Field measurements on monopile Dolphins

A. Sadeghi-Hokmabadi
Post-Graduate researcher, School of Civil Engineering, University of Tehran, Iran
A. Fakher
Associate Professor, School of Civil Engineering, University of Tehran, Iran

ABSTRACT: A monopile dolphin includes a single pile which is used for berthing and anchoring of large vessels in offshore or onshore terminals. Several methods have been developed to analyze piles under lateral loading. One of the most effective methods is the Strain Wedge Model (SWM) which has a number of advantages in comparison with traditional *p-y* curves. In the Pars Special Economic Energy Zone (Asalouyeh) in the south of Iran, a number of single piles as dolphins were constructed and some full-scale lateral loading tests were conducted on them under the supervision of the second author. In the present paper, a program called Lateral Analysis of Piles (LAP), which has been developed by the authors, is used to examine the Strain Wedge Model for pile analysis using the results of the mentioned full-scale loading tests. The research shows that the SWM calculates a greater pile head displacement than the test data, and illustrates the need for local calibration.

1 INTRODUCTION

In general, there are two types of berth structure; quay and jetty. A quay (or wharf) is a landing place parallel to a navigable waterway that provides access to ships and boats (Figure 1.a). Because of its high lateral resistance, the fenders must be well-designed to absorb the berthing energy of a ship. A jetty (or pier) extends out into the water from the shore. It is in the perpendicular direction to the shoreline serving as a landing place and where loading equipment allows the use of a lighter structure. Ships can berth directly at the structure, but usually require separate structures, such as dolphins, to absorb the high energy of the ship (Figure 1.b).

In some cases, dolphins consist of a number of piles. This type has low lateral deformation and, therefore, a reduced ability to absorb energy. A monopile comprises a single large-diameter pile which is embedded in the soil and behaves as a console. The ability of monopiles to absorb a high amount of energy, their low cost, and simple construction method has made them common alternatives for offshore structures such as wind turbines and mooring or berthing dolphins (Quinn 1972).

In the analysis of monopiles, lateral behavior is important and the interaction between the pile and soil should be modeled accurately. A number of researchers have investigated laterally loaded pile behavior, providing a number of different approaches. These methods can be classified in to the following categories:

(a) Continuum-base approaches;
(b) Load-transfer (or subgrade reaction) approaches.

In the first category, the soil has been modeled as a continuum media, requiring several soil properties inputs for analysis (Fleming et al. 1992). The complexity and unavailability of soil properties of this first approach make it less attractive. The load-transfer approach is more commonly used and was selected for this study.

The load-transfer method models the pile as an elastic member and the soil as series of nonlinear springs (*p-y* curves). The nonlinear soil springs describe the local variation of lateral soil–pile interacting resistance with lateral displacement. Traditional $p - y$ models were initially developed by Matlock (1970) and Reese et al. (1974). Later, a number of $p - y$ curves were developed by different researchers (like Murchinson & O'Neill (1984) & Scott (1980)).

Traditional $p - y$ curves do not consider pile properties such as pile bending stiffness, pile cross-sectional shape, pile head restraint, and pile installation method (Ashour et al. 2004). SWM is an advanced method in comparison with traditional *p–y* curves. It can consider three-dimensional behavior of soil, the effect of piles dimension and shape, and the piles head conditions. However, SWM, as like as traditional *p-y* curves, is a semi-empirical method. In the other words, the main drawback to these approaches is that they are based on empirical parameters (i.e. the modulus of subgrade reaction) which can only be back figured from the results of pile load tests (Basile 2003). The aim of this study is to assess the accuracy of SWM by using the results of some full-scale tests in the Pars special economic energy zone area (Asalouyeh) in Iran.

