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RECENT ADVANCES IN P-Y FORMULATIONS BASED ON EXPERIMENTAL RESEARCH WORK

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ABSTRACT

Pile foundations, in particular drilled shafts, are one of the most common foundation systems used in engineering practice. Using experimental test results from large-scale pile testing programs has become an evolving design practice to create safer and more economical foundation systems. Large-scale pile tests also help understand foundation behavior in specific boundary conditions, soil types and enable engineers to conduct a performance based design approach. Among several analytical design tools such as Limit Equilibrium Methods, Winkler Method, Elasticity Methods, *p-y* formulations, Hybrid Methods, Closed Form Solutions and FEM analyses, the *p-y* method has become a popular tool to design and analyze pile foundations subjected to lateral loading. Many advances have been made since the American Petroleum Institute (API) sponsored a pioneering large-scale research program in the 1960s to mid-70s. However, due to lack of knowledge and familiarity with recent advances, many engineers rely on traditional results and obtain pile designs that are over conservative in terms of ultimate pile resistance, misrepresent the initial stiffness under small lateral loading, and consequently cause uneconomical pile geometries. This paper will focus on recent developments on *p-y* curves derived from large-scale pile experiments conducted in the last decades. In particular, it will present a brief summary of studies that target effects like (1) effect of pile geometries (e.g. diameters& length), (2) effects of head fixities, (3) effects of cyclic loading & strength degradation, and finally, (4) effects of group configurations. In addition to the research review, a recently developed database of large-scale lateral pile tests will be introduced. The database is a uniquely designed resource that will help practicing geotechnical engineers to guide efficient and economical pile design for piles under lateral loading.

INTRODUCTION

Deep foundations are used extensively to resist lateral loads from wind, waves, vessel impact, earthquakes, inclined loads or earth pressures. The analysis of displacements and stability requires a good understanding and prediction of the highly non-linear soil-structure interaction response; however, predicting the lateral response of deep foundations accurately is not a straightforward task.

Load tests have been regarded as an effective tool in verifying the reliability of deep foundations. More and more design approaches rely on knowledge gained through a combination of experimental pile studies, finite element models, and load-displacement simulations to iterate to the most suitable and economical foundation design. This new approach, known as performance- based design (PBD), focuses on the actual foundation behavior under various loading scenarios (e.g., static, cyclic and dynamic, consideration of site effects, etc.), and uses state of the art test data in addition to knowledge gained from previous large-scale experiments and code documents.

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Accurate and reliable analysis of the entire structure-foundation-soil system predominantly depends on the prediction of soil resistance surrounding piles. Existing analytical design methods for predicting soil resistance can be classified into four categories. The first group includes limit equilibrium models as developed by Blum (1932), Brinch Hansen (1961) and Broms (1964). The pile and soil are assumed to be rigid and rigid, perfectly plastic, respectively. The soil is in a state of plastic equilibrium and no plastic hinge does develop within the pile. The **s**econd group includes all methods that are able to model the stiffness and strength of a pile, such as the characteristic load method (CLM), the non-dimensional load method (NDM), the subgrade reaction theory (Winkler method) and *p-y* curves analysis. The third category consists of elasticity methods (Paulos, 1971&1980) based on Midlin's solution for a horizontal point in a homogenous elastic half-space. The forth group entails the so-called hybrid models, such as Focht &Koch (1973), which combine the nonlinear *p-y* method with the elastic continuum.

Since the above-mentioned methods are based on different assumptions and empirical rules, the final predictions of soil resistance are very different among the same soil conditions and pile type. Due to their limitations in terms of simplicity, numerical efforts, time investment, and accuracy, only few methods are implemented in daily engineering practice. For instance, Broms and CLM are the least utilized given their complex and inaccurate procedure. Blum and Brinch Hansen are ultimate strength models and cannot be used under working loads. The most widely used tool to design and analyze laterally loaded piles in practice is the *p-y* method, given its relatively reliable results and simple derivation process. Limitations and advantages of this method, along with the recommendations, will be presented in this paper.

BACKGROUND:

General comments on the *p-y* method

The *p-y* method analyzes the lateral response of a pile or pile group by modeling the soil via a series of pre-defined, nonlinear springs attached along the vertical depth of a pile. Finite element or finite difference programs can be used to execute such analyses. Hereby, *p* represents the lateral soil pressure per unit length of the pile and *y* is the lateral soil displacement. *P-y* relationships are strongly influenced by external (boundary conditions) and internal (derivation methods) factors. External factors include the properties (type) of soil, characteristics of pile (e.g. diameter, cross-sectional shape, head boundary conditions, construction method), type of loading (static, dynamic, monotonic, cyclic or combinations thereof), group interaction effects, depth below the ground surface and pile-soil interactions (coefficient of friction between foundation and soil). Internal factors include the derivation method, i.e. via sensors instrumentation installed inside the pile or pressure sensors installed at the perimeter of the pile or the utilization or the accurate structural pile model selected during the derivation process.

