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Design of Piles in Liquefied Soils for Combined Inertial and Kinematic Demands

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ABSTRACT

The design of piles penetrating liquefiable soils poses a number of challenges, including 1) accounting for the effects of liquefaction on lateral soil springs (p-y), 2) estimating liquefaction-induced kinematic demands (lateral spreading displacements) including pile-pinning effects, and 3) combining the effects of inertial demands from superstructure and kinematic demands from liquefied ground. This paper illustrates two approaches to address the above questions: A) a simplified decoupled approach adopting equivalent static analysis (ESA), and B) a detailed, coupled approach adopting nonlinear dynamic analyses (NDA). These approaches are presented through example projects showing the limitations and the potential conservatism associated with the simplified decoupled ESA approach, the benefits of using more rigorous coupled NDA approach, and recommendations on how to modify ESA methods, when used, to approximate outputs from advanced NDA for slope instability-induced demands on piles. These modifications include using pile-restrained, rather than free-field, soil displacements in the ESA, to reduce conservatism associated with these methods.

Introduction

Previous earthquakes have shown that liquefaction-induced lateral spreading can cause extensive damage to deep foundations. A number of studies have focused on the response of pile foundations in liquefied soils using physical models, numerical models and case studies (e.g. Martin et al. 2002; Boulanger et al. 2007). However, the guidance varies in terms of how to modify soil springs (p-y) to account for the effects of liquefaction, and how to combine kinematic demands from laterally spreading ground with inertial demands from the superstructure. This paper describes two approaches used in the design of piles in liquefied soils through two illustrative examples.

The first example explains a simplified decoupled equivalent static analyses (ESA) approach based on Caltrans (2012) guidelines. The simplified approach is used to assist with the foundation design of a 90-meter long bridge in Northern California supported by large diameter cast-in-drilled-hole (CIDH) piles penetrating potentially liquefiable, loose granular soils. To reduce uncertainty in estimating lateral spreading displacements, 2D dynamic effective-stress analyses were performed as opposed to simplified empirical methods. The kinematic and inertial demands were then combined and applied to the piles pseudo-statically.

The second example illustrates the detailed fully coupled approach in the foundation design of a cruise ship terminal in the Dominican Republic. The facility is located in a region of

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high seismicity and subsurface soils include liquefiable medium dense granular strata and soft normally-consolidated clays; hence, the trestle foundations had to be designed for kinematically-induced demands. Two-dimensional nonlinear dynamic analyses (NDA) were performed in FLAC, in combination with the UBCSAND constitutive model that enables dynamic effective-stress modeling of soil liquefaction in addition to embedded pile and superstructure elements. The analyses were used to guide substructure design decisions, including size of piles and final trestle geometry.

Example 1: Decoupled Equivalent Static Analysis (ESA)

Background

The Riverfront Reconnection project consists of an approximately 90-meter long new bridge ramp in Sacramento, California. The ramp structure will be supported on five bents and will be cantilevered over Interstate Highway 5. Each bent will be supported on a single, 2.4-meter diameter CIDH pile. The subsurface soils include approximately 18-meter loose to medium dense non-plastic silts and sands overlying denser strata. The loose to medium dense sand was found to be susceptible to liquefaction using procedures by Idriss and Boulanger (2008) for the 975-year return period PGA of 0.3g and design earthquake magnitude equal to 6.9. The liquefaction-induced horizontal soil displacement associated with the elevation difference at the retaining wall will impose kinematic loads on the foundations and the proposed bridge (Figure 1a).

Approach

The loads and deformation demands on the bridge foundations associated with liquefaction-induced lateral spreading were computed using the procedures presented in Caltrans (2012). The recommended design procedures are based on conducting Equivalent Nonlinear Static Analyses (ESA) where the foundation is loaded by a laterally displacing soil mass in conjunction with inertial loading from the superstructure. Analyses were conducted using the software LPILE 6.0 for four cases:

- Case 1 – No Liquefaction: lateral pile analyses were conducted for a loading scenario with 100% of inertial loads estimated using spectral elastic analyses.
- Case 2 – With Soil Liquefaction: lateral pile analyses were performed for a loading case considering 100% kinematic + 50% elastic inertial loads. A reduction factor equal to 0.5 is applied on the inertial loads to account for the fact that individual peaks of the contributing dynamic loads and kinematic demands generally occur at different times. Kinematic loads associated with liquefaction-induced soil movement were developed based on the results of dynamic effective-stress analyses. Kinematic soil demands were imposed by means of depth varying “free-field” soil displacements. These springs in liquefied layers were factored using excess pore pressure-dependent p-multipliers to reduce the ultimate capacity along the length of liquefiable soils.
- Case 3 – No Liquefaction/Plastic Demands: lateral pile analyses were conducted using superstructure plastic demands provided by the structural engineer.
- Case 4 – With Soil Liquefaction/Plastic Demands: In the liquefied case, soil springs were factored with p-multipliers similar to Case 2 above. Similarly, kinematic soil demands were imposed by means of depth varying “free-field” soil displacement in the liquefied case. Finally, plastic demands were applied at the shaft head. Similar to Case 3 this case was analyzed to ensure that CIDH piles are capable to resist ultimate plastic demands

from the superstructure in terms of shear forces and bending moments along the piles. Unlike Case 2, no reduction in inertial plastic demands were applied as proposed by Khosravifar et al. (2014). This was considered as an upper bound scenario.

Free-field liquefaction-induced soil displacements

The existing simplified empirical methodologies available to estimate liquefaction-induced lateral spreading include considerable uncertainty and are not appropriate for the geometry and boundary constraints of this project. Therefore, two-dimensional effective-stress dynamic analyses were conducted in FLAC to refine the estimates of liquefaction-induced displacement demands in the vicinity of the proposed foundations (Figure 1b). FLAC analyses were also used to estimate average excess pore-water pressure (PWP) ratios in the sand layers to modify p-y curves (p-multipliers) per Caltrans (2012). Three input ground motions were spectrally matched to 975-year target spectra and used as input to the dynamic analyses.

Results

Figure 2 presents the detailed lateral pile analyses results for Case 2 load combination (100% of the kinematic demands and 50% of inertial demands). The figure presents depth varying: a) idealized p-multiplier used to modify the soil springs in sandy layers, b) horizontal kinematic soil demand and estimated pile deflection, c) soil reaction, d) shear force demand, e) bending moment demand. The estimated pile head deflection is approximately 7 cm for a case where soil displacement is about 0.3 meter at depths of 9 to 12 meters.

The total additional force transferred on the soil as a result of the pile deflection is estimated equal to approximately 3 MN over a vertical length of about 5.5 meters from the top of the shaft. Given the close proximity of the pile to the retaining wall, and the large magnitude of the additional force (e.g., larger than the estimated at-rest soil pressure), it was recommended that foundation isolations be used to reduce the potential of additional loading on the wall.

Example 2: Fully Coupled Nonlinear Dynamic Analysis (NDA)

Background

The Amber Cove Cruise Ship Docking Facility is located in Maimon Bay near Puerto Plata, Dominican Republic. The docking facility was designed to accommodate two cruise ships and includes a main pier (400 m long supported on 30 bents), trestle (200 m long supported on 17 bents), and mooring and berthing dolphins. Each bent is supported by a 1-by-3 pile group of open-ended steel pipes with outer diameter of 0.76 m spaced at 3.5 m center-to-center.

The project is located in a highly seismically active area, just within 15 km of the Mouchoir Bank subduction source to the north and additional local faults with large magnitude potential (M~7.5). Probabilistic seismic hazard analyses (PSHA) were conducted which incorporated local faults to develop design acceleration response spectra for the project location (Fugro, 2013). The estimated PGA was an outstanding 0.98g at rock for the MCE design level.

