

A PSEUDO-STATIC ANALYSIS FOR THE EVALUATION OF THE LATERAL BEHAVIOR OF PILE GROUPS

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ABSTRACT

The pseudo-static approaches for the seismic analysis of pile foundations are attractive for design engineers because they are simple, when compared to difficult and more complex dynamic analyses. To evaluate the internal response of piles subjected to earthquake loading, a simplified approach based on the “*p-y*” sub-grade reaction method has been developed. The method involves two main steps: first a non linear free-field site response analysis is carried out to obtain the maximum ground displacements along the pile. Next a static load analysis is carried out for the pile, subjected to the maximum free-field ground displacements and the static loading at the pile head. The solution derived firstly for the single pile, was extended to the case of a pile group introducing the *p*-multiplier (f_m) concept proposed by Brown *et al.*, (1988). To take into account of the group action in the soil-pile interaction, the model parameters have been modified and linked to that of the single pile. Numerical results obtained by the proposed simplified approach were compared with experimental and numerical results reported in literature and it has been shown that this procedure can be used successfully for a provision of the lateral behavior of pile groups.

Keywords: Pile, pseudo-static analysis, *p-y* curve, non-linear analysis, lateral deflection.

INTRODUCTION

Pile foundations are commonly used for many types of structures and lateral load design considerations are very important because the performance of the piles during an earthquake significantly influences the integrity of the structures supported by them. There is evidence that in the past, pile groups have undergone lateral translations severe enough to cause loss of bearing support for superstructures and structural failure in the piles.

Pseudo-static approaches for the seismic analysis of pile foundations are attractive for design engineers because they are simple, when compared to difficult and more complex dynamic analyses. In pseudo-static approaches, an equivalent static analysis is carried out to evaluate key design parameters, such as the pile head displacement, the maximum bending moment and the shear force developed in the pile due to earthquake loading.

To evaluate the internal response of piles subjected to earthquake loading, Liyanapathirana & Poulos (2005) proposed a simple pseudo-static approach where a single pile is considered, including the contribution of the superstructure to the pile and the interaction between the pile and the soil. The method involves two main steps: first a non linear free-field site response analysis is carried out to obtain the maximum ground displacements along the pile and the degraded soil modulus over the

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depth of the soil deposit. Next a static load analysis is carried out for the pile, subjected to the maximum free-field ground displacements and the static loading at the pile head based on the maximum ground surface acceleration.

In practice there are two sources of loading of the pile by the earthquake: “*inertial*” loading of the pile head caused by the lateral forces imposed on the over structure and “*kinematic*” loading along the length of the pile caused by the lateral ground movements developed during the earthquake.

According to the approach suggested by Liyanapathirana & Poulos (2005), in the paper a numerical model for the analysis of the behavior of a single pile or a pile group subjected to static loadings and/or to lateral soil movements based on the “*p-y*” sub-grade reaction method has been developed.

The accuracy of the proposed approach is examined by the comparison of computed results with experimental and numerical results reported in literature. For the examined cases a reasonable agreement is observed.

PSEUDO-STATIC APPROACH

Pseudo-static design, including a lateral force acting through the centre of gravity of the sliding mass, continues to be used in practice today. The term pseudo-static applies to static analyses that account for the earthquake induced inertia forces in the soil mass by applying a body force horizontally.

In Eurocode 8 part 5, it is stated that the equivalent pseudo-static forces, in a given horizontal direction, can be calculated as:

$$V = S a_g W \quad (1)$$

where:

- V = pseudo-lateral force;
- S = amplification coefficient, depending on soil type (over a depth of 30 m);
- a_g = peak acceleration on rock soil;
- W = total dead load and anticipated live load.

Deformability of the soil can produce amplification of acceleration, that is incorporated in coefficient S , but that can be better evaluated through a site response analysis.

In the last years several simplified approaches for the analysis of single piles or pile groups have been developed that can be used with little computational effort. These methods have given results that are often in remarkable agreement with the mathematical models (Novak, 1974; Makris & Gazetas, 1992; Tabesh & Poulos, 2001). Abghari & Chai (1995), in particular, examined the performance of a pseudo-static approach for the analysis of a soil-pile-superstructure interaction.

