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Full Range Analysis of Concrete Piles Subject to Lateral Loadings

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Summary A non-linear analysis methodology, suitable for design office applications, is presented for predicting the flexural response of concrete piles up to failure. Conventional methods for predicting the lateral response of piles are mainly based on the assumption of linear elastic pile and soil characteristics. These would underestimate the pile deflection and ignore load redistribution effects. Some designers have used p-y (soil reaction-local deflection) curves for representing soil non-linearity, but the concrete pile is assumed to remain linear elastic. However, concrete cracks at low tensile stress levels further contributing to non-linear pile response. The presented analysis methodology (NOLCOP) is based on a frame analogy simulation with beam-column pile elements and soil (Winkler) springs. Several typical calculations are presented, and the method is shown to predict the few reported test results with good accuracy.

1. INTRODUCTION

Pile foundations are generally used to reduce settlements and lateral deflections and also to enhance the foundation bearing capacity by transferring the applied surface loadings to deeper strata. As a result, piles would be subjected to simultaneous axial forces, flexural moments and shear forces. In addition, they can be subjected to deformation-induced loadings generated by the swelling and shrinkage of the surrounding soil or by lateral soil movements at bridge abutments, slopes, etc.

Conventional design criteria are primarily aimed at ensuring adequate foundation bearing capacity with an acceptable factor of safety under service loads generally. Lateral deflections under service loads were generally ignored and, even if considered, some methods are quite cursory by neglecting the pile-soil interaction, e.g., adoption of equivalent free standing cantilevers. However, in modern Standards (e.g., AS 2159) advocating limit state design philosophies, rational prediction of the pile response under different limit states is required including the serviceability limit state of limited deflections. The accurate analysis of pile foundations is complicated due to the soil-pile interaction involving relative stiffnesses, local yielding of soil, pile group interaction, etc. In the case of concrete piles, the structural design of the pile is in accordance with the concrete design Standard (AS 3600) while the geotechnical analysis of the pile-soil system is in accordance with the piling Standard (AS 2159) or any other acceptable design practice.

Conventional methods for predicting the lateral response of piles have been based primarily on the linear elastic behaviours of the soil and the pile. Several researchers (Poulos, 1980; Sogge, 1982; Gabr, 1994) have studied the pile behaviour assuming the soil to be non-linear, but the pile to be linear. Although steel piles would show a linear response up to the yield capacity, the flexural behaviour of concrete piles would not be linear at very low load levels as concrete cracks due to small values of concrete tensile strength.

In the design of concrete piles, two distinct steps are involved: (a) geotechnical design usually governing the pile depth, diameter, etc. and (b) structural design of the pile determining the amount of reinforcing steel or prestressing. In a majority of cases, these two steps are followed in isolation. A limited equilibrium method is adopted for the geotechnical strength design of the piles based on a prior decision about the pile type being short, intermediate or long (Broms, 1964) and this neglects the deformation compatibility between the pile and soil. The lateral pile deflection is predicted using standard design charts (e.g., Poulos, 1980) which are based on elastic analysis. The concrete pile is usually treated as an elastic uncracked member, but the structural design of the reinforcing steel is based on a cracked section ignoring concrete tensile strength [AS 3600], and this approach is therefore self-contradictory.

A non-linear analysis procedure, which considers the actual load-deformation characteristics of both the pile and the soil, should predict the lateral response and failure mechanism of the pile realistically (Vitharana, 1997). This does not require a prior knowledge of the failure mechanism (e.g., short or long pile), and would facilitate the load redistribution along the pile length from highly-stressed areas to areas of low stress. The proposed method for the non-linear analysis of concrete piles

(NOLCOP) would serve the above purpose, but needs further verifications based on in-situ load tests as there are only a few reported measurements of reinforced concrete pile behaviour which include sectional details.

2. PILE RESPONSE

The non-linear flexural behaviour of a pile can be expressed by the general differential equation:

$$\frac{d^2}{dx^2}\left(EI\frac{d^2w}{dx^2}\right)+p=f \qquad (1)$$

where x = given depth along the pile, EI = flexural rigidity or bending stiffness (FL^2), E = Young's modulus of elasticity of the pile material (F/L^2), I = pile moment of inertia (L^4), w = lateral deflection (L) at a given depth, p = soil reaction per unit pile length (F/L), and f = applied laterally distributed load at point x (F/L). The effect of a simultaneous axial force enters Eq. 1 as a second-order effect, and this can be generally ignored due to its insignificant effect.

Standard analytical solutions to Eq. 1, based on beam-on-elastic foundation systems, are available (Hentenyi, 1946) for special cases. The numerical solutions to Eq. 1 have been obtained by finite-difference techniques (Poulos, 1980). Solutions for non-linear pile responses have been obtained by Kramer (1988) for steel piles having idealised elastic-perfectly plastic moment-curvature relationships. However, the response of concrete piles is much more complex with unique characteristics dependent on concrete properties, sectional details, axial force-moment combination, etc.

