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IMPACT FROM BRIDGE SUPERSTRUCTURE ON PILES IN LATERAL SPREADING AREAS

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ABSTRACT: Past earthquakes have indicated that bridge superstructure could have a significant impact on the behavior of pile foundations in areas prone to liquefaction and lateral spreading since the pile deformations are generally restrained at the top of the abutment (or pier) while ground movement tends to push the piles towards the direction of the soil movement. This leads to the "back-rotation" of piles, which is different to those typically modeled assuming fixed or free pile head conditions. Although, the superstructure (deck) resistance is considered in simplified approaches such as the Caltrans (2012) method, the analysis cannot simulate the observed back rotation besides several other limitations in modeling of the actual soil-pile-superstructure interaction. This paper proposes a modification to the Caltrans approach where a user-defined "deck spring" is introduced into the pushover analysis to represent the superstructure resistance instead of applying it to the slope stability model. The differences and advantages of the proposed approach compared to the Caltrans method are highlighted using a case history and hypothetical example referenced in Caltrans.

KEYWORDS: Abutment, Deck, Caltrans, Lateral Spreading, Piles.

1 INTRODUCTION

Several case histories have highlighted the impact of the bridge superstructure on the behavior of bridge abutments and piles in areas subject to liquefaction induced lateral spreading [1,2,3,4]. Compared to the free-field movements, the superstructure resistance and pile pinning can significantly reduce the lateral spreading displacement near the bridge abutments. In particular, the resistance provided by the relatively rigid superstructure has a significant impact on the overall response of the substructure and foundation. Simplified methods have been developed to assess the performance of bridge pile foundations subject to lateral spreading. One of the commonly used approaches is the Caltrans approach [5] that explicitly accounts for the pile pinning force and bridge superstructure resistance, also known as deck resistance. However, as discussed in this paper, there are several key shortcomings related to how the deck resistance is considered in this method. Particularly, it was observed that the deformation pattern and pile demand estimated from these methods can be significantly different to those observed in actual earthquakes and those estimated using more advanced numerical analyses that incorporate substructure and superstructure elements. Using the basic framework outlined in Caltrans, this paper presents a revised approach to account for the soil-pile-deck interaction in bridge abutments impacted by lateral spreading. The differences between the proposed and Caltrans approaches are demonstrated using a case history and an example referenced in [5].

1.1 Caltrans method

The methods available to analyze bridge foundation subject to lateral spreading can be broadly classified into the following two categories:

- Force-based methods [6, 7, 8, 9]
- Displacement-based methods [5, 10]

In force-based methods, the soil loads acting on the pile from laterally spreading soil are applied as an equivalent static pressure or as a concentrated load. One of the key drawbacks of force-based methods is the inability to account for the actual soil-pile interaction. The force acting on the pile will depend on the relative displacement between the soil and pile. Initially, this relative displacement between pile and soil is unknown; thus, it is typical to apply the peak soil resistance from horizonal soil springs on to the pile. If the relative displacement is insufficient to mobilize the peak soil resistance, the soil load is likely to be overestimated.

In displacement-based approaches, the free-field horizontal ground displacement is applied to the fixed ends of soil springs (i.e., node that is not attached to the pile). The soil force acting on the pile is iteratively calculated based on the relative displacement between soil and pile. In some computer programs, this iteration is automated. Two commonly used displacement-based methods are [5] and [10]. Cubrinovski et al. method [10] is an extension of the work completed by [11]. The objective of this paper is to discuss the improvements that can be made to the Caltrans method to account for the contribution from the superstructure more accurately. The Caltrans method, also known as the Pacific Earthquake Engineering Research (PEER) method, was developed based on contributions made by various scholars such as [12], [13], [14] and [15]. The key steps involved in the Caltrans approach are summarized below:

- Step 1: Conduct a liquefaction assessment and determine the post-seismic shear strengths of different soil units. This information is used to develop a limit-equilibrium based slope stability model for the abutment in question.
- Step 2: Estimate the resistance provided by the superstructure (or bridge

deck) as the passive soil pressure acting on the abutment. This is applied to the slope stability model as a constant resistance, while the resistance from the piles (i.e., pile pinning force) is increased gradually in the slope stability model. For each pile pinning force, the corresponding yield acceleration is estimated, which in turn used to estimate the horizontal ground displacement using Newmark sliding block type methods. The pile pinning force is plotted against the calculated horizontal ground displacement (i.e., "demand curve").

- Step 3: Using a program such as LPILE [16], the entire pile group and abutment is modeled as an equivalent "super pile". The program used for this purpose should allow ground displacements to be applied to the fixed ends of the soil springs. A pushover analysis is then conducted by incrementally increasing the magnitude of the horizontal ground displacement. The shear force developed at the slide interface is the total resistance provided by the pile (i.e., pile pinning force). The running average of this shear force is computed and is plotted against the applied ground displacement ("resistance curve").
- Step 4: The intersection of above two curves will provide the compatible displacement. The corresponding pile demand can be determined by applying this ground displacement to LPILE model.

Although the above approach is a relatively simple and attractive method to account for the complex soil-pile-deck interaction, several limitations exist in this approach. Most of these limitations are associated with the way the deck resistance is incorporated.

1) There is an inconsistency in the way the pile pinning force and deck resistance are considered in the Caltrans approach. While the pile pinning force is gradually increased in the slope stability model, full deck resistance is applied from the beginning. However, it is recognized that full mobilization of passive earth resistance requires a relatively large displacement of the abutment. If the actual displacement is smaller than that required to develop the full passive resistance, the deck resistance is likely overestimated. Once the compatible displacement is known, it is possible to confirm the validity of the passive resistance (i.e., deck resistance) assumed in the slope stability model by checking against the estimated ground displacement. It can be argued that the passive earth resistance could be adjust iteratively to ensure that it is compatible with the final displacement. However, this additional round of iteration will complicate the analysis approach. Partly to overcome this limitation, [17] recommended to consider the running average of the shear force. This results in a larger compatible displacement than that estimated using the unmodified shear force from the pushover analysis. While this adds some conservatism into the final result by

lowering the resistance curve obtained from the pushover analysis, this is merely a manipulation of the analytical method to account for the shortcoming in the approach.

