Centrifuge modelling of the effect of tunnelling on buried pipelines: mechanisms observed

T.E.B Vorster, R.J. Mair, K. Soga & A. Klar
University of Cambridge, UK

ABSTRACT: Ageing infrastructure such as pipelines are often subjected to third party activities, such as tunnelling. If engineers are unable to confidently judge the effects on the pipeline in question, this may result in costly and possibly unnecessary diversions to avoid the problem. Current methods of assessing the effect of tunnelling on buried pipelines are mostly based on elasticity and although extremely useful for parametric studies and preliminary evaluation of the problem, may contradict true behaviour. The aim of this paper is to discuss the underlying mechanisms governing pipeline response to tunnelling based on results from centrifuge testing.

1 INTRODUCTION

Tunnelling is widely accepted as a sustainable means of reducing increasing traffic problems in large cities and often affects strategic ageing infrastructure such as pipelines (Clemmitt 2003). Developers and engineers are not always adequately equipped to assess the problem to an extent where they can confidently present the risk to clients and affected parties. Subsequently owners of pipeline infrastructure impose stringent regulations on construction near their assets with damage criteria frequently owner-based (Fujita 1994). A need arises therefore to increase the current understanding of pipeline-soil-tunnel interaction.

In the past different approaches have been followed to quantify solutions for continuous pipelines affected by tunnelling, all of which were based on simple numerical analysis of the problem in elasticity with limited validation from field trials or field data (e.g. O'Rourke & Trautmann 1982; Attewell et al. 1986; Bracegirdle et al. 1996). Although extremely useful for parametric studies and preliminary evaluation, most current methods lack identification of driving mechanisms and subsequent guidance for focusing on critical parts of the design or monitoring (e.g. bending strain versus axial strain). Such methods also frequently apply theories developed for piling which may not be applicable to shallow buried pipelines. The aim of this paper is to identify mechanisms governing pipeline response to tunnelling.

2 EXPERIMENTAL SETUP

To simulate the problem in the field, a series of centrifuge tests were performed on model pipelines buried in sand. Three model pipelines of different orders of magnitude bending stiffness, EI, were subjected to ground movement, at 75 g acceleration, induced by a model tunnel located transverse to the model pipelines. The pipelines comprise a 16 mm acrylic pipeline (Pipe 1, prototype diameter 1.2 m), a 15.9 mm aluminium-alloy pipeline (Pipe 2, prototype diameter 1.19 m) and a 34.9 mm aluminium-alloy pipeline (Pipe 3, prototype diameter 2.6 m). The model tunnel, 60 mm in diameter (4.5 m diameter in prototype), is similar to the one used by Jacobsz (2002).

Three configurations of normalized embedment depth, C_p/D_p, and normalized pipe-tunnel separation distance, H/D_t, were investigated in nine experiments. C_p is the pipe embedment depth measured from the pipe crown, D_p is the pipe diameter, H is the pipe-tunnel separation distance (pipe invert to tunnel crown) and D_t is the tunnel diameter. These include a shallow pipe depth case with $C_p/D_p = 1$ and $H/D_t = 2$, an intermediate pipe depth case with $C_p/D_p = 2$ and $H/D_t = 1.5$, and a deep pipe depth case with $C_p/D_p = 3$ and $H/D_t = 1$. A schematic representation of the problem is shown in Figure 1.

A typical view of the experimental setup during model preparation is shown in Figure 2.

The experiments were carried out with Leighton Buzzard Fraction E silica sand ($d_{50} = 0.142$ mm, $e_{max} = 0.97$ and $e_{min} = 0.64$; Lee 2001) with void ratios ranging between 0.65 and 0.68 (average of 0.67). More details about the model and the experimental setup can be found in Vorster (2002).

3 SYSTEM RESPONSE

Assuming that the pipeline does not fracture there are only two ways in which it could respond to

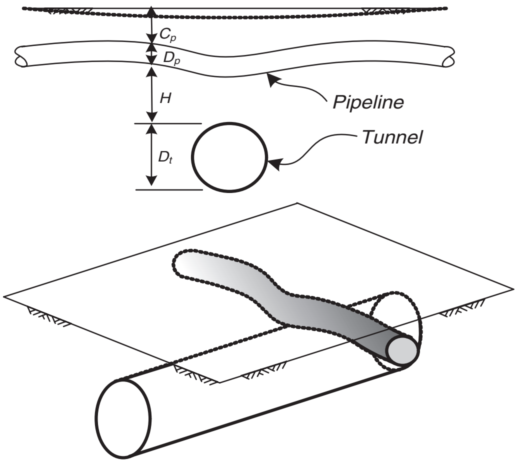

Figure 1. Schematic representation of the problem.

