Pile Tunnel Interaction: Literature Review and Data Analysis

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ABSTRACT: The underground space of densely populated cities contains deep foundations and tunnels. Considering the possibility that a new tunnel can be built close to existing piles it is necessary to assess the possible effects of that interaction. Most case studies have shown limited damage on pile supported structures; however, these constructions deal with great uncertainty as the mechanism of pile tunnel interaction is not completely understood. Physical tests at full and reduced scale are a valuable tool to improve that understanding and validate prediction methods. A descriptive review of studies on that matter is presented followed by a quantitative comparison of the results of tunnelling induced axial forces and settlements on the piles. Gathering and analysing these data provided a deeper understanding of the influencing geometrical and structural parameters as well as indicating where further research is needed.

1 INTRODUCTION

The underground space of densely populated cities contains parts of buildings, utility installations, deep foundations and tunnels. It is possible that new tunnels will be built within close proximity of existing pile foundations. Both structures might be located in the same soil layer to use the strength of a stiffer layer at greater depth. The pile tunnel interaction (PTI) must be assessed so that it is possible to ensure safety for both the tunnel construction and the pile-supported structures.

Quite some literature is available on the topic of the influence of new tunnels on existing piles and on the influence of new piles on existing tunnels (see section 2). Considering that most cities that need to develop underground transportation systems normally have an already constructed and consolidated urban centre, the first case is more common and therefore, more studied.

Field tests at full scale demand significant resources for instrumentation and monitoring. This is complicated because most cases will deal with piles already constructed and under operation. The consequence is that there are only a few case histories with consistent data from in-pile instrumentation, and from those most could only investigate a limited number of parameters of the pile considering the instruments and loading limitations.

Another investigative tool is the use of physical modelling with a geotechnical centrifuge or at 1g. The controlled aspects of soil constituency, drainage conditions, load and volume loss enable a very consistent layout for analysis. However, most construction procedures are adapted and their verisimilitude to model the real phenomena can be questioned. Another problem is that at small scale the soil dilatancy can have a disproportionate influence on pile resistance compared to the real case.

These physical models, at full and reduced scale, should be the base to validate mathematical representations of the phenomenon of pile tunnel interaction, namely numerical and analytical models. These mathematical models, validated to the range of physical tests performed, could extend the results to each specific case encountered. Therefore they enable the design of a pile tunnel interaction layout that was not tested by physical models.

The authors came across more than seventy studies regarding this topic. This literature must always be analysed under the terms of their nature (physical tests or numerical/analytical models). A standard layout for the three dimensional positioning of the advancing tunnel and the piles is also important to reach a common ground for discussions.

A general analysis revealed that the reproducibility of the results over the literature was quite poor. Therefore it was decided to focus on the analysis of physical tests to search for the patterns of pile response due to tunnel construction.

A descriptive review of these studies will be presented followed by a quantitative comparison of the results of tunnelling induced axial forces and settlements of the piles. These two parameters are most important in terms of serviceability analysis for both the piles and the pile-supported structures. Several authors also presented the pile bending moment and horizontal displacement. They compose the full picture of the tunnel effects on piles, however quite seldom excessive bending moments and possible pile cracks are a matter of concern.

Gathering and analysing these data will allow a deeper understanding of the influencing geometrical, structural and geological parameters. General trends and contrasting points will be highlighted, indirect results will be described and generally the reliability of the conclusions of the individual studies will be increased.

2 LITERATURE REVIEW

The first published analysis of pile-tunnel interaction regularly quoted dates back to 1979 (Morton and King, 1979). This work already raised interesting issues regarding the tunnel effects on pile bearing capacity and settlements as well as how these effects depend on the relative position of the pile regarding the tunnel.

As mentioned, a standard layout was developed to describe the pile tunnel relative positions and it is presented in Figure 1. The geometrical layout is composed of tunnel diameter (Dt), pile diameter (Dp), depth of the tunnel springline (Zt), depth of the pile tip (Zp), horizontal transversal distance between the tunnel centre and the pile centre (Ld), horizontal longitudinal distance between the face of the tunnel excavation and the pile (Fd). The working load of the pile (WL) is normally referred to in terms of its relation to the ultimate bearing capacity (UBC).
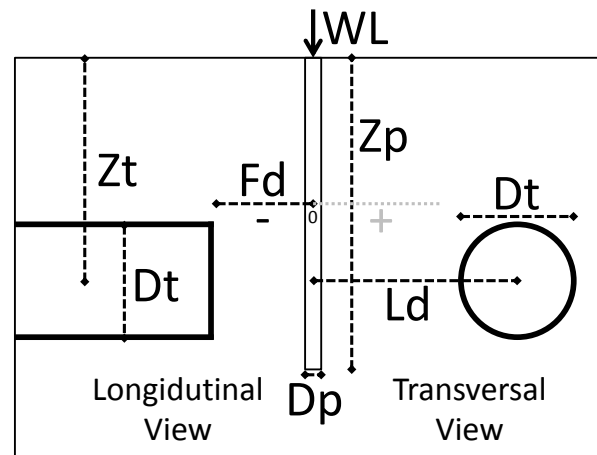

Figure 1.Pile-tunnel interaction layout

The literature will be presented in a division of case studies and physical models.

