EVALUATION OF A NEW P-Y ANALYSIS TOOL FOR LATERAL ANALYSIS OF DRILLED SHAFTS USING LOAD TESTS IN NEVADA

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ABSTRACT

Large-diameter drilled shafts (DS) are a critical part of a seismic-resistant system, and therefore it is important to be able to accurately model lateral resistance. To undertake the lateral load analysis of large-diameter DS, a finite-difference p-y analysis program, NVShaft, is being developed at the University of Nevada Reno. Validation of the new program involved numerous p-y analyses using commercially available programs. Subsequent comparisons suggested the program's ability to yield reasonable predictions for typical diameter DS. Based on the I-15/US 95 load test program carried out in Las Vegas, NV, this study presents the evaluation of NVShaft in Nevada's subsurface condition. Lateral load analyses of a 2 ft and an 8 ft diameter DS were attempted using NVShaft. The location of the applied lateral loads and the presence of cemented soil-layers were incorporated in the modeling. A good agreement between NVShaft prediction and the measured response was found for the 2 ft diameter DS. For the 8 ft diameter DS, a stiffer response was predicted. The inspections of the moment-curvature characteristics at different load levels suggested cracked response of the concrete due to prior axial load testing may be one of the reasons behind the deviation.

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

INTRODUCTION

The Ability to transmit large loads in a variety of soil and rock profiles within a small footprint and high flexural strength to counter seismic loads are some of the advantages of drilled shafts (DS) compared to other types of deep foundations. The use of large diameter DS has increased substantially both in the US and Europe in recent years, as a suitable foundation system to support offshore wind turbines, bridge piers, tall buildings, and bridge abutments. Some of the thousands of offshore wind turbines installed in Europe are supported by DS with the diameter as high as 6m (Byrne et al., 2015). Design codes (e.g. API 2010) recommend using beam on nonlinear Winkler foundation (BNWF) models, commonly known as the p-y method to perform lateral load analysis. In the p-y method, localized soil's lateral resistance is represented using nonlinear p-y models. The DS is assumed to behave like an Euler's idealized beam and commonly the finite difference numerical computation is done by discretizing such beam in several nodes (McClelland 1956; Reese et al. 1974). The nonlinear p-y relations that characterize the soil resistance are derived based on correlations with instrumented field tests undertaken using slender piles of diameter in the range of 1 ft to 2ft. Despite being adopted in numerous computer software packages for its simplicity and reliability, the p-y method appears to yield very conservative responses for large diameter DS (Ashour and Helal 2014; McVay and Niraula 2004). For dense sandy soil in the San Diego area, Bhushan (2002) found that LPILE requires p-multipliers between 2 to 8 to accurately predict the load-deflection response based on measured data. The problem originates from the inherent inability of the p-y model itself to capture the characteristics of soil resistance associated with large diameter DS. Also, lack of p-y curves to represent untested soil types, layered soil conditions, the direct use of p-y curves obtained based on field lateral load test conducted on the slender piles (Matlock 1970; Welch 1972; Reese et al. 1974) add to the constraints. To address these limitations, more appropriate p-y curves have been proposed based on lateral load tests on large diameter DS with advanced instrumentation, or new design methodology has been recommended. Liam and Dowling (2015) conducted a parametric study using LPILE (Reese 2000) and VERSAT-P3D (Wu 2006) to obtain load-deflection curves for different pile diameters ranging from 0.2 m to 2 m. While LPILE uses the finite difference method, VERSAT-P3D is a 3D finite element (FE) based program. A Significant diameter effect was observed only when lateral deflection exceeded 60 mm, particularly for large diameter DS. Based on VERSAT analysis, the relationship between pile diameter and load level at a given head displacement was developed. Byrne et al. (2015) proposed a 'Numerical-Based Design Method' involving more detailed site investigation, testing, and in-depth 3D FE analysis to proceed with the design of large diameter DS.

In this paper, a MATLAB based finite difference software, NVShaft is introduced, which is currently being developed in the University of Nevada, Reno to address some of the limitations in the p-y method. NVShaft is proved to be capable of conducting conventional p-y analysis based on comparison with numerous analyses conducted in LPILE, as well as field load tests and centrifuge data. One such example of comparison with multiple different soil layers and the nonlinear DS response is presented in this paper. Nonlinear stiffness properties of the DS can be incorporated in NVShaft in the form of user-input moment-curvature relationships. Any location for application of the lateral load as boundary condition can also be specified. Users can model tapered DS, specify p-multipliers in soil properties, define five different types of boundary conditions at the top and shear resistance model at DS tip. For layered soil media, NVShaft can perform layering correction (Georgiadis 1983) and perform p-y analysis using any of 19 p-y curve models included in the program. Some of these features of NVShaft were utilized in order to assess its ability to evaluate lateral DS response in local soil conditions in Nevada.

