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Failure analysis and safety evaluation of buried pipeline due to deflection of landslide process

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ABSTRACT

This paper focuses on the failure mechanisms of buried X65 pipeline due to non-uniform deflection of landslide process. First, fractographic tests are performed to analyze the failure causes of two typical accidents arising from landslide. Second, based on the non-linear stabilization algorithm, an improved finite element model is established to predict the load-bearing ability of buried pipelines under deflection load, and the nonlinear contact interaction between the pipeline and soil is considered. Particularly, effects of the surrounding soil, internal pressure and pipeline geometry are comprehensively investigated for the risk assessment of buried pipeline under large deflection. Finally, a strength failure criterion based on the maximum principal strain is proposed to determine the safe properties of buried pipeline under this special failure issue. Research in this paper can provide technical support for the safety assessment and standard formulation of buried pipelines.

1. Introduction

With the increasing demands of global energy resources, buried pipelines for natural gas transportation are facing good opportunities to make great progress. Buried pipelines, which often run in high pressure and harsh environment, have posed severe challenges to the life prediction and safety evaluation [1,2]. Therefore, it is imperative that the fundamental research of failure mechanisms should be developed rapidly.

Safety issues of buried pipeline under permanent ground deformation such as fault movements and landslides have aroused wide attention in these years. Under these special failure issues, buried pipelines are often subjected to excessive plastic deformation which is associated with additional axial, shear and bending loads, and the high stress/strain on the pipeline can finally result in local plastic collapse or local buckling at the critical location.

Already, safety problems of buried pipeline under the active fault movements of earthquake have been studied for many years. Newmark and Hall [3] first investigated the mechanical properties of buried pipeline due to large crossing fault, and the pipeline was simplified as a long cable. Kennedy et al. [4,5] improved the model in order to explore the non-uniform friction behavior between the pipe and its surrounding soil. Wang et al. [6,7] investigated the bending stiffness of buried pipelines during the fault movements. Takada et al. [8] proposed a simplified method for calculating the critical strain at longitudinal direction. Trifonov and Cherniy [9,10] presented a semi-analytical approach to analyze the mechanical behaviors of buried pipeline, which comprehensively considered the nonlinearities of material, large displacement and pipe/soil interaction. In the past decades, the finite element method was widely used to simulate the buried pipeline behaviors under permanent ground-induced actions. Karamitros et al. [11,12] employed shell elements to describe the pipeline, and the surrounding soil was simulated by nonlinear springs. The numerical results by finite element analysis were compared with

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the current analytical methods, and the distribution of axial strain along the pipeline was obtained. Vazouras et al. [13] and Zhao et al. [14] applied similar models to explore the failure properties of pipelines under different directions of fault movements.

However, little work was concentrated on the failure mechanisms of buried pipeline due to non-uniform deflection arising from landslide. In fact, accidents of buried pipeline in the landslide area occurred frequently in these years. In 2008–2009, two accidents appeared continuously in Zhejiang province of China. The pipelines developed excessive plastic deformation, which finally resulted in local plastic collapse at the critical location. Liu et al. [15] proposed a finite element method to predict the load-bearing ability of these pipelines, yet the contact boundary condition was not applied on the interface between the pipeline and its surrounding soil. Furthermore, since the fractography and the effects of some significant factors were not investigated in that paper, the strength failure criterion of this failure issue cannot be proposed.

Thus, in this study, the fractographic testing is first carried out to analyze the failure causes of two typical accidents. Second, the contact boundary is applied on the interface between the pipeline and soil in the finite element analysis. Particularly, effects of some factors such as the surrounding soil, internal pressure and pipeline geometry on the load-bearing ability are comprehensively investigated. Finally, a strength failure criterion of this failure issue is proposed, which can provide theoretical support to determine the allowable strain in the pipeline design process.

2. Two accidents of buried pipeline

In 2008–2009, two typical accidents of buried pipeline caused by landslide appeared in Zhejiang province of China. The earlier accident happened in Yuyao city, where the maximum deflection displacement reached about 1.9 m [15]. Fortunately, the offset segment of pipeline was detected timely and replaced before leaking. The latter accident occurred in Ningbo city as shown in Fig. 1. Terribly, the pipeline exploded at last. The site inspection indicated that the accumulated soil was deposited on the hillside, and the pipeline segment and its surrounding soil were severely pushed away from the original location during landslide process in rainstorm days.

Caption near the fracture location is shown in Fig. 2. As indicated in the picture, the pipeline is ruptured circumferentially in the butt-welded joint. The outer diameter of the fracture surface is 813.6–814.3 mm, which is close to the original value.

Fig. 1. Pipeline accident in Ningbo city of China.

Fig. 2. Captions near the fracture location of accident pipeline.