In the p-y method of analysis (Winkler-type), the soil reaction p (F/L) at a given point x is given by the product of local lateral deflection w, subgrade reaction modulus k (F/L^3), and the pile diameter (d_p):

$$p = d_p k w \qquad (2)$$

This p-y approach is quite useful in defining the non-linear soil response. The major draw back in the use of p-y curves (Winkler-type analysis) is considered to be its inability to represent the continuous nature of the soil mass in contrast to an elastic half-space analysis. However, the continuous nature of the foundation is indirectly incorporated in the Winkler-type analysis as p-y curves are mainly based on the results from in-situ load tests. Elastic half-space analysis methods would also become invalid in the non-linear range because the basic formulations involved are based on linear elastic soil behaviour, e.g., Mindlin formulations. The adoption of a subgrade reaction approach with p-y curves therefore permits the non-linear soil behaviour to be incorporated conveniently with computational simplicity. Strain-softening effects (in which the strength of the soil falls below the peak value at larger strains) can also be incorporated easily with the help of appropriate p-y curves.

3. METHOD OF ANALYSIS

Fig. 1 shows the method of analysis adopted in the current study. In the finite-element discretisation, the pile is represented by rigidly-connected beam-column members while the Winkler subgrade reaction (p) is represented by discrete springs or pin-ended struts. In the elastic range, this simulation can be simply carried out with a standard plane frame analysis software used by structural engineers. The struts are assigned with the respective subgrade modulus values times their contributory area as axial stiffness. The beam members are assigned with the respective flexural stiffness (EI) values. As shown schematically in Fig.1, depending on the values of the local deflections, the soil reactions vary in accordance with assigned p-y curves.

In the current study, an analysis software (NOLCOP) was developed to analyse the pile-soil system in the non-linear range. The non-linear analysis was carried out incrementally with the use of updated tangential stiffness values in each load increment. The basic details of the soil and concrete pile are given in Section 4.

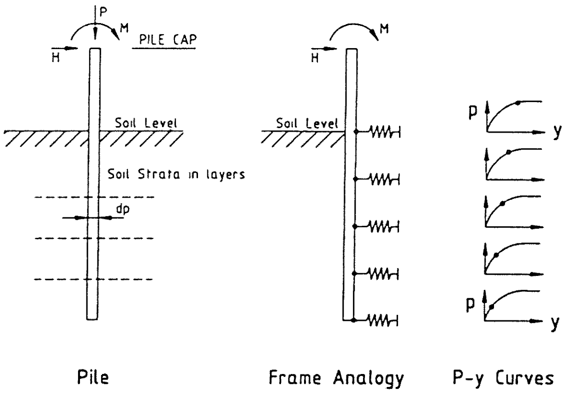


Figure 1. Modelling laterally loaded pile.

4. NON-LINEAR ANALYSIS PARAMETERS

4.1 Computation of p-y Curves for Soil

The determination of p-y curves can be either based on the available models proposed by several researchers (Reese, 1997; Gabr, 1994; Pender, 1993) or from in-situ load tests for a given site. The characteristics of p-y curves over a pile length are affected in two ways: (i) with respect to depth x, and (ii) with respect to lateral deflection w at a given depth. Both aspects should be considered in selecting p-y curves if an accurate prediction of the pile response is required.

The non-linear relationship between the subgrade reaction p and the lateral deflection w has been expressed in different forms by various researchers relating to initial (low strain) subgrade modulus k_i and the ultimate lateral soil pressure p_u (Fig. 2). Generally, hyperbolic relationships are popular, and Reese (1997) has recently proposed a variation to this by a 3-segmental curve. As shown by Ruesta (1997), the basic shapes of many available curves are quite similar and differ mainly in the computation of the ultimate soil resistance. The hyperbolic p-y curve given by Pender (1993) based on the results from in-situ load tests, is used in the current study, Eq. 3:

$$w = \frac{p}{k_i}\left(\frac{p_u}{p_u - p}\right)^n \tag{3}$$

where k_i = initial (low strain) subgrade reaction modulus (F/L^3), p_u = the ultimate lateral pressure at a given depth (F/L^2), and n = an empirical index. The secant subgrade modulus, Fig. 2, k_e (= p/w) is then given by:

$$k_e = k_i\left(\frac{p_u - p}{p_u}\right)^n \tag{4}$$

Typical values for the empirical index n are: n = 1 for *sand*, and n = 0.2 for *clay*. Typical values for p_u are: *sand* = 3 times the passive pressure allowing for 2-dimensional effects, and *clay* = $2c_u$ at the ground surface to constant $9c_u$ below a depth of $3.5d_p$ based on limited equilibrium assumptions (Broms, 1964), where c_u = undrained cohesion (kN/m^2).