Table 1. Classic derivation process of *p-y* curve

Deflection [y]	**Slope [S]**	**Curvature [ϕ]**	**Moment [M]**	**Shear [V]**	**Soil Reaction [p]**
2nd integral	1st integral			1st derivative	2nd derivative
$y(z)$	$S = \frac{dy}{dz}$	$\phi = \frac{d^2 y}{dz^2}$	$M = EI\frac{dS}{dz} = EI\frac{d^2 y}{dz^2}$	$V = \frac{dM}{dz} = EI\frac{d^3 y}{dz^3}$	$p = \frac{dV}{dz} = EI\frac{d^4 y}{dz^4}$

Table 1 explains the classical derivation process of experimental *p-y* relationships from internal pile curvature readings. After internal curvature measurements are obtained from strain gauges, LVDTs and inclinometers installed inside the pile, rotations and deflections, as well as bending moments, shear forces and lateral soil reactions in and around the pile are calculated following a traditional process of double derivation and double integration of pile curvature data.

Given the significance of the external influencing factors on the lateral pile response, many *p-y* formulations have been empirically derived over the past decade to study the actual in-situ pile behavior. Testing needs arose from project specific design requirements and were complimented through academic research programs as explained later in the paper.

Codes and standards

The earliest recommendations on *p-y* behavior date back to the 1950s and refer to work by Skempton and Terzaghi. Some of the initial experimental studies started in the 60s and 70s, including the work of Broms (1964), Matlock (1970) and Reese and Cox (1975). Full-scale lateral load tests were, and still remain, rare due to the high costs associated with mobilizing an experimental testing program. Therefore, engineers today rely on previous test results and institutional design recommendations in their daily practice. The biggest limitation in using *p-y* formulations provided therein is the reliance on test results that stem from single, small-diameter driven piles or drilled shafts for specific types of soils. New types of pile foundations such as pile groups, micro piles, large-diameter drilled, driven and battered piles, have been developed and need more specific design recommendations than available. Significant progress on the formulation has been done in the 1980s and thereafter. The following paragraphs summarize the most commonly implemented design recommendations.

The American Petroleum Institute (API) was one of the first agencies to develop recommendations based on funded research studies for their own offshore construction purposes. API (1993) suggests specific *p-y* relationships for soft clays, dating back to the 324 mm diameter steel-pipes piles by Matlock (1970), for stiff clays, based on 610 mm diameter steel-pipes pile by Reese and Cox (1975), and cohesionless soils based on the tests by O'Neill and Murchinson (1983). The API *p-y* curves represent pioneering works on lateral pile design, but they have not experienced significant revisions since its establishment 30-40 years ago. Moreover, the API *p-y* curves represent the basis for current practice, even if they were developed using data from lateral load testing of small diameter steel piles.

Federal Highway Administration (FHWA) directly suggests the utilization of software such as *LPILE* (developed by Ensoft), or *FBPIER* (developed by the Bridge Software Institute of Gainesville, FL), for the lateral pile design and relies on relationships provided therein. Additionally, it suggests consulting the FHWA-IP-84-11 Guideline "Handbook of design of piles and drilled shafts under lateral load" by L.C. Reese (1984). LPILE and FBPIER are programs that solve the beam bending equation with difference numerical methods and model the soil using nonlinear lateral load-transfer (*p-y*) curves. Pile-soil systems are characterized by conventional geotechnical data, pile properties and loading conditions. Procedures for constructing *p-y* curves cover various types of soil and water table conditions as well as static or cyclic loading conditions. LPILE and FBPIER provide models for cohesive soils by Matlock (1970), Reese (1975) and Reese & Welch (1972), for cohesionless soils by Reese, Cox and Koop (1974) and for silt, referring to the cemented c-φ procedure. Both programs also allow the input of user-defined *p-y* formulations.

