Effects of Loading on Stiffness and Capacity of Wind Turbine Hybrid Foundation under 1 g Modeling.

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ABSTRACT
Wind Energy generation from both onshore and offshore wind farms is growing rapidly. Monopile foundations offer construction advantages, and therefore are widely used, especially in North Sea offshore wind energy projects, as an effective foundation option for wind turbines. To further enhance the efficiency of monopole foundations for wind turbines applications, a new hybrid system that combines a monopile and a concrete plate is presented and tested in this work. Measured wind loads from boundary layer wind tunnel tests conducted at Western University on a model 5 MW NREL (National Renewable Energy Laboratory) wind turbine were used. A scaled-down non-dimensional framework of stiff foundation models installed in sand was used to conduct a series of static tests under 1-g. Two model foundations were tested in a laboratory setup. The test results were used to develop an equation to predict the plate effects of the proposed hybrid foundation system as an effective modeling technique in the lab.

RÉSUMÉ
La production d'énergie éolienne onshore et offshore croît rapidement. Les fondations monopieux offrent des avantages de construction et, par conséquent, sont largement utilisées, notamment en mer du Nord dans des projets d'énergie éolienne, car elles offrent une véritable option de fondation. Pour accroître l'efficacité des monopieux pour les applications éoliennes, un nouveau système hybride qui combine un monopieu et une plaque de béton est présentée et testée dans ce travail. Les charges de vent mesurées à partir d'essais réalisés dans le Boundary Layer Wind Tunnel de la Western University sur un modèle 5 MW NREL (Laboratoire national des énergies renouvelables) ont été utilisées. Un modèle réduit adimensionnel d'une fondation rigide installée dans du sable a été utilisé pour mener une série d'essais statiques sous 1 g. Deux modèles de fondations ont été testés au laboratoire. Les résultats du test ont été utilisés pour élaborer une équation afin de prédire les effets de la plaque sur le système de fondation hybride.

1 INTRODUCTION

Green energy resources are essential to meet the growing energy demands in the near future while reducing the effects of global warming. Offshore wind energy is one of the main efficient renewable energy sources, and therefore, offshore wind farms are continually expanding, especially in North Sea and China. One of the main cost items in the construction of offshore wind turbines is the foundation, which represents about 30-40% of the total cost (Byrne and Houlsby, 2003).

Several foundation systems are used to support wind turbines depending on the soil conditions and water depth. The gravity base foundation, which depends on its weight to resist the lateral load and overturning moment, is used in case of small water depth. It is usually cast onshore then moved to the offshore site to be erected in order to reduce its construction cost. Monopile foundations can be used to support wind turbines in wide range of soil conditions and water depth due to its versatility and suitability of construction in different conditions. Large offshore wind turbines are typically supported by a steel pile 4-6 m in diameter with length of 20-40 m (Houlsby et al. 2001). Suction caissons are also used to support wind turbines in a variety of soil conditions and water depth (Houlsby et al. 2001). A combination of the shallow footing and monopile can provide efficient foundation system for large offshore wind turbines (Leblanc 2010). The monopile and the combined, hybrid, offshore wind turbine foundations are presented schematically in Fig.1.

Most methods for analysis and design of offshore wind turbines foundations originated from the practices employed in the design of offshore oil and gas production rigs. However, there is a significant difference between the two foundation applications. Unlike the oil production rigs, the loading combination for wind turbines involves relatively small vertical loads but large horizontal and moment cyclic loads.

This relatively large lateral cyclic load can affect both the stiffness and the capacity of the foundation system. Additionally, while the monopole static capacity is important, the changes in its stiffness and accumulated rotation after long-term cyclic loading must be addressed as part of the stringent performance criterion that has to be satisfied. (Leblanc et al. 2010). As the long term cyclic loading could change the soil stiffness, the foundation stiffness can also be affected.

Cyclic response of laterally loaded pile is influenced by soil and pile yielding, soil-pile gapping and cyclic soil degradation. During cyclic loading, the response of piles installed in sand is also affected by soil cave-in and recompression. In addition, the soil may experience strength loss and modulus reduction. Procedures that are used in evaluating pile response should therefore be capable of accounting for these factors (e.g. Allotey and El Naggar 2008) and (Heidari et al. 2014).

