. Gayhurst Road Bridge

The northern approach to Gayhurst Road Bridge, on the inner bank of the river, suffered from severe liquefaction induced damage in both the Darfield and Christchurch earthquakes. Large settlements and lateral spreads developed, with the effects most severe following the Christchurch earthquake. After the Darfield earthquake there was evidence of cracking of the abutment, and relative movement between the wingwalls and backwalls. This was worsened following the Christchurch earthquake, with further cracking and movement leading to an overall rotation of 5°, and differential horizontal movement between the backwall and wingwalls of approximately one metre (Figure 2a). After the Christchurch earthquake there was evidence of significant hinging of the northern abutment piles at their connection to the base of the abutment. At the southern abutment, only very minimal lateral spreading cracks were evident following the Christchurch earthquake, and no significant structural damage was noted.

3.2. Avondale Road Bridge

Avondale Road Bridge was not damaged during the Darfield earthquake, however minor ejecta volumes were apparent to the south of the bridge, on the inner bank of the river. Both the north and south abutments were affected in the Christchurch earthquake, with large volumes of ejecta and significant lateral spreading surrounding the south abutment, and more moderate levels of ejecta and lateral spreading to the north. The south abutment back-rotated by 7° (Figure 2b), plastic hinges were identified in the piles below the abutment, and there was moderate cracking and settlement of the approach. The north abutment developed 3° of back-rotation, and there was only minimal cracking and settlement of the approach material.

3.3. ANZAC Drive Bridge

A minor volume of ejecta and lateral spreading in the area surrounding the ANZAC Drive Bridge was evident following the Darfield earthquake, however there was no observed damage to the bridge. Following the Christchurch earthquake there was significant volumes of ejecta and lateral spreading surrounding both the north and south abutments. Lateral spreading at the southern abutment resulted in 8-10° of back rotation, exposing the steel H-piles, and settlement and cracking of the approach embankments (Figure 2c). The north abutment developed approximately 4° of back rotation, with the piles again exposed. Settlement and cracking of the approach was again evident, but to a lesser degree than the southern abutment. Rotations of the abutments led to crushing and spalling at the interface with the bridge deck.

3.4. Pages Road Bridge

A moderate volume of ejecta was evident in the region behind the western abutment of Pages Road Bridge following the Darfield earthquake, but no observed damage to the bridge and approaches. Moderate lateral spreading at both abutments following the Christchurch earthquake resulted in cracking and settlement of the approach material. Wingwalls on both abutments all hinged and rotated at the interface with the abutment backwall. Non-structural rock facing around the wingwalls moved and cracked, however there was no visible rotation of the backwall of each abutment. This was due to the stiff bridge superstructure and the integral deck-to-abutment connection. Pages Rd Bridge seemed to perform much better than expected given it was constructed in the 1930s.

3.5. South Brighton Bridge

Both the approach embankments of the South Brighton Bridge, which were built over wetlands, developed severe cracking and settlements in both the Darfield and Christchurch earthquakes. Lateral spreading resulted in back-rotation of the eastern abutment by approximately 4°, with evidence of plastic hinging in the abutment piles and cracking of the abutment. This damage was exasperated following the Christchurch earthquake, with additional lateral spreading further damaging the piles and abutment, and increasing the rotation by 3°. Differential movements developed between the bridge deck and the abutments, with crushing and spalling of the deck beam flanges. Similar damage was evident at the western abutment following both earthquakes (Figure 2d). Minor flexural cracking developed in the central piers as a result of transverse inertial movement of the superstructure.

4. MATERIAL CHARACTERISATION AND LIQUEFACTION TRIGGERING

4.1. Site Investigation

In order to characterise the soil profile at each bridge abutment, a literature search was performed to collate all existing site investigation data (Soils & Foundations 1997, Tonkin & Taylor 2011a, 2011b). Additional site investigations were then performed as part of a recovery project funded by the New Zealand Natural Hazards Platform to fill any perceived gaps in the data. The minimum level of site investigation consisted of a cone penetrometer test (CPT) and a borehole with standard penetration test (SPT) data.

