Journal of Engineering Science and Technology Review 9 (1) (2016) 167-173

JOURNAL OF
Engineering Science and Technology Review

www.jestr.org

Research Article

Effect of Double-line Parallel Shield Excavation on Adjacent Underground Pipelines

G. Wei[1*], S. Hu[1], J.J. Xing[2] and Q. Ye[3]

[1] *Department of Civil Engineering, Zhejiang University City College, Hangzhou 310015, China*
[2] *Institute of Geotechnical Engineering, Zhejiang University, Hangzhou 310058, China*
[3] *Pypun Engineering Consultants Ltd., Hong Kong*

Received 28 January 2016; Accepted 10 February 2016

Abstract

Using a formula for the total settlement of deep soil and the Winkler foundation model, the force model for underground pipelines is simplified to obtain the ultimate bending moments and to calculate the stress and strain due to double parallel shield tunneling construction. The effects of soil conditions, tunnel horizontal spacing L, pipeline material, pipeline depth h and soil loss ratio η l/ η f on the force on an underground pipeline are investigated. The results show that changing the soil conditions and value of L can have a significant effect on the shape and magnitude of the pipeline's ultimate moment curve, although changing the value of h has less of an effect. It is found that the larger the bending stiffness of the pipeline, the larger the distribution range of the ultimate bending moment and its peak value, but there is less of an effect on the shape of the curve. Changes in the ratio η l/ η f have some influence on both the shape of the curve and the values of the bending stiffness of the pipeline.

Keywords: Double-line parallel shield, Underground pipelines, Ultimate moment, Soil settlement

1. Introduction

In recent years, there has been an unprecedented development of subway construction. This mostly uses double-line horizontal parallel shield tunneling, which causes a greater amount of settlement and produces a wider settlement trough than single-line shield tunneling. The effects of double-line parallel shield tunneling on underground pipelines (hereinafter referred to simply as pipelines) differ considerably from those of single-line shield tunneling [1], [2], [3]. Therefore, an investigation of the effects of double-line parallel shield tunneling on adjacent pipelines is of great importance.

In this study, a modified two-dimensional (2D) Peck equation is used to calculate the vertical ground displacement at the pipe surface orthogonal to the direction of double-line horizontal parallel shield tunneling. Based on the Winkler foundation model, the ultimate bending moment, stress and strain on the pipeline due to tunneling are determined. Example calculations are performed to analyze the effects of tunneling on the pipeline.

The principal approaches to this problem have included numerical analysis [4], [5], [6] and field measurements [7], [8], but an analytical solution is lacking. Deformation of a pipeline due to single-line tunneling is generally represented by a symmetric normal distribution and is relatively simple. However, deformation due to double-line tunneling is complicated, being represented by an asymmetric distribution, since two conditions occur, with "V" and "W" shaped distributions, respectively. Therefore, there is an urgent need for an analytical solution to evaluate the effect of double-line horizontal parallel tunneling on adjacent pipelines.

There have been numerous studies on the effect of single-line tunneling on pipelines. The principal methods employed have included analytical solutions [4], [9], [10], [11], [12] among other approaches. Wu et al. [4] proposed an analytical solution for the displacement of a pipeline using the theory of beams on an elastic foundation, but their calculation was rather complex. Attewell et al. [9] simplified the problem using the Winkler foundation model and provided analytical solutions. Vorster et al. [10] provided a continuous elastic solution, the viability of which was validated by a centrifuge model experiment. However, the equation provided only an upper limit solution that was conservative, and the calculation was complex. Wang et al. [13] analyzed the force on a pipeline orthogonal to the direction of tunnel excavation using the Loganathan equations on the basis of the Winkler foundation model in an attempt to remedy the deficiencies of the Attewell et al. solution. With regard to the deficiencies of the Wang et al. solution, Wei and Zhu [14] proposed a theoretical approach based on the Winkler foundation model that allowed calculation of the ultimate bending moment and the deformation of a pipeline due to pipe jacking excavation. Zhang and Zhang [15] proposed an equation describing the vertical displacement and internal force on a pipeline using the Pasternak foundation model, thus addressing the inherent drawbacks of the Winkler model. Taking into account the lack of continuity of pipeline connections, Zhang et al. [16] proposed an analytical approach to the displacement and bending moment of a pipeline based on a modified Winkler foundation model. However, all of the studies mentioned here dealt with a single-line tunnel only, with no consideration of double-line horizontal parallel tunnels. To date, very few studies have been conducted on the effect of double-line tunneling on pipelines.

* E-mail address: weig@zucc.edu.cn
ISSN: 1791-2377

2. Methodology

2.1 Analysis of pipeline deformation and its mechanism of action

The effects on the ground foundation due to double-line horizontal parallel shield tunneling can be attributed to ground loss, friction at the shield, bulkhead additive thrust, grouting force, friction at the cutter head, etc., with ground loss being the primary factor. Because the range of impact of bulkhead additive thrust and friction at the shield is limited to the vicinity of the excavating face of the tunnel and because the induced ground surface displacement beyond a certain distance from the excavating face is basically zero, ground loss alone needs be considered in the present context [17].