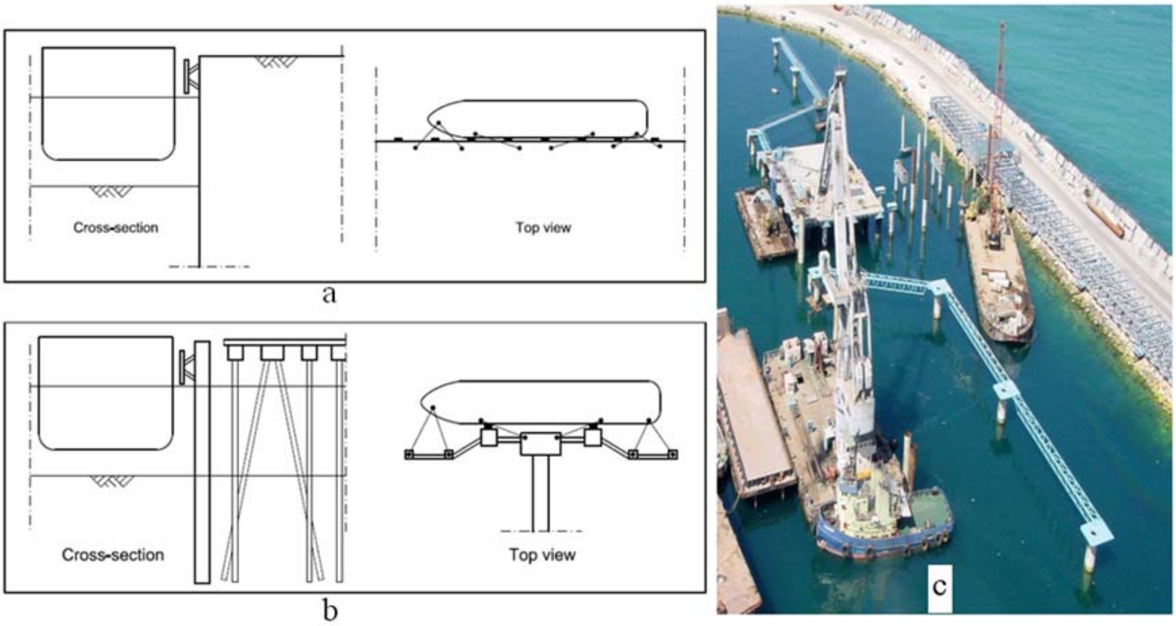


Figure 1. a) Schematic picture of a quay, b) Schematic picture of a Jetty with two berthing dolphins in middle and four mooring dolphins in sides, c) construction of Jetty with its Monopiles in Asalouyeh.

In the present paper, at first the characteristics of SWM are briefly discussed. Details and the results of the undertaken full-scale tests are shown and specs of developed computer program (LAP) has been describes. Later on, the tested monopiles are analyzed with LAP and the results are compared with the tests' data and general conclusions are made.

2 STRAIN WEDGE MODEL

The Strain Wedge Model (SWM) is an approach that has been developed to predict the response of a flexible pile under lateral loading (Norris 1986). In the Strain Wedge Model (SWM), the soil resistance against the lateral loading is determined by the three-dimensional passive wedge of soil that develops in front of the pile (Figure 2).

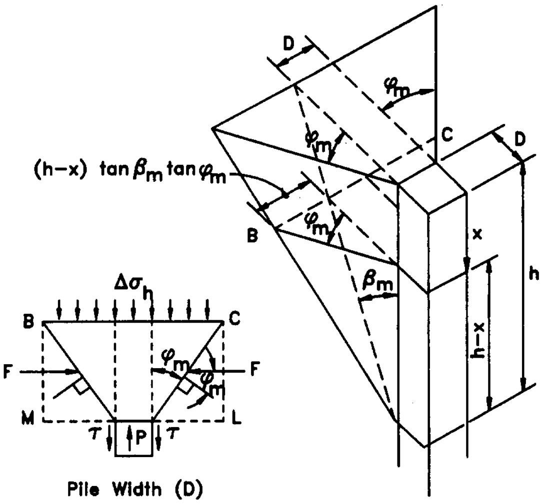


Figure 2. Basic Strain Wedge in Uniform Soil (Ashour et al. 1998)

As shown in Figure 2, this passive wedge is characterized by base angles, θ_m and β_m, the current passive wedge depth h, and the spread of the fan angle, φ_m (the mobilized friction angle). The horizontal stress changes at the passive wedge face, $\Delta\sigma_h$, and the side shear τ, act.