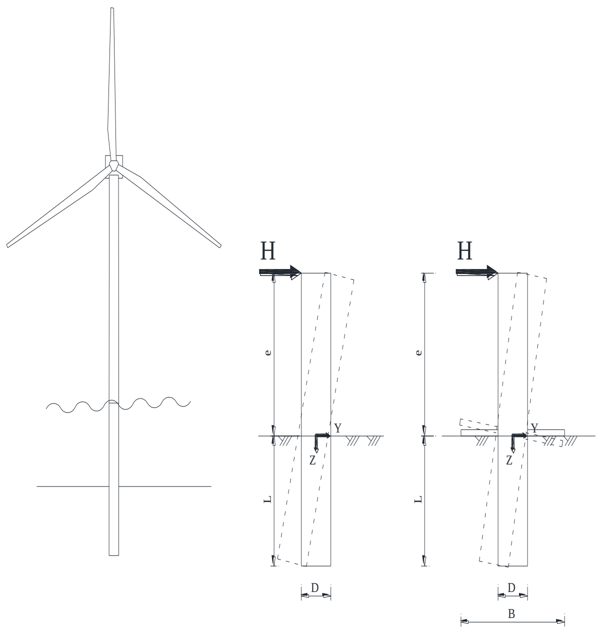


Figure 1. Offshore wind turbine foundations considered: a) offshore wind turbine; b) monopile; and c) hybrid foundation system.

The p-y curves approach (Reese & Maltock 1956); and (McCelleand & Focht, 1958) is widely used to evaluate the response of piles subjected to lateral loads. In this approach, the soil reaction, p, is related to the pile deflection, y. The shape of the p-y curve can be estimated based on laboratory results and back calculation of field performance data (e.g. Reese et al. 1974) or based on in-situ test results (Robertson, et al. 1986) through solving the pile equilibrium equation, i.e.,

$$E_p\, I_p \frac{d^4y}{dz^4} - p(y) = 0, z \in [0; L] \qquad [1]$$

Where E_p is the soil modules, I_p is moment or Inertia of the pile cross-section.

There are different methods available in the literature to establish the p-y curves for piles installed in saturated and unsaturated sand (Bhushan et al. 1981) and (Bhushan and Askari 1984) based on full-scale load test results. For long offshore piles installed in sand, (DNV 2011) proposed an equation to generate p-y curves, i.e.

$$p = Ap_u tanh\left(\frac{Bz}{Ap_u y}\right) \qquad [2]$$

Where A=0.9 for cyclic loading, B is initial modules of subgrade reaction and depends on the angle of friction and P_u is the pile ultimate lateral resistance. The p-y curves are mainly employed for the analysis of long and flexible piles. However, piles supporting offshore wind turbine are usually short and rigid, hence the p-y curves approach is not suitable for the analysis of their response.

The cyclic response of laterally loaded piles can also be evaluated employing the finite element method that treats the soil as a continuous medium discretized into elements (Aristonous et al 1991), (Bentley and El Naggar; 2000); and (Maheshwari et al. 2004).

The considered foundation systems included hollow steel monopiles with diameter equal to 6.0 m and a hybrid foundation system, which combines a monopile and a concrete plate as shown in Figure 2.

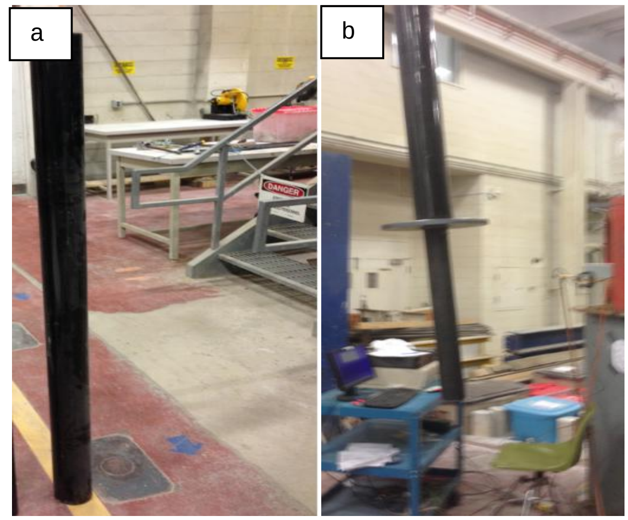


Figure 2. Different foundation models: (a) monopiles; (b) hybrid system with steel plate.