The problem is similar in principle to the problem of statically-induced ground movements (Figure 1), even if there are additional complexities that must be recognized (Poulos, 2006). There are two sources of loading of the pile by the ground movements: “*inertial*” loading of the pile head, caused by the lateral forces imposed on the structure by the earthquake and which are then imposed on the piles, and “*kinematic*” loading along the length of the pile, caused by the lateral ground movements developed by the earthquake (Dobry & Gazetas, 1988).

Generally in the pile design only the effects of inertial loading are considered, even if kinematic loading is also very important. Part 5 of the Eurocode 8 states that piles shall be designed for the following two loading conditions:

- a) inertia forces on the superstructure transmitted on the heads of the piles in the form of axial and horizontal forces and moment;

- b) soil deformations arising from the passage of seismic waves which impose curvatures and thereby lateral strain on the piles along their whole length.

While there is ample experience of carrying out the equivalent static analysis for the inertial loading (type (a)), no specific method or procedures are available to predict deformations and bending moment from the kinematic loading (type (b)).

To take into account these two sources of loading, Abghari & Chai (1995) proposed an analysis in which the pile was subjected to the free-field soil displacements at each node along its length. These displacements were obtained from a separate free-field site response analysis. The inertial forces acting on the pile were obtained from the product of mass and spectral acceleration. These forces were applied to the pile as static forces.

Similarly Tabesh & Poulos (2001) proposed a simple approximate methodology for estimating the maximum internal forces of piles subjected to lateral seismic excitation. The method involves the evaluation of the free-field soil movements caused by earthquake computed, as example, by the well known *SHAKE* (Schnabel *et al.*, 1972) program or similar computer code and the analysis of the response of the pile to the maximum free-field soil static movements plus a static loading at the pile head, which depends on the computed maximum spectral acceleration of the structure being supported.

According to this approach in the paper a pseudo-static push over analysis using a “*p-y method*” is adopted to simulate the behavior of a single pile and/or an individual pile of a group subjected to lateral soil movements (kinematic loading) and static loadings at the pile head (inertial loading).

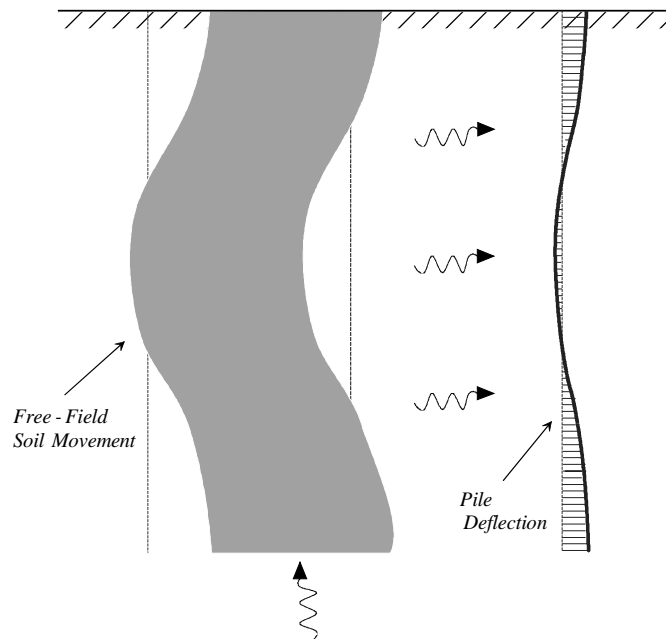


Figure 1. Profile of external soil movement caused by earthquake

MODELLING PROCEDURE

Inertial Loading

Several approaches are currently available to analyze the behavior of piles subjected to lateral load ranging from complex models, as non linear dynamic analysis and 2D or 3D finite element methods, to the use of simplified approaches as limit equilibrium (*LE*) approach and *p-y* analysis approach. The “*p-y method*” is still widespread in practice and based on literature review it appears to be one of the most attractive methods for civil engineers.