2.1 *Case Studies*

The redevelopment of the Lee House started in 1987 with the demolition of an office block in the intersection of the London Wall and Wood Street in the City of London, UK (Benton and Phillips, 1991). The project also included the deepening of the basement and construction of under-reamed bored piles. Underneath the redevelopment there were two cast iron tunnels for telecommunication installations constructed between 1920 and 1958 (Figure 4a). Field measurements indicated that the pile loading induced minor changes in the tunnel diameter and horizontal alignment. Those movements proved non detrimental to the tunnels and were less than predicted by a combination of numerical analysis and analytical solutions.

Another case is the construction of a 2 stage hand-excavated tunnel between under-reamed bored piles supporting a 7 storey building to house an escalator system for the Angel underground station in London, UK (Figure 4b) (Lee *et al.*, 1994). The ground movements were predicted by empirical and numerical methods and the structure was intensively instrumented. The inclinometer results have shown that the piles respond as a flexible body to tunnelling induced horizontal movements, deforming in the same way as the surrounding soil.

Along the Island Line of the Hong Kong mass transit railway, the response of several

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pile-supported buildings was monitored against the construction of deep excavations and tunnelling works. Forth and Thorley (1996) concluded that the induced settlements were highly variable, but generally causing little or no damage to the overlying structures.

The separation of the lift shaft, added to the London Bridge House after its construction, from the main building due to tunnelling induced relative settlements was a specific point of concern during the Jubilee Line Extension Project (Selemetas *et al.*, 2002). A jacking system was installed in the shaft basement and operated in response to a precise levelling system focused on avoiding differential settlement along the structure. It was observed that the construction of the pilot tunnel and the ground consolidation afterwards induced minimal settlements. The enlargement of the station tunnel caused more settlements and was the main trigger for jacking operations.

A site test was prepared along the construction alignment of the 2nd Heinenoord tunnel in The Netherlands to investigate pile tunnel interaction conditions for future tunnelling operations in Amsterdam (Kaalberg *et al.*, 2005). Clay columns were created in the sand to reproduce the typical soil profile and the wooden end bearing piles of the city. A numerical analysis was also performed to guide the parameters to be investigated on site. The registered pile settlements were slightly larger (A), equal (B) or smaller (C) than the surface settlements defining three zones around the tunnel (Figure 3). Based on CPT tests performed before and after the tunnel construction, it was concluded that for lateral distance (Ld) greater than one tunnel diameter (Dt) there is no significant stress relief on the pile toe due to tunnelling and no change in the pile bearing capacity.

Figure 2 shows the graphs presented in the study. The CPT test indicates a higher cone resistance after the tunnelling works. On the other hand, the pile load test, whose location was not specified, shows an evident decrease in capacity after the tunnel construction. The load settlement curve inflexion point is roughly 100kN smaller for the pile tested after the tunnel. Therefore the tunnelling induced stress effects are not as clear as described in the conclusions of that paper.
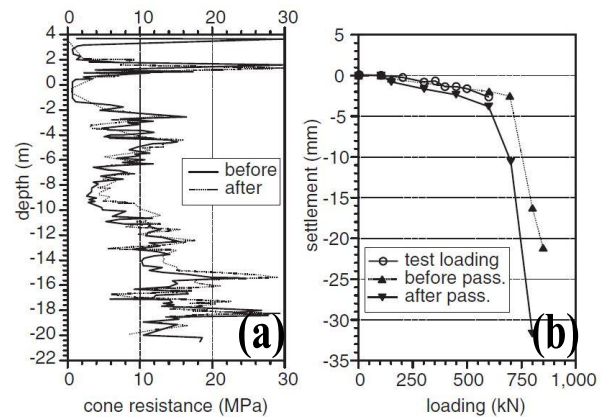

Figure 2.CPT test (a) and static pile load test (b) before and after tunnelling (after Kaalberg *et al*., 2005).

Another test site was prepared in the UK along the construction of the new Channel Tunnel Rail Link (Selemetas, 2005; Selemetas *et al.*, 2005). Friction and end bearing driven piles, loaded to 50% their ultimate bearing capacity (UBC), were monitored during the construction of twin EPB tunnels. In this case a reduction in the base load of the pile was measured and that mobilized the shaft friction capacity. This point presented a marked difference between end bearing and friction piles regarding their shaft resistance and it was most evident directly above the tunnel (Ld=0).

The base load magnitude was also highly dependent on the face distance, decreasing when tunnelling beneath the pile (Fd=0) but increasing again when tail grout was injected underneath the pile. The same three zones of relative pile/soil settlement around the tunnel could be defined as in the Kaalberg's publication. However, it was suggested that the angle between zones is probably a function of the shearing resistance of the soil and the tunnelling volume loss and therefore is not likely to be constant. The defined zones can be seen on Figure 3 for the conditions of the test. There it can be seen that for the Amsterdam conditions the zone A (pile settlements higher than surface settlements) is larger than for the conditions in Essex, UK.
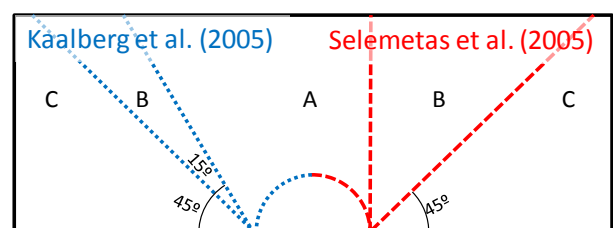