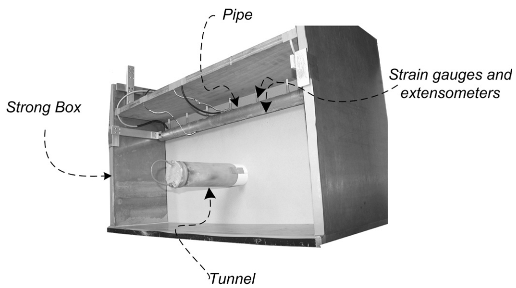

Figure 2. Experimental setup viewed during model preparation (figure rotated for sectional view).

tunnelling: it either follows ground movement exactly (no interaction), or interacts with the surrounding soil in accordance with the relative rigidity, R, of the pipe-soil system. R is usually regarded as the relation between soil modulus, E_s, and structure stiffness, in particular bending stiffness, EI, and axial stiffness, EA (e.g. Attewell et al. 1986 and others). Klar et al. (2004) showed from elasticity that, for the bending part of the problem, R may be defined as:

$$R = \frac{EI}{E_s . r_o . i^3} \tag{1}$$

where r_o is the outer radius of the pipe and i is the distance to the inflection point of the green field soil settlement curve, at a level corresponding to the pipe invert level, and measured from the tunnel centreline (also referred to as trough width). E_s is a representative soil modulus taken at pipe level.

In terms of relative pipe-soil longitudinal stiffness (K^*) Attewell et al. (1986) proposed the following relation based on elasticity and pile design:

$$K^* = \frac{E_p A_p}{E_s A_s} \tag{2}$$

where E_p is the pipe material stiffness, A_p is the sectional material area of the pipe and A_s is the full cross-sectional area of the pipe.

Assuming the pipe characteristics remain constant, it is clear that E_s and i are key factors in determining the relative system rigidity. If R and K^* are adopted to review the general response of pipelines affected by tunnelling, then from a design point of view, it is important to identify contributing factors affecting the choice of E_s and i. However, viewing the problem only from an elasticity point of view limits the designer's perception of influential events which may affect true behaviour. To overcome this problem the responses of the model pipelines in the centrifuge are considered in terms of the development of normalized bending moment, M_{norm}:

$$M_{norm} = \frac{Mi^2}{EIS_m} \tag{3}$$

where M is the pipe bending moment measured in the centrifuge and S_m is the maximum green field subsoil settlement (at a level corresponding to the pipe invert).

In Figure 3 the M_{norm} responses of Pipes 1 and 3 are shown compared to responses of similar pipelines being 'forced' to follow the curvature of the green field soil exactly (infinitely flexible response), denoted by $(M_{norm})_s$. For each pipe, tests results are shown for the shallow and deep pipe depths defined earlier. The $(M_{norm})_s$ curve would vary if the subsoil settlement profile changed shape during different stages of ground movement (e.g. when concentrated ground movement occurs above the tunnel during increased shearing). This was indeed the case, hence for comparison all the results are shown as a ratio of $(M_{norm})_{s,max}$, the maximum value of $(M_{norm})_s$ for the particular stage of ground movement in question; i.e. values of $M_{norm}/(M_{norm})_{s,max}$ (measured pipe response to be referred to as M_p^*) and $(M_{norm})_s/(M_{norm})_{s,max}$ (infinitely flexible response to be referred to as M_s^*).

