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Seismic Response of Piled Bridge Abutments in Multi-Layered Liquefiable Soils Deposits

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ABSTRACT

Evaluation of liquefaction-induced lateral displacements of bridge approach embankments is of considerably important for piled bridge abutments. Especially, in the case of multiple liquefiable layers embedded under abutments, more significant damages are expected on pile foundations if several liquefiable soils liquefied and displaced laterally. However, the currently recommended procedures in evaluating the seismic performance of bridge abutment piles do not account for such conditions. In this paper, a simplified procedure is presented to estimate lateral displacements of soils by considering pile pinning effects on multiple liquefiable layers. In addition, an example is demonstrated for evaluating performance of a bridge abutment with considering two liquefiable soil layers sandwiched between three non-liquefiable soil layers. The presented procedure showed conservative results for force and displacement demands on the piled bridge abutment as compared to the current procedures.

Introduction

Past earthquakes have shown that the performance of piled bridge abutment can be significantly affected by the earthquake-induced lateral deformations of approach embankment due to liquefaction in the underlying soils. Estimation of liquefaction-induced lateral displacements of bridge approach embankments is important for estimating force and displacement demands on the piled abutments. Originally, NCHRP 472 specifications for seismic design of bridges [Transportation Research Board (TRB) 2002; Martin et al. 2002] proposed the design procedures for the piles locating in slope ground with considering pile pinning effects. The pile pinning is considered as a beneficial effect for reducing the lateral displacement of soil around pile foundations. Over the years, the pile pining method has been continually refined by other researchers (e.g., Zha 2004; Boulanger et al. 2006; Ashford et al. 2011). Recently, CALTRANS (2013) adopted the procedures in the design guideline for pile designing in liquefiable and laterally spreading ground.

The current recommended design procedures (e.g., Ashford et al. 2011, CALTRANS 2013) require determining the compatible displacement between the pile foundation and the abutment soils. Although the procedure is straight forward in one layer of liquefiable soil

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profile, there is no guidance for determining the compatible displacement in the case of multi-layered liquefiable soils deposits. In this paper, currently recommended design procedures are briefly summarized with the limitation for multi-layered liquefiable layers. Then, the extended of current method is presented for estimating lateral displacements of multiple liquefiable layers. Additionally, an illustrative example is demonstrated with comparison of results obtained from current and proposed procedures.

Estimating abutment displacement considering pile pinning effect

Current method

Existing methods for estimating abutment displacement by accounting the pile pinning effect includes following three major steps (e.g., TRB 2002; Boulanger et al. 2006; Ashford et al. 2011; CALTRANS 2013)

(1) Performing pushover analysis to estimate the restraining forces from the foundation piles for a range of imposed abutment displacements. The restraining forces are estimated at the middle of liquefiable soil layer.
(2) Performing slope stability analysis to estimate the yield acceleration, k_y, (and subsequently displacement) for a range of restraining forces from the foundation piles, resulting into factor of safety of 1. The restraining forces are applied at the mid-depth of liquefiable layer from which failure surface is constrained to passes through. The abutment displacements are estimated from yield acceleration using Newmark rigid sliding block analysis.
(3) Determining the compatible displacement based on the force-displacement relationships established in step 1 and 2 above. The intersection of two curves gives the compatible displacement.

The simplest pattern for displacement versus depth (imposed in step 1 above) is trapezoidal shape (e.g., CALTRANS 2013), with constant magnitude (maximum) from ground surface to the top of the liquefiable layer and varying linearly to zero until the bottom of liquefiable layer (for example as shown in Fig 1 a or 1b]. This method of imposing lateral displacement is suitable when the soil profile consists of only one layer of liquefiable soil and the failure surface passes through mid of liquefiable soil layer, as described in current design methods (e.g., Ashford et al. 2011, CALTRANS 2013).