Figure 3.Zones of relative pile/soil settlement (modified from Kaalberg *et al.* (2005) and Selemetas *et al.* (2005))

In Singapore a piled viaduct bridge on the MRT North East Line was constructed just before twin tunnels were excavated around bored piles (Figure 4c) (Pang *et al.*, 2005). Early planning allowed in-pile instrumentation. The non-loaded piles recorded an increase in axial force when the tunnel was less than 4 tunnel diameter behind the pile section ($Fd=-4Dt$); the increase was proportional to the tunnel volume loss and most likely due to the development of negative friction.
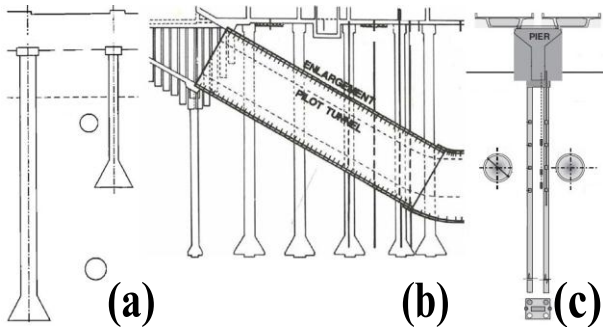

Figure 4. Different layouts for case studies on pile tunnel interaction (from Benton and Phillips (1991) (a), Lee et al. (1994) (b) and Pang et al. (2005) (c))

The, already presented Channel Tunnel Rail Link, was again analysed over three piled structures with friction and end bearing piles (Jacobsz and Bowers, 2005). The analysis converged to the same mechanism presented for the trial site. The tunnel construction causes a stress relief around the pile base that is transferred to the shaft to ensure equilibrium. The response of the pile will be dictated by its rigidity and capacity regarding this new load transfer mechanism.

2.2 *Physical Models*

Following the pioneer work of Morton and King (1979), who performed a 1g test on dry sand with a surface surcharge and a model tunnel of detachable cylinders with increasingly smaller diameters (Ghahremannejad *et al.*, 2006) and on clay, monitoring the lining stresses to detect a load transfer mechanism from the lining to the piles (Meguid and Mattar, 2009). Aluminium rods were also tested at 1g with a model tunnel composed of contractible segments around a cylinder (Shahin *et al.*, 2009; Shahin *et al.*, 2011). Despite being qualitatively comparable, these work's data are not adaptable to prototype scale.

Regarding the influence of new piles on existing tunnels, the results of centrifuge tests

on sand were analysed to find the induced bending moments on the tunnel lining due to pile loading (Chung *et al.*, 2006). When referring to centrifuge tests, all the parameters will be presented in prototype scale.

Centrifuge tests on PTI started 20 years ago (Bezuijen and van der Schrier, 1994), analysing a typical Dutch profile of soft clay over sand and driven piles, with the pile tip and the tunnel based on the sand layer (Figure 5). It was already detected that higher loads on the piles induce higher tunnel-induced settlements. For a tunnel at the same depth as the piles ($Zt=Zp$) the settlements were higher than for a deeper tunnel ($Zt>Zp$). However, for the latter case, the pile effects were significant over a larger distance from the tunnel. The differential settlements between pile and soil were described as a function of the volume loss. For a volume loss below 1%, negative friction developed along the pile as the pile settles less than the surrounding soil. For higher volume loss positive skin friction developed (Hergarden *et al.*, 1996).
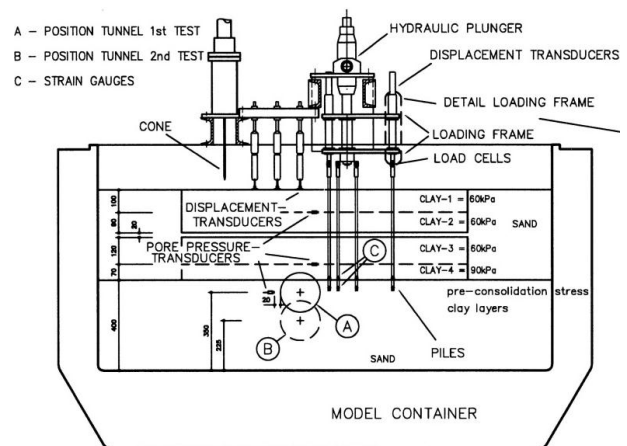

Figure 5. Test apparatus for the centrifuge test (after Bezuijen and van der Schrier (1994))

Studies that considered a layout where the piles and the tunnels are entirely built on clay have also been performed.

One study employed a model tunnel of a rigid cylinder enveloped by a rubber membrane (Loganathan *et al.*, 2000). The volume of the oil that filled the gap between the inner core and the membrane was reduced to simulate the tunnel volume loss. The model tunnel was installed at 1g and the pore pressure could dissipate for eight days between volume loss increments. The tunnelling induced negative friction was again detected by an increase in the pile axial force. For piles shallower than the

tunnel (Zp<Zt), there was an increase along all the pile length. For piles deeper and at the same depth of the tunnel, the increase was detected until the depth of the tunnel, followed by a decrease in the axial force.

