Proceedings of the 8th World Congress on Civil, Structural, and Environmental Engineering (CSEE'23)
Lisbon, Portugal – March 29 – 31, 2023
Paper No. ICGRE 106
DOI: 10.11159/icgre23.106

Interpretation of Static Load Tests on the Burj Khalifa's Foundation Piles

Anil Cherian [1]
[1]Strainstall Middle East L.L.C. (SME), Dubai, United Arab Emirates. dranilct@gmail.com

***Abstract* -** For assessing the geotechnical capacity of reinforced concrete bored piles, instrumented static compression load tests are often utilized in high-rise building projects. To better understand the geotechnical characteristics, this article discusses the axial behavior of preliminary compression test piles installed at the Burj Khalifa Foundation in Dubai. These tests were carried out for 1500mm and 900mm diameter piles prior to foundation construction, up to an ultimate test load of 60260kN. Vibrating wire concrete embedment strain gauges, comprising four units at six levels, were installed on the preliminary test piles. Results indicate that the skin friction was increased after the pile test as compared to the values used for the preliminary pile design. Subsequently, four working load tests were also performed during the construction of the foundation to confirm the final pile design. This article explores the results of static pile load test interpretation, an important factor in enabling reasonable construction settlement prediction, including the pile interaction effects.

***Keywords*:** Static load test, Skin friction, Pile stiffness, Pile Settlement

1. Introduction

With the large-scale construction of high-rise buildings in the Middle East, reinforced bore piles are used to support structures by transferring their loads to deeper and stronger soil layers. They are considered a favorable design option for sites with weak shallow soil layers or when supporting heavy structures. The type of structure, ground conditions, construction techniques, durability, load distribution, and cost are the few factors that may affect the selection of a tall building foundation [1].The complexity of the projects and the demands of their loading conditions have increased the need for more understanding of foundation systems and their behavior. The most suitable pile foundation test that could be employed would be a static load that replicates the interim and final geotechnical conditions as closely as possible. However, for practical reasons, it is desirable to carry out these initial load tests to minimize any design errors and allow construction to progress without any interruption.

The Burj Khalifa (formerly known as the Burj Dubai) is an 830-meter-high structure with a 163-story high-rise building in Dubai, United Arab Emirates (UAE). The tower is supported on a 3700mm thick raft and 192 bored piles of 1500mm diameter. The podium structures are founded on a 650mm to 1000mm thick raft, supported on 750 bored piles of 900mm diameter. The preliminary instrumented and working pile static load testing was performed by Strainstall Middle East LLC, Dubai. Studies on the process of foundation design and structural health monitoring of the Burj Khlaifa were discussed by various researchers [2-5]. Every pile, at the time of testing, has a single and unique behavior that can be determined by carefully controlled load application and pile displacement monitoring. ELPLA is a quick and practical software program for analyzing and resolving various foundation engineering problems [6]. With proper load application and pile-displacement monitoring, it is possible to identify each pile's distinct behavior at the time of testing and can be used for value engineering design. This article discusses the author's experience with the results of the pilot and working compression load tests and further evaluates pile foundation settlement using the ELPLA program.

2. Subsurface conditions

The subsurface geology of the United Arab Emirates, has been substantially influenced by the deposition of marine sediments associated with numerous sea level changes during relatively recent geological time. With the exception of mountainous regions shared with Oman in the north- east, the country is relatively low-lying, with near surface geology dominated by Quaternary to late Pleistocene age, mobile aeolian dune sands, and sabkha/evaporites deposits [7]. These superficial deposits were underlain by alternating beds of siliceous calcarenite, calcareous sandstone, siltstone, and conglomerates [3, 8]. The ground profile and derived geotechnical design parameters assessed from the investigation data with depth (DMD - Dubai Municipality Datum) are summarized in Table 1.