Single Pile

Current researches based on results of field tests on full scale piles, both in cohesive than in cohesionless soil, suggests to employ non linear p - y relationship (Reese *et al.*, 1974; Reese & Welch, 1975; Reese *et al.*, 2000; Juirnarongrit & Ashford, 2006), thus in the proposed approach a hyperbolic p - y relationship (Figure 2) has been adopted:

$$p(z) = \frac{y_p(z)}{\frac{1}{E_{si}(z)} + \frac{y_p(z)}{p_{lim}(z)}} \quad (2)$$

where E_{si} [FL⁻²] is the initial modulus of horizontal sub-grade reaction, y_p [L] is the pile lateral deflection, p and p_{lim} [FL⁻²] are the mobilized and the ultimate horizontal soil resistance respectively. The hyperbolic p - y relationship (2) is defined by the two parameters p_{lim} and E_{si} .

Another term that is sometimes used in place of E_{si} is the coefficient of horizontal sub-grade reaction k_h , expressed in units of force per unit volume [FL⁻³]. The relationship between E_{si} and k_h is $E_{si} = k_h/D$ being D the pile diameter. Nevertheless E_{si} is a more fundamental soil property because it is not dependent on the pile size

The numerical model is based on an iterative procedure taking into account the decrease of the model stiffness with the increasing of the applied horizontal load (Castelli *et al.*, 1995; Castelli & Maugeri, 1999; Castelli, 2002; Castelli, 2006).

Pile group

The response of a laterally loaded pile group with relatively closely spaced piles is different from that of a single pile, because of the interaction between piles through the surrounding soil. Experiments have been carried out in the last decade with the aim of deriving general rules to adapt p - y curves to take into account the group effects.

Brown *et al.*, (1988), as example, introduced the term “*shadowing*” to mean the phenomenon for which the soil resistance of a pile in a trailing row is reduced because of the presence of the leading pile ahead of it. Brown *et al.*, (1988) defined the concept of p -multiplier f_m , a multiplier of the limiting values p_{lim} capable of stretching the p - y curve for the single pile to account for the interaction among the piles in a group (Figure 2).

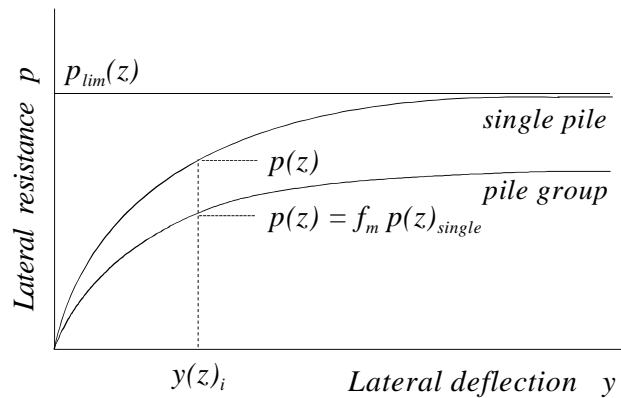


Figure 2. Hyperbolic p - y curves

This effect is related to the influence of the leading row of piles on the yield zones developed in the soil ahead of the trailing row of piles. Because of this overlapping of failure zones, the front row will be pushing into virgin soil while the trailing row will be pushing into soil which is in the shadow of

the front row piles. A consequence of this loss of soil resistance for piles in a trailing row is that the leading piles in a group will carry a higher proportion of the overall applied load than the trailing piles. This effect also results in gap formation behind the closely spaced piles and an increase in group deflection. It has been shown both theoretically and experimentally that the shadowing effect becomes less significant as the spacing between piles increases and is relatively unimportant for centre-to-centre spacing greater than about six pile diameters (Brown & Shie, 1991; Cox *et al.*, 1984; Ng *et al.*, 2001).

Similar to the design of axially loaded piles using a efficiency method, in a pile group subjected to horizontal load, the p - y relationships for the individual piles are modified to take into account the group effects by “stretching” the curve in the direction of deflection. To extend the method of the p - y curves to the case of a pile group, the p -multiplier concept proposed by Brown *et al.*, (1988) has been applied. The multiplier has obviously values in the range 0 to 1.