The tangential subgrade reaction modulus, Fig. 2, k_t (=dp/dw) can be expressed mathematically for use in the incremental non-linear analysis, Eq. 5:

$$k_t = \frac{dp}{dw} = k_i\frac{(p_u - p)^{n+1}}{p_u^n\{np + p_u - p\}} \tag{5}$$

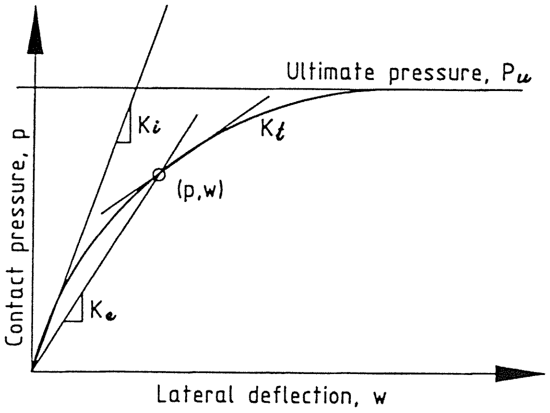


Figure 2. P-y curve for representing soil reaction.

P-y curves with strain-softening characteristics beyond the peak strength (Reese, 1975) can be treated similarly with known residual strength values and the shape of the unloading branch. While the accurate determination of k_i is important for calculating pile deflections at low load levels, its importance would diminish towards high load levels where plastic deformations of soil would take place. The variation of the initial subgrade reaction modulus k_i (F/L^3) with pile depth x has been investigated by several researchers (Reese, 1997; Sogge, 1982). The generalised equation proposed by Sogge (1982) for determining initial k_i along the pile depth is described by:

$$k_i = k_{\max}\left(\frac{x}{L}\right)^m \tag{6}$$

where x = depth at which k_i is being calculated, k_{max} = maximum value of k_i occurring at pile length L, and m = an empirical index the typical values of which are 0.0 for overconsolidated *clay* and 1.0 for *sand*. Typical values of k_{max} for sand and clay are given in (Sogge, 1982).

4.2 Determination of Concrete Pile Behaviour

In the design of reinforced concrete piles, the required amount of reinforcing steel is generally governed by the strength and durability criteria (AS 3600). In order to avoid the corrosion of reinforcing steel under aggressive exposure conditions, the crack widths under service load levels should be limited to allowable values (Grayson, 1995) corresponding with a reinforcing steel stress of 100 to 120MPa generally. Therefore, an accurate estimate of the flexural moments acting on the pile is important for ensuring adequate durability over a 50-60 year design life. The failure criterion of a pile foundation can also be determined by an acceptable level of deflection which could occur well before the ultimate failure load for the pile.

At loads well below the ultimate capacity, concrete cracks when the tensile strength f_t is exceeded. The behaviour of concrete pile sections is governed by the composite action of concrete and reinforcing steel, and this becomes more complex with the onset of cracking in concrete. A micro-level analysis, in which the behaviours of both concrete and steel are considered individually, will be permitted only in a sophisticated finite element analysis procedure having the capability to model the discrete cracks. This complexity has been avoided by the development of a simple beam-column element (Vitharana, 1998) which represents the non-linear behaviour of reinforced concrete by an effective flexural stiffness.

Typical characteristics of the moment-curvature response (M-ϕ) of reinforced concrete sections are schematically shown in Fig. 3. The stiffness of the cracked concrete section (E_cI_{cr}), in which there is no tensile resistance of concrete, is much lower than that of the uncracked gross section (E_cI_g), and E_c is the Young's modulus for concrete. The neutral axis depth (where the strain is zero) of a cracked concrete section can be calculated by satisfying strain compatibility between concrete and steel with the equilibrium of applied loadings and internal resisting stresses in Fig. 4 where β, equal to half of the subtending angle, defines the neutral axis depth. The formulations for determining the value of β are given in (Vitharana, 1997), and similar expressions can be obtained for any given pile shape.

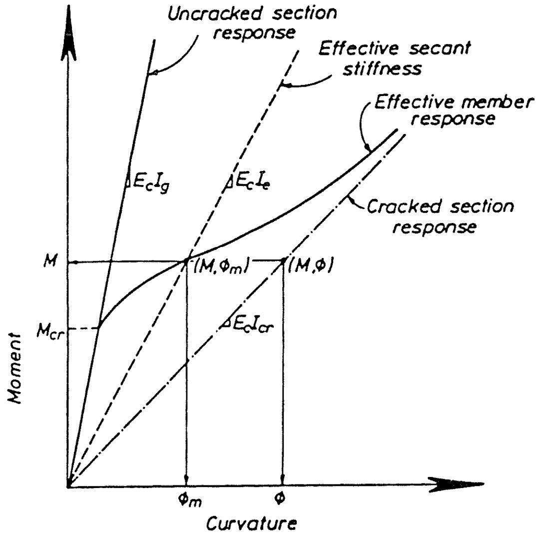
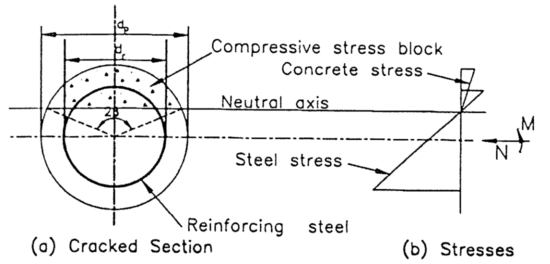


Figure 3. Typical M-ϕ relationship for a reinforced concrete member.