To simplify the calculation, only the force and displacement due to ground loss are evaluated in this study. An analysis shows that the force has the greatest effect when the pipeline is orthogonal to the direction of tunneling, and therefore only this condition is evaluated here. The effect of double-line parallel shield tunneling on the pipeline is shown in Fig. 1, in which d denotes the outer diameter of the pipe, h the depth of the centerline of the pipe from the ground surface, H the burial depth of the centerline of each tunnel, R the excavating radius of each tunnel and L the horizontal distance between the centerlines of the two tunnels. The ground deformation caused by double-line horizontal parallel shield tunneling is rather complex, and the ground surface settlement curve exhibits two conditions produced by variation of H, L and R: a "W" shape and a "V" shape. The resulting deformation of the pipeline, characterized by variations in h, H, L and R, therefore also exhibits both "W" and "V" shaped conditions, as shown in Fig. 1(a) and 1(b).

(a)

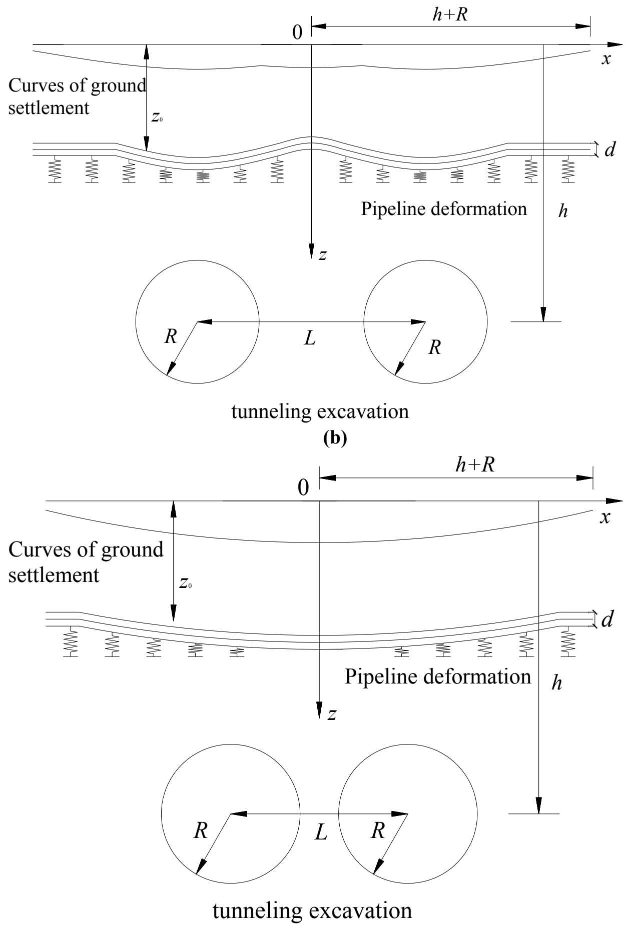

Fig.1. Effects of double-line parallel shield construction on an underground pipeline

Wei and Pang [18] evaluated the variation of the ground surface settlement curve due to double-line parallel shield tunneling and proposed that the shape of the curve is related to L and $H + R$. Let $C = L/(H + R)$ be the relative horizontal distance factor of the tunnel: when $C \leq 0.66$, the ground surface settlement curve is "V" shaped (i.e., a normal distribution); when $C > 1$, the curve is "W" shaped; in the range $0.66 < C \leq 1$, there is a transition from a "V" shaped to a "W" shaped curve.

The pipeline is closer to the tunnel than to the ground. Therefore, in this study, the relative horizontal distance factor of the pipeline is taken as $G = L/(H – h + R)$; when $G \leq 0.66$, the settlement curve of the pipeline is "V" shaped (a normal distribution) and when $G>1$, the curve is "W" shaped, with a transition from a "V" shaped to a "W" shaped curve in the range $0.66 < G \leq 1$.

For validation, we consider an example. Sun et al. [7] measured the settlement of a pipeline due to the construction of a double-line shield tunnel. The settlement of the pipeline appeared to be "V" shaped, but it did not match a normal distribution. The maximum settlement deviated to the side of the first tunnel. The method proposed in this study was used for validation, with parameters $L = 13.2$ m, $H = 15.12$ m, $R = 3.17$ m and $h = 0.9$ m. A calculation gave $G = 0.76$, which is slightly larger than 0.66 and smaller than 1. Therefore, the settlement curve did not fit a normal distribution but was in the transition region from a "V" to a "W" shape, although closer to the former. The results of this validation calculation match favorably with the measured results, and the reliability of the proposed method is validated.The mechanism of the interaction between pipeline and ground due to double-line parallel shield tunneling is similar to that in the case of single-line shield tunneling, as explained by Wei [14].

Assuming that no separation occurs between the pipeline and the surrounding ground, the settlement of the ground in the plane of the pipeline leads to additive pressure on the pipeline. As both tunnels are completed, the settlement of the pipeline gradually reaches its maximum value and so does the additive pressure, in which case, the pipeline is in the least favorable condition. In this study, the force characteristics are evaluated under this condition. It is assumed that when the pipeline is in the undeformed state, the bending moment of the pipeline is at its maximum, known as the ultimate bending moment. This can be used as a comprehensive indicator for evaluating pipeline deformation. The larger the ultimate bending moment, the greater the possibility that the pipeline will be damaged.

2.2 Assumptions and simplification of the force model of the pipeline

The assumptions and simplification of the force model of the pipeline proposed by Wei [14] are adopted in this study, although the details will not be discussed here because of space limitations. To determine the ultimate bending moment of the pipeline, the ground surface settlement in the plane of the pipeline needs to be found.