At a selection of bridge sites, especially the older bridges, concrete and reinforcement material characteristics were investigated. Concrete compressive strength and elastic modulus were determined through testing of core samples. Using indentation testing, estimate of reinforcing steel strengths were also made. These results were used in conjunction with the material properties defined in design to provide a good representation of the structural material characteristics.

4.2. Liquefaction Assessment

Triggering of liquefaction of each of the bridge soil profiles was calculated using the site investigation data from each bridge abutment and estimates of peak ground acceleration (PGA) in the absence of liquefaction. The cyclic stress ratios (CSRs) for the Darfield and Christchurch earthquakes were calculated using the approach outlined in Youd et al. (2001). Magnitude scaling factors were used to scale the CSRs to that of an equivalent M_w7.5 earthquake ($CSR_{7.5}$). The cyclic resistance ratio ($CRR_{7.5}$) profile for CPT and SPT data was calculated using the approach outlined in Youd et al. (2001), and scaled for the effects of overburden (Hynes and Olsen 1999). The factor of safety against liquefaction was then calculated using these profiles, where factor of safety for each earthquake is equal to $CRR_{7.5}/CSR_{7.5}$.

During the Darfield earthquake, calculations indicated that liquefaction was expected to have occurred at all sites apart from Pages Road Bridge. At South Brighton Bridge, the prediction of marginal liquefaction underestimated the severity that was observed. During the Christchurch earthquake, liquefaction was shown to be triggered at all sites, with the lowest factors of safety at the Gayhurst Rd Bridge north abutment and at South Brighton Bridge, where the most severe liquefaction induced land damage was evident.

4. LATERAL SPREADING MODELLING

The aim of the modelling carried out in this research is to investigate the effect of lateral spreading displacements on the response of bridges, and more specifically, the response of each abutment and its foundation system. The response of each abutment was modelled using an displacement based approach based on that outlined in Cubrinovski & Ishihara (2004, 2006). Using the computer program Ruaumoko (Carr 2005) the pile and abutment system (Figure 3a) were represented using a two-dimensional beam-spring model, where the soil was modelled using spring elements, and pile and abutment were modelled using beam elements (Figure 3b). Bi-linear spring elements were used to represent the soil lateral resistance. Moment-curvature characteristics of the pile and abutment were modelled to capture the non-linear structural response and the development of plastic hinges. Models were of a representative strip of the foundation, encompassing a single pile, and the tributary width of the abutment above. Element lengths and spring tributary lengths were varied to take into account changes in cross sectional properties, soil layering, and expected locations of plastic hinging.