In the case of multi-layered liquefiable soil deposits, the displacement versus depth pattern can be affected by all liquefiable soil layers. In current design, there is no any procedure to account the effects of multiple liquefiable soils for evaluating the seismic performance of bridge pile foundation. In multiple liquefiable soils conditions, there are two major issues in current methods: (1) identifying which liquefiable layer will be displaced and used for failure surface in limit equilibrium analyses, and (2) accounting interaction of multiple liquefied soil layers in pile pinning design approach. For simplicity, failure surface can be assumed to pass through any single liquefiable layer and displacement profile can be assumed as a single trapezoidal shape. However, such assumption may not suitable when the thicknesses liquefiable layers are relatively large in the profile. Although it is challenging to determine which liquefiable layer will displaced in multi-layered liquefiable soils deposits, the interaction of multiple liquefied soil layers may be accounted, as described in next section.

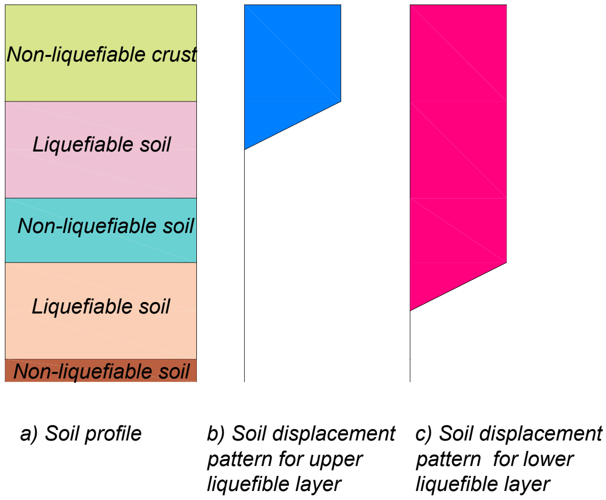

Figure 1: Schematic for lateral displacement pattern for single layer of liquefiable soil

Extended method

The idea for incorporating the interaction of multi-layered liquefiable soil in pile pinning approach is based on current design (i.e., Ashford et al. 2011, CALTRANS 2013), using multiple independent failure surfaces passing through each liquefiable soil layers. Following are the steps:

1) Estimate the restraining forces from the foundation piles for each liquefiable layers for a range of imposed abutment displacements. For example, Figure 1a and b show abutment displacement patterns for two layers of liquefiable soils profile to perform pushover analyses.
2) Estimate the abutment displacement for a range of restraining forces from foundation piles by considering failure surface passing through each liquefiable soil layers. For example, Figure 2 shows two independent failure surfaces (upper and lower) for two layers of liquefiable soils profile to perform slope stability analyses.
3) Determine the compatible displacements for each liquefiable soil layers based on the intersections of the relations established in steps 1 and 2. For example, Figure 3 shows the compatible displacements D_U (for upper liquefiable layer) and D_L (for lower liquefiable layer) for two layers of liquefiable soils profile.
4) Add all the compatible displacements (from step 3 above) together to determine multi-linear displacement pattern as a function of depth. Figure 3 shows the displacement pattern for two layers of liquefiable soils profile.

The proposed method directly assumes that liquefaction-induced lateral spreading occurs in all the liquefiable layer such that displacement in each liquefiable layers is independent of displacements in other layers. Moreover, no displacement is assumed to occur in the non-liquefiable soil between liquefiable layers. It is noted that these assumptions may not be able to fully capture the actual interaction occurring between the multiple liquefiable and non-liquefiable layer. In fact, it is reasonable to expect that displacement of upper liquefiable soil can be reduced by the occurrence of liquefaction in the subsequent lower liquefiable soil layers. Nevertheless, the purpose method will help to estimate approximate displacement profile pattern that can be used in conjunction with the current design guideline in situations where no information is available for soil displacements.

