

LATERAL BEHAVIOR OF MICRO-PILE GROUPS UNDER STATIC AND DYNAMIC LOADS

Alper Turan, M. Hesham El Naggar, Ph.D. P. Eng., Sean Hinchberger, Ph.D. P. Eng.

Department of Civil and Environmental Engineering, The University of Western Ontario, London, ON, Canada, aturan@uwo.ca

RÉSUMÉ

Des micro-pieux sont largement répandus dans des secteurs séismiquement actifs, où elles peuvent avoir à résister aux charges latérales significatives. Par conséquent, leur comportement sous chargement latéral est d'intérêt. Les facteurs tels que l'arrêt d'enveloppe, la flexibilité de chapeau de pile et l'existence des surcharges sur la surface de sol peuvent avoir un effet significatif sur la réponse latérale d'un micro-pieu. Par conséquent, l'évaluation de ces facteurs et de leur impact sur la réponse latérale est d'intérêt aux chercheurs et aux praticiens dans ce domaine. Dans cet article, la réponse pseudo-statique et dynamique d'une base soutenue par un groupe de neuf micro-pieux incorporées dans un sol homogène uniforme est étudiée en utilisant un modèle en élément finis 3D. Le sol et les piles sont modélisés en utilisant des éléments du solide 3D. Les conditions limites du sol sont modélisées en utilisant les éléments infinis, aux frontières modèles. Le comportement linéaire visco-élastique et de élasto-plastique du sol avec l'atténuation de Rayleigh sont assumés pour l'analyse des cas dynamiques et pseudo-statiques, respectivement. Des états non linéaires d'interface ont été également étudiés, entre les piles et le sol, mais en employant des analyses pseudo-statiques seulement. On assume que le groupe de micro-pile est rigidement relié au chapeau de pile. Le mouvement du sol est modélisé via une ondelette de Ricker et est appliquée à la base du modèle d'éléments finis pour des analyses dynamiques. Pour les analyses pseudo-statiques, une charge harmonique latérale a été appliquée au chapeau de pile. La distribution des moments de flexion, des pressions d'interface sur les interfaces de sol-à-pile, des ouvertures d'espace et le glissement entre le sol et la pile ont été examinés en fonction de la profondeur de pile, là où applicable. Des recommandations pour l'exécution des bases de micro-pile dans des régions sismiquement actives sont également données basées sur les analyses.

ABSTRACT

Micro-piles are widely used in seismically active areas, where they may have to resist significant lateral loads. Therefore, their behaviour under lateral loading should be of interest. Factors such as casing termination, pile cap flexibility and the existence of surcharge loads on the soil surface may have a significant effect on the lateral response of a micropile supported foundation. Therefore, evaluation of these factors and their impact on the lateral response may be of interest to researchers and practitioners in this field. In this paper, the pseudo-static and dynamic response of a foundation supported by a group of nine micro-piles embedded in a uniform homogeneous soil is investigated using a 3D finite element model. The soil and piles are modelled using 3D solid elements. The far field soil continuum is modelled using infinite elements placed at the model boundaries. Linear visco-elastic and elastic-plastic behaviour of the soil with Rayleigh damping are assumed for analysis of dynamic and pseudo-static cases, respectively. Non-linear interface conditions were also investigated, between the piles and soil, but using pseudo-static analyses only. The micro-pile group is assumed to be rigidly connected to the pile cap. A Ricker wavelet is used as the ground input motion and it is applied at the base of the FE model for dynamic analyses. In pseudo-static analyses, a lateral harmonic load was applied to the pile cap. The distribution of bending moments, interface pressures on the soil-to-pile interfaces, gap openings and sliding between the soil and pile have been examined along the pile depth where applicable. Recommendations for implementation of micro-pile foundations in seismically active regions are also given based on the analyses.

KEYWORDS

Micropile, Plasticity, Dynamic Response, Casing Termination, Contact Non-linearity.

1. INTRODUCTION

Micro-piles are bored and grouted small diameter piles that are widely used in foundation rehabilitation projects. They are well suited for foundations with difficult access, restricted clearance and poor ground conditions where minimal disturbance to the existing structure is permissible (FHWA 1997). Micro-piles have been used for retrofitting and rehabilitating existing foundations due to their ease of installation (e.g. Taylor *et al.* 1998, Zelenko *et al.* 1998, and Misra *et al.* 1999). Micro-piles have also been used to increase the overall capacity and to reduce deflections of existing foundations subjected to compression and uplift forces (Mascardi 1982, Laefer 1999, Bruce *et al.* 1997 and IWM99 1999).

