Investigation of Tunnel-Soil-Pile Interaction in Cohesive Soils

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Abstract: Underground tunnels are considered to be a vital infrastructure component in most cities around the world. Careful planning is always necessary to ensure minimum impact on nearby surface and subsurface structures. This study describes the experimental investigation carried out to examine the effect of existing piles installed in cohesive soil and extended to bedrock on the circumferential stresses developing in a newly constructed tunnel supported by a flexible lining system. A small scale testing facility was designed and built to simulate the process of tunnel excavation and lining installation in the close vicinity of preinstalled model piles. Lining stresses were measured for different separation distances between the lining and the existing piles Consistent decrease in the lining load was observed when the piles are located within a distance of one tunnel diameter from the tunnel. The results presented in this study indicated that measuring the lining response near existing pile foundations may be used to evaluate the extent of the interaction between the lining and the surrounding piles.

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Introduction

The construction of tunnels in soft ground is associated with changes in the state of stress in the close vicinity of the tunnel. Peck (1969) developed a method to determine surface settlement and lining stresses induced by tunneling based on field measurements and empirical data. Analytical solutions of similar problems were also developed by several researchers (Zurabov and Bugaeva 1962; Muir Wood 1975; Chou and Bobet 2002). A major concern during tunnel excavation is the potential damage to surrounding buildings and subsurface structures. Several studies investigated the effect of tunnel construction on nearby deep foundation systems (Breth and Chambosse 1974; Morton and King 1979; Attewell et al. 1986; Rankin 1988; Vermeer and Bonnier 1991; Burland 1995; Mroueh and Shahrour 1999; Coutts and Wang 2000; Cheng et al. 2006), however the reverse of the problem (the effect of existing pile foundation on the stresses developing in the tunnel lining) has not received enough research attention. Selected studies that have investigated the tunnel-pile-soil interaction are summarized below.

Chen et al. (1999) studied the pile response induced by tunneling using a two-stage approach. First, the ground movement caused by tunneling is analyzed (free from structures) using the analytical approach of Loganathan and Poulos (1998). The calculated soil movements are then imposed on the pile and boundary element analysis is conducted to compute the bending moment, lateral deflection, compressive, and tensile axial force in the pile. Factors influencing the pile response ranged from the tunnel geometry, volume loss, soil strength and stiffness, pile diameter, and ratio of pile length to tunnel depth below surface. Results indicated that bending moments in the pile increased as the separating distance reduced to about 1.5 the tunnel diameter. A faster rate of bending moment increase in the pile was also reported as the volume loss increased from 1 to 5%.

Lee and Ng (2005) numerically investigated the response of piles to an advancing open face tunnel in London clay. A zone of influence (one tunnel diameter around the pile) was defined, in which an excess pile settlement was found to develop in addition to an increase in bending moment along the pile. The increase in bending moments around the axis parallel to tunnel alignment increased more significantly as compared to the moments around the perpendicular axis, particularly in the zone of influence.

Kitiyodom et al. (2005) investigated the response of a single pile as well as a piled raft foundation to tunneling using the finite difference method. Results indicated that single piles experienced an increase in lateral deflection, bending moment, vertical movement, and axial force when the separation distance is less than one tunnel diameter. For piled raft foundation the closest piles in the group experienced greater movement as compared to the farthest with respect to the tunnel advance.

Lee and Yoo (2006) conducted experimental investigations using a two-dimensional (2D) physical model (910 mm $\times$720 mm$\times$75 mm) to quantify the tunnel movement during excavation in the vicinity of preinstalled model piles installed in an idealized 2D granular material. Tunnel excavation was modeled by reducing the diameter of a cylindrical shaped apparatus. Results indicated that horizontal and vertical shifting of the tunnel develops when excavated in the close vicinity of the piles.