The load test program performed as part of the Interstate I-15/US 95 Interchange project was chosen for this purpose (Kleinfelder 1996). Lateral tests on 2 ft diameter and 8 ft diameter DS in site No. 1 were intentionally chosen to observe the diameter effect when using the conventional p-y analysis. Axial load tests were conducted in both of these test shafts prior to the lateral load test, which made a significant impact on lateral response. Two boring logs from the site indicated the presence of caliche, partially cemented dense sandy clay and plastic silty sand layers at various locations of the soil profiles. Results from NVShaft analysis were compared with a COM624P prediction reported by Zafir and Vanderpool (1998) and field measurements. It was found that the difference between predicted and measured curvatures can be attributed to the deviation in the lateral load responses.

I-15/US 95 LOAD TEST PROGRAM

An extensive load test program as a part of geotechnical investigation associated with Interstate 1-15/US 95 reconstruction project was carried out in Las Vegas, Nevada in 1995 (Kleinfelder 1996). The goal was to get a better understanding of the response of DS under axial and lateral loading in Nevada's soil conditions. The outcome of the project was to ascertain axial and lateral design capacities and to help to develop the geotechnical design criteria of the DS. A total of 13 DS were constructed that included 2 ft, 8 ft diameter single DS, and four DS groups comprising of 2 ft diameter DS within 11 ft diameter cap. In total, five lateral and ten axial load tests were conducted in four different locations, about a one-half mile west of downtown Las Vegas. A primary motivation for the load tests was to use the response from the tests to develop side shear and end bearing capacity relative to the role of partially cemented soil and caliche layers. The Las Vegas area is known to experience some minor shaking from underground blasting and earthquakes. The load tests can also provide better insight into the lateral behavior of DS in such scenarios. In this study, 2 ft diameter and 8 ft diameter DS from site No. 1 were chosen for lateral load analysis using NVShaft.

Subsurface exploration and soil conditions

Five borings were made using truck-mounted drilled rigs and continuous flight, hollow stem auger. In site No. 1, two boring logs in close proximity, boring B-1, and B-5 were made. Hard to very hard cemented cores were obtained by using Nx size coring equipment. Standard penetration tests using a Central Mining Equipment (CME) auto hammer was carried out in all boring locations. Some of the major laboratory tests included grain size distribution, Atterberg Limits, direct shear strength, unconfined compression strength, and triaxial shear strength tests. The unconfined test was conducted on four caliche core samples, resulting in unconfined compressive strength value (q_u) ranging from 4060 psi to 9320 psi.

The upper 2.5 to 3.5 ft of the surface layer at site No. 1 consisted of low plastic to non-plastic silty sand, followed by a stiff, low to high plastic sandy clay up to 14 ft depth. Some remnants of alluvial and sheet wash soil from the original construction of interchange were also present near-surface. The first caliche layer was encountered from 14 to 13.5 ft depths and existed up to 18.5 to 21 ft depth in borings B-1 and B-5, respectively. Groundwater was at 13 ft and 10 ft depth after drilling as reported in logs for B-1 and B-5 respectively. The first caliche layer was followed by several layers of partially cemented, medium to very dense clayey and silty sand strata up to 32 ft depth. A 1 ft thick caliche layers were found at 32 ft depth in boring B-1, while in B-5 a clayey gravel layer was found at the same location. Another 2 ft thick caliche layer at 35 ft depth was preceded and followed by stiff sandy clay and dense to very dense clayey sand.

Information from the soil exploration program was used to develop soil profiles for numerical analysis in this study, which is summarized in Tables 1 and 2. The selected p-y curves for NVShaft analysis and required material properties, such as unit weight (γ), angle of friction (φ), cohesive strength (c_u), uniaxial compressive strength (q_u), soil modulus (E_s) and strain factor i.e. strain at 50% strain level (ε_{50}) values depending on soil types are shown. Several empirical formulas and correlations from FHWA (Brown et al. 2010) and Caltrans (2019) manual were used to obtain other needed soil parameters. For the first caliche layer in boring B-1, the p-y curve back-calculated from the triaxial test result was utilized as a user-input model. For other caliche layers, the p-y curve for vuggy limestone (Reese 1978) was adopted. A p-y curve incorporating a wide range of soil ductility and relative soil-shaft stiffness given by the Integrated Clay Method (O'Neill 1984) was used for the stiff clay layers.

Table 1. Representative soil profile for 2 ft diameter DS in I-15/US 95 load test program, Las Vegas

Depth (ft)	Soil Profile	p-y Curve Used	SPT-N	γ (pcf)	φ (Degree)	c_u (psf)	q_u (psi)	E_s (ksf)	ε_{50}
0-3.5	Low Plastic Silty Sand	Sand	26	91	37	-	-	-	-
3.5-6.5	Stiff Sandy Clay	Integrated Clay	31	113.1	-	4400*	-	1160	0.0043
6.5-10	Stiff Sandy Clay	Integrated Clay	50	120	-	4136	-	1036	0.0043
10-14	Stiff Sandy Clay	Integrated Clay	15	113.1	-	2369	-	914	0.0045
14-18.5	Caliche	User-input p-y	50	136	-	-	-	-	-
18.5-24	Dense Silty Sand	Sand	50	133	37	-	-	-	-
24-28	Dense Clayey Sand	Sand	49	133	32*	-	-	-	-
28-31.5	Dense Clayey Sand	Sand	39	133	37	-	-	-	-
31.5-33	Caliche	Vuggy Limestone	50	136	-	-	7819*	-	-
33-35	Dense Clayey Sand	Sand	40	130	37	-	-	-	-
35-38	Caliche	Vuggy Limestone	50	136	-	-	7778*	-	-