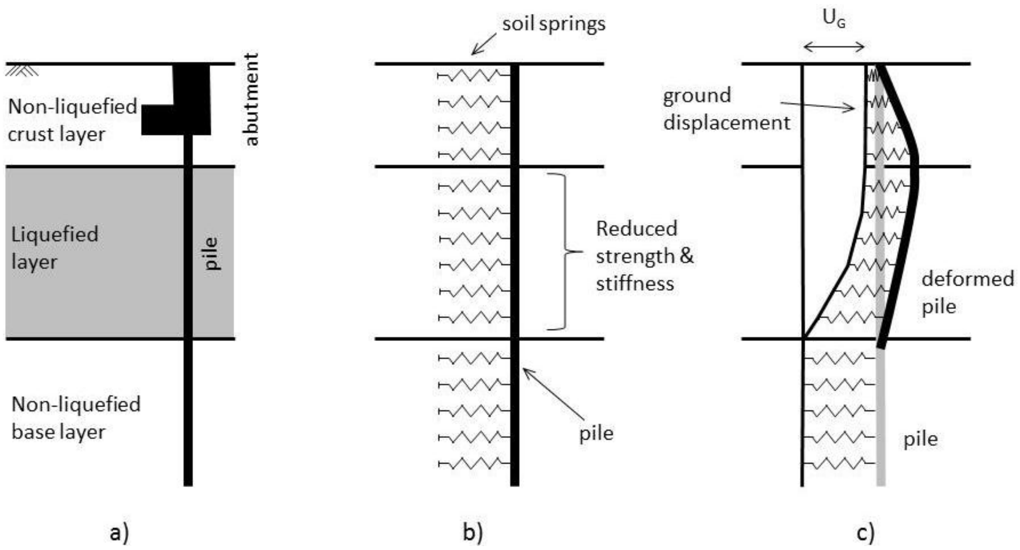

Figure 3. Lateral spreading modelling approach a) generalised soil profile and abutment-pile; b) beam spring model; c) applied ground displacement profile and deformation of abutment-pile

The focus of this modelling was the lateral spreading phase, as comparisons were made with the bridge damage resulting from the permanent displacements of the approaches to each bridge. Hence, the effects of cyclic ground displacements were not accounted for in this analysis. The effects of lateral spreading on the abutment pile system were modelled using the seismic displacement method, where a displacement profile is applied to the system. A simplified representation of the lateral spreading displacements using a cosine representation of displacements through the liquefied layer, and a constant displacement through the non-liquefied crust to the ground surface as shown in Figure 3c. This is applied to the ends of the soil springs, is transferred through to the abutment and foundation, leading to the development of foundation displacements and loads.

4.1. Geotechnical Characteristics

Soil springs were represented using bi-linear force displacement relationships for both non-liquefied and liquefied layers. For each of these situations, the initial stiffness (AIJ 2001), yield force, and post-yield stiffness of the springs down the length of the abutment and pile were defined. Spring yield forces were calculated using a Rankine passive earth pressure approach, and was modified for piles to account for the increase in passive pressures expected (Cubrinovski et al. 2006). In layers that were expected to liquefy stiffness characteristics were reduced to represent the degraded stiffness (Cubrinovski et al. 2006, O'Rourke et al. 1994, Orense et al. 2000). Spring yield forces in the liquefied layers were calculated using residual undrained shear strength values (Idriss & Boulanger 2007). To investigation the effects of soil variability, analysis using upper and lower bound representations of the liquefied soil properties were used. Upper and lower bound stiffness reduction factors were equal to 1/1000 and 1/50 respectively, and upper and lower bound values of residual undrained shear strength were used to represent the variability in the liquefied soil strength.

4.2. Structural Characteristics

Giberson beam elements with hinges at each end were used to represent the pile and abutment. CUMBIA (2007) was used to calculate moment curvature characteristics of the hinges, with axial loads defined using the vertical permanent loads from the superstructure. Moment curvature relationships were varied down the length of the pile to allow for changes in reinforcement details (both longitudinal and transverse reinforcement). Tri-linear moment curvature relationships were used to account for the cracking, yield and ultimate bending moment and curvature.

As the tops of the abutments were restrained by the bridge deck, there was assumed to be no lateral displacement at the top of the abutment. Depending on the connection characteristics between the abutment and bridge deck, a range of possible boundary conditions exist at this point, ranging from rotationally unrestrained (precast prefabricated bridges) to the rotational restraint provided by an integral bridge superstructure.

5. RESULTS AND DISCUSSION

For each model the soil displacement profile was gradually increased and the characteristics down the length of the pile were recorded at a range of performance points. These performance points were: (1) the point of first yield of the pile at the interface of the abutment, (2) the point of first yield in the pile at depth, (3) the point where the rotation of the abutment back wall was equal to that measured in the field. In this paper the results from Avondale Road Bridge are discussed.