State of the practice: the most popular *p-y* curve formulations

The simplicity and accuracy of the *p-y* method is the principal reason of its popularity. *P-y* curves for a given soil profile can be estimated using the following three step procedure:

1. Obtaining the best possible estimate of the variation of shear strength (τ, c), submerged unit weight (γ) and the value of the strain corresponding to one-half of the maximum principal stress difference (ε_{50}). If no stress-strain curves are available, typical values of ε_{50} are given in Table 2.
2. Computing the ultimate soil resistance per unit length of the pile (P_u).

3. Computing the deflection at one-half of the ultimate soil resistance (y_{50}) or other transaction points in the curve.

Table 2. Typical values for ε_{50} related to the undrained shear strength

c_u [kN/m^2]	ε_{50} [-]
<12	0.02
12-24	0.02
24-48	0.01
48-96	0.006
96-192	0.005
>192	0.004

The most popular models for cohesive (e.g. Matlock, 1970, Reese et al., 1975, and Reese and Welch, 1972) and cohesionless soils (e.g. Reese, Cox and Koop, 1974, O'Neill and Murchinson, 1983) are summarized hereafter.

P-y curves models for clays

Matlock (1970) and Reese et al (1975) suggested the following relationships for *p-y* formulations in clay in the presence of free water:

Table 3. Mathematical description of the *p-y* formulations for clay

Researcher	**Matlock (1970)**	**Reese et al. (1975)**
Soil type	**Soft Clays with free water**	**Stiff clay with free water**
	$p = 0.5p_u(\frac{y}{y_{50}})^{1/3},\ y < 8y_{50}$ $p = p_u,\ \mathrm{y} > 8\mathrm{y}_{50}$ (1) $p_{uz} = \min[(3+\frac{\gamma'}{c_{uz}}+\frac{J}{b}z)c_z b, 9c_{uz}b]$ (2)	$p = (k_{py}z)y$ (4) $p = 0.5p_u(\frac{y}{y_{50}})^{1/2}$ (5) $p = 0.5p_u(\frac{y}{y_{50}})^{1/2} - 0.055p_u(\frac{y - A_s y_{50}}{A_s y_{50}})^{5/4}$ (6) $p = 0.5p_u(6A_s)^{1/2} - 0.411p_u - \frac{0.0625}{y_{50}}p_u(y - 6A_s y_{50})$ (7) $p = p_u[1.225(A_s)^{1/2} - 0.75A_s - 0.411]$ (8) $p_u = \min[(2c_a b + \gamma' bz + 2.83c_a z), 11cb]$ (9)
	$y_{50} = 2.5\varepsilon_{50}b$ (3)	$y_{50} = \varepsilon_{50}b$ (10)

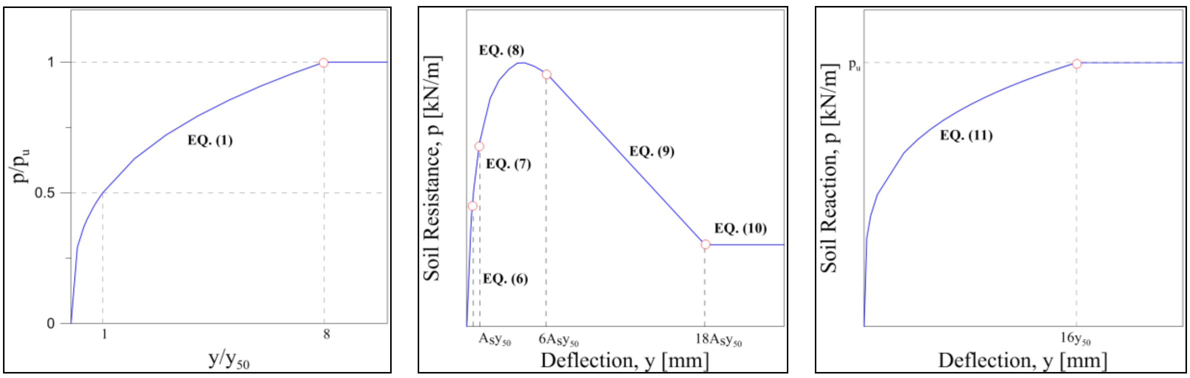

Figure 1. *P-y* curves for clay: (a) Matlock (1970); (b) Reese et al. (1975); (c) Reese and Welch (1972)

Table 3 and Figure 1 explain the segments of the *p-y* curves and how the respective lateral load and displacement can be calculated. All notations are explained at the end of this manuscript.

Reese and Welch (1972) proposed a *p-y* curve formulation to understand the response of stiff clays with no free water. The procedure is the same as Matlock (1970) for soft clay with the presence of free water. The only difference is the relationship between *p* and *y*, which is as follows:

$$p = 0.5 p_u (\frac{y}{y_{50}})^{1/4} \tag{11}$$

The value of p/p_u remains constant beyond $y = 16y_{50}$.

