

# EVALUATION OF EXISTING $t$ - $z$ MODELS FOR CALICHE BASED ON NUMERICAL ANALYSIS OF BI-DIRECTIONAL LOAD TESTS USING NVSHAFT

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## ABSTRACT

The difficulty in material characterization and the erratic response of cemented soil layers, such as caliche in the Las Vegas valley, creates challenges for practicing engineers to reliably predict the response of deep foundations. The focus of this paper is the numerical prediction of axial response in drilled shaft foundations, which are commonly used in infrastructure projects (i.e., bridges and tall buildings) in Las Vegas, NV. The prevalence of hard caliche layers with variation in the degree of cementation add to the complication in numerical modeling. In this study, the applicability of two existing  $t$ - $z$  models, developed for Florida limestone and soft rock, was evaluated based on numerical simulations of three bi-directional load tests conducted at caliche-dominant sites. The corresponding top-down load tests were also simulated for further assessment. A MATLAB-based finite-difference program, NVShaft, has been used to implement the mentioned  $t$ - $z$  models in axial load analysis. Complexity originating from drilled shaft construction and the interaction with caliche during one axial load test resulted in a stiffer predicted response compared to measure data. For the other two load tests, both  $t$ - $z$  models produced comparable load-displacement responses. It was observed that the  $t$ - $z$  model for Florida limestone predicted relatively stiffer responses and higher drilled shaft capacity.

**Keywords:** drilled shaft, axial load test, bi-directional static load test,  $t$ - $z$  analysis, cemented soil, caliche

## INTRODUCTION

Numerical simulation of axial load-deformation response of deep foundations, known as  $t$ - $z$  method is often used as a simplified design tool. In this method, the interactive soil-shaft response under axial loading is characterized through linear or nonlinear  $t$ - $z$  (i.e., side resistance) and  $q$ - $z$  (i.e., end bearing) models. The  $t$ - $z$  analysis can be performed using commercially available programs such as TZ-PILE (Ensoft 2014). Although the method offers a simple solution to perform trial design, Brown et al. (2010) advised to rely on multiple field load tests to verify the numerical predictions, as the derived load transfer (i.e.,  $t$ - $z$  and  $q$ - $z$ ) curves often fail to capture sensitive parameters relating to construction technique and subsurface materials. To address this issue, Stanton et al. (2015) implemented a semi-empirical procedure to calculate load transfer models in the computer program CGI-DFSAP and reported improved accuracy in numerical  $t$ - $z$  prediction.

Caliche is a rock-like, calcium carbonate cemented material (Werle et al. 2007) typically classified as an Intermediate Geomaterial (IGM) for engineering purposes (Brown et al. 2010; Motamed et al. 2016). Caliche is known to exist intermittently as discontinuous lenses across the Las Vegas valley, which poses significant challenges in site characterization. The unconfined compressive strength ( $q_u$ ) of caliche may vary between 3,000 psi to 10,000 psi (Cibor 1983; Saint-Pierre 2018). There have been reports of sudden changes in cementation of caliche within very small depth (Kleinfelder 1996). Performing axial load tests (conventional top-down and bi-directional) on drilled shafts embedded in cemented soils introduces additional complexity for design consideration. Karakouzian et al. (2015) observed monolithic behavior at the interface between the concrete of a drilled shaft and the encompassing caliche layer. This monolithic behavior failed to result in minimal displacement (about 0.2-0.3 inches) required to mobilize ultimate skin friction during axial load testing (Karakouzian et al. 2015). In a recent study, Afsharhasani et al. (2020) investigated the effect of the proximity of bi-directional cell relative to the caliche layer. The study concluded that placing bi-directional cells near

the most competent caliche layer ensured adequate mobilization of material which produced reliable shaft resistance.

The focus of this paper is to evaluate two existing  $t$ - $z$  models developed for Florida Limestone (McVay and Niraula 2004) and weak rock (Asem and Gardoni 2019), to simulate side resistance of caliche in numerical  $t$ - $z$  analysis. A MATLAB-based finite-difference program, NVShaft is implemented to simulate axial load tests in caliche dominant soils throughout Las Vegas. Unlike most available finite-difference  $t$ - $z$  analysis programs, NVShaft allows the users to specify the location of applied axial load and enables one to simulate both conventional top-loaded and bi-directional static load tests. NVShaft's capability to produce reliable axial load response is highlighted by producing identical outputs, by analyzing an example problem from the TZ-PILE user manual. Two bi-directional load tests from the I-15/US 95 reconstruction project (Kleinfelder 1996) and a bi-directional test from the Las Vegas City Center project (LOADTEST 2005) were used in developing the finite-difference model. The predicted responses from NVShaft analyses were compared with their respective measured shaft responses to assess the applicability of the mentioned  $t$ - $z$  models, for both static top-down and bi-directional load test simulations. The top-down axial responses obtained from NVShaft predictions were compared against calculated equivalent top load-settlement curves. The relative locations of the bi-directional cells along the shaft length were taken into consideration following recommendations by Afsharhasani et al. (2020) to interpret the results.