Indeed, SWM allows the assessment of the nonlinear p-y curve response of a laterally loaded pile based on the envisioned relationship between the three-dimensional responses of a flexible pile in the soil to its one-dimensional beam on elastic foundation parameters (Ashour et al. 1998) as in Figure 3.

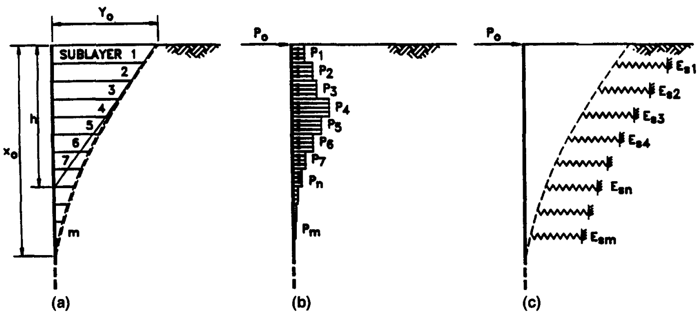


Figure 3. Soil-Pile interaction in Multisublayer Strain Wedge Model (Ashour et al. 1998)

The main objective behind the development of the SWM is to solve the beam on elastic foundation (BEF) problem of a laterally loaded pile based on the envisioned soil-pile interaction and its dependence on both soil and pile properties. Compared to other approaches, the SWM depends on well known on accepted principles of soil mechanics (the stress-strain-strength relationship) and an effective stress soil analysis. For more information about SWM refer to Ashour et al. (1998 & 2004). This method is used in the present research to analyze the full-scale tested monopiles.

3 UNDERTAKEN FULL-SCALE TESTS ON MONOPILES

Experimental researches conducted on the behavior of laterally loaded piles could be divided into two basic types, namely full-scale and small-scale or model testing. Full-scale tests are generally believed to provide the most accurate results, but they are rare because of the large costs required and difficulties involved. Therefore, the results of full scale tests are valuable. In the presented research, a number of full scale tests were performed on monopile dolphins.

3.1 *Tests location*

Asalouyeh is located in southern Iran on the Persian Gulf. It is 300 km east of the city of Bushehr on the coast of Iran. Pars Petrochemical Port in Asalouyeh has 15 berths. At piers 5 and 15, monopiles are used as berthing and anchoring dolphins (Fig. 1.c).

Four monopiles were tested. Monopiles No. 1 and 2 are the inner and outer piles of Berth 15 at a water depth of 14 m. Monopiles No. 3 and 4 are the inner and outer piles of Berth 5 at a water depth of 26 m. The final elevation of the monopile heads after installation was 5 m above mean sea level. These monopiles have a cylindrical shape and were made from three types of steel. The thickness and types of steel used are variable in depth and are shown in Table 1. Details of the monopiles are shown in Figure 4.

Table 1. Details of monopile sections.

Monopile No. 1

Section	Type of steel	Outer diameter (m)	Thickness (mm)	Yielding stress (kN/m^2)
1	ST52	1.778	25.40	360000
2	ST60	1.778	25.40	420000
3	ST70	1.778	28.58	490000
4	ST70	1.778	31.75	490000
5	ST70	1.778	34.93	490000

Monopile No. 2

Section	Type of steel	Outer diameter (m)	Thickness (mm)	Yielding stress (kN/m^2)
1	ST52	1.905	25.40	360000
2	ST52	1.905	28.58	360000
3	ST60	1.905	28.58	420000
4	ST70	1.905	34.93	490000
5	ST70	1.905	41.28	490000
6	ST70	1.905	44.45	490000

