Ground Movements – A Hidden Source of Loading on Deep Foundations

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Ground movements can arise from a large number of sources and can have a significant effect on nearby piles and deep foundations. The loading of the piles by ground movements is a different mechanism to that arising from direct applied loading to the pile head, and consequently it is not generally possible to adequately analyze the effects of ground movements simply by applying some type of equivalent loading to the pile head. The main effects of ground movements are the development of additional movements, axial forces and bending moments in the piles, and thus the key design aspects are related to movements and to the structural integrity of the pile. However, the ultimate geotechnical load carrying capacity is generally not affected by the ground movements themselves.

This paper will describe an approach to the analysis of ground movement effects on piles, considering axial and lateral movements separately. Some of the main features of pile response will be discussed for three specific problems involving ground movements:

1. Piles near and within embankments;
2. Piles near an excavation for a pile cap;
3. Piles subjected to seismic ground motions.

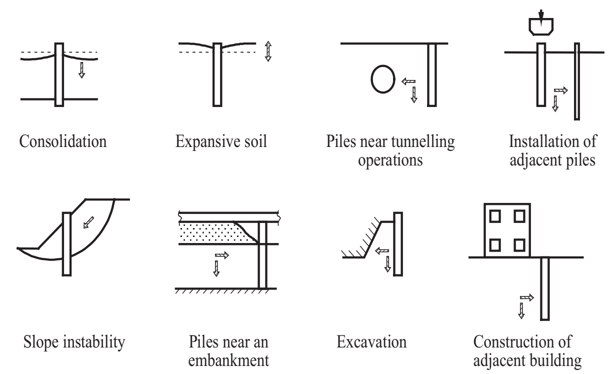
Introduction

There are many circumstances in which pile foundations may be subjected to loadings arising from vertical and/or lateral movements of the surrounding ground. Fig. 1 illustrates a number of these circumstances. In such cases, at least two important aspects of pile foundation design must be considered:

1. the movements of the piles caused by the ground movements
2. the additional forces and/or bending moments induced in the piles by the ground movements, and their effect on the structural integrity of the piles.

Problems involving the effects of ground movements on piles may be analyzed in at least two ways:

1. Via a complete single analysis (generally numerical) involving modelling of the pile, the soil and the source of the ground movements. This will give a complete solution for the behaviour of both the soil and the pile.
2. Via a simplified approach involving initial separation of the soil and the pile ("sub-structuring") so that the soil movements are first computed and then imposed on the pile. In this approach, the focus is generally placed on the behavior of the pile.



[Fig. 1] Some Sources of Ground Movements

This paper summarizes a consistent theoretical approach to the analysis of ground movement effects on piles, for both vertical and horizontal movements, which falls into the second category. Two distinct stages are involved in this analysis:

1. Estimation of the "free-field" soil movements which would occur if the pile was not present;
2. Calculation of the response of the pile to these computed ground movements.

Some specific cases of ground movement are then considered, and in each case, a discussion is given of the general features of pile behaviour revealed by the theory, and typical applications to practical field cases are described.

Analysis of Pile Response to Externally Imposed Soil Movements

Axial response

The analysis used for axial response of the pile has been described by Poulos and Davis (1980) and has been used to analyze problems of negative friction of piles in consolidating soil, and of tension and uplift of piles in expansive soil. It employs a simplified form of boundary-element analysis, in which the pile is modelled as an elastic column and the surrounding soil as an elastic continuum.

The pile is divided into a series of cylindrical elements. The vertical movement of each element is related to the applied load, the pile-soil interaction stresses, the pile compressibility, and the pile tip movement. The vertical movement of each supporting soil element depends on the pile-soil interaction stresses, the modulus or stiffness of the soil, and also on any free-field movements that may be imposed on the pile. To simulate real pile response more closely, allowance may be made for slip at the pile-soil interface, i.e., the pile-soil interaction stresses cannot exceed the limiting pile-soil skin friction.

The above analysis has been implemented via a FORTRAN computer program, PIES (Poulos 1989).

The analysis of axial pile response requires a knowledge of the pile modulus, the distribution of soil modulus and limiting pile-soil skin friction with depth, and the free-field vertical soil movements. The assessment of the pile-soil parameters (in particular the soil modulus and limiting pile-soil skin friction) has been discussed by several authors (e.g. Meyerhof 1976; de Cock and Legrand, 1997; Poulos 1989, 2001).

Lateral response

Details of the lateral-response analysis have been given by Poulos and Davis (1980), and it also relies on the use of a simplified boundary element analysis. In this case, the pile is modelled as a simple elastic beam, and the soil as an elastic continuum. The lateral displacement of each element of the pile can be related to the pile bending stiffness and the horizontal pile-soil interaction stresses. The lateral displacement of the corresponding soil elements is related to the soil modulus or stiffness, the pile-soil interaction stresses, and the free-field horizontal soil movements. A limiting lateral pile-soil stress can be specified so that local failure of the soil can be allowed for, thus allowing a nonlinear response to be obtained.

The analysis has been implemented via FORTRAN computer programs, including a proprietary program called ERCAP, and in an alternative approach, via the program PALLAS (Hull, 1996).

Group Effects

The analysis of ground movement effects on groups of piles has been reported by several authors, for example, Kuwabara and Poulos (1989), Teh and Wong (1993), Chow et al (1990), Xu and Poulos (2001). All these authors have found that under purely elastic conditions, group effects tend to be beneficial to the pile response as compared to single isolated piles, i.e. the group effects tend to reduce the pile movement and the forces and moments induced in the piles. This is especially so for the inner piles within a group, which, because of the pile-soil-pile interaction are, in effect, "shielded" from the soil movements by the outer piles. Experimental work reported by Chen (1994) indicates that the ultimate lateral pile-soil pressures are affected to some extent by grouping and that the group effect may either increase or decrease the pile response, depending on the pile configuration and spacings.

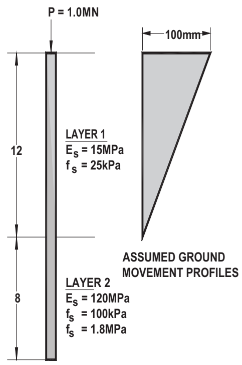
From the viewpoint of design, it is generally both convenient and conservative to ignore group effects and analyze a pile as if it were isolated. Thus, in the remainder of the paper, attention is concentrated on ground movement effects on single isolated piles.

Loading Via Ground Movements Versus Direct Applied Loading

There is a widespread misconception that the effects of externally imposed ground movements on piles can be estimated by the application of equivalent loadings at the pile head. To illustrate the consequences of this procedure, the case in Fig. 2 has been analyzed. A single pile in a two-layer soil profile is considered, and the pile is subjected to the following sources of loading:

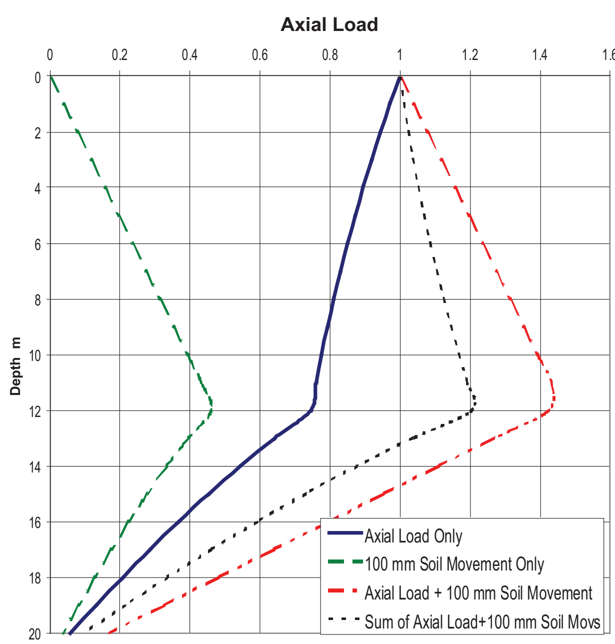
- An applied vertical load of 1.0 MN
- An applied lateral load of 0.1 MN at the pile head
- Vertical ground movement profile which decreases from 100 mm at the ground surface to zero at a depth of 12 m

- A lateral ground movement profile which also decreases from 100 mm at the ground surface to zero at 12 m depth.



