Study on the construction mechanical response of the double-shield TBMs of a long diversion tunnel crossing a fault fracture zone

Weiyang Xia\textsuperscript{1}, Kangjian Zhang\textsuperscript{2,3}, Runlin Hong\textsuperscript{2,3} and Zhiqiang Zhang\textsuperscript{2,3,4}

\textsuperscript{1}China Railway No.2 Engineering Group Co., Ltd., Chengdu 610032, China
\textsuperscript{2}School of Civil Engineering, Southwest Jiaotong University, Chengdu 610031, China
\textsuperscript{3}Key Laboratory of Transportation Tunnel Engineering, Ministry of Education, Southwest Jiaotong University, Chengdu 610031, China

Email: Clarkchang68@163.com

Abstract. The Bheri Babai Diversion Multipurpose Project (BBDMP) in Nepal is located in the southern foothills of the Himalayas. The tunnel of this project passes through the Siwalik rock stratum. In this paper, the mechanical numerical model of the double-shield tunnel boring machine (TBM) crossing the fault fracture zone is established based on the morphology of the developed joint in the Bheri fault zone of the BBDMP. By analyzing the influence of joint group on double-shield TBM tunneling, the variation rules of the surrounding rock displacement, plastic zone distribution, and internal force of TBM crossing the fault fracture zone are obtained. The results of the research demonstrate the following: (1) when the single-shield tunneling mode is used in the fault fracture zone, the surrounding rock displacement is larger than that in intact rock mass, and the dissymmetry of joint geometry leads to the dissymmetry of the displacement distribution. (2) The plastic zone of the surrounding rock is mainly distributed along the geometric plane of two joints. The maximum value appears in the area from the left arch waist to the left arch shoulder and from the right arch waist to the right foot of the wall. The joints at all the other sites reach the yield state, except for the area around the vault and inverted arch, with a range of about 0.3 times the tunnel diameter. (3) The existence of joint surface leads to the asymmetrical distribution of the internal force in the pipe segment. The minimum safety coefficient appears at the left shoulder and right foot of the wall.

1. Introduction
Tunnel boring machines (TBMs) have been increasingly used in recent years owing to its high efficiency and safety. However, its adaptability to bad geological conditions, such as fault fracture zones, is poor [1, 2]. The main challenges in underground excavation through fault rock are extremely short stand-up time, tunnel face instabilities, and high radial convergence [3-7]. TBM tunneling in the fault zone may cause numerous problems, which include high ground pressure, TBM jamming, tunnel face instability or collapse, and a high or sudden inflow of water [8]. TBM tunneling in zones with poor quality has been quite problematic, for example, in the case of the Tuzla tunnel in Turkey [9], the Kargi Kizilirmak headrace tunnel in Turkey [10], the Water Transmission Tunnel in Iran [11], and the Zagros water-transfer tunnel through the Zagros Mountains in Iran [12]. Therefore, in engineering, studying the mechanical response of TBM crossing the fault fracture zone is important for the improvement of the performance of TBM and efficiency of tunneling.
Evidently, having proper knowledge of the determination of the rock mass geomechanical characteristics in the fault fracture zone is essential. Hao and Azzam [13] conducted a parametric study with the use of numerical method (UDEC) and identified three fault parameters (fault dips, fault shear strength, and fault locations relative to the underground structure) that are critical to the stability of the underground structure. Kahrman and Alber [14] investigated a fault breccia with clasts weaker than the matrix. They found that its strength was related to the proportion of the blocks and the size of the specimen. Zhao et al. [15] conducted a 3D simulation of TBM excavation in brittle rock associated with fault zones and found that the condition of the local rock mass and the complex interaction between the rock mass, TBM components, and tunnel support play a role in the characterization of instability phenomenon. However, the mechanical behavior of the rock mass in a fault fracture zone is difficult to characterize and is still poorly understood.

Numerous scholars have investigated the interaction between the surrounding rock and tunnel structure in a fault fracture zone. Shrestha and Panthi [16] studied the effect of groundwater on the fault behavior of the Modi Khola tunnel in Nepal. Abdollahi et al. (2019) determined suitable reinforcement strategies for passing TBM through a fault zone using precision numerical methods and accurate simulation of the TBM motion. Zhang et al. [17], Kun and Onargan [18], Baziar et al. [19], and Gao et al. [20] investigated the influence of fault on the stability of the surrounding rock of a tunnel.

