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A SIMPLE NONLINEAR DESIGN MODEL FOR DRILLED SHAFTS IN COHESIVE SOIL

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ABSTRACT

A recent study conducted at Iowa State University (ISU) identified deficiencies in the current practice of accounting for soil-foundation-structure-interaction (SFSI) in the design of drilled shafts in cohesive soils. The existing simplified methods are unable to capture the lateral response of a soil-shaft-column system accurately over the elastic and inelastic ranges, thus making them inapt for seismic design. Following a demonstration of the shortcomings and limitations of the existing methods, a new approach suitable for seismic design of drilled shafts in cohesive soils is presented. The new approach models a column supported on a drilled shaft as a cantilever with a flexible base and the inclusion of soil resistance through the use of three springs, one rotational and two translational. This approach is shown to accurately capture the lateral response envelope of columns supported on drilled shafts, validating its potential application in future design practice.

Introduction

Currently, a significant amount of research is being performed in the area of SFSI in order to better understand the lateral response of structures in different soil types. As part of an ongoing research effort in the field of SFSI, a recent project at ISU has examined the lateral load behavior of continuous column-foundation systems in cohesive soils (see Fig. 1). The type of system examined was the bridge column that extended into the ground as a cast-in-drilled-hole (CIDH) shaft since this design is used in practice today due to the simplicity of construction, elimination of column-foundation connection and reduced construction costs.

Although CIDH shafts are commonly used in seismic regions because of the aforementioned benefits, the project identified a deficiency in the current practice of accounting for SFSI in the design of drilled shafts in cohesive soils. Currently, several different models exist for simplifying the Winkler soil spring concept that accounts for SFSI in cohesive soils that generally rely on the determination of an equivalent cantilever system with a base fully fixed against deformation, no surrounding soil and an effective height to the fixity point shown in Fig.1. None of the models, however, are able to capture the response of the system in both the elastic and inelastic ranges during a design level or greater seismic event. This paper presents an investigation into the existing models in order to demonstrate some of these deficiencies and

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provides a new approach to capture the lateral response of the system in both the elastic and inelastic ranges with the potential to provide information on local and global design parameters.