2) For multi-span bridges, the deck resistance is mainly contributed by the adjacent piers instead of the soil backfill of the opposite abutment as suggested in the Caltrans approach. Therefore, the deck resistance depends on the stiffness and strength of these pier(s), which can be obtained from a pushover analysis. Similar to the resistance provided by an abutment, the deck resistance in these bridges is expected to develop gradually with the deformation of the adjacent pier.
3) Using numerical modeling, [18] demonstrated the importance of the expansion joint gap on the foundation response. If expansion joints are included, the gap should be closed before the deck resistance is mobilized. The pile head movement is constrained once the gap is closed. The Caltrans approach does not consider the impact from expansion joint gaps. To overcome this limitation, [18] proposed to obtain a compatible solution by ignoring the expansion joint in the Caltrans approach and add the expansion joint gap to the estimated displacement. However, it is questionable if the impact from pile pinning and expansion gap can be decoupled in all scenarios.
4) After mobilizing the deck resistance, the movement of the soil will push the pile towards the river while the pile head movement is restrained. As explained earlier, this leads to a characteristic deformation pattern of the piles, usually referred to as the “back-rotation” of abutment piles (Figure 1). As discussed in the subsequent section, back-rotation was commonly observed in the past earthquakes. With this deformation mechanism, the maximum bending moments and their locations are not similar to those estimated using typical pile head fixity conditions such as the “fixed” or “free” pile head conditions (or with a certain rotational stiffness). For example, back-rotation will cause tension cracks on the river side which is different to the cracking that is predicted using free pile head conditions where piles tend to develop tension on the land side. Under certain circumstances, piles experiencing back-rotation can cause larger bending moments near the pile head than those estimated using other pile head fixity conditions. The inability to model this behavior is a key deficiency in the simplified approaches.
5) [19] highlighted the importance of checking the bridge superstructure for compression force triggered from convergence of abutments due to lateral spreading. The compressive forces can buckle the deck laterally or vertically, causing severe damage to the superstructure (e.g., Tawarayama bridge during the 2016 Kumamoto Earthquake). In the Caltrans approach, this compression force is considered equal to the full passive resistance acting on the abutments. As highlighted earlier, this is an upper bound value

and the actual mobilized deck resistance will depend on the relative movement between abutment and backfill. In other words, in the Caltrans method, the compression force acting on the deck is independent of the foundation type and soil movement. However, the compression force developed in the bridge deck can be smaller if the ground displacement is small or if the pile deformation is smaller due to the rigidity of piles.

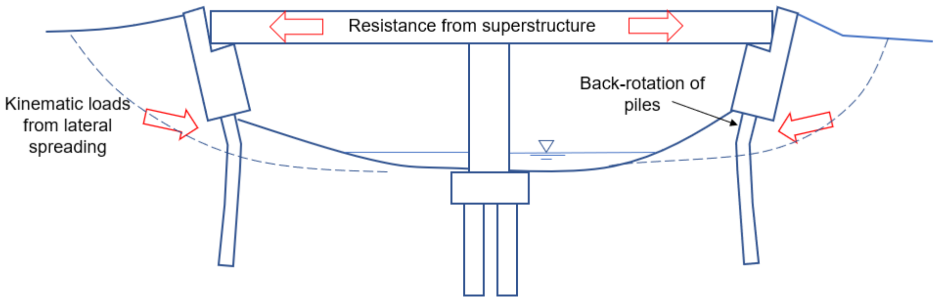


Figure 1. Schematic representation of back-rotation of abutment piles due to deck resistance.

6) Lastly, when a large resisting force such as the deck resistance is applied to a limit-equilibrium based slope stability model, the slide mass of the critical slip surface tends to increase by moving the slip surface further away from the abutment. However, at the time when the slip surface is formed, the failure surface is not likely impacted by the deck resistance due to the smaller displacement. As the horizontal ground displacement is increased with earthquake shaking, it is unlikely that the location of failure surface will move and grow in response to the increased deck resistance. To address this issue, [20] recommended to fix the critical slip surface similar to that created when the resistance force is zero. However, [17] stated that artificially restraining the slip surface will significantly increase the yield acceleration and produce unconservative results. Therefore, they recommended that critical slip surface should allow to grow as the applied resistance is increased to avoid overprediction of resistance from piles. [13] recommended to limit the slide mass length to four times the thickness of the embankment. Irrespective of the recommendation, above discussion demonstrates the issue of applying a large concentrated force on the limit-equilibrium based slope stability model. The revised approach proposed in this paper attempts to overcome these limitations.

1.2 Other numerical and analytical approaches

Using nonlinear dynamic analysis that included structural elements, [21] showed the potential conservatism in the simplified decoupled approach used in the Caltrans method. In particular, the pile head deflections estimated using the numerical modeling were much smaller than those estimated using the Caltrans method when the restraining effect from the superstructure is considered. In their study, the estimated pile head displacement from the dynamic analysis was in the order of 0.25 m while it was estimated to be 3.2 m using the Caltrans method. Similar to the numerical modeling conducted by [21], [18] showed that dynamic analysis with structural elements can be used to simulate the pile deformation mechanism involving back-rotation. Despite the advantages, it is not practical to conduct detailed dynamic analysis for all bridge projects as they require significant expertise and effort. Due to space limitations, this paper does not attempt to compare the results obtained from proposed method and numerical analyses.

Using the method proposed by [10], [4] attempted to simulate the back-rotation of pile observed during the 2011 Christchurch earthquake. This method requires the displacement at the pile head to be specified depending on the expansion joint gap size. It is not possible to predetermine the pile head displacement, which is not necessarily equal to the expansion joint gap in all scenarios. A somewhat similar approach was adopted by [22], where a lateral pile analysis was conducted by applying the estimated ground displacement. Subsequently, the horizontal load at the pile head is increased until the pile head displacement is zero or equal to the expected displacement. It was ensured that the horizontal force does not exceed the passive resistance of the backfill. The force required to achieve the target pile head displacement is considered as the deck resistance. [22] used case histories where the pile/abutment movement are already known, which is different to an actual design where the pile head displacement is unknown. Instead, the pile head displacement should be calculated based on the actual compliance of the backfill, expansion joint gap and pile-soil interaction.