As suggested by Brown *et al.*, (1988), a given row within the group could be represented by a single pile p - y curve multiplied by a load factor to account for row position. With this aim, in the proposed approach the p - y curve of the piles belonging to each row of the group is obtained by scaling down the p - y curve of the single pile, multiplying the values of p_{lim} by a group reduction factor named f_m . Experimentally determined multipliers are used to adjust the magnitude of load carried by each row of piles in the group.

According to Figure 2 it should be expected that the resulting initial modulus of horizontal sub-grade reaction of a pile group is softer than the value of an isolated pile (Ashour *et al.*, 2004). Thus to adapt the hyperbolic p - y relationships defined for a single pile to the case of a pile group, a multiplier should be considered also for the initial modulus of horizontal sub-grade reaction E_{si} . In essence, the use of this multiplier is similar to the traditional approach given by NAVFAC (1982) in which the modulus of sub-grade reaction is reduced by a factor taken as a function of pile spacing (a value equal to 1 is suggested at 8 diameter pile spacing varying linearly to 0.25 at 3 diameters).

The modulus of horizontal sub-grade reaction reflects the soil-pile interaction at any level of pile loading or soil strain. Then, E_{si} will account for the additional strains in the adjacent soil due to pile interaction within the group and, consequently, E_{si} (i.e. the secant slope of the p - y curve reported in Figure 2) of an individual pile in a group will be reduced (Ashour *et al.*, 2004). Consequently we can assume for the pile group:

$$p(z) = \frac{y_p(z)}{\frac{1}{\zeta_m E_{si}(z)} + \frac{y_p(z)}{f_m p_{lim}(z)}} \quad (3)$$

where ζ_m is the empirical factor introduced in the proposed method to extend the single pile analysis to the case of a pile group (Castelli, 2006). The multiplier ζ_m has obviously values in the range 0 to 1 and it can be defined as the ratio between the initial modulus of horizontal sub-grade reaction of the pile in a group $(E_{si})_{group}$ and that of the single pile $(E_{si})_{single}$:

$$\zeta_m = \frac{(E_{si})_{group}}{(E_{si})_{single}} \quad (4)$$

A rather large amount of experiments have been carried out in the last decade for the determination of the p -multipliers. The values of these group reduction factors seems do not be depend by the soil type, pile type and/or load level. The observations collected by Brown *et al.*, (1988), McVay *et al.*, (1998), Rollins *et al.*, (1998) suggest that the multipliers depend essentially on the pile spacing, thus it is reasonable to suggest values only in terms of number of piles and position in the group (Zhang *et al.*, 1999; Rollins *et al.*, 1998). At a spacing above 6 to 8 diameters in the direction of the load vector, and

4 diameters in the orthogonal direction, the interaction among piles is negligible and the multipliers can be assumed equal to 1 (Mandolini *et al.*, 2005). The p -multipliers can be assumed constant for each row and independent of the number of piles contained in the rows, so in the practice a unique multiplier for each row is generally fixed. In the paper the values of f_m reported in Table 1 suggested by FHWA (1996), Rollins *et al.*, (1998) and Mc Vay *et al.*, (1998) have been adopted.

The values of the empirical factor ζ_m can be back-calculated comparing the numerical results with experimental evidences (Castelli, 2006).

The advantage of the proposed approach is that the load-deflection response of the pile group can be calculated using solutions for the response of a single pile. Naturally, this approach is based on an estimation of only the average head deflection of the pile group.

Table 1. Common p -multiplier values at 3 diameters center-to-center spacing

p -multiplier value	FHWA (1996)	Rollins <i>et al.</i> (1998)	Mc Vay <i>et al.</i> (1998)
Lead Row	0.8	0.6	0.8
2 nd Row	0.4	0.4	0.4
3 rd Row	0.3	0.4	0.3

Kinematic Loading

The proposed numerical method is based on the idea to evaluate the effects due to the applied load at the pile head and lateral movements along the pile length by a series of independent “ p - y curves”, which relate soil reaction and relative soil - pile movements.

The implementation of the method involves imposing a known free-field soil movement profile. When the expected free-field movement is large enough to cause the ultimate pressure of laterally spreading soils to be fully mobilized, the ultimate pressure, instead of free-field soil movement, may be used.