The objective of this study is to investigate the effect of an existing pile group installed in soft ground and extended to bedrock on the circumferential stresses developing in a flexible lining installed in the close vicinity of the piles. Fig. 1 provides a gen-

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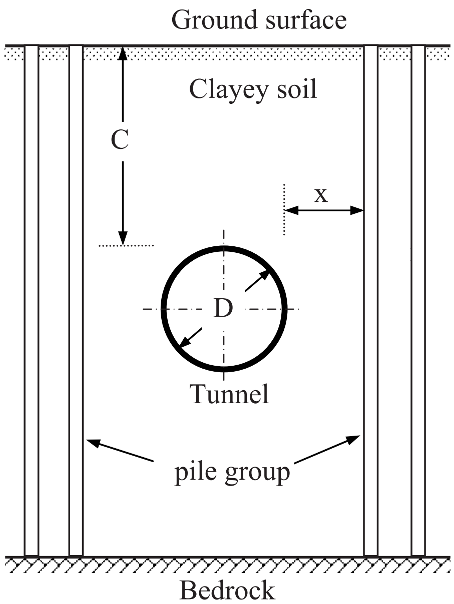

Fig. 1. Problem statement

eral outline of the problem. An experimental investigation is conducted with emphasis on examining the effects of the separation distance between the tunnel and the existing pile group on the bending moment in the lining.

Physical Model

Several physical models have been developed to study the ground response to tunneling in soft ground including the trap door method (Terzaghi 1936; Vardoulakis et al. 1981; Tanaka and Sakai 1993; Park et al. 1999), a preinstalled tube with vinyl facing (Chambon et al. 1991; Sterpi et al. 1996; Kamata and Masimo 2003), a dissolvable polystyrene foam core (Sharma et al. 2001), or a miniature tunnel boring machine (Nomoto et al. 1999). These methods are described in more detail elsewhere (Meguid et al. 2008). Tests are generally conducted under either 1*g* conditions or in a centrifuge. 1*g* models allow one to investigate complex systems in a controlled environment and are considered to be more economical compared to centrifuge or field investigations. The usefulness of 1*g* models is limited by the fact that in situ stresses are not realistically simulated. Despite this limitation 1*g* models have long been used in soft ground tunneling research. Centrifuge modeling is a convenient tool to reproduce gravity stresses in a small model. Limitations of centrifuge modeling include (Taylor 1995): grain size effects in small models and inconsistency of scaling factors for different measured quantities (e.g., length, inertia force, creep, etc.). In addition, the radial forces induced during centrifuge testing are not usually the same throughout the model.

Scale model experiments (particularly 1*g* tests) normally require the preparation of large size soil samples. For tests involving cohesive material, the soil is usually consolidated from slurry to reach the desired consistency and shear strength needed for the proposed test. Chapman et al. (2006) built a 1/50 scale test setup (1.80 m long, 0.60 m wide, and 0.45 m high) for modeling multiple tunnels constructed in soft ground. Kaolin clay powder was mixed with water and consolidated from slurry for 5 weeks to a prescribed strength (5 kPa). A water content of 120% or twice the liquid limit (LL) is usually used to create slurry that would facilitate pumping of the material into the test tank and allow the preparation of a uniform clay bed.

An alternative method of preparing cohesive soil that minimizes the need for consolidation is described by Stimpson (1970) and involves mixing clay, fine sand, water, and adding a small percentage of cementing agent. Dykeman and Valsangkar (1996) and Dunham et al. (2005) adapted Stimpson's work to simulate a stiff clay and soft rock medium which was used to investigate soil-pile interaction problems. The above research work provided an appropriate basis for investigating the design of a mixture that would simulate the basic behavior and mechanical properties of a soft, cohesive material suitable for tunneling applications.

A testing facility has been designed such that the entire model was contained in a rigid steel box with internally lubricated sides to minimize frictional restraint of soil movement. The tunnel location was selected such that overburden pressure applied over the tunnel is maximized ($C/D=5.3$) and in the meantime the effect of the rigid boundary location on the measured lining stresses is minimized. This was achieved by placing the lateral boundaries at a distance of four times the tunnel diameter (4*D*) from the tunnel circumference. The rigid base was located at a distance of 1*D* below the tunnel to represent the case where a strong layer (e.g., bedrock) is present. The dimensions of the rigid box were chosen such that they facilitate 2D simulation of the tunnel construction and minimizes the 3D effects that would develop at the tunnel face during incremental tunnel excavation in thicker testing facilities. The chosen design of the test facility is consistent with those reported in the literature as discussed in the previous section. The box is approximately 1.40 m wide $\times$1.20 m high$\times$0.30 m thick made of four steel sides and one detachable Plexiglas face. The entire box was stiffened using three 0.10 m hollow steel sections as shown in Fig. 2. A description of the different components used in the model is given below.

