

Article

Experimental and Numerical Investigation of the Dynamic Evolution and Interface Friction Characteristics of Pipe Jacking Thrust in Sandstone Formations

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Abstract

Accurately predicting pipe jacking thrust and addressing the limited applicability of existing frictional resistance models under rock formation conditions are key technical challenges in pipe jacking through sandstone formations. Based on a pipe jacking project in the sandstone formation of the Qianping Irrigation District in Henan Province, laboratory direct shear tests were conducted to systematically compare the interface friction characteristics of smooth planar and rough split-surface interfaces under dry-friction and slurry-lubricated conditions. Numerical simulations were subsequently performed using the section beneath Provincial Highway S243 as a representative case. The test results showed that, under dry-friction conditions, the planar interface exhibited a distinct peak, followed by a sudden drop and subsequent stabilization. In contrast, the rough split-surface interface exhibited typical strain-hardening behavior. Slurry lubrication reduced the friction coefficient of the planar interface by 36%; however, under the same lubrication conditions, the friction coefficient of the rough split-surface interface was 20% higher than that of the planar interface. The numerical results indicated that the contact state at the pipe–rock–slurry interface gradually evolved from an initially suspended, noncontact state to localized line contact and then to combined multipoint and multiline contact, ultimately developing into a widespread contact pattern in which the crown, invert, and sidewalls jointly carried the load. The simulated pipe jacking thrust agreed well with the field measurements, whereas the frictional resistance calculated using conventional empirical formulas was substantially higher than the measured values. This finding confirms that the conventional assumptions of uniform contact and constant friction systematically overestimate the actual frictional resistance encountered during pipe jacking through rock formations. By elucidating the dynamic evolution of the pipe–rock interface, this study advances the understanding of the mechanical behavior of pipe jacking in sandstone formations and provides a reliable engineering reference for the accurate prediction of pipe jacking thrust and the optimization of pipe jacking equipment.



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Keywords: pipe jacking construction; pipe jacking thrust; lateral frictional resistance; interface friction characteristics; direct shear test; numerical simulation; sandstone formation

1. Introduction

Pipe jacking is a well-established trenchless construction technique that offers high construction safety and minimal surface disturbance. It has been widely applied to urban underground pipelines, water conveyance tunnels, utility corridors, and cross passages between subway tunnels. Driven by China's dual carbon goals of carbon peaking and carbon neutrality, underground space development is progressively moving toward larger diameters, longer drives, greater burial depths, and hard-rock geological conditions [1–3]. Consequently, pipe jacking has expanded from conventional applications in loose soils to rock formations characterized by substantial variations in integrity, complex joint networks, and pronounced stiffness heterogeneity. During jacking, the accuracy of pipe jacking thrust prediction directly affects project safety and cost-effectiveness. Overly conservative estimates may result in excessive capacities and redundant configurations of jacking equipment and intermediate jacking stations, thereby increasing overall project costs. Conversely, underestimation may cause pipe sticking and substantially reduce jacking efficiency. In extreme cases, pipe jacking thrust exceeding the stability threshold of the surrounding rock may induce ground instability and other engineering failures [4–6]. These challenges have become major scientific and technical barriers to refined design and safe, efficient construction of pipe jacking projects in rock formations. Therefore, systematically investigating the evolution of pipe jacking thrust in rock formations and developing an accurate prediction method are of considerable theoretical and practical significance.

The total pipe jacking thrust consists of frontal resistance at the excavation face and lateral frictional resistance along the external pipe wall. The lateral frictional resistance varies dynamically with the jacking distance and interface contact conditions and is therefore a key variable governing pipe jacking thrust prediction. Existing models for calculating lateral frictional resistance have largely been developed from pipe jacking practices in soil formations and can be classified into three categories according to their assumed contact mechanisms. The first category comprises direct soil-contact models, which completely neglect the friction-reducing effect of lubrication slurry and assume full contact between the external pipe wall and the surrounding soil. Some studies have further refined the local contact mode using Hertzian contact theory [7,8]. The second category comprises slurry-suspension models. Based on the idealized assumption that a complete slurry envelope forms around the pipe, these models consider the pipe to be fully surrounded by thixotropic slurry and suspended within the bore. They primarily focus on the influence of slurry rheology on friction reduction. For example, Ye et al. (2015) coupled cylindrical cavity expansion theory with a planar fluid model and proposed a method for predicting pipe jacking thrust that accounts for the effects of thixotropic slurry [9]. Through laboratory interface direct shear tests, Namli and Guler (2017) demonstrated that the formation of a complete slurry film could reduce the friction coefficient at the sand–concrete interface by up to 90% [10]. The third category comprises soil–slurry coupled-contact models, which attempt to account for both pipe–soil and pipe–slurry contact mechanisms. Representative studies include the empirical contact distribution proposed by Shimada et al. (2004), in which one-third of the pipe circumference was assumed to be in contact with soil and two-thirds with slurry [11], and the modeling approach developed by Shou et al. (2010), who introduced Hertzian contact theory to quantitatively estimate the pipe–soil contact angle and width [12]. However, all three categories are based on fundamental soil-mechanics assumptions and simplified in situ stress conditions. The values of contact-state parameters, such as the contact angle and contact width, rely heavily on calibration against field measurements from individual projects. These models do not adequately account for the spatial heterogeneity of the initial in situ stress field caused by topographic variations and

geological structures. Consequently, their general applicability is substantially reduced in complex rock engineering settings.

Rock formations and loose soils differ fundamentally in their constitutive responses, interface contact configurations, and mechanical evolution. The interaction between a rock formation and concrete pipe segments does not conform to the uniform surface contact commonly assumed for pipe jacking in soil. Instead, it may involve complex discontinuous contact configurations, including point contact, line contact, and interlocking contact between discrete rock fragments. Based on Persson's contact mechanics theory for conformal surfaces, Zhang (2018) systematically clarified the differences in the mechanical responses of pipe–ground contact configurations between soft-soil and rock conditions [13]. Building on this work, Deng et al. (2021) further considered the dynamic evolution of pipe–rock contact in rock formations and classified it into typical modes, including point contact, bottom filling, and top filling [14]. They also incorporated the combined adverse effects of bentonite slurry buoyancy and rock-chip accumulation on the contact angle and interface friction coefficient, thereby developing a comprehensive model for calculating lateral frictional resistance in long-distance pipe jacking through rock formations [14]. Laboratory friction tests conducted by Li et al. (2025) demonstrated that the mass ratio of slurry to rock chips and the particle-size distribution of the rock chips are key factors governing interface friction behavior [15]. These parameters intensify viscous, plowing, and adhesion effects at the interface, leading to the progressive deterioration of frictional behavior and ultimately causing abnormal increases in pipe jacking thrust and pipe-sticking incidents [15]. Choo, Ong, and their co-workers systematically quantified the effects of rock strength, the interface friction coefficient, and the surrounding-rock arching effect on pipe jacking thrust through back-analysis of rock strength parameters and numerical simulations [16–19]. Although previous studies have recognized the complexity of the mechanical behavior involved in pipe jacking through rock formations, current lateral frictional resistance models generally retain the modeling frameworks originally developed for soil formations. These models do not systematically incorporate the dynamic evolution of the multiphase rock–slurry–pipe interface and therefore cannot accurately characterize the actual pipe–rock contact state or the distribution of normal stress in rock formations. This limitation is a primary cause of systematic errors in pipe jacking thrust predictions for rock formations.

Furthermore, existing studies have provided insufficient insight into the additional resistance caused by rock chips generated through interface friction. Most laboratory direct shear tests have been conducted by manually adding rock chips to slurry. Such procedures cannot reproduce the complex mechanisms of surrounding-rock compression, fragmentation, and grinding that occur during pipe jacking. Consequently, they cannot accurately represent the actual interface contact state or normal stress distribution. This knowledge gap further limits the reliability of frictional resistance models and contributes to systematic errors in pipe jacking thrust prediction under rock formation conditions.

This study investigates the pipe jacking section beneath Provincial Highway S243 of the Ruyang Water Supply Project in the Qianping Irrigation District, Henan Province. It directly addresses the engineering problem of redundant intermediate jacking stations resulting from inaccurate pipe jacking thrust estimates. Laboratory direct shear tests were conducted to accurately determine the composite friction coefficients of sandstone–concrete–slurry interfaces with planar and rough split-surface configurations. The experimental results were then incorporated into a three-dimensional finite-element model to clarify the frictional behavior arising from the coupled interaction between the pipe and the surrounding rock. The findings provide a robust theoretical basis for optimizing the design of similar pipe jacking projects.

Building on prior investigations of pipe–rock contact mechanisms, slurry-induced friction reduction, and jacking-force evolution law, this study addresses the inherent limitations of conventional laboratory direct shear tests on planar interfaces by systematically performing comparative shear tests on both smooth intact and split sandstone surfaces. The obtained results quantitatively demonstrate that, under the condition of slurry lubrication, the friction coefficient of the rough split surface is approximately 20% higher than that of the ideal planar interface. By eliminating the oversimplified assumption of full uniform contact and adopting a refined numerical scheme where interfacial contact properties are dynamically updated in real time as jacking progresses, the established model fully reproduces the complete evolution process of pipe–rock contact, spanning from initial full slurry suspension, subsequent localized contact formation, to final coordinated load-bearing along the entire circumference of the pipe–soil interface. A three-way comparative analysis among in-situ field measurements, high-fidelity numerical simulations, and classical theoretical predictions further confirms that the two default assumptions of constant friction coefficient and uniform circumferential contact are the primary root causes for conventional empirical formulas to systematically overestimate the actual jacking force. These novel findings provide a reliable theoretical basis for the refined jacking-force design of pipe jacking projects in hard sandstone formations.