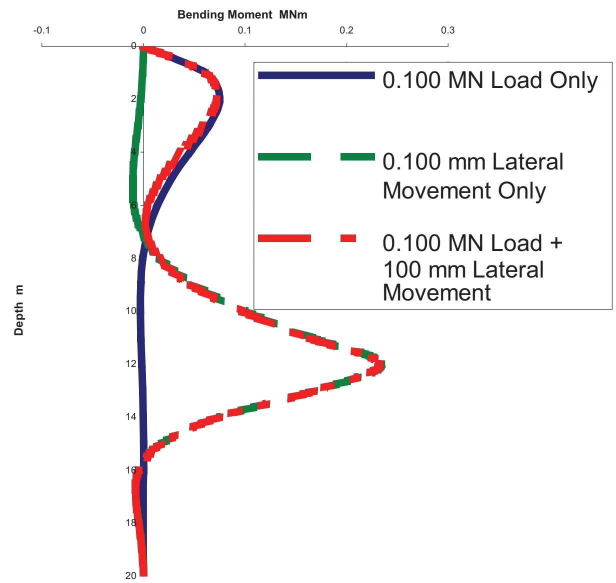
[Fig. 2] Typical Problem Analyzed

Fig. 3 shows the computed axial load distributions for the applied load acting alone, the vertical ground movement acting alone, and the applied load and the ground movement acting together. It can be seen that the distributions of axial load in the pile due to applied loading are very different from those induced by the ground movements. In the latter case, the maximum axial load occurs near the bottom of the upper soil layer which is subjected to movement. It can also be seen that the addition of the two profiles of axial load gives axial loads which are less than those computed for the combined loading and ground movement case.



[Fig. 3] Comparison of Axial Responses

Fig. 4 shows the corresponding distributions of moment computed for the lateral response of the pile. Again, it can be seen that the distribution of induced bending moment is very different for applied loading and for lateral ground movement. In the latter case, the maximum moment occurs well below the pile head, near the bottom of the zone of ground movement. The maximum bending moment under the combined loadings is also at the latter location, since the moment due to the applied loading is virtually zero where the moment due to the lateral ground movements is largest.



[Fig. 4] Comparison of Lateral Responses

In terms of practical pile design, the above example demonstrates the following important points:

- The effects of ground movements can not be simulated accurately by the application of a load to the pile head;
- The superposition of axial load distributions due to axial applied loading and vertical ground movements may underestimate the maximum axial load in the pile;
- The maximum load in a pile subjected to lateral ground movements may occur well below the pile head. In this particular case, having the pile reinforced only to resist applied lateral loading (for example, in the upper 6 m or so) will be inadequate to resist the ground-movement induced moments. The pile may well fail structurally at a considerable depth below the pile head.

Thus, it is important to consider the possibility of ground movements in pile design, and to allow for reinforcement to resist deep-seated moments that may be induced by these movements.

Generic Design Charts For Piles Subjected To Soil Movements

Vertical Soil Movements

Design charts for the settlement and maximum axial force in a single end bearing pile on rock and subjected to vertical soil movements, have been published by Poulos and Davis (1980), While similar charts for a single floating or friction pile have been presented by (Poulos, 1989; Poulos and Davis, 1980). Such charts will tend to give upper bound values of both pile settlement and induced pile force, because there is no limit to the pile-soil shear stress that is developed between the pile and the soil. In reality, the existence of an ultimate skin friction will result in a limit to the axial force and pile movement that can be generated within a pile. The use of elastic solutions therefore tends to be conservative when applied to practical cases. Corrections for pile-soil slip and other practical effects are presented by Poulos and Davis (1980) and Nelson and Miller (1992).

Lateral Soil Movements

If the distribution with depth of free-field lateral movements can be simplified, it is possible to develop useful design charts to enable approximate assessment of the pile head deflection and the maximum bending moment in the pile. Chen and Poulos (1997, 1999) have presented such charts, both for a pile in soil subjected to a uniform movement with depth (to a depth zs below the surface), and for a soil in which the horizontal movement decreases linearly with depth, from a maximum at the surface to zero at a depth zs. These solutions assume that the soil remains elastic, and they therefore generally give an upper bound estimate of the pile moment and deflection. The extent of the possible over-estimation increases with increasing lateral soil movements, due to the progressive departure from elastic conditions which results from the development of plastic flow of the soil past the pile.

From a study of several examples, Chen and Poulos (2001) have suggested the following preliminary guidelines for the determination of soil movements in making theoretical predictions via the generic design charts :

1) For unstrutted excavations or relatively small slope movements, a linear soil movement profile, with a maximum value at the ground surface and zero at a certain depth below the surface, may be adopted. The maximum value may be estimated from measured ground surface movements or via appropriate empirical approximations which relate movement to the height of the retained soil, for example, Peck (1969).
2) For landslides involving relatively large soil movements (for example, up to about 0.4 pile diameters), a uniform soil movement profile may be adopted.

The above study by Chen and Poulos also shows that the elastic design charts can give reasonably good estimations of the lateral pile response, provided that the ground movements are not very large, for example, less than about 30-40% of the pile diameter.

Piles Near Embankments

Introduction

The construction of embankments on clay can result in the development of substantial immediate and time-dependent vertical and horizontal movements of the soil beneath and adjacent to the embankment. In situations where bridge abutments adjacent to such embankments are supported on piles, these piles may experience significant axial and lateral loads which are induced by the soil movements. The design of abutment piles therefore requires a proper consideration of the pile response to the externally imposed soil movements, including an assessment of the consequent bending moment and lateral deflection profiles of the piles.

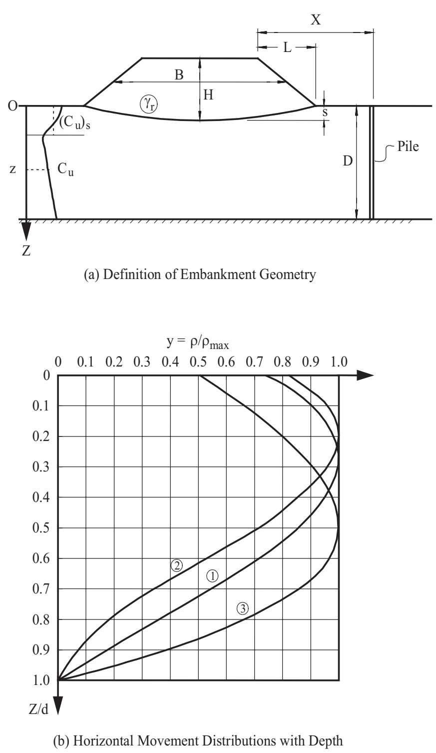
Marche and Lacroix (1972), Heyman (1965), and Heyman and Boersma (1965) have presented field data of the lateral pile response to embankment-induced soil movements. Poulos and Davis (1980) have compared the results of a boundary element analysis with this field data, and have found fair agreement. Stewart et al (1991, 1992) have conducted centrifuge model tests of piles bridge abutments and have developed an empirical procedure for estimating the lateral deflection and bending moment in a pile or pile group. Poulos (1996) has compared various methods of analyzing lateral pile behaviour, including some design methods based on estimation of pressures developed between the pile and the soil. Such methods appear to be generally less reliable than methods based on a proper pile-soil interaction analysis.

Estimation of Ground Movements

Settlements beneath and near embankments can, in principle, be predicted by conventional methods of settlement analysis, whether they are based on one-dimensional analysis, elastic-based analysis, or numerical analyses, such as the finite element method or the program FLAC. An important requirement for such settlement predictions is to make allowances for non-linear soil behaviour, including differences in compressibility in the normally- and over-consolidated states, local yielding or failure within highly-stressed zones, and the effects of consolidation and creep.