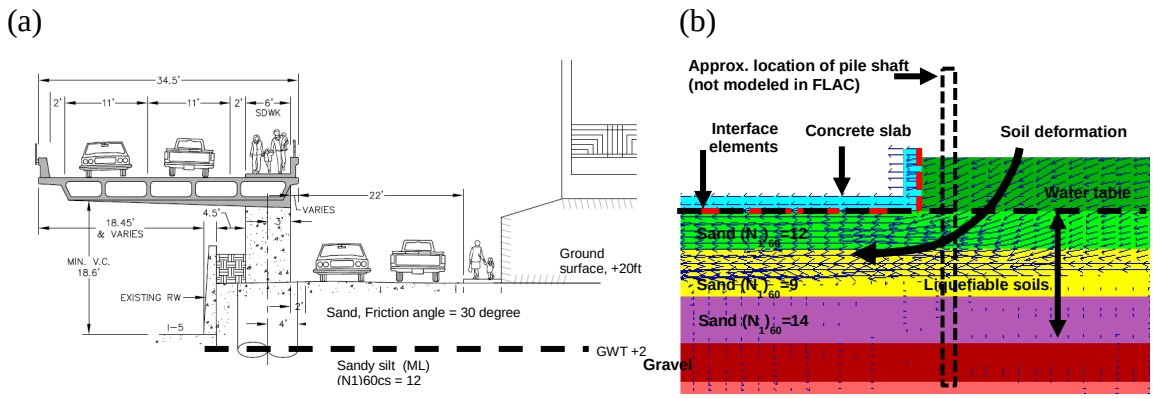

Figure 1: (a) Schematic cross section of riverfront bridge ramp, (b) FLAC model.

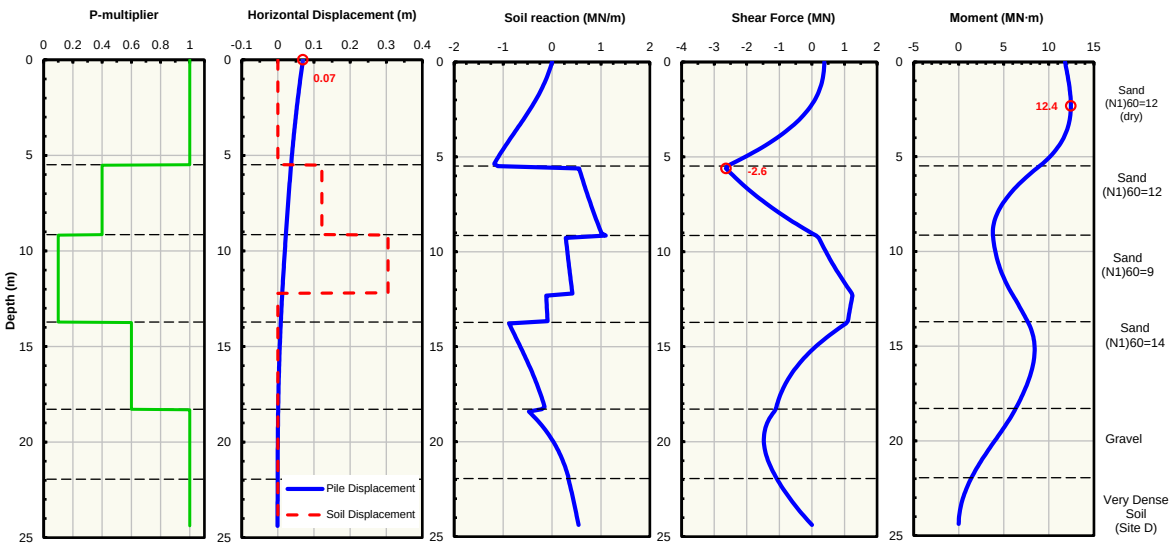

Figure 2: Profiles of p-multipliers, pile and soil displacements, shear, and moment for ESA Case 2 (100% kinematic + 50% elastic inertia).

In general, the subsurface condition consists of soft clayey silt with varying thickness from about 10 to 20 meters, overlying medium dense carbonate sand with varying thickness up to 5 meters, overlying gravel and rock. Despite its relative density the sand strata were shown to be liquefiable during the very high levels of shaking for the project MCE. The high seismic hazard in combination with the presence of very soft clays and liquefiable carbonate sands added to the challenges involved in developing a robust design for the pile foundations. Hence for this project a fully coupled NDA approach was adopted to estimate liquefaction-induced and slope instability demands on the pile foundation.