When the TBM tunnel passes through the fault fracture zone, the stratum characteristics exhibit a more prominent effect on the mechanical response of the TBM tunnel construction. The strength and deformation characteristics of the broken rock mass are mainly controlled by joint cracks, and its mechanical properties are largely dependent on the mechanical characteristics, geometric characteristics, and distribution laws of joint cracks. Thus, selecting a proper constitutive model for numerical simulation is extremely important. The ubiquitous joint model can simultaneously consider the role of the two media types in joint rock mass and truly simulate the strength and deformation characteristics of the rock mass. Moreover, this model has been widely studied [21]. Wang and Huang [22] proposed a constitutive model for the deformation of a rock mass containing sets of ubiquitous joints, which can present the joint-induced anisotropy in the strength and deformation of rock mass. Ismael and Konietzky [23] proposed a ubiquitous joint constitutive model that links two failure criteria, the sliding and non-sliding failure modes, for inherent anisotropic rocks. Das et al. [24] evaluated the stability of underground workings for the exploitation of an inclined coal seam by the ubiquitous joint model.

In this paper, a 3D numerical model of the double-shield TBM passing through the Babai fault fracture zone was established based on the Bheri Babai Diversion Multipurpose Project (BBDMP) in Nepal. The ubiquitous joint model of the ABAQUS software was utilized to simulate the stratum in the fault fracture zone. Moreover, the influence of joints on the double-shield TBM tunneling in the fault fracture zone was analyzed. Finally, the variation laws of the surrounding rock displacement, plastic zone distribution, and segment internal force of the TBM crossing the fault fracture zone were obtained.

2. Project Overview

The BBDMP is the first interbasin water-transfer project in Nepal. The BBDMP primarily comprises of a tunnel with a total length of 12.2 km that transfers the water from the Bheri River, which is rich in water resources, to the Babai River—a local river with intermittent drought and flood. The purpose of this project is to irrigate 51,000 ha of farmlands in Bardiya and Banke in central and eastern Nepal. The cross section of the BBDMP is round, with a diameter of 5.06 m and a gradient of 3%. Robbins double-shield TBM is selected for the excavation. The tunnel of the project passes through the Siwalik rock stratum. There are two major fault zones, Babai and Bheri, both of which are thrust faults.

The Babai fault, which developed along the bedding, occurs at N35°–45°W, NE∠40°–45°. It has a width of about 100 m and is composed of fractured rock, fault breccia, mylonite, fault gouge, etc. The Babai fault is roughly distributed along the north side of the Babai River. The Bheri fault, which developed along the bedding, occurs at N40°–50°W, NE∠40°–45°. It has a width of about 400 m and
is composed of fractured rock, fault breccia, mylonite, fault gouge, etc. The Bheri fault is roughly distributed along the Bharleni Khola River, and the main joints in the construction area occur at N0°–10°W, SW ≤70°–80°, EW, S(N) ≤80°–90°.

3. Numerical Simulation

3.1. Overview of Ubiquitous Joint Model
The Abaqus software provides a model that is suitable for materials rich in parallel joint planes. The model may be used for the simulation of regular fracture joint planes appearing in groups or rock mass with a large number of faults. The spacing between joints is sufficiently smaller than the structural size of the model. Therefore, the whole model can be combined with a continuous sliding system. The main characteristics of this model include the following: 1) elastic–plastic deformation is considered; 2) three relationships exist between the joint layers (frictional sliding, closed, and separated), and once the joint layers are separated, the materials immediately become orthogonal anisotropic bodies; 3) the damage caused by volumetric deformation based on the Drucker–Prager or Mohr–Coulomb model may be considered; and 4) the model provides a reasonable stress cycle, including joint opening/closing and shearing force cycle.

3.2. Mechanical Behavior of Joint Materials
The direction of a joint plane is mainly determined based on its normal direction by setting the normal vector of the joint plane \( \mathbf{n} \) to \( \mathbf{n}_a = \begin{bmatrix} l \\ m \\ n \end{bmatrix} \), where \( l \), \( m \), and \( n \) denote the cosine in three directions of the normal vector. Two unit vectors on the joint plane \( \mathbf{t}_{\alpha \alpha} (\alpha = 1, 2) \) together with \( \mathbf{n}_a \) form mutually orthogonal vectors as the local coordinate system describing the orientation of the joint plane, as presented in Figure 2.
Figure 2. Joint element and joint direction.

