EVALUATION OF EXISTING *P-Y* MODELS FOR CALICHE BASED ON NUMERICAL ANALYSIS OF RAIDERS STADIUM LATERAL LOAD TESTS

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ABSTRACT

Numerical lateral load analysis based on the beam on Winkler foundation (BNWF) model (i.e., the p-y method) is a simple, and widely used tool to predict lateral responses of drilled shafts from field load tests. The lack of a site-specific p-y model for cemented soil layers such as caliche is one of the major limitations of simulating lateral load tests conducted in Las Vegas Valley. In this study, some of the existing p-y models for rock materials, such as Florida Limestone and Vuggy Limestone have been evaluated to simulate the lateral resistance of caliche. This has been done in the context of four lateral load tests in caliche-dominant sites from the recently constructed Raiders Stadium project in Las Vegas (NV). The uncertainty in characterizing the material properties of caliche has been addressed. The p-y analyses were performed using NVShaft, a MATLAB-based, comprehensive load analysis program. The applicability of the considered p-y models for caliche material was evaluated by quantifying the normalized error margins between measured and predicted lateral responses from the simulations of the mentioned load tests. Correlations between such errors and the depth and thickness of caliche in soil layers affecting numerical predictions were established, for both up to and beyond design load levels.

Keywords: drilled shaft, lateral load test, *p-y* analysis, cemented soil, caliche

INTRODUCTION

The deep foundation system is widely adopted in civil engineering projects to transmit the load from the superstructure into the subsurface. The data obtained from subsurface exploration is one of the key information in the selection of the proper type of deep foundation system. The hard, cemented calcium carbonate material known as Caliche is known for its erratic, intermitted existence across the Las Vegas valley. As explained in Werle and Luke (2007), the fluctuation of the water table in the presence of calcareous deposits causes the formation of discontinuous layers of caliche. Typically classified as an Intermediate Geomaterial (IGM) (Brown et al. 2010; Motamed et al. 2016), the cementation in caliche material varies with the unconfined compressive strength (q_u) values ranging between 30 psi to 20,000 psi (Cibor 1983; Saint-Pierre 2018). Caliche poses major engineering challenges in site exploration techniques, expensive drilling and excavation, and unknown characteristics due to a limited number of studies (Rinne et al. 1996; Werle and Luke 2007; Stanton et al. 2017). Performing standard penetration tests often result in the refusal SPT-N values, and obtaining intact samples of weakly cemented deposits often proves to be problematic (Stanton et al., 2017; Saint-Pierre, 2018). Despite these limitations, the cemented soil in Las Vegas offers high load carrying capacity and strong monolithic interaction with foundation material (Karakouzian et al. 2015; Afsharhasani et al. 2020; Bhuiyan et al. 2021), making the deep foundation system often a viable choice in this region.

Performing lateral load analysis of a deep foundation is a necessary computational step for the design for lateral loading. To simplify complex soil-structure interaction in lateral load analysis, Hetényi and Hetbenyi (1946) introduced a subgrade reaction method where the pile is assumed to behave as an elastic beam, and the soil is simplified by a series of uncoupled elastic springs. Later, McClelland and Focht (1956) incorporated nonlinearity in soil lateral stiffness in the lateral resistance (*p-y*) model, followed by the development of other nonlinear *p-y* models for different soil conditions (e.g., Matlock 1970; Reese et al. 1974). This simplified method to perform lateral load analysis is now widely known as Beam on nonlinear Winkler foundation (BNWF) model, or *p-y* method, and is recommended by design guidelines (e.g., API 2014).

The *p-y* models on specific soil types are generally back-calculated from lateral field load tests data (Matlock 1970; Reese et al. 1974; Reese and Nyman 1978), or finite-element (FE) model simulations (Suryasentana and Lehane 2014; Zhang and Andersen 2017; Byrne et al. 2020). The obtained *p-y* models are validated using case studies from several field load tests conducted on similar soil types (Suryasentana and Lehane 2014; Taghavi et al. 2020). The complex soil-pile interaction and characteristics of key lateral resistance properties (e.g., coefficient of subgrade modulus, k_v, and ultimate soil reaction, p_{ult}) are integrated into the derived *p-y* model. In practice, the API (2014) recommended *p-y* models are widely used in commercial computer programs and by practicing engineers, and as noted by many before, cannot properly capture the lateral resistance mechanisms for site-specific unique soil conditions (Briaud and Wang 2018; Fu et al. 2020). The selection of proper *p-y* models in the numerical lateral analysis is imperative to obtain reliably predicted responses and to reinforce the design process.