Micro-piles can be advantageous for construction in seismic areas, mainly due to their flexibility, ductility and capacity to withstand extension forces. Micro-piles are used to support the foundations of both new structures and existing structures, which have suffered seismic damage (see Pearlman *et al.*, 1993, Juran *et al.*, 2001 and Shahrour and Juran, 2004). Most studies on dynamic soil-micropile-foundation interaction have been numerical solutions. Kishishita *et al.* (2000) performed 2-D finite element simulations of micro-piles considering different input motions and pile types using a linear elastic model and various nonlinear models for the soil and pile. Shahrour *et al.* (2001) conducted a 3-D FEM analysis of a single micro-pile and a

micro-pile group supporting a superstructure assuming a square micro-pile cross-section and elastic material with Rayleigh damping. Ousta and Shahrour (2001) analysed a single micro-pile and group of micro-piles in saturated soils using a cyclic elastic-plastic constitutive model. Sadek and Shahrour (2004) investigated the influence of pile inclination on the seismic behaviour of a micro-pile group. Wong (2004) investigated the seismic behaviour of micro-piles using different levels of soil non-linearity, load intensities and frequency contents and pile inclinations. Lastly, the influence of pile head and tip connections on the dynamic response of micro-pile supported foundations was studied by, for example, Sadek and Shahrour (2006) using a 3D finite element scheme and considering both vertical and inclined pile.

A number of reduced scale and full-scale micro-pile tests have been reported in the literature. Yamane *et al.* (2000) conducted lateral and vertical load tests on various full scale micro-piles. Yang *et al.* (2000) tested a single reduced scale micro-pile installed in dry sand on a shaking table. Juran *et al.* (2001) tested a single reduced scale micro-pile, micro-pile groups, and micro-pile networks in the centrifuge. The lateral performance of micro-pile groups and micro-pile networks was assessed in the field by Geosystems, L.P. (2002). Each of the preceding studies has considered various micro-pile inclinations, pile numbers, and loads.

Although micro-piles have been investigated extensively, there are some factors that may affect the long-term performance of micro-piles and that warrant further study. First, micro-piles are commonly constructed with a casing in the upper part of the pile only to resist lateral loads. Turan *et al.* (2007) showed that, for a single micro-pile subject to lateral dynamic loading, bending moments in the pile increase significantly just below where the casing terminates (e.g. in the uncased portion of the pile). However, such effects have not been studied in micro-pile groups. Secondly, the influence of soil-pile interface non-linearity on the dynamic behaviour of conventional pile groups was studied by Lu *et al.* (2005), using an equivalent linear elastic model for the soil. These authors investigated the distribution of pile surface pressures, gap opening-closing and sliding behaviour between soil and pile using various input motions causing different levels of non-linearity. Third, Comodromos and Bareka (2005) investigated the negative skin friction effects on pile groups. They considered different values of surface surcharge in their analyses. Therefore, it was considered useful to study the influence of (i) an upper casing (ii) cap stiffness and (iii) surface surcharge on the lateral behaviour of a micropile group taking into account the non-linear behaviour of soil-pile interface in (ii) and (iii).

In this paper, a series of 3D time domain dynamic analyses and some pseudo-static analyses of micro-pile groups have been performed. A linear elastic material model has been used for the piles and pile cap along with constant Rayleigh damping. The preliminary results summarized below provide significant information about the response of the soil–micropile–foundation systems under lateral loadings. Verification of the finite element model, details of numerical model, influence of various aspects such as the existence of pile casing, pile cap flexural stiffness and soil surface

surcharge has been investigated considering soil-pile interface non-linearity. Finally, comments on the results and recommendations for researchers and practitioners in this field have been provided.

2. METHODOLOGY

2.1. Problem Geometry

The problem investigated in this study is the dynamic and lateral pseudo-static response of a group of micro-piles embedded in a 25m deep homogeneous soil deposit. The pile group comprises nine 15m long \times 0.25m diameter micro-piles that were rigidly connected to a 3m \times 3m \times 0.3 m thick reinforced concrete pile cap. A single degree of freedom system was connected to the top of the pile cap as shown in Fig. 1 and discussed below. Each micro-pile was assumed to comprise grout and a 0.09m diameter concentric steel reinforcement bar extending from the pile head to toe. In addition, the following two conditions were investigated: (i) the micro-piles were equipped with a steel casing from the ground surface to a depth of 5m, and (ii) the micro-piles were not equipped with a casing (e.g. they comprised only grout and a concentric reinforcement bar). These cases will be referred to as ‘with’ and ‘without’ casing for the remainder of this paper. A series of dynamic analyses were carried out to study the micro-pile behaviour with and without a casing. In addition, several pseudo-static analyses were carried out to investigate the influence of pile cap flexural rigidity (EI) and the existence of a surface surcharge. The schematic of the problem is shown in Figure 1.