Piles surrounded by moving soil are relevant for many engineering applications. This soil movement will in turn displace the pile a certain amount depending on the relative stiffness between the pile and the soil. The soil loading must be considered by taking into account the relative movement between the soil and the pile. If the soil mass moves and the pile movement y_p is less than the soil movement y_s , the soil exerts a driving force on the pile. However, if the pile movement y_p is greater than the soil movement y_s , the soil provides the resistance force p_{lim} to the pile. The response of the pile can then be obtained by solving the following governing differential equation:

$$EI \frac{d^4 y}{dz^4} - p(y_p - y_s) \quad (5)$$

where EI = pile stiffness, p = soil reaction per unit pile length and z = depth.

This equation can be solved by a numerical procedure based on a pile finite element discretization, in which the pile load (force per unit area [FL⁻²]) due to relative pile-soil movement ($y_p - y_s$), can be represented by a series of p - y curves on both sides of the pile shaft for all its length (Figure 3). For a simple approach an idealized elastic p - y relationship could be used:

$$p(z) = \frac{E_{si}(z)}{D} [y_p(z) - y_s(z)] \quad (6)$$

where D is the pile diameter. To take into account that the lateral pile response to static or dynamic loading is non linear, the following p - y relationship could be adopted:

$$p(z) = \frac{[y_p(z) - y_s(z)]}{\frac{1}{E_{si}(z)} + \frac{|y_p(z) - y_s(z)|}{p_{lim}(z)}} \quad (7)$$

in which E_{si} , y_p , p and p_{lim} are defined as in equation (2). According to the proposed approach, when an individual pile of a group is taken into consideration, in equation (7) the empirical reduction factors f_m and ζ_m should be adopted.

ESTIMATION OF MODEL PARAMETERS

Construction of p - y relationships at each specified depth location depends on soil strength parameters, i.e., the friction angle for sands and the cohesion for clays. The main difficulty of applying the p - y curve approach is the appropriate evaluation of the functions parameters for a realistic estimation of single pile and/or pile group performance. In particular, the adoption of hyperbolic p - y curves requires the determination of the ultimate horizontal soil resistance p_{lim} and initial modulus of sub-grade reaction E_{si} .

In a single pile-soil interaction, the ultimate horizontal soil resistance p_{lim} can be evaluated according to the well known formulas existing in literature (Matlock, 1970) both for cohesive (Broms, 1964^a) than for cohesionless soils (Broms, 1964^b).

As concern the initial modulus of horizontal sub-grade reaction E_{si} in the proposed approach this parameter has been evaluated according to the relationships proposed by Matlock (1970), Welch & Reese (1972), Robertson *et al.*, (1989) for hyperbolic p - y curves. The most common assumption is that E_{si} is constant with depth for clays and E_{si} varies linearly with depth for sands, for sand and normally consolidated clay is often assumed to vary linearly with depth.

Most of the existing methods for obtaining p - y curves are empirical, because they are based on empirical relationships for the evaluation of the model parameters. Obviously the back-analysis of load tests is the best way to assess and to verify these relationships by the comparison between field measurements and numerical analyses.