Table 1: Summary of ground profile and geotechnical parameters

Strata	Sub-Strata	Subsurface Material	Level at top of stratum (m DMD)	Stratum Thickness (m)	UCS q_u (MPa)	Undrained Modulus Eu (MPa)	Ultimate Shaft friction, fs (kPa)
1	1a	Medium dense silty Sand	+2.50	1.50	-	34.5	-
	1b	Loose to very loose silty Sand	+1.00	2.20	-	11.5	-
2	2	Very weak to moderately weak Calcarenite	-1.20	6.10	2.0	500	350
3	3a	Medium dense to very dense Sand/ Silt with frequent sandstone bands	-7.30	6.20	-	50	250
	3b	Very weak to weak Calcareous Sandstone	-13.50	7.50	1.0	250	250
	3c	Very weak to weak Calcareous Sandstone	-21.00	3.00	1.0	140	250
4	4	Very weak to weak gypsiferous Sandstone/ calcareous Sandstone	-24.00	4.50	2.0	140	250
5	5a	Very weak to moderately weak Calcisiltite/ Conglomeritic Calcisiltite	-28.50	21.50	1.30	310	285
	5b	Very weak to moderately weak Calcisiltite/ Conglomeritic Calcisiltite	-50.00	18.50	1.70	405	325
6	6	Very weak to weak Calcareous/ Conglomerate strata	-68.50	22.50	2.50	560	400
7	7	Weak to moderately weak Claystone/ Siltstone	-91.00	>46.79	1.70	405	325

2.1. Pile design

Even though various empirical relationships are suggested by researchers to calculate the ultimate unit shaft friction from the Unconfined Compressive Strength (UCS) of rock (Table 2), the most suitable one for the current site conditions was given by Horvath and Kenney [9]. Presently, the empirical coefficient utilized in the Middle East ranges from 0.25 to 0.35, with a suitable value of 0.30 based on the author's expertise in a variety of high-rise infrastructure projects.

Table 2: Summary of empirical relationships suggest by various researchers

References	Correlation	Equation
Horvath and Kenney [9]	Power	$f_s = 0.25q_u^{0.5}$
Williams et al. [10]		$f_s = 0.44q_u^{0.36}$
Rowe and Armitage [11]		$f_s = 0.41q_u^{0.57}$
Rosenberg and Journeaux [12]		$f_s = 0.34q_u^{0.51}$
Meigh and Wolski [13]		$f_s = 0.22q_u^{0.6}$
Cherian [14 -15]		$f_s = 0.30q_u^{0.5}$
Reynolds and Kaderabek [16]	Linear	$f_s = 0.30q_u$
Gupton and Logan [17]		$f_s = 0.20q_u$
Reese and O’Neill [18]		$f_s = 0.15q_u$

Toh et al. [19]		$f_s = 0.25q_u$

Ultimate unit shaft resistance, $f_s = 0.25\ q_u^{\ 0.5}$
where f_s is in kPa, and q_u = uniaxial compressive strength in MPa

3.0. Materials and methods

The test method used for axial load testing was the traditional reaction method (Fig. 1). It consists of six adjacent reaction piles for TP1 and TP2 and four reaction piles for TP4. The five compression test piles were instrumented with vibrating wire concrete embedment strain gauges at six different levels, with four strain gauges in each level. Six to nine numbers of calibrated 1000 ton capacity compression load cells and hydraulic jacks were installed to measure the load applied on the piles. Four 100 mm range displacement transducers were installed at 90 degrees apart between the reference beam and test pile cap to measure the vertical pile head settlement. An on-site pile load test was performed after achieving the required concrete strength. The pile load test has been performed as per the ASTM D1143 standard [20]. A data logging system with a laptop was used to collect, monitor, and process the strain gauge, load cell, and displacement transducer raw data. The load distribution along the pile shaft, unit shaft friction, and the load transfer to the soil were calculated from the strain gauge readings. Later, four working piles were tested to understand the load and settlement behavior and verify the initial pile design. The results of test piles (TP1 to TP5) and working piles (WP1 to WP4) are summarized in Table 3.