2.2. Numerical Model and Boundary Conditions

To study the problem depicted in Fig. 1, the pile foundation and soil layer were modelled using mainly 8-noded linear hexahedron elements with three degrees of freedom per node (see Figure 2). In total, 5940 Hexahedral and 927 wedge elements and 315 infinite elements were used in the model. Since the higher frequency components of input motion are difficult to transmit if the element size is too large, the maximum element size used was between $1/6^{\text{th}}$ and $1/8^{\text{th}}$ of the minimum Rayleigh wavelength in accordance with Kramer (1996). Analyses were also performed using finer meshes (e.g. with 7182 and 11540 elements), which gave comparable results to the coarser mesh shown Fig. 2. The same mesh was used in both dynamic and pseudo-static analyses.

The initial step in each analysis consisted of a geostatic analysis, which was carried out to establish initial geostatic equilibrium. For the implicit dynamic analyses, the piles were assumed to be fully bonded to the soil (e.g. interface elements were not used) and thus separation on the soil-pile interfaces was not allowed. In addition, seismic loading was simulated by applying acceleration time histories at the base of the model. For pseudo-static analyses, a harmonic lateral load was applied to the pile cap. As noted below, pseudo-static analyses were used to investigate the influence of pile cap flexibility and surface surcharge on the lateral response of the pile group. In addition, interface elements were used

between the pile and soil permitting slip and separation in the pseudo-static calculations.

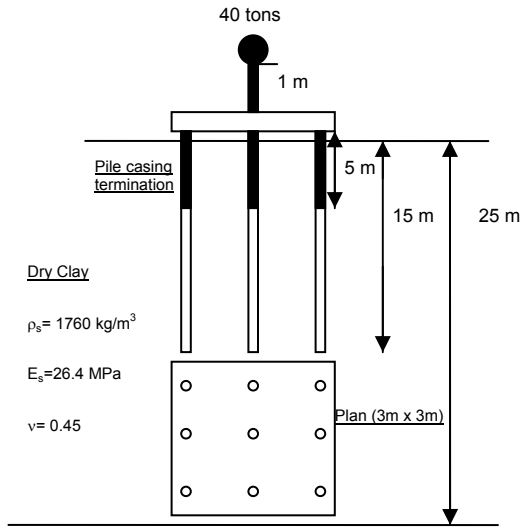


Figure 1. Schematic of problem geometry.

For all analyses, the lateral soil boundaries were represented using infinite elements (Lysmer and Kuhlemeyer, 1969), which are defined over a semi-infinite domain and were given a suitable decay function. The model base was constrained using rough-rigid boundary conditions. The infinite lateral boundaries were situated 100 m from the centre of the middle pile.

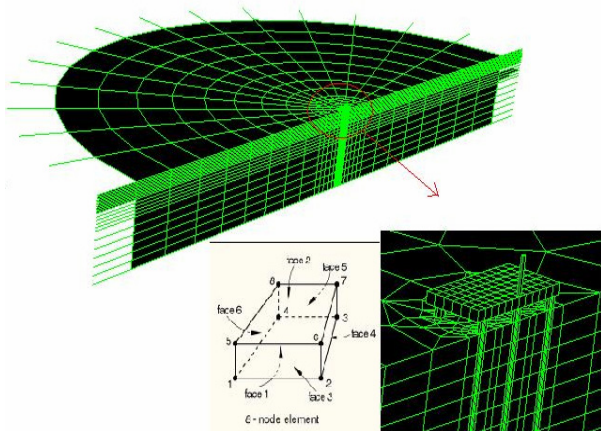


Figure 2. Model mesh composed of hexahedron and wedge element.

2.3. Verification of the Numerical Model

The FE procedures used in this study were tested using a verification problem comprising the monotonic lateral loading of a 52m long \times 1m diameter (d) pile embedded in a 70m thick soil deposit. Details of this case including the soil parameters used have been presented by Comodromos and

Pitilakis (2005). In the finite element model for the verification problem, the soil domain was extended to a distance of 50d from the pile. The problem was solved using interface elements between the pile and soil to permit separation and slip (if present) and the Mohr-Coulomb constitutive model was used for the soil. The mesh density was identical to that shown in Fig. 2. Figure 3 compares the FE calculations with the measured pile head response (see Comodromos and Pitilakis 2005). The good agreement between measured and calculated response is considered to provide some validation of the numerical approach taken in this paper.

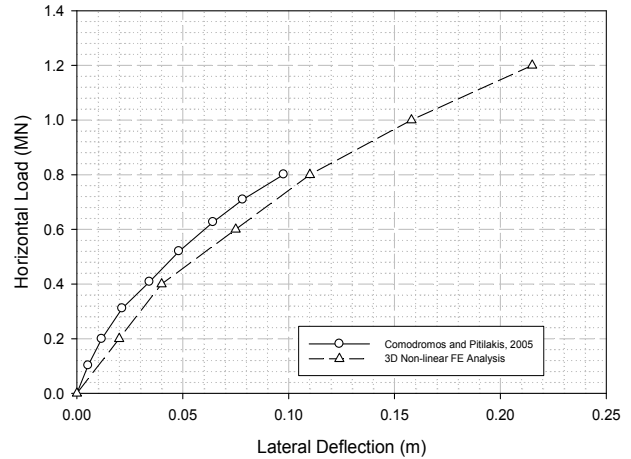


Figure 3. Load deflection curves from the present study and literature.