2 EVIDENCE OF ABUTMENT BACK ROTATION AND DECK COMPRESSION

Prof. Leslie Youd [2] compiled several bridge failures caused by liquefaction induced lateral spreading. Some of the oldest evidence of damage caused by bridge deck compression can be found during the 1868 Hayward (California) and 1886 Charleston (South Carolina) earthquakes. For example, the Bacous bridge over the Ashley River was damaged during the 1886 Charleston earthquake due to convergence of riverbanks, which resulted in one plank to overlap another by 0.18 m. Similarly, the railroad bridge over Rantowles Creek was damaged due to lateral spreading that was estimated to be about 0.7 m, causing the stringers in the bridge to bulge up and planks to overlap due to compression.

Lateral spreading damaged several bridges during the 1906 San Francisco earthquake, including the highway bridge over Salinas River, California, where the lateral spreading about 1.8 m occurred toward the river channel. According to [2], bridge deck remained undamaged and braced the tops of the piers in places while their bases moved towards the river due to lateral spreading. This caused back-rotation of southern pier with the top of the pier tilted away from the river.

The 1964 Great Alaskan earthquake damaged 266 railway and highway bridges, collapsing about 20 bridges and damaging many others beyond repair. In almost all instances, bridges were compressed due to lateral spreading at riverbanks [2]. [23] reported that the compression was generally in the order of 0.5 m or less, except in two bridges where the compression was in order of 1.6 to 2 m. The bridge at Milepost 37.3 caused the deck to buckle horizontally by about 1.2 m due to the compression developed as a result of converging riverbanks. A back-rotation of piers was observed in the Yachiyo Bridge after the 1964 Niigata earthquake. According to [24], lateral spreading was in the order of 4 m at both riverbanks, which pushed the pier bases towards the river. However, at the tops were restrained by the bridge deck. At one pier, the back-rotation caused a differential horizontal displacement of about 1.1 m between the top and bottom of the column.

During the April 22, 1991 earthquake in Limon Province, Costa Rica, eight major highway and railway bridges collapsed, and several other bridges were severely damaged [2]. All damages were attributed to liquefaction induced lateral spreading. Compressional forces generated in the bridge deck due to lateral spreading caused the deck to act as a strut bracing the abutments and piers. The foundations were pushed towards the river causing piers and abutments to tilt outward from the river. For example, [25] reported that Rio Cub bridge experienced a rotation of approximately 8.5 degrees at the east abutment. The rotation occurred after closing the expansion gap which was reported to be about 86 mm. At the west abutment, the measured rotation was about 6 degrees and the expansion gap was about 50 mm. The exposed piles beneath the east abutment showed significant bending and cracking caused by the rotation of the abutment. According to [25], the bridge failure was largely prevented by the bridge deck. However, in couple of instances, the connections between the foundation and deck were severely damaged neutralizing the benefit of the bridge deck. This caused abutments and piers to move or tilt toward the river channel and in turn the bridge to collapse. In Rio Bananito Highway bridge, the southern abutment rotated approximately 14 degrees, while the northern abutment rotated approximately 10 degrees [26]. Exposed piles supporting the north abutment indicated fractures directly below the pile cap. The bridge deck likely pinned the bridge abutment during the initial ground deformation, and additional horizontal movements may have occurred following the collapse of the deck.

Some of the most detailed and extensively investigated cases of back-rotation of abutments were reported during the 2010 Darfield and 2011

Christchurch earthquakes [10, 27, 28]. These bridges were typically one to three span reinforced concrete bridges with integral or simply supported type [4]. After the earthquake, most of these bridges were repairable as the deck remained intact and acted as a strut which reduced the abutment deformations. The pile response was clearly influenced by the resistance provided by the bridge deck. For example, in the Ferrymead bridge over Heathcote River, the abutment moved towards the river because of lateral spreading. However, the deck restrained the displacement of the abutments by bracing the two abutments causing piles to back rotate. A similar deformation pattern was observed at the two piers where the deck was seated on bearing pads. In contrast, the old east abutment that was not restrained by the deck showed a deformation pattern that is similar to a pile with a “free” head condition. Back-rotation and deck compression were observed in several other bridges such as Fitzgerald Avenue, Avondale Road, South Brighton, Anzac and Dallington pedestrian bridge. Except for the Anzac bridge modeled using the proposed approach, the details of other bridges are not provided in this paper for brevity.

3 PROPOSED MODIFICATIONS TO CALTRANS APPROACH

As highlighted in the previous sections, the main limitation in the Caltrans approach is the way the deck resistance is considered. As demonstrated in the subsequent sections, these limitations can be resolved by including the deck resistance in the pile pushover analysis instead in the slope stability model. Notably, specialized programs are not required, and the modifications can be implemented using the same computer programs suggested in Caltrans. The steps involved in the revised approach are given below:

1) A limit-equilibrium slope stability analysis is conducted similar to the Caltrans approach by applying only the resistance from the pile pinning force (i.e., deck resistance is not included). Similar to the Caltrans approach, yield accelerations and corresponding horizontal ground displacements for different pile pinning forces are estimated. The demand curve is developed by plotting the applied pile pinning force against the horizontal ground displacement.
2) Using the basic framework suggested in the Caltrans method, pile pushover analysis is completed using LPILE or similar program that allows ground displacement to be specified to the fixed ends of the soil springs. However, an additional “user-defined” p-y curve is introduced to represent the deck resistance (i.e., “deck spring”), which is not required in the Caltrans approach (see Figure 2).
3) The characteristics of this p-y curve representing the deck will depend on the type of bridge. For example, in a single-span bridge with fully integral abutments, the deck resistance is provided by the passive resistance of the soil backfill from the opposite abutment and piles supporting that abutment.