(a) Cracked Section (b) Stresses

Figure 4. Cracked section of a circular concrete pile.

For a circular pile, the moment of inertia of the uncracked section is given by Eq. 7:

$$I_g = \frac{\pi d_p^4}{64} \qquad (7)$$

where d_p = pile diameter. In a circular pile, the reinforcing steel is placed around the pile at a ring diameter d_r, and the area of reinforcing steel is given by the steel ratio ρ which is equal to total steel cross sectional area A_s divided by the pile uncracked sectional area A_g.

$$\rho = \frac{A_s}{A_g} \quad and \quad A_g = \frac{\pi d_p^2}{4} \qquad (8)$$

Due to space limitations, the derivation of cracked section moment of inertia I_{cr} and steel stress value f_s is not presented herein, and reference should be made to (Vitharana, 1997) for details. These are useful for the rapid evaluation of the cracked section behaviour of concrete piles under service loads.

Fig. 3 indicates a sharp decrease in the value of flexural stiffness (i.e., low cracked section stiffness E_cI_{cr}) once a concrete section cracks at the cracking moment M_{cr} corresponding with the tensile strength f_t. However, the observed behaviour of reinforced concrete sections (Vitharana, 1998; ACI318, 1989) has shown that the flexural stiffness drops more gradually due to the bonded concrete between discrete cracks according to the effective secant stiffness (E_cI_e) in Fig. 3. This is usually known as the tension stiffening effect and should be considered in the evaluation of the response of concrete members. The non-linear analysis method, adopted in NOLCOP, is based on the non-linear effective secant stiffness E_cI_e (in Fig. 3) for $M > M_{cr}$ and the involved formulations are given in (Vitharana 1997 & 1998).

$$M_{cr} = f_t \frac{I_g}{(d_p/2)} \qquad (9)$$

Once the reinforcing steel yields corresponding with the yield moment M_y (when the steel stress f_s = steel yield strength f_y), the yield plateau can be assumed to occur with a nearly-flat moment curvature response in Fig. 3.

5. TYPICAL CALCULATIONS AND CASE STUDIES

In order to examine the various factors, particularly the effect of the cracking of concrete, a number of solutions have been obtained for idealised cases. There are only a few reported measurements of concrete pile behaviours with sufficient sectional details available to carry out comparison between predicted and observed behaviours. Two case studies were also considered. For the idealised cases, the analyses were carried out based on the following:

(a) Pile dimensions and details are: pile dia (d_p) = 1000mm, annular steel ring dia (d_r) = 800mm, pile length (L) = 15m, E_c = 27,500MPa, f_t = 1.5MPa and reinforcing steel ratio ρ = .01. The pile properties were determined from the design aids given in (Vitharana, 1997) and some of the relevant values are: uncracked section moment of inertia I_g =

4.91×10^{10} mm^4, cracking moment M_{cr} = 150kNm, yield moment M_y = 800kNm, and cracked section moment of inertia $I_{cr} = 0.2I_g$.

(b) Soil is medium dense sand with m = 1.0 (Eq. 6), k_{max} = 40,000kN/m^3, and n = 1.0 (Eq. 3). The ultimate pressure p_u is given by $3K_p\sigma_v'$ where K_p = passive earth pressure coefficient equal to 3, and σ_v' = effective vertical stress at a given depth.

In order to show the effect of the non-linear behaviour of the pile, two basic types of analysis have been compared:

(i) Elastic pile behaviour as per the conventional analysis, but the soil behaviour is non-linear.

(ii) Non-linear behaviours of both the pile and soil (i.e., cracking of the concrete pile is allowed to occur).

TYPICAL CASES

CASE No. 1: Free-head pile subject to lateral force H

The pile is subjected to a lateral (horizontal) force H of 200kN at the pile head. The pile head is assumed to be free, i.e., allowed to rotate and translate.

Fig. 5a shows the variation of the pile head deflection with the applied load. Elastic pile behaviour was determined for both uncracked (I_g) and cracked (I_{cr}) moment of inertia values. Elastic pile behaviour with I_g would represent the conventional analysis method resulting in low deflection. Elastic pile behaviour with I_{cr} (= moment of inertia of the cracked section) grossly overestimates the deflection value. The non-linear analysis, carried out with NOLCOP incorporating the effective stiffness of the cracked pile, predicts a deflection value which is in between these two extremes. The lateral displacement at the pile head has been increased by about 100% due to the non-linear behaviour of the pile, and therefore this aspect should be considered rationally if an accurate prediction of the pile head deflection is required.

Figs. 5b and c show the distributions of the flexural moment and pile deflection along the pile depth at H = 200kN for non-linear and uncracked pile analyses. For this particular case, the non-linearity of the pile caused by the cracking of concrete has relaxed the peak flexural moment value by about 30%.