In contrast, horizontal movements below and near embankments are difficult to predict accurately (e.g. Poulos, 1972) and it is often more appropriate to make use of empirical information on such movements. Bourges and Mieussens (1979) have evaluated the results of a number of field observations and recommended a practical design approach which is summarized here. They have found (see Fig. 5a) that the significant parameters of the horizontal displacement profile are:

- The ratio of layer depth (D) to mean embankment width (B);
- The ratio of distance (X) of a point from the crest, to the horizontal extent of the slope (L_w);
- The undrained shear strength s_u;



[Fig. 5] Pile Near Embankment and Soil Movement Distributions

- The ratio of undrained shear strength of surface crust and underlying clay ($M=s_{us}/s_u$).

They identified three general distributions of horizontal displacement, which are illustrated in Fig. 5b. These distributions apply at or near the toe of an embankment, and can be expressed by the following equations:

- Curve 1 (Overall mean curve):

$$Y = 1.83Z^3 - 4.69Z^2 + 2.13\,Z + 0.73 \quad (1a)$$

- Curve 2 (where the compressible layer lies several metres below the surface):

$$Y = -2Z^3 + 1.5Z + 0.5 \quad (1b)$$

- Curve 3 (where the soil compressibility is reasonably uniform with depth):

$$Y = 3.42Z^3 - 6.37Z^2 + 2.14Z + 0.81 \quad (1c)$$

where Y = dimensionless horizontal displacement = ρ_h / ρ_{hmax}

ρ_h = horizontal displacement

ρ_{hmax} = maximum horizontal displacement

Z = z/D = dimensionless depth.

The magnitude of the maximum horizontal displacement, ρ_{hmax}, is given as the sum of the immediate and consolidation components, ρ_i and ρ_c respectively. ρ_i is related to the position of the point (X/L) and the safety level F, where:

$$F = 5.14 s_{uav} / \gamma H \quad (2)$$

where s_{uav} = average undrained shear strength in the clay layer

γ = unit weight of embankment fill

H = embankment height.

From measured data, Fig. 6 plots the dimensionless maximum immediate horizontal displacement (λ, where $\lambda = \rho_{imax} / D$) against F for three values of X/L_w. As would be expected, λ increases as F decreases, and becomes relatively large as failure is approached (i.e. F approaches 1.0).

The maximum consolidation lateral displacement at any time, $\rho_{cmax,t}$, is correlated with the consolidation settlement Sc of the centre of the embankment, as follows:

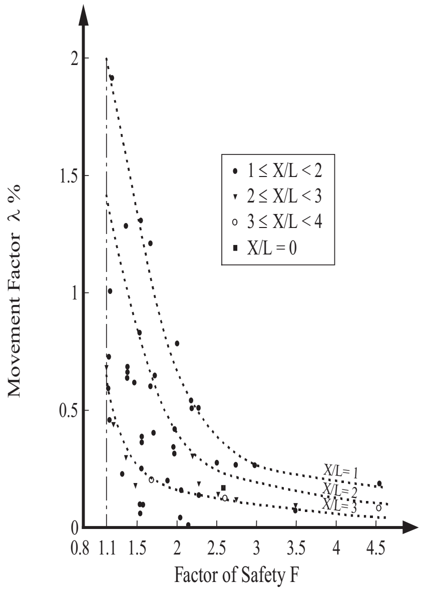
$$\rho cmax,t = 0.16\,S_c = 0.16\,(S_t - S_i) \quad (3)$$

where S_t = total settlement at time t

S_i = immediate settlement.

Thus, at any time t, the maximum horizontal displacement ρ_{hmax} can be approximated as:

$$\rho_{hmax} = \lambda.D + 0.16(S_t - S_i) \quad (4)$$



[Fig. 6] Dimensionless Immediate Horizontal Movement Factor I (Bourges & Mieussens, 1979)

Some Characteristics of Pile Behaviour

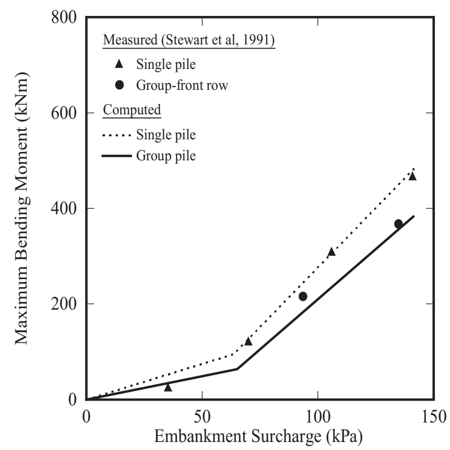
Poulos (1994) has undertaken a study of the behaviour of piles near and through embankments on clay, using a two-stage procedure:

1. assessment of the horizontal soil movements due to the embankment, using the empirical approaches developed by Bourges and Mieussens (1979), described above;
2. analysis of the response of piles to these movement, using the program ERCAP to compute lateral response and PIES for axial response.

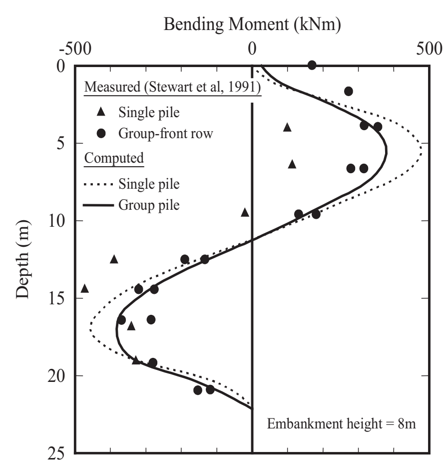
Comparisons were made between the analysis results and the results of centrifuge tests reported by Stewart et al (1992). Fig. 7 compares the relationship between maximum horizontal movement of a pile and embankment height. The centrifuge tests show that, beyond a certain embankment height (which corresponds to an average pressure of about $3s_{uav}$), the rate of pile displacement with increasing embankment height accelerates. This behaviour is reproduced quite well by the theoretical analyses. Fig. 8 compares the measured and computed distributions of bending moment along a pile, and shows fair agreement in both the magnitude and distribution of moment.

A study of the importance of a number of parameters was also made by Poulos (1994), and the factors which have the most significant influence on pile response were found to be: pile position – relative to the embankment toe, undrained shear strength of the clay, thickness of clay layer, embankment height, pile size, and delayed installation of the pile. The latter is particularly important, and it is found that, if the installation of the pile can be delayed until after embankment construction is completed, the maximum bending moments in the pile can be reduced to 10-15% of the values which would otherwise occur.

As mentioned previously, group effects are generally beneficial when piles are subjected to soil movements, leading to a "shielding" effect and reduced moments and shear forces in the piles, and it is therefore generally conservative to consider a single pile for design purposes.