In a multi-span bridge, the deck resistance is contributed by the adjacent bridge pier(s). The opposite abutment may also contribute if the displacements are large enough to deform the piers and close out the expansion gaps between piers and abutments. The resistance offered by the pier can be estimated from a pushover analysis of the piers and abutments. If expansion joints exist, the impact can be incorporated into the user-defined soil spring as shown in the subsequent examples. Analysis should also confirm that deck and abutment backwall have sufficient structural capacity to withstand the compressive and shear forces, respectively. For example, if the buckling resistance of the deck is smaller than the peak resistance obtained from the pushover analysis, the deck spring should be capped to recognize the structural limitation.

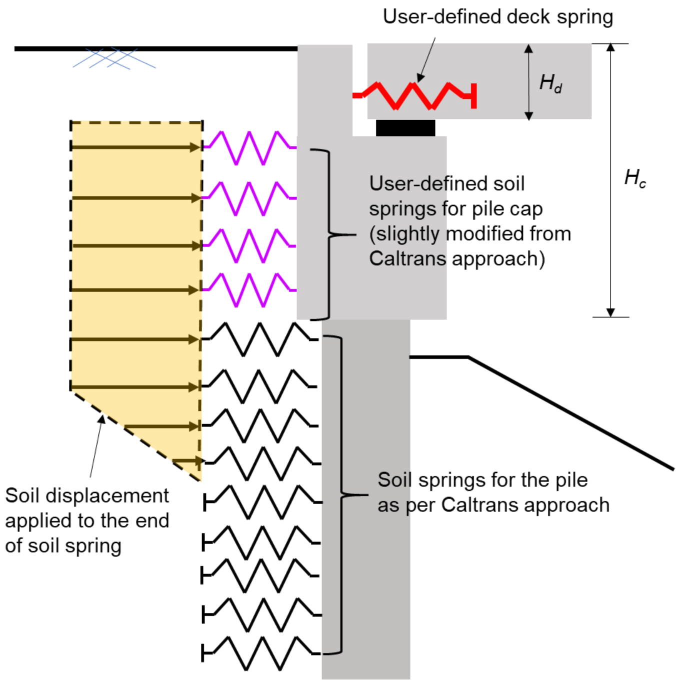


Figure 2. Schematic representation springs for deck, pile/abutment cap and piles using the proposed approach.

4) The pushover curves are converted into user-defined p-y curves for the height of the deck. In case of LPILE program, user-defined p-y curves should be appropriately scaled such that they represent the resistance of unit height of deck (i.e., divide the resistance obtained from the deck resistance (in kN) by the height of deck considered in the LPILE model). The number of pile elements should be carefully selected such that at least one pile element is included to represent the deck spring.

5) The ground displacement should not be applied to the user-defined spring that represent the deck resistance, as shown in Figure 2. For this reason, the soil spring representing the pile cap/abutment wall which is computed using the Caltrans approach should be scaled up to account for the lost kinematic loading by having the deck spring. For example, if the heights of the pile cap and deck are H_c and H_d, respectively, the resistance of the pile cap soil spring should be multiplied by $H_d/(H_d-H_c)$.
6) The remaining steps in the analysis approach are similar to the Caltrans method. After completing Steps 2 to 5, the resistance curve is developed by incrementally increasing the ground displacement applied to the LPILE model. The demand curve is obtained using the slope stability curves and Newmark sliding block method which allows to estimate the ground displacement using the yield acceleration coefficient. A compatible solution is obtained when the two curves intersect each other. In the proposed approach, it is not required to compute the moving average for the pile shear force since the flexibility of the superstructure is explicitly considered. Once the compatible ground displacement is known, pile response is obtained by applying this ground displacement in the LPILE model.
7) Compared to the Caltrans approach where the deck resistance is predetermined and assumed equal to the full passive soil resistance behind the abutment, in the proposed approach, the mobilized deck resistance can be obtained as the summation of soil reactions from the deck springs. As the soil reactions are typically shown as force per unit length, the total mobilized deck resistance is the summation of reactions (per unit length) multiplied by the element length resulting from the discretization of the deck thickness.

To demonstrate the differences between the proposed approach and Caltrans method, two examples are given in the following sections.

3.1 Case 1: Hypothetical example given in Caltrans

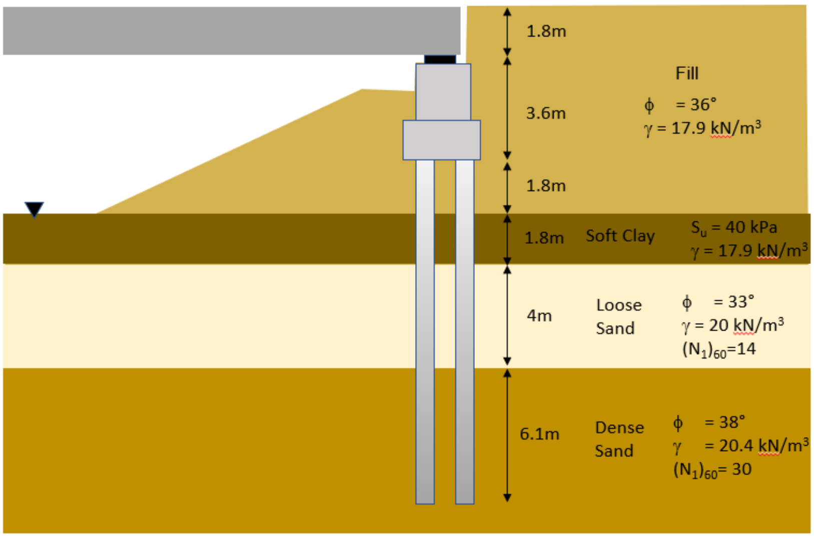
The following hypothetical example given in Caltrans [5] was reanalyzed using the revised approach. The abutment configuration and pile layout are shown in Figure 3. The soil profile consists of 7.3 m of fill, over 1.8 m of soft clay, over 4 m of liquefiable loose sand, over dense sand. The bridge abutment is 3 m wide and 14 m long and is supported on two rows of 610 mm diameter piles with a permanent steel casing of 12.5 mm thick. Concrete core of the pile was reinforced with seven #10 rebars. The bending moment-curvature relationship given in Caltrans was considered for these piles. According to Caltrans, the peak passive resistance offered by the abutment was 5516 kN which is mobilized at a displacement of 0.68 m. To allow a direct comparison of the two methods, the same rotational stiffness of the pile head, p-y scaling factors, soil input parameters used in the example were considered.