Fig.1: Static compression pile load test setup

Table 3: Results of Pile load tests (Preliminary and Working piles)

Pile type	Pile No.	Pile diameter (mm)	Working Load (kN)	Test Load (kN)	Settlement at Test Load (mm)	Stiffness at Test Load (kN/mm)
Preliminary piles	TP1	1500	30130	60260	21.26	2834
	TP2	1500	30130	60260	16.85	3576
	TP3	1500	30130	60260	20.24	2977
	TP4	900	10100	35070	26.62	1317
	TP5	900	10100	40160	27.45	1463
Working piles	WP1	900	10100	15150	4.20	3607
	WP2	900	10100	15150	4.40	3443
	WP3	900	7000	10500	2.10	5000
	WP4	1500	30130	45195	9.80	4611

4.0. Results and Discussion

4.1. Pile Stiffness

The stiffness values recorded during the preliminary (TP1 to TP5) and working (WP1 to WP4) load tests ranged from 1317 kN/mm to 5000kN/mm (Fig.2).

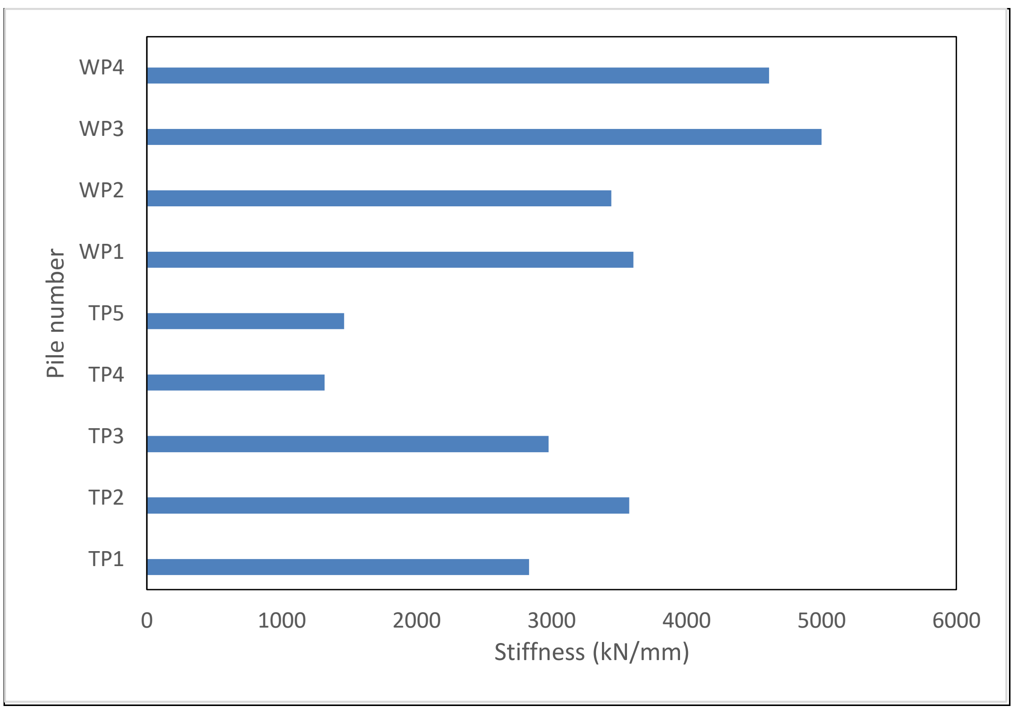

Fig.2: Stiffness values of preliminary and working piles

The measured stiffness values were slightly higher than expected for both preliminary and working piles due to the influence of polymer drilling fluid and the reaction piles. When a compression load is applied to the test piles, the reaction piles undergo tension, reducing the settlement of the test piles. Thus, the apparent high stiffness of the pile may not reflect the true stiffness of the pile beneath the structure [1]. Similar observations were made in different deep foundation projects [21]. Additional deflections are caused by the interaction effect of reaction piles and test piles. Additionally, the static load tests do not adequately account for the temporal effects of the surrounding soil, which could result in overestimated stiffness values. These effects are crucial where the reaction pile load was significant and therefore stressed the ground during load testing. Bidirectional static load tests are now mostly employed in foundation testing to avoid such interaction effects and inaccuracies from standard loading techniques [14 -15]. For large raft foundations like the present situation, the flexural stiffness is generally low, and hence the geotechnical parameters like load and settlement of supporting piles are important to minimize differential settlements.