Table 4. K_{py} as function of c_a (Reese et al. 1975) and φ (Reese, Cox and Koop 1974)

	c_a [kPa]				Loose (φ<30°)	Medium (30≤φ<36°)	Dense (φ≥36°)
	50-100	200-300	300-400				
K_{py} (static) [MN/m³]	135	270	540	K_{py} (below water table) [MN/m³]	5.4	16.3	34
K_{py} (static) [MN/m³]	55	110	540	K_{py} (above water table) [MN/m³]	6.8	24.4	61

P-y curves for sands

Reese, Cox and Koop (1974) and O'Neill and Murchinson (1983) proposed *p-y* formulations for the response of sand above and below the water table. The procedures are for short-term static loading and for cyclic loading. The following *p-y* curve parameterization has been proposed. Figure 2 shows the curve segments associated with the equations below.

Table 5. Mathematical description of the *p-y* formulations for sand

Researcher	**Reese, Cox and Koop (1974)**	**O'Neill and Murchison (1983)**
Soil type	**Sand**	**Sand**
	Ultimate soil resistance: $p_{u1} = \gamma z[\frac{K_0 z \tan\varphi \sin\beta}{\tan(\beta-\varphi)\cos\alpha_s} + \frac{\tan\beta}{\tan(\beta-\varphi)}(b + z\tan\beta\tan\alpha_s)$ $+ K_0 z \tan\beta(\tan\varphi\sin\beta - \tan\alpha_s) - K_a b]$ $p_{u2} = K_a b\gamma z \tan^8(\beta - 1) + K_0 b\gamma z \tan\varphi \tan^4\beta$ (12) $p = K_{py} zy$ (13) $p = Cy^{1/n}$ (14) With: $m = \frac{p_u - p_m}{y_u - y_m}$; $n = \frac{p_m}{m y_m}$; $C = \frac{p_m}{y_m^{1/n}}$ (15) $p_{ms} = B_s p_u$ (16) $p_{us} = A_s p_u$ (17) The value of p remains constant after $y = \frac{3b}{80}$. With K_{py}, depending on f (Table 3). A_s and B_s are dimensionless parameters used in static loading and function of the depth and pile diameter.	$p = A p_u \tanh(\frac{k_{py\max} z}{A p_u} y)$ (18) $p_u = \min[(C_1 z + C_2 b)\gamma z, C_3 c\gamma z]$ (19) With C_1, C_2, C_3= dimensionless coefficients related to φ; k_{py} is determined as function of φ.

RECOMMENDATIONS FOR *P-Y* CURVES WITH COHESION AND FRICTION ANGLE

There are no available recommendations on developing *p-y* curves for soil with cohesion and a friction angle that are generally accepted. The following procedure for developing *p-y* curves is for short-term static loading and cyclic loading (Evans and Duncan, 1982). The ultimate soil resistance is given by:

$$p_u = A_s p_{u\varphi} + p_{uc} \tag{20}$$

With $p_{u\varphi}$ is the smallest of (12), whereas p_{uc} is the smallest of (1). The dimensionless parameter, A_s, is from Reese, Koop and Cox (1974). This *p-y* curve is composed by an initial straight line (21), a parabolic portion (14) and two straight lines.

$$p = K_{py} zy \tag{21}$$

The value of p remains constant beyond $y_u = \frac{3b}{80}$. $K_{py} = K_c + K_\varphi$, where K_c and K_φ are respectively functions of c_{uz} and φ.