## NUMERICAL AXIAL LOAD ANALYSIS IN NVSHAFT

NVShaft is a MATLAB based, finite-difference program currently under development at University of Nevada, Reno. The program was originally developed as part of an NDOT funded research project to improve numerical lateral load analysis in Nevada's local soil condition (Bhuiyan et al. 2020). The capabilities of NVShaft have been extended to perform numerical axial load ( $t$ - $z$ ) analysis, where the drilled shaft is modeled as a linear elastic and perfectly plastic axially loaded beam similar to other commercial programs such as RSPile (Rocscience 2018). The complex soil-shaft interaction is simplified by replacing the encompassing soil with a series of  $t$ - $z$  springs along the length of the shaft, to characterize side resistance and a  $q$ - $z$  spring at shaft tip location, to characterize end bearing (Mosher and Dawkins 2000). The numerical representation of side resistance and end bearing resistance mobilization in axially loaded drilled shafts for conventional top-down and bi-directional static tests for a given applied load  $Q_a$  is shown in Fig. 1. In a bi-directional static load test, the axial load is applied via bi-directional cell. The cell assembly is embedded within the shaft concrete, typically at a depth intended to achieve equal mobilization above and below the cell (Brown et al. 2010). To obtain the axial load response, the following differential equation can be solved using a finite-difference approach (Rocscience 2018),

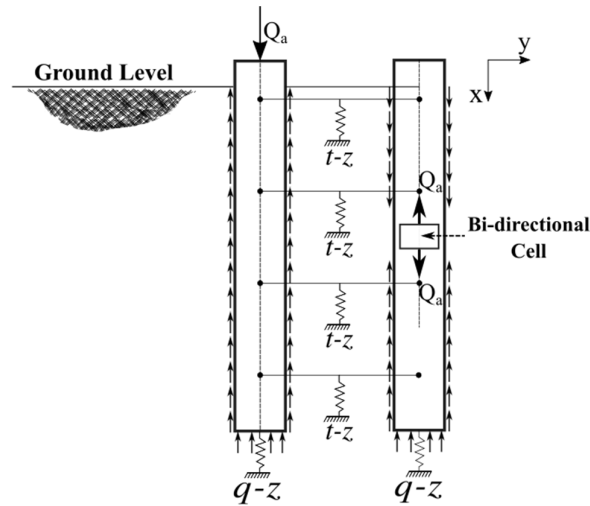
$$-EA \frac{d^2z}{dx^2} + tC = 0 \quad [1]$$

where,  $E$  = elastic modulus of shaft section,  $A$  = shaft cross-sectional area at depth  $x$ ,  $z$  = vertical shaft movement at depth  $x$ ,  $t$  = soil side resistance at depth  $x$ , and  $C$  = circumference of shaft segment at depth  $x$ . The mobilized axial force ( $Q$ ) in the shaft at depth  $x$  due to axial deformation is given by,

$$Q(x) = -EA \frac{dz}{dx} \quad [2]$$

While performing numerical  $t$ - $z$  analysis, NVShaft solves Eq. [1] by implementing relevant boundary conditions depending on the type of axial load test simulation. For a top-down static load test, the boundary condition is the applied load at the shaft head, and end bearing resistance at the shaft tip obtained from  $q$ - $z$  model. Shaft tip displacement can be also be specified as a boundary condition instead of applied load. To simulate bi-directional static load test, the user can specify the location of the applied axial load (i.e., bi-directional cell), which is used as an internal boundary condition in the finite-difference domain. Zero axial load at the shaft head and end bearing resistance at the shaft tip are also applied as boundary conditions in this case. Using Eq. [2], NVShaft computes mobilized axial load

along shaft length. Users can select suitable  $t$ - $z$  and  $q$ - $z$  curves from the NVShaft library to model side resistance and end bearing resistance for different material types. A summary of such available models included in NVShaft to date is shown in Table 1. Users can also specify their  $t$ - $z$  and  $q$ - $z$  models as user-defined inputs.



**Fig. 1. Numerical load transfer mechanism for axially loaded drilled shaft for conventional top-down and bi-directional static load test**

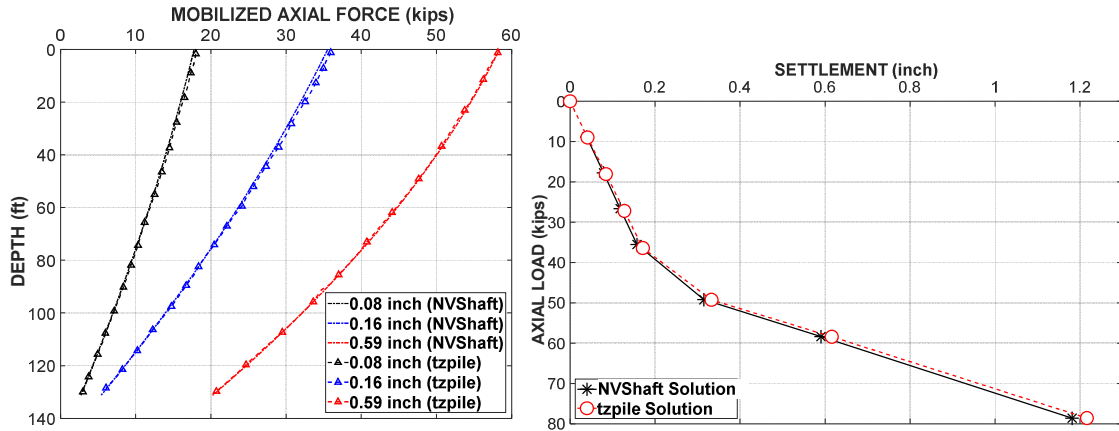
**Table 1. Summary of  $t$ - $z$  and  $q$ - $z$  models included in NVShaft**

