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Responses of pipeline to side-by-side twin tunnelling at different depths: 3D centrifuge tests and numerical modelling

Shaokun Ma, Yu Shao, Ying Liu*, Jie Jiang, XiaoLing Fan

College of Civil Engineering and Architecture, Guangxi University, Nanning, China

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ABSTRACT

The tunnel-pipeline-soil interaction has gotten momentum recently, but most of the previous studies were focused on the responses of pipeline to single tunnelling and simplified the interaction as a plane strain problem. Up to date, there is still a lack of research on the responses of a pipeline to twin tunnelling at different depths. This study presents a series of three-dimensional centrifuge model tests which were performed to investigate the effects of side-by-side twin tunnelling at varying depths on an existing buried pipeline in dry sand. In the centrifuge model tests, both the volume loss and the weight loss were simulated in-flight using a novel technique. To gain better understanding of the tunnel-pipeline interaction, the centrifuge tests were back-analysed numerically based on an advanced hypoplastic model, which accounts for strain- and path-dependent soil stiffness at small strains. Based on the physical and numerical investigations, it was revealed that the distribution of pipeline maximum shear force is greatly affected by the second tunnelling. The shifted distance of the pipeline maximum shear force induced by the second tunnelling in the case of cover-to-diameter ratio (C/D) of 2 was 80% larger than that in the case of C/D of 4. The major influence zone on the pipeline induced by tunnelling is within 1.2D (tunnel diameter) ahead of and after the tunnel face. When the pipeline is located within the major influence zone due to tunnelling, there was a sharp increase in the relative pipeline-soil stiffness and the additional bending strain in the pipeline. The tests results were compared with the results from a dimensionless chart and an analytic solution in the literature. Although these two approaches can reasonably predict the bending strain in the pipeline due to the first tunnelling, they fail to accurately capture the incremental bending strain solely caused by the second tunnelling. This implies that one should be cautious in using the superposition principle to predict the bending strain in a pipeline subjected to twin tunnelling. Because the deviatoric strain of the soil around the pipeline was increased by the second tunnelling, leading to further degradation of the soil stiffness around the pipeline.

1. Introduction

Tunnel excavation, which inevitably causes the ground movements and alters the stress distributions in the soil, often leads to additional stress and deformation of the existing adjacent structures, such as pipelines. The serviceability of the buried pipelines due to tunnelling has been seriously concerned recently, especially for the pipelines being close to or having deteriorated to a critical state (Royse et al., 2009; AWWA, 2012; Folkman, 2012).

The tunnel-pipeline-soil interaction has been studied in the past decades. Attewell et al. (1986) treated this problem on the basis of the Winkler type model, in which the soil is represented as a series of independent linear springs. Despite the popularity of Vesic (1961)'s method for determining the spring coefficients, the Winkler type model is not necessarily adequate for the tunnel-pipeline-soil interaction (Klar et al., 2005). An alternative subgrade model was proposed, which was based on a comparison to an elastic-continuum solution. Yu et al. (2013) suggested a more appropriate subgrade model, which reasonably predict the tunnelling-induced bending moment and axial load along the pipeline. The elastic-continuum solution was also applied to the problem of tunnelling effects on jointed pipelines (Klar et al., 2008).The results obtained from the elastic-continuum were compared with a more rigorous numerical analysis in which the pipeline was represented by the shell elements (Klar and Marshall, 2008). Based on the elastic-continuum solution, Klar and Marshall (2015) reported that the volume loss deduced from the greenfield displacement input and that the resulting deformed pipeline shape were equal. The shape of the deformed pipeline can be fitted by using the Gaussian distribution. Based on that, a simple expression for the prediction of pipeline bending moment was estimated on the basis of the two features.

* Corresponding author.
E-mail addresses: mashaokun@sina.com (S. Ma), shaoyu_1987@126.com (Y. Shao), lyily1112@126.com (Y. Liu), jie_jiang001@126.com (J. Jiang), 18176260802@163.com (X. Fan).

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Incorporating soil nonlinearity into the analytic solution is another main direction of current research. Vorster et al. (2005) proposed an equivalent linear approach, in which the stiffness of the soil was evaluated based on an average deviatoric strain due to greenfield settlements. Marshall et al. (2010) reported that the shear strain of the soil around a pipeline is not only due to the greenfield settlements but also due to the pipeline-soil interaction. On the basis of their reports, more accurate soil stiffness was estimated. In order to address the difficulty of evaluating the relative displacement between the pipeline and the greenfield settlement at the pipeline level, Klar et al. (2015) derived a set of simplified close-form solutions based on the previous research findings. Meanwhile, a calculation procedure of the maximum bending moment in the pipeline due to tunnelling was also given. To improve the accuracy of prediction, Saiyar et al. (2016) suggested that incorporating soil nonlinearity for the pipeline-soil interaction is more important than accounting for more realistic ground displacement profiles.

As another important analytical tool, numerical simulation was often applied for dealing with the problem of tunnel-pipeline-soil interaction (Marshall et al., 2010; Klar et al., 2007). Wang et al. (2011) conducted a series of numerical parametric studies to investigate the responses of the pipeline due to tunnelling. A dimensionless plot was developed for estimating the maximum pipeline bending strain.

Marshall et al. (2010) and Vorster (2005) carried out a series of centrifuge tests to investigate the effects of tunnelling with different volume losses on adjacent pipelines. Ground settlement, pipeline settlement, bending moment of the pipeline and the shear strain of soil around the pipeline were reported and discussed.