Tunnel Lining

The lining is an aluminum pipe constructed by rolling a 0.30 m wide by 0.40 m long aluminum sheet of 0.25 mm thickness around a 0.014 m diameter pipe, then closing it with a series of 3 mm nuts and bolts (see Fig. 3). A summary of the lining properties is provided in Table 1. The aluminum lining provided the necessary flexibility and elasticity to maximize strain detection. The compressibility and flexibility factors for the chosen lining according to Peck (1969) are 0.01 and 7,000, respectively, which represents an incompressible and highly flexible lining. The lining was instrumented with eight strain gauges and supported on the soil independent of the rigid boundaries. Strain gauge readings were taken for a gap closure that corresponded to a volume loss of about 3%.

Model Piles

Model piles are selected to simulate the case of deep, relatively stiff piles extended in soft clay and bearing on a strong soil or bedrock layer below the tunnel level. They consisted of 30 25 mm diameter steel bars distributed symmetrically in three rows of five piles on both sides of the tunnel. The piles are restrained from movements in the horizontal direction at the top and bottom of the box using a metal grid and a perforated wooden plate, respectively. In addition, piles were directly supported at the base of the box to prevent movements during testing. Pile compressibility and flexibility factors were calculated using the relationships

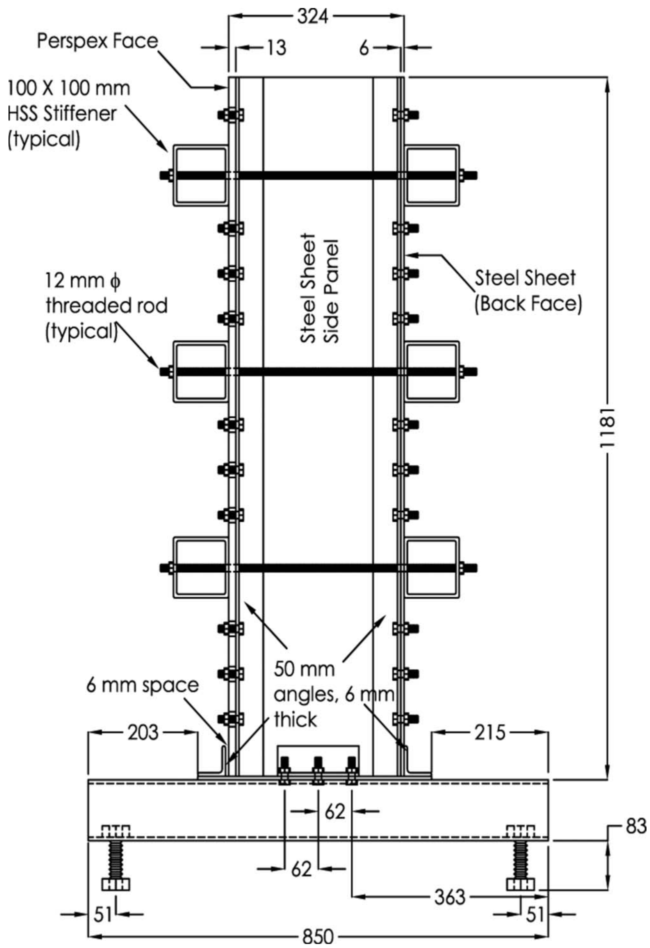

Fig. 2. End view of strong box

($C_p = E_p A_p / E_s$ and $F_p = E_p I_p / E_s L^4$) of Poulos and Davis (1974), where, E_p=Young's modulus of pile; E_s=Young's modulus of soil; L_p=length of pile; A_p=cross-sectional area of pile; and I_p=second moment of area of pile. Respective values of 40 and 0.0016 correspond to the incompressible pile of medium flexibility.

To facilitate the simulation of different pile arrangements around the tunnel, the piles are designed such that they are easily removable from their full grid setup to be rearranged in a multitude of grid patterns. Pile properties are summarized in Table 1.