***Measured from in-situ/laboratory**

Table 2. Representative soil profile for 8 ft diameter DS in I-15/US 95 load test program, Las Vegas

Depth (ft)	Soil Profile	p-y Curve Used	SPT-N	γ (pcf)	φ (Degree)	c_u (psf)	q_u (psi)	E_s (ksf)	ε_{50}
0-2.5	Dense Silty Sand	Sand	26	91	37	-	-	-	-
2.5-5.5	Stiff Sandy Clay	Integrated Clay	13	108	-	2215	-	726	0.0047
5.5-7.5	Stiff Sandy Clay	Integrated Clay	42	109	-	5277	-	1216	0.0045
7.5-12	Stiff Sandy Clay	Integrated Clay	16	125	-	2640	-	580	0.0052
12-13.5	Medium Dense Clayey Sand	Sand	13	120	33	-	-	-	-
13.5-21	Caliche	Vuggy Limestone	-	136	-	-	9583*	-	-
21-26	Dense Clayey Sand	Sand	50	138	40	-	-	-	-
26-32	Dense Silty Sand	Sand	50	133	40	-	-	-	-

***Measured from in-situ/laboratory test**

Details of test shafts and lateral load test configuration

For 2 ft diameter DS, 21” diameter O-Cell with hydraulic ram was placed 32.2 ft below grade. Longitudinal bars consisted of eight bundled groups of three #9 bars that were evenly spaced around 18” spiral-tied cage. The pitch of the tie bar was 3” up to the depth of 14 ft and 9” from that depth, continuing up to the location of O-Cell.

The 8 ft diameter DS was designed to carry 2000 kips of maximum lateral load. The reinforcement cage consisted of 24 2-bar bundles of #11 longitudinal bars tied within 84” diameter spiral cage. Tie bars consisted of continuous # 5 bar at a three-inch pitch. The schematic of 2 ft and 8 ft diameter DS along with the generalized soil profiles are shown in Fig. 1.

For both test shafts, the lateral load was applied using hydraulic jack (with the capacity of 600 kips and 2000 kips for 2 ft and 8 ft diameter DS respectively) about 20” below ground level. The 2 ft diameter DS was subjected to 228 kips of maximum test load, in 12 kips increments, and maximum horizontal deflection was 3.198”. For 8 ft diameter DS, in 100 kips increments, the maximum applied load was 1578 kips. The maximum recorded groundline lateral deflection was 1.37”.

Field observations during axial load tests and related uncertainties

During axial load tests, which were conducted prior to lateral load tests, 366 kips and 3914 kips of maximum axial loads were applied via O-Cells placed in the middle depth of 2 ft and 8 ft diameter DS respectively. For the 8 ft diameter DS, vertical displacements of top and bottom portions were 1.351” and 0.807”, respectively at the maximum axial load level. Loads were applied in three cycles on the 8 ft diameter