Approach

Preliminary slope stability analyses using simplified procedures showed that the seismic factor of safety (FOS) for the trestle area is below one with estimated seismically induced free-field displacements ranging up to 6 meters (Fugro, 2013). To more accurately

characterize the seismic displacements and their kinematic demands on the piles, 2D dynamic analyses were performed in FLAC. Figure 3 presents the FLAC model developed for a representative longitudinal cross-section along the trestle. The model included soil elements, one row of 17 piles, and the trestle deck. A spring element was added to model the resistance of the main pier section that is perpendicular to the offshore end of the trestle.

The dynamic response of the liquefiable sand layers (Sand1 and Sand2 on Figure 3) was modeled using UBCSAND constitutive model. This model has been extensively calibrated and validated and has been recently used in the vulnerability assessment of important structures (Travasarou et al., 2012). The changes in the strength of granular soils as a result of the development of excess pore pressure during cyclic loading and the resulting contraction and dilation of the granular soil and the flow of water are explicitly simulated in time domain in these analyses. The Clay, Silt, Gravel, Coral and Fill were modeled using the Mohr-Coulomb failure criterion in combination with a nonlinear stress-strain behavior characterized by a three-parameter sigmoidal shaped backbone curve. The Rock layer and the half space (below elevation -60 meters) were modeled as elastic materials with constant elastic modulus. The model was subjected at the base to one-directional horizontal dynamic

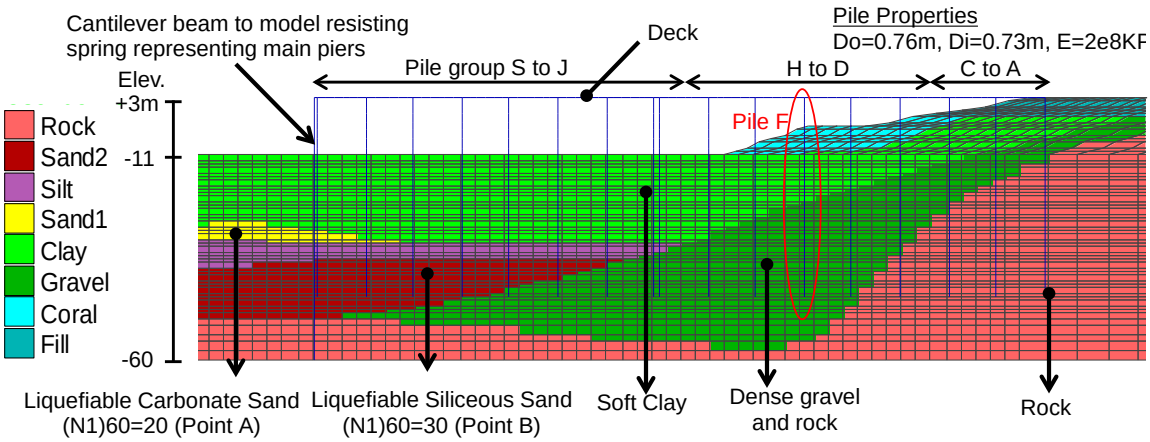

Figure 3: FLAC model of the representative longitudinal cross-section along the Trestle.

loading using the time histories that were spectrally matched to the target MCE and DE spectra developed for stiff soil. Dynamic excitation was specified using the compliant-base deconvolution procedure.

Calibration of a single pile response

The pile elements are connected to the grid nodes using vertical (t-z) and lateral (p-y) bilinear springs developed in general accordance with API (2008) for each pile specifically. Spring properties include the group effect factors, however, they do not include the effects of dynamic loadings, since these effects are captured through the nonlinear behavior of the soil elements. The piles are modeled using elastic elements. Figure 4 shows the results of a pushover analysis on a single pile. Comparison of displacement, shear and moment profiles with the results from LPILE shows general agreement between FLAC and LPILE. This step is important when performing NDA to ensure that the single pile response is adequately modeled.