The following is evident from Figure 3: **[1]** In almost all cases M_p^* differs from M_s^*. This implies that in every case the pipe is not behaving completely flexibly. **[2]** In both the deep pipe depth cases, at lower face loss, M_p^* is in closer agreement with M_s^*, compared to the shallow cases at similar face loss (i.e. more 'flexible'). At larger face loss, however, values of M_p^* in the shallow pipe depth cases sometimes exceed those of M_p^* in the deep pipe depth cases, indicating a higher R for the deep case (e.g. Pipe 3). In such cases a crossover in dominant

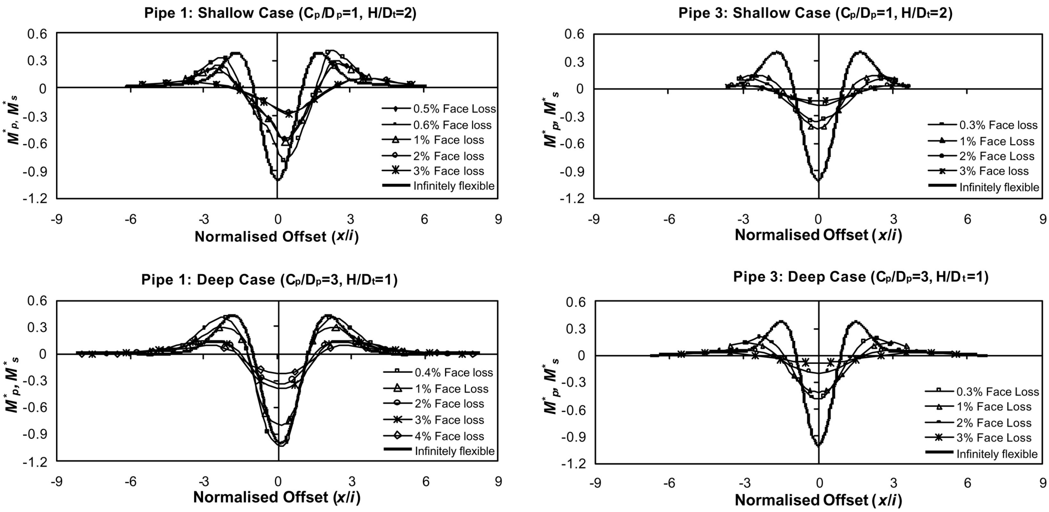

Figure 3. Normalised bending moment for Pipe 1 ($EI_{prototype} = 204$ MNm^2) and Pipe 3 ($EI_{prototype} = 52452$ MNm^2).

mechanism/s seems plausible to explain this observed change in R. **[3]** The crossover in [2] occurs earlier with increasing values of EI and EA. Pipeline response generally indicates increasingly flexible behaviour at low face loss, with M_p^* approaching M_s^*. This is followed by increasingly 'rigid' behaviour soon after when M_p^* deviates from M_s^* (e.g. shallow cases).

Ultimately, regardless of the pipe system tested, M_p^* becomes only a fraction of M_s^* at large face loss. **[4]** In normalized terms, it is possible for pipelines of varying orders of magnitude in stiffness, to exhibit similar normalized behaviour, albeit at different ground movement. This is shown for Pipes 1 and 3 in the shallow case. These pipes vary by two orders of magnitude in EI and an order of magnitude in EA. **[5]** The changing value of R is more pronounced in sagging than in hogging with smaller changes in M_p^* evident in hogging.

The observations imply that different mechanisms occur with increasing ground movement. These mechanisms influence pipeline responses. The onset and influence of such mechanisms vary with pipe system and the magnitude of ground movement achieved. These were experienced by every pipe system tested for the ranges of face loss shown in Figure 3.

4 MECHANISMS

It is useful to consider the development of pipeline behaviour as the result of a combination of global (effects not only confined to the vicinity of the pipe) and local effects (due to pipe-soil interaction). Global effects (denoted *Mechanism 1*) represent shearing

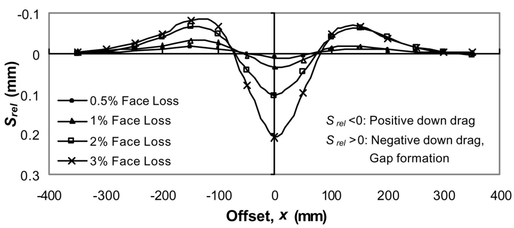

Figure 4. Relative pipe-soil settlement (fitted curves), Pipe 3.

caused purely by contraction of the tunnel cavity (i.e. as if a green field scenario is considered). Regardless of pipeline properties, this represents the minimum ground shear strain which should be taken into account when evaluating the effect of tunnelling on pipelines. Local effects on the other hand result in local shear strain increases over and above those caused by global effects. Further to Figure 3 the combined effect is illustrated when considering relative pipe-soil settlement (S_{rel}), defined as:

$$S_{rel} = S_s - S_p \tag{4}$$

where S_s is the green field subsoil settlement at a level corresponding to the pipe invert level and S_p is the pipe crown settlement. A typical example is shown in Figure 4 where Pipe 3 is subjected to the deep pipe depth case ($C_p/D_p = 3$, $H/D_t = 1$).