Material Type	Model Name	Author	Model Type
Sand	API Sand	API (2014)	$t$ - $z$ and $q$ - $z$
	Mosher Sand	Mosher (1984)	$t$ - $z$ and $q$ - $z$
Clay	API Clay	API (2014)	$t$ - $z$ and $q$ - $z$
	Coyle Reese Clay	Coyle and Reese (1966)	$t$ - $z$ and $q$ - $z$
Rock	Florida Limestone	McVay and Niraula (2004)	$t$ - $z$
	Soft Rock	Asem and Gardoni (2019)	$t$ - $z$

The capability of NVShaft to produce reasonable outputs after performing numerical  $t$ - $z$  analysis was verified by comparing the outputs generated from TZ-PILE analysis. Example problem 1 from the TZ-PILE user manual (Ensoft 2014) describes a 131.2 ft (40 m) long open-ended steel pipe pile with an outside diameter of 3.3 ft (1 m) with wall thickness of 0.8 in (20 mm). Other relevant input parameters such as the  $t$ - $z$  models at the top and bottom of the steel pile and the  $q$ - $z$  model at the tip location can be obtained from the example description. The NVShaft and TZ-PILE predicted mobilized axial load profiles for three different levels of tip displacement; and a load-settlement comparison plot up to 1.2 inches (30 mm) of maximum tip displacement is shown in Fig. 2. A good match between NVShaft and TZ-PILE generated axial load response can be observed. Relative to TZ-PILE response, NVShaft predicted shaft head settlement resulted in a maximum of 2.88% deviation. Similar satisfactory observations were made based on a series of additional examples.

## MODELING SIDE RESISTANCE OF CALICHE

Numerical modeling of bi-directional static load tests in cemented soil was attempted in NVShaft, to evaluate the capability of two existing  $t$ - $z$  models to represent side resistance of caliche dominant soils. Looking at the soil exploration reports of the considered load test programs, limited information was obtained from the caliche deposits. This makes the characterization of caliche in numerical models challenging. In many cases, the SPT-N values are inconclusive and also represent the surrounding weak soil material due to small thickness. As the sampling of caliche is difficult, obtaining laboratory-measured  $q_u$  at multiple depths is often not possible.



**Fig. 2. Comparison between NVShaft and TZ-PILE predicted responses for verification: mobilized axial force profiles at specified shaft tip displacements (left) and load-settlement plot (right)**

Based on laboratory shear wave velocity measurements and unconfined compressive strength tests, Saint-Pierre (2018) proposed the following empirical formula for caliche,

$$q_u = 4 * 10^{-9} * V_s^{3.0724} \quad [3]$$

where,  $q_u$  is in units of psi.  $V_s$  is the shear wave velocity and is in units of ft/s. In the same study, empirical correlations between shear wave velocity, unit weight, and Young's Modulus were also proposed. These relationships, if properly utilized, can make reasonable estimates of the strength and deformation properties of caliche. The obtained material properties of caliche were implemented to generate two existing  $t$ - $z$  models of rock material in numerical simulations. A brief description of these models is discussed below.

#### ***Side resistance ( $t$ - $z$ ) model for Florida limestone***

A  $t$ - $z$  model for Florida limestone was formulated based on multiple instrumented axial centrifuge tests, all performed on 6 ft (1.83 m) diameter shafts with 18 ft (5.5m) of embedment depth (McVay and Niraula 2004). The test shafts were founded in synthetic limestone with unconfined compressive strengths of 20 ksf, 40 ksf, and 80 ksf. The obtained side resistance ( $t$ ) was normalized with ultimate side resistance ( $t_u$ ) and the displacement ( $u$ ) was normalized with shaft diameter ( $D$ ). The following  $t$ - $z$  model for Florida limestone was developed based on the normalized measured centrifuge data,

$$\frac{t}{t_u} = 0.96 * R^{0.33}, \quad 0 \leq R \leq 0.5 \quad [4]$$

$$\frac{t}{t_u} = 0.86 * R^{0.16}, \quad 0.5 \leq R \leq 3.0 \quad [5]$$

$$\frac{t}{t_u} = 1.0, \quad R > 3.0 \quad [6]$$

Where,  $R = z/D * 100$ . The ultimate side resistance can be approximated based on the FDOT design equation given in McVay (1992).