2. Project Background

2.1. Project Overview

The North Main Canal of the Qianping Reservoir Irrigation District serves the dual functions of municipal and rural water supply and agricultural irrigation for Ruyang, Ruzhou, and Jiaxian. The Ruyang section has a total length of 22.7 km. As a critical control segment of this large-scale infrastructure project, the underground crossing beneath Provincial Highway S243 at chainage K18 + 320 is one of the sections with the most complex geological conditions and greatest construction challenges along the entire alignment, as shown in Figure 1. This study focuses on the mechanical behavior of the water conveyance pipeline during pipe jacking. The pipeline was installed directly beneath Provincial Highway S243 using a trenchless pipe jacking method.

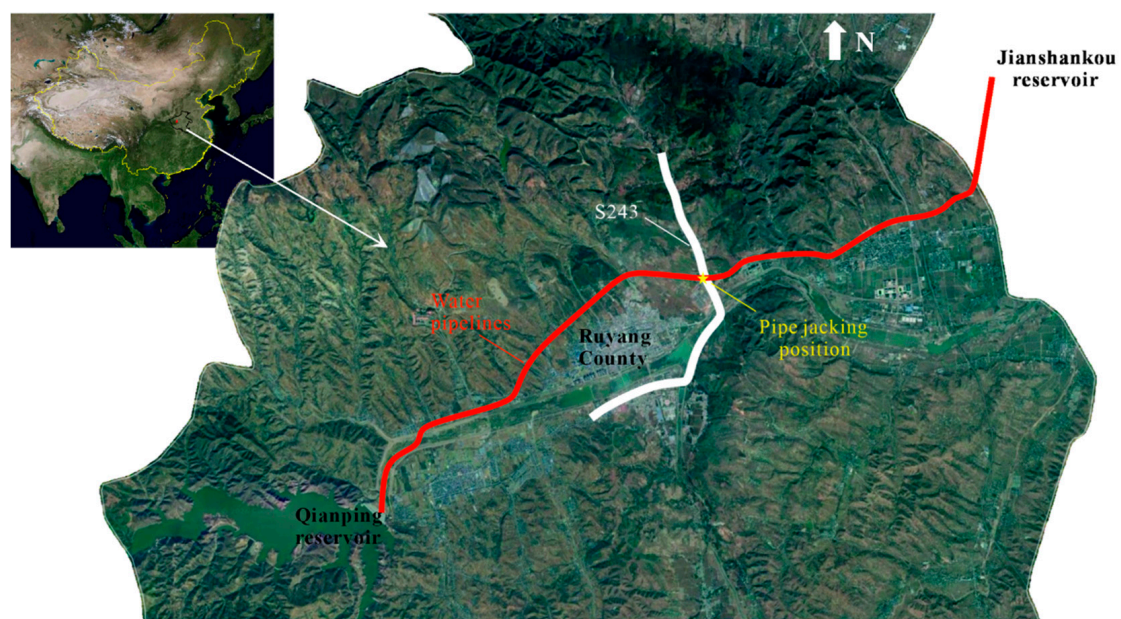


Figure 1. Pipeline layout of the Qianping Reservoir Irrigation District Project.

The pipeline consists of ductile iron reinforced concrete pipe (DRCP) segments with an outer diameter of 3.55 m and a wall thickness of 0.275 m, which classifies this project as a large-diameter pipe jacking case. The pipeline alignment is perpendicular to the highway, with a crossing angle of 90° . Along the entire jacking alignment, the soil cover above the pipeline ranges from 5 to 13 m. The launch and reception shafts were strategically located outside the physical boundaries of the highway embankment to ensure construction safety. Maintaining uninterrupted traffic on Provincial Highway S243 throughout the jacking operation was a mandatory requirement. Meanwhile, embankment settlement had to be strictly controlled to ensure high-precision pipeline installation throughout construction.

2.2. Stratigraphic Characteristics

Detailed site investigations and geological exploration revealed a medium-dense, variegated cobble layer directly beneath the provincial highway subgrade. The cobbles, primarily composed of andesite, quartz sandstone, and limestone, are generally well rounded, with particle sizes typically ranging from 20 to 100 mm. Quantitative particle-size analysis showed that particles larger than 20 mm accounted for 76.0% of the total mass. Isolated oversized boulders were also identified within the stratigraphic profile.

The voids within the cobble layer are predominantly filled with argillaceous material, along with a small amount of sandy material. Although weak cementation was observed locally, the overall degree of cementation within the cobble layer was low. Immediately below the cobble layer is a purplish-red to flesh-red sandstone formation characterized by a typical fine-grained texture and medium- to thick-bedded structure. The sandstone was classified as slightly weathered, and the recovered cores were generally columnar. Thin purplish-red shale layers were interbedded within the sandstone formation.

It is worth noting that the drilled geological boreholes did not penetrate the full depth of the sandstone formation. The maximum explored thickness was approximately 20.0 m, and the lower boundary of the formation was located at a depth greater than 20.0 m, corresponding to an elevation below 291.57 m, as shown in Figure 2a. The spatial relationship among the pipe dimensions, jacking alignment elevation, and surrounding strata is illustrated in Figure 2b. The mechanical parameters of the sandstone formation used in the numerical model were selected according to the Preliminary Design Report for the Ruyang Water-Supply Project of the North Main Canal in the Qianping Reservoir Irrigation District, Henan Province. Based on a comprehensive analysis of field in-situ tests and laboratory test results, the report provides recommended values by applying reasonably derived reductions to the physical and mechanical parameters of the rock and soil masses. These values are fully consistent with the empirical equivalent rock-mass parameter criteria for the slightly weathered, medium- to thick-bedded sandstone encountered in this project.

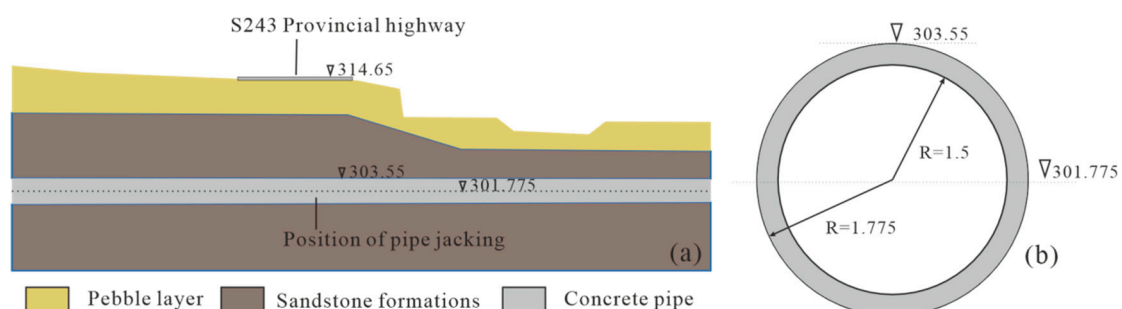


Figure 2. Pipe jacking design. (a) Geological cross section; (b) size and elevation (unit: m).

2.3. Key Parameters of the Pipe Jacking Operation

Comprehensive and continuous monitoring was conducted throughout the pipe jacking operation, and the principal field-monitoring results are summarized in Table 1. The jacking pressure listed in the table refers specifically to the oil pressure within the hydraulic propulsion system, whereas the pipe jacking thrust represents the structural load derived from the measured oil pressure. The raw hydraulic monitoring records exhibited a certain degree of scatter, primarily because of the inherent response lag of the hydraulic system, transient pressure surges during the startup and shutdown of hydraulic cylinders, and potential minor leakage. Therefore, although these measurements provide valuable insight into the overall mechanical evolution, they should be regarded only as indicators of general trends rather than as high-precision benchmarks for absolute validation. In addition, the slurry pressure within the excavation chamber was dynamically adjusted to balance the frontal resistance at the excavation face. The cutterhead support conditions were also continuously regulated to minimize construction-induced disturbance to the surrounding rock mass.

Table 1. Monitoring results during the jacking process.

Jacking Distance (m)	Jacking Force (kN)	Max Jacking Pressure (MPa)	Mud Silo Pressure (MPa)
0–19	2080–2600	8–11	0.02–0.07
19–35	2600–2860	10–11	0.06–0.07
35–55	2860–6240	15–16	0.06–0.07
55–82	6240–7280	16–18	0.06–0.07

The entire jacking process showed a clear stage-dependent mechanical response. The 76 m section covered by the numerical simulation was divided into three stages: 0–35 m, 35–70 m, and 70–76 m.

During the 0–35 m stage, the pipe jacking machine initially entered the sandstone formation and advanced progressively. This stage was further divided into two substages: 0–19 m and 19–35 m. Over the first 0–19 m, the cutterhead underwent a critical running-in process as it engaged with the intact rock mass, and the pipe jacking thrust increased steadily and predictably. Meanwhile, substantial fluctuations in the excavation-chamber pressure were observed, indicating that the operators were continuously adjusting the system to determine the minimum support pressure required to accommodate the inherent strength of the formation ahead while preventing excavation-face instability.