Behavioural patterns suggested in Figure 4 include gap formation and possible limit conditions achieved in positive down drag (pipe settles more than surrounding soil) and negative down drag (soil settles more than

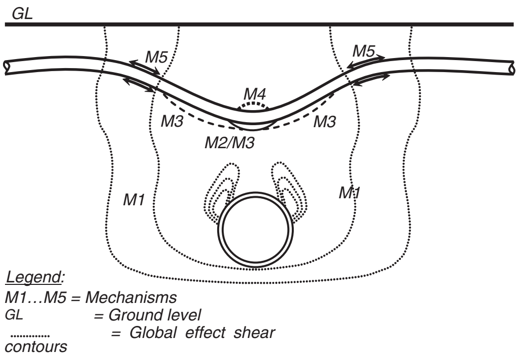

Figure 5. Schematic representation of global and local mechanisms and how they might impact on pipeline behaviour.

the pipe). By combining S_{rel}, bending moment and soil stress measurement possible local mechanisms could include the following, as illustrated in Figure 5: **[1]** Gap formation (*Mechanism 2*) **[2]** Decreased stability (*Mechanism 3*) **[3]** Negative down drag failure (*Mechanism 4*), and **[4]** Longitudinal interaction (*Mechanism 5*). Model pipeline behaviour in the centrifuge may therefore be regarded as the result of global shearing, the ability of a pipeline to distribute load through its stiffness and the formation of subsequent local mechanisms, which affect the relative pipe-soil rigidity. These mechanisms are described in more detail in the following sections.

4.1 *Global effects: Mechanism 1*

Green field response to tunnelling was investigated by a number of earlier authors, e.g. Potts (1976) on sand, Mair (1979) on soft clay and Grant (1998) on layered clay and sand. By considering shear distributions in green field soil close to failure (e.g. for sand from the work of Potts 1976) the contribution of global effects to total shear strain and how this contribution may be expected to change between soil surface and tunnel levels, may be visualized. It is also indicative of the expected direction and intensity of soil displacement vectors. Shear strains in the green field increase with depth below surface level and decreasing distance from the tunnel. The effect is reflected by the variation of the settlement trough width parameter, i, with depth (Mair et al. 1993). This parameter is convenient since it is widely accepted for describing the settlement profile as a Gaussian curve. The manner in which i develops provides insight into the impact of tunnel contraction at that particular depth.

During the current research the green field i was not only found to change with depth, but also with increasing ground movement. Similar observations were also made by Hergarden et al. (1996). This suggests an

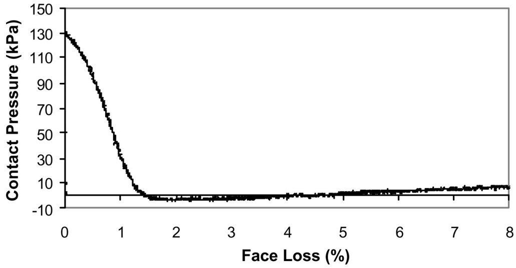

Figure 6. Gap formation confirmed by development of pipe-soil contact pressure (Pipe 3: Deep Case, $C_p/D_p = 3$, $H/D_t = 1$).

increased localization of ground movement as the tunnel cavity contracts, which in turn impacts on relative pipe-soil rigidity and the ability of local mechanisms to develop.

In addition, the state of the material also impacts on the shear contribution of *Mechanism 1* in particular the lateral extent of shearing. In loose sands and soft clays the shear zone is generally wider and strain levels higher than in dense sand (Potts 1976) and overconsolidated clay.