#### ***Side resistance ( $t$ - $z$ ) model for soft rock***

A  $t$ - $z$  model for drilled shafts socketed in soft rock material was proposed by Asem and Gardoni (2019). The parameters affecting the side resistance mobilization (i.e., initial shear stiffness, peak side resistance) were identified based on an axial load test database. An empirical framework was developed based on the compiled database to obtain necessary rock mass engineering properties to develop the  $t$ - $z$  model. Following the approach taken by Duncan and Chang (1970), Gupta (2012), and others, the

following hyperbolic equation was proposed up to the mobilization of peak side resistance,

$$t = \frac{z}{1/K_{si} + z \cdot R_f / t_{sp}} \quad [7]$$

where,  $K_{si}$  = initial shear stiffness,  $R_f$  = Fitting Ratio and  $t_{sp}$  = peak side resistance. Based on the assumption that the side resistance of soft rock decreases with post-peak displacement, the latter part of the proposed model is defined by introducing a brittleness index ( $I_B$ ), which is used to obtain the reduction in side resistance at the post-peak displacement of 0.59 inch (15 mm). Based on the observation made by Saint-Pierre (2018), for caliche,  $I_B$  may range between 0.67 to 0.89. The proposed  $t$ - $z$  model is formulated based on rock material properties such as mass rock modulus ( $E_m$ ), geological strength index (GSI), material constant ( $m_i$ ) and drained rock mass friction angle ( $\phi_m$ ). For carbonate rock sediments, the  $m_i$  ranges between 8 to 12 (Brown et al. 2010). Due to this small variation, the  $m_i$  for caliche was assumed to be 10 to calculate the soft rock  $t$ - $z$  model in all the load test simulations discussed in this paper.

## DETAILS OF AXIAL LOAD TESTS IN CALICHE

Two load tests from the I-15/US 95 reconstruction project (Kleinfelder 1996) and one from the Las Vegas City Center project (LOADTEST 2005) were modeled in NVShaft. The drilled shafts from the load test programs had diameters ranging from 2 ft to 8 ft, and embedded depths ranging from 32 ft to 116.8 ft. Table 2 summarizes the drilled shaft properties, location of the bi-directional cells, upper and lower depths, and strengths of the caliche layers. In all the load tests, the bi-directional cells were installed below the caliche layers. The 8 ft diameter shaft from the I-15/US 95 project had bi-directional cells located right below the caliche layer. The same can be said about the 2 ft diameter shaft, which also had a caliche layer below the bi-directional cell. The details on the mentioned load test programs are briefly described below.

**Table 2. Drilled shaft configuration; location, and strength of caliche layers from the mentioned load test programs**

Load Test Program	Shaft Diameter (ft)	Shaft Embedment Depth (ft)	Depth of Bi-Directional Cell(s) (ft)	Upper and Lower Depths of Caliche Layers (ft)	Unconfined Compressive Strength of Caliche (psi)
I-15/ US 95	8	32	21	13.5 - 21	9,583
				14-17	6,000
	2	82.5	39.1	30.5-37.5 43-44	6,000 6,000
City Center, LV	4	116.8	60	16.5-19	2,354
				32.5-36	2,354

### *I-15/ US 95 load test program*

A large-scale load test program carried out as part of the I-15/US 95 interchange upgrade at Las Vegas, Nevada consisted of a total of five lateral and ten bi-directional static load tests in four different locations (Kleinfelder 1996; Bhuiyan et al. 2020). The responses from the axial load tests were utilized to assess the side resistance and end bearing capacity of the drilled shaft in cemented soil conditions. Site characterization comprised standard penetration tests at five boring locations and several laboratory tests. The majority of the soil profiles consisted of partially-cemented dense clayey and silty sand with some intermittent hard to very hard caliche layers. Hard cemented soil and caliche were sampled using Nx size coring equipment. Based on unconfined and triaxial compressive strength tests performed on caliche samples, the unconfined compressive strength value ranged from 4,060 psi to 10,645 psi. For

both 8 ft and 2 ft diameter test shafts, strain gages were installed at seven levels from top of shafts to bi-directional cell depths.

The 8 ft diameter shaft was designed to carry a maximum of 18,000 kips of axial load. A maximum of 3,914 kips of axial load was applied in 200 psi increments, resulting in 1.351 inches of upward shaft head movement and 0.807 inches of downward bi-directional cell movement. It was concluded that the shaft failed simultaneously in side resistance and end bearing. The load was applied in two more cycles, which caused some degree of radial upward heave of soil and crack formation. At 2,228 kips of test load, the reported soil upward heave varied from 0.75 inches at shaft edge to 0.37 inches at a radial distance of 5 ft. The reported axial strain profile indicated tensile strain at the first strain gage level at 5.7 ft depth and also around the upper depth of the existing caliche layer at higher load levels. As stated by Karakouzian et al. (2015), strong bonding between the competent caliche layer and the shaft concrete often results in a monolithic response at the interface with an inadequate amount of slippage. The monolithic soil-shaft response is a possible reason for the upward movement of the ground surface in this case. As explained by Sinnreich (2012), the development of tensile strain is the indication of possible micro-fracturing of the shaft concrete material. The very stiff cemented soil or caliche may have restricted the elastic compression of shaft material during the curing process. When the internal stress within the shaft concrete exceeds the tensile limit, micro-fractures are formed (Sinnreich 2012). Crushing of concrete material resulting in the damage to the bi-directional cells was also observed (A. Bafghi, personal communication, October 24, 2019), which explains the tensile strain value near the bi-directional cells location. These special observations regarding this test shaft will be crucial in comparing the measured response with NVShaft predictions.