The compressive stress perpendicular to the joint plane and the two shear stresses in the joint plane are calculated using the following equations:

\[
p_a = n_a^T \cdot \sigma \cdot n_a = \begin{bmatrix} l & m & n \end{bmatrix} \begin{bmatrix} \sigma_x & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & \sigma_y & \tau_{yz} \\ \tau_{zx} & \tau_{zy} & \sigma_z \end{bmatrix} \begin{bmatrix} l \\ m \\ n \end{bmatrix}
\]

(1)

\[
\tau_{aa} = n_a^T \cdot \sigma \cdot l_{aa} = \begin{bmatrix} e & m & n \end{bmatrix} \begin{bmatrix} \sigma_x & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & \sigma_y & \tau_{yz} \\ \tau_{zx} & \tau_{zy} & \sigma_z \end{bmatrix} \begin{bmatrix} l_{aa} \\ m_{aa} \\ n_{aa} \end{bmatrix}
\]

(2)

The shear stress is defined as:

\[
\tau_a = \sqrt{\tau_{aa} \tau_{aa}}
\]

(3)

The strain perpendicular to the joint plane is defined as local strain:

\[
\varepsilon_{an} = n_a \cdot \varepsilon \cdot n_a
\]

(4)

The strain at an angle \( \alpha \) to the joint plane is defined as engineering shear strain:

\[
\gamma_{aa} = n_a \cdot \varepsilon \cdot l_{aa} + l_{aa} \cdot \varepsilon \cdot n_a
\]

(5)

Assuming that the total strain can be decomposed into the sum of elastic strain and plastic strain:

\[
d\varepsilon = d\varepsilon^{el} + d\varepsilon^{pl}
\]

(6)

The plastic strain \( d\varepsilon^{pl} \) consists of two parts:

\[
d\varepsilon^{pl} = \sum_i d\varepsilon_i^{pl}
\]

(7)

Where \( i = b \) denotes volume deformation, and \( i = a \) denotes joint deformation. When joints are closed, the materials become isotropic linear elastic bodies. Once joints are identified to be tensioned in the normal direction, i.e., \( p_a \leq 0 \), they are opened, and the materials become orthogonal anisotropic bodies. As long as Equation (8) is true, joints are considered to remain opened.

\[
\varepsilon^{el}_{an(ps)} \leq \varepsilon^{el}_{an}
\]

(8)
where \( \varepsilon_{\text{el}(ps)} \) denotes the elastic strain perpendicular to the joint plane calculated using the plane stress method as follows:

\[
\varepsilon_{\text{el}(ps)} = \frac{H}{E} \left( \sigma_{a1} + \sigma_{a2} \right)
\]

where \( E \) denotes elastic modulus and \( \mu \) Poisson’s ratio. \( \sigma_{a1} \) and \( \sigma_{a2} \) denote stresses in the plane and are calculated using the following equation:

\[
\sigma_{aa} = f_{ac} \cdot \sigma \cdot t_{ac}
\]

In Equation (9): \( \alpha = 1, 2 \).

The sliding failure criterion for the joint \( a \) is calculated as follows:

\[
f_a = \tau_a - p_a \tan \beta_a - d_a = 0
\]

where \( \beta_a \) denotes the friction angle, and \( d_a \) denotes the cohesion. When \( f_a < 0 \), joints will not slide. When \( f_a = 0 \), joints will slide. The plastic strain on the joint plane consists of two parts, which are calculated using Equations (11) and (12):

\[
d \gamma_{aa} = d \varepsilon_{a} \cdot \frac{\tau_{aa}}{\tau_a} \cos \psi_a
\]

\[
d \gamma_{an} = d \varepsilon_{a} \cdot \sin \psi_a
\]

Equation (13) is obtained as follows:

\[
d \varepsilon_{a} = d \varepsilon_{a} \left[ \sin \psi_a n_a n_a + \frac{\tau_{aa}}{\tau_a} \cos \psi_a \left( n_a + t_{ac} n_a \right) \right]
\]

In Equation (13), \( d \varepsilon_{a} \) denotes the plastic strain ratio, and \( \psi_a \) denotes the dilation angle of joints. When \( \psi_a = 0 \), the joints will be subject to shear flow only; when \( \psi_a > 0 \), the joints will be subject to dilation while sliding. If different joint planes intersect at the same point, the failure criterion on each joint plane is independent.