Results

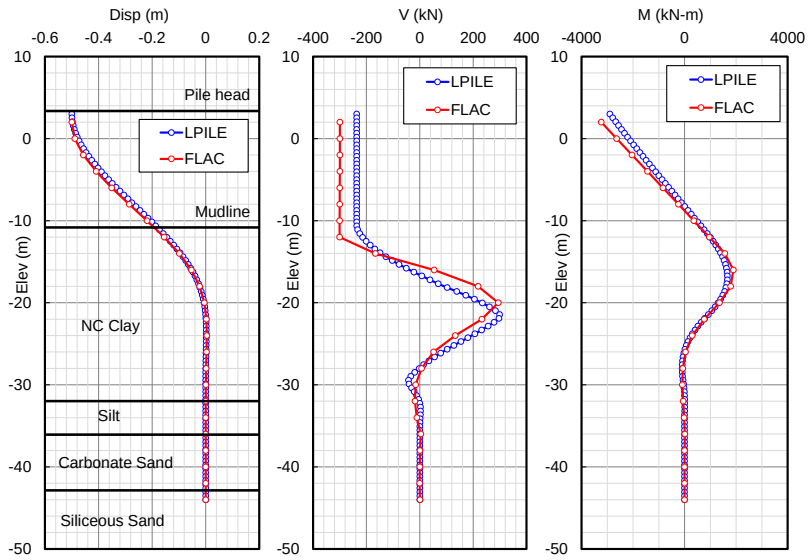

Figure 4. Calibration of lateral response of a single pile in FLAC compared to LPILE

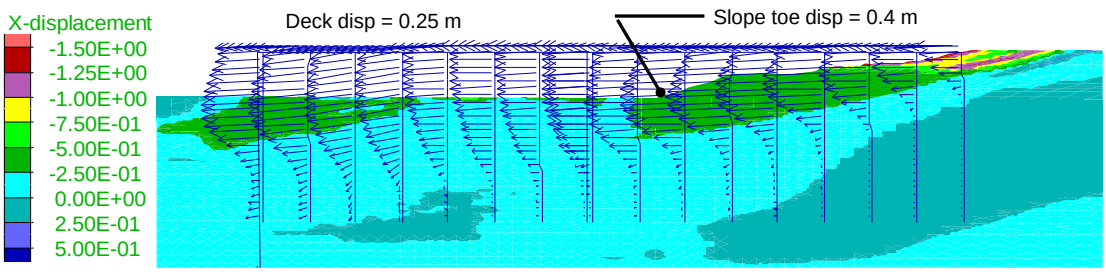

Figure 5: Horizontal soil displacement contours at the end of a design-level motion

Figure 5 shows horizontal soil and pile displacement at the end of motion. Permanent slope displacements are primarily associated with liquefaction in sand layers and failure through the soft clay layer. The soil displacements are reduced due to pile-pinning effects compared to free-field displacements (no pile) where slope displacements at toe is about 3.5 m (not shown here).

Figure 6 shows the time histories of horizontal displacement of slope and deck and excess pore-water pressure ratios of two points in the middle of liquefied layer shown in Figure 3. Relative displacements between slope and deck start to develop after about 10 seconds corresponding to the time of pore-water pressure ratios approaching one (1).

To better characterize the residual demands on the piles due to the soil movement, structural demands were extracted at the end of the design-level motions and plotted on Figure 7. The moment capacity is also plotted for reference as a vertical dashed line (i.e., 3 MN·m). The piles that tipped in liquefied sand (Bents Q to S) showed some displacement at tip elevation due to soil liquefaction. Overall, transient demands during shaking (including both inertial and kinematic effects) may be higher than the residual demands.