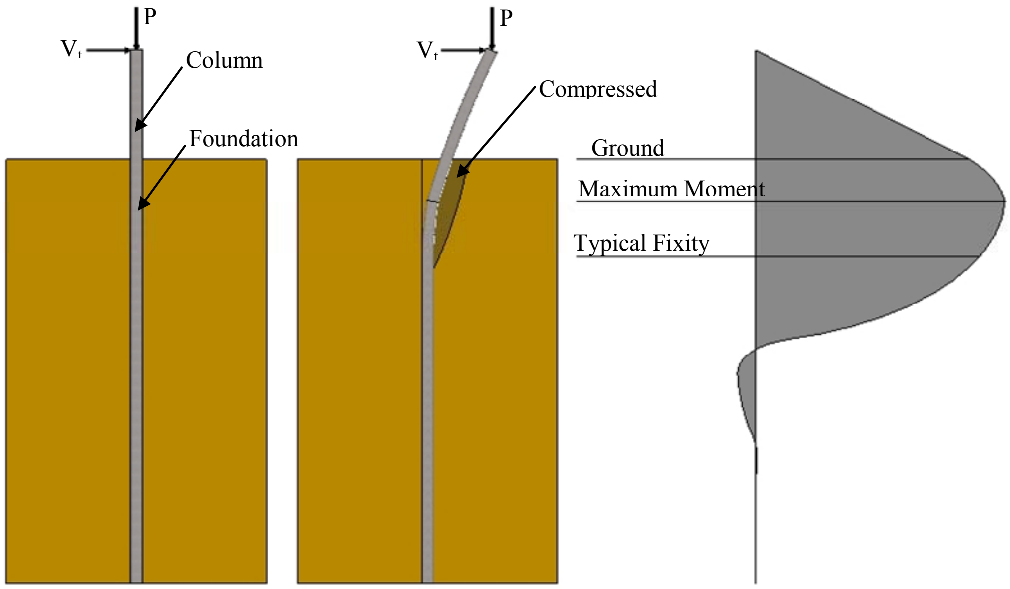

Figure 1. Expected lateral response of a bridge column supported on a CIDH shaft

Current Design Models

To minimize the complexity and to reduce the number of iterative steps needed for analysis and design when using the Winkler soil spring concept, many models have been developed in order to simplify the process of accounting for SFSI in the design of drilled shafts [e.g., Chai (2002), Priestley et al. (1996), Priestley et al. (2007)]. However, none of the models are able to capture the SFSI effects on the lateral load response throughout the entire loading range expected during design-level or greater seismic events. Furthermore, these models have not given consideration in quantifying all local design parameters. This section of the paper provides a summary of an in-depth investigation into some of the more common approaches recommended for use in practice.

Guide Specifications for LRFD Seismic Bridge Design (AASHTO 2009)

In the seismic design guidelines published by the American Association of State and Highway Transportation Officials (AASHTO), multiple methods are presented for determining the lateral response of pile foundations based on site location, bridge design and importance. Methods, from simple to complex, are discussed within the main guideline sections and additional simple methods are identified within the commentary. The complex method presented within the guidelines uses the soil spring methodology, in which p-y curves are created and placed along the shaft length in order to determine the structural equilibrium through a numerical iteration process. Although the accuracy has shown to be very good (e.g., Reese et al. 2004 and Sritharan et al. 2007,), this method requires a significant amount of knowledge on the surrounding soil parameters and the combined experience in geotechnical and structural fields.

The simple method models the bridge column with a shaft extended to an equivalent point of fixity, near the location identified in Fig. 1, within the soil using empirical means. The empirical equation determines the point within the soil that allows for the column/foundation system to be represented as an equivalent cantilever where the extended shaft is modeled without any surrounding soil and is fully constrained at the base from experiencing any deformation. The top end of the column/shaft unit is modeled using the constraints imposed by the bridge superstructure. The equivalent point of fixity within the system is located by determining the extended length of the shaft from an equation that is a function of the ratio between the flexural rigidity of the pile and a soil modulus using empirical means. The main deficiency of this approach is that it does not accurately capture the maximum moment location in the shaft as this point does not generally occur at the point of fixity, but rather above this point in most cases (Figure 1C). The maximum moment location is critical in design as this will determine where the confinement reinforcement should be provided. When plastic hinges are not designed correctly, a brittle failure of the foundation shaft will most likely ensue. Other deficiencies of these code-based methods may be summarized as follows: 1) no validation for the estimated fixity point is given; 2) no clear information is provided on how plastic action in the shaft will be included when estimating the lateral column displacement; and 3) shear demand in the column and foundation shaft are assumed to be constant.

Within the commentary of AASHTO (2009), additional simple approaches are suggested in lieu of the point of fixity method if the systems are believed to not behave in a linear elastic manner. This is the case in most seismic design situations and some of the models suggested to deal with these design problems include Chai (2002) and Priestley et al. (2007).

Chai (2002)

Chai proposed a model to determine the flexural strength and ductility for the lateral response of extended column shafts while accounting for the effects of soil. The model relies on the use of two points, fixity and maximum moment, along the length of the system. A visual representation of the model and the two points defining the fixity and the maximum moment locations used to determine the lateral loading and displacements of the column/foundation system are shown in Fig. 2.

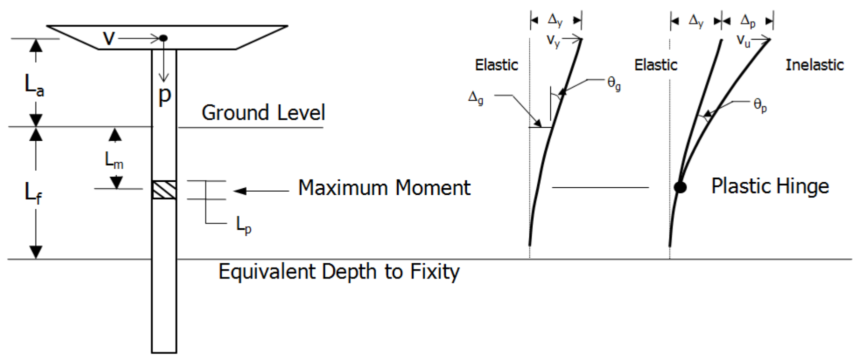

Figure 2. Equivalent fixed-base cantilever (from Chai 2002)