**Figure 3.** Joint system model.
**Figure 4.** Dilatancy material model.

In addition to the closing and opening behaviors of the joints, the D-P failure criterion shall also be considered in calculating the joint plane.
In Equation (14), $\beta_b$ denotes the friction angle of volumetric strain, and $d_b$ denotes the cohesion of volumetric strain.

Once the volumetric strain meets the above criteria, volumetric plastic strain will occur:

$$\mathbf{d}_{\varepsilon_b}^{pl} = \mathbf{d}_{\varepsilon_b}^{pl} \cdot \left(1 - \frac{1}{3} \tan \psi_b\right)^{-1} \cdot \frac{\partial g_b}{\partial \sigma}$$

(15)

In Equation (15), plastic potential is calculated as follows:

$$g_b = q - p \tan \psi_b,$$

where $\psi_b$ denotes the dilation angle of volumetric strain. The properties of the joints are not changed by the above criteria.

### 3.3. Modeling and Parameter Selection

The conditions of the surrounding rock are extremely poor in the fault fracture zone. Thus, the single-shield tunneling mode shall be adopted for tunneling in this stratum. The geometric parameters of the front shield, rear shield, gripper shield, and segment are presented in Table 1. The surrounding rock includes three joint groups, i.e., $N0^\circ-10^\circ W$, $SW \angle 70^\circ-80^\circ$ and $EW$, $S(N) \angle 80^\circ-90^\circ$, mainly developed in the construction area of the BBDMP. The tunnel is located at $N26.84^\circ E$ lengthways.

| Table 1. The main parameters of the double-shield TBM used in the BBDMP. |
|---------------------------------|------------------------|
| **Parts**                      | **Parameters**         |
| Segment                        |                        |
| Inner diameter                 | 4200 mm                |
| Thickness                      | 300 mm                 |
| Outer diameter                 | 4800 mm                |
| Elastic modulus                | 30 GPa                 |
| Excavation diameter            | 5060 mm                |
| Maximum thrust                 | 20826 kN               |
| Cutter head                    |                        |
| Maximum over-excavation amount | 80 mm                  |
| Maximum excavation diameter    | 5140 mm                |
| Number of disc cutters         | 33                     |
| Diameter of front shield       | 4990 mm                |
| Thickness of front shield      | 40 mm                  |
| Diameter of outer telescopic shield | 4990 mm          |
| Thickness of outer telescopic shield | 30 mm         |
| Diameter of inner telescopic shield | 4910 mm            |
| Thickness of inner telescopic shield | 40 mm            |
| Shield                         |                        |
| Diameter of gripper shield     | 4910 mm                |
| Thickness of gripper shield    | 40 mm                  |
| Diameter of tail shield        | 4910 mm                |
| Thickness of tail shield       | 24 mm                  |
| Length of front shield (including outer telescopic shield) | 5000 mm |
| Length of gripper shield       | 3320 mm                |
| Length of tail shield          | 3745 mm                |
The surrounding rock of the Bheri fault zone is classified as Class V. Table 2 presents the mechanical parameters of the surrounding rock and joints. With regard to the tunneling parameters, the thrust is set to 3000 kN and the torque to 389 kN·m. The single-shield tunneling mode is adopted. The weight of the TBM component is presented in Table 3. The cutter head and shield are based on their actual weights. The backup structure is complex and has little influence on the mechanical response during tunneling. Thus, its mass is reflected by equivalent segment increase.

The ideal elastoplastic constitutive model is adopted for soil, and the Mohr–Coulomb criterion is applied as the yield criterion. The pea gravels and segments are considered as elastic bodies. Due to the time effect of filling, the elastic modulus of the pea gravel is considered as two stages of soft and hard. Assuming that TBM will not shift or rotate during tunneling, the horizontal degree of freedom and three rotational degrees of freedom of TBM are constrained. The numerical model is presented in Figure 6, and the schematic of measures for supporting and hydraulic filling of pea gravels is illustrated in Figure 7.