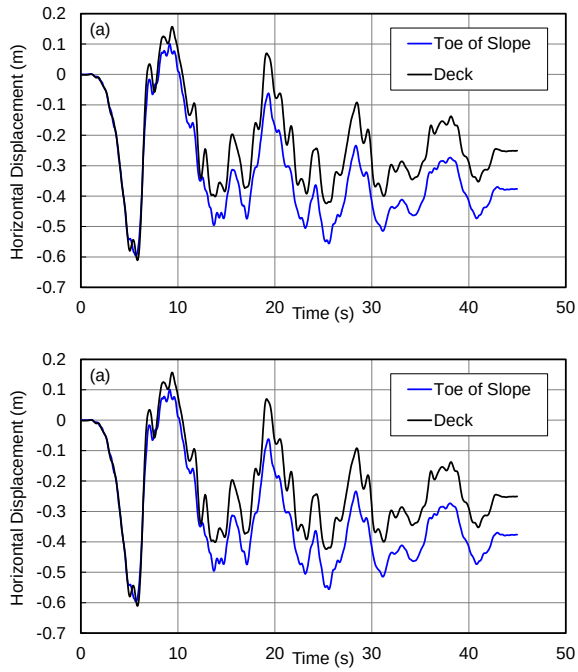

Figure 6: Time histories of (a) slope and deck displacements and (b) excess pore-water-pressure ratios in the middle of liquefied layers during a design-level motion.

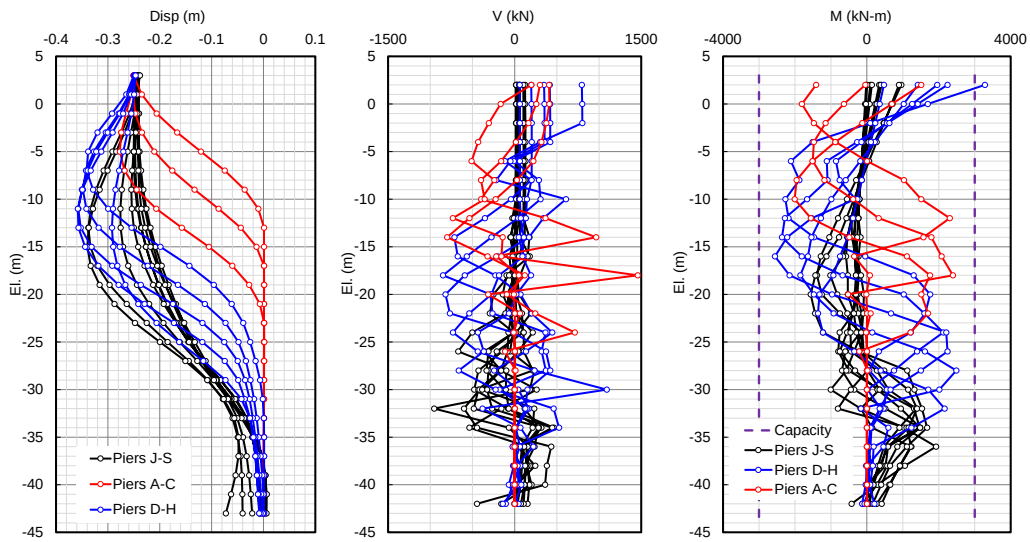

Figure 7: Displacement, shear and moment pile demands at the end of a design-level motion.

The results of NDA were compared to those obtained from simplified Equivalent Static Analyses (ESA) to assess the level of conservatism of these simplified approaches and explore potential modifications to improve their accuracy. The results of these comparisons are shown for Pile F (as an example) on Figure 8. In the first approach, the free-field soil displacements (blue dashed line on Figure 8a) are applied pseudo-statically to the pile using LPILE. The free-field soil displacements are estimated from a separate dynamic analysis in FLAC without the piles, to simulate free-field conditions (similar to Example 1). Alternatively, the free-field displacements can be estimated using slope stability analysis and