Let the vertical displacement of the ground in the plane of the pipeline (the location of the centerline of the pipeline; the same meaning is used throughout this paper) due to double-line parallel shield tunneling be $S_z(x)$. This can be viewed as the unloading process of the ground in the plane of the pipeline, and thus the force on the pipeline can be expressed as $P = kS_z(x)d$, where k is the reaction force factor of the foundation in the plane of the pipeline, which can be further modified after experimental determination of the reaction force factor of the foundation, k_0, from loading plate tests. The reaction force factor k_0 can be related to the stiffness and elastic characteristics of the foundation. Vesic [19] proposed the following expression for the reaction force factor of the foundation for a long beam (with $l/b>10$, where l and b are, respectively, the length and width of the beam):

$$k' = \frac{0.65E_0}{b(1-\mu^2)}\left(\frac{E_0 b^4}{EI}\right)^{\frac{1}{12}} \quad (1)$$

where E_0 is the modulus of deformation of the ground, μ is Poisson's ratio of the ground, E is the elastic modulus of the pipeline, EI is the bending stiffness of the pipeline and the width of the foundation beam, b is taken as equal to d.

Taking account of the effect of the burial depth of the pipeline, Attewell et al. [9] proposed that the reaction force factor of the foundation in the plane of the pipeline is $2k'$, which is assumed in this study.

2.3 Calculation of ground surface settlement in the plane of the pipeline

Ground surface settlement in the plane of the pipeline due to double-line parallel shield tunneling can be precisely described by the amended 2D Peck equation proposed by Chen et al. [20]. The total ground surface settlement is taken to be equal to the sum of the ground surface settlements due to each tunnel. The equation for the settlement at any point (x, z) of the ground is

$$S_z(x) = \frac{S_{\max,f}}{(1-z/H)^n}\exp\left[\frac{-(x-0.5L)^2}{2i_f^2(1-z/H)^{2n}}\right] + \frac{S_{\max,l}}{(1-z/H)^n}\exp\left[\frac{-(x+0.5L)^2}{2i_l^2(1-z/H)^{2n}}\right] \quad (2)$$

where $S_{\max,f} = \frac{\pi R^2 \eta_f}{i_f\sqrt{2\pi}}$, $S_{\max,l} = \frac{\pi R^2 \eta_l}{i_l\sqrt{2\pi}}$, $S_{\max,f}$、$S_{\max,l}$ are the maximum ground surface settlements above the centerline due to the excavation of the tunnels, x is the horizontal distance to the central axis of the double-line shield, z is the vertical distance to the ground surface, η_f and η_l are the ground losses produced by each tunnel, i_f and i_l are the factors of the ground surface settlement troughs of each tunnel and n is the settlement trough parameter. When solving the above equation, a total of five parameters need to be determined: η_f、i_f、η_l、i_l and n.

From a statistical analysis of the measured values of ground surface settlement due to tunneling in different geologic conditions, O'Reilly and New [21] proposed that the factor of the ground surface settlement trough i can be calculated from the empirical equation $i = mH$, where m is a factor related to the earthiness. For sandy soil, m = 0.2–0.3; for clay soil and soft soil, m = 0.7; for medium clay soil, m = 0.5; and for hard clay soil, m = 0.4. The value of i_f can thereby be obtained [22]. The specific value of η_f has been found by Wei [23].

According to Chen et al. [20], i_l / i_f=0.97, η_l/η_f=0.96 and n should be in the range 0.35–0.85 for clay soil and 0.85–1.0 for sandy soil.

2.4 Solution for the force on the pipeline based on the Winkler model

As proposed by Wei and Zhu [14], when the stiffness of the pipeline is not large, the Winkler foundation model can be used to calculate the force on the pipeline. The differential equation describing the deformation of the pipeline due to tunneling is

$$\frac{\partial^4 w}{\partial x^4} + 4\beta^4 w = 4\beta^4 S_z(x) \quad (3)$$

where w is the vertical deflection of the pipeline and $\beta = \sqrt[4]{kd/4EI}$.

For an infinite beam, when a concentrated load P is applied at a point, the bending moment produced at a distance x from the loading point is

$$M = \frac{P}{4\beta}\exp(-\beta x)(\cos\beta x - \sin\beta x) \quad (4)$$

It is evident from the force on the pipeline equivalent that the infinitesimal concentrated load at a distance x from the center of the pipeline is $dP = dk\, Sz(x)\, dx$.

Assume that the origin of coordinates is taken above the central axis of the double-line parallel tunnel, as shown in Fig. 1. It is also the vertical center of the pipeline. From the equations above, the ultimate bending moment at the vertical center of the pipeline is

$$M\max = \int_{-\infty}^{\infty} dM(x) = \int_{-\infty}^{\infty} \frac{kS_z(x)d}{4\beta}\exp(-\beta x)(\cos\beta x - \sin\beta x)dx \quad (5)$$