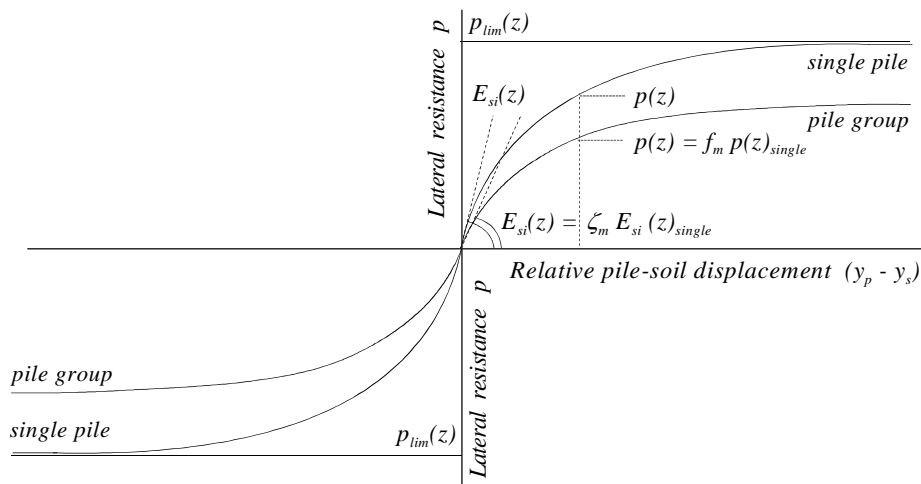


Figure 3. p - y curves to model loading due to lateral movements

An improvement of these approaches can be achieved if more reliable and relevant soil properties are adopted in defining the input parameters even if the design method is empirical or semi-empirical. The shear modulus G_o or, for clayey soils, the undrained shear strength c_u , can be considered essential parameters for the estimation of the soil response, especially in the case of dynamic loading.

The resonant column test (*RCT*) is commonly used to determine G_o in laboratory, alternatively it can be determined in situ by measurements relating it to the propagation velocity of seismic shear waves V_s and the mass density ρ , or by empirical relationships with the undrained shear strength. Kuwabara (1991), as example, reports values of the ratio G_o/c_u varying between 400 and 900. By the back-analysis of the results of lateral loading tests carried out on full scale bored piles, the following relationships for the estimation of the initial modulus of horizontal sub-grade reaction E_{si} have been proposed (Castelli *et al.*, 1995):

$$E_{si}/c_u = 200 \div 280 \quad (8)$$

Thus comparing equation (8) and the usual values of the ratio G_o/c_u it is possible to find that the ratio E_{si}/G_o assumes values ranging between 0.5 and 0.3.

VALIDATION OF THE PROPOSED APPROACH

The approach proposed was employed and implemented in an original computer code. Such code allows the assessment of the lateral response (deflection, moment and shear force distribution) of an isolated pile and a pile group including the *p-y* curve along the length of the isolated pile or the individual piles in the group.

The isolated pile or the individual pile of a group can be considered subjected to the simultaneous application of a lateral force and/or moment at its head and a free-field soil movement profile along its length. Comparison with field measurements, shows that this procedure can give lateral deflection, bending moment and shear force distributions on piles which agree with experimental ones.

Comparison with Measured Behavior of Piles under Lateral Loading

Validation of the proposed approach has been carried out comparing the numerical results with those measured in load tests reported in literature, both on single pile and pile groups. Model parameters have been assessed to obtain numerical results close to those measured.

The numerical analysis was carried out for the single piles and then for the pile groups, considering in this last case, the experimental curves in terms of average lateral load per pile versus average pile head lateral deflection. A detailed description and additional data concerning the comparison with measured behavior of single pile and pile groups under lateral loading is provided by Castelli (2006). In the following, as example, the back-analysis of the experimental results reported by Rollins *et al.*, (1998) is reported.

Experimental results reported by Rollins et al., (1998)

The first case history analyzed was reported by Rollins *et al.*, (1998) and regards the static lateral loading test carried out on a pile group (3x3) at three-diameter spacing, driven into a soil profile consisting of soft to medium stiff clays and silts underlain by sand. A single pile test was conducted for comparison. The piles were 0.305 m I.D. closed-end steel pipes with a 9.5 mm wall thickness and were driven to a depth of approximately 9.1 m. The elastic modulus of the steel was 200 GPa. The single pile and the pile group was in free-head condition. The geotechnical soil parameters have been determined by in situ and laboratory tests. The undrained shear strength was typically between 25 and 60 kPa, even if the strength increases with depth with a linear shape.

In the numerical analysis the initial modulus of horizontal sub-grade reaction was determined with the

relationship proposed by Welch & Reese (1972): $E_{si} = (E_{soi} + k_i z)$, being z the depth, k_i [FL^{-3}] the gradient of the initial modulus of horizontal sub-grade reaction (a value $k_i = 22 \text{ MN/m}^3$ was assumed) and E_{soi} the initial modulus at the ground surface, that in the analysis was assumed equal to 35 MN/m^2 .

The ultimate horizontal soil resistance was determined according to Broms' theory (1964^a) and the values of p_{lim} range between 0.14 MN/m^2 at the ground surface up to 0.72 MN/m^2 at the pile tip. In Figure 4^a is reported the comparison between measured and computed lateral head deflection of the single pile and the middle row of the pile group.