This hypothetical example does not provide details of the bridge structure such as the type of bridge (integral or semi-integral), number of spans, locations of expansion joints and their gaps. These factors are not explicitly considered in Caltrans method although they are recognized as critical parameters that influence the pile/abutment response as discussed in the preceding sections. For the sake of simplicity, the bridge is assumed to be a single-span bridge. To demonstrate the significance of the expansion gap, two analyses were completed assuming a fully integral-type abutment without an expansion joint and a semi-integral seat-type abutment with an expansion joint gap of 50 mm.

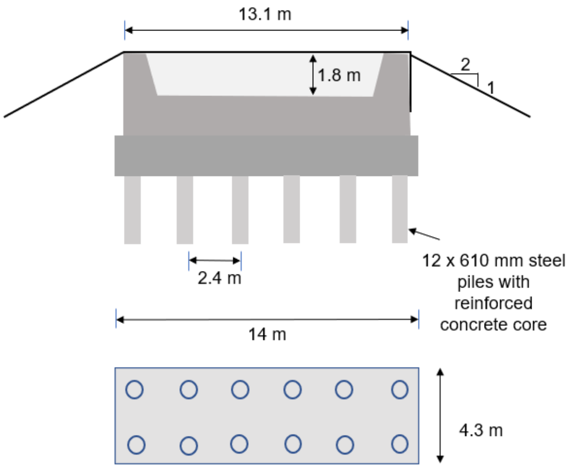
If the deck is pushed against the opposite abutment, in addition to the resistance provided by the abutment backfill, piles supporting that abutment will also contribute to this resistance. As explained under Step 4, this total resistance provided by the opposite abutment can be estimated by conducting a pushover analysis. In this example, the passive soil resistance from the opposite abutment was determined using the approach recommended in Caltrans [29]. Using this approach, the peak passive soil resistance was estimated as 33,000 kN and is mobilized at a displacement of 37 mm. This soil spring is applied as a user-defined spring in the LPILE model to represent the reaction from the opposite abutment. A pushover analysis is then performed to determine the combined resistance from the abutment wall and pile. The deck spring obtained from this approach has a much larger peak resistance and stiffness compared to that estimated using the Caltrans approach where the resistance is assumed to be provided only by the passive resistance of the abutment backfill.

Once the deck spring is applied as a user-defined soil spring, a separate pushover analysis is performed for the abutment in question. To demonstrate the impact of the expansion joint gap, an additional analysis was conducted by including an initial gap of 50 mm. The deck springs selected for these two analyses are shown in Figure 4. In this example, the deck resistance is linear within this displacement range because the peak passive resistance from the backfill was not fully mobilized and pile deflections are relatively small such that soil and pile stiffnesses have not reached the nonlinear range. In other situations, deck spring can reach the nonlinear range depending on the pile and soil responses or if the deck spring is capped at a certain value depending on the structural capacity of the deck or abutment backwall. It was assumed that deck and abutment backwall are structurally capable of withstanding the mobilized compressive and shear forces, respectively.

Using the resistance and demand curves, the compatible ground displacement was estimated as 116 mm based on the Caltrans method and 135 mm using the revised approach proposed in this paper. The displacement profiles and bending moments obtained from these two methods are shown in Figure 5. Though the difference in ground displacements estimated from these two methods is inconsequential, the pile deformation pattern and bending moments are markedly different. In this example, if there is no expansion joint gap, the maximum bending moment estimated using the proposed approach is about 1080 kNm at a depth of 9 m. If the gap is 50 mm, the maximum bending moment increases to about 1210 kNm, which occurs approximately at the same depth. The maximum bending moment calculated using the Caltrans approach is 1550 kNm. The predicted pile deformation using the revised approach is consistent with the back-rotation observed in piles when the deck resistance is mobilized. Also, the impact of expansion joint gap on the pile response is clearly visible in the proposed approach.



(a)



(b)

Figure 3. Details of the example considered in Caltrans [5]: (a) Soil profile and (b) pile configuration at the abutment.

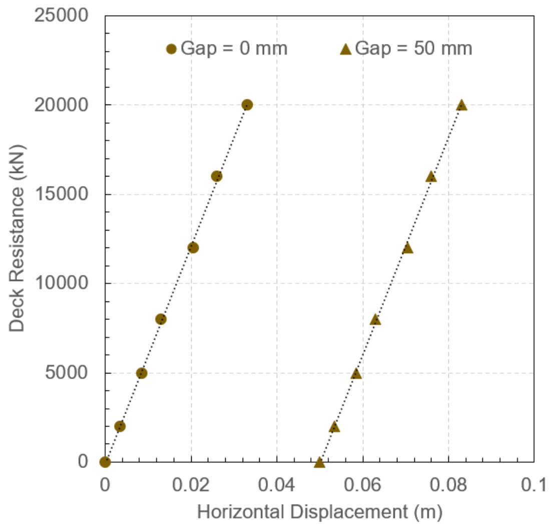


Figure 4. Deck springs considered for analysis

Using the soil reactions, the mobilized deck resistance can be calculated using the proposed approach as explained earlier. In this example, the mobilized total deck resistance is 28,975 kN without the expansion joint gap and 23,750 kN when there is an expansion joint gap of 50 mm. As expected, when there is an expansion joint, the piles should deform more to achieve force equilibrium. As a result, the mobilized deck resistance is expected to be smaller when there is an expansion joint. Caltrans method cannot estimate the mobilized deck resistance.