The ground surface settlement in the plane of the pipeline can be calculated from Equation (2). The ultimate bending moment M_L at any point (x_0, h) of the pipeline in the region affected by tunneling can be calculated as

$$\begin{aligned} M_L =& EI\beta^3 \frac{\pi R^2\eta_f}{i_f\sqrt{2\pi}}\left(1-\frac{h}{H}\right)^{-n} \cdot \int_{-\infty}^{+\infty}\left(\cos\left(\beta|x-0.5L-x_0|\right) - \sin\left(\beta|x-0.5L-x_0|\right)\right)\cdot \\ & \exp\left[\frac{-(x-0.5L)^2}{2i_f^2(1-h/H)^{2n}} - \beta|x-0.5L-x_0|\right]dx \\ & + EI\beta^3\frac{\pi R^2\eta_l}{i_l\sqrt{2\pi}}\left(1-\frac{h}{H}\right)^{-n}\cdot\int_{-\infty}^{+\infty}\left(\cos\left(\beta|x+0.5L-x_0|\right)-\sin\left(\beta|x+0.5L-x_0|\right)\right) \\ & \cdot\exp\left[\frac{-(x+0.5L)^2}{2i_l^2(1-h/H)^{2n}} - \beta|x+0.5L-x_0|\right]dx \end{aligned} \quad (6)$$

The ultimate stress on the pipeline can be calculated as

$$\sigma = \frac{M_L}{W} = \frac{32M_L d}{\pi\left(d^4 - d'^4\right)} \quad (7)$$

where d' is the inner diameter of the pipeline. The ultimate strain produced on the pipeline can be calculated as

$$\varepsilon = \frac{\sigma}{E} = \frac{32M_L d}{\pi E\left(d^4 - d'^4\right)} \quad (8)$$

If only the first tunnel is considered ($S_{\max,l}$=0), then the method discussed in this paper can be used for a single-line tunnel.

2.5 Assessment of the safety of the pipeline

It is proposed in this study that the pipeline is first affected by a single-line tunnel (i.e. the first tunnel) and then by the second tunnel. Therefore, to determine the safety of a pipeline affected by double-line parallel tunneling, two safety assessments should be conducted, first for a single-line tunnel (i.e. the first tunnel) and then for the double-line parallel tunnel.

When the stiffness of the pipeline is large, it can be assumed that no deformation occurs at the pipeline. Therefore, Equations (6), (7) and (8) can be used to calculate the ultimate bending moment, stress and strain on the pipeline, and safety can be evaluated in terms of the allowable stress and strain.

When the stiffness of the pipeline is small, it can be assumed that the settlement of the pipeline is equal to the ground surface settlement in the plane of the pipeline due to double-line shield tunneling. Equation (2) is used for this calculation. Once the settlement curve of the pipeline has been obtained, the maximum angle of rotation of the pipeline can be calculated and the safety evaluated in terms of the allowed angle of rotation.

The method proposed in this study is conservative. It is assumed that the pipeline is orthogonal to the direction of the double-line parallel tunnel, that the ground is homogeneous (layering of the ground is not considered) and that only ground loss is significant.

3. Results

3.1 Analysis of example calculations

The following conditions are assumed. This is a double-line horizontal parallel shield tunneling project with an excavating radius R = 3.17 m, the burial depth of the tunnel centerline H = 12 m, the horizontal distance between the centerlines of the two tunnels L = 12 m and the right tunnel is excavated first. The tunnels are excavated in clay soil with a Poisson's ratio of 0.35, a modulus of deformation E_0=3087.6 kPa and a unit weight of 18 kN/m^3. Assuming that η_f =1.56% owing to the excavation of the first tunnel, the width of the ground surface settlement trough can be calculated using the approach of O'Reilly and New [21] as i_f = 6 m. According to Chen et al. [20], η_l / η_f=0.96, and it is clear that η_l=1.50%. If i_l / i_f = 0.97, then i_l=5.8 m. A value of n = 0.4 is taken.

There is an underground rigid steel pipeline orthogonal to the direction of excavation of the tunnel, lying above the tunnel. The outer radius of the pipeline is 0.4 m, the centerline is a depth h = 1.5 m below the ground surface, the tube thickness is 16 mm, E = 2.06×10^5 MPa and the bending stiffness EI =1 246 300 kN·m^2. The reaction force of the foundation is calculated from Equation (1) and doubled, so that k=3219 kN/m^3. It is calculated that β=0.151.

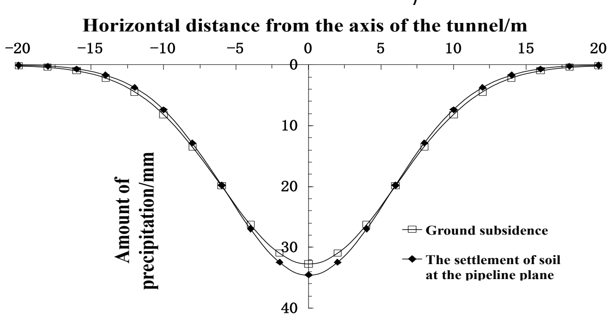

Fig.2. Soil settlement due to single shield construction

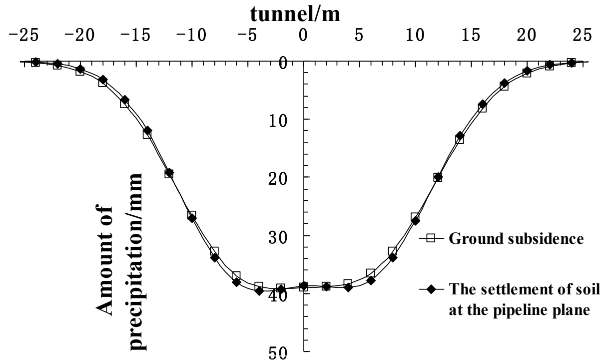

Fig.3. Soil settlement due to double-line parallel shield construction

Fig. 2 shows the ground surface settlement and ground settlement in the plane of the pipeline due to single-line shield tunneling using the parameters (η_f , i_f) of the first tunnel. Fig. 3 shows the ground surface settlement and ground settlement in the plane of the pipeline due to double-line shield tunneling. As can be seen from these figures, the ground settlements due to single-line and double-line shield tunneling are significantly different. The maximum value of the ground settlement trough and the range of impact due to double-line shield tunneling are larger than in the case of single-line shield tunneling. Therefore, double-line shield tunneling cannot be evaluated by simply applying the same approach as for single-line shield tunneling.