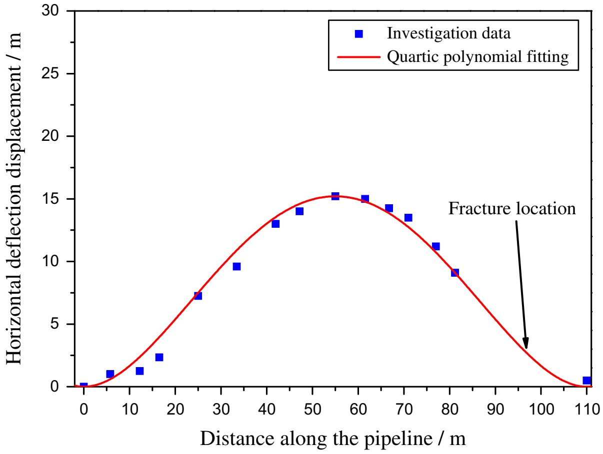
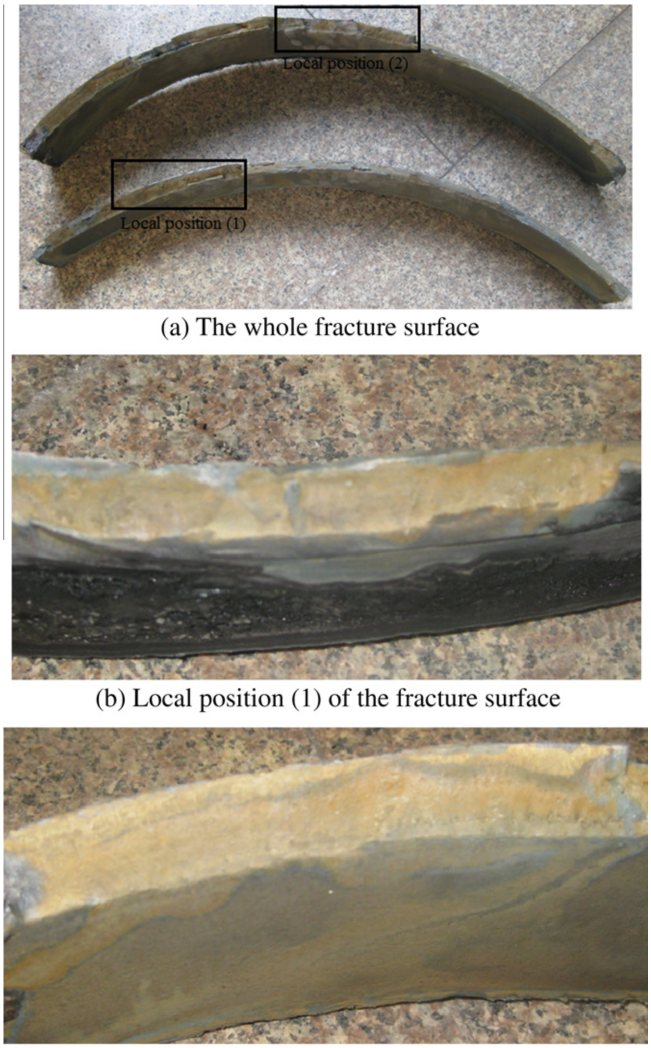


Fig. 3. Investigation data and the quartic polynomial fitting of deflection displacement.

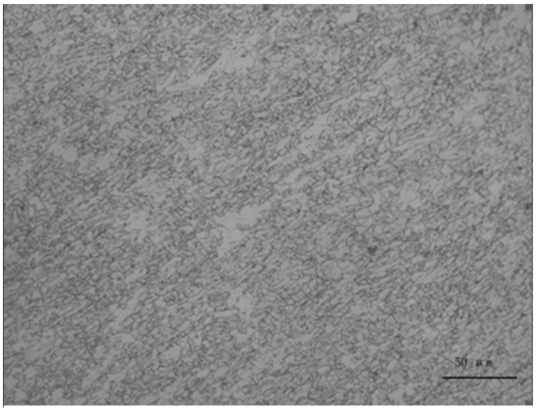


(a) The whole fracture surface

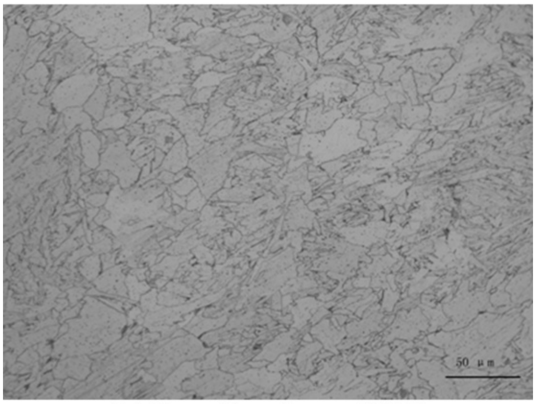
(b) Local position (1) of the fracture surface

(c) Local position (2) of the fracture surface

Fig. 4. Captions of the fracture surface.



(a) Base metal (500X)



(b) Heat affected zone (500X)

Fig. 5. Microstructures of the base metal and HAZ in the welded joint of fracture surface.

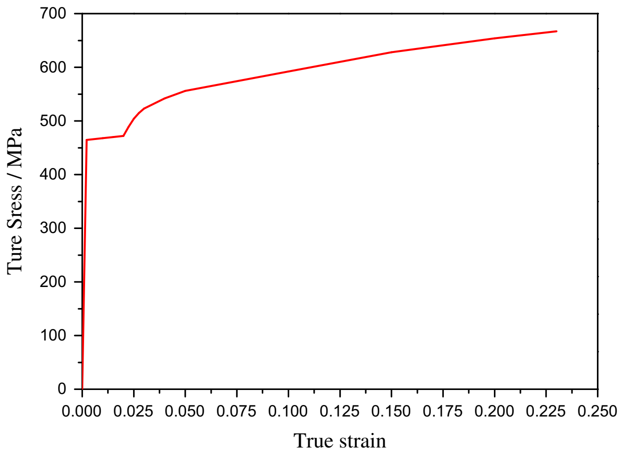


Fig. 6. True stress–strain curve of X65 steel.