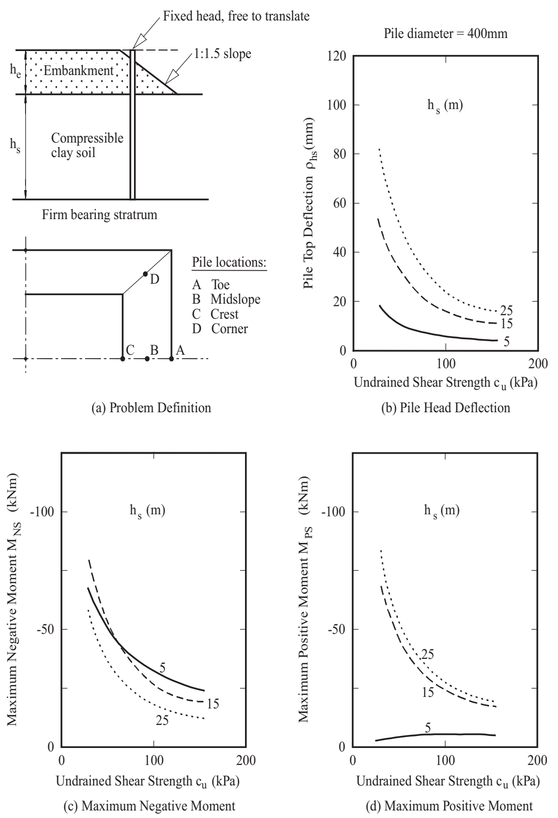
[Fig. 7] Comparison Between Measured and Theoretical Maximum Moments – Deep Clay Layer



[Fig. 8] Comparison Between Measured and Computed Moment Distributions – Deep Clay Layer

Typical Design Charts

For the idealized problem shown in Fig. 9(a), a series of design charts has been developed for a range of values of embankment height, soil layer thickness, soil strength and pile size. For a typical case of a vertical pile located at the crest of the embankment, Fig. 9(b) to 9(d) shows the maximum positive and negative moments and the pile head deflection for a 400 mm square precast concrete pile, for an embankment height of 8 m. It is assumed that the pile head is restrained from rotation and that the pile is installed after construction of the embankment has been completed. Such charts have the potential to provide a more rational design approach for piles within embankments. Successful use of such charts demands judicious selection of the necessary geotechnical parameters and appropriate modelling of the real problem to reduce it to the idealised cases for which the design charts are derived.



[Fig. 9] Design Charts for Pile at Embankment Crest – 4 m Embankment Height

Application to Case Study

De Beer and Wallays (1972) reported a field test in Belgium that aimed to study the influence of embankment construction on adjacent pile foundations. Measured results were presented for a steel pipe pile and a reinforced concrete pile. The steel pipe pile was 28 m in length, 0.9 m in diameter, and 1.5 cm in wall thickness, while the reinforced concrete pile was 23.2 m in length and 0.6 m in diameter. The pile heads were restrained from lateral displacement. The soil deposit consisted mainly of sand, with a Young's modulus E_s of about 30 MPa and the limiting soil pressure of about $2p_p$ (where p_p is the Rankine passive pressure) (see Chen & Poulos, 1997). The measured free-field lateral soil movements, shown in Fig. 10(a), generally decrease with depth, after reaching a maximum at a relatively shallow elevation.

Chen & Poulos (1997) have shown that a full analysis via the computer program PALLAS can give estimations of pile bending moments and deflections very close to those measured, using the measured soil movement profile shown in Fig. 10(a). The pile bending moment and deflection profiles estimated using PALLAS are shown in Fig. 11, together with those measured, and a fairly good agreement between the estimated and the measured values can be observed.

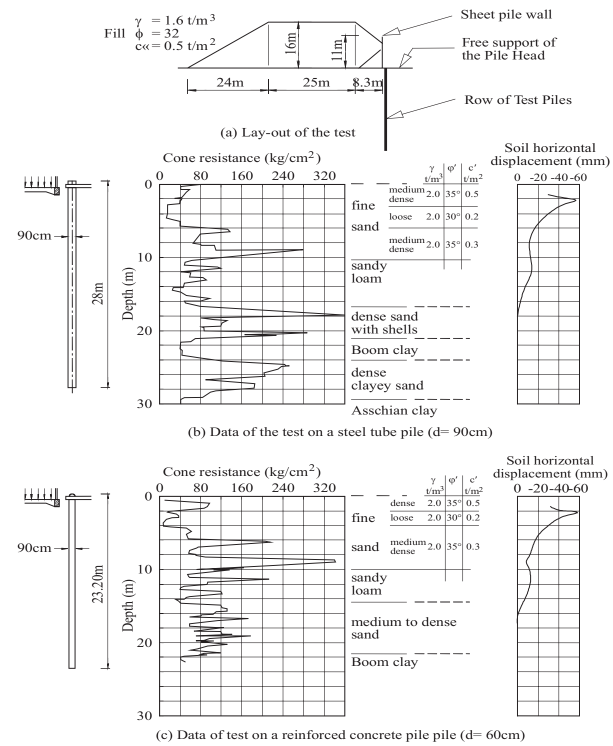
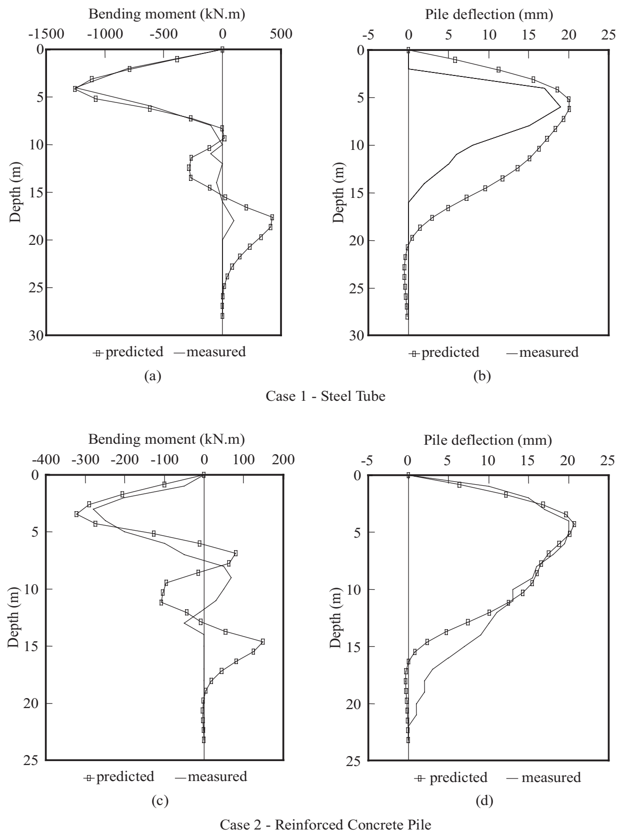


Fig. 10 Test at Zelzate (De Beer & Wallays, 1972)

The soil movement profile has also been simplified to a linear profile for two cases, to assess whether simplified soil movement estimates can give adequate predictions of pile response. One case has a surface displacement (s_0) of 20 mm, while the other has a s_o value of 40 mm, with both cases having a zero value occurring at a depth of about 18 m. It has been found that the measured profiles are encompassed by those estimated for the above two s_o values.



[Fig. 11] Calculated and Measured Pile Responses

Piles Near a Pile Cap Excavation

Introduction

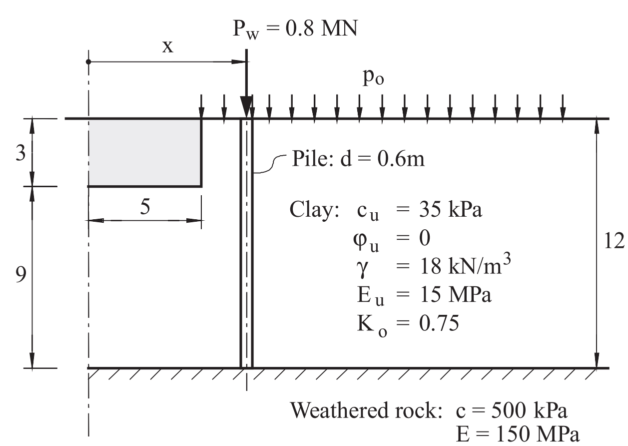
When an excavation is carried out for a new pile cap in the vicinity of existing piles, there is often little or no support provided for the excavation, since pile cap thicknesses are typically 1-3 m and the excavation is therefore relatively shallow. However, under conditions in which the ground is highly stressed (for example, within the footprint of an existing building), even such modest excavations deserve careful consideration as ground movements will inevitably be generated by the excavation process. In addition, it is possible that dewatering may also be necessary, in which case additional ground movements (both vertical and lateral) will be generated by the process of groundwater lowering.