2.4. Numerical Analyses

Both dynamic and pseudo-static analyses were performed in this study as described in the following sections. For both types of analyses, the bending moments in the central pile (P5) were used as an indicator of the pile group response. Bending moments were calculated from the strains in pile elements viz.

$$M_i = \frac{\varepsilon_i EI_i}{c_i} \quad [1]$$

where, M_i is the pile bending moment at depth i of the pile, ε_i is the vertical strain at the extrados of the pile elements, E is the elastic modulus of the pile material, I_i is the moment of inertia of the pile cross-section and c_i is the distance from the neutral axis to the extrados.

2.4.1 Dynamic Analyses of Micro-pile Group

Dynamic analyses of the problem depicted in Fig. 1 was carried out to study the consequence of designing each micro-pile with a 5m long casing. Since input ground motions were specified at the base of the FE model, a

Table 1. Soil parameters used in analyses.

	Mass Density (kg/m ³)	Elastic Modulus (MPa)	Poisson's Ratio	Angle of Friction(°)	Flow Stress Ratio	Yield Stress (KPa)	Rayleigh Damping Ratio (%)
Dynamic Analyses	1760	26.4	0.45	N/A	N/A	N/A	5
Pseudo-static Analyses	1760	26.4	0.45	5	1	275	N/A

simplified superstructure, previously used by Sadek and Shahrour (2006), was placed on the pile cap to simulate interaction between the pile group and a superstructure. The simplified superstructure composed a concentrated mass of 40 tons supported by a 1m high column connected centrally to the pile cap (see Figure 1). The fixed base fundamental frequency of this SDOF system is 1.36 Hz. As noted above, dynamic analyses were carried out for two cases: (i) the micro-piles were equipped with a steel casing extending from the ground surface to a depth of 5m and (ii) the micro-piles were uncased. Five percent Rayleigh damping was assumed in the uniform homogeneous soil layer (see Table 1). The concrete and rebar were assumed to be fully bonded and modelled (meshed) separately. The Elastic modulus and Poisson's ratio of concrete and steel were 28 GPa and 0.3 and 120 GPa and 0.25, respectively. The pile behaviour was assumed to be linear elastic with 2% Rayleigh damping, which was considered to be a reasonable approximation for cases where the maximum stress developed in the pile did not exceed the yield stress of the concrete. A linear elastic material model was used for the soil and the pile-to-soil interface was fully bonded, to facilitate numerical convergence.

A horizontal acceleration with peak amplitude of 0.3g and predominant period of 0.16s was applied to the model base, which is considered to be far enough from the resonance condition of the system. The duration of the Ricker wavelet was 2 seconds. However, the analyses were continued for four seconds. The input time history was a Ricker wavelet (see Gazetas, 2001), which is of the form

$$a(t) = A \cdot \frac{1 - 2[\pi \cdot f \cdot dt \cdot (t - t_0)]}{\exp[[\pi \cdot f \cdot dt \cdot (t - t_0)]^2]} \quad [2]$$

where, $a(t)$ is the acceleration time history, t is the time, dt is the sampling interval, t_0 is the duration of interest, A is the maximum acceleration, and f is the predominant frequency of the motion.

2.4.2. Pseudo-Static Analyses of Micro-pile Groups

Pseudo-static analyses of the problem depicted in Fig. 1 were carried out to investigate the influence of EI of the pile cap and the effect of a soil surface surcharge on the lateral group behavior. The simplified superstructure described

earlier in this study was used. Analyses were performed using uniform homogenous soil layer and the Drucker-Prager constitutive model (see Table 1). The pile cap was loaded using a 1 KN lateral load applied in sinusoidal form. The period of the applied load was 1 second. The Lagrange multiplier method of constraint enforcement was used for the normal component of the soil-pile contact. The tangential component of the contact was modeled using the stiffness (penalty) method that permits some relative motion of the surfaces (Hibbit *et al.*, 1996).

3. RESULTS

3.1. Behavior of Micropiles with and without a Casing (Dynamic Analysis)

Figure 4 compares distributions of the bending moments along pile P5 for the cases: (i) with and (ii) without casing. In Figure 4, the maximum bending moments of P5 have been shown from the bottom of the casing (5m depth) to the pile tip (15.3 m depth), only. The results show that the bending moments for piles with a 5m casing, are significantly higher than those for piles without a casing at a depth of 6.5m. Thus, in this case, equipping the micro-piles with an upper casing resulted in over 100 % increase in the bending moments just beneath the point where casing was terminated. This may have a significant impact on the structural performance of the micro-piles, since there is already a complex stress regime at the location due to the casing termination. Beyond a depth of 9 m, the difference in bending moments of the two cases is minor.