Local buckling on the pipeline wall is not detected, but the thickness of pipeline at the fracture location is sharply reduced. Additionally, some hard obstacles are detected in the soil near the fracture location.

Failure segment of buried pipeline is detected by electromagnetic induction. Investigation shows the length of buried pipeline with non-uniform deflection (the width of landslide area) is about 110 m, and the deflection is mainly at the horizontal direction. The maximum deflection displacement is 15.2 m in the middle of this pipeline segment, and the distribution is close to quartic polynomial curve, which is illustrated in Fig. 3.

3. Fractographic testing

Before the fractographic testing, a series of pipeline tests (including tensile test, bending test, impact test and drop weight tear test) indicate that the properties of base metal meet the requirements of each standard. In this section, the fractography is presented to preliminarily analyze the failure cause of the accident. Captions of the fracture surface are shown in Fig. 4. As depicted in Fig. 4a, the thickness of pipeline at the fracture location is sharply reduced, which means the excessive plastic deformation is developed before ductile damage. Fig. 4b and c are captions of local positions. As indicated in Fig. 4b, shear zone can be clearly observed at the margin of fracture surface. From Fig. 4c, there are several uneven steps on the fracture surface, which suggests that the fracture surface may probably locate in the heat affected zone (HAZ).

Microstructures of the base metal and HAZ in the welded joint of fracture surface are demonstrated in Fig. 5. As shown in the pictures, no obvious flaw is observed. Combined with the results of base metal tests, it can be concluded that the major cause of this accident may not be the potential defects in the material but the large deflection caused by landslide.

Table 1
Soil properties of different depths near the accident pipeline.

Layer index	Soil depth (m)	Compressibility coefficient (MPa^{-1})	Compression modulus (MPa)	Internal friction angle (°)	Cohesive force (kPa)
1	3.70	0.92	2.62	9.8	15.3
2	4.50	0.64	3.28	14.1	32.7
3	7.80	1.23	2.00	7.6	16.0
4	21.30	1.21	2.10	8.8	18.0
5	22.60	0.35	5.64	15.0	34.7

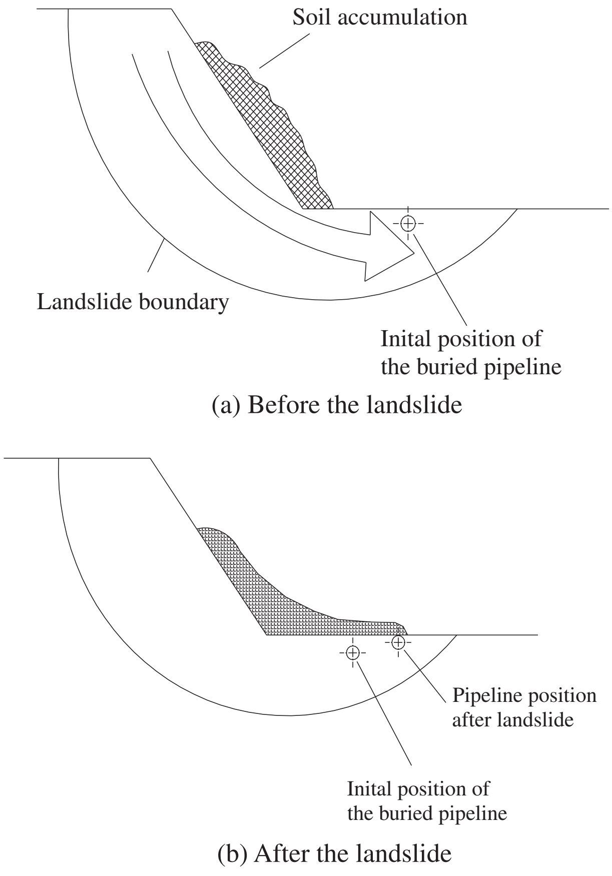


Fig. 7. Schematic diagram during the landslide process.

4. Numerical analysis

4.1. Basic assumptions

(1). Property of soil is linear-elastic and pipeline is elastic–plastic and isotropic.
(2). Contact property of the interface between pipeline and soil is finite sliding.
(3). Appearance of pipeline and surrounding soil during landslide process is quartic polynomial along the pipeline.