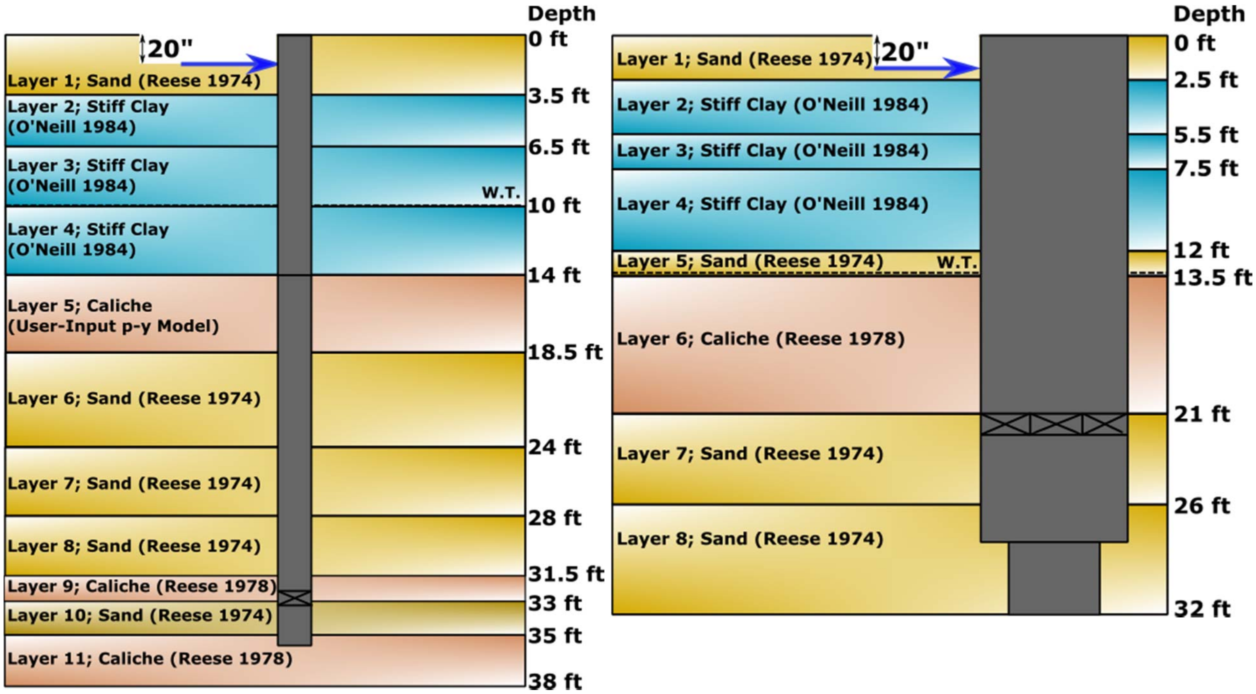

Fig. 1. Schematics of DS details and generalized soil profile for 2 ft diameter (left) and 8 ft diameter (right) test shafts in I-15/US 95 load test program, Las Vegas

DS, which caused the formation of cracks and upward heave at the nearby ground surface. At the level of 2228 kips of axial load, an average upward heave from DS edge up to the radial distance of 5 ft was reported to be around 0.53”. Also, the tensile strain reported in the strain profile diagram near O-Cells location indicated a possible crushing of concrete. For the 2 ft diameter DS, the deformation in the axial load test was comparatively lower (0.041” of upward and 0.807” of downward movement of the DS at maximum axial load).

From the field observations during the axial load test response, it is clear that the lateral load test conducted on the 8 ft diameter DS could have been significantly affected by the prior severe soil-shaft response. The presence of cementation in soil layers is another major factor to add to the complication. As stated by Brown et al. (2010), cementation in the sand may deteriorate during drilling and sampling, which may cause the sample to appear to be uncemented. Lack of reported soil properties led to the use of multiple empirical formulas, which also adds to the uncertainties in the prediction of DS response in the lateral load analysis.

DEVELOPMENT OF NVSHAFT

A MATLAB based, finite difference program NVShaft is being developed as part of a Nevada of Department of Transportation (NDOT) funded research project aiming to improve the lateral load analysis in the context of Nevada soil conditions. A total of 19 different p-y curves, based on different soil/rock conditions, have been included in the program’s library at this time in the project. Some of the recently developed p-y curves (e.g. O’Neill 1984) claim to provide a better representation of lateral resistance. A list of the p-y models available in the current version of the program is shown in Table 3.

Table 3. Summary of p-y curve models included in NVShaft

Soil type	Clay	Sand	Rock	Special/Other Types
Number of p-y Models	6	4	4	5

While performing the p-y analysis, NVShaft solves for DS displacement, slope, moment and shear force using the fourth-order differential equation in eq. [1],

$$EI\frac{d^4y}{dx^4} + P_x\frac{d^2y}{dx^2} - p - w = 0 \qquad [1]$$

Where E = elastic modulus of DS section, I = moment of inertia of DS section, y = deflection at depth x, P_x = vertical load on head, p = soil reaction per unit length, and w = applied distributed load per unit length.

NVShaft can perform p-y analysis on the DS with multiple sections including nonlinear properties using user-defined moment-curvature relationships. The program can implement the layering correction method proposed by Georgiadis (1983) for layered soil profiles. In this method, the effect of stiffness characteristics of upper soil layers to the lower ones is defined in the form of equivalent depths. Tapered DS can be modeled by assigning different values for the diameter at the head and the tip. Similar to the current state of practice, p-multipliers can also be specified for specific p-y curves. Users can specify five different types of boundary conditions at the top, and also shear resistance model at the tip of the DS. The location of boundary conditions, such as shear force can also be set as an input. NVShaft also has additional features like calculating the critical length of DS based on analysis with multiple loading conditions.

Each of these features and p-y models was verified by comparing the responses from identical models made in popular commercial p-y program and NVShaft. One such example of validation by comparing the outputs with LPILE in terms of deflection and bending moment profiles is shown in Fig. 2. The model includes a nonlinear DS with 3 ft diameter and 50 ft in length, embedded in 8 different types of soils. The soil parameters and nonlinear DS properties were retrieved from examples 23 and 15 respectively from LPILE's user manual (Isenhower et al. 2017). The analyses were carried out after applying the layering correction, which ended up producing almost identical responses. Relative to LPILE response, the difference in maximum deflections and bending moments were 2.66% and 1.24% respectively. This small deviation in outputs mostly stems from different numerical integration schemes adopted by these two programs while performing layering correction for soil layers.