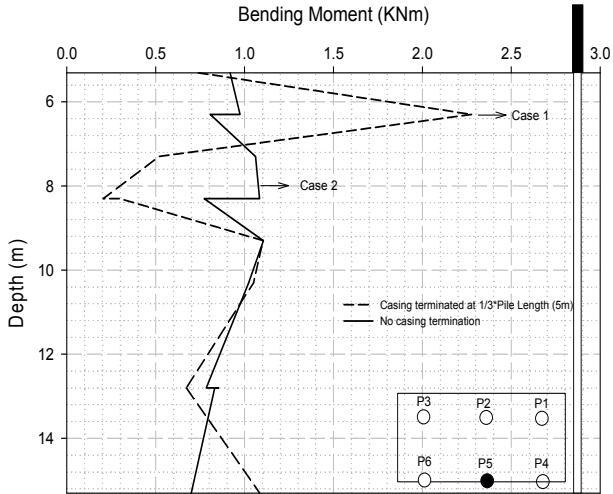


Figure 4. Effect of casing termination on bending moments in pile P5.

3.2. Effect of the Flexural Stiffness of the Pile Cap (Pseudo-Static analysis)

As noted above, the effect of EI of the pile cap on the soil-pile group interaction was also investigated for the problem depicted in Figure 1. The flexural rigidity of the pile cap was varied by changing the elastic modulus of the pile cap, while keeping the thickness of the pile cap constant (0.3 m). EI values of 8.7×10^8 and 1.9×10^8 were considered to evaluate rigid and flexible cap configurations. In Figure 5, the maximum bending moments in P5 are shown for each of the cases from 0.3 m depth, which is the level of soil surface to the pile tip at 15.3 m depth. Figure 6 shows the maximum values of pile-to-soil contact pressures and gap openings on the pile-soil interfaces.

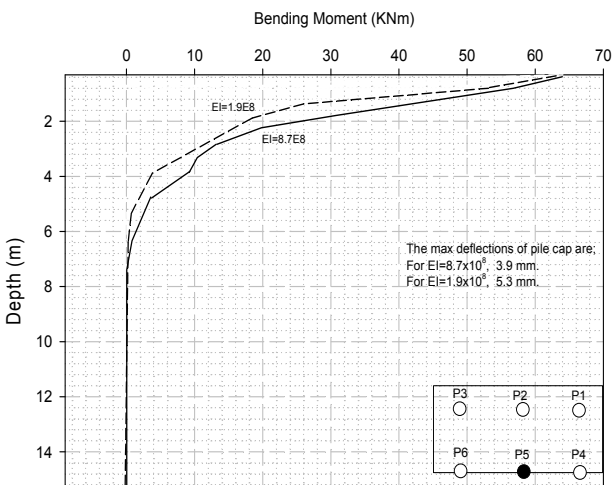
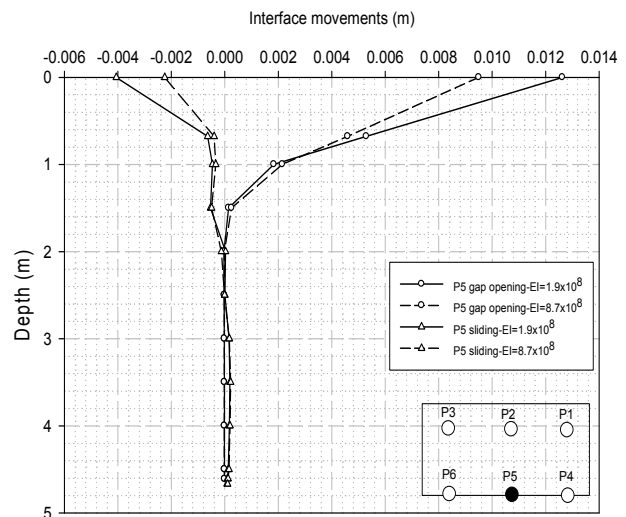


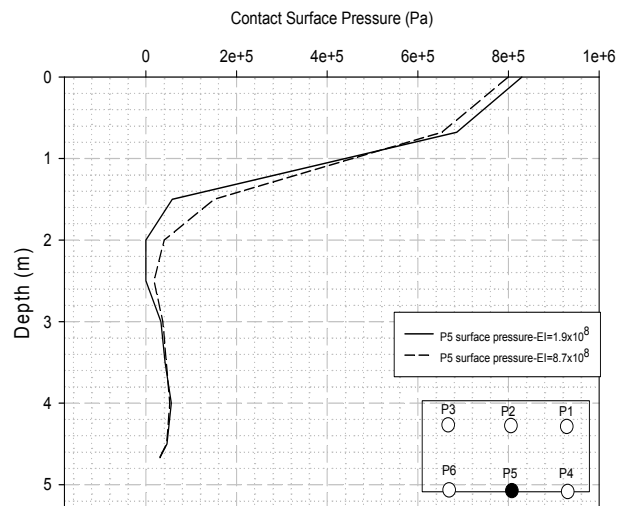
Figure 5. Effect of pile cap flexural stiffness on bending moments along P5.