Using a model tunnel composed of a high density polystyrene foam that was dissolved on flight inside a brass foil, Ran (2004) achieved the same conclusion of negative friction until the depth of the pile regardless of the pile-tunnel lateral distance (Figure 6a). It is worth noting that these results were measured after just 2 days of pore pressure dissipation and that the constructed model tunnel profile was an ellipse, pushing the soil away from the tunnel around springline.

Ong *et al.* (2006) applied a similar procedure of dissolving foam in-flight but in this case it was between a rubber membrane and a steel cylinder, allowing a better control of the volume loss. The model piles were installed at 1g. Instantaneously the axial force increased over about half the pile length. With time the axial force profile would shift to the long term response observed in the other studies (Figure 6b). Therefore what Ran (2004) observed might be just the short term response, that might change with more time for consolidation. Ong *et al.* (2006) also recorded that the pile settlements were always smaller than the soil surface settlements on the same position, for piles with tips below the tunnel.
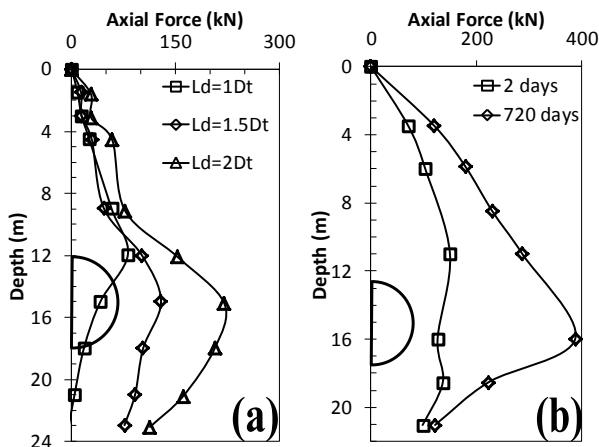

Figure 6.Axial force along depth for different lateral distances (modified from Ran (2004)) and different consolidation times (modified from Ong *et al.* (2006)).

Interaction of piles and tunnels entirely built on sand were also a matter of study.

The same type of model tunnel from Loganathan *et al.* (2000) was used, this time

with water filling the void *(Jacobsz et al.,* 2002). The model piles were jacked 2m in flight. The results converged to a similar type of zones of the case studies presented (Figure 7). In these cases zones A and C present equal pile-soil settlements, in zone B the pile settled more than the surface and in zone D less. Inside zone A, the vertical distance between the tunnel and the pile was of great influence for the pile response.

The load transfer mechanism was studied in a later paper (Jacobsz *et al.*, 2004). Piles inside zones A and B experienced a reduction in the base load and an increase in the shaft friction. On the other hand, piles in zone D experience a small increase in the base load as there is negative friction on the upper part of the pile and no base resistance degradation.
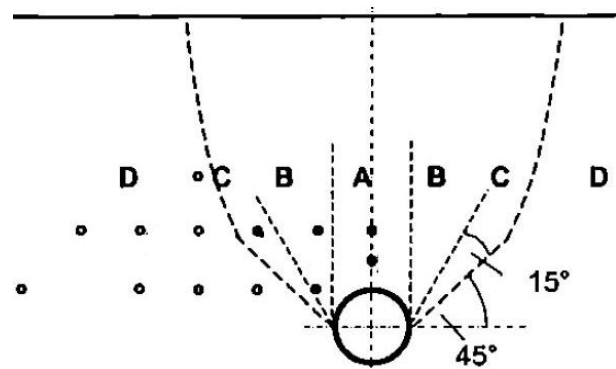

Figure 7.Zones of tunnel influence on piles (from Jacobsz et al. (2002))

Marshall (2009) conducted a study using the same model tunnel from the previous study and model piles also driven in-flight. Figure 8 presents the surface settlements from tests at greenfield (G), that is without the piles, and with the piles (T) for a 13.5m deep tunnel with a 4.6 m diameter. Contrary to what was found in previous studies, Marshall concluded that the greenfield displacements cannot be used to predict pile displacement, as the presence of the piles has a profound effect on the surface displacements.

Regarding the load transfer mechanism it was detected that the described steps might be cyclic as the settlements due to full base and shaft mobilization would compress the base and mobilize resistance again (Marshall, 2009).

Another study used the type of model tunnel from described by Ran (2004) and model piles already in place for the sand pouring. Negative friction was measured until the tunnel depth (Zt) for piles regardless of the lateral distance (Ld),

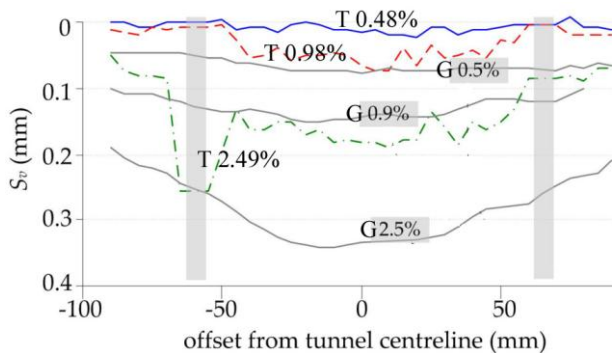enforcing the results for clay presented by Ran (2004) (Feng, 2004; Feng *et al.*, 2002).