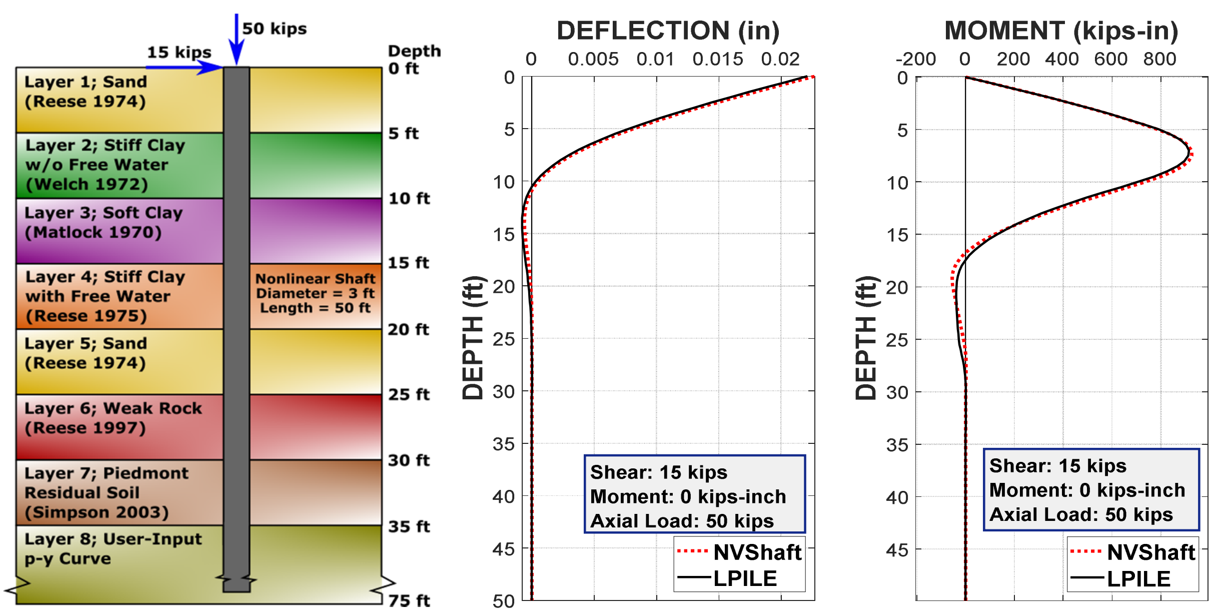

Fig. 2. Numerical model and outputs from identical p-y analyses performed for comparison between NVShaft and LPILE

PREDICTION OF I-15/US 95 LOAD TEST BY NVSHAFT

Numerical p-y analyses were carried out using the NVShaft program, to evaluate the program's capabilities to predict the response of the large-diameter DS based on the I-15/US 95 load test program. To investigate the diameter effect, analyses were conducted for 2 ft and 8 ft diameter DS from site No. 1. Zero bending moment and multiple shear forces applied at 20" from DS head were specified as boundary conditions. While calibrating the model, it was observed that the location of the applied lateral load caused a significant impact on the results. Moment-curvature relationships for the DS were generated in CSiBridge (2016) program, which is shown in Fig. 3.

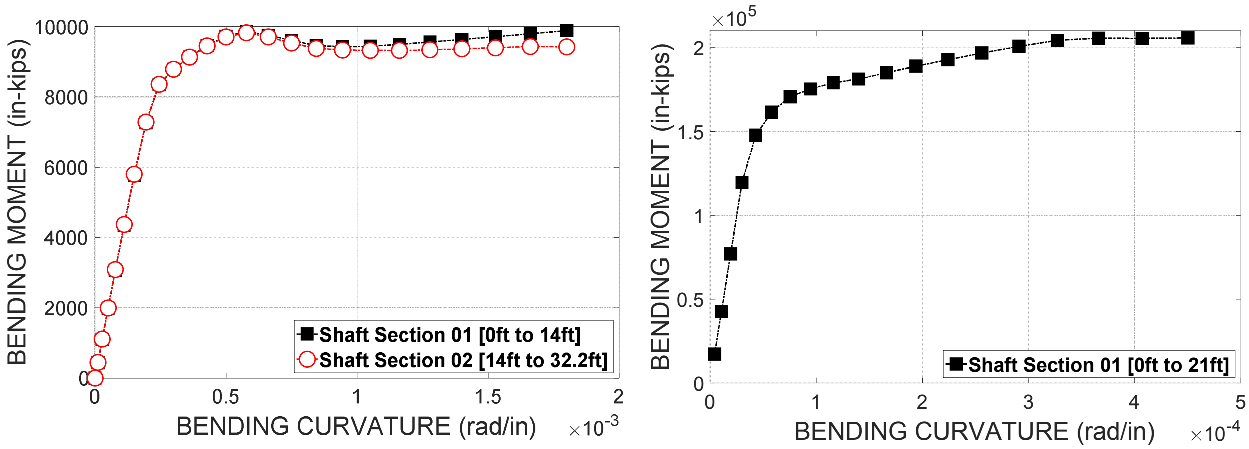

Fig. 3. CSiBridge generated moment-curvature curves for 2 ft diameter (left) and 8 ft diameter (right) DS

The deflection profile and load-deflection plot for 2 ft diameter DS obtained from NVShaft analysis are presented in Fig. 4. The Corresponding measured DS and COM624P (currently LPILE) predicted response by Zafir and Vanderpool (1998) are also included in the plots for comparison. A similar type of comparison plots is shown in Fig. 5 for 8 ft diameter DS. A very good match between NVShaft predicted response and measured data at smaller load levels can be observed for both of these DS. Compared to NVShaft, the measured response is slightly stiffer at these load levels for 8 ft diameter DS, indicating possible diameter effects in the p-y analysis. The head deflections predicted by NVShaft at corresponding maximum lateral loads are 3.39" and 1", for 2 ft and 8 ft diameter DS respectively. Predictions yielded by COM624P are softer compared to other types of responses, as seen in load-deflection plots.