Ground Movements

It is now common for the ground movements around excavations to be estimated via detailed numerical analyses such as the finite element method. When numerical analyses cannot be carried out, it is possible to use approximations developed by Clough and his co-workers to estimate vertical and horizontal distributions of ground movements. The distributions of movement with depth are difficult to estimate without some form of analysis, as they depend on wall flexibility and excavation support conditions, but it may sometimes be adequate to assume a linear distribution with depth (Chen and Poulos, 2001).

Common design practice employs two-dimensional analyses, and near the centre of an excavation, two dimensional analyses can give reasonable soil movement estimates (for example, Yong et al, 1996). Thus, in the following examples, a two-dimensional analysis, employing the computer program FLAC, has been used to estimate the ground movements due to excavation for a pile cap.

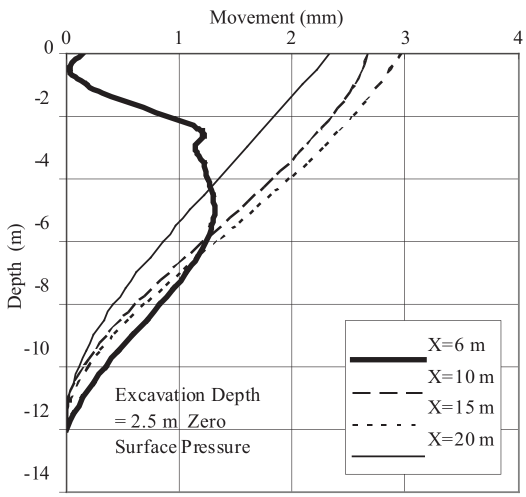
The case examined is shown in Fig. 12, and involves an excavation in medium-soft clay for a 3 m deep pile cap, 10 m in width, with no lateral support provided for the excavation. Figs. 13 a-d show typical distributions of the vertical and lateral movements with depth, at various distances from the excavation. Two different values of the surface pressure are considered, 0 kPa (a “green-field” situation) and 50 kPa, a typical situation that may arise beneath an existing building. It can be seen that, as would be expected, the movements for the 50 kPa surface pressure are considerably larger than those for zero pressure, and that the movements tend to decrease with increasing distance from the excavation. It is further assumed that the excavation is carried out relatively rapidly, and that no drop in the level of the water table arises from the excavation.



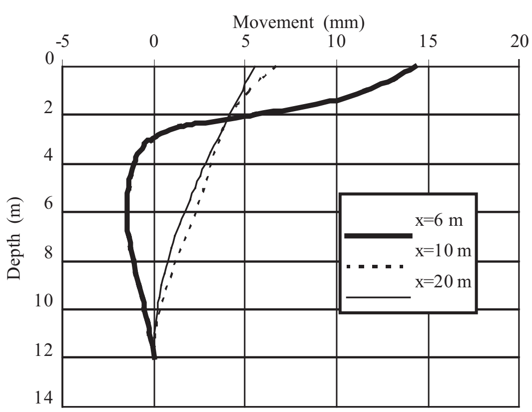
[Fig. 12] Pile Cap Excavation near Existing Pile

Pile Response to Ground Movements

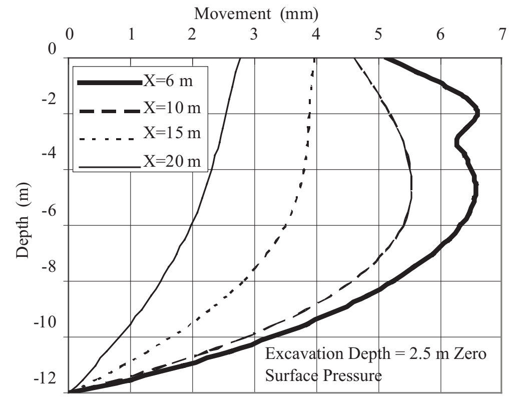
For the case as shown in Fig. 12, Figs. 14 and 15 summarize the computed maximum bending moment and shear in an adjacent pile, as a function of the distance from the excavation and the surface pressure. It will be seen that the induced maximum bending moment is very large when the pile is close to the excavation. Indeed, for a 0.6 m diameter reinforced concrete pile with 1% reinforcement, carrying a working axial load of 800 kN (corresponding to a factor of safety of about 2), the maximum design moment capacity is about 0.56 kNm. Thus, Fig. 14 implies that piles within about 10 m of the axis of the excavation could have induced moments that exceed the design capacity of the pile, if the surface pressure is 50 kPa.



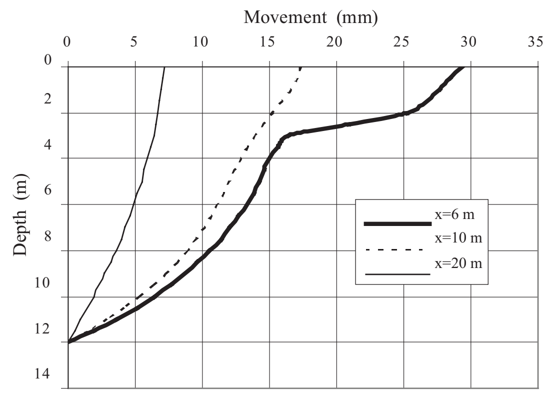
[Fig. 13(a)] Computed Vertical Ground Movements Due to Cap Excavation – p_0 =0



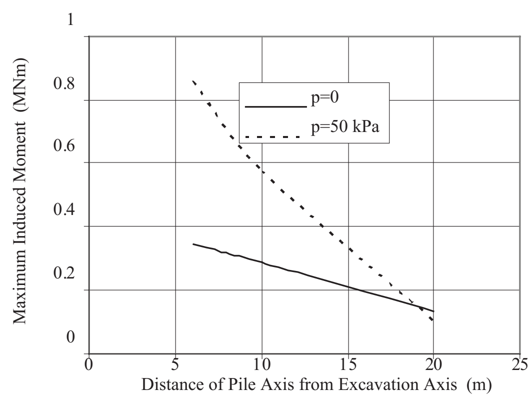
[Fig. 13(b)] Computed Vertical Ground Movements Due to Cap Excavation – p_0 =50 kPa



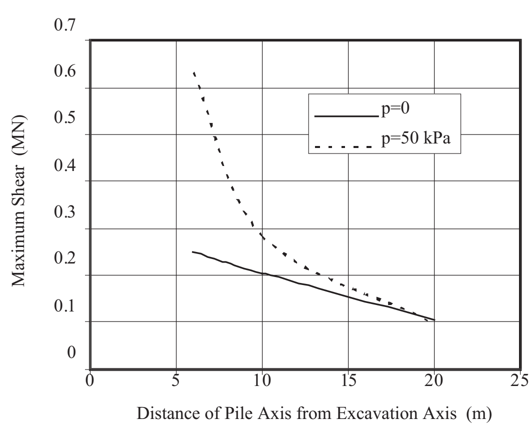
[Fig. 13(c)] Computed Horizontal Ground Movements Due to Cap Excavation – p_0 =0



[Fig. 13(d)] Computed Horizontal Ground Movements Due to Cap Excavation – p_0 = 50 kPa



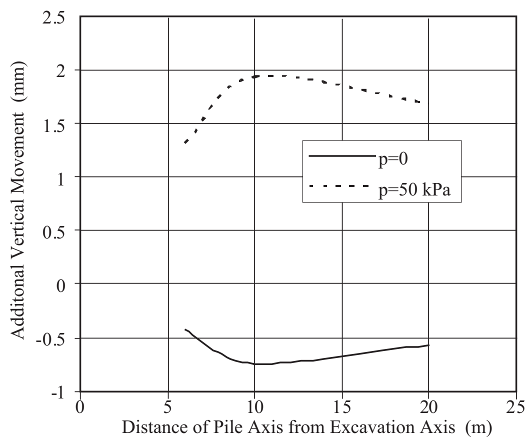
[Fig. 14] Computed Pile Moment Due to Cap Excavation



[Fig. 15] Computed Pile Shear Due to Cap Excavation

Fig. 16 summarizes the computed additional movement of an existing pile adjacent to the excavation. In this case, if there is zero surface pressure, the adjacent pile tends to move upwards slightly because of the excavation, but it settles if the surface pressure is 50 kPa. In the latter case, the additional axial force induced in the pile by the vertical ground settlement is small, even if the pile is relatively close to the excavation.