The results in Figure 5 show that, maximum values of the bending moments remained comparable for the cases with $EI=1.9 \times 10^8$ and $EI=8.7 \times 10^8$. Thus, EI does not appear to have a major impact on the bending moments in the piles.

As shown in Figure 6 (a), the calculated maximum gap opening for the case with $EI=1.9 \times 10^8$ was 12.5 mm, whereas the opening for the case with $EI=8.7 \times 10^8$ was 9.4 mm. A similar trend has been observed for sliding behaviour as well. Increasing EI of pile cap caused a decrease in the magnitude of sliding between the pile and soil (4 mm versus 2.3 mm for the cases with $EI=1.9 \times 10^8$ $EI=8.7 \times 10^8$, respectively). Beyond the depth of 2 m, there was no slip on the interface. Figure 6 (b) shows that the contact pressures for the case $EI=1.9 \times 10^8$ are higher for the depth of 1 m. However, the other case ($EI=8.7 \times 10^8$) gave higher surface pressures between 1 and 3 m. Beyond the depth of 3 m, the influence of pile cap flexural rigidity is negligible. It is hypothesized that although bending moments are not significantly influenced by EI, variations of the moments, and contact conditions will have an impact on the period of the pile ground.



(a) Pile-Soil Contact Separation



(b) Pile-Soil Contact Pressures

Figure 6. Effect of the flexural stiffness of pile cap on the contact movements and pressures.

3.3. Effect of the Surface Surcharge (Pseudo-Static analysis)

Finally, the effect of a surface surcharge on the soil-pile interaction was investigated for surcharges of 0, 20 and 50 KPa applied at the ground surface. In Figure 7, the maximum bending moments of P5 are shown for each of these surcharges. and Figure 8 shows the maximum values of contact pressures and gap openings on the pile-soil interfaces.

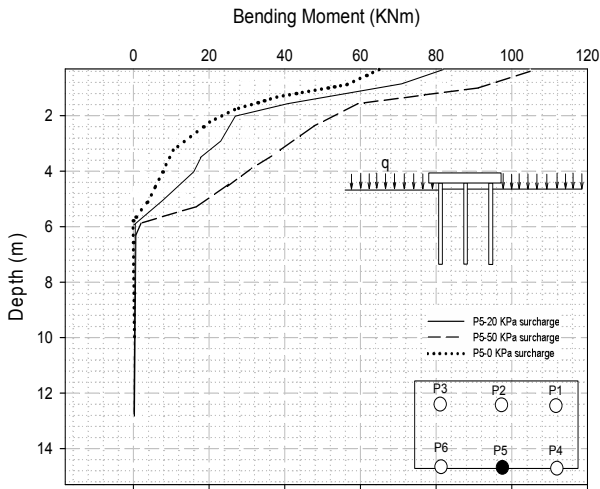
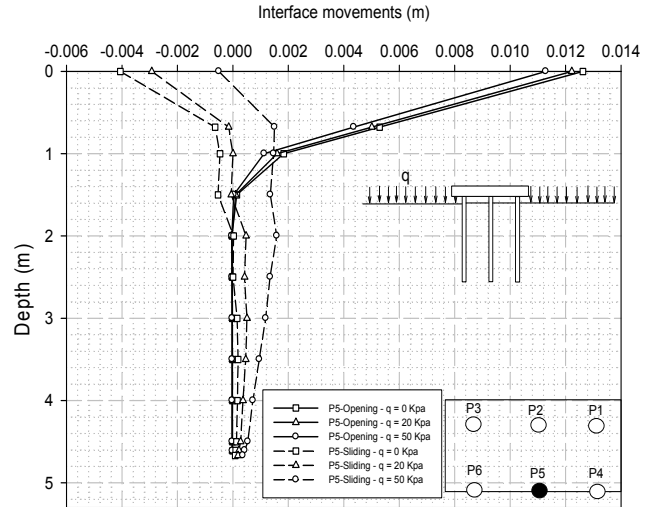
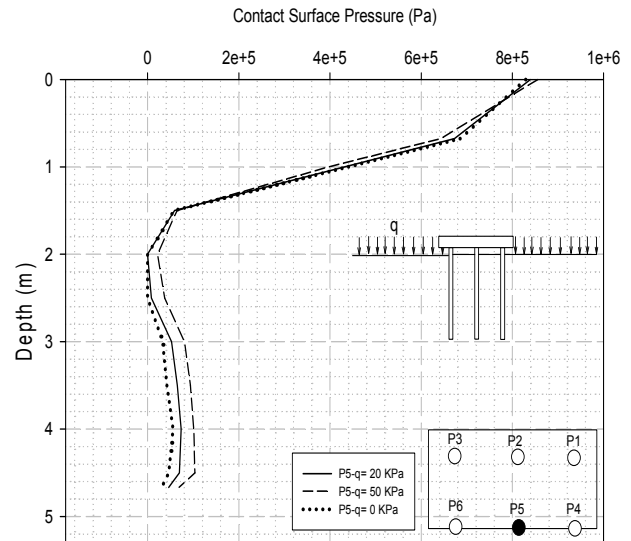