4.2. Material properties

The pipeline material is American Petroleum Institute (API) X65 steel. The diameter and thickness of the pipeline are 813 mm and 12.5 mm, respectively. The design pressure is 6.3 MPa, the operating pressure is 2.0–4.0 MPa. True stress/strain curve is shown in Fig. 6, and the density of pipeline is 7850 kg/m^3. Soil properties of different depths near the pipeline have

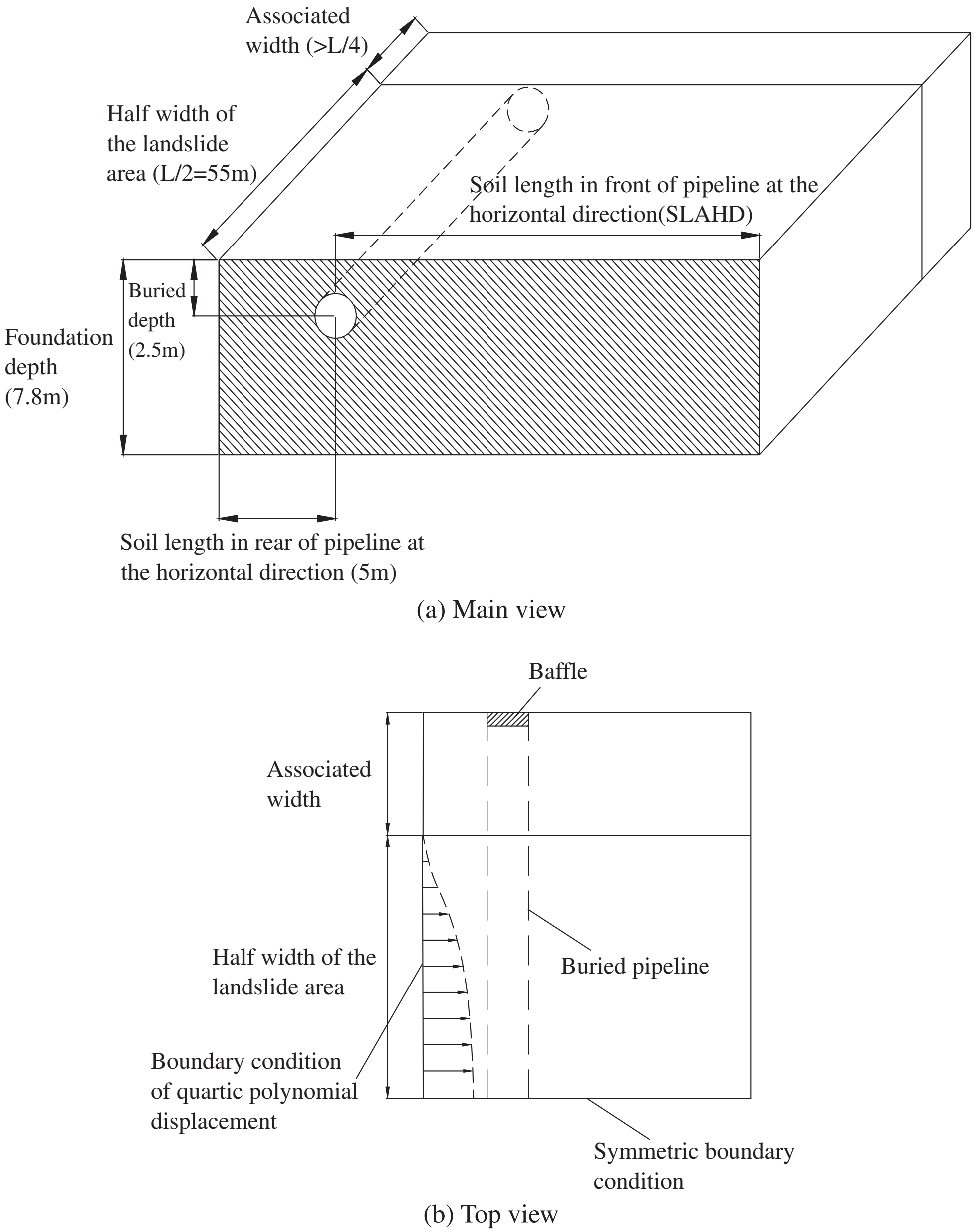


Fig. 8. Geometric features of the FEA model.

been listed in Table 1. Since the depth of buried pipeline is about 2.5 m, the property of soil is assumed as the parameters of index 1–1, and Poisson's ratio is 0.45. Friction coefficient of the interface between the pipeline and soil is 0.3.

4.3. Finite element model

In order to reproduce the landslide process, the schematic diagram of this accident is illustrated in Fig. 7. The piping system of finite element model contains the pipeline segment and its surrounding soil. Geometric features of the finite element model are illustrated in Fig. 8. Assuming the deflection of pipeline is at horizontal direction, and the length of pipeline with non-uniform deflection is close to the width of landslide area. During the landslide process, the reaction force from the compressed soil will increase the stress on the pipeline, which effectively reduces the limited deflection displacement. Furthermore, the value of reaction force depends on the soil configurations, such as the soil property and soil length in front of pipeline at the horizontal direction (SLAHD). Because the SLAHD is intractable to determine in the landslide area, different values from 5 m to infinite length will be analyzed.

A parametric 3D finite element model using the software ABAQUS is established to explore the failure mechanisms of buried pipeline. The mesh of pipeline segment and its surrounding soil use the eight-node and hexagonal solid element C3D8, which includes 35,890 nodes and 33,710 elements, respectively. When the infinite SLAHD is considered, the elements of CIN3D8 are updated at the outermost surface of soil as shown in Fig. 9. During the landslide process, the quartic polynomial displacement is applied on the soil of landslide field (as shown in Fig. 8b), and the entire piping system will deform under the given ground-induced actions.