4.2 *Local effects*

4.2.1 *Gap formation: Mechanism 2*

In Figure 5 gap formation is shown to develop for Pipe 3 (deep pipe depth case). A rapid decrease in vertical contact pressure is shown at pipe invert level ($x = 0$ in Fig.3). This observation is considered with the gap formation indicated in Figure 4 based on the development of S_{rel}. The apparent discrepancy between Figures 4 and 6 in terms of the volume loss at which gap formation occurs can be explained by the finite thickness (1.02 mm) of the stress cell fixed to the perimeter of the pipe. From studying S_{rel} it follows that gap formation probably occurred for all three pipes tested, depending on the magnitude of ground movement at pipe level. Hachiya et al. (2002) also observed gap formation during a trapdoor experiment in the centrifuge on a model pipeline of similar EI to Pipe 1.

4.2.2 *Decreased stability: Mechanism 3*

In response to ground movement, horizontal (σ'_h) and vertical stresses (σ'_v) in positive down drag regions were measured by stress cells and found to increase. Combined with unloading during gap formation, the stability and stiffness of soil around the gap may be reduced (similar to a footing on a slope) possibly resulting in local failure. Part of the reduction in stability around the gap also arises from stress changes occurring below the gap.

The argument is explained by considering that during tunnelling in dense sand, arching is common,

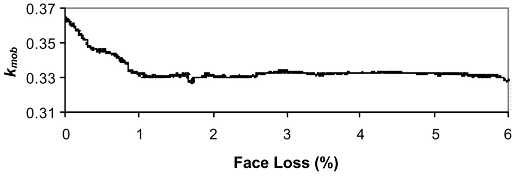

Figure 7. Lateral earth pressure coefficient mobilized below pipe (Pipe 3: Deep Case, $C_p/D_p = 3$, $H/D_t = 1$).

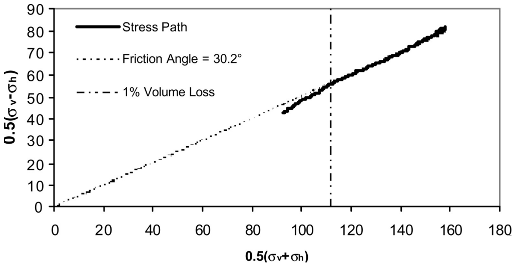

Figure 8. Lateral earth pressure coefficient mobilized below pipe (Pipe 3:Deep Case, $C_p/D_p = 3$, $H/D_t = 1$).

resulting in increased σ'_h above the tunnel centreline, while σ'_v decreases. Combined with unloading and gap formation, the stability of the soil below the gap may ultimately be affected in so far as it concerns the soil's ability to support itself when the tunnel cavity contracts (support pressure conditions such as observed by Atkinson & Potts 1977). This may be visualized by considering that, following gap formation, the body of soil between the pipe and the tunnel (below the gap), has less ability to develop the same extent of arching to support itself than soil at the same depth in the green field. Depending on the extent of stiffness reduction and subsequent reduced support stability around the gap perimeter, a slope-like failure adjacent to the gap may occur in a plane parallel to the pipe as shown schematically in Figure 4.

The mechanism is supported by the development of the mobilized lateral earth pressure coefficient, K_{mob}, for Pipe 3 (deep case) as shown in Figure 7. K_{mob} is defined as σ'_h/σ'_v, where σ'_h is measured in a plane transverse to the tunnel.

After the onset of ground movement, K_{mob} reduces steadily before stabilizing at a value of 0.33. This value corresponds to the active earth pressure coefficient, K_a, for a material with $\phi'_{crit} = 30°$ and is close to the value of $K_a = 0.31$ associated with $\phi'_{crit} = 32°$ (applicable to the sand used in this study). A plane strain stress path derived from stress cell measurements indicate stress conditions approaching critical state conditions in the region of maximum positive down drag (Fig. 8).

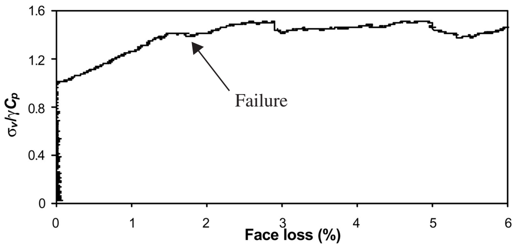

Figure 9. Normalized negative down drag response (Pipe 3, $C_p/D_p = 1$, $H/D_t = 2$).