At higher load levels, particularly for the case of 8 ft diameter DS, the response obtained from NVShaft is significantly stiffer compared to measured data. This deviation can be explained by comparing the maximum curvatures at different load levels based on the numerical and measured response as shown in Fig. 6. Based on CSiBridge simulated nonlinear flexural model used in the analysis, predicted maximum curvature closely follows the measured one for 2 ft diameter DS, at small to medium load levels. This indicates the ability of NVShaft to make a reasonable prediction in terms of curvature response for 2 ft diameter DS.

A similar type of investigation done for 8 ft diameter DS shows significantly higher maximum curvature predicted by NVShaft. One of the possible reasons is the cracked response of concrete originating from the prior crushing during the axial load test. The rigid movement of the cracked upper portion of the DS that follows at higher applied lateral loads explains the softer measured DS response. Observed excessive upward heave and crack formation in the soil during axial load tests prior to the lateral load tests make the continuum-based p-y analysis somewhat challenging. The Presence of cemented soil layers, higher stiffness at O-Cell locations, and the use of empirical formulas to obtain soil properties also add to the complication.

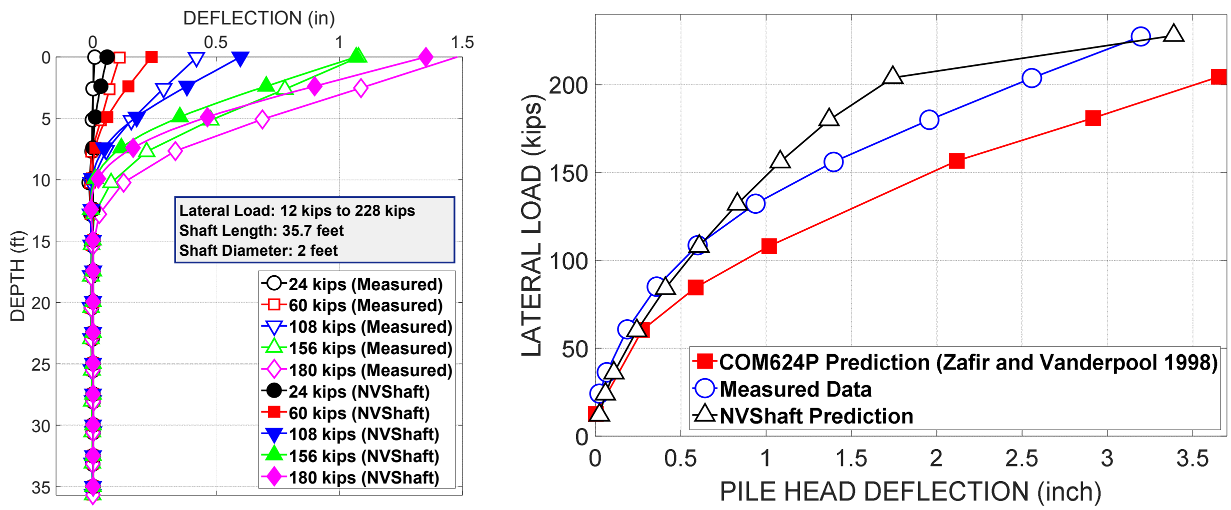

Fig. 4. The response of 2 ft diameter DS based on numerical calculation (NVShaft and COM624P) and measured data