It should be noted that in the study of impact factors described below, for ease of analysis, when considering a given impact factor, it is assumed that all other conditions remain the same.

3.2 Effect of geologic conditions

To evaluate the effect of variation of geologic conditions on the pipeline with other conditions held constant, it is assumed that the tunnel is excavated in round grain sand with a Poisson's ratio of 0.25, a modulus of deformation of 14 320 kPa and a unit weight of 18 kN/m3. Using O'Reilly and New's [21] approach, it is calculated that i_f =3.6 m. Following the recommendation of Chen et al. [20] that i_l / i_f = 0.97, it can be calculated that i_l=3.49 m, and, taking n=0.85, the ground loss rate is identical to that with clay soil. The reaction force factor of the sandy soil foundation is calculated from Equation (1) and doubled, so k=15.881×10^3 kN/m^3, and thus β=0.225.

The ultimate bending moment of the pipeline due to double-line shield tunneling calculated using the method proposed here is shown in Fig. 4, from which it can be seen that geologic conditions significantly affect the shape and magnitude of the ultimate bending moment curve of the pipeline. There are three negative and two positive peak values of the bending moment for a rigid pipeline in sandy soil, with the pipeline being subjected to a maximum ultimate negative bending moment $M^-_{\max}$ = –813.66 kN·m at x = 16 m and a maximum ultimate positive bending moment $M^+_{\max}$ = 1426.76 kN·m at x = 6 m. $M^+_{\max}$ is much larger than $M^-_{\max}$. Because of the interaction between the two tunnels, the curve is slightly asymmetric. For clay soil, the bending moment curve of the rigid pipeline is close to a normal distribution. There are two negative and one positive peak values of the bending moment. M^+_{max} is also larger than M^-_{max}, with M^-_{max} = –258.31 kN·m and M^+_{max} = 352.89 kN·m.

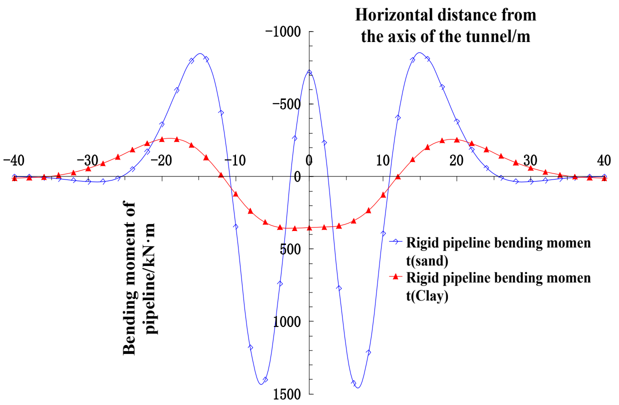

Fig.4. Ultimate moment of pipeline due to double-line parallel shield construction

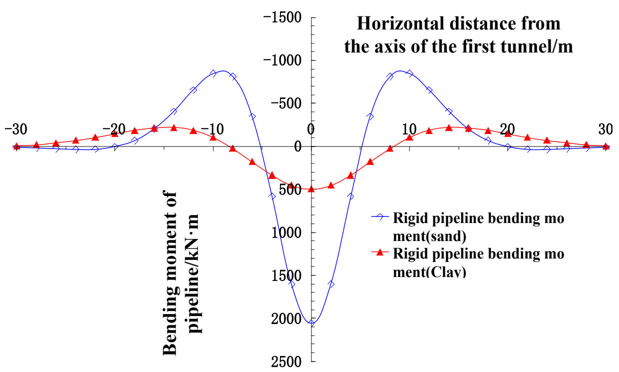

Fig.5. Ultimate moment of pipeline due to first shield construction

The ultimate bending moment of the pipeline due to the first tunnel (a single-line tunnel) is shown in Fig. 5. From comparison of Fig. 4 and 5, it is evident that the ultimate bending moments due to single-line and double-line shield tunneling are significantly different. In the case of sandy soil, the shapes of the curves are clearly different, although for clay soil, the curves are similar. In sandy soil, M^+_{max} due to single-line tunneling is 2059 kN·m, which is much larger than for double-line tunneling, and M^-_{max} = –847.8 kN·m, which is slightly larger than in the case of double-line tunneling. In clay soil, M^+_{max} due to single-line tunneling is 494.8 kN·m, which is much larger than for double-line tunneling, and M^-_{max} = –217.2kN·m, which is smaller than for double-line tunneling. This suggests that the interaction between the double-line parallel tunnels counteracts part of the bending moment of the pipeline, and consequently the ultimate positive bending moment of the pipeline is reduced significantly. However, because the pipeline has already been affected by the first tunnel before the second is bored, the construction of a double-line parallel shield tunnel does not reduce damage to the pipeline.