3.5. Avondale Road Bridge

Avondale Road Bridge was built in 1960 with seat-type abutments and a simply supported superstructure. Retrofit of this bridge in the 2000s increased the seat length and provided some level of connection between the abutment and superstructure using bolted steel angles. In the computer model it was assumed these would provide little rotational restraint, and the top of the abutment was restrained from movement in the horizontal direction only. Each abutment was supported by seven 12.2 m long 406 m square lightly reinforced concrete piles (four D32 reinforcing bars). This bridge was not affected during the Darfield earthquake, however the increased intensity of ground motions during the Christchurch earthquake led to significant damage as a result of lateral spreading. During the Christchurch earthquake (M_w6.2), for an estimated PGA of 0.34g a 3 m thick layer liquefied layer was calculated between depths of 2.5 m to 5.5 m below the bridge deck level. Using the method proposed by Zhang et al. (2004), 65 cm of lateral spreading was calculated for this soil profile. As there is considerable uncertainty in this calculation, an appropriate range of 32 cm and 130 cm was defined equal to half and double the calculated value.

For both the upper and lower bound models the pile section at the abutment interface yielded after only a few centimetres of ground surface displacement as a result of the low strength of the piles. At a ground surface displacement of 6 cm, both the upper and lower bound models developed a hinge approximately half a metre below the bottom of the liquefied soil layer. At this displacement the top of the pile had displacement by 5.5 cm. Bending moment characteristics were also very similar for the two bounding models at their respective ground surface displacements. At these small displacements it was clear that the soil bounds have little effect on the response of the abutment-pile system, and that the lightly reinforced piles were damaged by small amounts of lateral spreading displacements.

As indicated previously, the south abutment of Avondale Road Bridge developed a back-rotation of 7° and pile hinging was evident at the interface between the abutment and piles. For the upper bound model, a ground surface displacement of 51 cm resulted in 7° of back rotation, with a 60 cm plastic hinge beneath the abutment and plastic hinging at depths between 2.8 and 3.6 m below the top of the pile (Figure 4). This amount of ground surface displacement was within the range of calculated lateral spreading displacements using the Zhang et al. approach. The ultimate curvature of the pile section beneath the abutment was exceeded at this level of ground displacement, which relates well to the significant pile damage at this location.