In the first axial load tests conducted on the 2 ft diameter shaft, a maximum of 978 kips of the axial load was applied in 200 psi pressure increments. The maximum upward movement of shaft top and downward bi-directional cell movements were 0.012 inches and 0.221 inches, respectively. The test was terminated as inadequate strength of shaft concrete was observed from the concrete cylinder strength test.

### ***Las Vegas City Center load test program***

A bi-directional static load test conducted on a 4 ft diameter, 116.8 ft long shaft as part of the Las Vegas City Center project (LOADTEST 2005) was considered in this study. The shaft head and tip were located 5.2 ft and 122 ft below the ground surface, respectively. Pair of strain gages were installed at three levels each, both above and below the bi-directional cell assembly. To measure elastic compression between shaft top and bi-directional cell, steel pipes along with telltales were used. Subsurface investigation at the test shaft location revealed the presence of clayey sand, gravel, sandy clay, and caliche. Two caliche layers with 2.5 ft and 3.5 ft thickness were identified above the bi-directional cell located at 60 ft. No laboratory test results regarding the caliche layers were mentioned in the available load test report, and only the SPT-N value at the bottom of the second caliche layer was reported. A maximum of 4,720 kips of axial load was applied, resulting in 0.32 inches and 1.29 inches of the upward top of and downward bottom of bi-directional cells movements, respectively. It was mentioned that the shaft movement exceeded the approximated creep limit, particularly in side resistance below the bi-directional cell. Crushing of concrete near the cell location is reported to be the possible reason. The equivalent top load-settlement curve was back-calculated, indicating 0.25 inches of shaft settlement with 0.18 inches of elastic compression corresponding to 3,350 kips of maximum axial load.

### **AXIAL LOAD TESTS SIMULATIONS IN NVSHAFT**

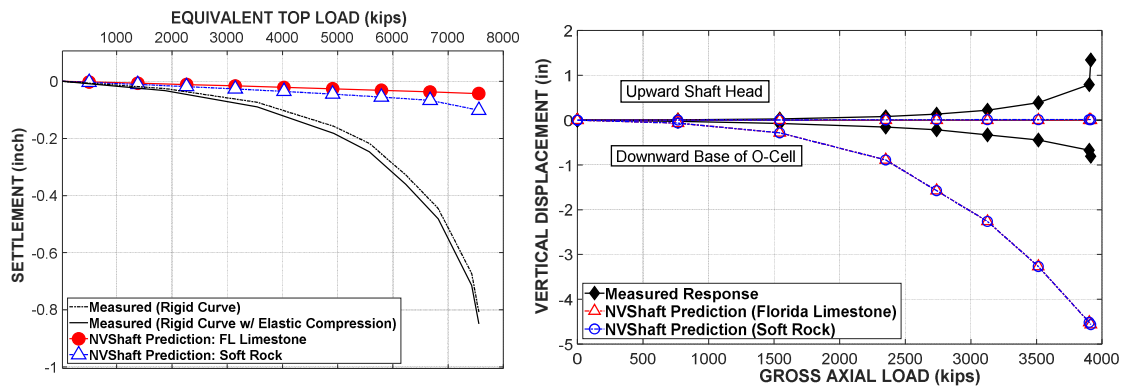
The mentioned load test programs were modeled in NVShaft, to simulate both conventional top-down and bi-directional static tests through numerical  $t$ - $z$  analysis. The side resistance and end bearing of sand and clay material were generated using the API Sand and API Clay  $t$ - $z$  and  $q$ - $z$  models (API 2014). The relevant soil material properties to use as input parameters were either obtained from their respective load test reports or calculated from several empirical correlations from FHWA (Brown et al. 2010) and Caltrans (2019) manual. Separate NVShaft simulations were carried out after implementing the Florida

limestone (McVay and Niraula 2004) and soft rock  $t$ - $z$  models (Asem and Gardoni 2019) for all the caliche layers. The empirical formula proposed by Saint-Pierre (2018), as shown in Eq. [3], was used to obtain the unconfined compressive strength of caliche material in cases where laboratory measured data were not available. It should be noted that using such formula adds to the uncertainty of the numerical models.

### *Numerical predictions of I-15/ US 95 load test program*

The NVShaft predicted and field-measured axial load responses for both conventional top-down and bi-directional static load test simulations of 8 ft diameter shaft are presented in Fig. 3. The single caliche layer encountered in this particular load test was modeled using the side resistance ( $t$ - $z$ ) models given for Florida limestone (McVay and Niraula 2004) and soft rock (Asem and Gardoni 2019). The results of the analyses including the corresponding NVShaft predicted responses are shown in Fig. 3. The constructed equivalent top-down rigid curve based on field load test data, considering both scenarios of rigid and elastic compression is also presented. For both types of load test simulations, the measured field responses were significantly softer compared to the NVShaft predicted responses. This discrepancy can be attributed to the deficiency in drilled shaft construction i.e., the possibility of micro-fracturing of concrete, leading to highly nonlinear shaft stiffness (Sinnreich 2012). The severe monolithic soil-shaft response due to the strong bonding between caliche and shaft concrete (Karakouzian et al. 2015) is another possible reason behind the softer response in the field. Since there was no caliche layer below the bi-directional cells assembly, identical downward movement was predicted in both bi-directional load test analyses using both  $t$ - $z$  models.