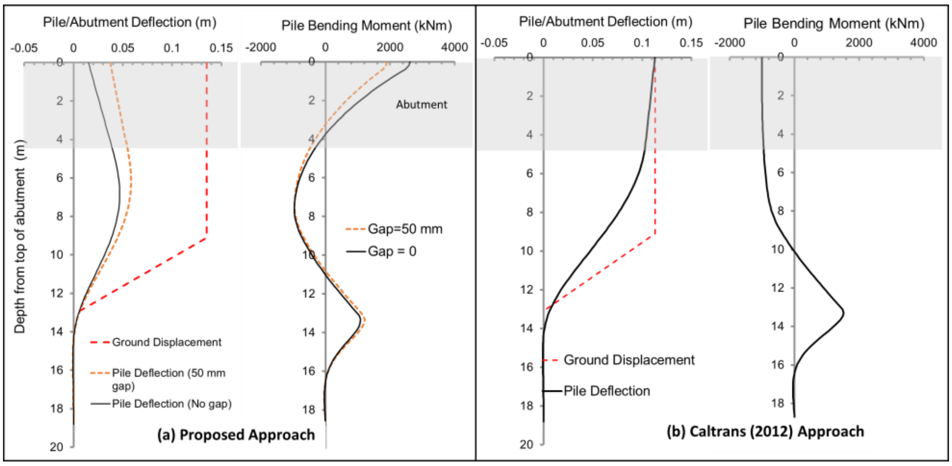


Figure 5. Estimated pile deformation and bending moment using (a) proposed approach and (b) Caltrans method.

3.2 Case 1: Anzac bridge

The Anzac bridge in Christchurch, New Zealand was selected to demonstrate the applicability of the proposed analysis approach to a real-life situation because its performance during the 2011 Christchurch earthquake is well documented in several publications [4, 10, 27] including the details of soil conditions interpreted from in situ testing, ground and abutment movements. The 48m long, three-span bridge was constructed in 2000 on the Woolston-Burwood Expressway and supports two lane traffic in each direction. The bridge has a skew angle of 13 degrees and consists of two end spans of 14.9 m long and a mid-span of 18.6 m long (Figure 6). The deck consists of precast prestressed reinforced concrete hollow units.

The piers consisted of four 1 m by 1 m rhomboid sections spaced at 7 m spacing. Each column is transitioned into a 20 m long, circular reinforced concrete piles of 1.5 m diameter with 8 mm thick permanent steel casing. The north abutment was supported on a single row of 16 steel HP 310x137 kg/m piles, while the south abutment was supported on 15 H-piles. The spacing between two adjacent H-piles was 1.5 m. Each H-pile was about 22 m long, reaching approximately the same depth as the pier piles. The bridge deck was continuous at piers. At the abutments, [4] considered the expansion joint gaps to be about 30 mm to 40 mm.

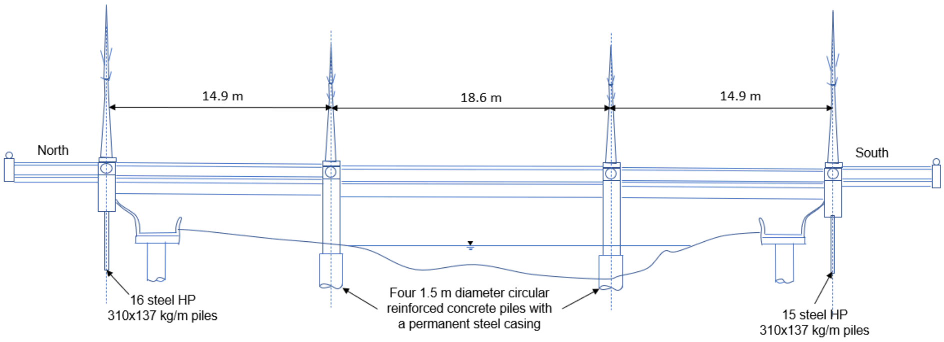


Figure 6. Simplified sketch of the Anzac bridge

3.2.1 Ground movements and observed damage

Although the bridge was not damaged during the 2010 Darfield earthquake, it experienced extensive damage during the 2011 Christchurch earthquake from extensive liquefaction and lateral spreading at the site. Large volumes of sand ejecta were observed at the south side of the bridge. LiDAR measurements indicated “free-field” lateral spreading displacements in the range of 0.9 to 1.1 m at the south abutment and about 0.4 m at the north abutment [4]. Some of the visible damage to the superstructure included cracking of the interior beam-pier connection and spalling of concrete due to compression at exterior beam-pier connections. The deck was undamaged and braced the abutments from inward movement. The resulting back-rotation of the south abutment was about 6 degrees, while the north abutment rotated by 3.5 to 4 degrees (Figure 7). Given the magnitude of ground displacement and observed abutment rotation, it was suspected that H-piles have surpassed their plastic moment capacity.

Cone Penetration Tests (CPT) and Standard Penetration Tests (SPT) conducted at the bridge site indicated that the south abutment is underlain by sand in the upper 1.5 m over a 1.5 m thick layer of silty sand followed by a sand layer to the termination depth of boreholes. Layers between 3.5 to 15 m depth were generally classified as compact and becomes dense below this depth. The north abutment is underlain by a gravelly fill, topsoil and silt in the upper 1 m, followed by compact sand to a depth of 18 m and dense to very dense sand. A simplified liquefaction triggering assessment completed by [4] identified potentially liquefiable layers extending to a depth of 14 m at the south abutment. One of the most prominent liquefiable layers was observed between 3.3 m to 6 m depth from the ground surface. Details of the liquefaction assessment is given in [4]; therefore, not repeated herein for brevity.

Figure 7. Back-rotation of the south abutment (reprinted from GEER [30]).