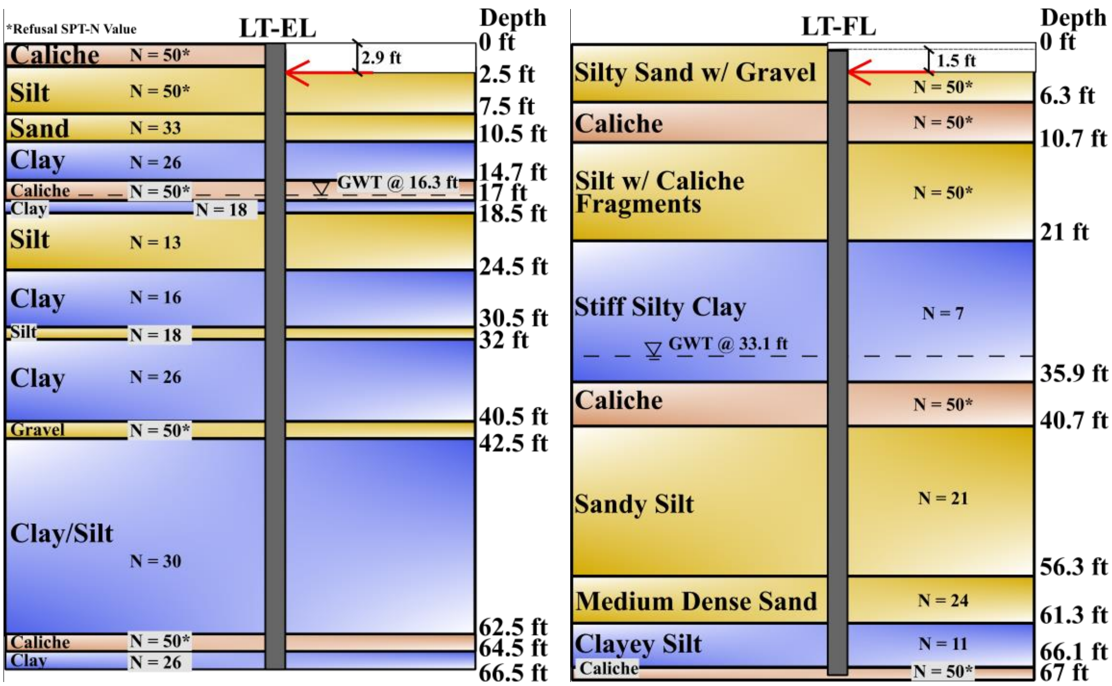
The focus of this paper is the evaluation of lateral resistance (i.e., *p-y* model) of caliche in the numerical *p-y* analysis. As of date, there is no site-specific *p-y* model developed for caliche material. The *p-y* model for vuggy limestone (Reese and Nyman 1978) has been used in the past to simulate lateral resistance of caliche for the I-15/US 95 load test program (Rinne et al. 1996; Bhuiyan et al. 2020), but the applicability of the model in this context remains to be investigated. In this study, the *p-y* models for vuggy limestone and Florida limestone (McVay and Niraula 2004) were evaluated for caliche material based on simulations of four lateral load tests from the Raiders Stadium project (Fiorelli et al. 2018a). Apart from the mentioned *p-y* models, the authors also considered the stiff clay *p-y* model (Reese and Welch 1975) for evaluation. The predicted responses from the preliminary analyses using the stiff clay model showed significant deviations from the measured responses and hence the model was excluded from this study. The considered load tests were conducted in cemented soil with the presence of caliche layers, making the load test program a viable choice for evaluating the aforementioned *p-y* models. The authors acknowledge that a limitation of this study is the limited amount of data on the caliche materials at the project location, which has been addressed in the numerical modeling. A MATLAB-based, finite-difference program, NVShaft (Bhuiyan et al. 2020, Bhuiyan et al. 2022) was used to perform the *p-y* analysis of the mentioned load test program. The applicability of the mentioned *p-y* models was further assessed by investigating the error margin in measured and predicted pile head deflections, concerning varying depths to the first caliche layers from the Raiders Stadium load tests. This assessment provided some insight on the reliability of these rock *p-y* models considering the relative distances of first caliche layers and the amplitude of applied lateral load.

RAIDERS STADIUM LOAD TEST PROGRAM

Inaugurated on July 31st, 2020, the new Allegiant stadium in Las Vegas is the home venue to the Las Vegas Raiders football team. Also known as the Raiders Stadium, the facility can accommodate 65,000 spectators, and features a sliding field tray and operable walls towards the views of the Las Vegas strip. Initially, the stadium was planned to be supported by drilled shaft foundations with a 48 inches diameter and 600 kips of axial load capacity, given the previous records of competent cementitious soil conditions in the region (Fiorelli et al. 2018b). Considering the foundation load to be distributed along a large project area, continuous flight auger (CFA) piles were chosen, to shorten the construction schedule and cut down costs (Fiorelli et al. 2018a). Approximately 1600 CFA piles with 24 inches diameters (*D*), and lengths (*L*) ranging from 30 to 65 feet were chosen as the final design (Fiorelli et al., 2018b). In this paper, four lateral load tests conducted on CFA piles with 24 inches diameter and around 66.5 ft length were considered. The considered test piles were designated in their corresponding load test reports as follows: 1) test pile LT-EL (LOADTEST 2018d), 2) test pile LT-FL (LOADTEST 2018c), 3) test pile LT-CL (LOADTEST 2018b) and, 4) test pile LT-AL (LOADTEST 2018a). The former two test locations (LT-EL and LT-FL) have caliche layers near the excavated pile head locations, making these load tests ideal to study the effect of caliche in *p-y* analysis. The generalized soil profiles for four lateral load tests from the Raiders Stadium project, along with the recorded SPT-N values, the depths of groundwater tables (GWT), and the locations of applied lateral loads relative to the top of the test piles are shown in Fig. 1. The details of site sub-surface conditions are discussed in the following section.