To reduce the computed load-carrying capacity of the piles in the group, a value of the p -multiplier f_m equal to 0.35, 0.39 and 0.55 was assumed for the middle row group, back row group and front row group respectively. These values of the group reduction factor adopted in the numerical analysis are very close to the values of f_m reported by Rollins *et al.*, (1998) and back-calculated for the middle row group, back row group and front row group respectively (0.38, 0.43 and 0.60).

The values of the initial modulus of horizontal sub-grade reaction for the pile group were reduced assuming for the factor ζ_m approximately the same values of f_m . With reference to the back row and front row of pile group, in Figure 4^b is reported the comparison between measured and computed load-head deflection curves. The comparison shows a good agreement between experimental and numerical results.

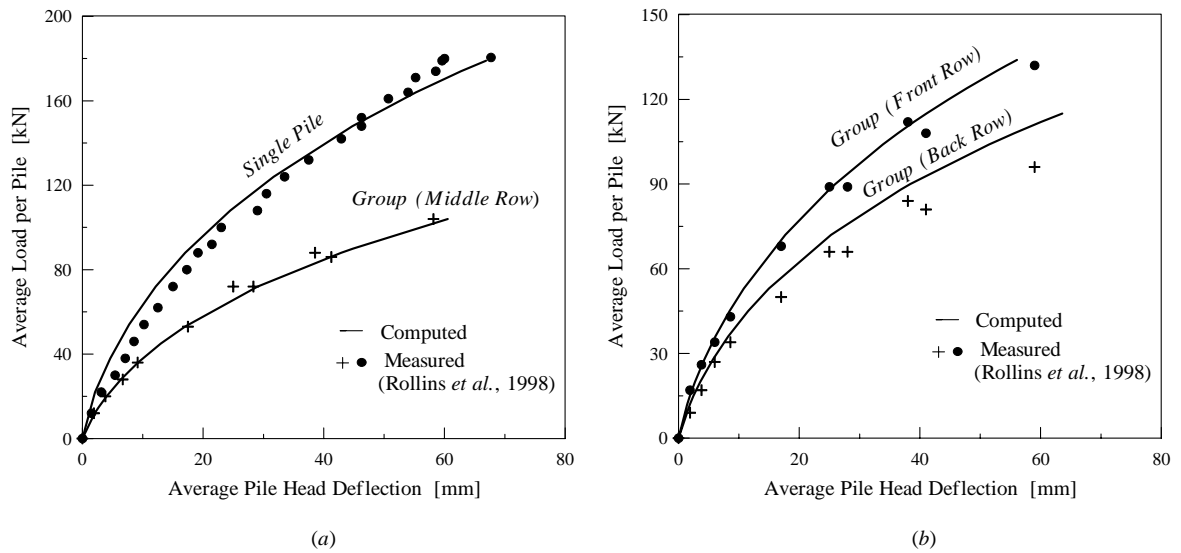


Figure 4. Comparison between measured (Rollins *et al.*, 1998) and computed pile head deflection: a) single pile and middle row group, b) back row and front row group

Comparison with a Numerical BDW Formulation

Validation of the proposed approach for kinematic loading along the length of the pile caused by the lateral ground movements developed during the earthquake has been carried out comparing the computed results with a numerical Beam-on-Dynamic-Winkler-Formulation (BDWF) proposed by Kavvas & Gazetas (1993).

The system studied refers to an end-bearing pile embedded in a two layer soil deposit named layer (a) and (b), underlain by rigid bedrock and subjected to vertically propagating S-waves (Kavvas & Gazetas, 1993). The soil is assumed to be a linear solid with Young's modulus E_a and E_b , mass density $\rho_a = \rho_b$ and Poisson's ratio $\nu_a = \nu_b = 0.40$. The pile is assumed to be elastic with Young's modulus E_p and it is in free-head condition. The sensitivity of the model has been investigated varying the values of the crucial dimensionless parameters: the pile-to-soil stiffness ratio E_p/E_a , the ratio of the S-waves

velocities V_b/V_a of the two soil layers, the pile slenderness ratio L/d , the ratio of the thicknesses of the soil layers H_a/H_b .