4.4. Solution algorithms

Since the conventional Newton–Raphson method cannot describe the strain softening properties of pipeline under excessive plastic deformation, the arc-length algorithm and non-linear stabilization algorithm are two effective methods to predict the limited deflection displacement. Liu et al. [15] supposed that results of non-linear stabilization algorithm and

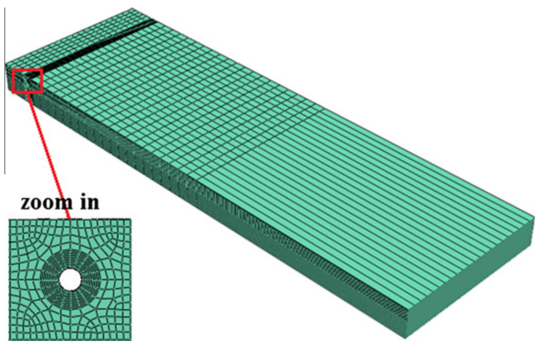


Fig. 9. Finite element model of the infinite SLAHD.

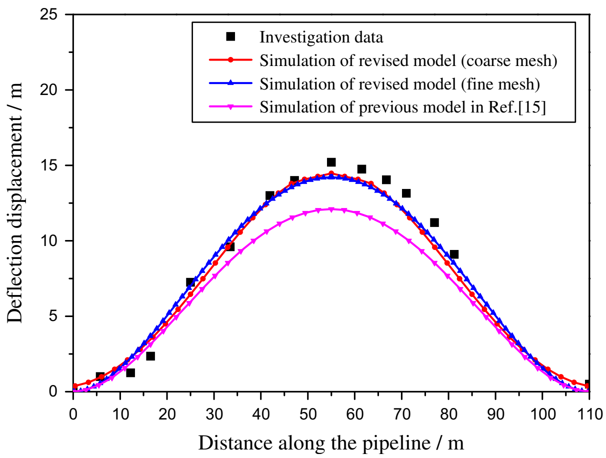


Fig. 10. Predictions of different finite element models.

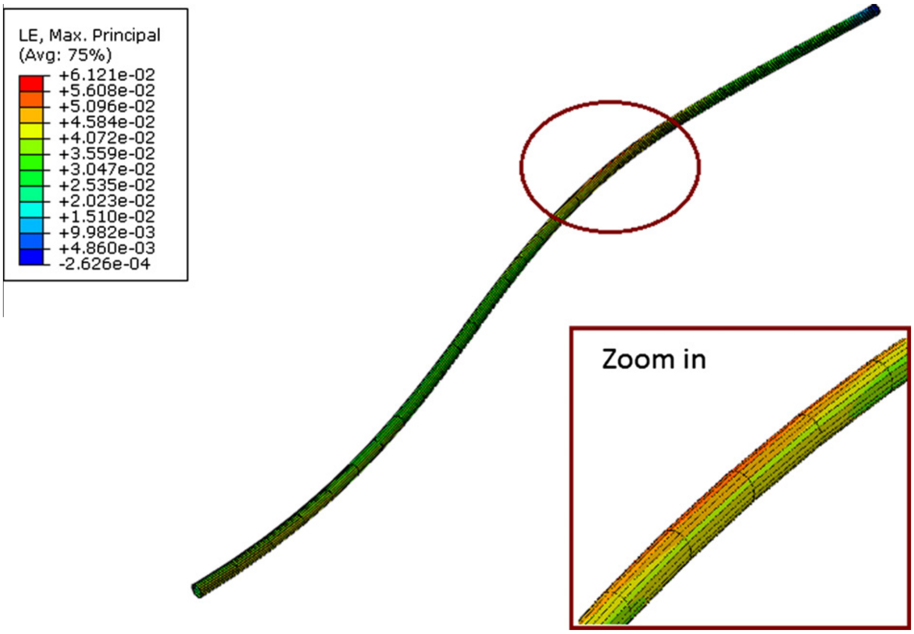


Fig. 11. Distribution of the maximum principal strain along the pipeline.

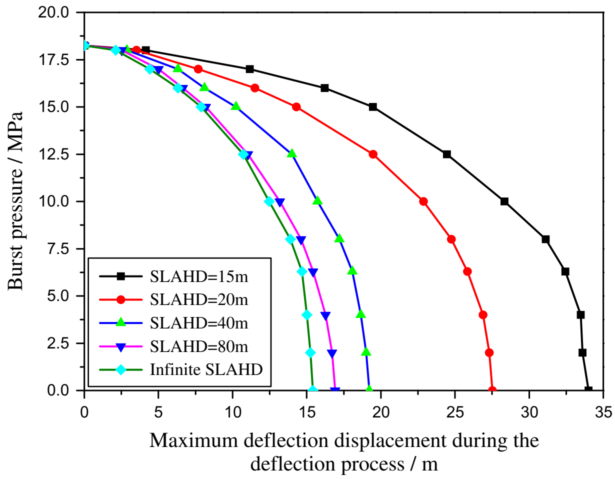


Fig. 12. Relationships between the burst pressure and the maximum deflection displacement under different SLAHD.

arc-length algorithm are in good agreement with each other. Furthermore, the non-linear stabilization algorithm exhibits better efficiency than the arc-length algorithm. Therefore, non-linear stabilization algorithm is used in this paper.