4.2. Shaft friction capacity from load tests

As a part of the preliminary pile testing (PPT) program, on the main rebar of the preliminary test piles, vibrating wire concrete embedment strain gauges were inserted vertically at six elevations, four of them per level, and firmly attached to the rebar cage. Out of five test piles (TP1 to TP5), the compression load tests on preliminary piles TP1, TP2, and TP4 were taken into consideration for this study. The measured unit skin friction from preliminary load tests using strain gauges (Table 4) along with theoretical and recommended design values were presented (Fig.3). Based on the results, it was observed that the skin friction values up to -30.00 m DMD (meter Dubai Municipality Datum) appear to be fully mobilized and are the ultimate values (up to 839 kPa). The unit shaft friction values less than 200 kPa was recorded at depth below -30.00 m DMD and do not show any indication of fully mobilized ultimate values [1]. The initial pile design assumptions were comparable upper segment of the ground profile. As the lower segment of the pile was not fully mobilized, the values deduced from the geotechnical evaluation were used for the final pile design. A similar approach has been adopted in different tower foundation designs in the Middle East [22]. The unit skin friction appeared to be more than sufficient to resist loads considerably in excess of the working load, and there was no indication of tip resistance mobilization during the axial load testing [1].

Table 4: Measured unit skin friction from preliminary load tests

TP1		TP2		TP4	
Depth (m DMD)	Skin friction (kPa)	Depth (m DMD)	Skin friction (kPa)	Depth (m DMD)	Skin friction (kPa)
-11.10 to –17.56	468	-11.55 to -17.55	668	-9.00 to -17.545	335
-17.56 to –24.00	367	-17.55 to -24.00	236	-17.545 to -24.00	320
-24.00 to –30.00	839	-24.00 to -30.00	307	-24.00 to -30.00	407
-30.00 to –39.00	122	-30.00 to -44.00	171	-30.00 to -39.00	193
-39.00 to –48.00	26	-44.00 to -58.00	40	-39.00 to -48.00	56