The method suggested for use by Chai was developed for both cohesionless and cohesive soils. For each soil type, the effective fixity location was determined by relating the stiffness of the soil-shaft system to that of an equivalent cantilever system using equations produced by Poulos and Davis (1980) and standard cantilever displacements. The maximum moment location was then determined by using a soil distribution along the length of the foundation shaft [e.g., modified version of Broms (1964) in a cohesive soil] to define shear and moment equations.

After defining the two key locations for the model, the plastic deformation of the foundation shaft was defined using an idealized elasto-plastic moment-curvature response at the section level, effectively ignoring the combined nonlinearity of soil, steel and concrete beyond the first yield limit state. The analytical plastic hinge length used in this process is that found through experimental means in Chai and Hutchinson (2002), which relates the plastic hinge length only to the above ground column height and varies between 1.0D and 1.6D, where D is the shaft diameter. This approach was suggested for use by Chai for both soil types even though the experiment was only performed in a cohesionless soil. By multiplying the distance from the maximum moment location with the plastic curvature and analytical plastic hinge length, the plastic deformation of the system is determined. Using the yield and plastic deformations, a relationship is produced that relates the displacement ductility of the system to the curvature demand of the foundation shaft to complete the method suggested by Chai.

The cohesive soil model proposed by Chai was compared with the full-scale outdoor test of Suleiman et al. (2006) to examine the efficiency of the model in cohesive soils. The comparison found that the maximum moment location was under predicted by 30%, the analytical plastic hinge length was under predicted by 36% and the initial secant stiffness to the yield point was 25% lower than the experimental stiffness. The global comparison of the results, provided in Fig. 3, shows that the idealized force-displacement response largely over predicts the yield point and fails to capture the secondary slope beyond the first yield limit state.

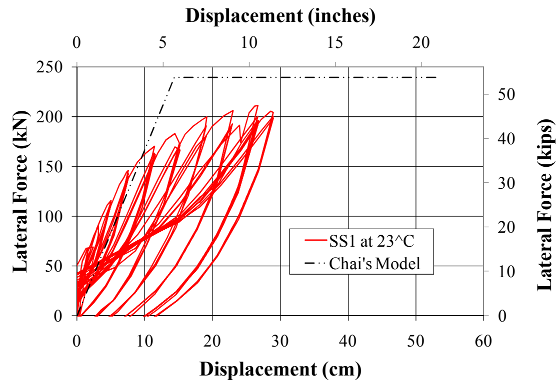

Figure 3. Global comparison of Chai's model with full-scale testing by Suleiman et al. (2006)

After noting the differences with the experimental test data, additionl shortcomings and limitations associated with this model are summarized as follows: 1) the perfectly plastic response between the yield and ultimate limit states ignores the strength gained from the combined nonlinear effects of the soil and steel reinforcement after yielding; 2) a structural analysis program may not be easily used for analysis purposes as it uses two characteristic lengths to define the maximum moment location and the fixity point; and 3) the plastic hinge length will generally be under-predicted since cohesionless soils are typically stiffer than cohesive soils.

Priestley et al. (2007)

The effort by Priestley et al. (2007) on displacement-based seismic design included the topic of bridge columns supported by CIDH shafts. To determine the design displacement values of this system, a model was introduced based off of the recommendations of Suarez and Kowalsky (2007). Accordingly, to determine the design displacement, the following steps are required: 1) locate the in-ground plastic hinge using nomographs or equations; 2) determine the yield and ultimate limit state curvatures using damage-control limit strains of steel and concrete; 3) find the length of the analytical plastic hinge based off of Chai's model (2002); 4) determine a coefficient to account for the effects of soil type and boundary conditions using nomographs or equations; and 5) find the yield and design displacements using the equations presented in Priestley et al. (2007). This process defines the design displacements, but does not provide an easy way to determine the lateral shear demand in the shaft for the seismic event.