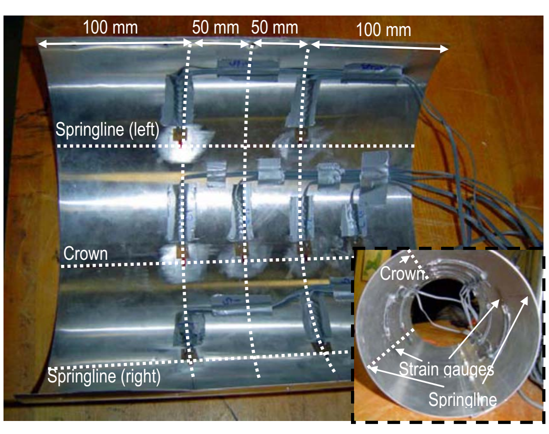

Fig. 3. Inner surface of lining with strain gauge locations

Table 1. Lining and Pile Properties

Lining properties	
Lining diameter (m)	0.15
Thickness (m)	0.00025
Young's modulus (GPa)	64
Poisson's ratio	0.3
Pile properties	
Length (m)	1.1
Diameter (m)	0.025
Young's modulus (GPa)	200
Poisson's ratio	0.15

Shield Casing and Hydraulic Jack

A galvanized steel pipe with an outer diameter of 152 mm, and an inner diameter of 150 mm was chosen to perform the excavation work within the model. The casing pipe was approximately 405 mm in length, which was longer than the inner depth of the steel box. The leading edge of the casing was bevelled using an angle grinder to create a sharp cutting lip. Four 4 mm thick steel plates approximately 25 mm deep were welded in a cross-like formation on the inside of the pipe, at a recessed depth of 25 mm from the leading edge. A 19 mm hexagonal nut was welded at the center of the cross-form and connected to a threaded rod 12 mm in diameter and 915 mm in length. Details of the steel casing are shown in Fig. 4. A hydraulic jack was used to incrementally excavate the tunnel opening within the model. A piston head with a threaded bored hole 12 mm in diameter was mounted on the end of the driving shaft of the jack. Each depression of the lever arm advanced or withdrew the piston through 25 mm. Fig. 5 presents a graphical summary of the hydraulic system.

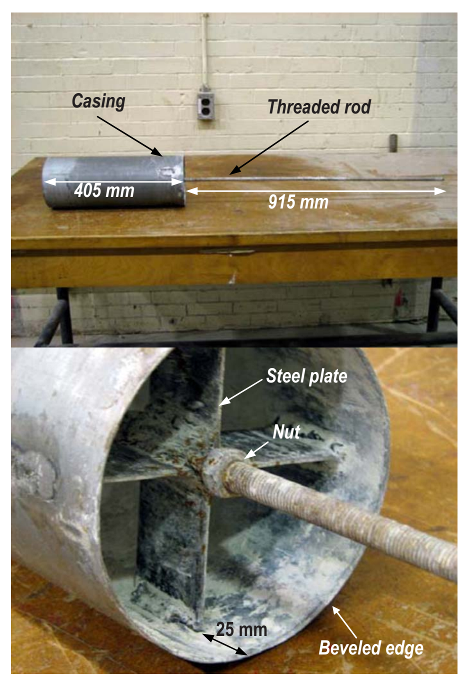

Fig. 4. Steel casing and threaded rod

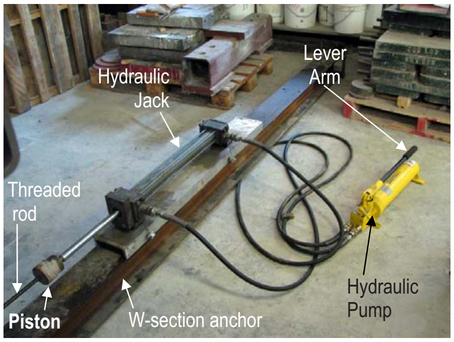

Fig. 5. Hydraulic jack mounting and connection to threaded rod

Table 2. Measured Soil Properties

Fine sand	
Specific gravity (G_s)	2.66
Effective size (D_{10}) (mm)	0.28
Uniformity coefficient (C_u)	1.25
Bentonite	
Liquid limit (LL) (%)	84
Plastic limit (PL) (%)	60
Clay mix	
Moisture content (W_c) (%)	40
Undrained shear strength (c_u) (kPa)	3–4
Saturated unit weight (γ_{sat}) (kN/m^3)	18