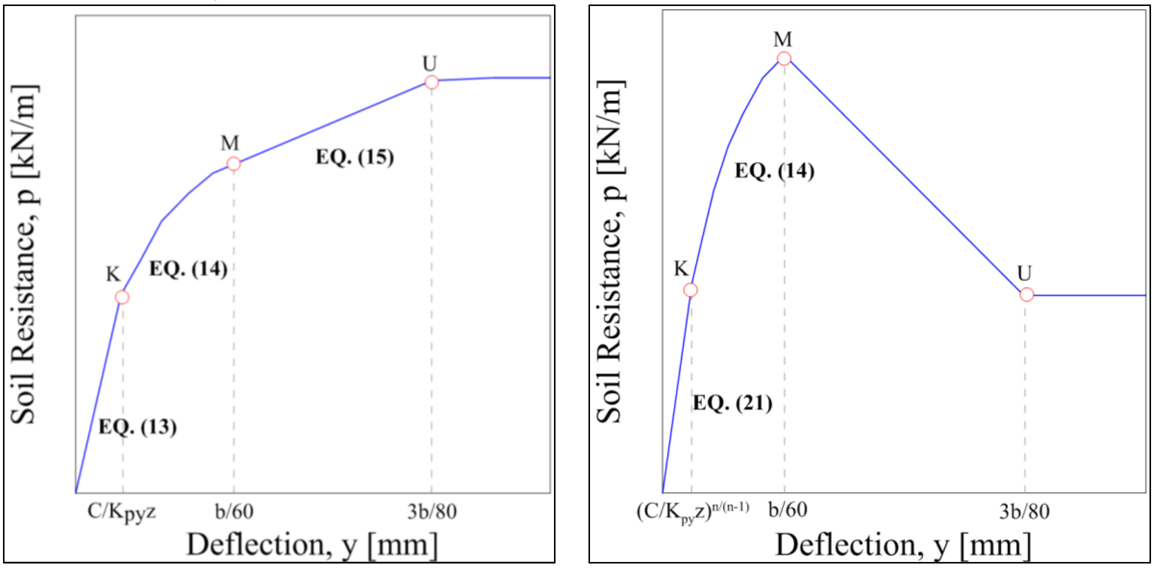

Figure 2. (a) Reese, Koop and Cox's *p-y* curve for sands; (b) c-φ *p-y* curve model

RECENT ADVANCEMENTS OF *P-Y* CURVES:

In order to identify the current state of knowledge in *p-y* design and to recognize recent advances with respect to *p-y* formulations, an extensive literature review of experimental pile testing programs in non-liquefiable soils over the last 40 years was conducted. The predominant soils are sands (43%) and clays (38%), followed by silts, gravels and cemented materials, for which much less information is available. Several research programs have addressed this issue and conducted studies on silt (Naramore and Feng 1990, Kramer et al. 1991), peat (Kramer et al. 1991), rock (Seychuck 1970, Cho et al. 2001), gravel (Hokmabadi et al. 2012) and loess (Dapp et al. 2011). Additional studies investigating the effects of various deposits and strongly layered soils can be found in Rollins and Sparks (2002), Ng et al. (2001), and Gerber et al. (2008). While studies in silts showed similarities with clays, *p-y* results for rock and gravel led to different results. For instance, Hokmabadi et al. (2012) performed lateral tests on four monopoles in granular marine soil and it has been observed that the API curves for sands resulted in larger pile head displacement in comparison with the field measurements. Cho et al. (2001) observed that back-calculated *p-y* curves had a softer response than those obtained by using Reese et al. (1997).

The following portion of the paper will narrow its object on issues such as the influence of pile dimensions, head fixity, load applications, and group configurations.

Pile Diameter (D)

The majority of pile tests in the literature were conducted on shafts with diameters ranging between 0.3m-0.6m. Several recent studies focused on larger diameter piles (Dunnavant et al. 1989, Naramore and Feng 1990, Ng et al. 2001, Hokmabadi et al. 2006, Janoyan et al. 2012, Khalili-Tehrani et al. 2014) and/or compared geometric differences to assess the impact of increased side friction around the pile perimeter.

Khalili-Tehrani et al. (2014) compared two field studies on RC flagpole (free head) specimens with diameters of 0.6m and 1.8m and showed that the 1.8m diam. flagpole had a soil resistance that was 5 times greater than the smaller pile. The ultimate soil resistance increases with pile diameter because the larger pile ultimately mobilizes more soil, which results in higher soil resistance.

Compared to recommendations by API (1993), Janoyan (2006) found the 1.8m diameter pile to have a 60% larger capacity than the recommended API curves. For the smaller diameter pile, API and measured *p-y* curves were closer in comparison with the experimental curves having 20% less capacity.

Juirnarongrit and Ashford (2004) performed four tests on RC free head piles with diameters of 0.40, 0.60, 0.90 and 1.20 m and compared experimentally derived curves with and standard code recommendations. The API *p–y* curves for sands underestimated the measured soil resistance for the small diameter pile, whereas better agreement was found with larger piles. The measured lateral loads were greater by 40% for the 0.4m diam. pile and 11% for the 1.2m diam. pile than the API curves for sand. Test data for both studies are included in the pile database.