Figure 8.Surface settlements for different volume losses for greenfield (G) and with the piles in place (T) (modified from Marshall (2009))

Lee and Chiang (2007) modelled the tunnel controlling the air pressure inside a thick cylindrical rubber bag on which a sheet of filament tape was pasted. The piles were already in place during sand pouring. The tests were performed with no load, and a load of ¼ and ½ of the pile bearing capacity. With no head load, the axial force profile agrees with the previous studies. However, for a loaded pile the tunnel degrades the end bearing capacity of the pile, which is compensated with an increased frictional force. This can be seen in Figure 9 in which Z_p=27 m; Z_t=27 m; D_t=6 m and 1% volume loss.
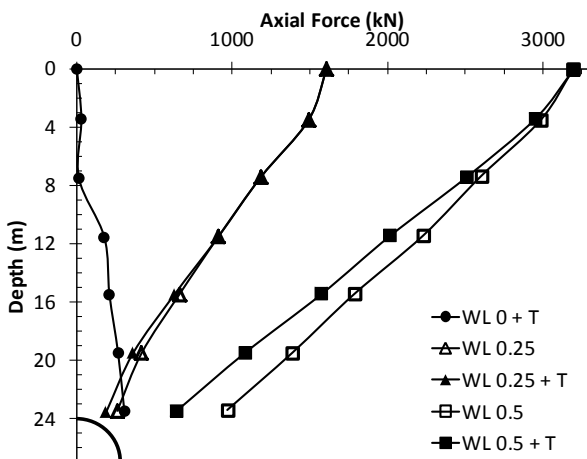

Figure 9.Axial force profile for piles with different working loads (modified from Lee and Chiang (2007))

It is quite evident that there are similarities and differences over the qualitative conclusions presented. To trace mechanisms from qualitative observations might be misleading and achieve wrong interpretations, as the scale

of the results might be very different among different studies even when the behavior is qualitatively the same.

Therefore a quantitative analysis is proposed. The published data was analyzed using software to digitize images, obtaining the measured values for analysis and comparison. The results will be presented next.

3 DATA ANALYSIS

The geometric aspects that compose a pile tunnel interaction layout were shown in Figure 1. However, when distinguishing several different layouts it might be confusing to group similar layouts by several characteristics. Considering this 5 assumptions were made: (i) the tunnel depth (Z_t) itself is not determinant; (ii) the zone around the tunnel can be normalized by the tunnel diameter (D_t); (iii) the pile settlements can be normalized by the pile diameter (D_p); (iv) the soil settlements can be normalized by the tunnel diameter (D_t) and (v) the axial force can be normalized by the pile area.

Considering these conditions a new layout is proposed in Figure 10. The vertical axis takes advantage of (i) assuming all tunnel at depth 0 and of (ii) normalizing the vertical (Z_t-Z_p) and the horizontal (L_d) distances between the tunnel and the pile by the tunnel diameter (D_t).
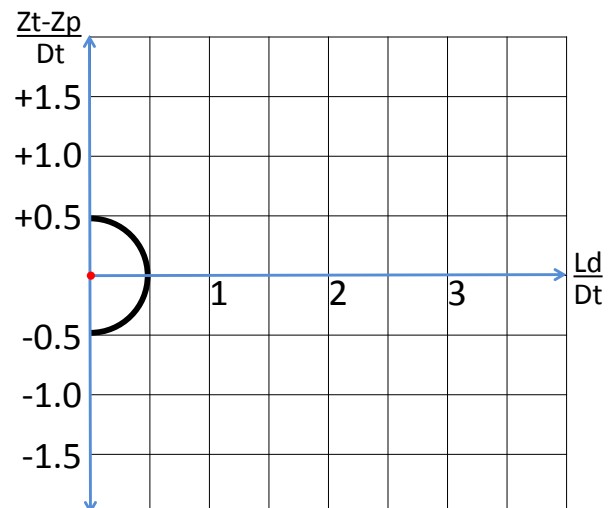

Figure 10.Layout for multiple cases presentation

The pile settlement is presented as a function of the volume loss for fifteen cases from (1-5) Jacobsz *et al.* (2004), (6-9) Marshall (2009) and (10-15) Hergarden *et al.* (1996). It is worth

noting that the first two studies were performed with constant load on the piles while the last one employed springs to apply the load. After 1% volume loss the applied load reduced quite significantly for this case, what probably cause a less steep settlement curve.