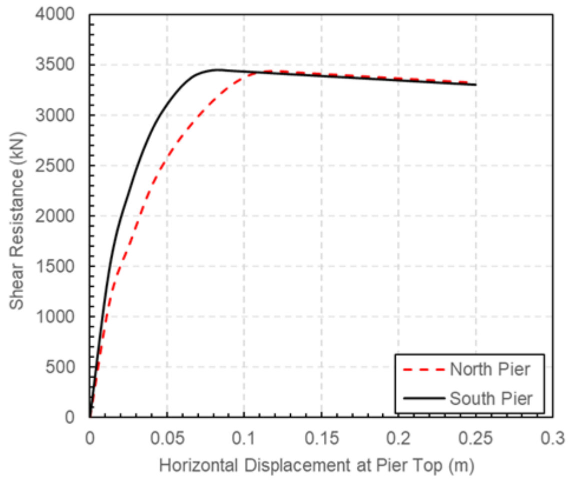
For brevity, this paper discusses only the predicted performance of the south abutment which experienced more severe damage than the north abutment. According to [4], the lateral spreading displacement at the south abutment ranged between 0.5 to 0.8 m, with a best-estimate value of 0.66 m. This value was considerably less than the free-field displacements measured away from the bridge influence zone. Earthquake ground motions were not available to estimate the lateral spreading displacements based on Newmark type analysis methods. However, since somewhat reliable horizontal ground displacements are available for this site, analysis was undertaken using the ground displacement profile estimated by [4]. As a result, the iterative approach recommended in Caltrans is not required to estimate the lateral spreading after accounting for the deck resistance and pile pinning effects. [4] estimated the ground displacement profile using the method proposed by [31] where the relative magnitude of the movement contributed by each soil layer is related to the Lateral Displacement Index. In the longitudinal direction, [4] concluded that seismic inertial loads had little to no impact on the response of the abutment piles; therefore, the inertial load was ignored in this calculation.

Compared to the previous example, the superstructure resistance is expected to be provided by the adjacent piers. To estimate the resistance from the superstructure, a pushover analysis was conducted on the adjacent piers using the super pile approach outlined in Caltrans. In the LPILE model, the concrete columns were included which extended to the bridge deck level. The top of column was assumed fixed due the resistance provided by the bridge deck. The pushover curves obtained for the north and south piers are shown in Figure 8(a). The resistance from the passive resistance of the north abutment was not considered since most of the superstructure reactions were taken by two piers. As discussed in the subsequent section, the estimated pile head displacement (i.e., deck movement) is 40 mm, which is not sufficient to mobilize the passive resistance from the north abutment. These two soil springs were combined to develop a deck spring to represent a bridge deck of 400 mm thick (Figure 8(b)). Once the deck spring is known, the pile response was obtained by applying the ground displacement profile developed by Winkley (2013). Some of the key soil input parameters considered in the LPILE analysis are shown in Figure 9.

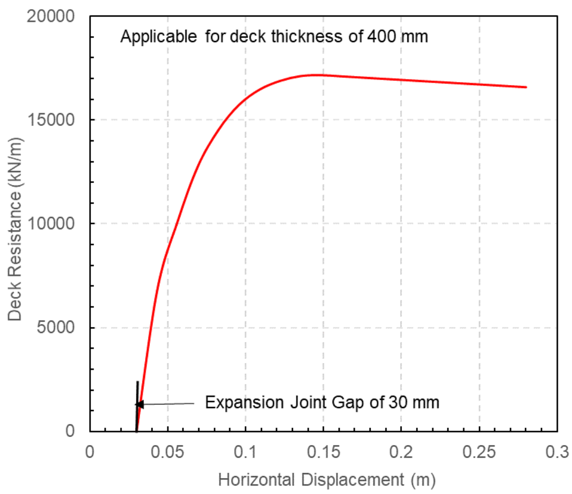
3.2.2 Results

The estimated pile deformation profile and bending moments are shown in Figure 10. The results indicate that H-piles at the south abutment are likely to yield at depths between 0.5 and 1.5 m below the pile head and at a depth of about 10 m at the boundary between liquefied to non-liquefied soils. This conclusion is consistent with the analytical estimations by [4] who used the method proposed by [10] for the pile-soil interaction analysis.

However, compared to the analysis approach proposed by [4], where the pile head displacement was set to zero, the proposed method estimates a displacement of about 40 mm. Using a global bridge model, [3] estimated that the bridge has displaced by about 30 mm to the north. The predictions made using the proposed approach is consistent with the above estimate.



(a)



(b)

Figure 8. (a) Pushover curves obtained for the north and south pier at the deck level and (b) resultant deck spring considered for south abutment.

According to [27], the horizontal displacement of the base of the south abutment wall was in the order of 50 cm. This was consistent with the prediction made in this analysis where this displacement was estimated as 55 cm. The estimated abutment back-rotation was 14 degrees which is approximately similar to the predictions made by [4] using the method proposed by [10] but greater than the actual measurement of 6 degrees reported by [1]. It appears that [1] considered the point of rotation of the abutment about between half and two-thirds of the way down the deck beam (i.e., 300 to 400 mm from the deck surface), which may have impacted the measured rotation. More importantly, this example demonstrates the benefits of the proposed approach to replicate the back-rotation observed for this bridge, which is not possible mimic using the Caltrans approach. It is interesting to note that the existing pedestrian underpasses were founded on 1.2 m diameter reinforced concrete piles independent of the bridge structure (Figure 6). These piles were only 6 m long; therefore, likely to have moved with the soil and provided no to little resistance to lateral spreading. However, their deflection shape is consistent with that estimated using the Caltrans approach where conventional "free" pile head conditions can be assumed in the absence of any restraining forces at the top of the pile head.