5. Results and discussion

As mentioned in Section 3, defects in the material are not the major cause of this accident. According to the site inspection, accumulated soil in the hillside brings about landslide after rainstorms. The buried pipeline develops excessive plastic deformation and finally leads to local plastic collapse at the critical location. Captions near the fracture surface also indicate that the direction of the maximum principal stress/strain is longitudinal since the pipeline is ruptured circumferentially. During the landslide process, the stress/strain of pipeline arises with the offset increasing. When the deflection displacement reaches a critical value, the stress/strain of pipeline at the critical location will exceed the limitation, which leads to ultimate failure of this pipeline segment. Additionally, the soil performance is heterogeneous, hard obstacles will be unavoidably contained in the surrounding soil. With the increase of deflection, the pipeline will encounter these objects easily, which exacerbate the stress concentration.

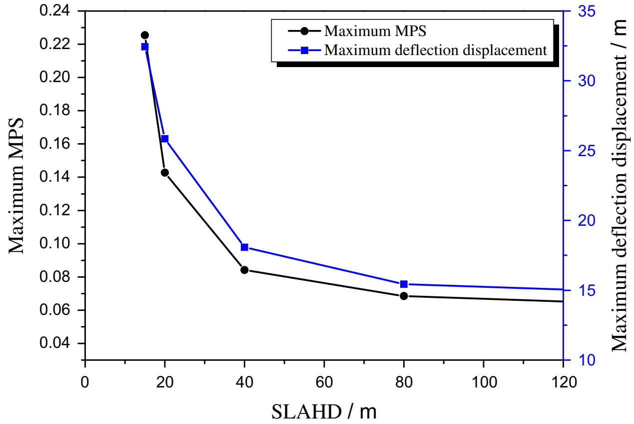


Fig. 13. Relationships of the maximum deflection displacement and maximum MPS as pipeline failure under different SLAHD.

In order to check the effect of current model, an original model from Ref. [15] and a refine-meshed model (including 104,698 nodes and 98,484 elements) are used simultaneously to reproduce the accident process. Predictions of these models have been illustrated in Fig. 10. It can be seen that, compared with the original model in Ref. [15], simulations of the improved models are more close to the investigation, and the result of current mesh density does not change largely with the mesh number increasing. Furthermore, it can be seen that there is a relative sliding between the pipeline and soil. If the nodes on the pipeline/soil interface are coupled as Ref. [15], the extra constraint will result in additional stress/strain at the critical location which reduces the simulated accuracy. Distribution of the maximum principal strain (abb. as MPS) along the pipeline is illustrated in Fig. 11. It indicates the location of maximum MPS is near the fracture location. Since the direction of maximum principal stress/strain is longitudinal, the MPS along the pipeline can be approximately described as the longitudinal strain.

5.1. Internal pressure

During the landslide process, the failure of buried pipeline appears when the deflection displacement reaches its maximum value. At this time, the internal pressure can be approximately assumed as the burst pressure under this limited offset displacement. In this section, the load-bearing ability of buried pipeline under different deflection displacements will be investigated. Relationships between the burst pressure and the maximum deflection displacement under different SLAHD have been demonstrated in Fig. 12. As depicted in the figure, it can be concluded that:

(a) The burst pressure of buried pipeline without offset is independent of SLAHD, which is close to 18.2 MPa. With the growth of SLAHD, the reaction force from the compressed soil increases the stress of buried pipeline. Therefore, during the landslide process, the limited deflection displacement declines with the SLAHD increasing.
(b) Under the condition of the same SLAHD, the burst pressure of buried pipeline decreases logarithmically with the offset increasing. It means the burst pressure does not decline obviously under the light deflection of buried pipeline. However, when the pipeline develops severe deflection, the burst pressure will be slumped.
(c) Considering the design pressure is 6.3 MPa, and the operating pressure is 2.0–4.0 MPa. As depicted in Fig. 12, the maximum deflection displacement under the operating pressure is only 1.03–1.05 times as large as that under the design pressure. Therefore we can come to conclusion that, the internal pressure under the normal operation is not the major factor to affect the limited deflection displacement of buried pipeline.

Extracting data from Fig. 12, relationships of the maximum deflection displacement and the maximum MPS as pipeline failure under different SLAHD have been given in Fig. 13. The internal pressure is taken as 6.3 MPa. It can be seen that the maximum deflection displacement and the maximum MPS decrease exponentially with the SLAHD increasing. Once the SLAHD is greater than 80 m, two variables do not change largely. When SLAHD is close to be infinite, the limited deflection displacement is about 14.6 m, the maximum MPS is about 0.061. Because of the complex circumstance in the landslide area, it is intractable to determine the SLAHD at the offset direction. Therefore, the following work is to find a variable that can reduce the effect of SLAHD.