2 OBJECTIVES AND SCOPE OF WORK

The main objectives of this paper are twofold. First, is to evaluate the characteristics of the static response of monopiles and hybrid foundation systems and compare their performance experimentally. Second, is to establish an equation to evaluate the system capacity of the proposed hybrid foundation considering the contribution of the concrete plate. To achieve these objectives, 1-g small scale models of the monopiles and hybrid foundations are subjected to static loads in order to study their effects.

3 METHODOLOGY

According to Poulos & Hull (1989) defined pile flexibility factor, K_R can be given as:

$$\boldsymbol{K_R} = \frac{E_s L^4}{E_p I_p} \qquad [3]$$

Where E_s is elastic modules of the soil, L embedded pile length.

(Poulos & Hull 1989) suggested a range for K_R where the pile can be considered short and rigid, and will rotate without flexing, which is given by:

$$4.8 < \frac{E_s L^4}{E_p I_p} < 388.6 \qquad [4]$$

Considering the geometrical properties of monopoles with 4.0 to 6.0 m diameter of length up to 36 m, these piles can

Figure 3. Foundation systems considered in analysis (pile length L=36 m: (a) pile system D_p=6 m, upper section, D_t = 6m; (c) Hybrid System with D_{Pl}=12, 16 m. be considered rigid according to Eq. 4. On the other hand, the plate of the hybrid foundation system can be considered rigid if its flexural rigidity falls within the range suggested by i.e.:

$$0.005 < (E_s t^3 / E_{pl} D_{pl}^3) < 2 \qquad [5]$$

Where E_{pl} elastic modulus of the plate, t is the plate thickness and D_{pl} is the plate diameter.

LeBlanc et al. (2010) developed a non-dimensional framework for scaling stiff piles in sand, and applied it to interpret the test results of 1-g monopile small models. This methodology simulates the monopile lateral and rocking response accounting for the frictional behaviour of the sand, which depends on the isotropic stress level, and can be represented by:

$$\begin{bmatrix} \frac{M}{L} \\ H \end{bmatrix} = DG \begin{bmatrix} K_1 & K_2 \\ K_2 & K_3 \end{bmatrix} \begin{bmatrix} L\theta \\ u \end{bmatrix} \qquad [6]$$

Where: M is applied moment on the pile, L is embedded pile length is, H is applied lateral loads, D is pile diameter, G is shear modulus, θ is pile rotation, and u is pile lateral displacement. The parameters k_1, k_2 and k_3 are dimensionless constants. The full development of the method was provided by (Leblanc et al. 2010). The non-dimensional parameters that are used to scale down the model monopiles are presented in Table1.

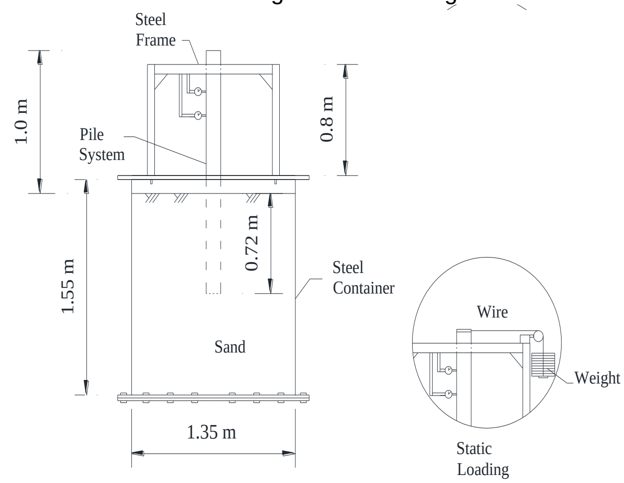
Table 1. Non dimensional parameters (Leblanc et al., 2010).

Non Dimensional Parameters	
Moment Loading	$M' = \frac{M}{L^3 D\gamma'}$
Vertical Force	$V' = \frac{V}{L^2 D\gamma'}$
Horizontal Force	$H' = \frac{H}{L^2 D\gamma'}$
Rotation Degree	$\theta' = \theta\sqrt{\frac{p_a}{L\gamma'}}$
Load Eccentricity	$e' = \frac{M}{HL}$
Aspect Ratio	$\eta = \frac{L}{D}$

Where γ is soil unit weight, p_a atmospheric pressure, parameters with hyphenation refer to normalized parameter.