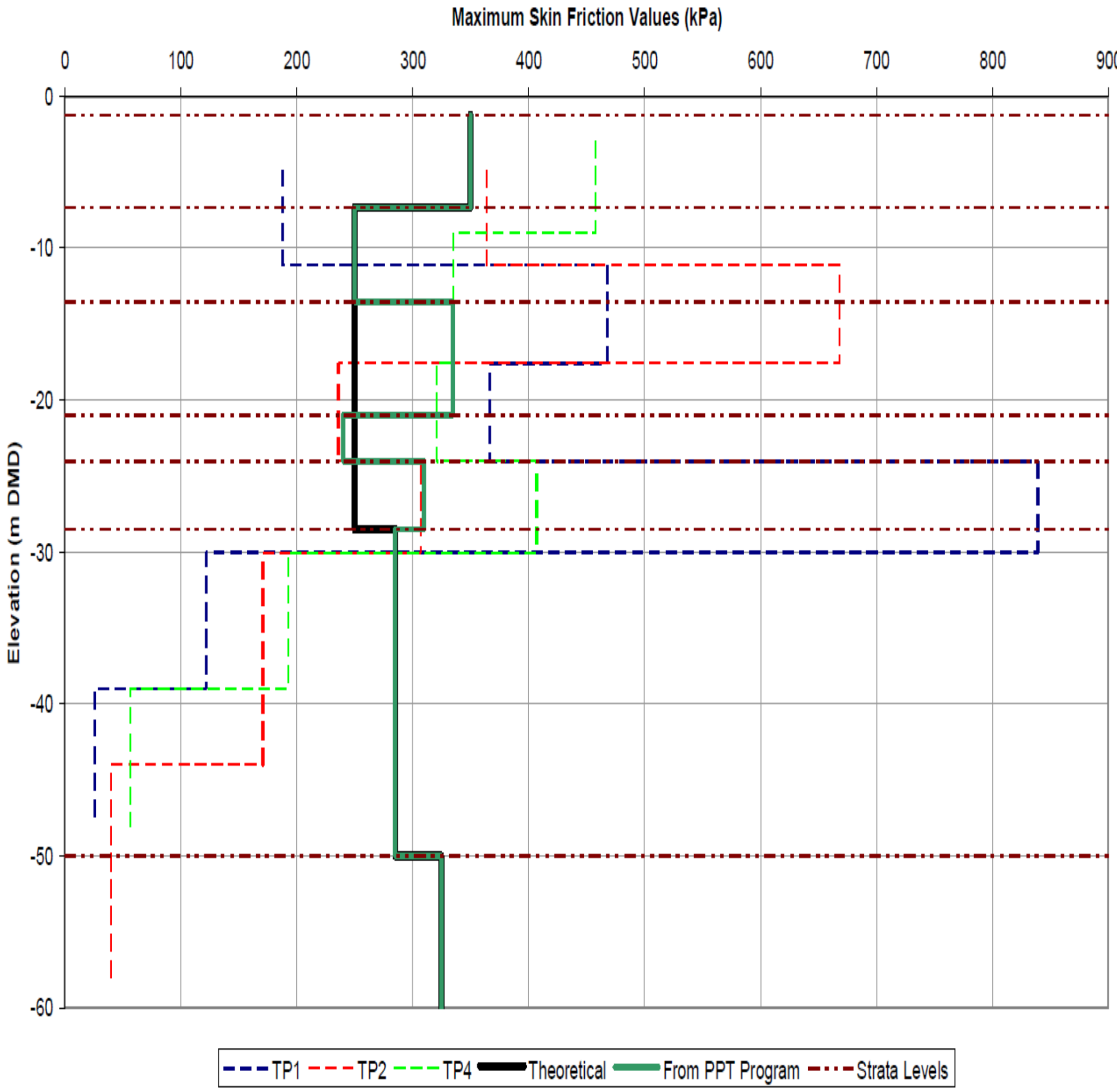

Fig.3: Skin friction values of preliminary and working piles

4.3. Settlement analysis

The settlement analysis was performed using various methods, which are presented in Table 5. The predicted total settlement of piles using ELPLA analysis is shown in Fig.4.

Table 5: Estimated settlement values

Analysis method	Maximum settlement (mm)	Minimum settlement (mm)	Difference (mm)
Design values [3]	78	60	18
NAPRA program [23]	52	25	27
Preliminary pile load tests	28	17	11
ELPLA program	58	29	29