In the following safety assessment, because $M^+_{max} > M^-_{max}$, M^+_{max} is used for the calculation.

If the first (single-line) tunnel is excavated in clay soil, M^+_{max} = 494.8 kN·m, d = 0.8 m and d' = 0.784 m, the tensile stress is 126.86 MPa and the tensile strain is 6.16×10^{-4}. If it is excavated in sandy soil, M^+_{max} =2059 kN·m, the ultimate tensile stress is 527.92 MPa and the ultimate tensile strain is 2.56×10^{-3}.

If the double-line tunnel is excavated in clay soil, M^+_{max} =352.89 kN·m, the ultimate tensile stress is 90.48 MPa and the ultimate tensile strain is 4.39×10^{-4}. If it is excavated in sandy soil, M^+_{max} = 1426.76 kN·m, the ultimate tensile stress is 365.81 MPa and the ultimate tensile strain is 1.78×10^{-3}.

According to Wu [24], the allowed stress for a steel pipe $[\sigma]$ = 166.3 MPa. Therefore, the rigid pipeline is safe in clay soil, but in sandy soil, both single- and double-line tunneling result in the allowed stress being exceeded and both are therefore dangerous.

3.3 Effect of horizontal distance *L* between double-line tunnels

To evaluate the effect of horizontal distance, the value of L is changed, while the other parameters are identical to the standard conditions. The ultimate bending moment of a pipeline at different values of L calculated using the method proposed here is shown in Fig. 6. It can be seen that varying L has a significant effect on the shape of the ultimate bending moment curve of the pipeline and the value of the bending moment at the central axis. First, when L = 8 m, the behavior of the ultimate bending moment of the pipeline is similar to that in the case of single-line shield tunneling, with a maximum positive bending moment at the central axis and M^+_{max} = 669.74 kN·m, which is larger than the value 494.8 kN·m due to the excavation of the first tunnel. This suggests that when the horizontal distance between the double-line shields is small, the detrimental effect on the pipeline is greater than in the case of single-line shield tunneling. Second, with increasing L, the ultimate bending moment of the pipeline varies significantly, in particular at the central axis, where the positive bending moment changes toward a negative bending moment. When L = 20 m, at the central axis, M^-_{max} = –216.22 kN·m, and the curve around the central axis transforms from a "V' shape to a "W" shape. Third, the location of the maximum negative bending moment of the pipeline drifts toward the two sides and its value decreases. When L = 8 m, M^-_{max} = –335.33 kN·m, which occurs at x = –16 m; when L = 20 m, M^-_{max} = –210.65 kN·m, which occurs at x = –24 m.

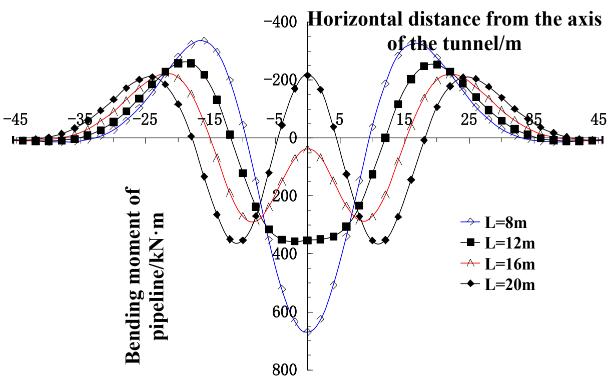

Fig.6. Ultimate moment of pipeline at different values of *L*

3.4 Effect of burial depth *h* of the centerline of the pipeline

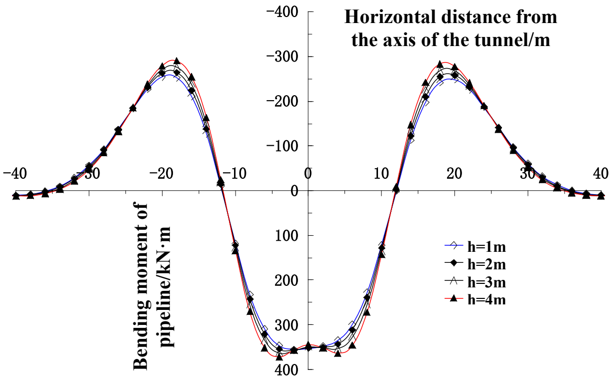

Fig.7. Ultimate moment of pipeline at different values of *h*

The value of the burial depth *h* of the centerline of the pipeline is varied from 1, 2, 3 to 4 m, with other parameters being identical to the standard conditions. The ultimate bending moment of the pipeline at different values of *h* calculated using the method proposed here is shown in Fig. 7. It can be seen that the variation of *h* has little effect on the ultimate bending moment of the pipeline, and that the shapes of the four curves are similar. Under the same conditions, the higher the value of *h* (i.e., the closer the pipeline is to the tunnel), the larger is the induced ultimate bending moment. When h = 1 m, $M^{+}_{\max}$ = 355.56 kN·m, which occurs at x = –2 m; when h = 4 m, $M^{+}_{\max}$ = 371.76 kN·m (an increase of 4.56%), which occurs at x = –4 m. The maximum negative bending moment occurs around –18 m in all cases. When h = 1 m, $M^{-}_{\max}$ = –252.52 kN·m, and when h = 4 m, $M^{-}_{\max}$ = –290.47 kN·m (an increase of 15.03%).