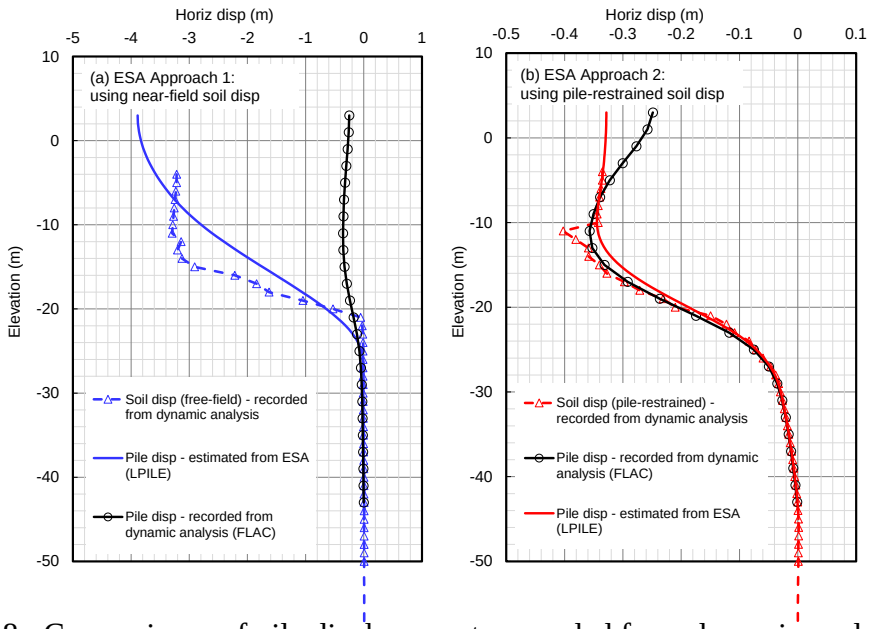

Figure 8: Comparisons of pile displacements recorded from dynamic analysis and estimated from two different ESA approaches: (a) using near-field soil displacements, and (b) using pile-restrained soil displacements.

simplified procedures (e.g. Bray and Travasarou, 2007). The pile displacements estimated using this approach (blue solid line) significantly overestimates the pile displacements obtained from the coupled-dynamic analysis (black line). This overestimation is due primarily to the free-field soil displacements being larger than "near-field" pile-restrained soil displacements.

In the second ESA approach (Figure 8b), pile-restrained soil displacements are used as input, estimated from the coupled-dynamic analysis which includes the effects of soil-pile interaction (red dashed line). The pile displacements estimated in this approach (red solid line) compare better with the pile displacements obtained from dynamic analysis (black line) which shows the importance of using realistic soil movements (pile-restrained) as input to ESA. In a forward analysis, the pile-restrained soil displacements can be estimated by incorporating pile-pinning effects as described in Caltrans (2012). Note that the difference between the pile displacements from NDA and ESA at shallow depths is primarily due to the restraining effect of the perpendicular main pier that was modeled in the NDA but cannot be simply modeled in ESA. The ESA approaches above estimate residual kinematic pile demands only. To combine the kinematic demands with inertia, similar approaches described in Example 1 can be used.

Concluding Remarks

Two different methods with different levels of complexity were adopted to assess liquefaction-induced horizontal demands on piled foundations for two projects of different magnitude of scope and budget. In the first example, lateral analyses of piles for a bridge foundation were performed using a decoupled Equivalent Static Analyses (ESA) following recommended guidelines by Caltrans (2012). Due to complexities in the problem geometry, dynamic analyses were used to better estimate liquefaction-induced soil displacements for this project.

The second example illustrated a detailed approach using nonlinear dynamic analyses (NDA)

for the design of piles in liquefied soils for a trestle structure subjected to lateral spreading. The first step in these analyses included calibration of the finite-difference model and interface springs to appropriately simulate the lateral load-deformation response of a single pile. This step is essential to ensure adequate response of the soil-pile model. The NDA analyses inherently accounts for the effects of liquefaction on inertial loads, and the combination of inertia and kinematic demands. Using NDA for this project allowed to optimize the size of piles compared to what would have been selected based on decoupled ESA.

The primary advantage of using equivalent static analyses (ESA) is related to speed and cost effectiveness associated with problem simplification. However, simplifying a dynamic response with liquefaction into a static problem involves assumptions which have inherent limitations and are usually conservative. Conservatism in estimating slope instability-induced pile demands using ESA can be reduced by incorporating pile-restraint, rather than free-field soil displacements. When project schedule and budget permit, more rigorous NDA can be used to reduce uncertainty and conservatism in estimating slope instability-induced demands. These analyses appropriately model the time-dependence of the system's dynamic response, pile pinning effects, transient character of excess pore-water pressure affecting the soil spring in the case of soil liquefaction, and combination of inertial and kinematic demands. Despite their increased complexity and cost these analyses can be performed efficiently by qualified engineers and given appropriate calibration and validation generally result in reduced conservatism, optimizations in design and savings during construction.

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