Node force–displacement relationships during the landslide process under different SLAHD have been demonstrated in Fig. 14, and the node is obtained from the maximum MPS location. As depicted in this picture, the node reaction force increases first and then decreases with the growth of deflection. The maximum points of these curves represent that the structural comes into the stage of plastic collapse, which indicates a sudden transition from material hardening to softening. The

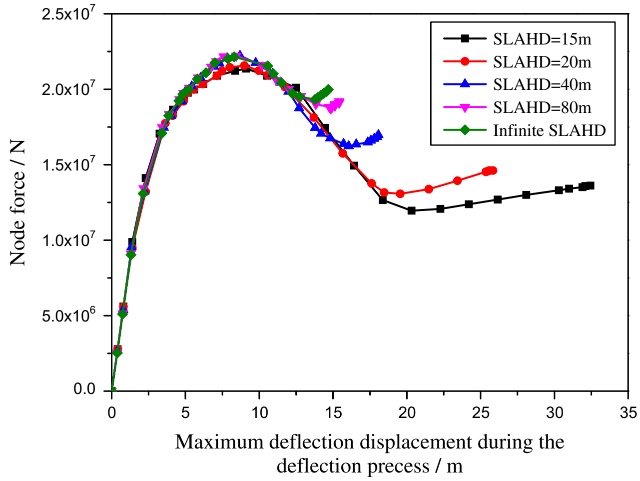


Fig. 14. Node force–displacement relationships during landslide process under different SLAHD.

Table 2
The maximum MPS of buried pipeline as plastic collapse stage and ultimate failure.

SLAHD (m)	Maximum MPS	
	Plastic collapse	Ultimate failure
15	0.0302	0.2258
20	0.0283	0.1428
40	0.0278	0.0841
80	0.0273	0.0683
160	0.0270	0.0612

Table 3
Relationship between the maximum deflection displacement and D/t ratio under different internal pressures.

D/t	Maximum deflection displacement (m)		
	Pressure = 2 MPa	Pressure = 4 MPa	Pressure = 6.3 MPa
50.81	17.26	16.95	16.44
57.25	16.14	15.89	15.54
65.04	15.42	15.03	14.67
73.91	14.57	14.05	13.87
85.58	13.75	13.42	13.01

end points of these curves show that the ultimate failure of buried pipeline appears. As depicted in Fig. 14, some conclusions are obtained:

(1) With the increase of SLAHD, the maximum deflection displacement as ultimate failure decreases obviously.
(2) SLAHD only affects the mechanical behaviors of buried pipeline in the strain softening stage, while the counterparts in the strain hardening stage do not change largely with the SLAHD increasing.
(3) The peak values of these curves (plastic collapse) are independent of SLAHD, and the corresponding displacements are frozen at 8.81–9.32 m.

The maximum MPS of buried pipeline as plastic collapse stage and ultimate failure has been listed in Table 2. It shows the maximum MPS as ultimate failure decreases obviously with the SLAHD increasing. However, the counterpart as plastic collapse is frozen at 0.027–0.030. Since the maximum MPS of buried pipeline as plastic collapse is independent of SLAHD, it is an appropriate variable to develop the strength failure criterion later.

When the SLAHD is close to be infinite, the reaction force from the compressed soil reaches the greatest value, which can be approximately considered as the most dangerous condition. Therefore the infinite SLAHD will be used in the following analysis without extra instructions.

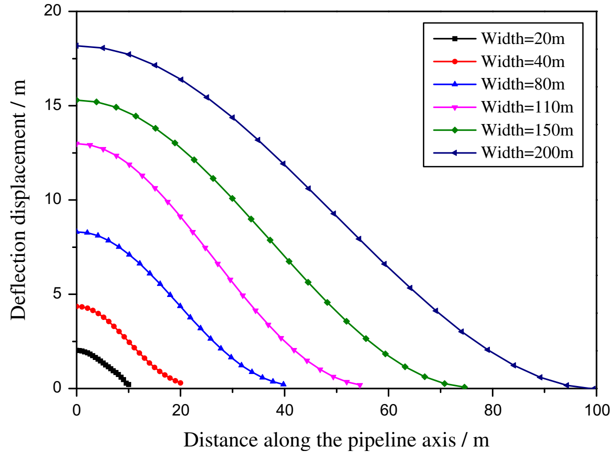


Fig. 15. Distributions of the deflection displacements under different landslide widths.

5.2. Ratio of diameter and thickness

Ratio of diameter and thickness (abb. as D/t) has significant effect on the pipeline performance. Table 3 shows the relationship between the maximum deflection displacement and D/t ratio under different internal pressures. As listed in the table, the maximum deflection displacement decreases with the growth of D/t ratio. Considering the design D/t ratio is 65.04 (t = 12.5 mm), and the nominal D/t ratio is about 85.58 (t = 9.5 mm). The limited deflection displacement of the design D/t ratio is 1.12–1.13 times larger than that of the nominal D/t ratio. However, under the same D/t ratio, the limited deflection displacement of the operating pressure is only 1.04–1.05 times as large as that of the design pressure. Therefore, the D/t ratio has greater effect on the limited deflection displacement than the internal pressure. Furthermore, it can also be concluded that the maximum MPS of buried pipeline as plastic collapse is independent of the D/t ratio, which stabilizes at 0.028–0.030.

5.3. Width of the landslide area

In this section, the width of landslide area is assumed as 20 m, 40 m, 80 m, 110 m, 150 m, and 200 m. Distributions of the deflection displacements under different landslide widths have been illustrated in Fig. 15. It can be concluded that the pipeline can reserve greater offset-bearing capacity as the landslide width increasing. Additionally, the maximum MPS of buried pipeline as plastic collapse is independent of the landslide width, and the value is frozen at 0.027–0.030.