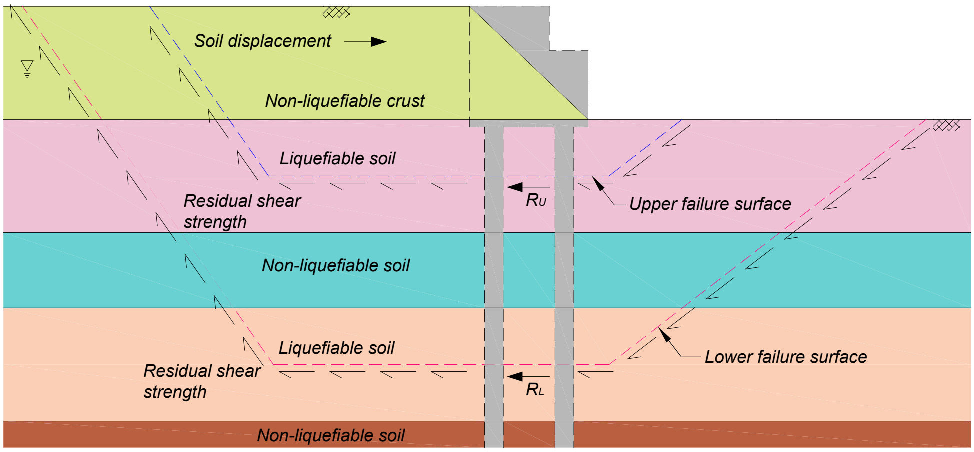

Figure 2: Schematic for failure surfaces in multi-layered liquefiable soils

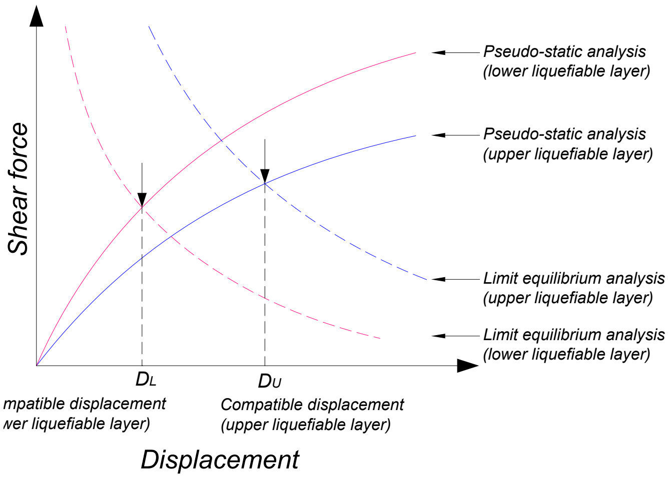

Figure 3: Schematic for force-displacement relations in multi-layered liquefiable soils

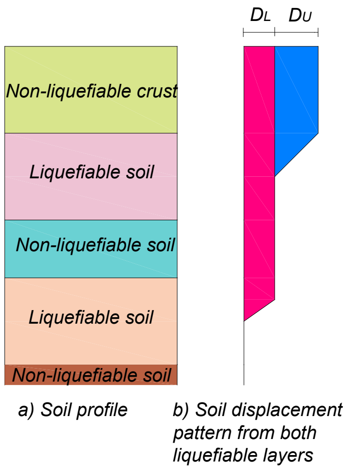

Figure 4: Schematic for lateral displacement pattern in multi-layered liquefiable soils

Illustrative example in two-layered liquefiable soils profile

Soil profile and foundation layout

An example is demonstrated to illustrate the proposed methodology for assessing the performance of abutment piled bridge foundation in multi-layered liquefiable soil profile. The idealized soil profile consists of two (upper and lower) liquefiable layers having 4-m and 2-m thick loose sand, respectively; the upper loose sand layer is overlain by 6-m thick crust of sand backfill and underlain by 2-m thick medium-stiff clay, followed by the lower loose sand layer, which is underlain by 4-m dense sand. The water table is assumed to be located at 3-m below the ground surface. The bridge abutment is founded on 1-m diameter and 14-mm thick steel pipe piles grouped in 3 x 7 layout. The axial load on each pile is assumed to be 1000 kN. The schematic of soil profile and foundation details with typical engineering properties for the materials and foundation are presented in Figure 4.