Also, the cohesive model was created through a parametric study using OpenSees on medium to soft cohesive soils (Suarez and Kowalsky, 2007), with undrained shear strengths, s_u, of 20 kPa and 40 kPa (420 psf and 840 psf). The ability of the model to characterize columns supported by drilled shafts in stiff soils has not been studied and the model is therefore limited to soft cohesive soils only even though s_u of cohesive soils may be as high as 400 kPa (8350 psf). A shortcoming with this model is that the plastic hinge length is determined using the Chai's method (2002), which has been previously reported to generally under predict the analytical plastic hinge length in cohesive soils.

Proposed New Approach

Since current design models are unable to capture both the local and global responses of a continuous column/foundation system during a seismic event, a new model was developed that is to not only capture these responses but also to account for the effects of seasonally frozen clay due to its impacts detailed in Sritharan et al. (2007). To realize this approach, a free-body-diagram (FBD) of the expected deflected shape of the column-shaft system is shown in Fig. 4a. The system is cut at the maximum moment location, where the confinement will be needed, and the effective height of the model is defined from this location to the column top (see Fig. 4b). As shown, to accurately capture the lateral response, a minimum of three springs are required: two to represent the flexible foundation at the maximum moment location and one to account for soil resistance above this location. At the flexible base, the rotational spring captures the elastic rotation caused by movement below the maximum moment location as well as any plastic rotation occurring in the maximum moment region. The second spring is intended to capture the translation occurring at this point due to the lateral movement below the maximum moment location. The soil resistance above this location is modeled by a single translational spring located halfway between the maximum moment location and the ground surface with a stiffness based on the soil properties surrounding the CIDH shaft. Presented below is a summary of the proposed approach while derivations of all equations can be found in Shelman (2009).

Maximum moment location

In the new approach, the location of the maximum moment is first defined as this will determine the effective height of the system and identify locations of all three springs. In accordance with the terminology introduced in Fig. 5, the maximum moment location is found using Eq. 1, where α_{ma}, β_{ma} and χ_{ma} are coefficients based on the soil's undrained shear strength.

$$L_{ma} = D\left[\alpha_{ma}\left(\frac{L_{col}}{D}\right)^2 + \beta_{ma}\left(\frac{L_{col}}{D}\right) + \chi_{ma}\right],\ \text{where}\ \alpha_{ma} = -0.000005ac_u^{\,2} + 0.0003bc_u + 0.028,$$

$$\beta_{ma} = 0.0038bc_u + 0.3247,\ \chi_{ma} = -1.28\ln[c_u(\text{psi})] + 7.1307,\ \chi_{ma} = -1.28\ln[c_u(\text{kPa})] + 9.6021,\quad (1)$$

$$a = 1.0 \text{ for psi and } 0.021 \text{ for kPa, and } b = 1.0 \text{ for psi and } 0.145 \text{ for kPa}$$