Figure 11 presents the geometrical layout of the cases (a) and the settlement curves (b). The first characteristic that can be taken from the results is that piles located just above the tunnel are critical in terms of settlement. Pile 2 for example failed before 1% volume loss. The rule of thumb for pile design is to consider a pile as failed for a head settlement of 10% the pile diameter or more. Using this rule of thumb failure was not reached in these studies; Piles 1, 2, 3 and 6 presented a typical failure response at very low volume loss. A difference can be seen on this point also, Piles 1 and 6 had very similar characteristics, they both failed at about 2% volume loss, but Pile 6 was more rigid before failure than Pile 1.
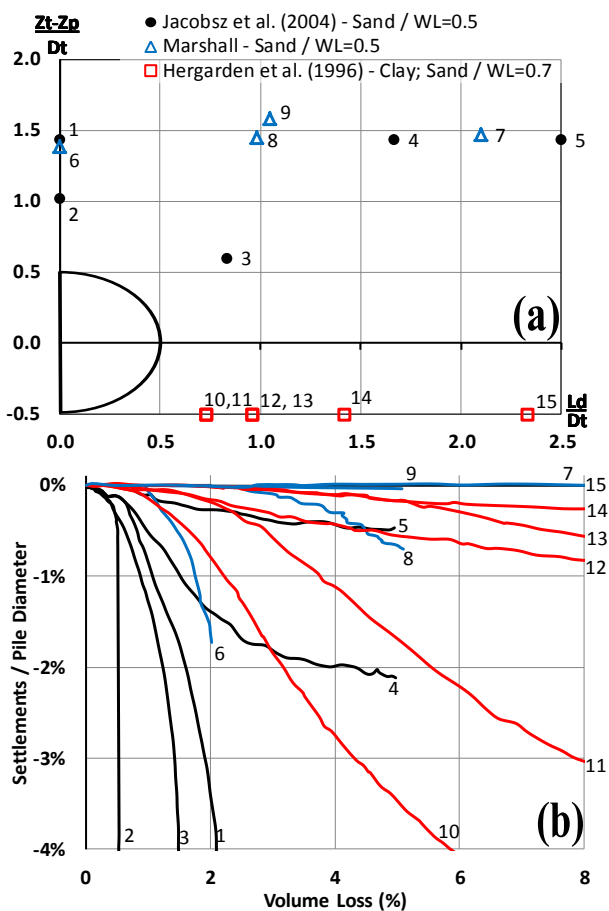

Figure 11.Layout of the analysed cases (a) and pile settlements as a function of volume loss (b).

Another comparison can be made between Piles 3, 10 and 11, as they have similar lateral

and vertical distance but Pile 3 is on top of the tunnel springline and Piles 10 and 11 are below. Pile 3 failed before 2% volume loss while Piles 10 and 11 were much more rigid also for low volume losses. That indicates that piles located above the tunnel springline are much more susceptible to settlement than piles below. However, it must be kept in mind that they were tested in different soil layouts and under different loads and also that the setup of Piles 10 and 11 was under decreasing load after 1% volume loss.

From just before $L_d=1.D_t$ on the tunneling induced settlement was generally under 1%D_p for all cases on the vertical range studied. The exception for this was Pile 4. Piles 9 and 8 were more rigid than Pile 4, despite being located closer to the tunnel. This again illustrates the differences between the studies of Jacobsz *et al*. (2004) and Marshall (2009) as discussed for Piles 1 and 6.

An interesting feature is the comparison of pile and soil settlements, which was done for a tunnel volume loss of 1%. Piles 1-9 from the previous case were analyzed again and data for Piles 10-17 are from taken from Bezuijen and van der Schrier (1994). It must be mentioned that the soil settlements presented on this last study were related to a volume loss of 3%. A linear relation of volume loss and settlements was assumed to adjust the results. Figure 12 presents the geometrical layout of the piles (a) and the piles and soil settlement (b).

The decision to normalize the settlement over the tunnel diameter was intended to scale the soil settlements, and consequently the pile settlements. This should not be understood as the controlling parameter for pile settlements, which are also controlled by the pile diameter itself. Piles 7, 8 and 9 did not presented measureable pile or surface settlements for 1% volume loss; therefore they were suppressed from the presentation as it was Pile 2 that failed before 1% volume loss.

Here it may be questionable if assumption (i) can be used to analyze pile tunnel interaction data. Considering the proposed layout the soil settlements should not be different between Piles 10/14, 11/15, 12/16 and 13/17, but they are. In the study from Bezuijen and van der Schrier (1994) the piles are at fixed depth and the tunnels are in different positions. The shallower tunnel for points 10, 11, 12 and 13 resulted in higher soil settlements than their counterparts 14, 15, 16 and 17. Therefore this

proposed layout should be employed focusing on the pile responses and not the soil settlements. Keeping this scaling issue in mind this data can still be used to analyze the relation between pile and soil settlements, as it was intended to.