Monopile No. 3

Section	Type of steel	Outer diameter (m)	Thickness (mm)	Yielding stress (kN/m^2)
1	ST52	1.778	25.40	360000
2	ST60	1.778	25.40	420000
3	ST70	1.778	28.58	490000
4	ST70	1.778	31.75	490000
5	ST70	1.778	34.93	490000
6	ST60	1.778	34.93	420000

Monopile No. 4

Section	Type of steel	Outer diameter (m)	Thickness (mm)	Yielding stress (kN/m^2)
1	ST52	1.905	25.40	360000
2	ST60	1.905	25.40	420000
3	ST70	1.905	28.58	490000
4	ST70	1.905	31.75	490000
5	ST70	1.905	34.93	490000
6	ST70	1.905	44.45	490000
7	ST70	1.905	41.28	490000

The soil parameters in the field were obtained for each layer using borings. Because of the high soil stiffness, it was not possible to perform in-situ tests such as the standard penetration test. The geotechnical properties of the soil are shown in Table 2. These parameters were obtained by describing the disturbed samples and laboratory tests. For instance the internal friction angle is determined from laboratory shear box. Table 2 presents the drained density (γd), wet density (γt), estimated value of standard penetration test (*Nspt*), effective cohesion (C'), internal friction angle in degrees (φ') and undrained cohesion (Cu).

Table 2. Geotechnical properties of the soil in the field.

Layer description	Sand	Sand and gravel	Sand stone
Depth (m)	0.0-8.0	8.0-21.0	21.0-30.0
classification	SP	GP	---
γ_d (ton/m^3)	1.7	1.95	1.8
γ_t (ton/m^3)	2	2.1	2.1
N_{SPT}	>50	>50	---
C' (ton/m^2)	0	0	---
φ ' (°)	38	40	---
C_u (ton/m^2)	0	0	---

3.2 *Tests method and results*

A heavy duty tension system was designed and constructed that uses a hydraulic jack to provide force and a cable to transfer tension force from one monopile to another. The testing followed ASTM D3966-81, item 24 (ASTM 1995).