The cases taken into consideration, which represent two limits of a wide range of possible two-layer profiles (stiff upper crust and very soft upper layer) are: $E_p/E_a = 5000$, $L/d = 20$, $H_a/H_b = 1$, $V_b/V_a = 0.58$ (Case A) to 3 (Case D). The corresponding values of the model parameters adopted in the numerical analysis carried out by the proposed approach are: pile length $L = 20$ m, pile diameter $d = 1$ m, Young's modulus of pile $E_p = 30.000$ MPa, mass density $\rho_a = 1.63$ kNsec²/m⁴ then S-waves velocities $V_a = 36.26$ m/sec and $V_b = 21.03$ m/sec for Case A and $V_b = 108.78$ m/sec for Case D. Finally the initial modulus of horizontal sub-grade reaction according to equation (8) is $E_{si(a)} = 1071$ kN/m² and $E_{si(b)} = 360$ kN/m² to $E_{si(b)} = 9644$ kN/m² for Case A and D respectively, even if for a best agreement between measured and computed results values greater than 30÷40 % were adopted.

The numerical analysis has been carried out in linear elastic conditions according to the elastic p - y relationship given by equation (6).

Figure 5 shows the distribution with depth of the lateral ground movements (*free field*) y_s , and the pile y_p . A very good agreement can be observed between the pile deflection profiles computed by the proposed approach and those reported by Kavvadas & Gazetas (1993).

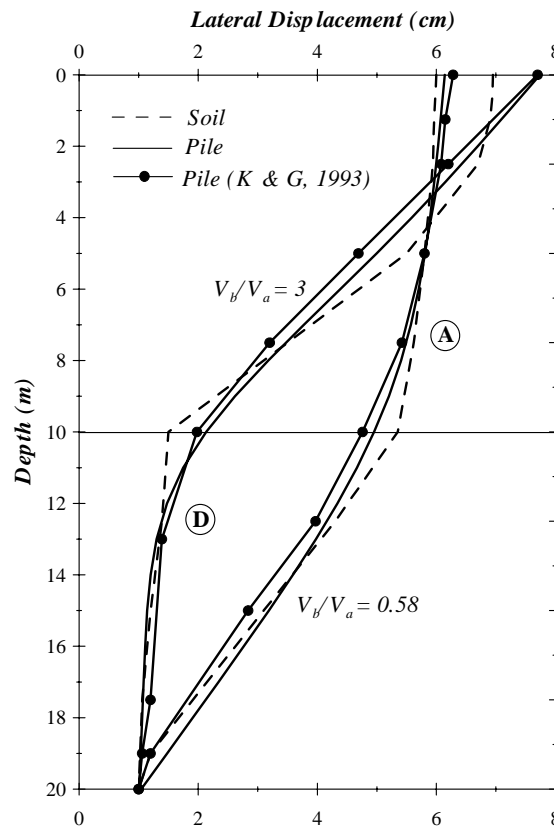


Figure 5. Distribution with depth of the lateral ground movements (free field) and the pile deflection

CONCLUDING REMARKS

To evaluate the internal response of piles subjected to earthquake loading, a simplified pseudo-static method based on the “ p - y ” sub-grade reaction approach has been developed. The method involves the

evaluation of the free-field soil movements caused by earthquake and the analysis of the response of the pile to the maximum free-field soil static movements plus a static loading at the pile head.

The approach proposed was employed and implemented in an original computer code. Such code allows the assessment of the lateral response (deflection, moment and shear force distribution) of an isolated pile and a pile group.

The solution derived firstly for the single pile, was extended to the case of a pile group by the adoption of the p -multiplier (f_m) concept proposed by Brown *et al.*, (1988). To take into account of the group action in the soil-pile interaction, the model parameters have been modified and linked to that of the single pile. The modulus of horizontal sub-grade reaction E_{si} , which reflects the soil-pile interaction at any level of pile loading, will be reduced for the additional strains in the adjacent soil due to pile interaction within the group.

Numerical results obtained by the proposed simplified approach were compared with experimental and numerical results reported in literature and it has been shown that this procedure can be used successfully for a provision of the lateral behavior of a single pile or a pile group.

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