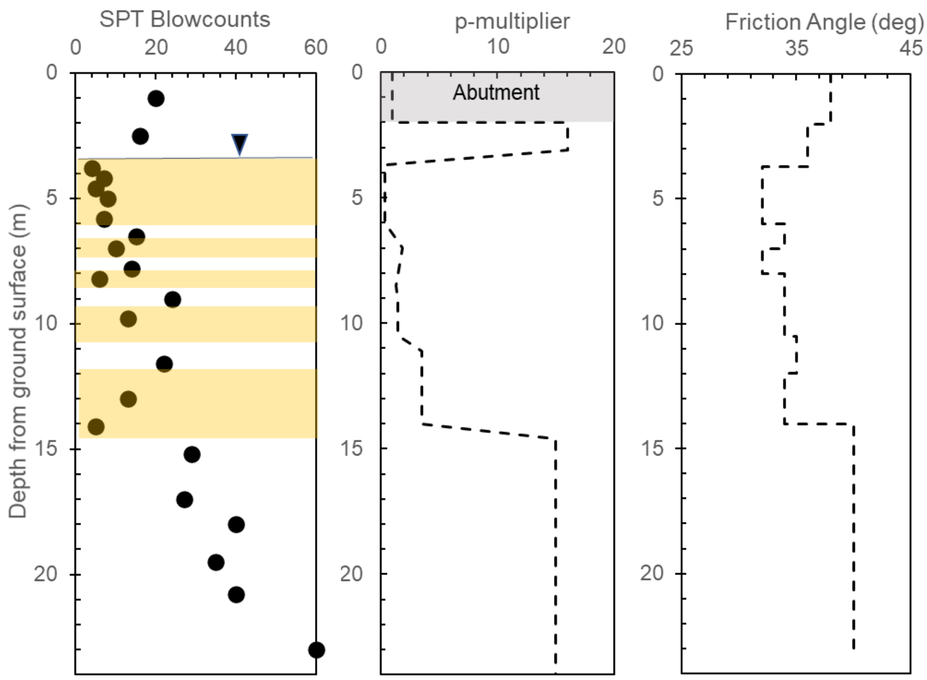


Figure 9. Key soil input parameters considered for LPILE analysis conducted for south abutment. Liquefiable soil layers are highlighted.

When the estimated abutment displacement of 40 mm is applied to the south and north piers, the estimated bending moment using LPILE reached the yield moment of the concrete section which was estimated to be about 1,800 kNm. This is consistent with the observations by [1] who noted cracking at the top of the pier column.

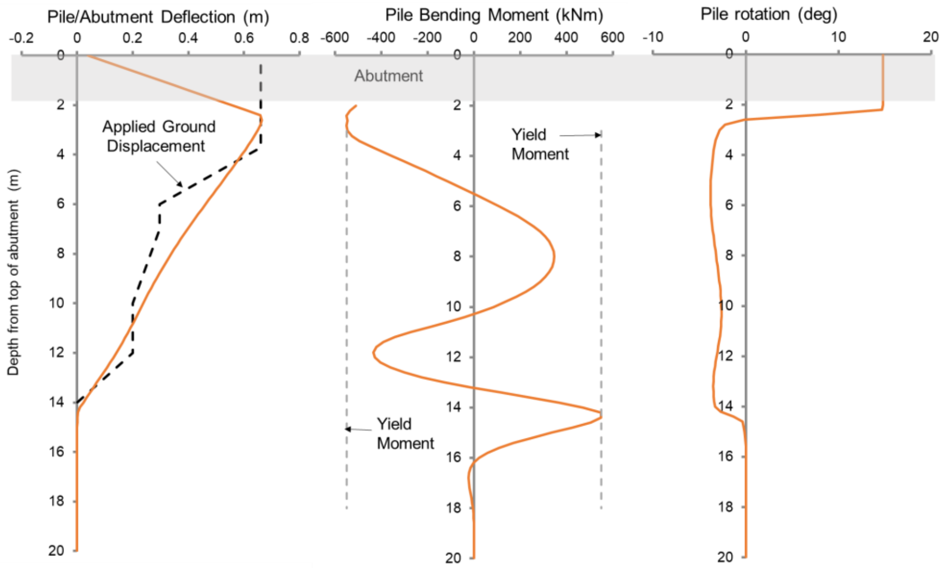


Figure 10. Pile deformations, bending moments and pile rotations estimated using the proposed approach for south abutment of Anzac bridge.

4 DISCUSSION

The proposed modification to the Caltrans approach is minor but has a significant impact to the estimated the performance of the pile and abutment. If the superstructure can resist the abutment/pile movement, the revised approach will estimate smaller pile head deflections; however, the estimated maximum bending moment could either be higher or smaller than that estimated from the Caltrans approach. Since the bending moment distributions are significantly different in these two approaches, the reinforcement cage details are likely to be different.

Throughout this paper, the benefits of deck resistance towards minimizing the pile demand and preventing collapse were discussed. The deck resistance can be relied upon for relatively short-span bridges with stiff decks. However, if the deck is weak and prone to buckling, the benefits of deck resistance cannot be relied. For example, during the 1991 Costa Rica earthquake, while the benefit of deck resistance in resisting the kinematic loading was evident in the Rio Cuba bridge, many other bridges with similar spans and abutment types lost their bridge decks [25]. The poor performance may be attributed to extensive liquefaction and slumping of the backfill behind the abutment. The bridge deck resistance will depend on the skew angle of the bridge. For example, according to [25], the collapse of the Rio Bananito Highway Bridge deck demonstrated the detrimental consequences of skewed abutments and deck configuration. In this case, the compression of the bridge deck due to lateral spread displacement at the bridge abutments caused rotation of the bridge deck sections, leading to the collapse.

These simplified de-couple analysis involve several simplified approximations that are expected to contribute to some uncertainty in the predicted responses. Nonetheless, the methodology can still be useful for bracketing the range of likely responses given the inherent uncertainties in the input parameters and analysis results. It is not practical to conduct complex numerical modeling for all projects. This paper introduces a slightly revised approach to account for the deck resistance and in turn able to account for the backward rotation of the abutments and piers observed in past earthquake events. The proposed approach also solves several issues associated with the Caltrans approach including the impact of expansion joint gap, ability to estimate the compressive load transferred to the bridge deck and avoid some limitations in limit-equilibrium method when applying large horizontal loads to represent the resistance from the superstructure.

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