Figure 7. Effect of soil surface surcharge on the bending moments along P5.

From Figure 7, it can be seen that increasing the surface surcharge increases bending moments. The bending moments for cases with 0, 20 and 50 KPa surcharge values were 64, 82 and 105 KPa, respectively. The results, shown in Figure 8(a) reveal that increasing the surface surcharge decreased the magnitude of the soil-pile contact opening. The calculated separations for surcharges of 0, 20 and 50 KPa were 12.5, 12.1 and 11.2 mm, respectively. Sliding at the top portion of the soil-pile contact showed similar trends (e.g. increasing the surcharge decreased the magnitude of sliding). The relative slip for surcharges of 0, 20 and 50 KPa were 4, 3 and 0.7 mm, respectively. The contact stresses along the soil-pile interface remained almost unaffected from increasing surcharges up to the depth of 1.5 m. However, the values increased with the increase of surface surcharge beyond the depth of 1.5 m.



(a)



(b)

Figure 8. Effect of soil surface surcharge on (a) the contact movements and (b) pressures.

4. SUMMARY AND CONCLUSIONS

This paper has presented the results of a series of dynamic and pseudo-static analyses of a micropile group. The primary objective of the analyses was to study the implications of designing micro-piles with an upper cased section versus using no casing at all. In addition, some preliminary sensitivity studies were done to assess the importance of EI of the pile cap and the effects of a surface surcharge at the ground surface. The following is the summary of the results and conclusions arising from this study.

1. This study has shown that, when micro-piles are equipped with an upper cased section, the bending moments in the pile may increase by more than 100% just below the casing termination compared to micro-piles with no casing at all. This could have a significant impact on micro-pile performance given the stress concentrations that already exist at this location.
2. Maximum values of the bending moments were not significantly affected by the flexural rigidity (EI) of the pile cap. The maximum pile-soil contact separation for the flexible cap was 33 % more than that for rigid pile cap case. A similar trend was observed for sliding behaviour as well. Increasing pile cap flexural rigidity caused a decrease in the magnitude of sliding between pile and soil. Beyond the depth of 2 m, interface movements decreased to negligible levels. The contact pressures for the flexible cap case were higher for the depth of 1 m. However, the rigid cap case resulted in higher surface pressures between 1 and 3 m. Beyond the depth of 3 m, influence of pile cap flexural rigidity disappeared.
3. Increasing the surface surcharge from 0 to 50 KPa resulted in a 64 % increase in the maximum bending moments in the pile caused by a harmonically varying static lateral load. The results showed that increasing the surface surcharge decreased the magnitude of pile-soil separation. The surface pressures on the soil-pile interface were almost unaffected by increasing surcharges up to 1.5 m depth. However, the values increased with the increase of surface surcharge beyond 1.5 m depth.

5. ACKNOWLEDGEMENTS

The research reported in this paper has been funded by grants from the National Research Council of Canada and the University of Western Ontario Academic Development Fund. The authors also wish to acknowledge the useful advice and suggestions of reviewers Dr. Ayman Shama and Nadir Ansari.