Following local failure, the gap may partially close, resulting in the pipe coming into contact with the soil again and repeating the process of gap formation as ground movement increases. The notion is supported in Figure 6 where a gradual increase in stress is shown to develop at pipe invert level for larger values of ground movement.

4.2.3 *Negative down drag failure: Mechanism 4*

In addition to gap formation and decreased stability, pipelines may also experience negative down drag failure if relative pipe-soil rigidity is sufficiently high. Evaluating the development of S_{rel} for the pipes tested (e.g. Fig. 4) it is evident that $(S_{rel})_{max}$ generally occurs in negative down drag. Figure 9 shows the increase in negative down drag pressure measured (and ultimately failure) at the crown of Pipe 3 close to the tunnel centreline ($C_p/D_p = 1$, $H/D_t = 2$). The approximately 1.5 to 2% face loss associated with the negative down drag failure is not unlike face losses achieved in tunnelling practice. Furthermore, the form of the results is similar to pipe pullout models such as reported by Trautmann et al. (1985) and others.

4.2.4 *Longitudinal pipe-soil interaction: Mechanism 5*

Attewell et al. (1986), and later Bracegirdle et al. (1996), pointed out that pipelines are usually more vulnerable in hogging than sagging as a result of the contribution of longitudinal interaction to total tensile strain. This observation was made in the light of work performed primarily on small diameter pipelines. Changes in pipe strain, however, are a function of soil properties, problem geometry, pipe sectional properties (*EI* and *EA*), pipe coating and the state of the pipe, which all contribute to the development of pipe-soil interface shear (Attewell et al. 1986; Scarpelli et al. 2003). In conjunction with the effects of global and local mechanisms (such as gap formation causing reduced pipe-soil contact) a possible scenario exists where total tensile pipe strains may not always be critical in regions of maximum hogging. This is especially true if the distinction is made between a conservative design approach and monitoring. Since the strain contribution from longitudinal interaction results in decreased tensile strain in the sagging region (where tensile bending strains are large), a decision should be taken by the engineer on whether taking account of longitudinal pipe-soil interaction may indeed be necessary (or even conservative).

This hypothesis was tested by applying Attewell et al.'s (1986) method to estimate strains due to longitudinal interaction (no pipe-soil slippage is allowed) and combining these values with bending strains measured in the centrifuge. For the purpose of design, the strains were combined in hogging regions only. For monitoring purposes on the other hand, the total contributions of bending and longitudinal interaction in both hogging and sagging regions were considered. Preliminary indications are that: **[1]** Relative pipe soil bending rigidity rather than relative longitudinal stiffness determines the criticality of a combination of longitudinal interaction and bending strains, versus taking account of bending strains only for design and monitoring. **[2]** The more flexible the pipe-soil system (rather than the pipe itself), the more important the contribution of longitudinal interaction is to both design and monitoring. **[3]** Since the relative rigidity is a function of pipe and soil characteristics, as well as global and local mechanisms, these factors should be taken into account when deciding whether longitudinal interaction should be considered in design and monitoring. Work is currently underway to further investigate the influence of longitudinal interaction.

5 FURTHER RELATED WORK

Guidelines for implementing the discussed mechanisms into design and monitoring of continuous pipelines will be published in future. Furthermore, it is known that understanding the effect of pipe joints on pipeline behaviour is fundamental in the design of jointed pipelines and as such may influence the occurrence of pipe-soil interaction mechanisms. Work is currently underway to investigate this.

6 SUMMARY AND CONCLUSIONS

Normalized bending moment and relative pipe-soil settlement responses of continuous pipelines when subjected to tunnelling transversely to the pipelines have been presented. By examining pipeline responses and considering these in conjunction with stress and strain distributions in the ground it was found that pipeline behaviour may be affected by global (ground movement due to tunnelling as represented by the green field soil response) and local mechanisms (due to pipe-soil interaction). Local mechanisms include gap formation, decreased stability, negative down drag failure and longitudinal pipe-soil interaction. Applying these mechanisms may vary between design and monitoring since the respective objectives may differ. Future work will provide guidance on how to apply these mechanisms to design and monitoring. Work is also underway in examining the effect of joints on pipeline behaviour when subjected to tunnel-induced ground movement.