During the subsequent 19–35 m substage, both the pipe jacking thrust and the corresponding hydraulic pressure reached a stable plateau, with only marginal increases. The excavation-chamber pressure stabilized at a relatively high and uniform level of 0.06–0.07 MPa, indicating that the construction team had established an effective pressure-equilibrium condition. This condition not only provided effective support for the excavation face but also allowed the externally injected slurry envelope to achieve optimal friction reduction around the pipe, thereby maximizing construction efficiency.

After the jacking distance exceeded the 35 m threshold and the operation entered the 35–70 m stage, a pronounced surge in pipe jacking thrust was observed immediately after the pipe jacking machine passed this critical location. This surge indicated a fundamental transition in the mechanical interaction between the pipe and the surrounding geological medium.

3. Direct Shear Tests on the Frictional Characteristics of the Sandstone–Concrete Interface

Existing models for calculating lateral frictional resistance in pipe jacking generally neglect the influence of excavation-surface geometry on the frictional behavior of the rock-

concrete interface, which can lead to substantial errors in pipe jacking thrust predictions for rock formations. To address this research gap, this study is designed to characterize the evolution of frictional behavior at the sandstone–concrete interface during pipe jacking through rock formations, thereby providing a reliable experimental basis for selecting appropriate friction coefficients in pipe jacking thrust prediction models.

Based on the average density of the strata in the study area, the normal stress acting on the pipe jacking section was estimated from the overburden load to range from 200 to 400 kPa. This range was therefore adopted as the engineering basis for specifying the normal stress conditions in the subsequent laboratory tests.

Two representative interface geometries were investigated: a smooth planar surface (FS) and a splitting surface (SS). The FS configuration represents the idealized contact condition associated with regular excavation and an intact slurry film, whereas the SS configuration represents practical conditions involving underexcavation and fragmentation of the surrounding rock. Two construction-site typical interface conditions were also set as control variables: dry friction (DF) and bentonite–slurry lubrication, hereafter referred to as the slurry-lubricated condition (MF). Together, these interface geometries and friction conditions represent the contact characteristics commonly encountered during pipe jacking through rock formations. More complex contact conditions, such as those associated with highly jointed rock masses and slurry mixtures with different proportions, were beyond the scope of this study.

3.1. Experimental Methods

3.1.1. Materials and Specimen Preparation

The sandstone specimens were obtained from slightly weathered sandstone blocks exposed at the project site. The saturated uniaxial compressive strength of the sandstone ranged from 30 to 40 MPa, and its natural unit weight was 25 kN/m³. The shear strength parameters included an internal friction angle of 29° and a cohesion of 0.6 MPa. The concrete specimens were cast using the same mixture proportions as those of the pipe segments used in the project. The concrete had a strength grade of C30 and was cured under standard conditions for 28 days. The bentonite slurry used under the slurry-lubricated condition had the same mixture proportions as those used in the field, with a bentonite-to-water-to-soda ash ratio of 1:6:0.07. The slurry was thoroughly mixed for at least 30 min for each batch. Its density ranges from 1.1 to 1.6 g/cm³, Marsh funnel viscosity ranges from 31 to 49 s, pH value ranges from 7 to 9, and API fluid loss ranges from 10 to 25 cm³ per 30 min. No free-water separation was observed after the slurry had remained undisturbed for 24 h.

Two interface geometries were prepared: a smooth planar surface and a split surface. The smooth planar surface specimens were fabricated via precision cutting, while the split surface specimens were produced through indirect mechanical splitting. The sandstone blocks used in the tests measured 75 × 75 × 75 mm, and the concrete blocks measured 150 × 150 × 75 mm. These dimensions ensured that the sandstone shear surface remained fully covered by the concrete block throughout the shearing process, thereby promoting a uniform stress distribution across the interface.

3.1.2. Surface Roughness Measurement

A portable surface roughness tester was used to measure the surface roughness, expressed as the arithmetic mean roughness (R_a , in μm), of the sandstone and concrete specimens before shearing (Figure 3). Measurement points were selected via random sampling over a uniformly distributed grid. The points were evenly distributed across the 75 mm side-length shear surface, to ensure that the measurements adequately represented the roughness characteristics of the entire interface.

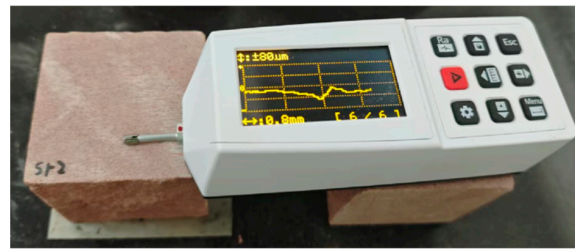


Figure 3. Surface roughness measurement procedure.

A total of 27 measurement areas on the sandstone specimens and 36 measurement areas on the concrete specimens were evaluated. All collected roughness data were statistically summarized using mean values and corresponding standard deviations. The median surface roughness values of the sandstone and concrete specimens were $7.282\ \mu\text{m}$ and $4.322\ \mu\text{m}$, respectively. The interquartile range (IQR) of the sandstone surface roughness was $1.556\ \mu\text{m}$, with measured values ranging from $5.859\ \mu\text{m}$ to $8.979\ \mu\text{m}$. These results indicated substantial variability in the sandstone surface roughness and pronounced differences among the measurement points. For the concrete specimens, the IQR was $1.914\ \mu\text{m}$, and the measured values ranged from $2.095\ \mu\text{m}$ to $5.982\ \mu\text{m}$, indicating greater dispersion in surface roughness. This variability was directly attributable to the inherent heterogeneity of concrete.

3.1.3. Test Program and Loading Conditions

Interface shear tests were conducted using an RLJW-2000 computer-controlled servo-controlled rock direct shear rheological test system (Figure 4a). During each test, a controlled normal stress was applied, and the interface shear stress was measured at different shear displacements to determine the friction coefficient. Normal stresses ranging from 200 to 400 kPa were applied to represent the in situ stress levels corresponding to the actual burial depth of the pipeline. The shear rate was set to 6–10 mm/min, which was converted from the actual on-site jacking speed to align with the field construction conditions.

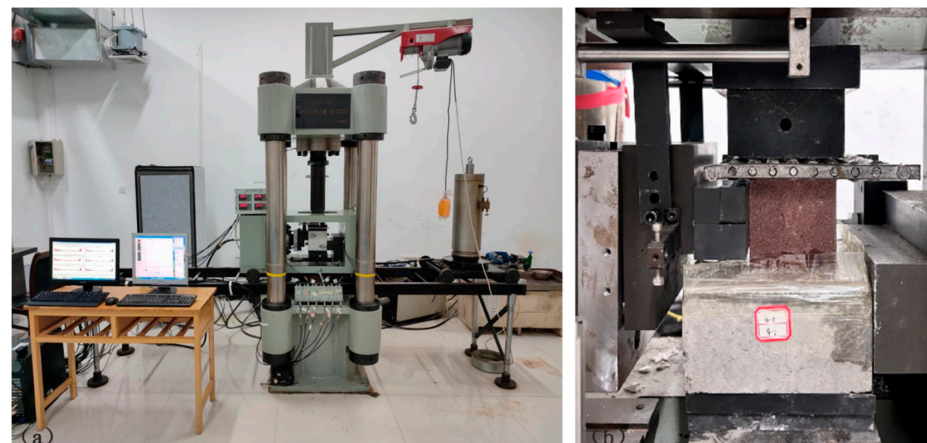


Figure 4. Direct shear testing apparatus and test procedure. (a) RLJW-2000 computer-controlled servo-controlled rock direct shear rheological test system; (b) test under the slurry-lubricated condition.

For each combination of normal stress, interface morphology, and lubrication condition, three replicate specimen sets were prepared and tested. Owing to the inherent heterogeneity in the roughness of the artificially split surfaces, slight scatter was observed among the replicate test results. The results obtained from representative specimens with similar surface morphological characteristics were therefore selected as the basis for the subsequent analysis, while anomalous data with a deviation exceeding 15% were excluded to ensure the reliability of the experimental conclusions.

For each test under the slurry-lubricated condition, 0.12 L of bentonite slurry was introduced into the interface to form an initial uniform slurry layer of approximately 0.5 cm in thickness (Figure 4b). A waterproof sealing sleeve was installed around the specimen assembly to prevent slurry leakage.

3.1.4. Determination of the Friction Coefficient

To determine the comprehensive friction coefficient of the sandstone–concrete interface under slurry lubrication, the residual shear strengths obtained at different normal stresses in the direct shear tests were linearly fitted. The slope of the fitted line was taken as the friction coefficient [20]. According to the shear stress–displacement curves, the shear stress essentially stabilized at a shear displacement of 6 mm. Therefore, the shear stress corresponding to a shear displacement of 6 mm was taken as the steady-state shear stress for linear fitting and analysis.