Site sub-surface conditions

For the test pile LT-EL, from the boring log LB-13, a caliche layer with a thickness of 2.5 ft is reported at the excavated ground level near the pile head location. This is followed by cemented fine to coarse sand, silt, and sandy clay layers up to another caliche layer at the depth of 14.7 ft. A third, 2 ft thick, caliche layer is encountered at a depth of 62.5 ft, overlain by a series of clay, silt, sand, and gravel layers with some cementation.



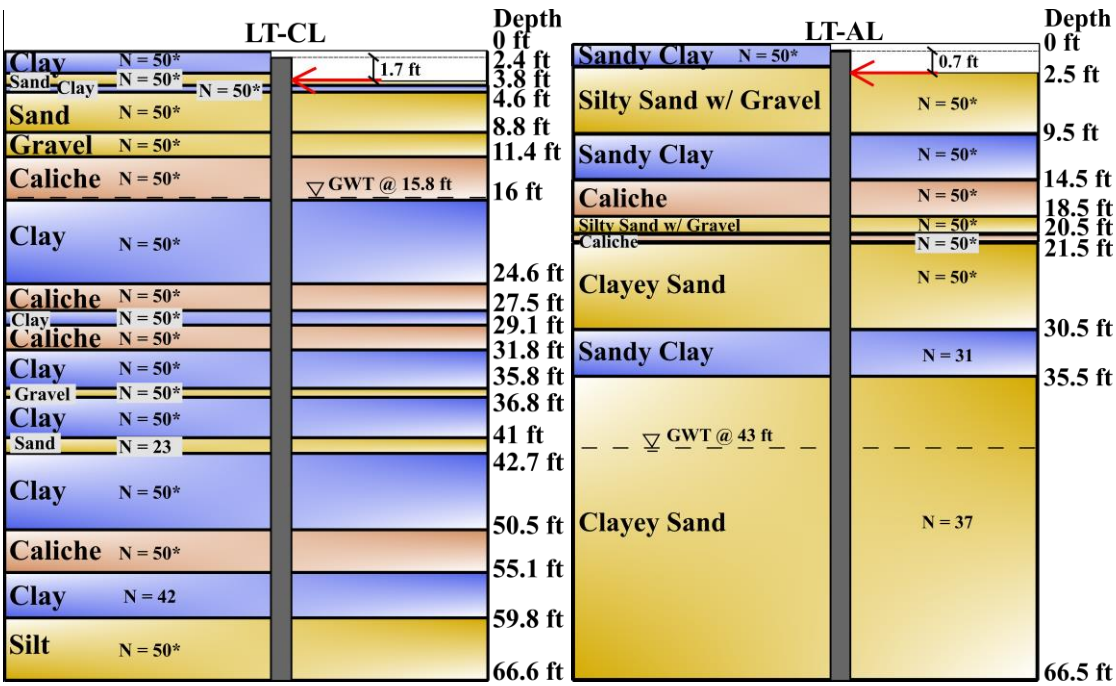


Fig. 1. Schematics of the generalized soil profile for test pile LT-EL (upper left), LT-FL (upper right), LT-CL (lower left) and LT-AL (lower right).

The subsurface condition of the test pile LT-FL indicates the first caliche layer to be located at a depth of 6.3 ft, as reported in the boring log LB-11 from the respective load test report (LOADTEST 2018c). This is overlain by very dense silty sand and coarse gravel layers. The second and third caliche layers are located at depths of 35.9 ft and 66.1 ft, respectively. Slightly cemented clayey silt and silty clay with coarse gravel-sized caliche fragments are reported at several depths.

As reported in the boring log LB-7 in the load test report, the soil layers at the location of test pile LT-CL consisted mostly of sandy clays, silts, and four layers of caliche (LOADTEST 2018b). The site has a series of slightly cemented silty clay, gravelly silty sand, and coarse gravel layers over the first 11.4 ft of depth from the ground level near the pile head location. This is followed by the first caliche layer with a 4.6 ft of thickness. Three more subsequent caliche layers at greater depths with thicknesses of 2.9 ft, 2.7 ft, and 4.6 ft are also present at this particular test site.