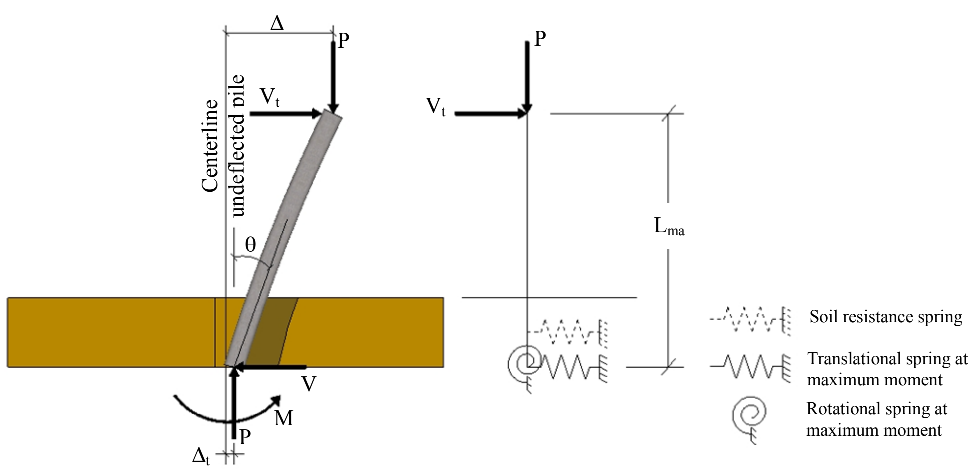

Figure 4. (a) FBD of column-foundation system; (b) proposed new approach

Zero moment location

The next point to be determined is the point at which the moment profile first crosses back over the zero point below the maximum moment location. This point is needed in order to determine the properties of the rotational and translational springs used in the flexible foundtion. The zero moment location is found using Equation 2, where α_{m0} and β_{m0} are coefficients based on the above ground height of the column.

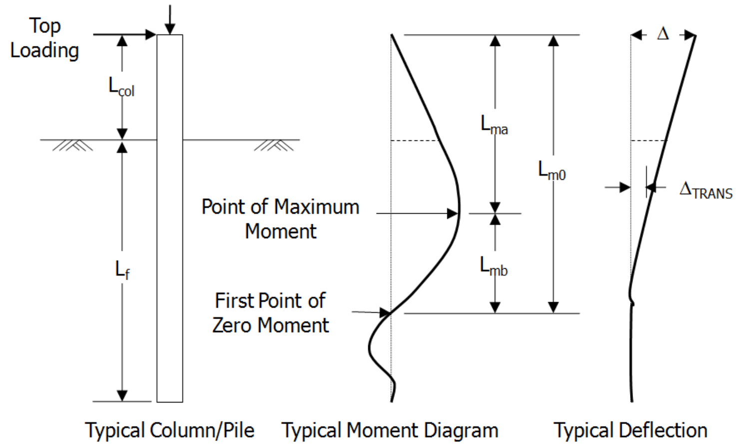

Figure 5. Definition of different variables used in the proposed new model

$$L_{m0} = D\alpha_{m0}\left[c_u(\text{psi})\right]^{\beta_{m0}} \text{ or } L_{m0} = D\alpha_{m0}\left[0.145c_u(\text{kPa})\right]^{\beta_{m0}}$$
$$\text{where } \alpha_{m0} = 0.11\left(\frac{L_{col}}{D}\right) + 22.3 \text{ and } \beta_{m0} = 0.021\left(\frac{L_{col}}{D}\right) - 0.33 \qquad (2)$$

Maximum moment translational spring

Once L_{ma} and L_{m0} are computed, the spring properties can be readily determined, starting with the bilinear response of the translational spring at the maximum moment location. The translation of the foundation shaft at the ultimate condition is found using Equation 3, while the translation at the yield condition is found using Equation 4.

$$\Delta_{tu} = D\left[0.0255\psi\left(\frac{L_{mb}}{D}\right) - 0.0652\right], \quad \text{where } \psi = 0.0157\left(\frac{L_{col}}{D}\right) + 0.9342 \qquad (3)$$

$$\Delta_{ty} = \frac{\Delta_{tu}}{4.37} \qquad (4)$$

In Equation 3, ψ is a correction factor used only if c_u is less than 70 kPa (10 psi); otherwise the value is input as 1. The lateral forces needed to activate the aforementioned displacements are computed by ensuring static equilibrium of the simplified model representing the system shown in Fig. 5.