3.2. Result

3.2.1. Shear Behavior of the Planar Interface

The shear stress–displacement curves of the sandstone planar interface are shown in Figure 5. The shear stress–displacement curves of the sandstone planar interface under dry friction conditions (DF) exhibited distinct evolutionary patterns at different normal stress levels. Under high normal stress conditions, namely 300 kPa and 400 kPa, the shear stress showed a typical three-stage evolution with increasing shear displacement. In the initial static friction stage, the shear stress increased rapidly to a peak value. In the asperity failure stage, the shear stress dropped abruptly after reaching the peak. In the steady sliding stage, the shear stress fluctuated slightly within a relatively narrow range. Specifically, under a normal stress of 300 kPa, the peak shear stress was 370 kPa, and the residual shear strength at a shear displacement of 6 mm was approximately 310 kPa. Under a normal stress of 400 kPa, the peak shear stress was 405 kPa, and the residual shear strength at a shear displacement of 6 mm was approximately 273 kPa. Under the lower normal stress conditions of 200 kPa and 250 kPa, the peak shear stress was not pronounced, and the residual shear strengths at a shear displacement of 6 mm were 212 kPa and 242 kPa, respectively. The amplitude of shear stress fluctuation was also related to the normal stress level: it was relatively small under the 200 kPa and 400 kPa conditions, whereas it was relatively large under the 250 kPa and 300 kPa conditions.

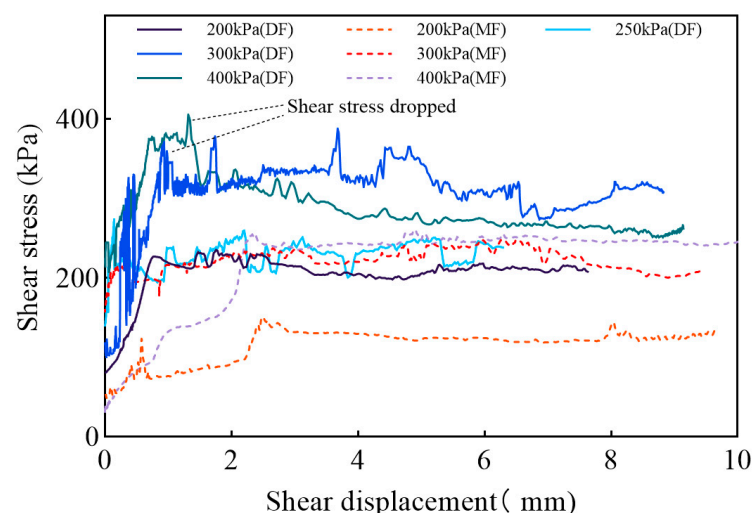


Figure 5. Stress–displacement curves from the planar friction test.

The shear stress–displacement curves of the sandstone planar interface under slurry-lubricated conditions differed significantly from those under dry friction conditions. Under high normal stress, the peak shear stress and the abrupt post-peak drop observed under dry friction conditions disappeared completely, and the curves generally exhibited a gradual increase in shear stress with displacement followed by stabilization. The fluctuation amplitude in the steady sliding stage was significantly lower than that under dry friction conditions. The residual shear strengths at a shear displacement of 6 mm were 126 kPa under 200 kPa normal stress, 234 kPa under 300 kPa normal stress, and 251 kPa under 400 kPa normal stress. At all normal stress levels, the shear stress under slurry-lubricated conditions decreased significantly compared with that under dry friction conditions, with the greatest reduction observed at 200 kPa, reaching only 41% of the corresponding dry-friction test value.

Surface roughness measurements from multiple parallel tests showed that under a normal stress of 400 kPa, the average roughness R_a of the sandstone specimen surface before shearing was $7.4\ \mu\text{m}$, which decreased to $6.6\ \mu\text{m}$ after shearing. This indicates that changes in surface roughness, together with the development of abrasion bands and scratches during shearing, were closely related to the observed shear behavior. Under dry friction conditions, abrasion bands convex toward the shear direction formed on the contact surface after shearing. Under the 400 kPa condition, multiple abrasion bands were distributed on the sliding surface, whereas under the 300 kPa condition, only one main abrasion band was observed. Red scratches were visible on the white sliding bands under the high normal stress conditions of 300 kPa and 400 kPa, and this feature corresponded to the peak and abrupt post-peak drop characteristics of the shear stress–displacement curves (Figure 6a,b). In contrast, the shear surface morphology under slurry-lubricated conditions differed clearly from that observed under dry friction conditions. Under high normal stress, scratches parallel to the shear direction were distributed on the sliding surface, and these scratches were concentrated within a single banded zone parallel to the shear direction, without crossing the entire sliding surface obliquely (Figure 6c,d).

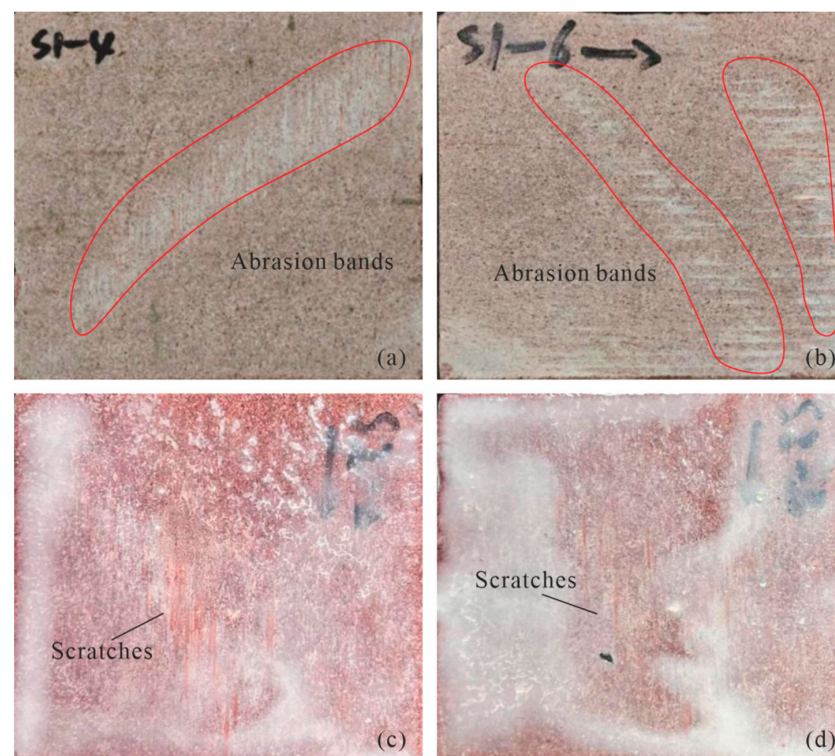


Figure 6. Shear surface of the smooth planar surface. (a) 300 kPa (DF); (b) 400 kPa (DF); (c) 300 kPa (MF); (d) 400 kPa (MF).

3.2.2. Shear Behavior of the Split Curved Interface

The shear stress–displacement curves of the sandstone split curved interface are shown in Figure 7. Except for the dry friction condition under a normal stress of 300 kPa, where the curve exhibited a distinct pattern of initial increase, subsequent decrease, and renewed increase, all other curves showed a gradual increase in shear stress with increasing shear displacement. Under dry friction conditions, when the normal stress was 300 kPa, the peak shear strength was 374 kPa, and the shear stress corresponding to a shear displacement of 6 mm was 260 kPa, indicating a pronounced shear-off sliding behavior. Under dry friction conditions, the shear stresses corresponding to a shear displacement of 6 mm were 270 kPa at a normal stress of 200 kPa and 467 kPa at a normal stress of 400 kPa. Under slurry-lubricated conditions, the shear stresses corresponding to a shear displacement of 6 mm were 194 kPa, 161 kPa, and 378 kPa at normal stresses of 200 kPa, 300 kPa, and 400 kPa, respectively. These results indicate that the introduction of slurry reduced the shear stress of the curved interface by 20–38%.

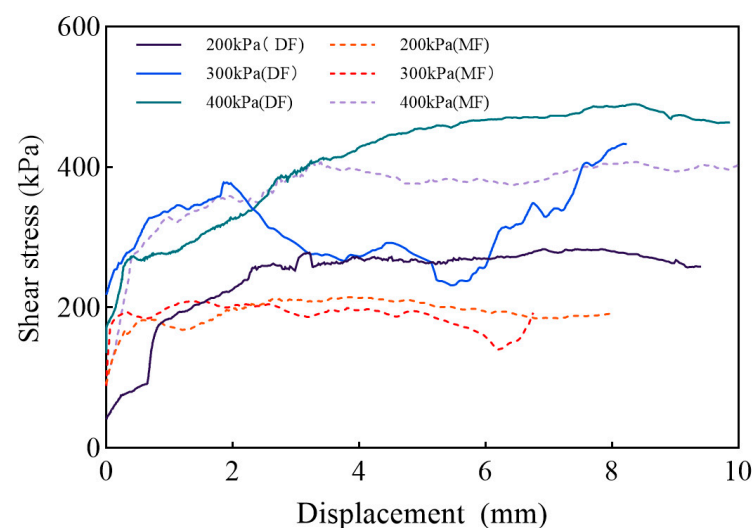


Figure 7. Stress–displacement curves from the curved-interface friction test.