The soil profile for the test pile LT-AL can be characterized by the reported boring log B-105 in the load test report (LOADTEST 2018a). The excavated ground level near the pile head is reported to consist of 2.5 ft thick, very stiff, slightly cemented sandy lean clay material. This is underlain by a 7 ft thick, dense silty sand layer with gravel. The first caliche layer is reported to be located at a depth of 14.5 ft with 4 ft thickness, followed by silty sand with a gravel layer and another thin caliche layer.

Details of test piles and lateral load test configuration

All the CFA test piles had nominal diameters of 24 inches and were embedded up to around 66.5 ft of depth into the subsurface. Drillings were done up to a tip elevation of 2093 ft in all cases. The grouting process was immediately followed by the insertion of reinforcing cages into the wet grout. The reaction system consisted of a beam positioned across two similar CFA reaction piles. The lateral load was applied through a hydraulic jack calibrated to a vibrating wire load cell and a set of spherical bearing plates. Both pile head and reaction beam displacement were monitored using attached Linear Vibrating Wire

Displacement Transducers (LVWDT). For each test pile, two inclinometers were installed at the pile head, and oriented in the parallel and transverse loading direction, to monitor tilting.

The lateral load tests were conducted following the standard loading schedule by ASTM D3966 (ASTM 2007). Except for test pile LT-FL, lateral load tests were carried out in two loading cycles. In the first cycle, the load was applied in 10 increments up to 60 kips (200% of design load) of maximum lateral load and then unloaded in four decrements. In the second cycle, a maximum of 90 kips of the lateral load was applied in fourteen increments, followed by unloading in four decrements. For the test pile LT-FL, a third additional cycle of the lateral load was applied, up to 187 kips of maximum load in nineteen increments, and then unloaded in four decrements.

A summary of test pile geometries, location and thickness of caliche layers, and location of the centerline of lateral load below grade level for the Raiders Stadium load test program are shown in Table 1 below.

Table 1. Summary of key parameters for test pile, caliche layers, and location of applied load for the Raiders Stadium lateral load tests

Test Pile	Pile Diameter (ft)	Pile Length (ft)	Depth to Middle of First Caliche Layer (ft)	Thickness of First Caliche Layer (ft)	Centerline of Lateral Load Below Top of Pile (ft)
LT-EL	2	66.5	1.25	2.5	2.9
LT-FL		66.5	8.5	4.4	1.5
LT-CL		66.2	13.7	4.6	1.7
LT-AL		66.54	16.5	4	0.7

ROCK *P-Y* MODELS USED FOR EVALUATION

Lateral resistance (p-y) model for Vuggy limestone

A p-y model for vuggy limestone was developed based on a lateral field load test on a bored pile at Islamorada, Florida (Reese and Nyman 1978). The 4 ft diameter pile was embedded 43.7 ft into the limestone material, with a 14 ft thick overburden fill at the top. The characterization of the vuggy limestone was limited by the challenges in obtaining intact rock samples, and q_u values of only two rock specimens were reported (69.4 ksf and 54.4 ksf). Although the test indicated no failure of rock material, brittle failure is assumed when soil reaction reaches the ultimate value. The derived relationship between the lateral soil reaction (p) and lateral displacement (y) can be expressed as follows,

$$p = 1000 * \mathrm{q}_u * y,\ \ 0 < y \leq 0.0004 * D \quad [1]$$

$$p = 0.4 * \mathrm{q}_u * D + 50 * \mathrm{q}_u * (y - 0.0004 * D),\ \ 0.0004 * D < y \leq 0.0024 * D \quad [2]$$

$$p = 0.5 * \mathrm{q}_u * D,\ \ y > 0.0024 * D \quad [3]$$

Lateral resistance (p-y) model for Florida limestone

A total of 12 centrifuge lateral load tests conducted at the University of Florida, on drilled shafts with 6 ft and 8 ft of diameters in prototype, and *L/D* ratios of 2, 3, and 4 were used to formulate a *p-y* model for Florida limestone (McVay and Niraula 2004). Due to the difficulty in coring and fitting the rock material within the centrifuge container, Florida limestone was synthesized with similar properties by mixing ground-up limestone, type I cement, and water. Lateral tests were conducted on material with unconfined compressive strengths of 20 ksf and 40 ksf. To obtain a single trendline, the back-computed *p* values were normalized relative to $q_u^{0.25}D^{0.9}$, leading to a *p-y* model given by the following equations,

$$p = 895.41 * \mathrm{D}^{0.90} * \mathrm{q}_u{}^{0.25} * \frac{y}{D},\ 0 < \frac{y}{D} \leq 0.004 \quad [4]$$

$$p = \mathrm{D}^{0.90} * \mathrm{q}_u{}^{0.25}\left(68.57 * \frac{y}{D} + 3.35\right),\ 0.004 < \frac{y}{D} \leq 0.1 \quad [5]$$

$$p = 10.21 * \mathrm{D}^{0.90} * \mathrm{q}_u{}^{0.25},\ \frac{y}{D} > 0.1 \quad [6]$$

where units for *p*, *y*, *D*, and q_u are kips/inch, inch, inch, and ksf, respectively.