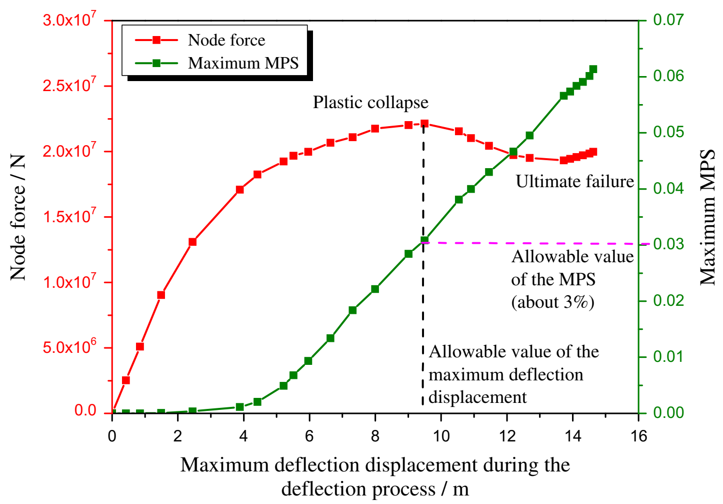


Fig. 16. Schematic diagram of obtaining the allowable MPS.

5.4. Strength failure criterion

From a series of finite element analysis, the maximum MPS at the critical location is appropriate to assess the safe properties of in-service pipeline during landslide process. In order to ensure the safety of buried pipeline under this special failure issue, the maximum MPS should be less than the allowable value of each material. This section will propose a method to obtain the allowable MPS of X65 pipeline due to non-uniform deflection of landslide process, which is illustrated in Fig. 16. Since the pipeline coming into the strain softening stage is not permitted in engineering, the maximum deflection displacement at the top of node-force curve (plastic collapse) can be defined as the allowable displacement, and the corresponding MPS can be served as the allowable strain. From previous study, the reason of obtaining this allowable MPS is:

(a) The maximum MPS of buried pipeline as plastic collapse is independent of the D/t ratio, width of landslide area and SLAHD. The results of finite element analysis indicate the corresponding value of X65 steel is close to 3%;
(b) When the maximum MPS of buried pipeline reaches 3%, the burst pressure will decline to 11.2 MPa. Considering the limited operating pressure is 4.0 MPa, the burst pressure is 2.8 times as large as the operating pressure;
(c) When the thickness of pipeline is reduced to the nominal value, the burst pressure of buried pipeline under 3% maximum MPS will decline to 9.63 MPa, which remains to be 2.4 times as large as the operating pressure.

Therefore, the buried pipeline under the maximum MPS of 3% can reserve sufficient load-bearing ability for the safety operation. Compared with the permitted strain of 2% in ASME B31.8 [16], the current standard has a safety factor of 1.5 times.

6. Conclusions

Continuous failure accidents of buried pipeline due to non-uniform deflection of landslide process have drawn wide attention in these years. In this paper, the interaction between the pipeline and soil is considered in the finite element analysis, and the effects of significant factors on the load-bearing ability are investigated to explore the failure mechanisms of buried pipelines. From our work, some conclusions are obtained:

(1) The fractographic testing and finite element analysis indicate that, during the landslide process, the large plastic deformation caused by excessive axial, shear and bending loads will easily result in the local plastic collapse at the critical location of buried pipeline.
(2) The relative sliding on the interface between the pipeline and soil should not be ignored especially when the buried pipeline develops large deflection.
(3) Compared with the internal pressure under normal operation, the D/t ratio and the width of the landslide area have greater effect on the limited deflection displacement of buried pipeline.
(4) The maximum MPS of buried pipeline as plastic collapse is independent of the D/t ratio, width of landslide area and SLAHD. The maximum MPS at the critical location can be used to determine the safe properties of buried pipeline under different deflection displacements.

In the pipeline design process, it is significant to determine how much the allowable stress/strain is under the excessive permanent ground deformations. Since most of them in current standards are based on experience, it is imperative that the fundamental research should be developed to obtain the accurate value with respect to each material and specific failure issue. Here, a strength failure criterion based on the maximum principal strain is proposed by a series of finite element analysis. From previous study, when the maximum MPS at the critical location reaches about 3%, the buried pipeline of X65 steel will result in the local plastic collapse. At this time, the pipeline segment under large deflection should be replaced at once. The strength failure criterion proposed in this paper can provide constructive suggestion for the pipeline design and safety evaluation.

Since excessive permanent ground deformations caused by landslide can easily lead to the failure of buried pipeline, a great amount of work has been concentrated on how to restrict the large deformations. Here, several points can effectively improve the ability of buried pipeline to resist the excessive deformations: (1) Excessive soil accumulation on the hillside near the buried pipeline should be strictly prohibited; (2) Slope area of the hillside should be consolidated properly; (3) The pipeline coatings should be paid particular attention in order to minimize longitudinal friction force; (4) The potential anchoring elements in the landslide field should be eliminated to improve the deflection displacement flexibility of buried pipeline; (5) Detection should be made periodically to discover the abnormal deflection as early as possible.

Acknowledgment

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