Observations of the shear surface contact characteristics showed that all split curved specimens had at least two contact regions (Figure 8). The splitting surface was a highly undulating curved surface. During shearing, only part of the rock surface was in contact with the concrete surface. As a result, the normal stress was concentrated within a limited number of small rock-contact regions, leading to a significant increase in the local normal stress at these contact areas. After the contact points were flattened, shear sliding occurred. During dry friction sliding, debris generated by compression and shearing continuously accumulated on the shear surface, resulting in an increase in frictional resistance. After slurry was introduced, the slurry encapsulated and carried away the debris generated during shearing, thereby inhibiting debris accumulation and reducing the shear stress.

3.2.3. Friction Coefficients

In this study, the average steady-state value at a shear displacement of 6 mm was uniformly adopted as the basis for determining the friction coefficient. The shear stress–normal stress envelope was linearly fitted using the Mohr–Coulomb model, and the fitting results are shown in Figure 9.

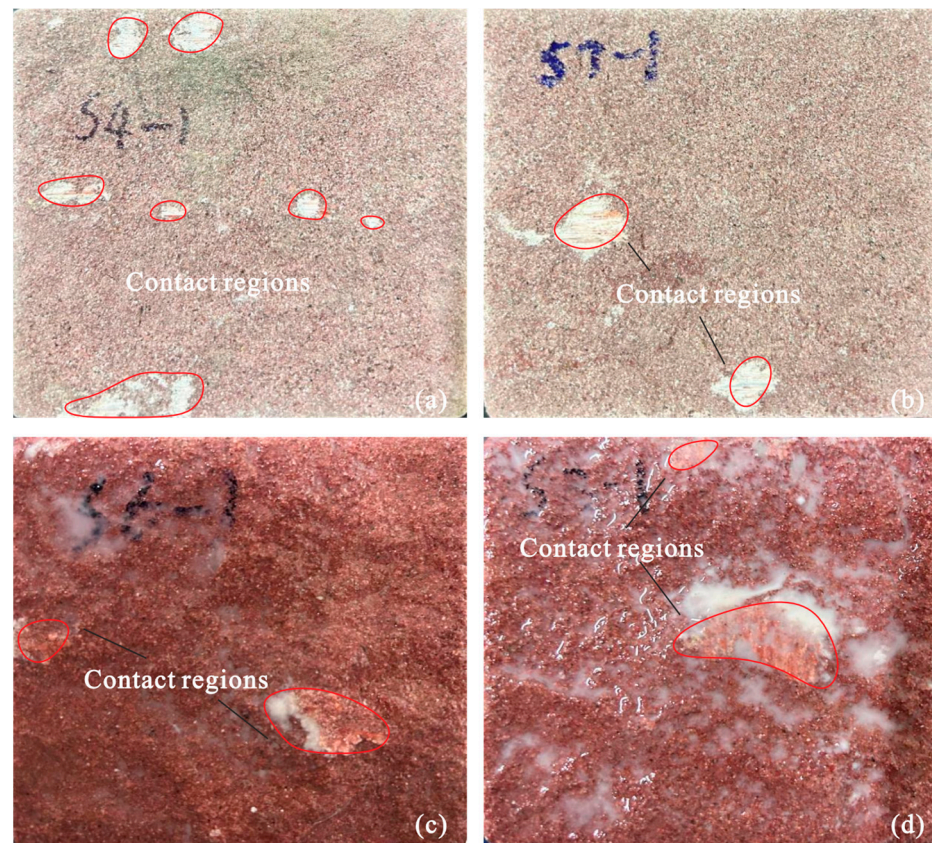


Figure 8. Shear surface of the splitting surface. (a) 300 kPa (DF); (b) 400 kPa (DF); (c) 300 kPa (MF); (d) 400 kPa (MF).

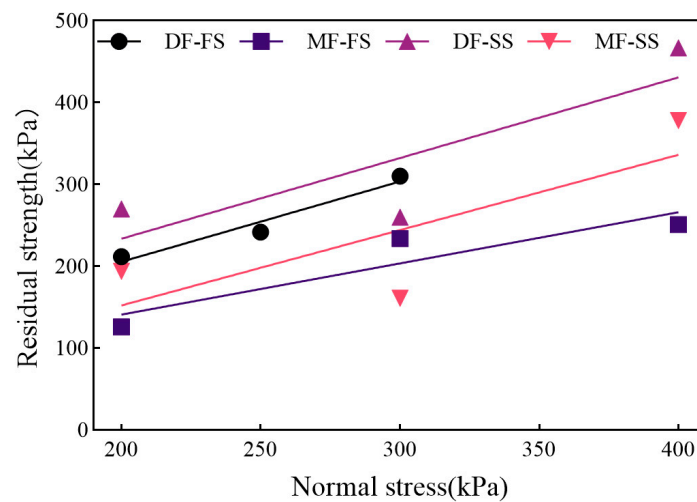


Figure 9. Fitting curves for the friction coefficient.

The shear strength parameters obtained by fitting the direct shear test data under different interface conditions are as follows: for the planar dry friction condition, the cohesion c was 9.7 kPa, the internal friction angle φ was 44.4° , and the coefficient of determination R^2 reached 0.95; for the planar slurry-lubricated condition, c was 16.2 kPa, φ was 32° , and R^2 was 0.85; for the split curved interface under dry friction, c was 36.8 kPa, φ was 44.7° , and R^2 was 0.72; for the split curved interface under slurry lubrication, c was 10.4 kPa, φ was 38.5° , and R^2 was 0.62.

Overall, the friction coefficients of the planar and split curved interfaces under dry friction conditions were very close, but the split curved interface exhibited a significantly

higher cohesion. This is attributed to the fact that, during shearing of the rough curved surface, rock debris generated by compression and abrasion at the interface filled the gaps between asperities, creating an additional interlocking resistance to sliding. As a result, a higher shear stress was required to initiate sliding. Under slurry-lubricated conditions, the friction coefficient of the planar interface was 0.63, representing a reduction of 36% compared with the dry friction condition, whereas the friction coefficient of the split curved interface was 0.8, representing only a 20% reduction. This difference further confirms that the debris-induced sliding resistance at the curved interface remained active during shearing and partially offset the friction-reducing effect of the slurry. In addition, the cohesion values of the two interface types under slurry-lubricated conditions were in a similar range.

4. Three-Dimensional Numerical Simulation of the Entire Pipe Jacking Process

4.1. Computational Model

The computational model primarily comprised two components: the excavated ground and the continuous pipe string, as shown in Figure 10. The modeled pipe had an outer diameter of 3.55 m and an inner diameter of 3.0 m. Structurally, the pipe jacking system consisted of a steel cutterhead section and a trailing concrete pipe section. The cutterhead section was 6 m long, and the continuous pipe string immediately behind it was 76 m long.

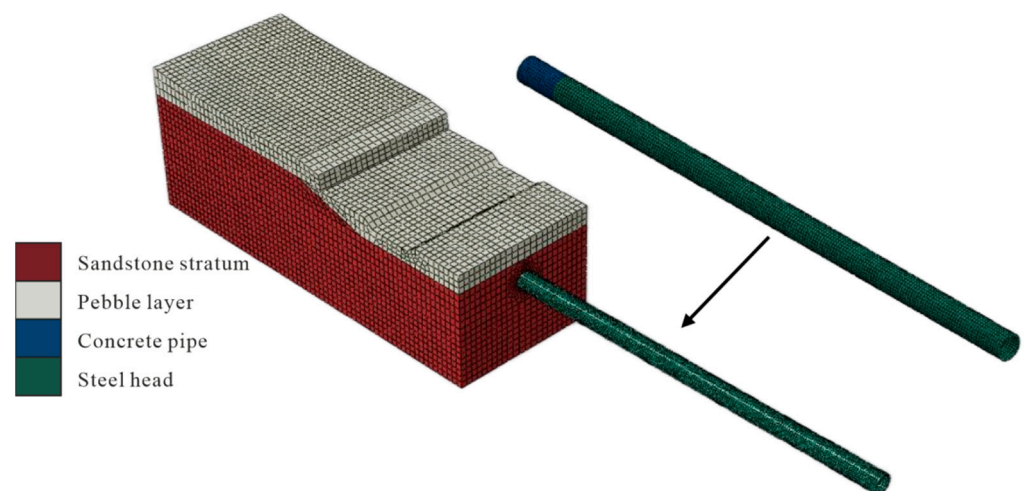


Figure 10. Numerical simulation calculation model.

Notably, the simulated jacking distance was set to 76 m, instead of the full modeled length of 82 m. The 6 m length of the cutterhead was excluded from the incremental jacking distance, thereby accurately representing the actual construction sequence.

To minimize the influence of boundary and scale effects on the numerical results, the surrounding ground domain was extended to approximately 3–5 times the excavation diameter in all radial directions from the pipe centerline. The model geometry was defined as follows. Taking the initial jacking face as the reference plane, the ground domain extended 82 m in the jacking direction, defined as the X-direction, and had a width of 30 m perpendicular to the jacking direction, defined as the Z-direction. The top of the model represented the ground surface and was treated as a free boundary normal to the Y-direction. The upper cobble layer and the underlying target sandstone formation were arranged sequentially with depth. The bottom boundary was located 15 m below the pipe centerline.