CASE No. 2: Fixed-head pile subject to lateral force H

In this case, the pile is subjected to a lateral (horizontal) force H of 200kN similar to CASE No. 1 above, but the pile head is assumed to be restrained against rotation, i.e., fixed-head. Fig. 6 shows the comparisons, and the general trends are similar to those for CASE No. 1 above. The condition of head fixity against rotation, has reduced pile head deflection in all the cases (Fig. 6a). However, the non-linearity of the pile has relaxed the negative moments induced at the pile head by about 30% (Fig. 6b). This relaxation is quite significant and should be considered for economical design of the pile and pile cap.

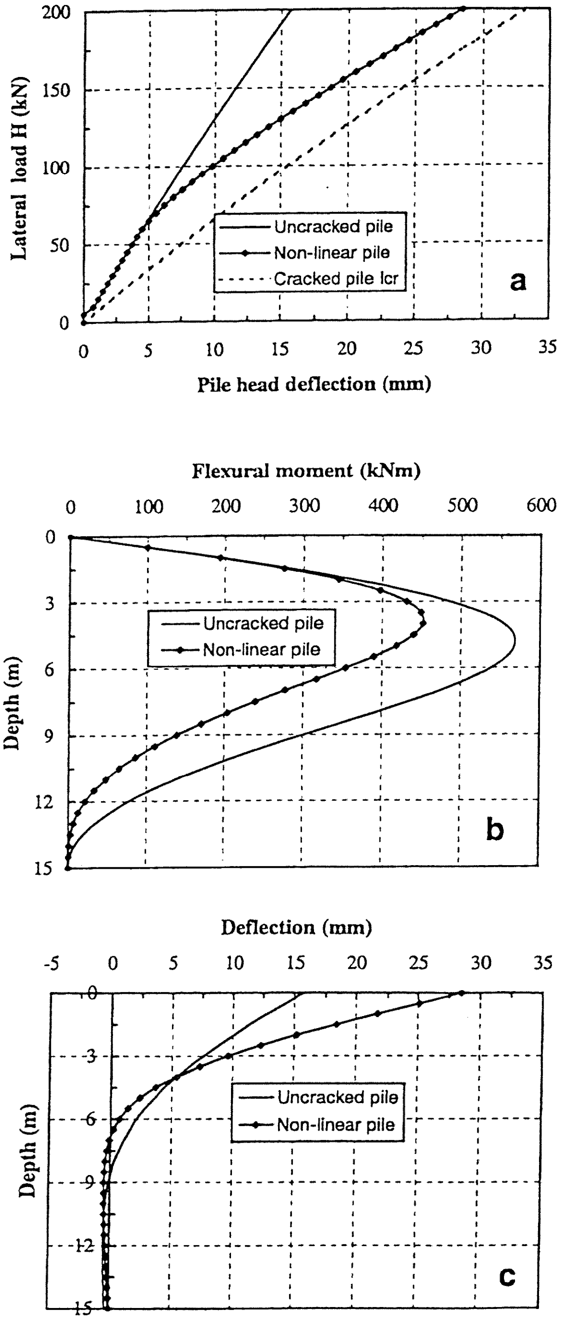


Figure 5. "Free-head" pile subject to lateral force H – *Case No. 1*

CASE No. 3: Free-head pile under lateral soil movement

Piles are subjected to lateral movements imposed by various structures such as bridge abutments, slopes, and tectonic movements of the top soil strata. This deformation-induced loading condition is different from the applied loading conditions considered above in that the deformation-induced loadings are stiffness-dependent, i.e., reduction in stiffness relaxes the magnitude of the deformation-induced loadings. Therefore, it can be expected that the cracking of concrete piles with a reduced stiffness would relax the magnitude of the loads developed on the pile. Based on elastic half-space theory, Poulos (1973) has studied the significance of horizontal soil movements on the pile response under different conditions.

In this example, it was assumed that the pile is subjected to a free-field lateral soil movement of 75mm at the pile head linearly reducing to zero at a depth of 5 m, i.e., within top 30% of the pile length. The pile head is allowed to rotate and translate, i.e., free-head condition. Fig. 7 shows the variations of the flexural moment, subgrade reaction pressures (kN/m^2), and lateral displacement along the pile depth. As can be seen, the pile non-linearity has relaxed the peak flexural moment by about 60% while the pile head deflection has increased due to less flexural rigidity associated with the cracking of the pile. Therefore, the evaluation of deformation-induced loadings on a pile should be based on a non-linear analysis if an economical and safe design is to be achieved.

In the absence of a detailed parametric study, a reasonable approach would be to reduce the deformation-induced flexural moments, calculated from an uncracked elastic pile analysis, in proportion to the reduction in the pile flexural stiffness in the appropriate stress range.