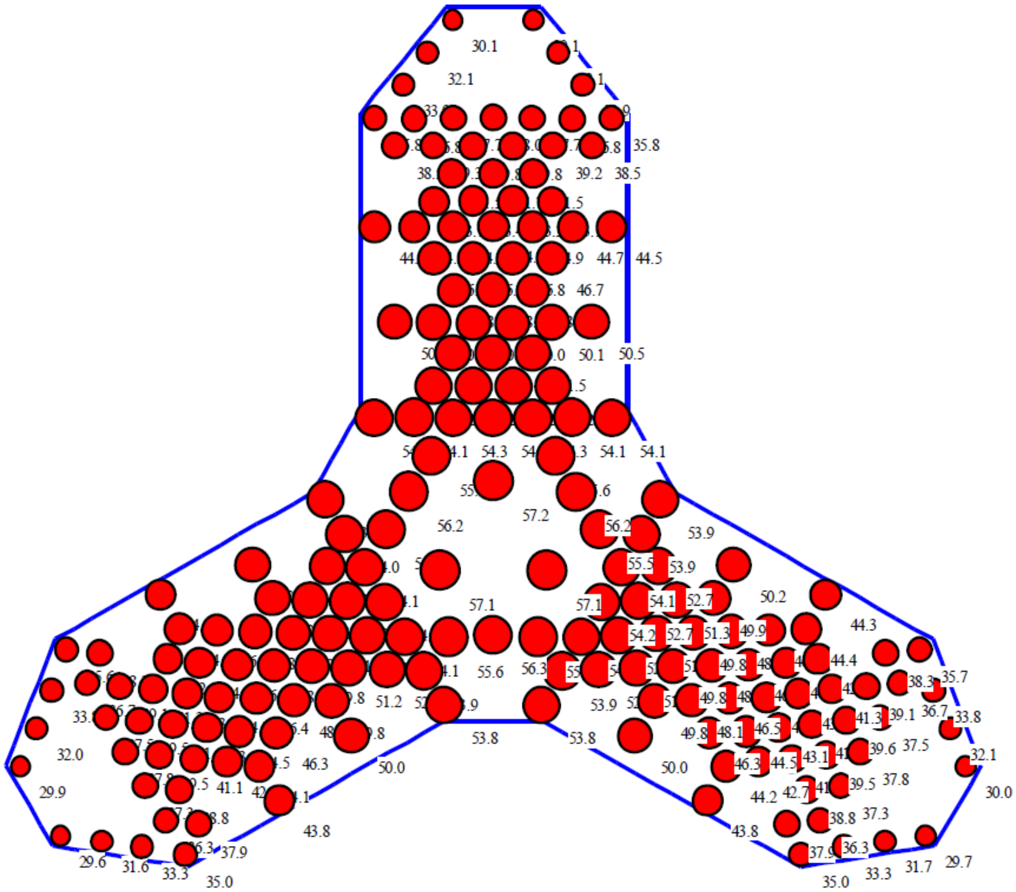

Fig.4: Predicated settlement of foundation piles

The anticipated settlement value exceeded the result of the initial load test. This is primarily caused by the extra expected settlement brought on by dead and live loads operating both during and after the superstructure's construction. Other likely explanations include the possibility that the consolidation rate was much slower than anticipated, an overestimation of pile interaction effects, a slower consolidation rate, a different distribution of ground stiffness with depth, etc. In comparison to other complex software models, it was determined that the results of the ELPLA analysis could provide a useful, quick, and acceptable estimation for pile settlement.

5. Conclusion

The importance of selecting suitable geotechnical parameters, particularly for problems involving traditional static loading of piles, is crucial. Most theories need empirical correlations to be employed in order to acquire appropriate design verification parameters for useful implementation of production piles. This study complemented the importance of using instrumented static load tests by taking proper account of pile interaction effects, ground stiffness, and settlement analysis to improve the understanding of pile-soil behavior. The pile foundation system in the study area is subjected to multidirectional loads, and the presence of cavity rocks as well as the chemical deterioration of steel and concrete may degrade the overall capacity. Therefore, the piled-raft foundation is a practical and cost-efficient solution for high-rise buildings in the region.

Acknowledgements

The author is grateful to Strainstall Middle East LLC (SME), United Arab Emirates for their continued support and encouragement.

References

[1] H. G. Poulos, "Tall building foundations: design methods and applications," *Innov. Infrastruct. Solut., vol.1, pp.1-51, 2016.*

[2] W. F. Baker, D. S. Korista and L. C. Novak, "Burj Dubai: engineering the World's Tallest Building," *Struct. Des Tall Spec Build.*, vol. 16, pp.361–375, 2007.

[3] H. G. Poulos and G. Bunce, "Foundation design for the Burj Dubai—the world's tallest building,'' *in Proceedings of 6th International Conference on Case Histories in Geotechnical Engineering*, Arlington, VA, 2008, Paper 1.47, CD volume.