The 2 ft diameter shaft had two caliche layers above and one below the bi-directional cell location. As shown in Fig. 4, using both  $t$ - $z$  models resulted in fair agreements between NVShaft predicted and measured responses, in both types of load test simulations. Unlike the previous case, the presence of a caliche layer below the bi-directional cell location resulted in a difference in predicted downward movement of the shaft, when different  $t$ - $z$  models were implemented in that location. As seen in Fig. 4, the axial load response of this particular drilled shaft up to the maximum applied axial load is really small, indicating insufficient mobilization of side resistance. Limited displacement between the shaft and surrounding caliche layer during bi-directional load tests often results in partial mobilization of side resistance (Fellenius and Ann 2010; Karakouzian et al. 2015). These types of outcomes from the axial load test present challenges in achieving the necessary accuracy in drilled shaft design and reducing costs of construction (Paikowsky and Tolosko 1999). One benefit of performing  $t$ - $z$  analysis is that higher axial loads can be assigned to the models to achieve hypothetical failure conditions. For this particular test shaft, further numerical simulations using the  $t$ - $z$  model developed for Florida limestone predicted significantly higher axial load capacity (around 12,000 kips) compared to the  $t$ - $z$  model for soft rock (around 3,000 kips). The  $t$ - $z$  model for soft rock also resulted in softer axial load response in all the axial load test simulations mentioned in this study.

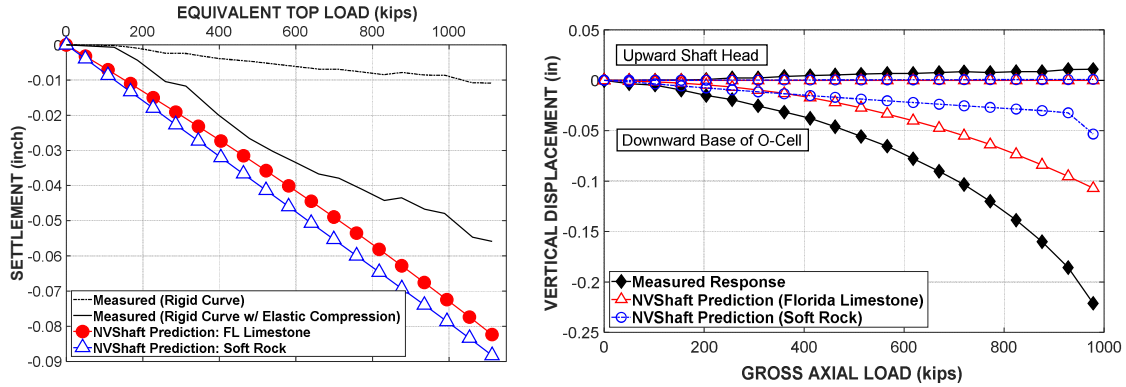


**Fig. 3. NVShaft predicted responses of 8 ft diameter shaft (I-15/US 95 project) from conventional top-down (left) and bi-directional static (right) load test simulations, along with measured data.**



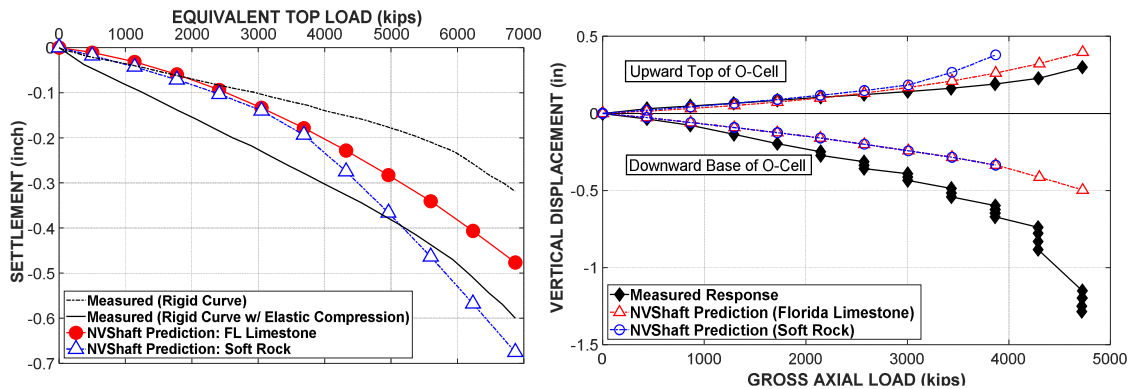
### Numerical predictions of Las Vegas City Center load test program

Similar to the previous two analyses, the 4 ft diameter shaft from the Las Vegas City Center project was modeled and analyzed in NVShaft. The load-displacement plots shown in Fig. 5, suggest reasonable agreement between the measured and predicted responses for both of the considered  $t$ - $z$  models.