NUMERICAL MODELLING OF RAIDERS STADIUM LOAD TESTS IN NVSHAFT

Numerical *p-y* simulations of four lateral load tests on 2 ft diameter CFA piles from the Raiders Stadium project were done using the finite-difference program, NVShaft. Based on the available subsurface information from load test reports, soil parameters were calculated using empirical formulas and correlations from FHWA (Brown et al. 2010) and Caltrans (2019) manuals. As the maximum measured lateral displacements from the considered load tests is only 0.13 inches, the CFA piles were assumed to have elastic section properties in the numerical model. The lateral loads were applied at their respective locations as summarized in Table 1. For test pile LT-EL, the lateral load were applied slightly below the first caliche layer near the ground level, and for the remaining three test piles loads were applied above the respective first caliche layers (Fig. 1). The mobilization of lateral resistance from caliche can be expected to be small given this test condition, which has been assessed by observing the mobilization of the *p-y* models at the respective depths (Fig. 4).

One of the major limitations of this study is the very limited information on caliche in the load test reports. In the boring logs, refusal SPT-N values are reported at several caliche depth locations, and no laboratory test data was reported. Stanton et al. (2017) evaluated four different methods used by local engineers and NDOT practitioners to assume caliche material properties. Typically, an SPT-N value of 50 and unit weight of 140 pcf (22 kN/m^3) are assumed in local practice (Stanton et al. 2017). Following the classification of caliche as cohesive intermediate geomaterial (IGM) by Brown et al. (2010), the maximum value of q_u is assumed as 100 ksf (4.79 MPa). In this study, 12 ksf is used as the q_u of caliche at all test locations following a correlation suggested by Abu-Hejleh (2003) based on SPT-N for weak rocks.

The selected *p-y* models and relevant soil properties such as effective unit weight (γ'), angle of friction (φ), cohesive strength (c_u), unconfined compressive strength (q_u), soil modulus (E_s), and strain at 50% stress level (ε_{50}) for test pile LT-FL is shown in Table 2. The characterized soil profiles for the remaining three load tests are almost similar. Although this study focuses on evaluating existing *p-y* models for caliche material, a slight degree of cementation reported in the sand and silt layers also presents a unique opportunity to evaluate the *p-y* model for weakly cemented sand (Juirnarongrit and Ashford 2004). Finally, the lateral resistance of clay was simulated using the *p-y* model given by the Integrated Clay Method (O'Neill and Gazioglu 1984).

Table 2. Representative soil profile for test pile LT-FL from Raiders Stadium lateral load test

Depth (ft)	Soil Profile	p-y Model Used	γ' (pcf)	φ (Degree)	c_u (psf)	q_u (ksf)	E_s (ksf)	ε_{50}
0-6.3	Silty Sand w/ Gravel	Weakly Cemented Sand	130	43	-	-	-	-
6.3-10.7	Caliche	Vuggy/Florida Limestone	135	-	-	12	-	-
10.7-21	Silt w/ Caliche Fragments	Weakly Cemented Sand	130	43	-	-	-	-
21-35.9	Stiff Silty Clay	Integrated Clay	106	-	1407	-	750	0.007
35.9-40.7	Caliche	Vuggy/Florida Limestone	72.6	-	-	12	-	-
40.7-56.3	Sandy Silt	Weakly Cemented Sand	51.6	35	-	-	-	-
56.3-61.3	Medium Dense Sand	Weakly Cemented Sand	61.6	35	-	-	-	-
61.3-66.1	Clayey Silt	Integrated Clay	56.6	-	2036	-	232	0.007
66.1-67	Caliche	Vuggy/Florida Limestone	72.6	-	-	12	-	-

NVSHAFT PREDICTED RESPONSES AND EVALUATION OF ROCK P-Y MODELS

The comparison between measured data and predicted lateral responses from numerical *p-y* analysis of test piles LT-EL and LT-FL are shown in Fig. 2. For test pile LT-EL, the predicted response obtained after implementing the *p-y* model for vuggy limestone resulted in a reasonably good match with the measured data, particularly up to 200% of design load (60 kips). For this test, the *p-y* model for Florida limestone resulted in a stiffer predicted response compared to measured data at all load levels. The predicted responses for the test pile LT-FL (also shown in Fig. 2) indicate a better match with the measured data, particularly for the Florida limestone *p-y* model. The bar plots for these two tests, showing the error between predicted and measured pile head deflections (y_{head}), normalized by the pile diameter at 100% and 200% of design loads are shown in Fig. 3. The normalized error (e_{head}) was calculated using the following formula,

$$e_{head}(\%) = (y_{measured} - y_{predicted})/D * 100 \qquad [7]$$

where $y_{measured}$ = measured pile head deflection from field load test and $y_{predicted}$ = pile head deflection predicted from NVShaft simulations.