4 EXPERIMENTAL TEST SETUP

The experimental test setup comprised a steel cylinder container, shown in Fig. 3, to enclose the test sand bed. It has a diameter of 1.35 m and depth of 1.55 m. A steel frame was installed on top of the container in order to guide the installation and leveling of the model piles and as a platform to support two linear variable displacement transducers (LVDTs), the static and dynamic load cells. A pulley system was used to conduct the static lateral loading. To facilitate loading at different load eccentricity, e, values (i.e. 0.50, 0.75 and 1.00 m) in order to produce horizontal load and rocking moment combinations representative of wind turbine loading conditions, C clamps were used to facilitate applying the load at different elevations loading as shown in Fig.3.



(a)

(b)

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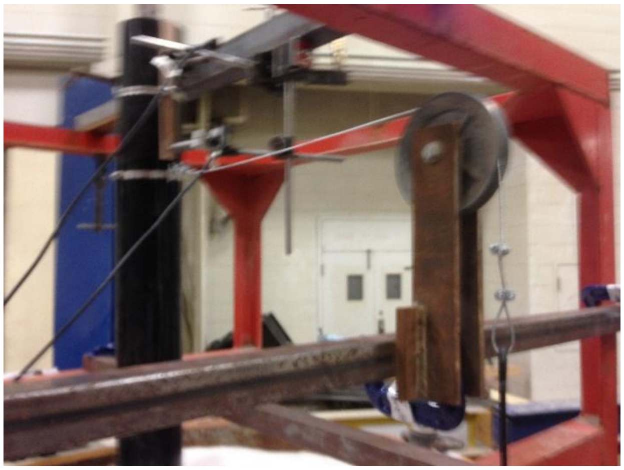
(c)

Figure 3. (a) System setup, (b) support system and (c) Load test setup for static loading

4.1 Soil model

The framework for small scale model tests of stiff piles installed in sand developed by (Leblanc 2010) depends on scaling the soil stiffness considering its angle of internal friction and relative density in order to scale down the vertical stress at 0.8 L. For yellow Leighton Buzzard sand, the scaling relationship between the model and full scale sand properties were provided by (LeBlanc 2010).

Ottawa sand F(50) was well characterized by Karl (2012) through extensive laboratory testing, which involved sieve analysis and direct shear tests, and the variation of its angle of internal friction, ϕ, with confining pressure for different values of relative density, D_r, The sand physical and engineering properties are provided in Table 2.

Table 2. Characteristics of Ottawa Sand (F50) – (after Helimigk et al, 2012).

Property	Value
Particle sizes, D_{10}, D_{30}, D_{50}, D_{60} mm	0.17, 0.24, 0.28, 0.32
Specific Gravity G_s	2..65
Maxim void Ratio, Minimum void ratio %	0.79, 0.59
Unit Weight KN/m^3	14.142
Critical Angle of Friction, ϕ_{cr}	32

(Karl 2012) concluded that the Ottawa sand F(50) exhibits different behavior compared to that of Leighton Buzzard sand. Several tests were conducted for different dry densities with range of 1378 kg/m^3 to 1682 kg/m^3. The range of stress at 0.8 L will be in the range of 9 kPa that require relative density of less than 0% in the model which is not possible. (Karl 2012) suggested using critical state approach to scale the soil (Altaee, 1994).

$$e_m = e_p + \lambda \, ln(n) \qquad [7]$$

Where e_m model void ratio, e_p prototype void ratio, λ is the slope of the critical state line (-1.46) and n is the geometric scale ratio.

4.2 Foundation model

Two different foundation models were tested: monopile with diameter 1.2 m; and hybrid system as the model was scaled with 1:50 scale taking into consideration that the geometry scaling is not related to soil scaling. One hybrid foundation had a steel plate 0.32 m in diameter while Table 3 presents their geometrical details.

The pile models were driven into the sand bed employing a hammer falling from fixed dropping distance. It took approximately 350 and 500 hammer blows to reach the final penetration depth for piles with diameter of 4.0 m and 6.0 m, respectively. hybrid foundation models was tested.

Table 3. Properties of steel pile.