3.5 Effect of the ratio η_l/η_f

The interaction between the two tunnels is a significant factor. Different ratios η_l/η_f are examined in this study. It is assumed that η_f due to the first tunnel is 1.56%, and η_l/η_f is taken as 0.8, 1.0 and 1.2, giving η_l = 1.25%, 1.56% and 1.87%, respectively. The effect of the interaction of the two tunnels on the pipeline is evaluated, with other parameters identical to the standard conditions.

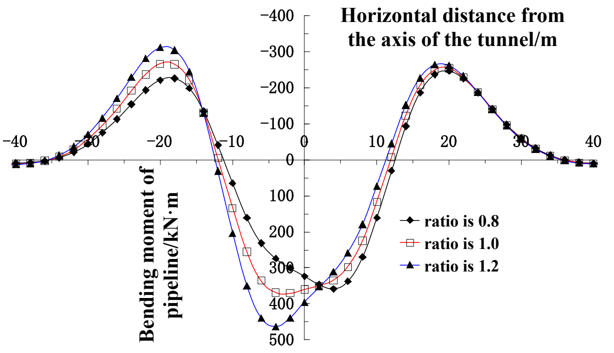

Fig.8. Ultimate moment of pipeline for the different ratios η_l/η_f

The ultimate bending moment of the pipeline calculated using the method proposed here is shown in Fig. 8. It is evident that the value of the ratio η_l/η_f affects the shape of the curve and the magnitude of the ultimate bending moment of the pipeline. With increasing η_l/η_f, the ultimate bending moment of the pipeline increases, and the maximum negative bending moment of the second tunnel varies significantly from –226.45 to –312.52 kN·m (an increase of 38.01%). However, there is no significant variation in the maximum negative bending moment of the first tunnel. The location of the peak positive bending moment shifts from x = 4 m to –4 m, and the value increases from 358.32 to 463.26 kN·m (an increase of 29.29%).

3.6 Effect of pipeline material

In this part of the study, the pipeline material is varied, as are the material parameters and the thickness of the pipeline tube. However, *d* remains constant at 0.8 m. With reference to the Manual of Water Supply and Drainage [25], the parameters of different pipelines are listed in Tab. 1. The other parameters are identical to the standard conditions.

The ultimate bending moment of the pipeline calculated using the method proposed here is shown in Fig. 9 for a number of different materials. It can be seen that the pipeline material has a significant effect on the magnitude of the ultimate bending moment, but not on the shape of the curve. The distribution range and the peak value are largest for a steel pipeline and smallest for a PVC pipeline. The ultimate bending moment curves for cast iron and concrete pipelines are very similar, although the magnitude for the concrete pipeline is slightly larger. These results suggest that pipelines with a larger bending stiffness have a larger distribution range and peak value of the ultimate bending moment.

Table 1. Parameters of the pipeline

Pipeline material	Tube thickness (mm)	Elastic modulus (MPa)	Bending stiffness (kN · m²)	Poisson's ratio	k (kN/m³)	β
Steel	16	2.06×10^5	1,246,300	0.3	3219	0.151
Cast iron	12	9.0×10^4	414,900	0.275	3528	0.203
Concrete	60	2.5×10^4	480,000	0.17	3486	0.195
PVC	30	2.26×10^3	24,871	0.35	4461	0.435

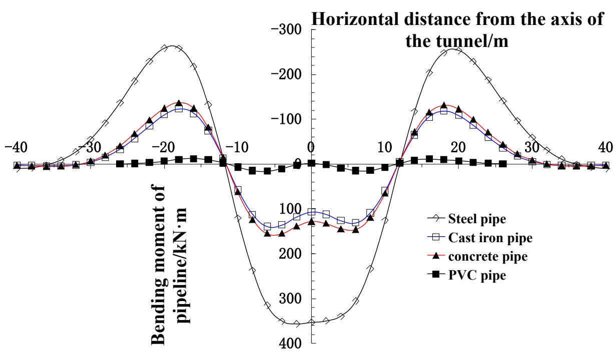

Fig.9. Ultimate moment of pipeline with different pipeline materials

4. Conclusion

For pipelines affected by double-line parallel tunneling, two safety assessments should be made, first for a single-line tunnel (i.e., the first tunnel), then for a double-line parallel tunnel.

With increasing *L*, the distribution curve of the ultimate bending moment of the pipeline varies significantly. In particular, at the central axis, where the positive bending moment changes toward a negative bending moment, the shape of the curve transforms from a “V” to a “W” shape, and the location of the maximum negative bending moment of the pipeline drifts toward the two sides and its value decreases.

The ratio η_l / η_f affects the shape of the curve and the magnitude of the ultimate bending moment of the pipeline. With increasing η_l / η_f, the ultimate bending moment of the pipeline increases gradually, and the location of the peak positive bending moment shifts.

Geologic conditions have a significant effect on both the shape and magnitude of the ultimate bending moment curve of the pipeline. The pipeline material has a significant effect on the magnitude of the ultimate bending moment, but not on the shape of the curve. For pipelines with a larger bending stiffness, the distribution range and peak value of the ultimate bending moment are larger. The value of *h* has little effect on the ultimate bending moment of the pipeline.

Acknowledgements

This work was financially supported by public technology projects of Science Technology Department of Zhejiang Province (NO:2016C33051) and science and technology project plan of Mnistry of Housing and Urban-Rural Development of the People's Republic of China in 2015 (NO:2015-K5-026).