**Fig. 4. NVShaft predicted responses of 2 ft diameter shaft (I-15/US 95 project) from conventional top-down (left) and bi-directional static (right) load test simulations, along with measured data.**

The equivalent top load-settlement curve with elastic compression based on measured bi-directional axial load response was softer, compared to the NVShaft predicted top-down load responses. As explained by Afsharhasani et al. (2020), the mobilization of side resistance in caliche is less in this case during the bi-directional test, compared to the case when the axial load is applied at the top. The bi-directional cell was installed 24 ft below the nearest caliche layer, which caused the applied load to be transferred in weaker soil layers before reaching caliche, compared to the top-down case scenario. Also, the  $t$ - $z$  model for soft rock failed to predict the shaft response at maximum test load (4720 kips), indicating lower axial load capacity in side resistance in numerical analysis (Fig. 5). A comparison between measured and predicted mobilized gross axial load profiles at three different loads, along with generalized soil profile is presented in Fig. 6. The sharp decreases in mobilized loads can be observed at the locations of caliche layers, which contradicts the relatively gradual change in mobilize load based on measured data. The existing  $t$ - $z$  and  $q$ - $z$  models failed to capture the exact load transfer mechanism in both side resistance (along shaft length) and end bearing (at shaft tip) resistances in this case.

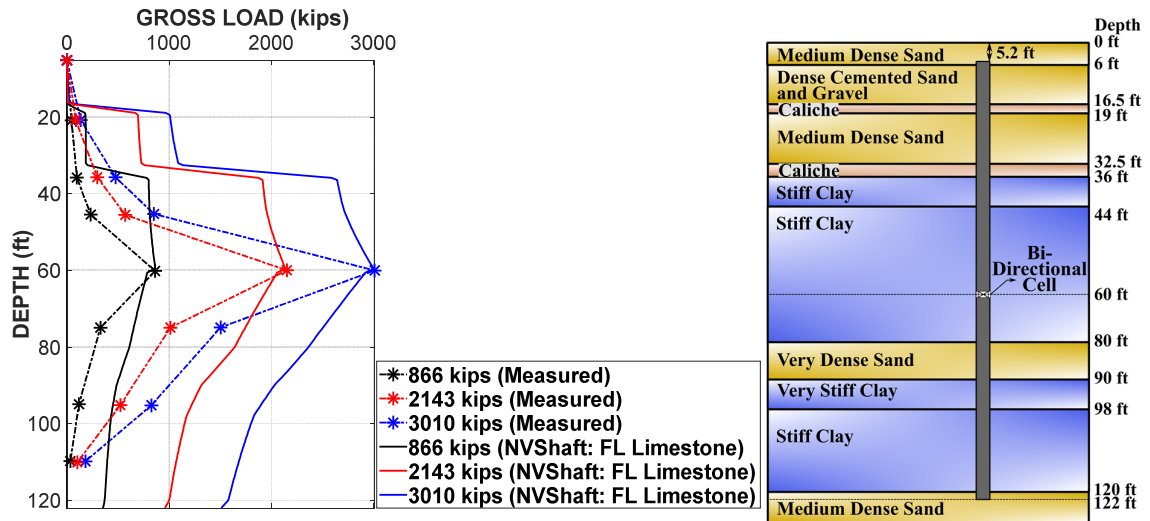


**Fig. 5. NVShaft predicted responses of 4 ft diameter Shaft (Las Vegas City Center Project) from conventional top-down (left) and bi-directional static (right) load test simulations, along with measured data.**



## SUMMARY AND CONCLUSIONS

Two existing  $t$ - $z$  models, formulated for Florida Limestone and weak rock, to simulate side resistance for caliche were evaluated based on numerical axial load analysis. Three bi-directional static load tests performed in caliche dominant sites from I-15/US 95 and Las Vegas City Center projects were considered. A MATLAB-based, finite-difference program, NVShaft was used to perform  $t$ - $z$  analyses, which is capable of simulating both conventional top-down and bi-directional static load tests. The side resistance characteristics of the caliche layers reported from site investigations were generated using the mentioned  $t$ - $z$  models.



**Fig. 6. Comparison between measured and predicted mobilized axial gross load at three different load levels (left) and schematics of the generalized soil profile (right) for 4 ft diameter shaft (Las Vegas City Center Project).**

From the I-15/US 95 project, drilled shafts with 8 ft and 2 ft diameters were modeled. For the 8 ft diameter shaft, the deficiency in drilled shaft construction leading to the possible formation of micro-fractures in shaft concrete resulted in highly nonlinear stiffness. Upward soil heave and radial crack formation due to possible monolithic soil-shaft response were also reported. Both of these facts attribute to the softer measured axial shaft response compared to NVShaft predicted ones for the 8 ft diameter shaft. For the 2 ft diameter shaft, NVshaft analysis produced a reasonably good predicted response compared to measured data. In this case, the measured response was small at the maximum applied bi-directional load, and similar to many axial load tests conducted in caliche, the axial load capacity of the shaft was inconclusive. By exploiting NVShaft's capability to simulate response at higher axial load, it was observed that using the  $t$ - $z$  model for Florida limestone resulted in higher capacity, and stiffer response compared to the  $t$ - $z$  model for soft rock. A similar observation was made based on axial load test simulations of all the test shafts mentioned in this study.