Rotational spring properties

The rotational spring placed at the maximum moment location is defined next. In order to determine this bilinear moment-rotation spring, Eqs. 5 to 8 are used for the ultimate condition, where L_p is the analytical plastic hinge length used for calculating the plastic rotation, θ_p. Eq. 9 defines the elastic rotation below the maximum moment location at first yield, θ_{eby}. The moment value for each point shall be taken as the ultimate moment and the first yield moment of the foundation shaft, respectively.

$$\theta_{ebu} = 0.0031\frac{L_{mb}}{D} + 0.0006 \tag{5}$$

$$L_p = 2(0.16L_{mb}) \tag{6}$$

$$\theta_p = \varphi_p L_p = (\varphi_u - \varphi_e)L_p \tag{7}$$

$$\theta_u = \theta_{ebu} + \theta_p \tag{8}$$

$$\theta_y = \theta_{eby} = 0.002\frac{L_{mb}}{D} + 0.00001 \tag{9}$$

where θ_{ebu} = elastic rotation below the maximum moment location at ultimate condition; ϕ_p = plastic curvature of the section which is defined as the ultimate curvature, ϕ_u, minus the elastic curvature, ϕ_e, at ultimate condition; θ_u = rotation at the maximum moment at maximum moment; and θ_y = rotation at the maximum moment location when first yielding occurs.

Soil Spring

The final translational spring used within the system is a soil spring. This spring, which may be replaced by multiple springs, is placed halfway between the maximum moment location and the ground surface in order to account for the resistance provided by the soil in this region. By locating a soil spring within the system, the influence of seasonal freezing on the soil properties and thus the system behavior may also be accounted for. The seasonal temperatures effects on other construction materials can be addressed in the model by revising the moment-curvature response to account for the temperature effects on the material properties.

The properties of the translational spring can be found using the procedure for establishing a p-y curve as outlined by Reese (1975). If a hand calculation is performed to determine the tip lateral load and displacement, first determine the ultimate soil pressure, p_u, and multiply this by the height of the soil column to determine the resistance of the soil at the ultimate, V_{su}. When dealing with the yield condition, multiply this value by a coefficient, η, to adjust the soil resistance value to the limit state being analyzed. The coefficient, η, is presented in Equation 10 and was found experimentally using data obtained from the analytical models produced at ISU (Shelman 2009).

$$\eta = -0.03\ln[c_u\,(\text{psi})] + 0.7536 \text{ or } \eta = -0.03\ln[c_u\,(\text{kPa})] + 0.8115 \tag{10}$$

Force-displacement response

Simple calculations may be performed in a computer program (e.g., MS Excel) to determine the global response of the system using the summation of the following individual parts presented above: 1) the total elastic displacement of the system; 2) the total plastic displacement; and 3) initial translation, Δ_t, at the maximum moment location. The total elastic displacement is a summation of the displacement due to elastic rotation below the maximum moment location, Δ_{eb}, and the elastic displacement above the maximum moment location, Δ_{ea}, due to the cantilever action produced from loading applied at the column top. The initial translation at the maximum moment location is computed using either Eq. 3 or 4 depending on the limit state being analyzed. The total plastic displacement, Δ_p, is due to the plastic rotation, θ_p, located at the maximum moment location. The final equation, Eq. 11, requires iteration as the final top elastic displacement must be computed using the cantilever equation with a modification for P-Δ effects.

$$\Delta_{final} = (\Delta_{eb} + \Delta_{et}) + \Delta_p + \Delta_t$$

$$\text{where } \Delta_{eb} = \theta_{eb} L_{ma};\ \Delta_{et} = \frac{V_{top} L_{ma}^{3}}{3EI_e};\ \Delta_p = \theta_p L_{ma} \text{ and } \Delta_t = \Delta_{tu} \text{ or } \Delta_{ty} \tag{11}$$

Model verification

Verification of the proposed method was performed using the data from full-scale testing of CIDH shafts in low plasticity clay (Suleiman et al. 2006). This test program included cyclic load testing of two identical column-foundation systems (i.e., SS1 and SS2). They were tested at 23°C and -10°C (~73 °F and 14 °F), respectively. Using the material properties established for the test units, the proposed model was found to well simulate the lateral load response of both systems. The warm weather comparison, SS1, is provided in Fig. 6a and the cold weather comparison, SS2, is provided in Fig. 6b.