References

1. Tomislav Hrestak, Antonia Jaguljnjak Lazarević, Lidija Frgić., “Stress and strain analysis during the sleme tunnel excavation”. *Tehnički vjesnik - Technical Gazette*, 22(3), 2015, pp.703-709.
2. Tadeusz Majcherczyk, Zbigniew Niedbalski, Michał Kowalski., “3D numerical modeling of road tunnel stability–the Laliki project”. *Archives of Mining Sciences*, 57(1), 2012, pp.61-78.
3. Gomez-Cabrera, Jose., “Pioneer system for risk control high speed tunnel in sants-segrera of barclona”. *DYNA*, 88(2), 2013, pp.161-163.
4. Wu, W. Y., Sun, Y. K. & Zhang, T. Q., “Analysis of the effects on the adjacent underground pipelines by shield tunneling construction”. *China Railway Science*, 29(3), 2008, pp.58-62.
5. Bi, J. H., Liu, W. & Jiang, Z. F., “Analysis of effects of tunnel excavation on underground pipeline”. *Rock and Soil Mechanics*, 27(8), 2006, pp.1317-1321.
6. Fan, D. W. & Song, X. G., “Analysis of buried pipelines caused by metro tunneling”. *Journal of Yanshan University*, 33(3), 2009, pp.247-253.
7. Sun, Y. K, Wu, W. Y. & Zhang, T. Q., “Analysis on the pipeline settlement in soft ground induced by shield tunneling across buried pipeline”. *China Railway Science*, 30(1), 2009, pp.80-85.
8. Lei, C. H., Wang, X., Liu, W. & Han, X., “Risk analysis of underground pipeline through the tunnel of Beijing subway line 8”. *Modern Urban Rail Transit*, (Suppl.), 2011, pp.9-14.
9. Attewell, P. B., Yeates, J. & Selby, A. R., “*Soil Movements Induced by Tunnelling and their Effects on Pipelines and Structures*”, Blackie Academic & Professional, London, U.K., 1986.
10. Vorster, T. E. B., Klar, A., Soga, K. & Mair, R. J., “Estimating the effects of tunneling on existing pipelines”. *Journal of Geotechnical and Geoenvironmental Engineering*, 131(11), 2005, pp.1399-1410.
11. Zhang, Z. G., Huang, M. S. & Wang, W. D., “DCBEM-FEM coupling method for response analysis of adjacent pipelines due to tunneling”. *Chinese Journal of Geotechnical Engineering*, 33(10), 2011, pp.1554-1561.
12. Hikmet Erbıyık, Eray Can & Şenol Kuşçu., “Potential risks and their analysis in the application of geographic based engineering projects”. *Tehnicki Vjesnik -Technical Gazette*, 21(1), 2014, pp.09-15.
13. Wang, T., Wei, G. & Xu, R. Q., “Prediction for influence of tunneling on adjacent pipelines”. *Rock and Soil Mechanics*, 27(Suppl.), 2006, pp.483-486.
14. Wei, G. & Zhu, K., “Prediction for response of adjacent pipelines induced by pipe jacking construction”. *Rock and Soil Mechanics*, 30(3), 2009, pp.825-831.
15. Zhang, H. & Zhang, Z. X., “Vertical deflection of existing pipeline due to shield tunnelling”. *Journal of Tongji University (Natural Science)*, 41(8), 2013, pp.1172-1178.
16. Zhang, C. R., Yu, J. & Huang, M. S., “Responses of adjacent underground jointed pipelines induced by tunneling”. *Chinese Journal of Geotechnical Engineering*, 35(6), 2013, pp.1018-1026.
17. Wei, G. & Xu, R. Q., “Prediction of longitudinal ground deformation due to tunnel construction with shield in soft soil”. *Chinese Journal of Geotechnical Engineering*, 27(9), 2005, pp.1077-1081.
18. Wei, G. & Pang, S. Y., “The definition of close range between parallel shield tunnels based on numerical simulation”. *Municipal Engineering Technology*, 32(1), 2014, pp.76-80.
19. Selvadurai, A. P. S., “*Elastic Analysis of Soil and Foundation Interaction*”, China Railway Publishing House, Beijing, 1984.
20. Chen, C. L., Zhao, C. L., Wei, G & Ding, Z., “Prediction of soil settlement induced by double-line shield tunnel based on Peck formula”. *Rock and Soil Mechanics,* 35(8), 2014, pp.2212-2218.
21. O’Reilly, M. P. & New, B. M., “*Settlements above tunnels in the United Kingdom—their magnitude and prediction*”. Proceedings of Tunnelling’82 Symposium, London, 1982, pp.173-181.
22. Wei, G., “Study on calculation for width parameter of surface settlement trough induced by shield tunnel”. *Industrial Construction*, 39(12), 2009, pp.74-79, 109.
23. Wei, G., “Selection and distribution of ground loss ratio induced by shield tunnel construction”. *Chinese Journal of Geotechnical Engineering*, 32(9), 2010, pp.1354-1361.
24. Wu, W. Y., “*Study on Mechanical Behaviors of Buried Pipelines Induced by Shield Tunneling Construction*”, Doctoral Dissertation of Zhejiang University, Hangzhou, China, 2008.
25. North China Municipal Engineering Design and Research Institute. “*Water supply and drainage design manual*”, China Building Industry Press, Beijing, China, 2011.