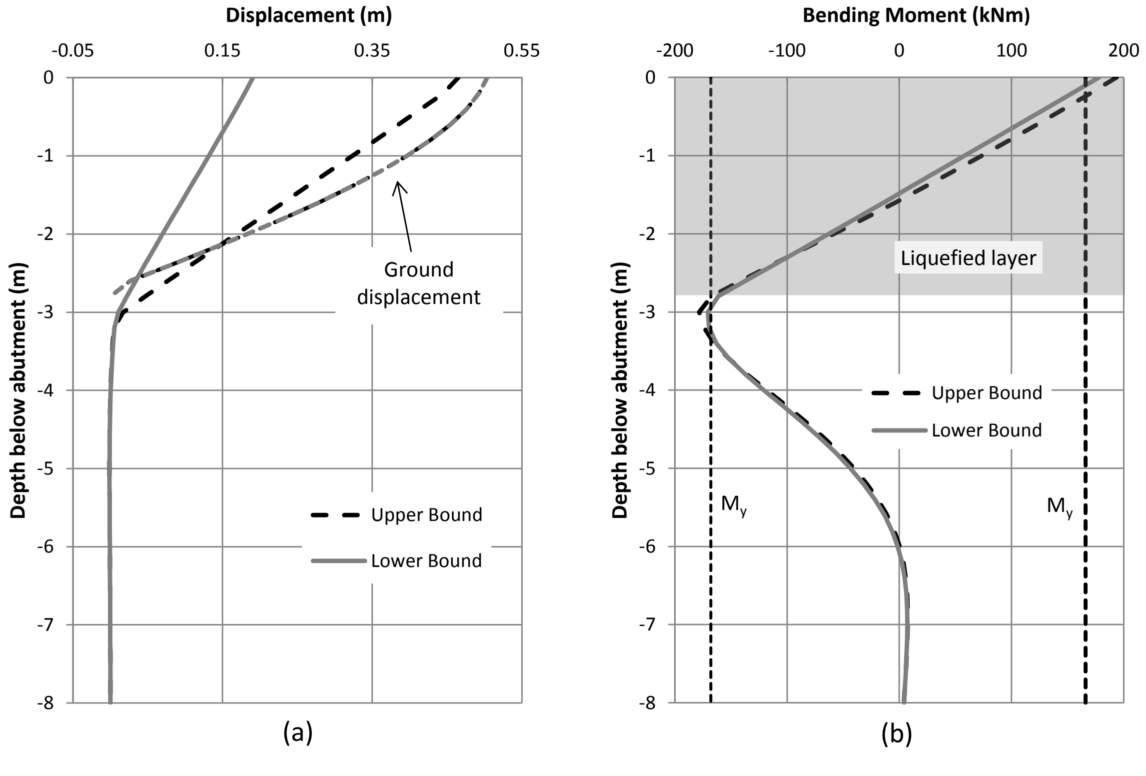

Figure 4. Avondale Road Bridge southern abutment at 51 cm of ground surface displacement; a) ground and pile displacement; b) pile bending moment

For the lower bound model the abutment developed only 3° of back-rotation at 51 cm of ground surface displacement. The displacement of the lower bound and upper bound model at 51 cm of ground surface displacement is shown in Figure 4a, along with the free field displacement profile. The lower soil stiffness and strength characteristics of this model meant that once the soil in the crust and liquefied layer had reached their ultimate pressure values, any additional lateral spreading displacement had little effect on the response of the abutment and pile. The fact that the abutment developed 7° of back-rotation suggests that the stiffness and strength of the liquefied layer may not have reduced by as much as the values used for the lower bound model. The pile bending moment profiles for each model at this displacement are compared in Figure 4b, showing that once plastic hinges had developed the bending moment characteristics showed little variation between the two soil bound models.

3.5.1 Effect of pile capacity on response

As Avondale Road bridge was constructed in the 1960s, there was no allowance for the effects of lateral spreading in the design process. The effect of the pile capacity on the response of the abutment-pile system was therefore investigated by increasing the yield moment of the pile. The upper bound model from the previous section was again subjected to lateral spreading ground displacements and the new response defined. For the same pile dimensions, an increase of the yield moment of 65% prevented the development of any plastic hinging in the piles. This increase could be obtained with the addition of an additional D32 bar to each side of pile cross section.