The main computational domain was discretized entirely using eight-node linear hexahedral solid elements with reduced integration (C3D8R). This element adopts one-point reduced integration, and viscous hourglass control was employed to ensure convergence of the nonlinear contact analysis while effectively suppressing shear locking under large deformation and numerical errors associated with hourglass modes. A small number of six-node linear wedge solid elements, accounting for less than 0.3% of the total element count, were generated solely as transition elements in non-critical regions with local geometric transitions or stringent shape constraints. These elements were not susceptible to hourglass modes and therefore have no significant effect on the overall computational accuracy or the effectiveness of hourglass control.

To rigorously verify mesh independence, three levels of computational meshes with coarse, medium, and fine discretization densities were successively constructed, and the calculation results of different grids are shown in Table 2. The final numerical model comprised 62,473 nodes and 53,164 elements. The complete mesh parameters and final computational results are summarized below:

Table 2. Computational results for different mesh densities.

Mesh Level	Total Number of Elements	Total Number of Nodes	Global Element Size Range (m)	Local Element Size at the Pipe–Rock Interface (m)	Local Element Size Around the Cutterhead (m)	Calculated Total Jacking Force at 76 m (kN)
Coarse mesh	30,436	35,645	0.6–2.5	0.6	0.2	6169
Medium mesh	62,473	53,164	0.4–1.25	0.4	0.2	6823
Fine mesh	111,671	127,065	0.3–1.0	0.3	0.2	6601

Comparison of the three sets of results showed that the relative difference in the calculated jacking force between the medium and fine meshes was only 3.3%. Further mesh refinement produced numerical variations within the acceptable range for engineering applications; therefore, further increasing the mesh density did not significantly improve accuracy but substantially increased computational time. Considering the balance between computational accuracy and efficiency, the medium mesh was selected for the formal simulations. Three layers of locally refined elements were assigned to the pipe–rock contact interface, while five layers were used in the stress-concentration region around the cutterhead. These refined regions fully covered the critical zones experiencing pronounced changes in contact conditions during jacking. The reliability of the adopted meshing scheme was thus confirmed through the mesh-independence analysis.

Several complex geological conditions and construction-induced influencing factors were reasonably simplified in this full engineering-scale numerical simulation. The model did not explicitly account for the discrete effects of geological discontinuities, such as joints and bedding planes, nor the spatial differential distribution of rock weathering degree. In addition, key phenomena such as overexcavation, progressive rock fragmentation at the cutterhead face, and the dynamic evolution of the slurry-layer thickness during jacking were not explicitly modeled. The simulation primarily focused on evaluating the pipe–rock contact state induced by excavation-related deformation in the sandstone formation under slurry-lubricated pipe jacking conditions. The slurry pressure at the pipe–rock interface was applied as an equivalent boundary stress.

4.2. Material Parameters

The parameters for the sandstone formation and cobble layer were primarily obtained from the geological investigation report, and supplemented by standard empirical parameters of similar geotechnical materials. The interface friction parameters were selected

based on the direct shear test results. Specifically, the concrete–sandstone interface friction coefficient was determined directly from the laboratory shear tests. Since no dedicated tests were conducted in this study to measure the steel–concrete interface friction coefficient, this parameter was selected based on established values reported in the literature and experience from representative engineering projects [21–24].

The input parameters for the numerical model were determined by integrating the field investigation data with the laboratory test results. All key parameters are summarized in Table 3.

Table 3. Model parameters.

Type	Density kg/m ³	Elastic Modulus /Pa	Poisson 's Ratio	Internal Friction Angle /°	Cohesion /Pa	Friction Coefficient
Sandstone stratum	2450	1×10^9	0.3	33	0.7e6	-
Pebble layer	2050	1.5×10^8	0.38	30	0.04e6	-
Concrete pipe	2600	31×10^9	0.2	-	-	0.8
Steel head	7850	210×10^9	0.18	-	-	0.2

An elastoplastic Mohr–Coulomb constitutive model was adopted to characterize the mechanical response of the sandstone formation. This choice was primarily based on three project-specific applicability considerations. First, under conventional confining pressure conditions in geotechnical engineering, the elastoplastic Mohr–Coulomb model could accurately describe the shear yielding and plastic flow behavior of rock masses. Its parameters had clear physical meanings and were fully compatible with the geotechnical investigation parameter system adopted for this project, without requiring additional complex parameters that are difficult to obtain via conventional laboratory and field tests. Second, the burial depth covered by the model ranged only from 5 to 13 m, corresponding to an initial in-situ stress of less than 400 kPa. Under these conditions, excavation-induced unloading of the surrounding rock was limited, and deformation dominated by nonlinear dilation or dilatancy under high-stress conditions was not expected. Therefore, there was no need to introduce a more complex constitutive model, such as the Drucker–Prager model, which is specifically designed for high in-situ stress environments. Third, the primary objective at this stage was to investigate the evolution of pipe–rock contact and the macroscopic jacking force response, rather than the progressive failure of internal joints and fractures within the rock mass. Therefore, the Hoek–Brown criterion was not required for refined characterization of discontinuous rock-mass strength. For the present application, the Mohr–Coulomb model provided an optimal balance between numerical accuracy and computational efficiency.

The concrete pipes and steel cutterhead were modeled as linearly elastic materials. The concrete–sandstone interface friction coefficient used in the numerical model was obtained from the direct shear tests described in Section 3.

4.3. Model Governing Equations, Boundary Conditions, and Loading Scheme

The three-dimensional numerical model developed in this study was established within a continuum-mechanics-based finite element framework. The governing equations and boundary conditions employed in the simulation are summarized below:

(1) Equilibrium equation

Under the small-deformation assumption, the static equilibrium condition was defined as:

$$\nabla \cdot \sigma + \rho g = 0 \quad (1)$$

where σ is the total stress tensor, ρ is the equivalent density of the formation, and g is the gravitational acceleration vector.

(2) Geometric equation

The small-strain symmetrical gradient formulation was adopted for the formulation:

$$\varepsilon = \frac{1}{2} (\nabla u + \nabla u^T) \quad (2)$$

where ε is the strain tensor and u is the nodal displacement vector.

(3) Constitutive relationship

Both the rock mass and concrete pipe segments were simulated using an elastoplastic constitutive framework. In the elastic deformation regime, the generalized Hooke's law was implemented:

$$\sigma = D^e : \varepsilon^e \quad (3)$$

where D^e is the elastic stiffness tensor and ε^e is the elastic strain tensor.

(4) Boundary conditions

Normal displacement constraints were imposed on all lateral boundaries of the model, while the bottom boundary was fully fixed in all three translational directions. The ground surface was defined as a traction-free boundary, and the initial in-situ stress field was generated automatically via self-weight gravitational equilibrium.

(5) Multi-body contact model

The normal interaction at the interface between the pipe segments and the surrounding sandstone was modeled using a standard hard-contact algorithm. Compressive stress can be fully transmitted once the normal gap between the contact surfaces is completely closed, whereas contact is automatically released if any tensile stress is detected at the interface. The tangential interaction was defined using a penalty-based friction formulation, which permits a small magnitude of "elastic slip" between mating surfaces. The input friction coefficient was directly determined from the composite interface friction parameters obtained from the laboratory direct shear tests conducted in this study. The mechanical effects at both the slurry-pipe and slurry-surrounding rock interfaces were represented by an equivalent uniformly distributed pressure load coupled with an equivalent buoyancy force.

Because the pipe was installed at a relatively shallow depth, the initial in situ stress field was generated by applying gravitational loading to the entire mesh. The resulting stress distribution is shown in Figure 11. As illustrated, the initial in situ stress at the pipeline depth differed substantially between the river channel and embankment sections. This spatial variation represents a typical topographic effect caused by variations in terrain and overburden thickness.

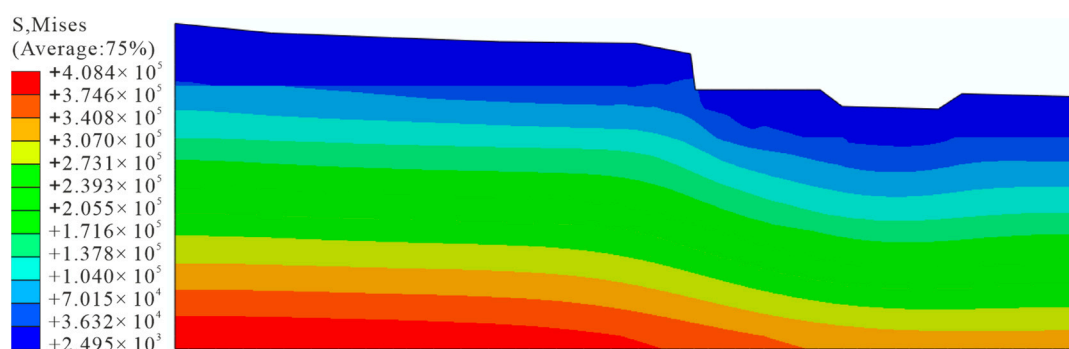


Figure 11. Initial in situ stress distribution (unit: Pa).

The excavation process was simulated using the element birth-and-death technique. During the initial stage, the ground elements within the cutterhead region were deactivated

to represent material removal, and the resulting excavation cavity was simultaneously occupied by cutterhead structural elements. The subsequent continuous jacking process was simulated incrementally using displacement control. After each excavation step, the corresponding concrete pipe segment elements were activated. Based on the actual construction schedule, the entire jacking process was divided into four discrete operational cycles, resulting in a total simulated jacking distance of 76 m. This 76 m jacking distance excludes the inherent 6 m length of the steel cutterhead.