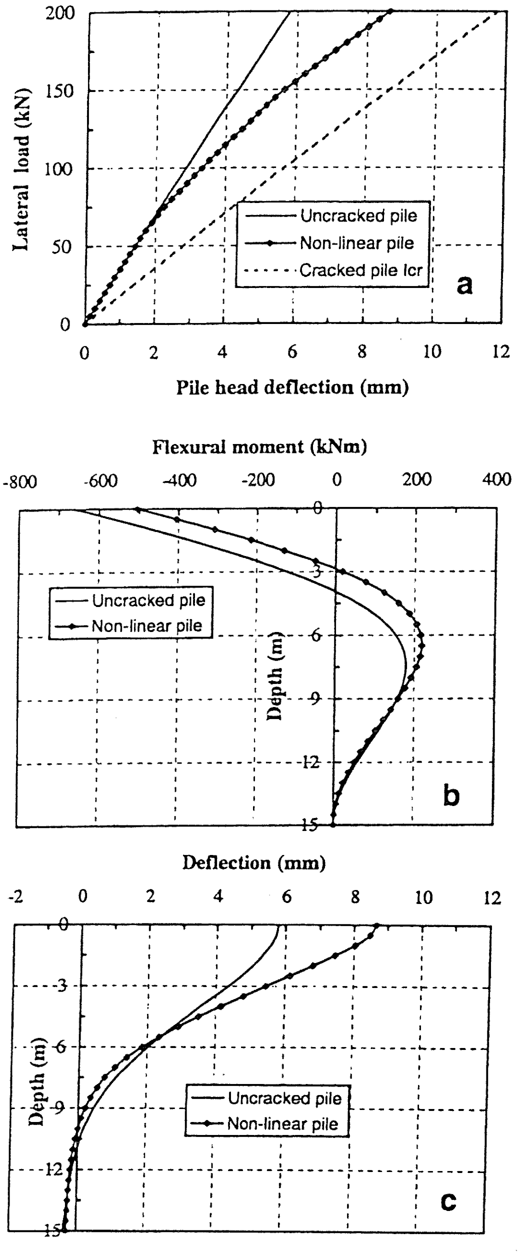


Figure 6. "Fixed-head" pile subject to lateral force H – *Case No. 2*

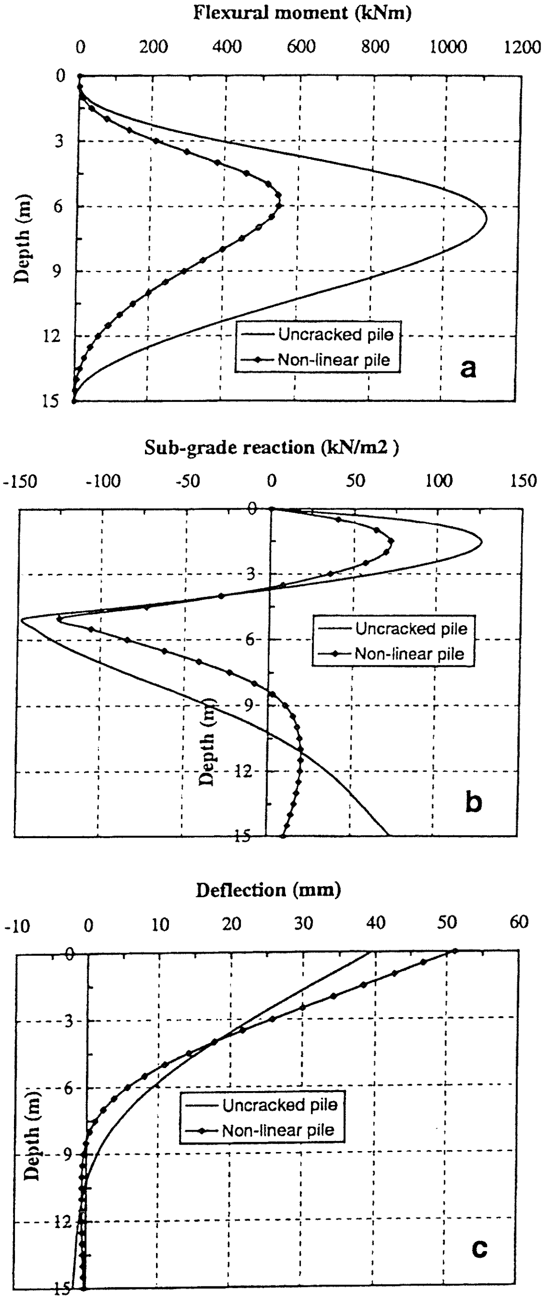


Figure 7. "Free-head" pile subject to soil movement – *Case No. 3*

CASE STUDIES

Ruesta and Townsend (1997)

Ruesta and Townsend (1997) described the in-situ load tests on prestressed concrete piles; pile diameter d_p = 760mm, and total pile length L = 15m. Extensive predriving field tests were carried out to determine the soil parameters including p-y curves. The foundation consisted of sand (4m thick) overlying a partially cemented sand (10m thick). The horizontal load was applied at a height of 2.0m from the mud line (ground surface). The foundation was under submerged conditions with the sea water level being at 1.8m above the ground surface. Some of the relevant soil parameters are: *sand* - k_i = 16,300kN/m^3, submerged density = 9.0kN/m^3; *cemented sand* - k_i = 34,000kN/m^3, submerged density = 11.0 kN/m^3.