Previous researchers have accomplished a series of prominent and outstanding achievements on the topic of tunnel-pipeline-soil interaction. With the growing utilization factor of underground spaces in cities, twin adjacent tunnels or even multiple tunnels are increasingly favored and constructed (Ng et al., 2014; Hong et al., 2015). On the other hand, there is a still lack of systematic research on the tunnelling-induced influence zone around a pipeline and the responses of the pipeline to twin tunnelling at different depths. For these reasons, a series of three-dimensional centrifuge model tests and numerical back-analysis were carried out to investigate the effects of side-by-side twin tunnelling at different depths on an existing buried pipeline in medium dense dry sand. An advanced hypoplasticity constitutive model was adopted to capture the state-dependent dilatancy, and strain- and path-dependent stiffness at small strains. Meanwhile, a displacement controlled finite element model (DCM) based on ground loss was also applied for the numerical simulations. Ground settlements, pipeline settlements, bending strain in pipeline, and the mechanism of tunnel-pipeline-soil interaction were presented and discussed. In addition, the results were compared with the results obtained from a published dimensionless chart and an analytical solution, and discussed with their practical implications.

2. Three-dimensional centrifuge modelling

2.1. Experimental program and setup

Two centrifuge model tests were carried out at the Geotechnical Centrifuge Facility of the Hong Kong University of Science and Technology (Ng et al., 2002, 2001). Fig. 1 shows a plan view of the two centrifuge model tests, which were carried out at 60*g* (*g* is the gravitational acceleration). The pipeline was modeled by a multi-functional 6010 aluminum tube with Young's modulus of 69 GPa. The outer diameter, thickness, and length of the tube were 31.75 mm, 2.08 mm and 1150 mm, respectively. According to the scale factors summarised in Table 1 (Taylor, 2003), the dimensions of the model pipeline were equivalent to 1.905 m, 0.126 m and 69 m in prototype, respectively. Since the length of the model box was 1245 mm and there was 47.5 mm space between each end of the pipeline and the side wall, the boundary effect was anticipated to be minimized. Based on the suggestion proposed by Jacobsz et al. (2004), the pipeline was long enough to exceed the influence zone of tunnelling in sand. Each end of the hollow pipeline was sealed to prevent soil flow (which may lead to additional volume loss) entering into the pipe.

Fig. 2 shows the elevation view of the two tests. The cover-to-diameter (C/D) ratios of the two tests were 2 and 4. In both tests, the vertical distance from the ground surface to the crown of the pipeline was 80 mm, which was equivalent to 4.8 m in prototype. The pillar depth-to-diameter ratios (P/D, where P is the vertical distance from the invert of the pipeline to the crown of the tunnel) were 2.9 and 0.9, respectively. The distance from the invert of the tunnel to the bottom of the strongbox in the two tests were 2.5 and 4.5D, which were sufficient to minimize the boundary effects.

2.2. In-flight simulation of twin tunnelling

The horizontal clearance distance between the two parallel-aligned tunnels (excavated one after the other) was 1D. Each tunnel consisted of six sections, which represented six excavation steps. The advancing distance in each step was 0.6D. Fig. 3 shows a cross-section of a donut and the tunnel model. This tunnel model had been adopted by Ng et al. (2013) to investigate the effects of tunnelling on the deformation of an adjacent tunnel. The effects of the volume loss and the weight loss can be simulated simultaneously. The donut consisted of three components, including an outer rubber membrane, an inner rubber membrane, and an aluminum alloy tube serving as the tunnel lining. Each inner and outer rubber membrane was filled with heavy fluid ($ZnCl_2$) having the same density as the soil. Then, the two drainage tubes for heavy fluid of the inner and outer rubber membrane were connected by a two-way joint. During the tests, the heavy fluid was drained from both the inner and outer membrane. The drainage was sequentially regulated through twelve pneumatic valves. Volume loss equivalent to 2% was simulated by releasing the heavy fluid from the outer rubber membrane. Similarly, the effect of weight loss was simulated through draining out the heavy fluid from the inner rubber membrane. As shown in Fig. 1, the heavy fluid drained from the inner and outer rubber membrane was collected in a reservoir. The aluminum alloy tube serving as the tunnel lining was adopted to prevent the soil from moving towards the tunnel face. The diameter and thickness of the aluminum tube were 100 mm and 3 mm, which were equivalent to 6 m and 0.18 m in prototype scale. If the compressive strength of concrete was assumed to be 50 MPa, then the elastic modules was to be 33 GPa (ACI, 2011). According to the scale factors (Taylor, 2003), the thickness of the tunnel model lining was converted to that of concrete with equivalent flexural stiffness. In the prototype scale, the tunnel model thickness were thus equivalent to 420 mm and 230 mm in the longitudinal and transverse directions. Each end of the model tunnel was sealed to prevent the soil from moving into the tunnel.

2.3. Model preparation

Dry Toyoura sand, which consists of sub-angular particles, was adopted for the two centrifuge model tests. The mechanical properties of the Toyoura sand are shown in Table 2. The coefficient of the lateral pressure at rest was 0.5, which is calculated by the equation of Jáky (1944) with effective friction angle at critical state ($\varphi'_{cr} = 30°$). As shown in Fig. 4, the two stiffness degradation curves were obtained from two triaxial tests on isotropically consolidated and K_0 consolidated sand (Yamashita et al., 2000).

Dry pluviation technique was adopted in the two tests to prepare the sandy ground. The drop height and rate of the pluviation were 600 mm and 100 kg/h, respectively. Table 3 shows that the soil densities of the two test samples were 1549 kg/m^3 and 1542 kg/m^3. The density difference fell within the acceptable rang (± 0.5% or ± 8 kg/m^3)