Table 2. Physical and mechanical parameters of TBM component.

| Material                        | Unit Weight (kg/m³) | Elastic Modulus (GPa) | Poisson’s Ratio | Cohesion (MPa) | Friction Angle (°) |
|---------------------------------|---------------------|-----------------------|----------------|----------------|--------------------|
| Broken surrounding rock         | 2150                | 0.2                   | 0.32           | 0.15           | 23                 |
| Joint                           | —                   | —                     | —              | —              | —                  |
| Sandstone                       | 2550                | 2.0                   | 0.32           | 0.6            | 20                 |
| Mudstone                        | 2350                | 0.4                   | 0.32           | 0.15           | 23                 |
| Segment                         | 3000                | 30                    | —              | —              | —                  |
| Pea gravel (including mortar)   | 2200                | 0.5 (soft)            | —              | —              | —                  |
|                                  |                     | 1.0 (hard)            | —              | —              | —                  |

Table 3. Weight of the TBM component (unit: kg).

| Position | Cutter head | Shield | Backup Structure |
|----------|-------------|--------|------------------|
| Name     | Cutter head | Front shield | Rear shield | Gripper shield | Segment erector | Belt conveyor | Rig | Crane span structure |
| Weight   | 46000       | 96700    | 27000           | 90900          | 17000           | 17700        | 3000 | 21000 |
3.4. Contact Settings

(1) Contact between the shield and surrounding rock
Setting of the contact relation between the shield and surrounding rock is extremely important for this model. At the start of the calculation, the shield and surrounding rock are in a non-contact state and have uneven gaps, which greatly influences the convergence of the numerical solution. During tunneling, the shield and surrounding rock are in a sliding friction state, and the tangential behavior of the contact surface is considered as dynamic friction. In this model, the exponential decay method is employed to describe the relationship between static friction coefficient and dynamic friction coefficient and expressed by Equation (16) [25]. The static friction coefficient \( \mu_s \) is 0.3; the dynamic friction coefficient \( \mu_k \), 0.24; and the decay coefficient \( d_c \), 0.8. \( \gamma_{eq} \) denotes the tangential sliding rate. Because a large normal contact pressure will be exerted on the contact surface between the shield and surrounding rock, the normal behavior is set as Hard Contact.

\[
\mu = \mu_s + (\mu_k - \mu_s) e^{-d_c \gamma_{eq}}
\]

(16)

(3) Contact between the pea gravel and surrounding rock
In the calculation, the pea gravel and surrounding rock are estimated to be extremely close to each other, with a good force transmission state. Thus, Tie Contact is set, that is, the degrees of freedom of the corresponding nodes between the pea gravel and surrounding rock are coupled for deformation together.

Figure 6. Numerical model.

Figure 7. Schematic diagram of the measures for supporting and hydraulic filling of pea gravels.
(4) Contact between the pea gravel and segment
The inner side of the pea gravel is in contact with the outer side of the segment. The pea gravels are filled as force transmission media for the transmission of the load released by the excavated surrounding rock to the segment, as well as for load sharing. In the numerical model, the Tie Contact is also set between the inner side of the pea gravel and the segment for deformation together.

4. Result Analysis

4.1. Displacement Analysis
Figures 8 and 9 present the nephograms for the vertical displacement and horizontal displacement of the surrounding rock when the fault fracture zone is penetrated.

![Figure 8. Nephogram for vertical displacement following penetration.](image1)

![Figure 9. Nephogram for horizontal displacement following penetration.](image2)

As can be seen from the figures, the displacement distribution in the vertical direction is symmetrical on the whole. After penetration by TBM, the vault settlement is 9.429 cm, and the bottom uplift is 7.288 cm. The horizontal convergence displacements at all nodes exhibit convergence toward the inside of the tunnel. The maximum value appears around the left arch shoulder and the right arch foot, which have horizontal convergence displacements of 5.518 and 5.347 cm, respectively.