For this analysis, Piles 1 and 6 agree in terms that the soil settles more than the pile directly above the tunnel. That goes against the expected response for the zones just above the pile traced by Kaalberg et al. (2005) and Selemetas et al. (2005). From just before Ld=1.5.Dt on the pile settlements were always smaller than the soil settlements, as it can be seen from Piles 4, 5, 12, 13, 16 and 17. From Ld=0.5.Dt to Ld=1.Dt both conditions exist. Piles 3, 10, 14 and 15 settled more than the soil while Pile 11 settled less. That goes with the hypothesis of an inclined boundary marking this difference in relative settlements, explaining the difference between Piles 11 and 15. On the other hand it does not confirm that this boundary would originate around the tunnel springline, as Pile 10 responded the same way as Pile 14.
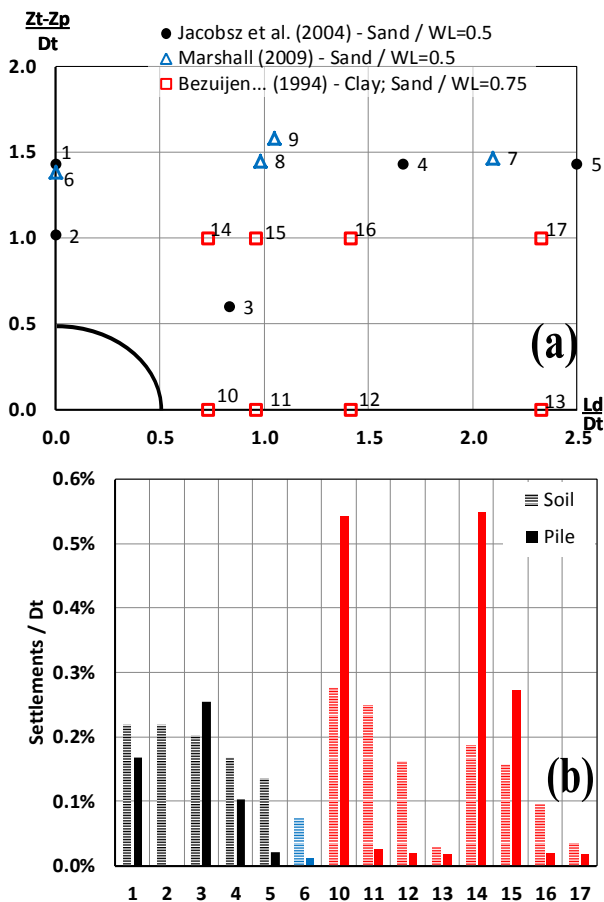
One might be interested to match these results with the field tests from Kaalberg et al. (2005) and Selemetas et al. (2005). Figure 13 present this data. Considering the discussed difference just above the tunnel it can be seen that Piles S1 and S3 actually settled more than the soil, as opposed to Piles 1 and 6 responses. Pile K4 presented significant settlements when their counterparts, Piles 9 and 8, did not. The same was observed between Pile K5 and 7.

The response of Piles K1 and K2 were matching their similar Piles 11 and 12 settling less than the soil. An interesting issue was the position of Piles S2 and S4 between a division zone of Piles 15 and 16, when the response changed from the pile settling more to less than the soil. The piles in this transition zone presented the same pile and soil settlements.
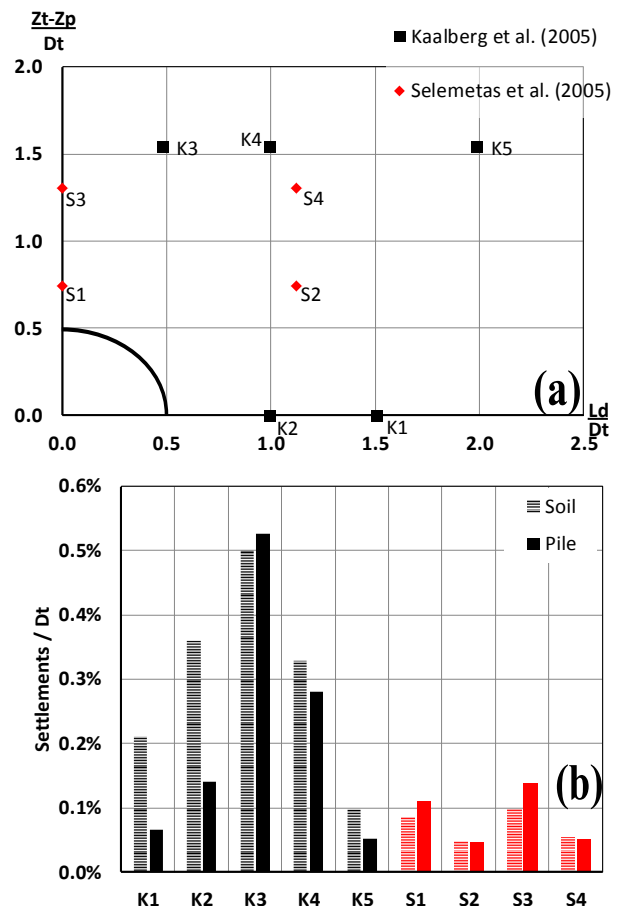

Figure 13.Layout of the analysed cases studies (a) and pile and soil settlements (b).

Considering the tunnelling induced axial forces on non-loaded piles nine conditions on sand could be analysed from (1-5) Feng (2002) and Feng et al. (2004) and (6-9) Lee and Chiang (2007). Considering assumption (v) the axial force was normalized by the pile area obtaining

Figure 12.Layout of the analysed physical models (a) and pile and soil settlements (b).

the axial stress, and the depth along the pile was normalized by the pile depth. Figure 14 presents the geometrical layout of the piles (a) and the axial stress (b), marked in the vertical axis are the positions of the tunnel depth for each case.