For test pile LT-EL, the *p-y* model for vuggy limestone resulted in 0.006% and 0.01% of normalized error for the design load and two times of design load, respectively. For test pile LT-FL, the overpredicted responses from the vuggy limestone *p-y* model resulted in -0.06% and -0.14% of normalized error at these load levels. As seen in Fig. 3, the Florida limestone *p-y* model yielded relatively smaller normalized errors at design load, which is 0.015% and -0.05% for test piles LT-EL and LT-FL, respectively. At 200% of design load level, using the Florida limestone *p-y* model resulted in 0.08% and -0.13% of normalized error for test piles LT-EL and LT-FL, respectively. These observations indicate that both rock *p-y* models can produce reasonable predicted responses for caliche, at least up to the design load level, in the context of the Raiders Stadium load test.

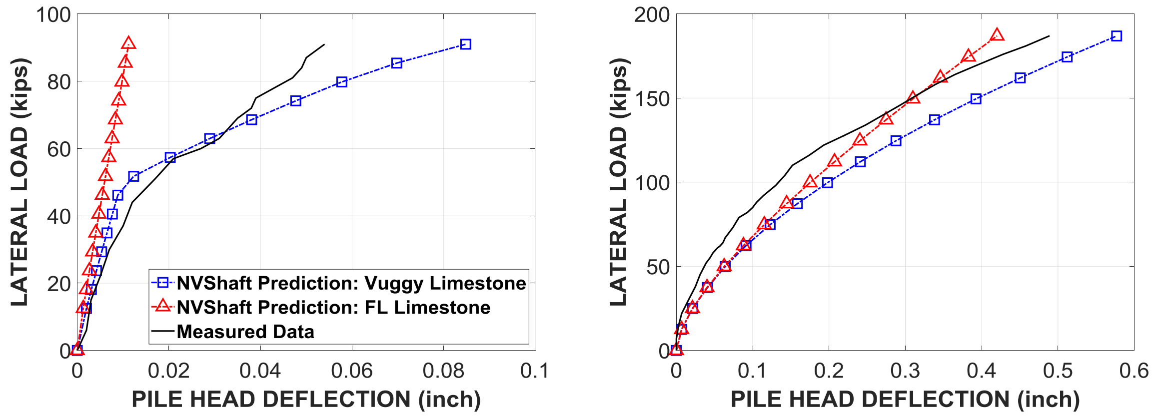


Fig. 2. NVShaft predicted responses of test piles LT-EL (left) and LT-FL (right) from lateral load test simulations with measured data.

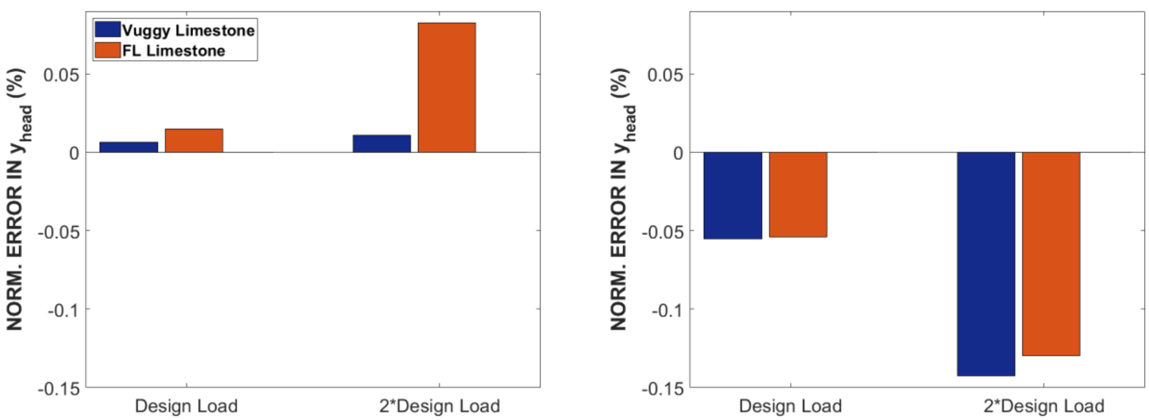


Fig. 3. Bar plots showing normalized error in pile head deflection (y_{head}) at design load and two times design load for test piles LT-EL (left) and LT-FL (right).