ACKNOWLEDGEMENT

The authors wish to express their sincere gratitude to the Cambridge-MIT Institute (CMI) for sponsoring the research described in this paper.

REFERENCES

Atkinson, J. H. & Potts, D. M. 1977. The stability of a shallow circular tunnel in cohesionless soil. *Geotechnique*, 27(2), pp 203–215.

Attewell, P. B., Yeates, J. & Selby, A. R. 1986. *Soil movements induced by tunnelling and their effects on pipelines and structures*, Blackie and Son Ltd, UK.

Bracegirdle, A., Mair, R. J., Nyren, R. J. & Taylor, R. N. 1996. A methodology for evaluating potential damage to cast iron pipes induced by tunnelling. *Geotechnical Aspects of Underground Construction in Soft Ground*, Mair & Taylor (eds). Balkema, Rotterdam, pp 659–664. ISBN 90 5410 856 8.

Clemmitt, I. 2003. Channel Tunnel Rail Link section 1: Diversion of utilities. *ICE Civil Engineering, CE156 (6).*

Fujita, K. 1994. Soft ground tunnelling and buried structures. *Proceedings of the Thirteenth International Conference on Soil Mechanics and Foundation Engineering, New Delhi, India*, pp 89–108.

Grant, R. J. 1998. *Movements around a tunnel in two-layered ground*. PhD dissertation, City University, London, UK.

Hachiya, M., Inoue, Y., Tohda, J., Takatsuka, Y. & Takagi, R. 2002. Response of buried pipelines subjected to differential ground settlement. *Physical Modelling in Geotechnics: ICPMG '02,* Phillips, Guo & Popescu (eds), Swets & Zeitlinger Lisse, ISBN 90 5809 389 1.

Hergarden, H. J. A. M., Van der Poel, J. T. & Van der Schrier, J. S. 1996. Ground movement due to tunnelling: Influence on pile foundations. *Proceedings of the International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground*. R. J. Mair and R. N. Taylor (Eds), pp 519–524.

Jacobsz, S. W. 2002. *The effects of tunnelling on piled foundations*. PhD dissertation, University of Cambridge, UK.

Klar, A., Vorster, T. E. B., Soga, K. & Mair, R. J. 2004. *Soil-Pipe-Tunnel Interaction: Comparison Between Winkler and Elastic Continuum Solutions*. Rep. No. CUED/D-SOILS/TR 332, Engineering Department, University of Cambridge, UK.

Lee, S. W. 2001. The *Effects of Compensation Injections on Tunnels*, PhD Dissertation, University of Cambridge.

Mair, R. J. 1979. *Centrifuge modelling of tunnel construction in soft clay*. PhD dissertation, University of Cambridge.

Mair, R. J., Taylor, R. N. & Bracegirdle, A. 1993. Subsurface settlement profiles above tunnels in clays. *Geotechnique*, 43(2), pp 315-320.

O'Rourke, T. D. & Trautmann, C. H. 1982. Buried Pipeline Response to Tunnel Ground Movements. *Europipe 82 Conference*, Basel Switzerland, paper 1.

Potts, D. M. 1976. *Behaviour of lined and unlined tunnels in sand.* PhD dissertation, University of Cambridge, UK.

Potts, D. M. & Addenbrooke, T. I. 1996. The influence of an existing surface structure on the ground movements due to tunnelling. *Geotechnical aspects of underground construction in soft ground*. Mair & Taylor (eds), Balkema, Rotterdam. ISBN 90 5410 856 8.

Scarpelli, G., Sakellariadi, E. & Furlani, G. 2003. Evaluation of soil-pipeline longitudinal interaction forces. *Rivista Italiana Di Geotecnica*, 4.

Trautmann, C. H., O'Rourke, T. D. & Kulhawy, F. H. 1985. Uplift force-displacement response of buried pipe. *Journal of Geotechnical Engineering*, Vol 111, No 9, pp 1061–1076.

Vorster, T. E. B. 2002. *The effect of tunnelling on buried pipelines and tunnels*. First year report for the registration for the degree of Doctor of Philosophy, University of Cambridge, UK.