Pile head fixity

The rotational restraint at the pile head is an important boundary property that influences the pile deflection and curvature distributions. An increase in rotational restraints (with the extreme case being a complete fixed head pile) leads to a reduction in lateral deflections, a shift in location of plastic hinge development and the introduction of shear deformations in RC piles. Approximately 86% of the tests analyzed in the current literature review studied free-head piles (e.g. Brown et al. 1987, 1988, Kramer et al. 1991, Ruesta and Townsend, 1997, Juirnarongrit and Ashford 2004, Rollins et al. 2005, 2006, Ismael et al. 2010). The remaining 14% of the tests were performed on hinged (e.g. Meimon et al. 1986, Ng et al. 2001) or fixed piles (e.g. Mokwa 1999, Rollins and Sparks 2000, 2002, Huang et al. 2001, Walsh et al. 2005, Lemnitzer et al. 2010).

Khalili Tehrani et al (2014) compared test results from RC fixed and free head pile researched by Stewart et al (2007) and Lemnitzer et al (2010). It was observed that *p-y* curves derived for the flagpole piles are approximately up to1.5 stiffer than *p-y* curves derived for the fixed head shaft. However, the ultimate capacity of the fixed-head *p-y* curves are typically about up to 50% greater than that for the flagpole *p-y* relations at a given lateral displacement at identical pile depths. Experimental curves were also compared with the API formulations for stiff clay, which revealed that the measured fixed-head pile had 100% larger capacity than the recomended API curves. API in turn overestimated the actual initial stiffness at small deflections by 30%.

Effect of cyclic loading and strength degradation

In addition to strong cycling loading through seismic events or impact loading, wind, waves and thermal factors can produce more than 10^4 loading cycles during the typical design life of a pile. While cyclic degradation was early known among foundation researchers (e.g. Sandeman, 1880), Feagin (1937) was the first researcher to document the observed loss of resistance in contractive soils under drained condition. He noticed a slight increase in displacement under repeated lateral load cycles, but a stabilization of displacements after 5 to 25 cycles. Matlock (1970, 1980), Lee and Gilbert (1979), Alizadeh (1969) and Robertson et al (1984) tested piles in contractive soils in undrained conditions and noticed displacement increases of 10% between the first and repeated cycles at low load levels. For large load levels, displacement increases of up to 100% were observed, after which they tended to stabilize. Turner et al. (1987) summarized more than 82 load test case histories in an EPRI EL 5375 report that compares deep foundations for electrical transmission line structures subjected to static and repeated loading. Trends in Drilled shafts were found to be very similar compared to driven pile studies mentioned above.

Cyclic loading along with the draining condition has a different effect on cohesionless and cohesive soils. Brown et al (1998) tested steel pipes in sand and noticed load differenced of 4% between the first and the 100^{th} cycle. Rollins et al. (2006) detected that the peak load was reduced by about 15% after only 15 cycles for a pile group in clay. Gaps play a significant role in reducing the resistance for small deflections. During the first cycle of loading, the shallower soils quickly mobilize their full strength in response to pile displacement due to their relatively low ultimate resistance, whereas deeper soils usually have a greater ultimate resistance and smaller pile displacement. Other experimental studies that developed cyclic *p-y* curves were performed by Little and Briaud (1988) with a typical number of 20 lateral loading cycles, and those by Long and Vanneste (1994), whose experimental study included up to 500 cycles in some tests.

Group configurations

Piles are most frequently used in a closely spaced group to resist higher lateral loads in structures such as highway bridges, dams, waterfront facilities and offshore constructions.

Piles in closely spaced groups behave differently than single isolated piles because of pile-soil-pile interactions that take place in the group. The unequal distribution of load among piles within a group is caused by the so-called "*shadowing effects*", which describe the overlap of shear zones and consequent reduction of soil resistance. Similarly, deflections observed in a single pile are less than deflections measured for the same pile located in a group due to the loss of soil resistance. Model tests and full-scale tests indicate that piles are not influenced by group effects when center-to-center pile spacing exceeds 6 pile diameters (6D) in a direction parallel to the load and when they exceed 3D measured in a direction perpendicular to the load (Mokwa 1999).

The resistance in the group piles is not uniform. Many researchers (e.g. Holloway et al. 1981 and Brown et al. 1988) observed that piles in trailing rows of pile groups have significantly less resistance to a lateral load than piles in the lead row, and therefore exhibit greater deflections.

A standard for quantifying group interaction effects is the "*p-multiplier*", an empirical reduction factor for *p-y* curves, suggested by Brown et al. (1988). This method accounts for the loss of soil resistance, due to both the shadowing effect, and the non-uniform distribution of the resistance within the group. Different values of p-multipliers are assigned to each row of the group.