Thus, it would appear that the issue that may cause most concern is the induced bending moment and shear in the pile due to the lateral ground movements.



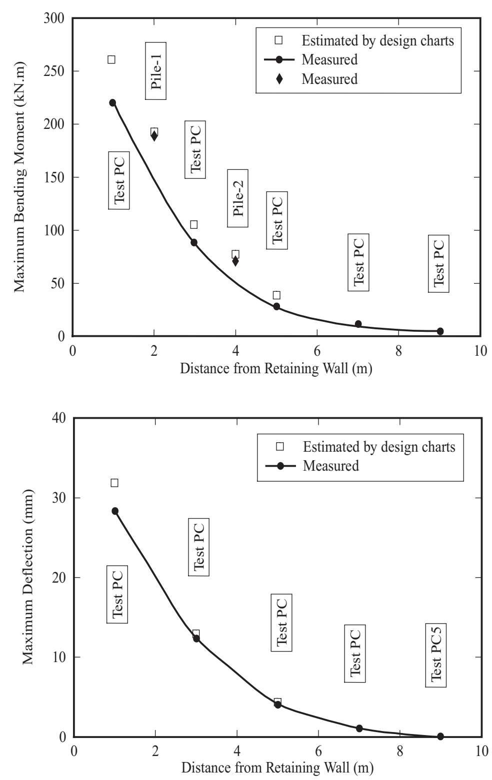
[Fig. 16] Computed Vertical Movement of Pile due to Cap Exacavation

Comparisons Between Computed and Measured Pile Response in Centrifuge Tests

There do not appear to be any measurements of pile response to cap excavation-induced ground movements. However, Leung et al (2000) have presented results from centrifuge model tests on a single pile adjacent to unstrutted deep excavations in dense sand. The model pile was fabricated from a hollow square aluminum tube and instrumented with 10 pairs of strain gauges protected by a thin layer of epoxy. The model pile simulated a prototype concrete bored pile of 0.63 m in diameter, and 12.5 m in total embedded length. The retaining wall supporting the excavation had an embedment depth of 8 m. The Young's modulus of the sand, E_s, was estimated to increase linearly with depth, z (in metres), and expressed approximately as $E_s = N_h z = 6z$ MPa.

Several tests were carried out in which the pile was located at different distances from the retaining wall. The free-field soil movements, pile bending moments and deflections were measured for different depths of excavation. The measured free-field soil movements at different distances from the wall and corresponding to an excavation depth of 4.5 m were found to decrease almost linearly with depth, from a maximum value at the surface.

The generic elastic design charts (Chen and Poulos, 1997) were used to calculate the pile response, with the soil movements simplified to linear profiles. The estimated results are shown in Fig. 17, together with those measured. It can be seen that, in this case, the elastic generic design charts using the simplified soil movements generally give fairly good estimates of the pile response.



[Fig. 17] Measured and Computed Pile Moments due to Excavation

Response of Piles to Seismically-induced Ground Motions

Introduction

Many pile failures during earthquakes have occurred due to the inadequacy of the pile to withstand large induced moments and shears (Mizuno 1987, 1996). The pile designer (who often is not the person who performs structural dynamic analysis) needs to know approximately what the maximum seismically-induced internal moment and shear of the pile would be. This problem is similar in principle to the problems of statically-induced ground movements discussed above, but there are additional complexities that must be recognized. In particular, there are two sources of loading of the pile by the ground movements:

1. "Inertial" loading at the pile head, caused by the lateral forces imposed on the structure by the earthquake and which are then imposed on the piles;
2. "Kinematic" loading along the length of the pile, caused by the lateral (and to a lesser extent, the vertical) ground movements developed by the earthquake.

Traditionally, many foundation designers have considered only the effects of inertial loading, but kinematic loading can also be very important.

A relatively simple pseudo-static approach for analyzing this problem, taking into account both inertial and kinematic effects, has been developed by Tabesh and Poulos (2001) and is described below.

Pseudo-Static Approach

The pseudo-static methodology developed by Tabesh and Poulos (2001) involves the following steps:

1. The superstructure is modelled by a single degree of freedom system whose natural frequency is equal to the fundamental frequency of the superstructure. The simplest way is to reduce the superstructure to a cap-mass. The designer should be careful about such a crude approximation; the mere eccentricity of the superstructure mass may have a profound effect on the response.
2. The natural lateral period T of the pile head is estimated through published formulae and charts, or in the case of a pile-cap-mass system, via the approximate relationship:

   $$T = 2\pi \, (\text{Cap Mass}/\, K_x)^{0.5} \qquad (5)$$

   where K_x = lateral pile head stiffness.
3. A free-field site response analysis is performed to obtain both the time history of the motion at the surface and the maximum displacement of the soil mass along the pile. In this analysis the well-known SHAKE (Schnabel et al (1972)) program, or similar computer codes such as the ERLS program used herein, may be used. As the moment and shear depend on the curvature of the pile, the points whose maximum displacements are to be obtained must be closely spaced, especially near the surface.
4. The maximum values of the displacements along the pile obtained in step 3 are treated as a static soil movement profile, although the displacement at each point may have occurred at different times.
5. The surface motion obtained in step 3 is used in an ordinary spectral analysis of a single degree of freedom system whose period is equal to the period obtained in step 2. The spectral acceleration is calculated.
6. The lateral force to be applied to the pile is obtained from multiplication of the spectral acceleration obtained in step 5 and the cap-mass or the mass of the single degree of freedom model of the superstructure calculated in step one.
7. A static analysis, in which the pile is subjected to the simultaneous application of a lateral force at its head equal to the force obtained in step 6, and a soil movement profile formed in step 4, is performed and the maximum pile moment and shear are obtained.

Verification of The Pseudo-Static Method

In order to examine the performance of the proposed pseudo-static methodology, Tabesh and Poulos (2001) considered a soil mass consisting of two layers. Various ratios of layer stiffness were considered, and a range of pile diameters was analysed. The Newcastle 1994 earthquake was used as the excitation source. Eighty different pile-soil configurations were considered for which the envelopes of the positive and negative moment and shear along the pile were obtained via a more complete dynamic analysis, and the shear and moment distributions along the pile were also calculated from the proposed pseudo-static analysis with the maximum computed free-field soil movements as input. The cap-mass (and hence the vertical applied load) was assumed to be zero. Without any exception, excellent agreement was obtained between the dynamic analysis and the pseudo-static methodology.

These comparisons suggest that, regardless of the soil non-homogeneity, the static methodology gives very good results for the maximum values of the moment and shear along the pile. Thus, when the response of the pile is dictated by the free-field ground movements, the internal response of the pile can be easily estimated by a very simple static analysis. When the effects of cap-mass were taken into account, it was observed that, while in many cases the agreement between the proposed pseudo-static method and dynamic analysis was close, while in some others the pseudo-static approach overestimated the maximum moment and shear.

Significance of Inertial and Kinematic Effects

The influence of inertial effects (via vertical loading and/or cap-mass) on the seismic response of pile foundations depends on the frequency content of the earthquake and the natural period of the pile-soil-cap-mass system. Mylonakis et al (1997) have identified the following characteristics:

1. Inertial bending can be significant, especially in the upper part of the piles, when the dominant period of the earthquake is similar to the fundamental period of the soil-pile-structure system.
2. Kinematic bending can be significant when the dominant period of the soil motions are similar to the natural period of the soil strata.
3. The three most likely areas of damage of a pile are the pile head, interfaces between layers of different stiffness, and the pile toe. Pile head damage is most likely in homogeneous strata while damage at strata interfaces is most likely when there is a marked stiffness contrast between the layers. The kinematic bending strains at the pile toe may be significant when the toe is restrained.