The mechanical interface between the pipe and the surrounding rock mass was modeled using a penalty-based frictional contact algorithm. The friction coefficient governing tangential behavior was taken directly from the direct shear test results for the slurry-lubricated rock–concrete interface, thereby representing the complex pipe–rock contact behavior. A uniform equivalent pressure of 100 kPa was also applied directly to the interface with the surrounding rock mass. It should be emphasized that the numerical framework did not explicitly include a separate discrete slurry layer. Therefore, the applied pressure of 100 kPa represented the support pressure generated by concurrent slurry injection during construction. Accordingly, the interface results should be interpreted strictly as an equivalent interface interaction analysis, rather than as an exact simulation of the three-phase rock–slurry–pipe contact mechanism.

The Technical Specification for Pipe Jacking of Water Supply and Drainage Engineering provides a standardized calculation formula for the frontal resistance in pipe jacking engineering:

$$N_F = \frac{\pi}{4} D_g^2 \gamma H \quad (4)$$

where D is the outer diameter of the pipe jacking cutterhead, γ is the unit weight of the ground, and H is the buried cover depth above the pipe centerline.

The frontal resistance calculated using this equation ranged from approximately 1000 to 4000 kN. To represent this variable resistance in the numerical model, a piecewise constant loading strategy was implemented instead of a fully continuously varying load. In the river channel section, where the overburden was relatively shallow and the jacking distance was 36 m, a uniform pressure of 200 kPa was applied to the front face of the cutterhead. In the embankment section, a higher uniform pressure of 400 kPa was applied to account for the elevated terrain and increased overburden stress. Both sets of pressures were applied in the direction opposite to the jacking movement to represent the frontal resistance encountered by the advancing cutterhead.

The total pipe jacking thrust in the model was obtained by summing the axial reaction forces at the rear-boundary nodes subjected to displacement control. This extraction method accounted for both the integrated frontal resistance generated by the pressure acting on the cutterhead face and the penalty-based contact friction distributed along the entire pipe string.

4.4. Evolution of Pipe Jacking Thrust

To rigorously evaluate the reliability of the numerical model, a multidimensional validation framework was adopted instead of relying on a single statistical metric. The simulated pipe jacking thrust was obtained by summing the nodal reaction forces at the axial loading nodes along the pipe string. Comparison with the field measurements showed close agreement in several respects, including the locations of stage transitions, peak-value deviations, and mean relative error. The correlation coefficient was 0.98, indicating strong overall consistency in both the evolutionary trend and magnitude of pipe jacking thrust (Figure 12).

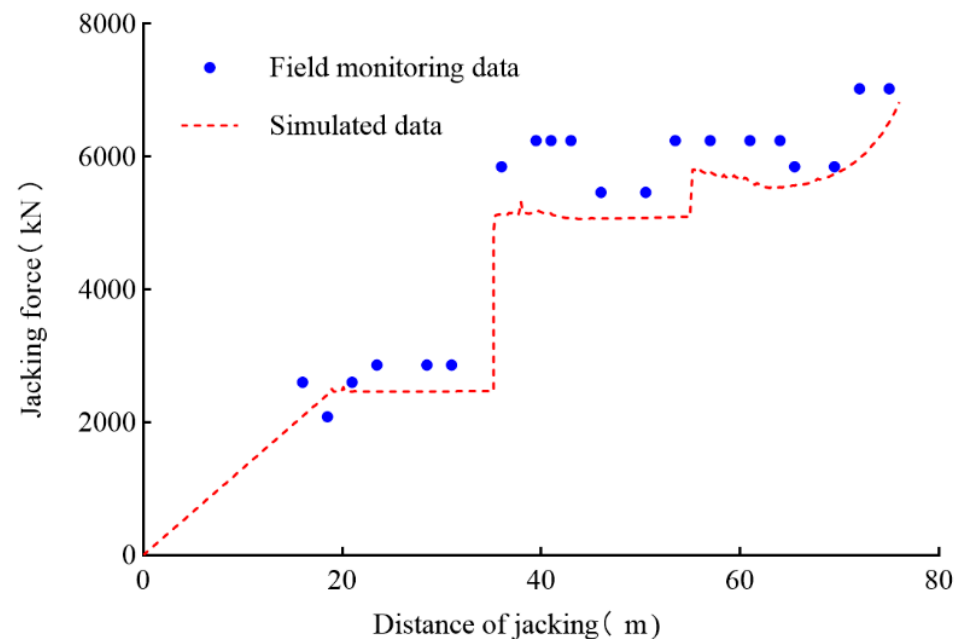


Figure 12. Jacking force in the sandstone stratum.

Nevertheless, the field measurements exhibited pronounced random fluctuations, whereas the numerical results showed a smoother, stepwise increase. This discrepancy arose from a combination of field conditions and modeling assumptions. During jacking, excavation by the cutterhead in the sandstone formation induced microcrack propagation, localized rock spalling, and varying degrees of overexcavation and underexcavation, all of which contributed directly to real-time fluctuations in pipe jacking thrust. As jacking progressed, progressive rock fragmentation and continuous rock chip intrusion into the annular gap also caused fluctuations in the friction-reducing performance of the slurry. In addition, unavoidable modeling simplifications introduced systematic discrepancies. In particular, the idealized representation of discontinuous rock-mass fracturing and the assumption of uniform slurry properties resulted in a smoother simulated response.

Although the field measurements fluctuated because of geological heterogeneity, variations in slurry performance, and construction-induced disturbances, the numerical results clearly revealed a three-stage evolution of pipe jacking thrust with jacking distance. This characteristic pattern was strongly coupled with the spatial variability of the in situ stress field governed by surface topography.

Initial stable stage (0–35 m): The pipe jacking thrust remained relatively constant, fluctuating within a narrow range of 2600–2800 kN. This section corresponded geographically to the relatively flat river channel area, where the overburden was shallow, the initial in situ stress was low, and the mechanical interaction between the pipe and surrounding rock was limited. Notably, the simulated pipe jacking thrust remained nearly constant over the 20–35 m subinterval. This plateau primarily resulted from the uniform piecewise loading scheme, the establishment of a stable contact state, and the constant interface parameters assigned to this geological section. However, the pronounced plateau may also have been partly attributable to inherent model simplifications that prevented the full reproduction of minor local variations observed under field conditions.

Pronounced surge stage (at 35 m): A distinct stepwise increase in pipe jacking thrust occurred at a jacking distance of 35 m. This location corresponded to the point at which the pipeline passed beneath the embankment slope of Provincial Highway S243. The abrupt topographic transition substantially increased the stress in the surrounding rock and sharply intensified the normal confinement acting on the pipe. Field construction

records supported this mechanical transition: the number of operating hydraulic jacks was increased from four to six to accommodate the sudden increase in pipe jacking thrust.

High-level stable and secondary-growth stage (35–76 m): From 35 to 70 m, the pipe jacking thrust stabilized at an elevated level, ranging from 5800 to 6300 kN. After the jacking distance exceeded 70 m, the pipe jacking thrust increased further and reached a maximum of approximately 7000 kN. This secondary increase resulted from the superimposed influence of local topography on the in situ stress field under already elevated stress conditions.

These results demonstrate that the stage-dependent evolution of pipe jacking thrust was not random but was intrinsically governed by the spatial distribution of the in situ stress field.

4.5. Contact-State Analysis

Figure 13 presents the distribution of contact states between the surrounding rock mass and the concrete pipe at different jacking distances. The numerical results indicated that line contact was identified as the predominant contact pattern during pipe jacking through the rock formation. At a jacking distance of 19 m, almost no mechanical contact occurred between the external pipe wall and the surrounding rock mass, indicating that the pipe was primarily supported in a slurry-suspended state during the initial stage. At 35 m, localized contact began to develop between the external pipe wall and the surrounding rock mass. As the jacking distance increased to 55 m, the contact zones gradually extended toward the pipe crown, invert, and sidewalls. After the full 76 m jacking construction was completed, the contact area expanded further and reached its maximum extent during the entire jacking process.

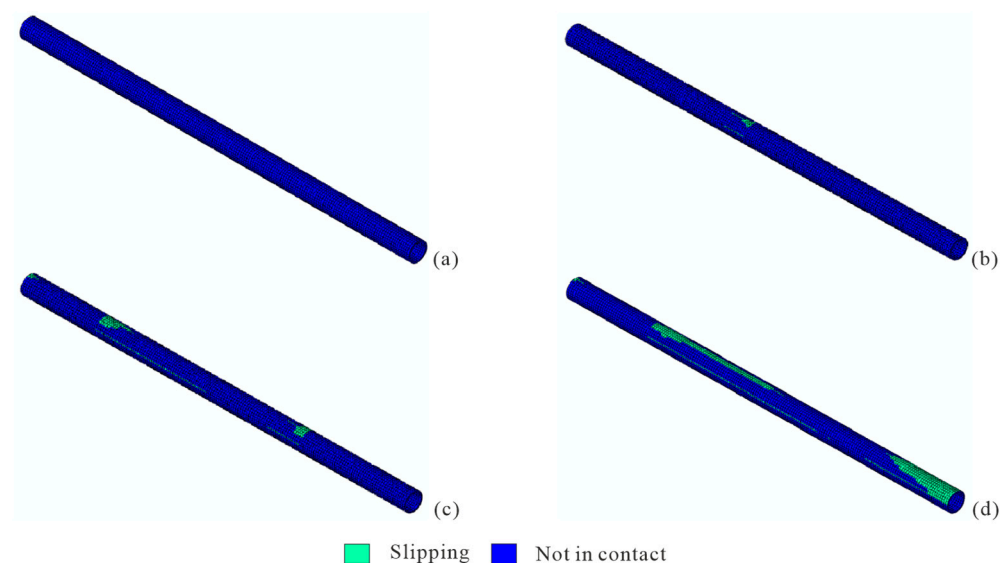


Figure 13. Contact state of the concrete pipeline at different jacking distances. (a) Jacking 19 m; (b) Jacking 35 m; (c) Jacking 55 m; (d) Jacking 76 m.