For test piles LT-EL and LT-FL, caliche layers were located at shallower depths compared to other lateral field load tests. In practice, to obtain the design length of piles to achieve lateral stability, the initial length of the pile is often taken as 10 to 15 times the pile diameter (ADOT 2010). As observed from past studies, the soil layers at this range of depths (20 ft to 30 ft in this study) for flexible piles exhibit wedge-type soil flow mechanism, affecting a major part of the overall lateral pile response (Hong et al. 2017; Wang et al. 2020). This fact is also reinforced in Fig. 2, which shows that using different *p-y* models for caliche for these two load tests caused major changes in predicted lateral responses from *p-y* analyses. The influence of *p-y* models to represent lateral resistance of caliche on numerical simulations can be further explained in Fig. 4. The NVShaft generated *p-y* plots presented in Fig. 4 show mobilization of lateral soil reactions (*p*) in caliche at lateral displacements (*y*) corresponding to 100% and 200% of design load for the considered lateral load tests. It can be seen that mobilization of lateral soil reaction in caliche is higher for test piles LT-EL and LT-FL, compared to test piles LT-CL and LT-AL, as caliche materials are located at greater depths for the former cases. For test pile LT-EL, a change in lateral stiffness can be seen in the mobilization of the vuggy limestone *p-y* model when the lateral load is increased beyond the design load level. This is also reflected in the predicted response corresponding to the vuggy limestone model, as shown in Fig. 2 (left). Figure 4 also reveals that none of the considered rock *p-y* models achieved full mobilization in the context of Raiders Stadium load test simulations.

The subsurface conditions for test piles LT-AL and LT-CL along the upper 19 ft depth are almost identical, as can be seen from Fig. 1. Performing lateral analyses using different *p-y* models for caliche for these two test piles resulted in almost no variation in predicted responses. The lateral responses for these test piles are mostly affected by the cemented sand, gravel, and clay material overlying the first caliche layer. Also, these two test piles have identical geometric properties and were subjected to the same lateral loads. Considering all these facts, it can be construed that these two lateral tests were almost identical, and the overall responses can be summarized by a composite plot, as shown in Fig. 5. It can be seen that the predicted responses for test pile LT-CL and LT-AL reasonably brackets the measured responses obtained at the field. Particularly for test pile LT-AL, a good match between predicted and measured responses can be observed. Due to the cyclic nature of the loading, the measured response for test pile LT-AL appears to be almost constant around two times of design load level (Fig. 5). The overall load-displacement responses from the numerical simulations of all the mentioned lateral load tests also indicate the reliability of the *p-y* model for weakly cemented sand given by Juirnarongrit and Ashford (2004).

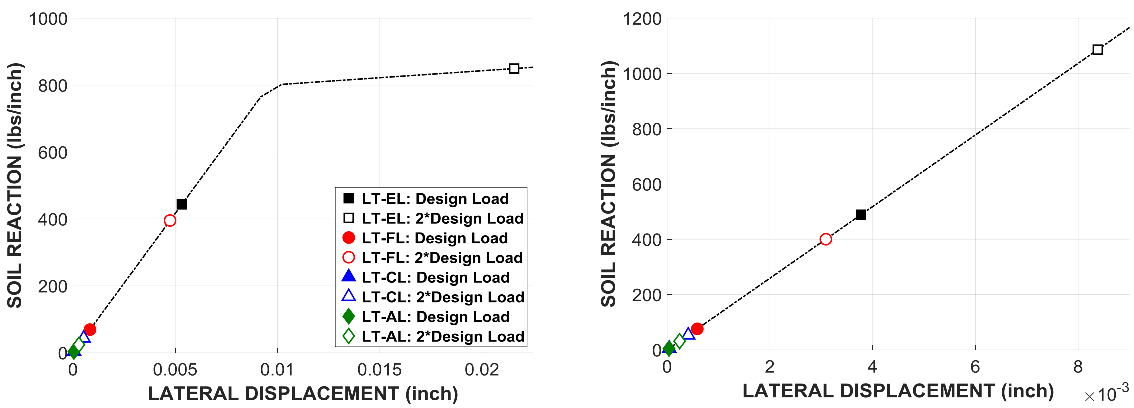


Fig. 4. Mobilization of lateral soil reaction at 100% and 200% of design loads, considering the *p-y* models for vuggy limestone (left) and Florida limestone (right) from the simulations of Raiders Stadium lateral load tests.

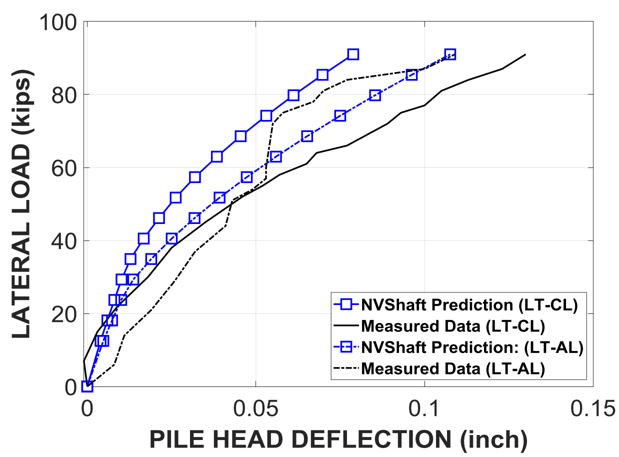


Fig. 5. Composite load-deflection plots for test piles LT-CL and LT-AL.