Perspex Caps

The caps were used to seal the 152 ϕ mm holes in both the Perspex and steel panels in order that the box would be sealed completely before any material was poured inside. They were attached to the box by using the 5 ϕ mm threaded rods found near the edge of the main holes. The cap that was used on the Perspex panel was sawn in half and a rubber gasket was glued to one sawn face to ensure a properly sealed joint. Circular rubber gaskets were also installed between the caps and the abutting faces of the box to ensure tight connections.

Soil Preparation and Characterization

Several mixes with different clay/sand content were tried to obtain sufficient strength and prevent soil from flowing into the excavated opening. The selected mix consisted of (44% fine sand, 10% bentonite clay, 4% type-10 cement, and 42% water). The mix provided enough workability and produced a uniform soil bed after placement. The following section presents the results of the tests done to characterize each constituent of the mix used throughout this study.

Sand

A fine sand (*Quartz Industrial 7030*) was used as a constituent of the cohesive mix. A sieve analysis was performed on several randomly selected samples of this sand and a particle-size distribution was generated. The coefficients of uniformity (C_u) and curvature (C_c) of the sand were estimated to be 1.90 and 0.89, respectively.

Bentonite Clay

Atterberg limit tests were conducted to determine the plasticity index (PI) of the clay. The plastic limit (PL) of this clay was found to be about 31%. The LL was found to be approximately 85%, thereby yielding a PI=54.

Soil Density

The density of the soil mix was determined in order to estimate the initial stresses that would arise inside the box. This was achieved first by calibrating the inner volume of a vessel, into which samples would be poured and weighed. A cylinder was used for this purpose, having a volume of 223.13 mL. Sample batches with a total mass of 500 *g* each were prepared using the same proportions that were used in the model. The cylinder was filled completely with mix, and the resulting net increase in mass was recorded for each trial. The average density was then calculated to be 17.9 kN/m^3.

Moisture Content

Moisture content values were taken from samples recovered from pilot access holes in the Perspex face approximately 2 h after mixing, once all testing was completed. A uniform moisture content of about 40% was measured across the sample which indicated that the prepared clay bed is homogenous. This was also confirmed by the results of shear strength tests described below.

Shear Strength

Vane shear tests were conducted on the clay sample at five different locations in the box. The shear strength was found to range from 3 to 4 kPa throughout the clay material, which is consistent with the mix design and the initial readings taken from the trial clay mixes. A summary of the clay constituents and mix properties is provided in Table 2.

Testing Procedure

The procedure consisted of pulling a 0.15 m diameter shield (lubricated stiff steel cylinder) progressively through the soil using a hydraulic jack as illustrated in Fig. 5. The soil inside the shield is then removed incrementally and the lining is placed inside the newly created void just before the shield is removed completely from the steel box to allow the soil to move toward the center of the tunnel. To improve the repeatability of the test the shield and the jack are placed on a steel track which consists of a large I-beam section to guide the shield during testing and ensures a consistent path for different tests.

To minimize boundary friction between the soil and the steel sides as well as the Plexiglas face, the inner faces of the testing box are painted with silicon grease before soil placement. The different constituents are mixed for an average mixing time of about 15 min and the mix is slowly poured into the box using a feed hopper. Excess water was allowed to drain outside the box. For tests involving pile groups, the upper and lower fixities are installed first. The fixities consist of a metal grid at the top resting on both sides of the test box and a slotted wooden plate at the bottom of the box and the steel model piles are lined with the corresponding slots before the soil placement. The piles are then

Fig. 6. Photograph of aluminum lining during gap closure process

pushed slowly into the soil one row at a time to minimize soil disturbance. In general, the magnitude of the pore pressures induced due to pile driving decreases rapidly with distance from the pile wall, becoming negligible at a distance of 5–10 pile diameters (Guo 2000). In this study, the distance between the outer perimeter of the model piles and the tunnel lining ranged from 0.1 to 0.4 m which corresponds to a separation distance of 4–16 pile diameter. Minimum impact is, therefore, expected at the tunnel location due to the pile installation process. In addition, and to ensure consistency in initial stress conditions throughout the experiments, the tunnel excavation always started 90 min after pile installation was completed. This is considered necessary to allow for the pore pressure to stabilize in the close vicinity of the piles.