A reasonable match between measured data and NVShaft predicted top-down and bi-directional static axial load response for the 4 ft diameter shaft from the Las Vegas, City Center project was obtained, after applying both  $t$ - $z$  models. The equivalent top load-settlement curve obtained from measured bi-directional cell movement indicated a softer response, compared to the top-down predicted response. This observation substantiates the findings by Afsharhasani et al. (2020), and emphasize the location of bi-directional cell relative to caliche in interpreting the measured load test results.

## ACKNOWLEDGMENTS

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## REFERENCES

- Afsharhasani, R., Karakouzian, M., and Farhangi, V., 2020. Effect of competent caliche layers on measuring the capacity of axially loaded drilled shafts using the Osterberg test. *Applied Sciences*, 10(18), 6169. MDPI AG. <http://dx.doi.org/10.3390/app10186169>. Accessed 4-3-21.
- API., 2014. API recommended practice 2A-WSD - planning, designing, and constructing fixed offshore platforms – working stress design. 22<sup>nd</sup> ed. American Petroleum Institute.
- Asem, P., and Gardoni, P., 2019. A load-transfer function for the side resistance of drilled shafts in soft rock. *Soils and Foundations*, 59(5), pp. 1241-1259.
- 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.
- Bhuiyan, F. M., Siddharthan, R. V., Motamed, R., and Sanders, D. H., 2020. Evaluation of a new p-y analysis tool for lateral analysis of drilled shafts using load tests in Nevada. DFI 45th Annual Conference on Deep Foundations, October 13-16 2020, pp. 303–312.
- 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.
- Cibor, J. M., 1983. Geotechnical considerations of Las Vegas valley. American Society of Civil Engineers, In Geological environmental and soil properties, pp. 351-373.
- Coyle, H.M. and Reese, L.C., 1966. Load transfer for axially loaded piles in clay. *Proceedings, ASCE*, Vol. 92, No. SM2.
- Duncan, J.M. and Chang, C.Y., 1970. Nonlinear analysis of stress and strain in soils. *J. Soil Mech. Found. Div.* 96 (SM5), pp. 1629–1653.
- Ensoft., 2014. Analysis of load versus settlement for an axially-loaded deep foundation. Austin, TX, Ensoft, Inc.
- Fellenius, B. H., and Ann, T. S., 2010. Combination of O-cell test and conventional head-down test. In *Art of Foundation Engineering Practice*, pp. 240-259.
- Gupta, R.C., 2012. Hyperbolic model for load tests on instrumented drilled shafts in intermediate geomaterials and rock. *J. Geotech. Geoenviron. Eng.* 138 (11), pp. 1407–1414.
- Karakouzian, M., Afsharhasani, R., and Kluzniak, B., 2015. Elastic analysis of drilled shaft foundations in soil profiles with intermediate caliche layers. *IFCEE 2015*, pp. 922-928.
- Kleinfelder, Inc., 1996. I-15/US 95 Load test program, Las Vegas, Nevada. Final Report, January 26.
- LOADTEST, Inc., 2005. Report on drilled shaft load testing (Osterberg method), City Center - test shaft 1, Las Vegas, NV. Project No. LT-9160-1, September 16, 2005.
- McVay, M. C., Townsend, F. C., and Williams, R. C., 1992. Design of socketed drilled shafts in limestone. *Journal of geotechnical engineering*, 118(10), pp. 1626-1637.
- McVay, M.C., and Niraula, L., 2004. Development of PY curves for large diameter piles/drilled shafts in limestone for FBPIER. No. Final Report.
- Mosher, R. L., 1984. Load transfer criteria for numerical analysis of axially loaded piles in sand. U. S. Army Waterways Experiment Station, Automatic Data Processing Center, Vicksburg, Mississippi.
- Mosher, R.L. and Dawkins, W.P., 2000. Theoretical manual for pile foundations. Report ERDTC/ITL TR-00-5, US Army Corps of Engineers, Washington, DC.
- Motamed, R., Elfass, S., and Stanton, K., 2016. LRFD resistance factor calibration for axially loaded drilled shafts in the Las Vegas valley. Nevada Dept. of Transportation, Report No. 515-13-803.
- Paikowsky, S.G., and Tolosko, T.D., 1999. Extrapolation of pile capacity from non-failed load tests. Publication. No. FHWA-RD-99-170, U.S. Department of Transportation, Washington, DC.
- Rocscience., 2018. Axially loaded piles. Toronto, Canada, Rocscience Inc.
- <https://link.springer.com/content/pdf/10.1007/BF02885899.pdf>. Accessed 4-3-21.
- Saint-Pierre, E. C., 2018. The Development of a material model for engineering behavior characteristics of cemented soils for the Las Vegas valley. M.S. thesis, University of Nevada, Reno.

- Sinnreich, J., 2012. Strain gage analysis for nonlinear pile stiffness. *Geotechnical Testing Journal*, Vol. 35, No. 2, pp. 367-374.
- Stanton, K., Ellison, K., Motamed, R., and Elfass, S., 2015. An evaluation of tz analysis methods. DFI 40th Annual Conference on Deep Foundations, Oakland, CA, September 2015, pp. 303–312.
- Werle, J. L., and Luke, B., 2007. Engineering with heavily cemented soils in Las Vegas, Nevada. In *Problematic soils and rocks and in situ characterization*, pp. 1-9.