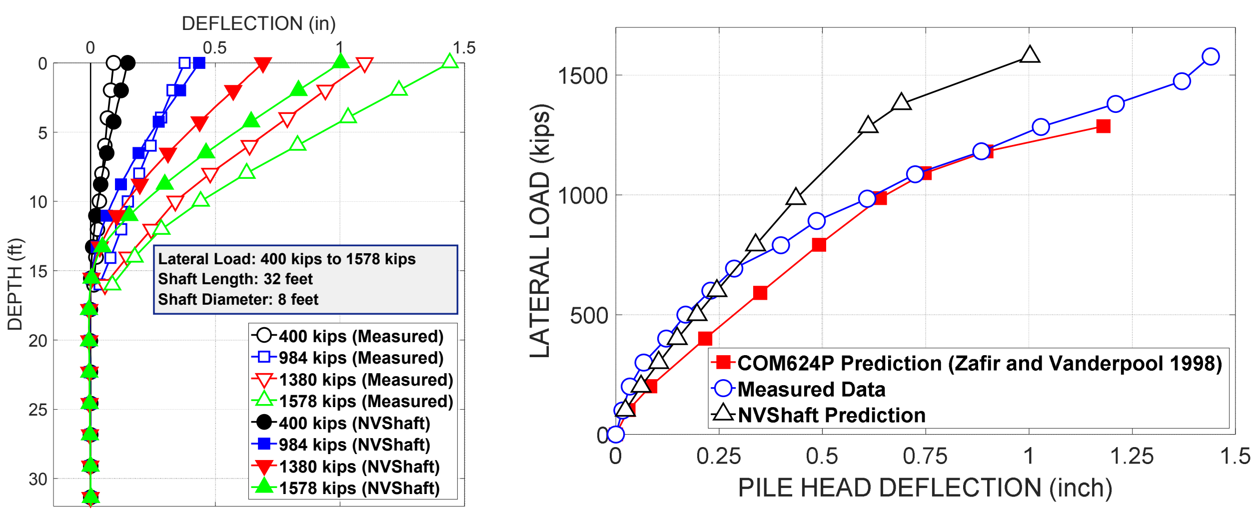

Fig. 5. The response of 8 ft diameter DS based on numerical calculation (NVShaft and COM624P) and measured data

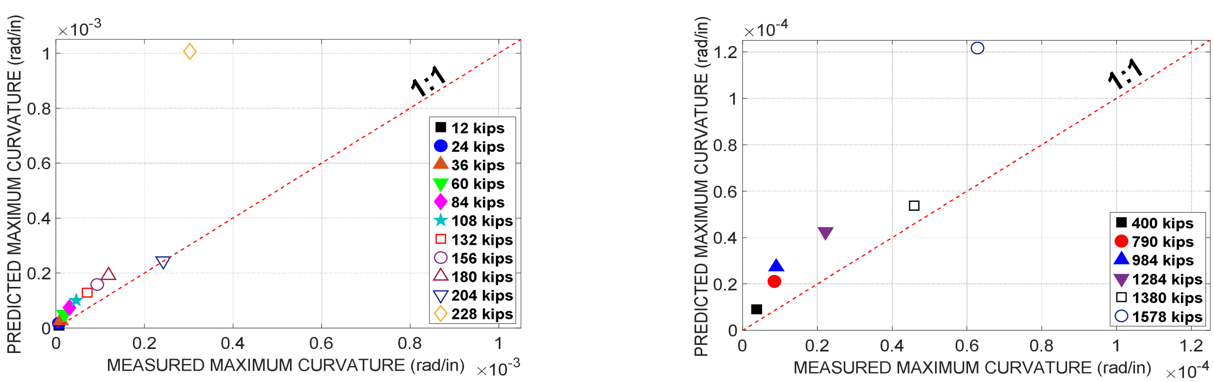

Fig. 6. Predicted vs measured maximum curvature for 2 ft diameter (left) and 8 ft diameter (right) DS

SUMMARY AND CONCLUSIONS

To improve the overall p-y method of lateral load analysis on large diameter shafts, a MATLAB based numerical, finite difference program, NVShaft, is currently being developed at the University of Nevada, Reno. As of now, a total of 19 p-y models have been included in the program's library. Users can specify p-multipliers, different types of boundary conditions, the location of applied lateral load, and relevant tip shear resistance in the model. Each of these features has been validated based on comparison from many analyses done in similar programs used in current practice. One such example is presented to compare the response obtained by NVShaft and LPILE from p-y analysis on a nonlinear DS embedded in layered soil medium. Differences of 2.66% and 1.24% in maximum deflection and bending moment, respectively, from separate, identical analyses indicate the new program's capability to perform conventional p-y analysis.

To verify NVShaft in a more practical perspective, I-15/US 95 load test program in Las Vegas, Nevada was used as a reference. Lateral load tests on a 2 ft diameter (35.7 ft in length) and an 8 ft diameter (32 ft in length) DS from site No. 1 were modeled in NVShaft to study the diameter effect. Two adjacent boreholes near test shafts revealed the presence of partially cemented soil, mostly in the form of stiff sandy clay and dense silty sand layers. Starting from 13.5 ft of depth, fully cemented caliche layers at several depths were encountered. The p-y model for vuggy limestone and the user-input model back-calculated from triaxial test results were used to attenuate the unpredictability of the caliche layers. In both test shafts, lateral loads were applied 20" below grade level from the hydraulic jack. The maximum measured horizontal deflection of the 2 ft diameter DS was 3.198" at a maximum lateral load of 228 kips. For the 8 ft diameter DS, the maximum recorded horizontal deflection was 1.37" at a maximum lateral load of 1578 kips.

A very good match with measured response was obtained in both cases from NVShaft analyses at small load levels. The maximum head deflections captured in numerical analysis at corresponding final lateral loads were 3.39" and 1", for the 2 ft and 8 ft diameter DS respectively. For the 8 ft diameter DS, the measured response was relatively stiffer than the predicted response at lower load levels, indicating possible diameter effects in play. At higher lateral loads, measured responses were softer in comparison, particularly for the 8 ft diameter DS. The record of the axial load test performed beforehand indicates severe crushing of concrete, the upward heave of soil, and crack formation along with the radial distance from the 8 ft diameter DS location. This extreme response of the soil-shaft system adds to the complexity in the nonlinear flexural behavior and the overall lateral load test. An inspection of the maximum measured and predicted bending curvature also points out the cracked response of concrete during the lateral test on the 8 ft diameter DS. On the other hand, for the 2 ft diameter DS, the maximum predicted curvatures at small to medium load levels complied well with the measured response. This particular observation reinforces NVShaft's capability to reasonably capture the nonlinear DS response during p-y analysis. Beyond the scope of this study, NVShaft also showed promise based on comparison with several other field load and centrifuge load test responses.