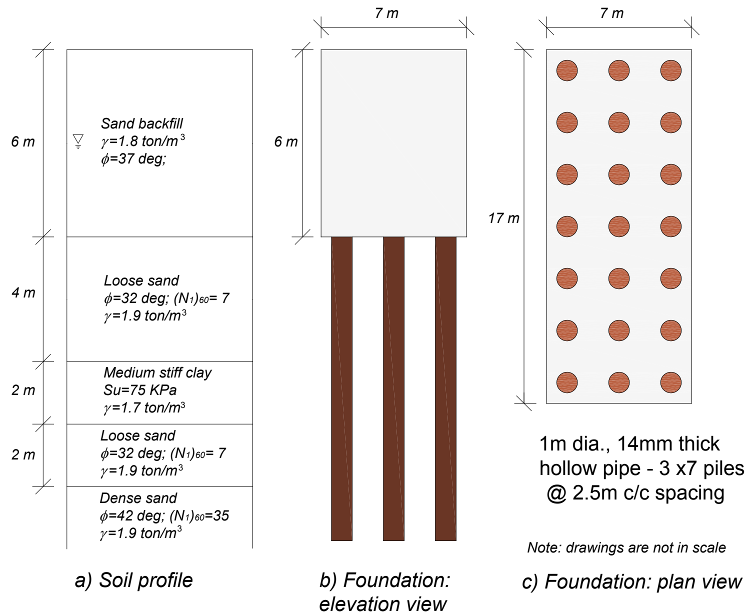

Figure 5: Soil profile and abutment foundation layout

Pushover analysis of abutment foundation

Pushover analyses of bridge abutment foundation were carried out using a beam on nonlinear Winkler foundation (BNWF) model as described in CALTRANS (2011). The LPILE 2013 is used for the pushover analyses.

The pile group section is converted into an equivalent non-linear single pile for the use in pushover analyses and the abutment section is modeled as 100 times stiffer in bending than the equivalent single pile. The rotational stiffness of pile cap is estimated as $1.1\text{x}10^7$ kNm/rad. The lateral response of soils are used in the pushover analyses using *p-y* curves. The interaction of abutment and crust is accounted by using user-defined p-y curve with tri-linear force-displacement relationship having ultimate lateral resistance of 3105 kN/m and full mobilizing displacement for this lateral resistance is estimated to be 0.11 m. The upper and lower loose sand were modeled as "soft clay (Matlock)" model, with residual strength of 8 kPa and 10 kPa computed using Kramer (2008); the medium stiff clay is modeled using "Stiff Clay with Free water (Reese)" model; and bottom dense sand layer is modeled as "API sand (O' Neill)" in LPILE. Group effects are considered using p-multiplier estimated for leading and trailing rows using Mokwa and Duncan (2001), with average group multiplier 13.23 (= 0.63 x21). Group effects are ignored in liquefied zone. The smearing effects of liquefiable soils are considered within 1-diameter of pile. For simplicity, unseating type of bridge deck is assumed so that lateral resistance from deck is not included in the pushover analysis.

A series of lateral displacements (with pattern shown in Fig 1) are imposed and shear forces are computed at the mid of both upper and lower liquefiable layer. Running average shear forces are used for computing the compatible force-displacement states as shown in Figure 6.

Slope stability analysis and estimated deformation

Slope stability analyses models are developed in SLOPEW (GeoStudio 2012). The loose sand were model as cohesive material with cohesive strength as the residual strength of liquefied soil. Two models are developed (as shown in Figure 2), in which the failure surface is constrained to pass through the mid of the liquefiable layer. The lateral extend of failure surface was constrained to 4H (where H is the height of abutment). Limit equilibrium analysis was carried out using Spencer (1967) method. As mentioned earlier, unseating type of bridge deck is assumed so that deck does not provide any passive resistance to the abutment movement i.e. F_{deck} =0 kN/m. A series of resisting forces are applied at the center of respective liquefiable layers (i.e., both upper and lower), and corresponding series of k_y are computed such that the slope FS=1.0.

Newmark rigid sliding block analysis is used to estimate the displacement corresponding to *ky*. The predictive model of Bray and Travasarou (2007) is used to estimate the displacement corresponding to *ky* obtained in slope stability analysis. The design earthquake level hazard considered in this example have magnitude M_w=8.0 and peak ground acceleration of 0.40g. Assuming crest width =17m and embankment slope 2H:1V, the tributary width of embankment is estimated as 25.5m. The resisting force per unit width used in the slope stability analysis, is multiplied with this width to estimate the forces. The force-displacement curves obtained from slope stability and deformation analysis are shown in Figure 6.

Compatible displacements

The compatible force-displacement states for upper (DU) and lower (DL) liquefiable soil are determined graphically as shown in Figure 6. The compatible displacement for upper and lower liquefiable layer is determined to be 8.4 cm and 5.1 cm, respectively.