Property	Value
Pile diameter, D mm	120
Plate Diameter, B mm	320
Wall thickness, t mm	5
Penetration depth, L mm	720
Load eccentricity, e mm	10000

5 TESTING AND DISCUSSION

A series of 6 static load tests for the two systems for different eccentricities as (0.5, 0.75 and 1m) .Pulley and C clamps were used with steel bars to set the test for each eccentricity. Fig. 4 shows the moment capacity determined from static load tests. The failure were defined when rotation of foundation, θ' = 4°=0.0698 rad.

$$\frac{3}{K}\frac{M}{DL^3\gamma'} = \alpha + 1 \pm 2\left(\frac{1}{2} + \alpha + \frac{1}{K}\frac{H}{L^2D\gamma'}\right)^{\frac{3}{2}} \qquad [8]$$

$$\alpha = \left(c_3\frac{V}{DL^2\gamma'} + \frac{\pi}{4}\frac{D}{L}\right)\frac{\sin\phi_{cr}}{K} \qquad [9]$$

Where K is a factor depending on the friction angle, critical state friction angle Φ_{cr} and c_3 is a dimensionless constant between 0 and 1.

A comparison between the lab results and the theoretical equation shows good agreement for both the monopiles cases in Fig 5. For the case of the hybrid system a new equation can be provided that describe the effect of the plate on improving the capacity of the system. The equation was developed by first considering the hybrid system with a plate as a monopole and established its moment-rotation relationship.

The additional rotational stiffness due to the plate was then accounted for by revising the equation of the hybrid system. It was observed that the first part of the hybrid foundation moment-rotation curve reduces to that of the monopile response curve. The developed equation makes it possible to predict the safe combination of forces in a hybrid foundation system as a function of the plate width B by plate factors, i.e.

$$\frac{M}{DL^3\gamma'} = -\frac{a}{B} + 0.33K\left(\alpha + 1 \pm 2\left(\frac{1}{2} + \alpha + \frac{1}{K}\frac{cBH}{L^2D\gamma'}\right)^{\frac{3}{2}}\right) \quad [10]$$

For the case of hubrid foundation system with plate width, B = 16 m it was found that a=5 and c =0.0136.

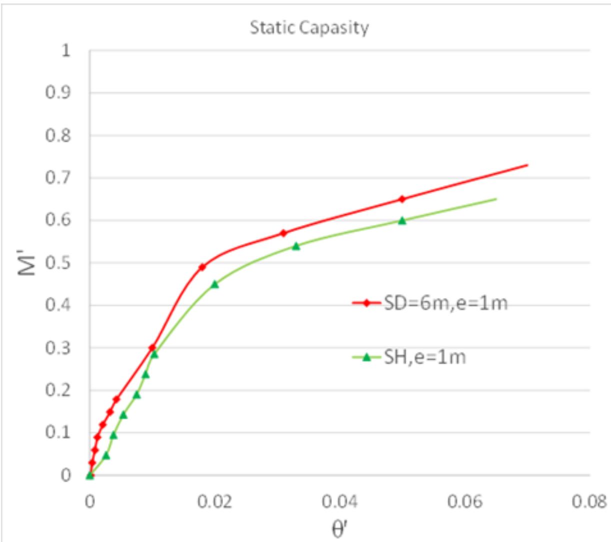

Figure 4. Static moment-rotation response of different systems with e = 1 m.

The developed equation is plotted in Figure 5, which presents the relationship between them moment and horizontal load resistance. It can be noted from Figure 5 that the addition of the plate in the hybrid system increased both the moment and lateral load resistance significantly.

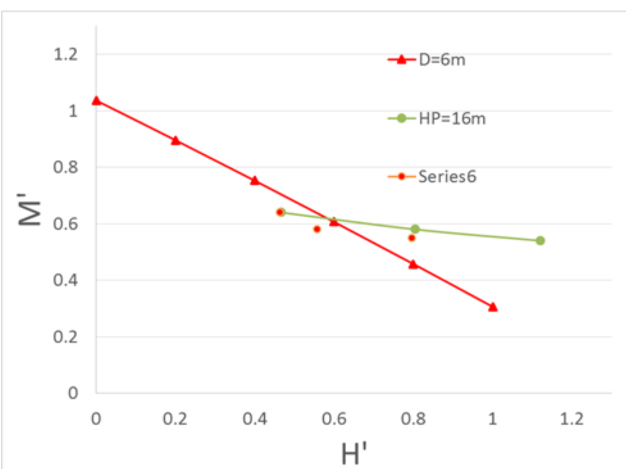

Figure 5. Interaction diagram between moment and horizontal forces

REFERENCES

Allotey, N. and El Naggar, M.H. 2008a. Generalized Dynamic Winkler model for nonlinear soil-structure interaction analysis, *Canadian Geotechnical Journal*, Vol. 45, No. 4, pp. 560-573.