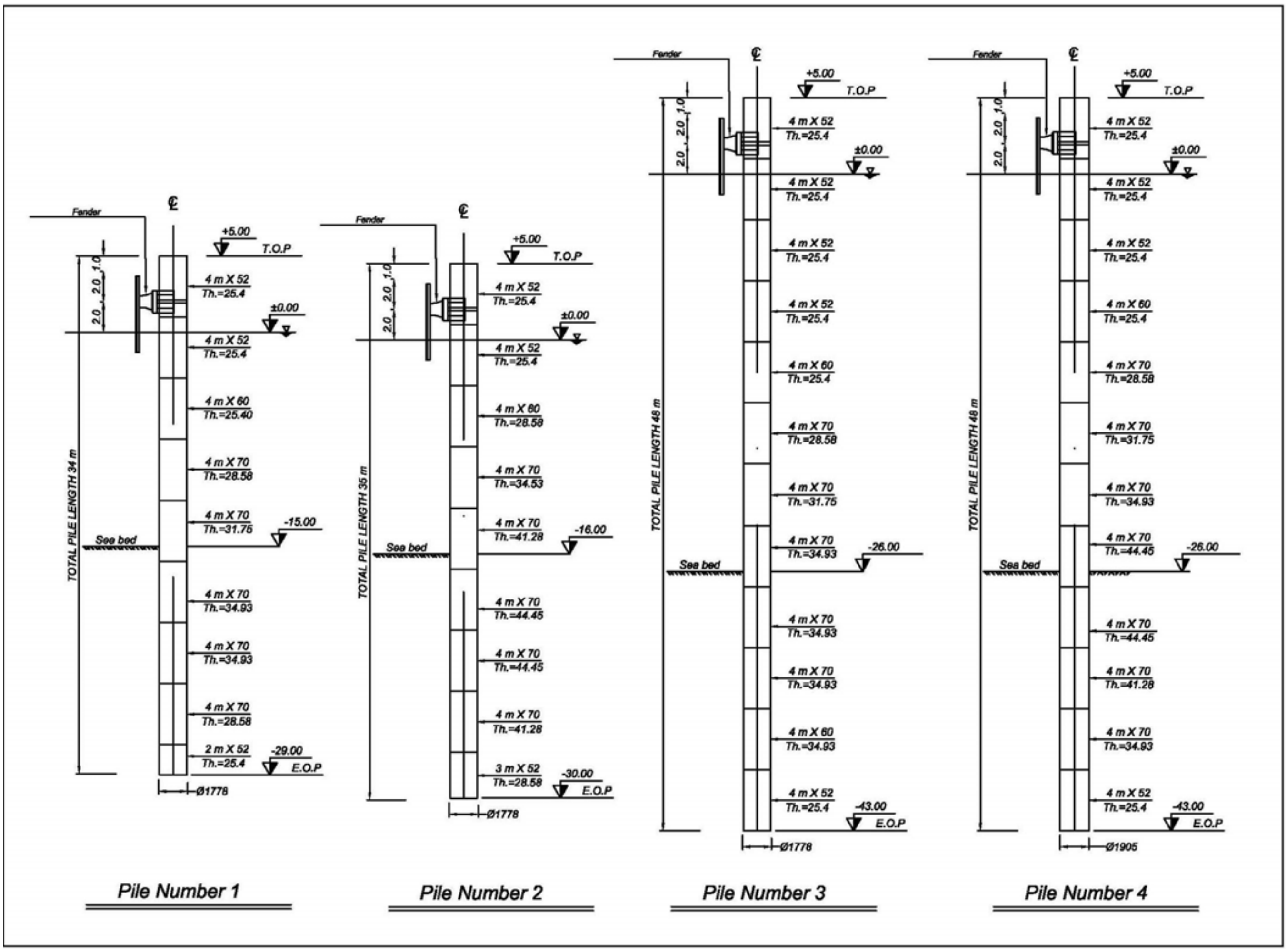


Figure 4. Details of monopiles in Asalouyeh

The tension system sat on one monopile and pulled the other one. Bolts placed in the head of the monopiles for a quick release system were used for the temporary installation of the tension system on one monopile and a pulley on the other. Cables were installed between the tension system on one monopile and support on the other with a 56 in diameter pipe between them to support the weight of the cables and avoid any initial force from them. This pipe is allowed to have axial displacement. Analysis shows that the maximum friction between the cable and pipe was less than 3% of the applied load and may be disregarded. Also, since the spacing between the piles (21.5 m) is more than eight times the diameter of the piles, there is no pile group effect (Fleming et al. 1992).

Four monopiles were tested under lateral static loading. Monopiles No. 1 and 2 were loaded in five steps. Monopiles No. 3 and 4 were loaded in three steps to accommodate the displacement limitation of the jacking system. At each step, the displacement of each pile was measured using Total Station. The loading steps increased and, for each step, the load was applied for 15 min for small loads and 30 min for large loads. Figure 5 shows the results. As it mentioned, the loads are applied at the head of the monopile dolphins and the displacement is measured in their head as well.

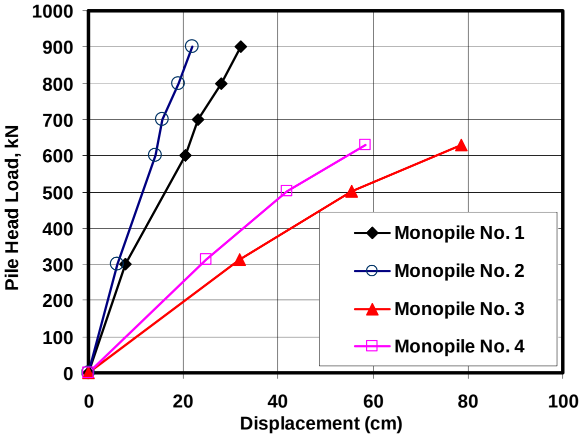


Figure 5. Results of lateral loading tests on monopile dolphins

4 LATERAL ANALYSIS OF PILES (LAP) PROGRAM

A program was developed to analyze the monopiles. The Lateral Analysis of Piles (LAP) program was written in FORTRAN programming language to solve the governing equation for a beam on an elastic foundation (Equation 1) by Hetenyi (1946),