Tunnel Excavation

The shield casing was first lubricated and advanced by the hydraulic jack until the cutting edge was about 25 mm from the outer face of the front Perspex cap. At this time, the two halves comprising this cap were removed and the cutting edge of the casing was advanced into the exposed hole. The casing was incrementally advanced at a rate of about 25 mm every 2 s, which corresponded to one complete cycle of lowering and rearming the lever arm of the hydraulic pump. The cuttings inside of the casing were removed continually as the casing was advanced. When the cutting edge of the casing was approximately 6 mm from the inner surface of the rear Perspex cap, the nuts holding this cap in place were removed, along with the cap itself, and the casing was advanced until the leading edge had passed out of the hole by about 50 mm. The aluminum lining was then placed inside of the shield casing, and the data acquisition system was armed. Finally, the casing was then advanced further, allowing the lining to become exposed to the walls of the soil cavity. At the instant that the surrounding soil came into contact with the lining, the data acquisition system was triggered, and recorded data for a period of 15 min. Fig. 6 shows a photograph of the typical contact between the soil and the aluminum lining.

Results and Discussion

A total of four tests were conducted, one control test and three tests that involved pile groups, namely, x/D=0.7, 2, and 2.7, which correspond to separation distances of 0.1, 0.30, and 0.40 m, respectively between the outer perimeter of the pile and lining sections (see Fig. 7). The convention used throughout this study is that compressive strains will be taken as negative and tensile strains, positive. All test results revealed the same chrono-

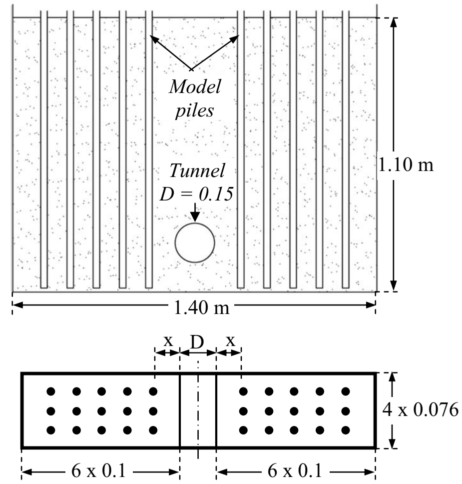

Fig. 7. Schematic of pile groups symmetrically arranged around tunnel

logical pattern based on the strain gauge readings. First, there was a range of values that corresponded to the initial movements of the soil. Then there was an intermediate range that corresponded to the gap closure. The final range represented a state of static equilibrium between the lining and the surrounding soil. These were considered to represent the plane-strain condition. All three phases are denoted as Regions I, II, and III, respectively, as shown in Fig. 8. It should be noted that during the initial stages of the gap closure (Region I), variation in the strain gauge readings was observed (particularly along the tunnel crown). This is explained by the slight nonuniformity in soil movement as it comes in contact with the lining. The readings became more consistent in Regions II and III as shown in Fig. 8.

Averaged strains of 59 and -122 $\mu\varepsilon$ at the crown and springline, respectively, were measured. The bending stresses in the circumferential direction were calculated by multiplying the measured strains by the elastic modulus of the aluminum (E_{measured}