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REFERENCES

API., 2010. RP 2A-WSD - recommended practice for planning, designing and constructing fixed offshore platforms. Washington: American Petroleum Institute.

Ashour, M., and Helal, A., 2014. Contribution of vertical skin friction to the lateral resistance of large diameter shafts. Journal of Bridge Engineering, 19(2), pp. 289–302.

Bhushan, K., and Scheyhing, C., 2002. Lateral load tests on drilled piers in San Diego area residual and formational soils. Proceedings, 27th Annual Conference on Deep Foundations, San Diego, CA.

Brown, D.A., Turner J.P., and Castelli, R.J., 2010. Drilled shafts: construction procedures and LRFD design methods. US Department of Transportation, Federal Highway Administration.

Bridge, C.S.I., 2016. Computers and Structures. CSI Berkeley, California.

Byrne, B.W., McAdam, R., ... and Schroeder, F.C., 2015. New design methods for large diameter piles under lateral loading for offshore wind applications. 3rd International Symposium on Frontiers in Offshore Geotechnics (ISFOG 2015), Oslo, Norway, June, pp. 10-12.

California Department of Transportation., 2019. Caltrans geotechnical manual. https://dot.ca.gov/-/media/dot-media/programs/engineering/documents/geotechnical-services/soil-correlations-mar2013-a11y.pdf. Accessed 3-10-20.

Finn, W. L., and Dowling, J., 2015. Modelling effects of pile diameter. Canadian Geotechnical Journal, 53(1), 173-178.

Georgiadis, M., 1983. Development of p-y curves for layered soils. American Society of Civil Engineers, Proceedings, Geotechnical Practice in Offshore Engineering, ASCE, pp. 536-545.

Isenhower, W.M., Wang, S.T. and Vasquez, L.G., 2017. User's manual for LPile 2018. Ensoft, Austin, TX, United States.

Kleinfelder, Inc., 1996. I-15/US 95 Load test program, Las Vegas, Nevada. Final Report, January 26.

Matlock, H., 1970. Correlations for design of laterally loaded piles in soft clay. Proceedings, 2nd Offshore Technology Conference, Vol. I, pp. 577-594.

McClelland, B., 1956. Soil modulus for laterally loaded piles. Journal of the Soil Mechanics and Foundations division, 82(4), pp. 1-22.

McVay, M.C., and Niraula, L., 2004. Development of PY curves for large diameter piles/drilled shafts in limestone for FBPIER. No. Final Report.

O'Neil, M., and Murchinson, J.M., 1984. An evaluation of p-y relationship in cohesive soils. American Society of Civil Engineers, Symposium on Analysis and. Design of Pile Foundations, San Francisco, pp. 192-213.

Reese, L.C., Cox, W.R. and Koop, F. D., 1974. Analysis of laterally loaded piles in sand. Proceedings, 6th Offshore Technology Conference, Vol. II, pp. 473-484.

Reese, L.C., and Welch, R. C., 1975. Lateral loading of deep foundations in stiff clay. American Society of Civil Engineers, ASCE J. of the Geotechnical Engineering Division, Vol. 101, No. GT7, pp. 633-649.

Reese, L.C., and Nyman, K.J., 1978. Field Load tests of instrumented drilled shafts at Islamorada, Florida. Report to the Girdler Foundation and Exploration Corporation, Clearwater, FL, February.

Reese, L.C., 1997. Analysis of piles in weak rock. American Society of Civil Engineers, Journal of the Geotechnical and Geoenvironmental Engineering Division, pp. 1010-1017.

Reese, L.C., Wang, S.T., Isenhower, W.M., Arrellaga, J.A., and Hendrix, J., 2000. A program for the analysis of piles and drilled shafts under lateral loads. Ensoft, Inc., Austin, Texas.

Simpson, M., and Brown, D.A., 2003. Development of p-y curves for piedmont residual soils. Highway Research Centre, B-424(2), 112.

Welch, R.C., and Reese, L.C., 1972. Laterally loaded behavior of drilled shafts. Research Report No. 3-5-65-89, Center for Highway Research, The University of Texas at Austin, May 1972.

Wu, G., 2006. VERSAT-P3D: A computer program for dynamic 3-dimensional finite element analysis of single piles and pile groups. Wutec Geotechnical International, Vancouver, Canada. Available from http://www.wutecgeo.com/versat-p3d.aspx. Accessed 3-31-20.

Zafir, Z., and Vanderpool, W.E., 1998. Lateral response of large diameter drilled shafts: 1-15/ US 95 load test program. Proceedings of the Thirty-Third Engineering Geology and Geotechnical Engineering Symposium, University of Nevada, Reno, pp. 161–176.