The progressive expansion of the non-uniform pipe–rock contact zones was the primary mechanical factor governing the three-stage evolution of pipe jacking thrust identified in Section 4.4. During the initial jacking stage, the nearly complete slurry-suspended state corresponded to a low and stable pipe jacking thrust plateau. At the topographic transition section at a jacking distance of 35 m, the sudden development of localized contact triggered a sharp, stepwise increase in pipe jacking thrust. Subsequently, the continued circumferential expansion of the contact area led to the gradual accumulation of additional frictional resistance, ultimately resulting in a secondary stable increase in the total pipe jacking thrust at the later stage of the entire jacking process.

5. Discussion and Limitations

5.1. Analysis of Experimental and Numerical Results

The direct shear tests conducted in this study clarified the differences in frictional behavior between two typical sandstone–concrete interfaces. Under the identical slurry lubrication condition, the friction coefficient of the rough split surface was 20% higher than that of the smooth planar surface. The primary mechanism underlying this phenomenon was that asperities on the split interface locally penetrated the slurry film, resulting in direct interlocking contact between the rock and pipe asperities. During interface shearing, the ploughing effect of these asperities was superimposed on the basic sliding friction, leading to a significant increase in interface frictional resistance. This finding explained the common engineering observation that the actual jacking force is generally higher than that predicted by idealized slurry-suspension models. It also confirmed that the idealized modeling assumption of a uniform slurry film completely surrounding the pipe circumference deviates substantially from actual engineering conditions.

The dynamic evolution of the contact state obtained from the numerical simulations indicated that, during the initial stage of jacking, the pipe remained in an approximately suspended, non-contact state under slurry buoyancy. As the jacking distance increased, the self-weight of the pipe first induced localized line contact at the pipe invert. Subsequently, formation heterogeneity and eccentric jacking loads gradually caused the contact state to evolve into a combined multi-point and multi-line contact pattern. Eventually, an extensive contact state developed in which the pipe crown, invert, and sidewalls jointly carried the load. This evolutionary path demonstrated that the skin friction acting on a pipe jacked through rock was not constant but increased nonlinearly with the jacking distance and growth of the contact area. This was the primary reason why conventional empirical formulas based on uniform contact and constant frictional resistance systematically overestimated the measured jacking force. Conventional methods directly included the entire circumferential surface area of the pipe in the friction calculation, whereas during a substantial portion of the actual jacking process, the pipe–rock contact area accounted for only a small fraction of the total circumferential area. Consequently, the frictional resistance was inevitably overestimated.

5.2. Study Limitations

The current findings have clearly defined applicability boundaries and limitations that should be addressed in future research:

1. The laboratory direct shear tests covered a normal stress range of only 200–400 kPa, corresponding to conventional sandstone pipe-jacking conditions at burial depths of 5–16 m. They did not cover high-stress pipe-jacking conditions in deep rock formations at burial depths exceeding 30 m. Under higher normal stresses, the crushing of asperities on rough rock surfaces may further alter the evolution of interface friction. Therefore, the present findings should not be extrapolated indiscriminately to ultra-deep pipe-jacking projects.
2. An equivalent homogeneous constitutive model was adopted for the sandstone formation in the numerical simulations. Discontinuities such as joints and bedding planes within the rock mass were not explicitly incorporated. Consequently, the model cannot directly reproduce extreme nonlinear failure behaviors, such as fracture propagation within the rock mass or pipe blockage caused by localized rock spalling.
3. The experiments did not consider the accumulation of continuously generated rock cuttings within the interface gap during construction. Prolonged grinding of these cuttings may alter both the interface roughness and the rheological properties of

the slurry. The degradation of the interface friction coefficient during long-distance jacking therefore requires further investigation through longer-duration shear tests.

Furthermore, inherent discontinuities within the rock mass, such as joints and bedding planes, can fundamentally alter the localization of surrounding-rock deformation and its progressive failure mechanisms [25]. The geometry and propagation patterns of fractures directly govern stress-release paths and failure modes within the rock mass, and this mechanism may also exert a significant influence during pipe jacking through sandstone formations. Although the equivalent homogeneous numerical model used in this study cannot directly reproduce these discontinuous failure processes, future engineering applications may account for them by modifying the contact-state evolution characteristics identified herein according to the joint orientation and joint density obtained from site investigations. Introducing a range-based correction coefficient for the effects of discontinuities could further improve the accuracy of jacking-force predictions in jointed sandstone formations.

6. Conclusions

Taking the large-diameter pipe jacking project located in the sandstone stratum of Qianping Irrigation District, Henan Province, as the research background, this study combined laboratory interface direct shear tests, three-dimensional numerical simulations, and comparative verification with field measurements to systematically investigate the dynamic evolution characteristics of the multiphase pipe–rock–slurry interface contact. The primary mechanisms responsible for systematic prediction errors in conventional frictional resistance models originally developed for soil strata pipe jacking scenarios were also clarified. The main conclusions are drawn as follows:

The shear behavior varied substantially between different sandstone–concrete interfaces. Under dry-friction conditions, the smooth planar interface exhibited a characteristic response comprising a peak shear stress, a sudden post-peak drop, and subsequent post-peak gradual stress stabilization. In contrast, the rough split-surface interface exhibited pronounced strain-hardening behavior. The interlocking effect of rock chips generated during the shearing process along the rough split-surface interface substantially increased the interface cohesion. The steady-state friction coefficients under both dry-friction conditions were approximately 0.98.

Thixotropic slurry lubrication substantially reduced the interface shear stress, although its effectiveness was strongly governed by the interface geometric morphology. For the smooth planar interface, slurry lubrication reduced the shear stress by up to 41% relative to the value under dry-friction conditions. For the rough split-surface interface, the reduction was only 14%, because the friction-reducing effect of the slurry was partially offset by the sliding resistance generated by interlocked rock chips. Under slurry-lubricated conditions, the cohesion values of the two interface configurations remained at a similar level.

During pipe jacking through the sandstone formation, the pipe–rock contact state did not conform to the conventionally assumed uniform full-circumference contact pattern. Instead, it followed a complete dynamic evolution process from initial slurry-supported suspension with no contact, through localized line contact and combined multipoint contact, to widespread full-circumference contact, where the pipe crown, invert, and sidewalls jointly carry the normal load. This typical nonuniform contact pattern directly resulted in actual lateral frictional resistance being substantially lower than the prediction from conventional empirical equations.

The three-dimensional numerical model calibrated with experimentally determined interface friction parameters produced pipe jacking thrust values that agreed well with the field measurements, demonstrating the reliability of the equivalent interface interaction analysis framework adopted in this study. In contrast, conventional empirical methods

based on the assumptions of uniform full-circumference contact and constant friction coefficient systematically overestimated the actual frictional resistance during pipe jacking through rock formations. Such overestimation may lead to excessive capacity design and redundant configurations of intermediate jacking stations and jacking equipment, thereby imposing unnecessary extra project costs.

Taking the S243 Provincial Highway undercrossing pipe jacking project as the engineering case, the maximum jacking force over the entire drive was estimated to be 13,192 kN using the conventional empirical formula. In contrast, the maximum jacking force obtained from the dynamically updated interfacial contact simulation calibrated with the laboratory experimental parameters in this study reached only 6823 kN. The relative deviation between this simulated value and the final peak jacking force recorded by field monitoring was within 4%, which demonstrated close agreement with the actual engineering conditions. This accurate jacking force prediction could directly support a 48% reduction in the rated load selected for the main jacking cylinders, and eliminated two redundantly designed intermediate jacking stations along the jacking alignment. It therefore provides a quantitative optimization scheme that reduces the equipment and construction costs associated with the jacking system, and can be directly extended to similar pipe jacking projects in hard sandstone formations.

All experimental and numerical findings obtained in this study are applicable only to shallow-buried pipe-jacking projects in slightly weathered sandstone formations. These conclusions should not be directly extrapolated to other scenarios, such as highly weathered rock formations with extensively developed joints, or deep-buried hard-rock pipe-jacking projects under high in-situ stress. The engineering applicability and limitations of the proposed method were systematically discussed and clearly defined in Section 5, Discussion and Limitations. Future parallel tests covering different lithologies and burial depths could further extend the applicable scope of the present findings.

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any core scientific reasoning, academic judgment, research innovation decision-making or conclusion generation processes.

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