Evaluating performance of foundation

The displacement pattern having maximum displacement of 13.6 cm (i.e. DU+DL) with the shape shown in Figure 4 is imposed in the BNWF model. Figure 7 shows the plot displacement, moment, and shear force along the depth. From the figure, it can be observe that patterns for overall responses are as expect and similar patterns are observed for lower liquefiable layer case and combined one. The maximum the amplitude of responses (e.g. displacement, moment, and shear) is consistently greater for combined upper and lower layer condition than considering single either upper or lower layer. Table 1 shows the maximum responses obtained from these displacement patterns. In this particular example, in combined case, the maximum pile displacement is ~10-55%, the maximum moment is ~40-55%, and the maximum shear is ~ 40-70% higher than considering soil movement in either upper or lower liquefiable layers only.

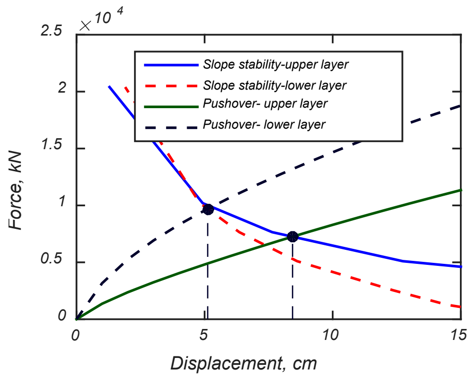

Figure 6: Force-displacement curves for design example

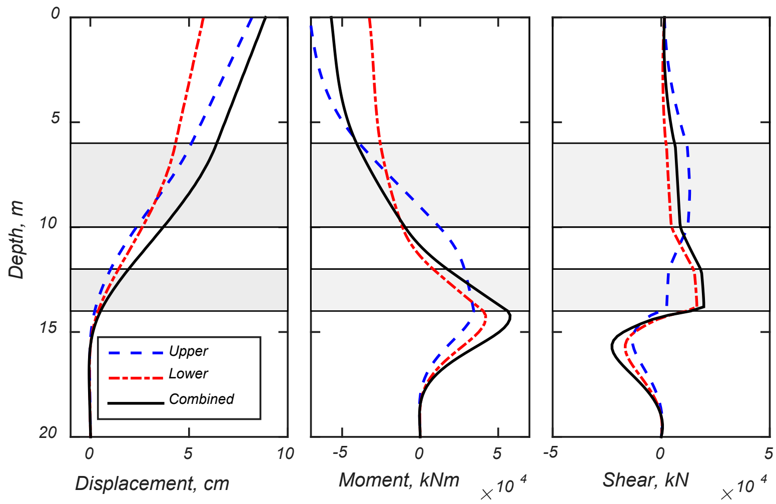

Figure 7: Responses of pile foundation

Table 1: Comparison of maximum responses for different soil displacement patterns

Failure surface cases	D_{soil} (cm)	D_{pile} (cm)	M_{pile} (kNm)	V_{pile} (kN)
Upper layer disp. (DU)	8.4	8.2	36740	13305
Lower layer disp. (DL)	5.1	5.7	41986	16604
Combined DU+DL	13.6	8.9	57622	22672

Discussion

It is important to note that the estimated compatible displacement may varies significantly depending on the simplification used in the pile pinning approach (e.g. foundation model, parameters used for soil models, and method of analyses). The objective of this paper is to demonstrate how to account the effects of multi-layered liquefiable soil in pile pinning approach. Besides all other variability in the pile pinning approach, results from the sample example showed that the proposed method gives conservative estimate (i.e., upper bound) in terms of pile foundation responses. The proposed method is based on an assumption thus is just a concept, and therefore future research with physical model tests are warranted to further validate the method for accounting the pile pinning effects in multi-layered liquefiable soil deposits.

Conclusions

In this paper a simplified extended design method is presented for using pile pining approach in determining the lateral displacements pattern in multi-layered liquefiable soil deposits. An illustrative example is demonstrated for two-layers of liquefiable soils sandwiched between three non-liquefiable soil layers. The results from the sample example showed that the proposed method gives conservative estimate (i.e., upper bound) in terms of pile foundation responses compared to current procedures; however future research with physical model tests are warranted to further validate the method in accounting the pile pinning effects in multi-layered liquefiable soil deposits.

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