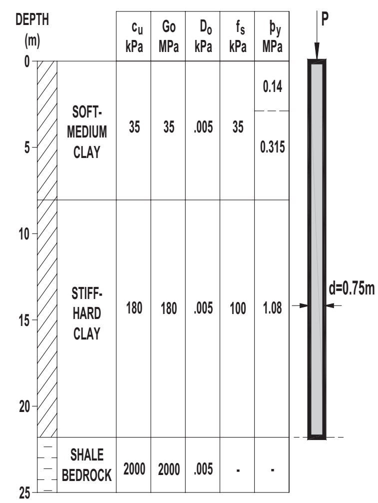
To facilitate an understanding of the relative importance of inertial and kinematic effects, analyses have been performed on the fixed head single pile shown in Fig. 18. The analysis has been carried out via the pseudo-static approach described above, so that the results provide an envelope of maximum bending moments and shears along the pile. It is assumed that the site is subjected to the 1994 Northridge earthquake with a maximum bedrock acceleration of 0.2 g. Three cases have been considered:

- A pile with no vertical load/cap mass;
- A pile with a lateral inertial load of 0.2 MN
- A pile with the same lateral inertial load as in the second case, but where the kinematic ground movements are not included in the analysis.

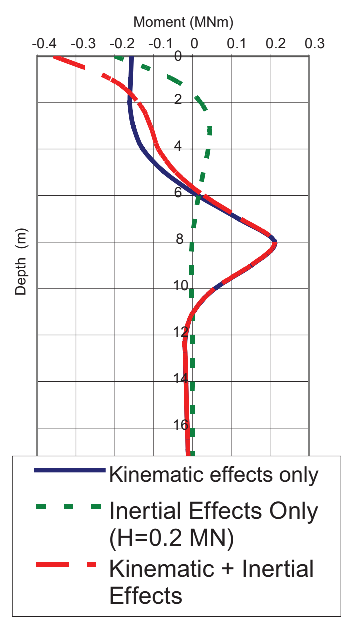
Fig. 19 shows the computed distributions of bending moment along the pile. Two key points emerge from this figure:

1. If kinematic effects are ignored, and only inertial (lateral load) effects are considered, the maximum moment at the pile head can be seriously under-estimated.
2. If only inertial effects are considered, the moment at depths in excess of about 7m becomes insignificant, but with the kinematic effects incorporated, there is a significant moment between depths of about 7 to 10m, i.e. in the vicinity of the interface between the softer upper layer and the stronger lower layer.

The importance of considering both kinematic as well as inertial effects is clearly emphasized in this example.



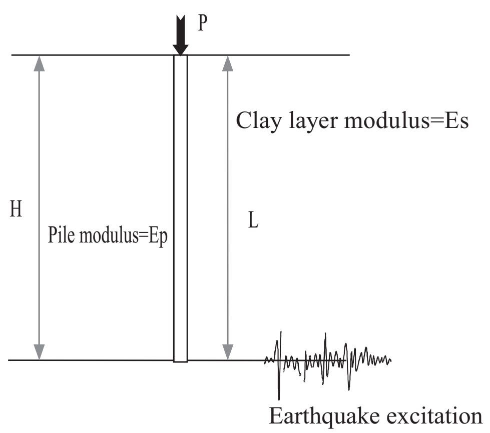
[Fig. 18] Example Analyzed



[Fig. 19] Effects of Kinematic and Inertial Loading of Pile on Moment Distribution

Design Charts

Tabesh and Poulos (2006) have attempted to provide a simple means of making preliminary estimates of maximum bending moment and shear in single piles embedded in homogeneous clay layers and subjected to seismic excitation (Fig. 20). To develop these design charts, a time domain method has been employed in which the earthquake motion has been input, and the moment, shear and relative displacement of the pile obtained at all time steps during earthquake. The maximum values at any time during the earthquake have then been extracted and used for the design charts.



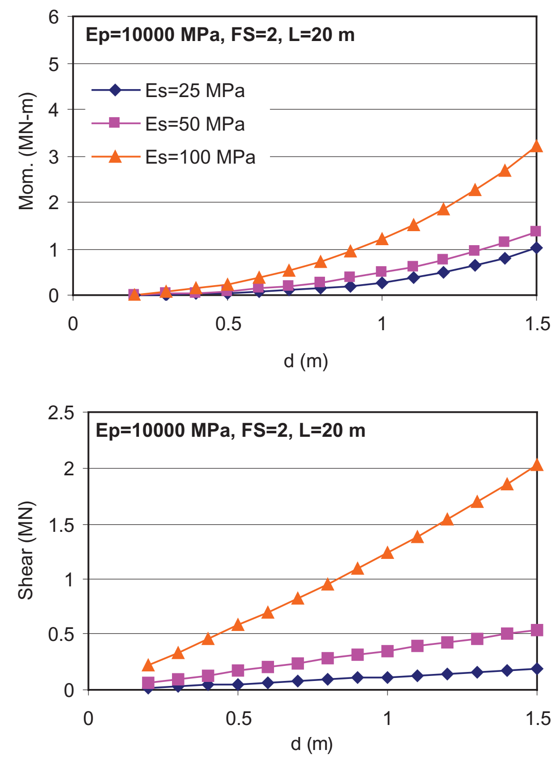
[Fig. 20] Idealized Case for Design Charts

Eight earthquakes were selected to develop the design charts, covering a wide range of prominent earthquake frequencies. The piles were assumed to have a fixed head (i.e. zero head rotation), and to be embedded in homogeneous clay layers with Young's modulus values of 25, 50 and 100 MPa. The pile diameters covered the range of values between 0.2 to 1.5 metres in 0.1 m increments. Two pile modulus values, 10000 MPa and 30000 MPa were considered, being representative of timber and concrete respectively.

Fig. 21 shows typical design charts for the maximum moment and shear developed in a 20 m long pile with an axial load equivalent to a factor of safety of 2. As would be expected, both the moment and shear increase with increasing pile diameter d.

Case Study

The pseudo-static methodology is used to estimate the maximum moment developed in the Ohba-Ohashi bridge in Japan. This bridge is located in Fujisawa city, Kanagawa prefecture,

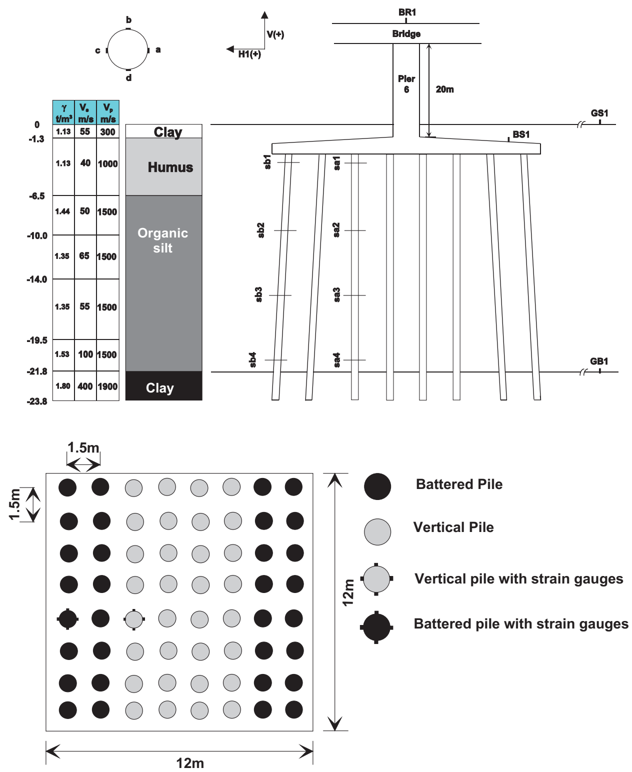


[Fig. 21] Typical Design Charts for Maximum Moment and Shear Induced in a Pile.

near Tokyo. The seismic observations at this bridge and one of its pile foundations were conducted between 1981 and 1985 by Shimizu Corporation. During this period, 14 earthquakes hit the region. Among them, the twelfth earthquake induced the largest peak horizontal surface acceleration which was 0.11 g.