The prestressed piles were also strengthened with a 350mm dia. steel pipe (9.5mm thick) thus enhancing the stiffness of the pile in the cracked state. Some of the material and section details of the pile are: E_c = 34,475MPa, effective prestressing force N = 6,000kN, total steel ratio ρ = .025, cracking moment M_{cr} = 850kNm, yield moment M_y = 1400kN, uncracked section moment of inertia I_g = 1.64x10^{10} mm^4, and cracked section moment of inertia (neglecting N) $I_{cr} = 0.266\ I_g$.

The analysis of the pile was undertaken with NOLCOP using Pender's hyperbolic p-y curves (Eq. 3). It was shown that the best-fit was given with n = 0.8 and p_u = 5 times the passive pressure. The piles were loaded laterally up to failure in eight load increments and the cracking of the pile occurred when the load H was at 225kN. Fig. 8a shows the load-pile head deflection curves obtained for the pile. As shown, the proposed non-linear analysis method predicted the pile response with excellent agreement while the conventional linear pile behaviour assuming an uncracked pile grossly underestimated the pile head deflection for load levels above 275kN.

Figs. 8b and c show the distributions of the flexural moment and lateral deflection along the pile depth for a non-linear pile and an uncracked elastic pile at H = 350kN. As can be seen, the magnitude of flexural moment is less sensitive to the value of the pile stiffness due to reduced tendency for load redistribution because M_{cr} is quite high in this particular case (being a prestressed pile). The peak moments occurred near the ground surface governed by the cantilever action with a 2m free span.

Reese (1997)

Reese (1997) reported an in-situ load test carried out by the California Department of Transportation on two (2) bored reinforced concrete piles constructed in weak rock: pile diameter d_p = 2.25m, and total pile length L = 15m including a 1.5m cantilever length above the ground surface. The compressive strength of the rock q_{ur} varied from 1.85MPa at the ground surface to 16.0MPa at the pile base.

Some of the material and sectional details of the pile are: E_c = 28,050MPa, d_p = 2.25m, steel ring dia d_r = 1.89m, steel ratio ρ = .0146, concrete tensile strength f_t = 1.25MPa, cracking moment M_{cr} = 1600kNm, yield moment M_y = 18,000kNm, uncracked moment of inertia I_g =1.258m^4, and cracked section $I_{cr} = 0.282\ I_g$.

In the in-situ load tests, the load was applied at 1.4m above the ground surface up to failure at the maximum load H = 9,000kN. The analysis of the pile was undertaken with NOLCOP using Pender's hyperbolic p-y curves (Eq. 3). It was found that the best-fit was given with n = 0.4 and p_u = rock compressive strength q_{ur} at the surface to 5 times q_{ur} below depth 3d_p. Fig. 9a shows the load-pile head deflection curves obtained for the pile. As can be seen, the proposed non-linear analysis method predicted the pile response with excellent agreement while the linear uncracked pile analysis grossly underestimated the pile deflection. In this case, the difference in the responses between linear uncracked and non-linear pile behaviours is large being a reinforced concrete member in which significant stiffness reduction occurs with the onset of cracking at M_{cr}. The difference between deflections in Fig. 9a is higher compared with that in Fig. 8a for a prestressed pile.

The distributions of the flexural moment and lateral deflection along the pile depth are shown in Figs. 9b and c for a non-linear pile and an uncracked elastic pile at H = 9000kN. As can be seen, the flexural moment is less sensitive to the value of the pile stiffness, and a major part of the pile deflection occurs in the first four meters.

6. CONCLUSIONS

- Modern Standards and design practices, advocating limit state design philosophies, require the designers to rationally predict the behaviour of concrete pile foundations. Reinforced concrete piles crack at very low load levels thus resulting in a non-linear pile response, and this is significant even at service (working) load levels. Therefore, the conventional assumption of uncracked elastic piles would become invalid.
- A pile-soil system can be easily simulated by a frame analogy using p-y curves to represent soil reaction. This simulation has much more flexibility in incorporating various pile and soil details compared with conventional methods

based on elastic theories. The analysis methodology developed in the current study, NOLCOP, is capable of predicting the non-linear response of concrete piles up to failure.

- As shown by typical examples, under applied loadings, the pile non-linearity (associated with the cracking of concrete) has significant influence on the pile response. It would redistribute and reduce the magnitude of flexural moments along the pile length while the pile deflection would be increased significantly. The conventional adoption of an uncracked concrete pile assumption would result in inaccurate predictions being either conservative or unconservative under different conditions.
- As shown, the pile non-linearity would greatly relax the flexural moments generated by lateral soil movements due to the stiffness dependency characteristics of deformation-induced loadings. In the absence of a detailed parametric study, an "interim" approach would be to reduce the deformation-induced flexural moments, based on an uncracked pile assumption, in proportion to the reduction in effective flexural stiffness. This reduction should not be applied to flexural moments caused by applied loadings unless confirmed by a non-linear analysis. A detailed parametric study is being currently undertaken in this regard.
- Comparisons between theoretical predictions using NOLCOP and observed pile responses on field load tests showed excellent agreement. However, further verifications are suggested based on field load tests on concrete piles, covering a range of practical situations. There are only a few reported measurements of concrete pile behaviour which include pile sectional details.