[4] A. Smith, "Burj Dubai: designing the world's tallest," *CTBUH 8th World Congress, Dubai*, 2008.

[5] A. Cherian, "Geotechnical and geological aspects of Structural Health Monitoring in high-rise buildings ," J. Ind Geoph Union, vol.24, pp. 10-24 , 2020a.

[6] O. El. Gendy and A. El. Gendy, "Analysis of Piled raft of Burj Khalifa in Dubai by the program ELPLA," Canada, 2018.

[7] H. G. Poulos, "A review of geological and geotechnical features of some Middle Eastern countries," *Innov. Infrastruct. Solut., vol.3, pp.1-18, 2018.*

[8] A. Cherian, "Static Behavior of Strain Gauge Instrumented Continuous Flight Auger (CFA) Piles," *Jordan J. Civil Engg*, vol.16, pp. 229-241, 2022.

[9] R. G. Horvath and T.C. Kenney, Shaft resistance in rock socketed drilled piers," *in Proceedings of the Symposium on Deep Foundation*, American Society of Civil Engineers, New York, 1979, pp.182-214.

[10] A.F.Williams, I.W. Johnston and I.B. Donald, "Design of socketed piles in weak rock," *in Proceeding of International Conference on Structural Foundations on Rock*, Australia, 1980, pp. 327-347.

[11] R.K.Rowe and H.H.Armitage, "Design of piles socketed into weak rock [Geotechnical research report]," Ontario, Canada, 1984, pp.11-84.

[12] P. Rosenberg, N.L.Journeaux, "Friction and end bearing tests on bedrock for high capacity socket design,'' *Can. Geotechnical J.*, vol.13, pp. 324 – 333, 1976.

[13] A.C.Meigh, W. Wolski, "Design parameters for weak rocks," *in Proceeding of the 7th European conference on soil mechanics and foundation engineering*, British Geotechnical Society, Brighton, 1979, pp. 59-79.

[14] A.Cherian, "Assessment of pile capacity using Bidirectional Static Load Test (BDSLT) ," Ind Geot J ., vol. 51, pp.369-375, 2020b.

[15] A.Cherian, "Bi-directional static load test (BDSLT) for economically sustainable pile capacity appraisal," *Arabian J. Geosciences*, vol.14, https://doi.org/10.1007/s12517-021-08753-1, 2021.

[16] R.T.Reynolds and T.J. Kaderabek, "Miami limestone foundation design and construction,'' J. Geotechnical Eng Division, vol. 107, pp.859-872, 1981.

[17] C. Gupton and T. Logan, "Design guidelines for drilled shafts in weak rocks of South Florida" *in Proceeding of the South Florida annual ASCE meeting*, USA, 1984.

[18] L.C.Reese and M.W.O'Neill, "Drilled shafts: construction procedures and design methods," Federal Highway Administration, Washington, D.C., 1988.

[19] C.T.Toh, T.A. Ooi , H.K.Chiu, S.K.Chee and W.N.Ting, "Design parameters for bored piles in a weathered sedimentary formation," *in Proceeding of the 12th International Conference on Soil Mechanics and Foundation Engineering*, Rio De Janeiro, 1989.

[20] ASTM D1143, "Standard test method for piles under static axial compressive load," ASTM International, USA, 1994.

[21] H.G. Poulos, "Pile testing—from the designer's viewpoint.,'' *in Kusakabe, Kuwabara, Matsumoto, Balkema (eds) Statnamic loading test*, Rotterdam, 2000, pp. 3-21.

[22] H. G. Poulos and F. Badelow, “Geotechnical Parameter Assessment for Tall Building Foundation Design,” *Int.J. High-Rise Buildings*, vol.4, pp.227-239, 2015.
[23] G. Russo, V. Abagnara, H.G. Poulos and J.C. Small, “Re-assessment of foundation settlements for the Burk Khalifa, Dubai,” *Acta Geotec*, vol. 8, pp.:3–15, 2013.