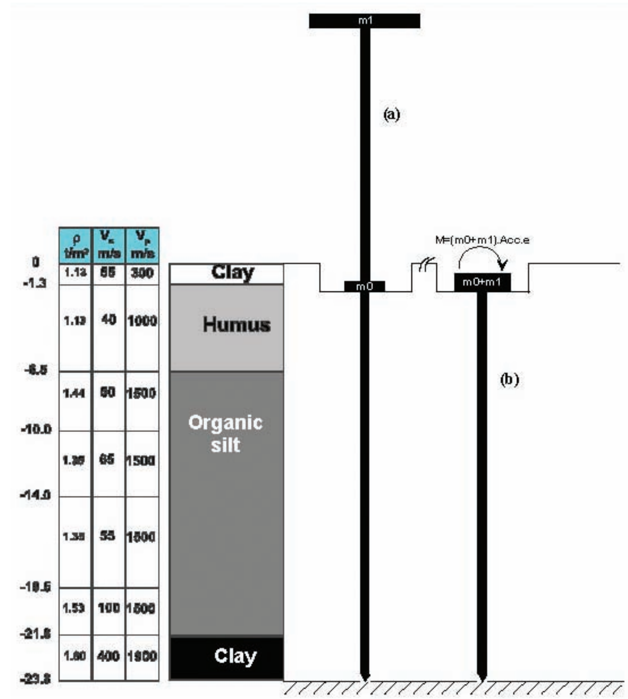
The bridge was supported by 11 piers and is 484.8 m long and 10.75 m wide. The girder is continuous from pier 5 to pier 8. Piers 5, 7, and 8 are equipped with movable bearings, and pier 6 is of the fixed-shoe type. Fig. 22 shows the details of the bridge and the pile foundation of pier 6 where the strain metres were installed. The soil profile shown in Fig. 22 was obtained from a borehole near pier 6. The top soil layers were extremely soft with SPT-N values being almost zero. The shear wave velocity of the top layers was between 40-100 m/s. The tips of the piles were embedded in a stiff layer of clay with a shear-wave velocity of 400 m/s. The length of the vertical piles was 22 m, with 2 m in the stiff clay.

The piles were steel pipes with a diameter of 0.6 m; the thickness of the vertical piles was 9 mm, and the wall thickness of the battered piles was 12 mm. A total of 32 wall strain metres were installed on one vertical and one battered piles at four different elevations (sa1-sa4, sb1-sb4 in Fig. 22). A total of 11 units of accelerometers of servo type were installed, with one unit (GS1) at the ground surface, four units (GB1-GB4) at the bearing substratum, three units (BS1-BS3) at the footings, two units (BR1 and BR3) at the piers and one unit (BR2) at the girder.



[Fig. 22] Ohba-Ohashi Bridge Pile Foundation.

For this analysis, the eighth earthquake was considered and the pseudo-static methodology was used to estimate the maximum moment developed in the vertical instrumented pile. Group effects were ignored. In the Ohba-Ohashi bridge measurements the free-field displacements along the length of the pile were not measured and only the base and surface motions were monitored. A free-field analysis was therefore required for which ERLS program was employed and the motion monitored at GB1 was used as the input motion. The soil was modelled as a system of 7 horizontal layers, and the mass of the superstructure was concentrated in two points, as shown in Fig. 23. The maximum value of the free-field response was obtained at 48 points corresponding to the centre of 48 pile elements. The spectral acceleration corresponding to the pile natural period and based on the surface motion was calculated to be 0.092 m/s2. All piles were assumed to carry equal loads. The pier was very stiff and was considered to be rigid. The eccentricity was calculated to be 16.3 m.

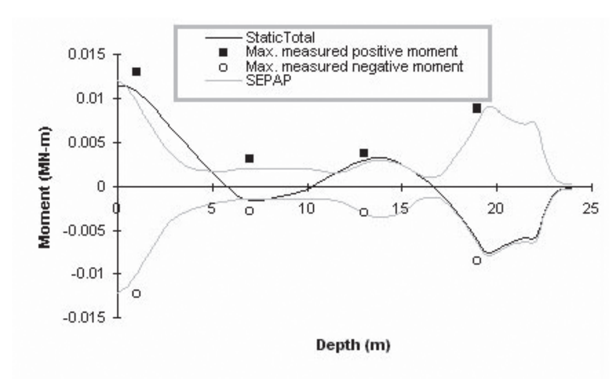


[Fig. 23] Pile Model used in Analysis

With these assumptions the pseudo-static approach was used to obtain the response of the pile to the combination of the following disturbances:

- a soil movement profile formed from the maximum free-field soil movement obtained at each pile element,
- a head force equal to F= m_s. A_{spec} in which ms = mass of superstructure and A_{spec} is the spectral acceleration obtained for a period equal to that of the pile and based on the surface motion,
- a moment equal to M=F.e in which e is the eccentricity.

The profile of the moment along the pile obtained from the pseudo-static method, along with the maximum moments measured at 4 locations along the pile are shown in Fig. 24.



[Fig. 24] Comparison of Calculated and Measured Moments Along a Vertical Pile in the Ohba-Ohashi Bridge Foundation.

In this figure the envelope of the positive and negative moment along the pile obtained from an independent dynamic analysis (SEPAP) is also shown (Tabesh and Poulos, 2001).

The proposed static methodology gives good estimates of the measured values, despite the fact that the Ohba-Ohashi bridge was very complicated and was over-simplified for the analysis. Similar results were obtained for several other measured earthquakes (Tabesh, 1997).

It is believed that the pseudo-static analysis can be used by practicing engineers to obtain a reasonable estimate of the maximum moment and shear which is likely to be developed in a pile when it is subjected to earthquake excitation. This analysis is likely to give good results in many practical cases, but it may overestimate the maximum moment and shear in certain other cases, especially when the period of the pile is close to the natural period of the soil mass in which case significant interaction may occur between the pile and soil. One reason is that the maximum free-field effects and maximum inertial effects have been assumed to act simultaneously, which does not occur in a dynamic analysis. More importantly, the assumption behind using spectral acceleration is that the cap-mass is excited by the surface motion. In reality, the cap-mass is excited not through the soil surface, but via the pile head whose motion is different from the surface motion. The pile head and surface motions are very close for a homogeneous soil mass, but when the soil is strongly layered, the pile head motion is often less severe.

Conclusions

There are several circumstances in which ground movements may influence significantly the behaviour of piles. This aspect of pile behaviour has often been overlooked or not recognized, often leading to excessive foundation deformations and possible structural damage of the foundation system. This paper has described a consistent procedure for analyzing the response of piles to ground movements, via simplified boundary element analyses. For such analyses, it is necessary to be able to predict the distribution of the 'free-field' ground movements, and then to use these in the analyses together with the pile-soil parameters which are required for the normal analysis of piles subjected to applied head loadings.

Three specific examples of piles subjected to ground movements have been considered:

1) piles near an embankment.
2) piles near a pile cap excavation.
3) piles in ground subjected to seismic action.

Some of the features of behaviour have been discussed, and some typical design charts have been presented to assist in the practical estimation of the forces, moments and displacements induced in the piles by ground movements.

Examples of comparisons between theoretical and measured behaviour are described, and these generally show a reasonable measure of agreement.

Attention has been concentrated on single piles, but it has been found that, in general, the consideration of a single pile is conservative from a design viewpoint, as interaction among piles in a group subjected to ground movements usually has a beneficial effect and reduces the induced deflections, forces and moments as compared with a single isolated pile. In addition, in contrast to the case of piles subjected to direct head loading, elastic analyses tend to give conservative estimates of pile responses to ground movements because there is no limit to the pile-soil pressures developed by the moving soil.

Acknowledgements

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