Relationships between the absolute normalized error in pile head deflection ($e_{abs,head}$) and the ratio of depth to the middle of the first caliche layer to pile diameter, for both design load and twice the design load levels are shown in Fig. 6. For design load level shown on Fig.6(left), it can be seen that $e_{abs,head}$ seems to show a relatively steadily increases with the increase in caliche depth with a maximum approximate value of 0.05%. A similar plot as shown in Fig. 6(right) for twice the design load also presents a smaller $e_{abs,head}$ when the caliche layer is located near the grade level, particularly for the vuggy limestone model. In this case, the $e_{abs,head}$ increases to a maximum approximate value of 0.14% corresponding to the caliche depth to diameter ratio of 4.25 and then decreases for greater depths. These observations indicate the reliability of both considered rock *p-y* models to simulate the lateral resistance of caliche, at least up to the applied design lateral load based on the findings from the Raiders Stadium load test study. In summary, the *p-y* model for vuggy limestone resulted in smaller $e_{abs,head}$, compared to the *p-y* model for Florida limestone in this regard.

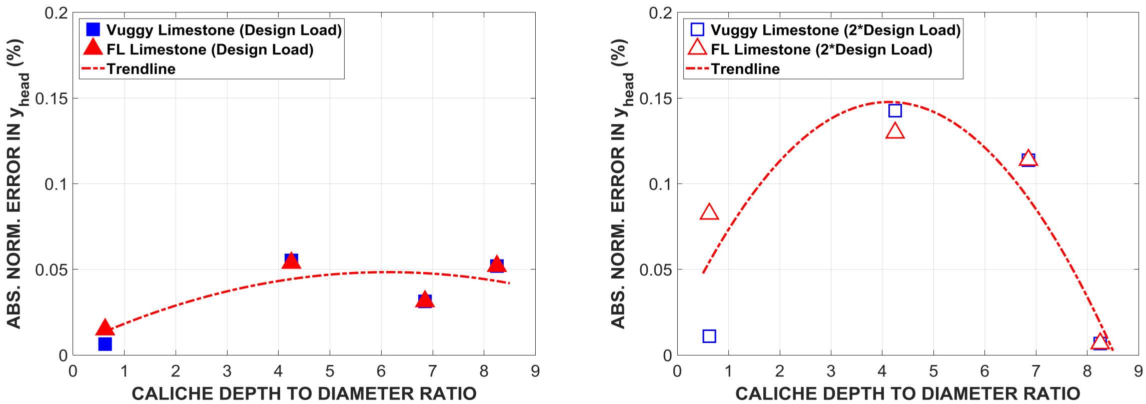


Fig. 6. Variation in absolute normalized error in pile head deflection with increasing caliche depth to diameter ratio for design load (left) and two times design load (right).

SUMMARY AND CONCLUSIONS

In this study, the applicability of two existing lateral resistance (*p-y*) models based on vuggy limestone and Florida limestone to simulate the lateral soil reaction of caliche were evaluated. This was done in the context of four lateral load tests performed in cemented soil conditions, as part of the Raiders Stadium construction project in Las Vegas. Field lateral load tests were carried out on CFA piles with 2 ft of diameter and around 66.5 ft of embedded depth. Unavailability of laboratory test data and limited information on caliche are the major limitations of this study. This limitation was addressed by characterizing the caliche based on some assumptions used in local practice and by using correlations to obtain the strength of the material.

A MATLAB-based comprehensive load analysis program, NVShaft was used to evaluate the mentioned rock *p-y* models. For test piles, LT-EL and LT-FL, the locations of the first caliche layer were relatively closer to the grade level, significantly affecting the lateral responses. For this reason, the simulated lateral responses from these test piles were extensively studied to evaluate the applicability of both rock *p-y* models. The predicted lateral responses of test pile LT-EL and LT-FL from using the considered rock *p-y* models showed reasonable agreements with the measured data, up to the design load level. The error margin between measured and predicted pile head deflection was calculated and then normalized relative to pile diameter. At design load level, the *p-y* model for vuggy limestone resulted in 0.006% and -0.06% of normalized error, for test pile LT-EL and LT-FL, respectively. At the same load level, the *p-y* model of Florida limestone resulted in 0.015% and -0.05% of normalized error for test pile LT-EL and LT-FL, respectively. These observations add to the reliability of the considered *p-y* models to perform *p-y* analysis in caliche-dominant sites.

Even though the lateral responses of test piles LT-CL and LT-AL were not significantly affected by the caliche, good matches between NVShaft predicted response and measured data were observed in these cases. This proves the capability of the *p-y* models used for other soil materials (e.g., weakly cemented sand) to produce reasonable predicted responses in cemented soil conditions. Finally, it has been shown that both *p-y* models are capable of producing reasonable predicted responses for caliche located at shallower depths, at both design load and twice the design load levels. Some discrepancies between measured and predicted responses presented in this paper can be attributed to the lack of site-specific *p-y* model and uncertainty in caliche material properties due to insufficient site characterization. Further research on quantifying lateral resistance of caliche is strongly encouraged in this context.

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