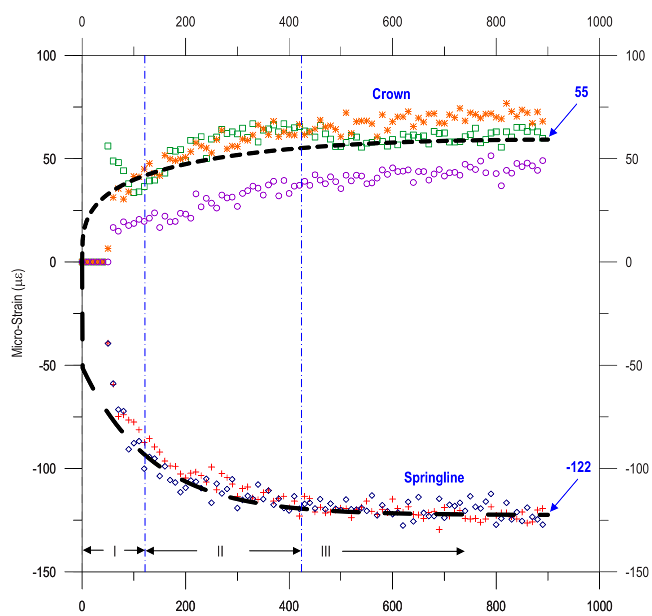

Fig. 8. Strain gauge readings for control test

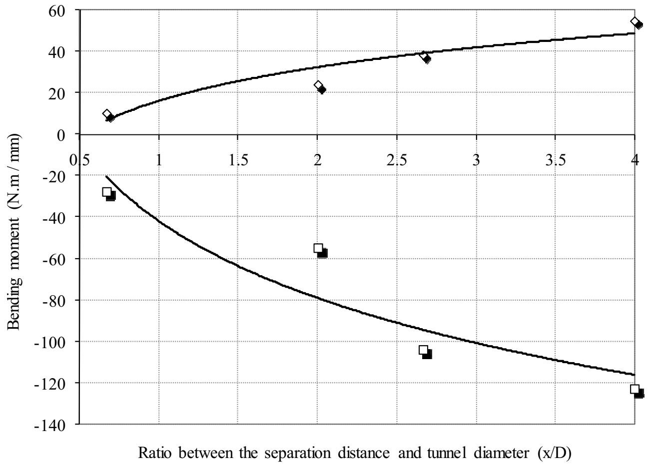

Fig. 9. Summary of experimental results

=64 GPa). Stresses at the crown and springline were found to be 4.9 MPa (tensile) and 7.8 MPa (compressive), respectively. Bending moments are then calculated based on the flexural theory ($M=\sigma . I/y$) using the converged strain values for all tests.

A summary of the bending moment results for all conducted tests is presented in Fig. 9. Bending moment in the lining generally decreased as the separation distance between the pile group and the tunnel circumference decreased. The results also indicated that tunnel-soil-pile interaction is significant only when the tunnel is located at a distance of less than one tunnel diameter from the surrounding piles. Within this distance the stresses generally redistribute such that reduction in the lining stresses corresponds to an increase in the stresses transferred to the nearby piles. This is consistent with the findings of previous researchers who investigated the reverse of the problem with emphasis on the stresses developing in the piles (Chen et al. 1999; Kitiyodom et al. 2005).

Practical Significance

The results presented in this study have an interesting practical significance related to the impact of new tunnel construction on the existing pile foundations. Since it is practically impossible to monitor the stress changes in existing piles during tunnel excavation, this study concluded that monitoring the stresses in the tunnel lining could be used to assess the extent of the tunnel-soil-pile interaction as the tunnel advances. A decrease in lining stresses below expected values implies that a portion of the stresses will be transferred to the surrounding piles and extra care should be taken in constructing these sections. It should be mentioned that the above results are considered to represent cases where piles are symmetrically arranged around the tunnel and, therefore, further investigation is needed to examine nonsymmetrical pile conditions.

Summary and Conclusions

Experimental setup has been designed and used to investigate the effect of existing pile foundation on the stresses developing in a flexible lining system installed in soft cohesive soils. Pile groups were symmetrically installed around the tunnel at separation distances of 0.7D, 2D, and 2.7D from tunnel perimeter. A total of four tests were conducted and strains were measured at the crown and springline of the lining. For the investigated range of separating distance (x/D ratio) tunnel-pile-soil interaction started to develop when the piles were located at a distance of less than one tunnel diameter from the lining circumference. Although the lining response was the prime interest in this study, the piles in the close vicinity of the tunnel usually experience an increased stress level due to the stress redistribution around the excavated tunnel.

Acknowledgments

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