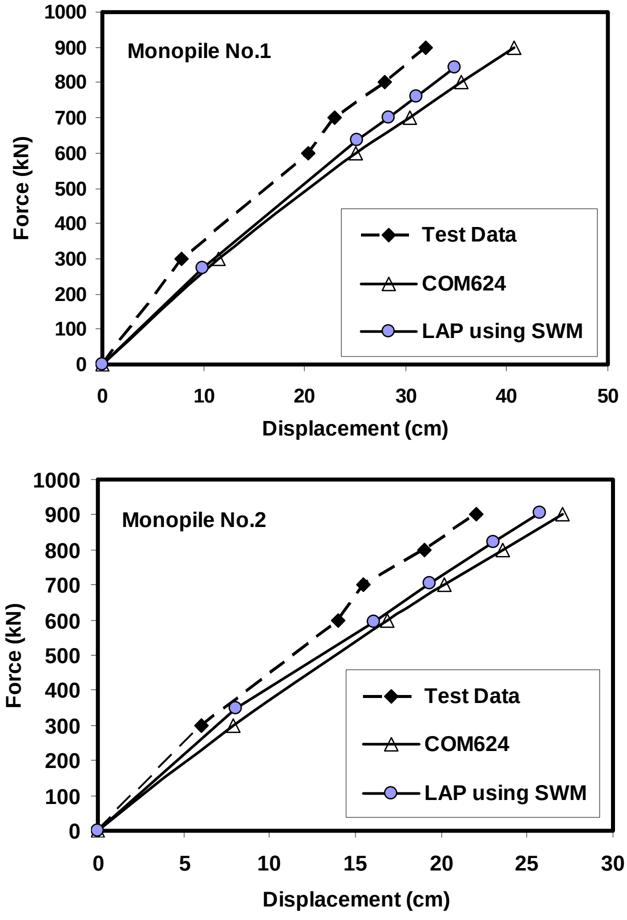
$$EI\left(\frac{d^4y}{dx^4}\right)+P_x\left(\frac{d^2y}{dx^2}\right)+E_s(y)=0 \qquad (1)$$

Where *EI* = bending stiffness of the pile; *Px* = axial load on the pile; *y* = lateral deflection of the pile at point *x* along the length of the pile; and *Es* = soil subgrade reaction (spring stiffness). LAP uses the finite difference method proposed by Matlock and Reese (1961) to solve Equation 1. It considers four sets of boundary conditions at the top of the pile, such as free-head or fixed-head pile. Also, LAP can use different types of spring stiffness (Sadeghi-Hokmabadi et al. 2009) like linear springs, Non-linear *p-y* curves, and SWM.

In addition, LAP can assess pile group behavior under lateral and dynamic lateral loading such as earthquake loads (Sadeghi-Hokmabadi 2009). In the present research, it was used as a means to analyze the dolphins at Asalouyeh and examine the accuracy of SWM.

5 COMPARISON AND DISCUSION

In the present research, LAP is used to analyze the tested monopiles using SWM. In addition, the mentioned monopiles are analyzed using COM624 (Resse & Sullivan 1980) as well. COM624 is a program for analyzing single piles under lateral loads and uses the *p-y* curves suggested by Reese et al. (1974). Figure 6 presents the results of analyses for the four monopiles in the term of head displacement versus lateral load at the head of monopiles.