Literature has shown the following group interactions:

1. **Type of soil**: Since sand generally has a higher φ than clay, the passive wedge would be wider in sands than in clay (Rollins et al., 2005). As a result, more group interaction would be expected from adjacent piles in sands than in clays.
2. **Pile Dimensions**: Mostafa et al. (2002) showed that piles with larger diameter would have less pile-soil interactions and greater p-multipliers. The same effect was obtained with an increase in pile length. As the pile length increases, the relative contribution of the soil layers along the pile are more homogenous and the pile-soil interactions along the shaft length decreases.
3. **Installation process**: The driving process is expected to increase the soil density making the soil stiffer. Huang et al. (2001) reported that bored pile group construction appeared to loosen the soil surrounding the piles, whereas the driven pile construction caused a densifying effect.
4. **Spacing**: Group interaction effects became progressively more important in reducing lateral soil resistance as pile spacing decreased.
5. **Row location**: Literature clearly shows that the location of a pile within the group is important. The first row, or the leading row, carries the greatest load in the group and is associated with the biggest p-multiplier values. The second, the third and the following rows carry progressively smaller loads and the associated p-multipliers are smaller.
6. **Deflections**: Group interactions have been found to vary with the group deflection. The bigger group deflection, the smaller p-multipliers. This means that the group ultimate load capacity becomes progressively smaller than the single pile capacity with the increase of pile head deflection.
7. **Head Fixity**:The importance of head fixity is evident in the comparison of the deflections of a fixed head pile and a flagpole. For example, specimens constructed in concrete (Huang et al. (2001)) and steel (Rollins et al 2005), both on a 3*3 pile group with a 3-D spacing in sand, illustrate the role of fixity. For the fixed headed group, Huang et al. (2001) observed deflections of approximatley 0.02D, whereas Rollins et al. (2005) measured deflections 10 times larger with free headed configuration. The fixed-headed pile group was shown to exhibit significantly smaller reductions in soil reaction (i.e., the *p*-multipliers were larger) than for the free-headed pile group. Further comparisons of the *p*-multipliers are presented in Table 6.

Table 6. Review of p-multipliers in the literature

Tests Reference	Soil Type	# of rows in group	Spacing	Group efficiency factor	P-multipliers by row				
					1	2	3	4	5
Meimon et al. (1986)	Clay	2	3d	0.5 - 0.71	0.9	0.5	-	-	-
Morrison and Reese (1986)	Sand	3	3d	0.58 - 0.9	0.8	0.4	0.3	-	-
Brown et al. (1987)	Clay	3	3d	0.68 - 0.8	0.7	0.5	0.4	-	-
Brown et al. (1988)	Sand	3	3d	0.63 - 0.7	0.8	0.4	0.3	-	-
Ruesta & Townsend (1997)	Sand	4	3d	0.6 - 0.91	0.8	0.7	0.3	0.3	-
Rollins et al. (1998)	Clay	3	3d	0.59 - 0.8	0.6	0.38	0.43	-	-
Mokwa (1999)	Sand	2	4d	-	0.85	0.75	-	-	-
Rollins and Sparks (2000)	Silts & Clays	3	3d	-	0.6	0.38	0.43	-	-
Cho et al. (2001)	Rock	-	-	0.59-0.80	0.6	0.38	0.43	-	-
Huang et al. (2001)	Sand	3	3d	0.92 - 1.02	0.93	0.7	0.74	-	-
Huang et al. (2001)	Sand	4	3d	0.72 - 0.89	0.89	0.61	0.61	0.66	-
Rollins & Sparks (2002)	Silts & Clays	3	3d	-	0.6	0.38	0.43	-	-
Snyder et al. (2004)	Clay	5	3.92d	0.85-0.90	1	0.81	0.59	0.71	0.59
Rollins et al. (2005)	Sand	3	3.3 d	0.72 - 0.935	0.8	0.4	0.4	-	-
Rollins et al. (2006)	Clay	3	5.65d	0.87 - 1.08	0.95	0.88	0.77	-	-
	Clay	4	4.4d	0.75 - 1.0	0.9	0.8	0.69	0.73	-
	Clay	5	3.3d	0.45 - 0.67	0.82	0.61	0.45	0.45	0.51

The group efficiency factor mentioned in Table 6 is another measurement used to quantify the reduction of capacity of a pile located in a group vs. a single pile and utilizes the pile head load deflection relationship as an indicator.