In addition to the experimental verification, the model was verified against multiple nonlinear analyses of column-foundation systems using a computer program based on the Winkler soil spring method (Shelman 2009). The computer program produced a basis for comparing the global and local responses of the model. The local responses were verified by examining the translation at the maximum moment location, the elastic and plastic rotations at the maximum moment location, the location of the maximum moment, and the location of the zero moment. Multiple points of comparison, including displacements and curvatures, were made in addition to the analytical models produced by Sritharan, et al. (2007) and have demonstrated that the model has the ability to handle a wide range of soil shear strengths and column axial loads.

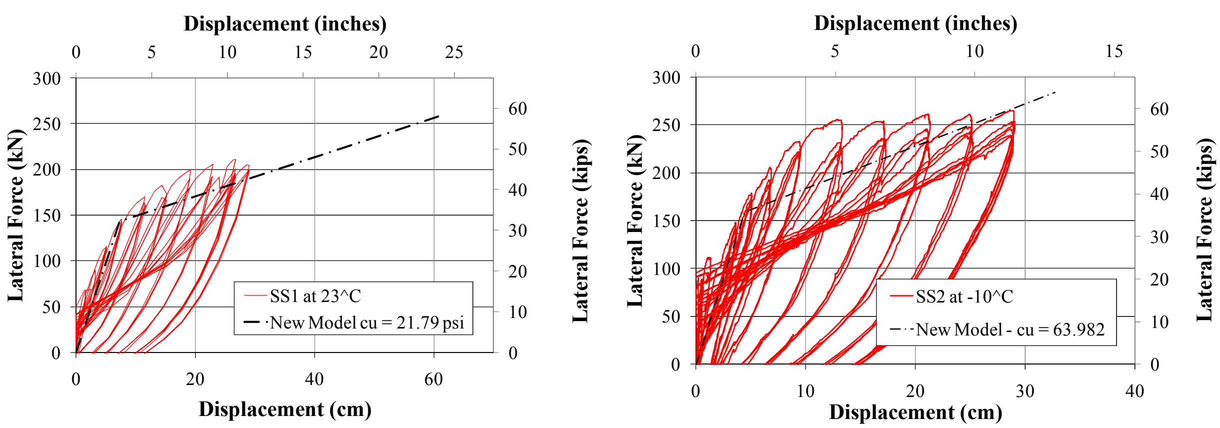

Figure 6. Verification of the proposed method with full-scale test results (a) warm weather conditions, (b) cold weather conditions

Conclusions

This paper first examined the current state of practice in regards to the seismic design of drilled shafts in cohesive soils and found that although several simplified models currently exist in order to determine the design displacements of drilled shafts, the models are not able to accurately capture the local and global response of the system. Next, an alternative model for predicting the seismic response envelope of a column supported on a drilled shaft in cohesive soils was provided. Based on the study summarized in this paper, the following conclusions are drawn:

1 Previous models specified to be used for cohesive soils [e.g., Chai (2002) and Priestley et al. (2007)] have only been verified against experimentation on drilled shafts in cohesionless soils, leading to inaccuracies in modeling drilled shafts in cohesive soil.
2 The models developed specifically for cohesive soils are typically for soft soils only and are not applicable for stiff clay.
3 Plastic action in the existing methods is significantly underestimated for CIDH shafts in clay. Although this is conservative for design, the system may be overly reinforced leading to higher costs.
4 The maximum moment location is not accurately identified within many of the models. This is a crucial point in the design to ensure that enough confinement is provided in the required region of the shaft to create a dependable response for the column-foundation system;
5 The proposed new model uses a series of three springs and has been shown to well simulate the global response of continuous column/foundation systems while enabling accurate estimates of the local design parameters at the critical locations.
6 The new model has also been shown to handle the effects of seasonal freezing on the response of column/shaft system due to the presence of a soil spring above the maximum moment location and the modification of construction material properties with the temperature. The effects of seasonal temperature variation are not addressed in any of the others methods discussed in this paper.

Acknowledgements

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