6. REFERENCES

- Bruce, D. A., Bruce, M. E. C. and Traylor, R. P. (1999). High capacity micropiles – Basic principles and case histories, in *Geo-Engineering for Underground Facilities*, Geotechnical Special Publication No. 90, (edited by G. Fernandez and R.A. Bauer). ASCE, Reston, Virginia. pp. 188–199.
- Comodromos, E. M. and Bareka, S. V. (2005). Evaluation of negative skin friction effects in pile foundations using 3D nonlinear analysis. *Computers and Geotechnics.*, 32:210–221.
- Comodromos, E. M. and Pitilakis, K. D. (2005). Response evaluation for horizontally loaded fixed-head pile groups using 3-D non-linear analysis. *Int. J. Numer. Anal. Meth. Geomech.*, 29:597–625.
- FHWA (1997). *Drilled and Grouted Micropiles: State-of-Practice Review Volume I-IV*, in Report No. FHWA-RD-96-016, 017, 018, 019, Federal Highway Administration, U.S. Department of Transportation, McLean, Virginia.
- Gazetas, G. (2001). The 1999 Parnitha (Athens) Earthquake: soil effects on distribution of damage. XV ICSMGE TC4 Satellite Conference on "Lessons Learned from Recent Strong Earthquakes", Istanbul, Turkey.
- Geosystem, L.P. (2002). Description of full scale tests conducted and data obtained in the three phases of tests conducted for the U.S. Military in Baltimore, M.D. Federal Highway Administration, Order DTFH61-02-P-00162, Requisition/Reference No. 41-08-2011.
- Hibbitt, Karlsson and Sorensen (1996), *ABAQUS/Standard User's Manual*, Version 5.6, Hibbitt, Providence, Rhode Island,
- IWM99 (1999). *Proceedings of Second International Workshop on Micropiles*, Yamaguchi University, Ube City, Japan.
- Juran, I., Benslimane, A. and Hanna, S. (2001). Engineering analysis of the dynamic behavior of micropile systems, transportation research record, Paper No. 01-2936; p. 91–106.
- Kishishita, T., Saito, E. and Miura, F. (2000). Dynamic-response characteristics of structures with micropile foundation system. 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 1-8.
- Laefer, D. F. (1999). Geotechnical procedures for at-risk and in-distress structures, In: L. B. Sickels-Taves (eds.). *The Use of and Need for Preservation Standards in Architectural Conservation*, ASTM STP 1355, ASTM, West Conshohocken, PA. pp. 211–225.
- Kramer, S.L. (1996). *Geotechnical earthquake engineering*. Prentice-Hall Inc., Englewood Cliffs, N.J.
- Lu, X., Li, P., Chen, B. and Chen, Y. (2005) Computer simulation of the dynamic layered soil–pile–structure interaction system, *Canadian Geotechnical Journal*, 42: 742–751.
- Lysmer, J. and Kuhlemeyer, R.L. (1969). Finite dynamic model for infinite media. *Journal of the Engineering Mechanics Division, ASCE*, 95(4): 859–877.
- Mascardi, C. A. (1982). Design criteria and performance of micropiles, In: *Recent Developments in Ground Improvement Techniques*, A. A. Balkema, Rotterdam. pp. 439–450.
- Misra, A., Oberoi, R. and Kleiber, A. (1999). Micropiles for Seismic Retrofitting of Highway Interchange Foundation, In: *IWM99, Proceedings of Second International Workshop on Micropiles*, Yamaguchi University, Ube City, Japan. pp. 215–223.
- Ousta, R. and Shahrour, I. (2001). Three-dimensional analysis of the seismic behavior of micropiles used in the reinforcement of saturated soil. *International Journal for Numerical and Analytical Methods in Geomechanics*, 25: 183-196.
- Pearlman, S.L. and Wolosick, J.R.(1993). Pinpiles for seismic rehabilitation of bridges. *Proceedings of the 10th international bridge conference*, Pittsburg, Pennsylvania.
- Sadek, M. and Shahrour, I. (2004). Three-dimensional

- finite element analysis of the seismic behaviour of inclined micropiles. *Soil Dyn Earth. Eng.* 24: 473–85.
- Sadek, M. and Shahrou, I. (2006). Influence of the head and tip connection on the seismic performance of micropiles. *Soil Dyn. Earth. Eng.* 26: 461–468.
- Shahrou, I. and Juran, I. (2004). Seismic behavior of micropile systems. *Ground Improv J* [in press].
- Shahrou, I., Sadek, M. and Ousta, R. (2001). Seismic behavior of micropiles used as foundation support elements: three-dimensional finite element analysis. *Transportation Research Record No. 1772. Soil Mech.* 84–91.
- Taylor, G. E., Gularte, F. B., and Gularte, G. G. (1998). Seismic retrofit of Fourth Street & Riverside viaducts with micropiles, In: A. Maher and D. S. Yang (eds.). *Soil Improvement for Big Digs*, Geotechnical Special Publication No. 81, ASCE, Reston, Virginia. pp. 313–325.
- Turan, A., El Naggar, M. H. and Hinchberger, S. (2007). Seismic performance of a single micro-pile in layered soil. 8th International Workshop on Micropiles.
- Wong, J. C. (2004). *The Seismic Behavior of Micropiles*. Master Thesis. Washington State University. WA. USA.
- Yamane, T., Nakata, Y. and Otani, Y. (2000). Efficiency of micropile for seismic retrofit of foundation system. 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 1-8.
- Yang, J.X., McManus, K.J and Berrill, J.B. (2000). Kinematic soil-micropile interaction. 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 1-8.
- Zelenko, B. H., Bruce, D. A., Schoenwolf, D. A. and Traylor, R. P. (1998). Micropile applications for seismic retrofit preserves historic structure in old San Juan, Puerto Rico, In: L. Johnsen and D. Berry (eds.). *Grouts and Grouting*, Geotechnical Special Publication No. 80, ASCE, Reston, Virginia. pp. 43–62.