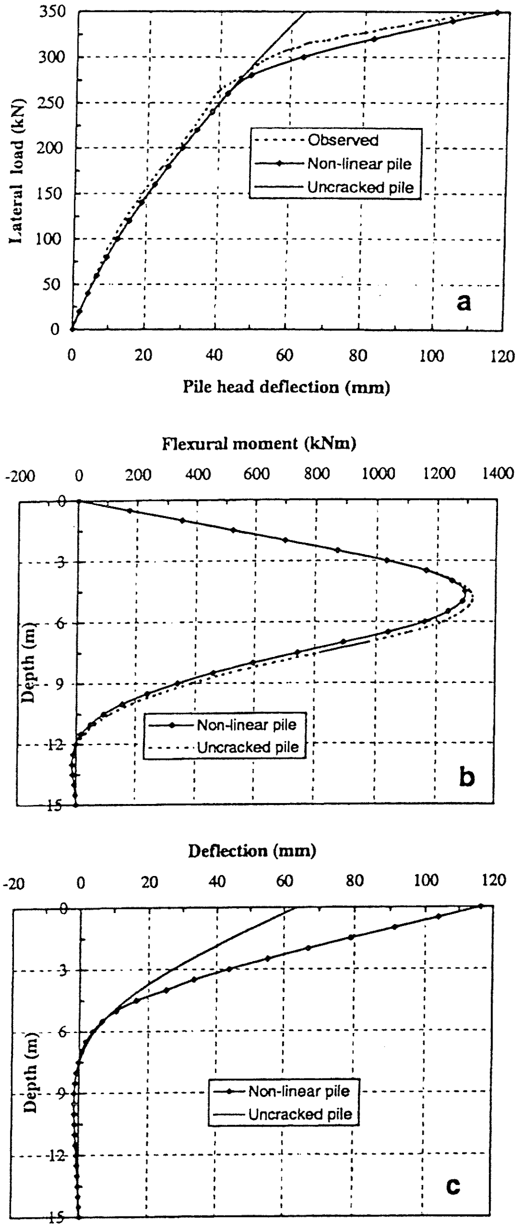


Figure 8. Comparison between observed *(Ruesta, 1997)* and predicted responses.

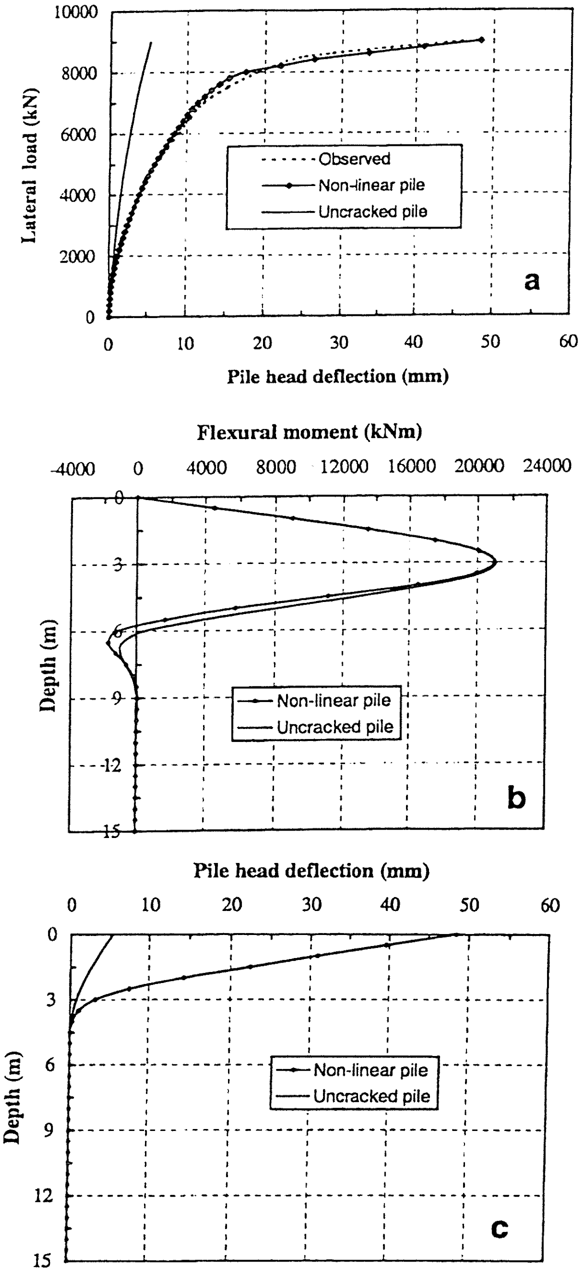


Figure 9. Comparison between observed *(Reese, 1997)* and predicted responses.

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