Allotey, N. and El Naggar, M.H. 2008b. A numerical study into lateral cyclic nonlinear soil-pile response, *Canadian Geotechnical Journal*, Vol. 45 (9), pp. 1268-1281.

Bhushan K., and Haley, S. C. 1980.Development of Computer Program Using P-Y Data from Load Test Results for Lateral Load Design of Drilled Piers. a research report prepared for *Woodward-Clyde Consultants Professional Development Committee*, San Francisco, California.

Bhushan, K., and Askari, S. 1984. Lateral Load Tests on Drilled Pier Foundations for Solar Plant Heliostats, Laterally Loaded Piles, ASTM STP 835, James A. Langer, Ed., *American Society of Testing and Materials*, pp. 141-155.

Bhushan, K., Lee, L. J., and Grime, D. B. 1981. Lateral Load Tests on Drilled Piers in Sand, Proceedings of a Session on Drilled Piers and Caissons, sponsored by *the Geotechnical Engineering Division at the ASCE National Convention*, St. Louis, Missouri, pp. 131-143.

Bhushan, K., Haley, S. C., and Fong, P. T. 1979. Lateral Load Tests on Drilled Piers in Stiff Clays, *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 105, No. GT8, Proc. Paper 14789, pp. 969-985.

Budhu, M., and Davies, T. G. 1987. Nonlinear analysis of laterally loaded piles in cohesionless soils. *Canadian Geotechnical Journal*, 24: 289-296.

L.C. Reese, R.C. Welch, 1975. Lateral loading of deep foundations in stiff clay, *Journal of Geotechnical Engineering,* ASCE, Vol. 101(7) 633-649.

Aristonous, M., Trochanis, J.B., and Paul, C. 1991. Three Dimensional Nonlinear Study of Piles, *Journal of Geotechnical Engineering*, Vol.117, No. 3, pp 429-447.

Bentley, K.J. and El Naggar, M.H. 2000. Numerical analysis of kinematic response of piles. *Canadian Geotechnical Journal*, Vol. 37, No. 6, pp. 1368-1382.

Maheshwari, B.K., Truman, K.Z., El Naggar, M.H. and Gould, P.L. 2004. 3D nonlinear analysis for seismic soil-pile-structure interaction. *Soil Dynamics and Earthquake Engineering*, vol. 24, No. 4, pp. 343-356.

Altaee, A., & Fellenius, B. H. 1994. Physical modelling in sand. Can. Geotech. J.

Byrne, B.W. and Houlsby, G.T. 2003. Foundations for Offshore Wind Turbines, *Philosophical Transactions of the Royal Society of London*, A(361), 2909-2930.

DNV-OS-J101, Offshore Standard, 2011, Design of Offshore Wind Turbine Structures, Electronic Version available at http://www.dnv.com/ (On Jan. 25, 2013)

El-Marassi, M., Newson, T., El Naggar, M.H. and Stone, K. 2008. Numerical modelling of the performance of a hybrid monopiled-footing foundation. Proceedings of 61st *Canadian Geotechnical Conference*, Edmonton, pp. 97-104.

Soliman, A., Nehdi, M. 2010. Effect of drying conditions on autogenous shrinkage in ulta-high performance concrete at early-age. *Material and Structures* (2011) 44/;879-899.

Reese, L. & Matlock, H. 1956. Nondimensional solutions for laterally loaded piles with soil modulus assumed proportional to depth. Proc. *8th Texas Conf. on Soil Mech. and Found*. Engng, Austin, TX.

Reese, L. C., Cox, W. R., and Koop, F. D.,1974. Analysis of Laterally Loaded Piles in Sand, Paper No. OTC 2080, Proceedings, *6th Annual Offshore Technology Conference*, Houston, Texas, Vol. 2, pp.473-483.

Robertson, P. K., Davies, M. P. and Campanella, R. G., 1989. Design of laterally loaded driven piles using the flat dilatometer. ASTM *Geotechnical Testing Journal*. Vol. 12, No. 1, pp. 30-38.

McClelland, B. & Focht, J. 1958. Soil modulus for laterally loaded piles. Trans. ASCE 123, 1049–1086.

Poulos, H. & Hull, T. 1989. The role of analytical geomechanics in foundation engineering. *In Foundation engineering: Current principles and practices*, Vol. 2, pp. 1578–1606. Reston, VA:ASCE.