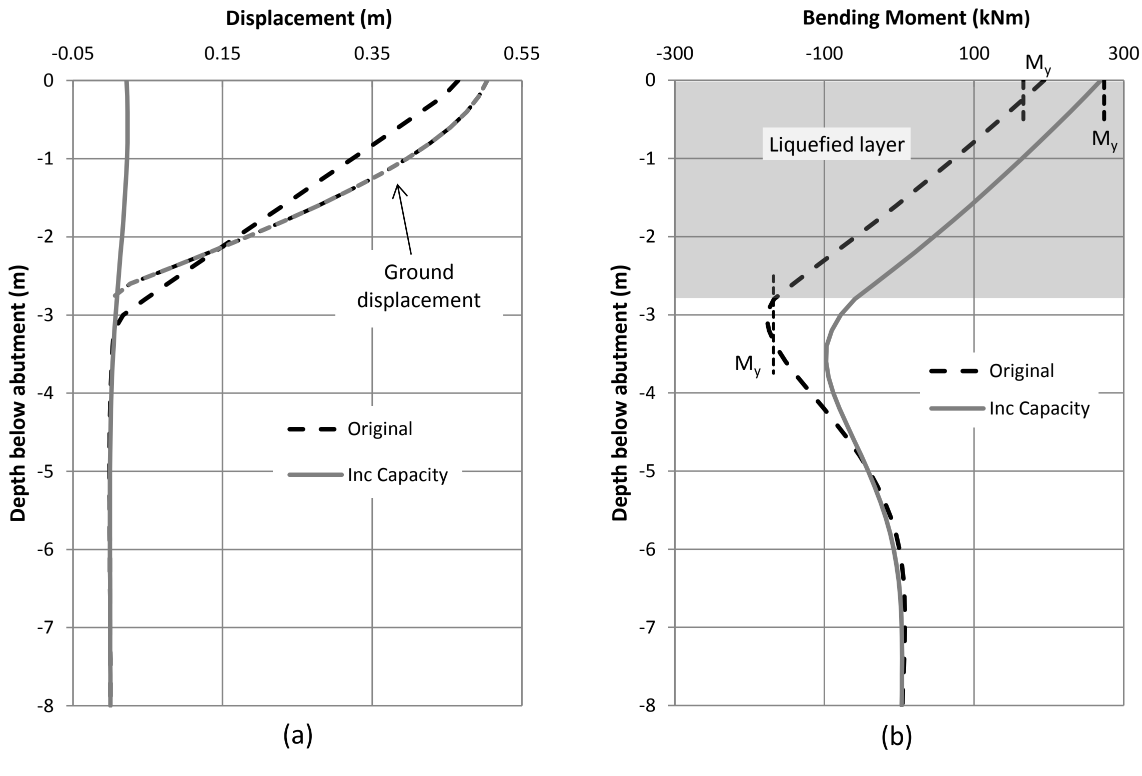

Figure 5. Effect of increased pile capacity on response of Avondale Road Bridge southern abutment at 51 cm of ground surface displacement; a) ground and pile displacement; b) pile bending moment

The significant effect of the elimination of plastic hinging is evident in the displacement and bending moment profile in Figure 5, comparing the original pile response with the increased pile section response for the upper bound soil conditions at a ground surface displacement of 51 cm. The displacement of the pile has been significantly reduced as a result of the lack of plastic hinge development, with the pile head displacement of the increased capacity pile approximately 5% of the original pile displacement. The bending moment profile in Figure 5b again shows the response of the original pile with plastic hinging at the top of the pile and beneath the liquefied layer. This is in contrast to the increased capacity pile, which remains elastic down its entire length for this and all other ground surface displacements. The increased yield moment means that the pile head bending moment increases in comparison to the original model, and is just below the increased yield moment as shown in Figure 5b. This increase in bending moment at the pile head shifts the bending moment profile down the pile, reducing the bending moment that develops below the liquefied soil layer. At this depth, the bending moment is lower than the bending moment in the original model, and much lower than the yield moment of 273 kNm (not shown on the figure).

If the ground surface displacement applied to this increased capacity model, there is little change to both the displacement profile and bending moment characteristics. Beyond a displacement of approximately 20 cm, the soil profile had softened to the point that any additional displacements had almost no effect on the abutment-pile system. These characteristics indicate the significant effect of pile plastic hinging on the response, and the reduction in displacements and rotation of the abutment-pile system when this structural non-linearity is avoided. The reduction in stiffness of the pile foundations as a result of plastic hinging of the original model meant that even though the soil profile had softened, the structural system still experienced additional displacements.

5. CONCLUSIONS

The Canterbury earthquakes series of 2010 and 2011 resulted in widespread liquefaction damage in the eastern suburbs of the city of Christchurch, New Zealand. In these eastern areas, many bridges along the Avon River were damaged as a result of lateral spreading of the river banks. The highly variable soil conditions and array of excitation levels resulted in a range of damage levels for each earthquake, not only between each bridge site, but in many cases from one end of the bridge to the other.

Post-earthquake inspections identified back-rotation of abutments and pile damage at their connection to the base of the abutments. Simplified abutment-pile models were developed that were able to capture these damage characteristics and the back-rotation of the abutment structure. These models also indicated that it is likely that many bridges developed pile damage at depth due to the differential displacements at the interface of the liquefied and non-liquefied soil layers, damage that would be difficult and costly to identify in the field.

Liquefied soil property bounds were shown to have a significant effect on the displacements of the abutment system, but once plastic hinges had developed there was little difference in the pile bending moment characteristics. The low capacity of the foundation system for the older bridges was also shown to have a significant effect on the response of the bridge abutments, with pile plastic hinging leading to increased displacement of the piles and permanent rotation of the abutments.

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