All piles present an increase in the axial force until tunnel depth (Zt) followed by a decrease. For Pile 6, the deepest, this reduction was reversed at about half the pile length. For Piles 4, 5 and 9 there was just axial force increase as they were at the same depth or shallower than the tunnel. With the exception of Pile 8, Piles 6, 7 and 9 present the same maximum stress and are at the same lateral distance (Ld). Piles 1, 2 and 3 present the maximum axial stress roughly at the same depth but with a decreasing value as the lateral distance (Ld) increases. The axial stress on Piles 4 and 9 are higher than on Pile 5, indicating that the tunnel effects regarding the axial force may be higher for $Zp>Zt$.
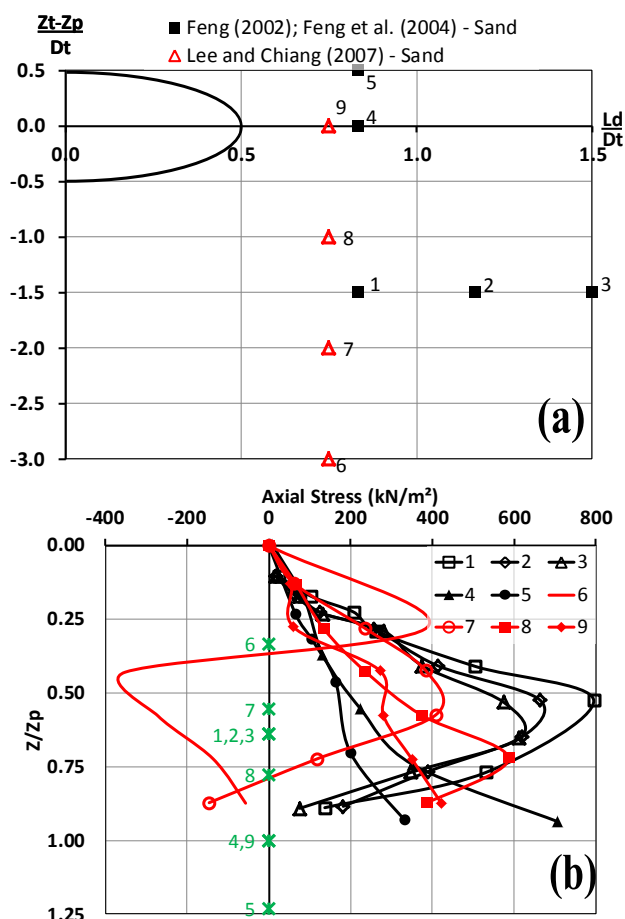

Figure 14.Layout of the analysed cases (a) and pile axial stress (b).

Considering loaded piles, the physical model of (1-6) Lee and Chiang (2007) at 1% volume loss and the case study from (S1-S4) Selemetas et al. (2005) at 0.2% volume loss were analyzed

by the increment of axial stress due to tunneling (Figure 15). It can be noticed that the typical increment for non-loaded piles changes with load. Pile 1 presented no change in the axial force due to tunneling when loaded. Piles 2, 3 and 4 had an increase in axial stress, but with a profile of maximum increment around 70% the pile length, regardless of the tunnel position. On the other hand, Piles 5 and 6 had a decrease in axial stress, higher for higher working load. The same reverse response was measures on Piles S1 and S3, but not on Piles S2 and S4.
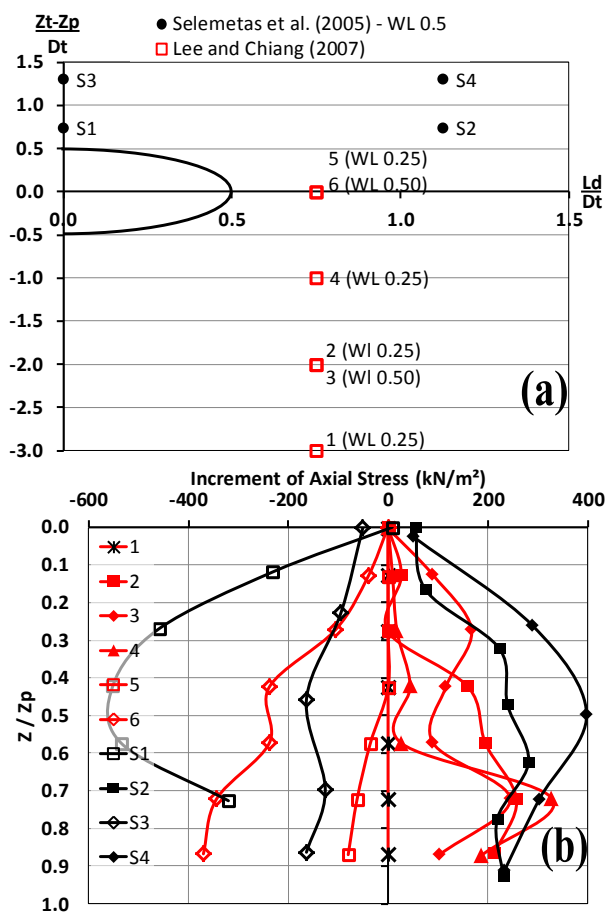

Figure 15.Layout of the analysed cases (a) and increments of pile axial stress (b).

4 CONCLUSION

Most case studies have shown limited damage on pile supported structures due to tunnelling operations. However, these constructions are often executed with a great deal of uncertainty as the mechanism of pile tunnel interaction is not completely understood. This work could show how far the physical tests on pile tunnel interaction have come, both by full and reduced scale test. Valuable information could be

combined and analysed depicting the convergences and divergences of the studies, indicating where further research is needed.

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