INTRODUCTION TO THE DATABASE: FindAPile.com

The results of an ongoing, extensive literature review have been integrated in www.FindAPile.com, a database developed by the authors of this paper, funded through the Deep Foundation Institute (DFI) and the University of California, Irvine. FindAPile.com provides educational information about the *p-y* curve method, the general behavior of laterally loaded piles and a review of the state of practice in the United States and represents an invaluable reference reservoir for the foundation industry. The core of the website is a database that documents previously conducted experimental studies on full-scale lateral load tests of piles and experimentally derived *p-y* curves. The assimilation of field data in an organized format is intended to provide an efficient resource of information for the geotechnical engineering community that can assist in developing safer and more economical foundation systems by providing large-scale test data for various pile boundary conditions and field conditions. The database in its current state consists of 64 load test records (with data of 20 tests already uploaded) from various parts of the world. The majority of tests are located in the United States. The predominant soils are sands (41%) and clays (36%), followed by silty soils, rocks and gravels. Soil profiles and data from in-situ investigations are provided, as available, for each respective test. The database interface allows the user to collect large quantities of information by performing simple queries such as author’s name (if known), soil type, test type, pile material, head boundary condition and test configuration.

A future extension of the database in intended and can be recognized in the “Test type” window, which suggests the possibility to expand the database to centrifuge tests, model tests and analytical studies. The authors welcome the recommendation of additional pile studies to be integrated in the database and invite fellow researchers to submit (1) their test results/publications or (2) recommend valuable studies that would be of benefit to the geotechnical community.

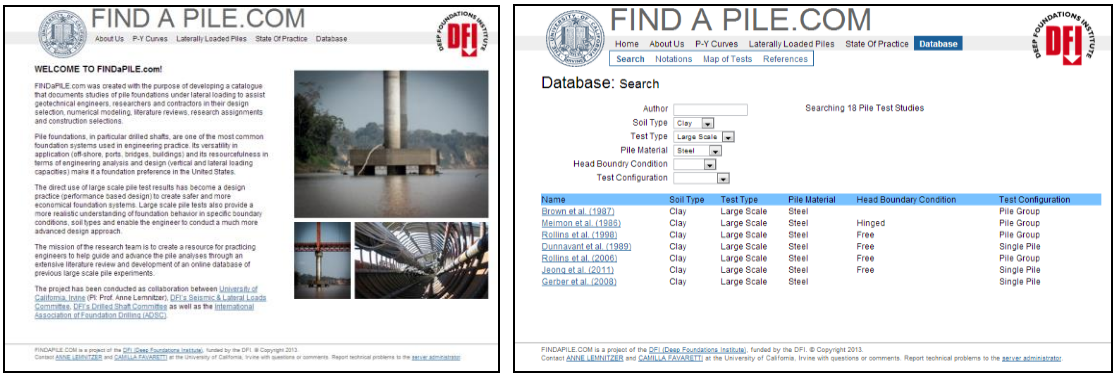

Figure 3. Screenshots from FindAPile.com: (a) welcome page; (b) database

SUMMARY

The *p-y* curve method has been widely validated in the last forty years but the most popular models used in the everyday practice still refer to single small-diameter driven piles or drilled shafts. Therefore, the selection of an adequate *p-y* curves model represents a crucial component in the analysis of laterally loaded piles since the results are very sensitive to the *p-y* curves used. Moreover, available *p-y* curve models have an empirical nature and the applicability criteria need to be reviewed carefully. A laterally loaded pile database was developed as a resource for geotechnical design engineers to assist with the selection of *p-y* formulations and offers references and test data from previously conducted large scale studies for various boundary and field conditions.

NOTATIONS

The following symbols are used in this paper:

A = 0.9 for cyclic loading; (3.0-0.8H/D) ≥ 0.9 for static loading [-];
b = Diameter (width) of the pile [m];
c_{uz} (c_a) = Undrained (average) shear strength at depth z [kN/m^2];
J = 0.5 for soft clays, 0.25 for medium clays. The value of 0.5 is frequently used [-];
K_0 = Coefficient of earth pressure at rest = 1- sin φ [-]
K_a = Minimum coefficient of active earth pressure = tan^2(45 – φ/2) [-];
K_{py} = initial modulus of subgrade reaction [kN/m^3];
p_{uz} = Ultimate soil resistance at depth z [kN/m];
z = Depth below ground surface to *p-y* curve [m];
α_s= φ/2;
β = Approximated to be 45 + φ/2 [°];
γ’ = Average effective unit weight from ground surface to *p-y* curve [kN/m^3];
γ = Average unit weight from ground surface to *p-y* curve [kN/m^3];
φ = Friction angle [°].

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