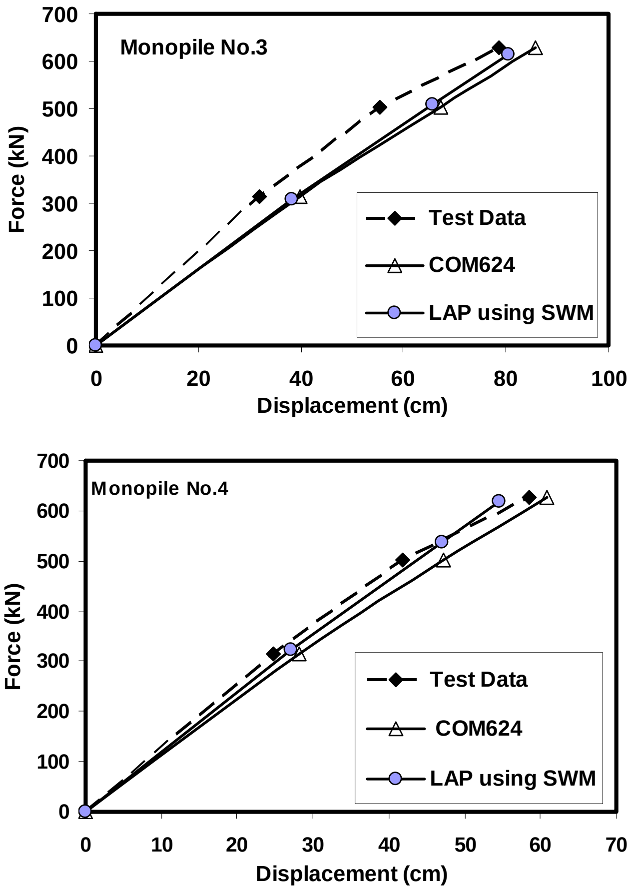


Figure 6. Pile head horizontal displacement versus lateral load according to test data and LAP analysis for monopiles No.1, 2, 3, and 4.

As Figure 6 shows, both the SWM and COM624 calculate a greater pile head deflection than the measured data. In comparison, the SWM gives closer answers with measurements undertaken in the presented case study than COM624.

The SWM receives force at the pile head as input and gives the pile head displacement as output (Ashour 1998). This method calculates *p-y* curves during the computation for each case. In other words, the SWM does not use pre-defined *p-y* curves like the traditional *p-y* method (Fakher et al. 2009), and it is not possible to define a certain modification factor for this method like *p-y* method.

The average of ratio between the calculated pile head displacement and the observed one for monopiles number 1 to 4 is calculated as 0.82, 0.85, 0.87, and 0.98 respectively. Also, the total average of this ratio for these 4 set of monopiles is 0.88. It means that the data of performed tests are 12 percent less than predicted pile head displacement using the SWM.

The real behaviour of pile head displacement is non-linear. The *p-y* curves and SWM have difference with real situation in the flexure of pile head-displacement curves. Indeed, the total lateral stiffness in the real situation declines sooner, but in these methods it decline later and have approximately linear behaviour in the tests loads. The difference between proposed *p-y* curves and SWM with the real situation is occurred because of the development of plastic region near the soil surface. In fact, in the real situation under the testing loads the near surface soil has a plastic manner and yields, but *p-y* curves and SWM do not show this behaviour under the tests loads level. It should be noted that the total behaviour of pile-spring system is very sensitive for the near surface soils, and these soils should be modeled carefully (Fakher et al. 2009).

6 CONCLUSION

The results of full-scale tests on large diameter piles showed that the monopile dolphins behave like long piles.

The LAP program was developed to analyze piles under lateral loading. This program has the ability to consider different boundary conditions and types of spring stiffness like *p-y* curves and Strain Wedge Model.

According to the results of full-scale in-situ test, the accuracy of Strain Wedge Model has been investigated.

In granular marine soils, the traditional *p*-y curves and SWM calculate the pile head displacement as being greater than the test data from the present study. This means that real piles withstood large amounts of force for the specified displacements. Thus, using these curves without calibration leads to overestimating the piles displacements and demonstrates the need for local calibration.

In comparison, the SWM gives the closest answers to the measurements undertaken in the presented case study than COM624.

The shape of the pile head displacement under real conditions declines sooner than in the calculated results because the analytical models do not show the soil plasticity near ground depth. To modify, the ultimate resistance of the non-linear springs should be decreased and the primary stiffness should be increased.

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