Table 4 presents both the vault settlement and inverted arch uplift values after tunneling for the total length of 60 m. Compared with the result of tunneling in intact rock mass, the trend of vertical displacement distribution is unchanged, and the specific values increase to a certain extent for tunneling in the fault fracture zone.

| Stratum                               | Vault Settlement Value | Inverted Arch Uplift Value | Horizontal Convergence Displacement |
|---------------------------------------|------------------------|-----------------------------|-------------------------------------|
| Mudstone (single-shield tunneling mode)| 7.312                  | 6.253                       | 1.942                               |
| Sandstone (double-shield tunneling mode)| 4.769                  | 4.012                       | 0.948                               |
| Fault fracture zone (single-shield tunneling mode)| 9.429                  | 7.288                       | 5.347                               |

4.2. Plastic Zone Distribution and Joint Yield of the Surrounding Rock
Since the mechanical properties of the joint plane also follow the D-P criterion, the rock mass will generally be subject to shear sliding along the structural plane during tunneling, which results in a certain yield on certain joint positions. From Figure 10, it can be seen that the contour line of the node yield point is about 0.3 times the diameter of the tunnel. In addition to the joints, the rock mass will also be yielded to a certain extent. The nephogram for the plastic zone distribution of the surrounding...
rock following TBM tunneling is presented in Figure 11. The plastic zone is mainly distributed along the geometric plane of two joints, as well as in the area from the left arch waist to the left arch shoulder and from the right arch waist to the right foot of the wall. The maximum plastic strain is 0.1667 and is located between the left arch waist and the vault, which is close to the arch shoulder. Table 5 presents the maximum plastic strain of the surrounding rock after tunneling for a total length of 60 m. The plastic zone due to tunneling in the fault fracture zone is obviously larger than that due to tunneling in intact rock mass.

![Figure 10. Contour line of the joint yield points.](image1)

![Figure 11. Plastic strain distribution of the surrounding rock.](image2)

Table 5. Plastic strain caused by TBM tunneling in different strata.

| Stratum                                      | Maximum Plastic Strain |
|----------------------------------------------|------------------------|
| Mudstone (single-shield tunneling mode)      | 0.09773                |
| Sandstone (double-shield tunneling mode)     | 0.06956                |
| Fault fracture zone (single-shield tunneling mode) | 0.16670                |

4.3. Internal Force Analysis of Segment

The internal force and the safety coefficient of the segments during TBM tunneling to 20 m, 30 m, and total length in fault fracture zone are presented in Table 6.

Table 6. Internal force and safety coefficient during TBM tunneling.

| Position            | TBM Tunneling to 20 m | TBM Tunneling to 30 m | TBM Tunneling to Total Length |
|---------------------|-----------------------|-----------------------|-----------------------------|
|                     | Axial Force (kN)      | Bending Moment (kN·m) | Safety Coefficient           |
|                     | Axial Force (kN)      | Bending Moment (kN·m) | Safety Coefficient           |
|                     | Axial Force (kN)      | Bending Moment (kN·m) | Safety Coefficient           |
| Vault               | 1268                  | 72.18                 | 2.76                         |
| Left arch shoulder  | 1477                  | 89.75                 | 2.31                         |
| Left arch waist     | 1329                  | 79.60                 | 2.58                         |
| Left wall foot      | 962                   | 60.59                 | 3.51                         |
| Inverted arch       | 1125                  | 84.11                 | 2.81                         |
| Right wall foot     | 1628                  | 91.28                 | 2.15                         |
| Right arch waist    | 1147                  | 76.75                 | 2.87                         |
Table 6 demonstrates that:
(1) The dissymmetry of the mechanical properties of joint rock mass leads to the dissymmetry of the internal force distribution of the segment.
(2) Due to the inherent properties of joint rock mass, “potential” shear sliding surface appears on the structural plane following excavation, and the internal force distribution law of the segment changes. With regard to the axial force and bending moment, the maximum values are roughly located at the left arch shoulder and the right foot of the wall. The axial force and bending moment also exhibit an increasing trend during the whole process of excavation.
(3) The minimum safety coefficient is located at the left arch shoulder and the right foot of the wall. After the tunnel penetration, the safety coefficient of the left arch waist is decreased, with the minimum value up to 2.02.

Compared with the calculation results of tunneling in intact rock mass, the internal force of the segment in joint rock mass is much larger, but the safety coefficient is much smaller. The maximum internal force and the minimum safety coefficient and their positions are compared and analyzed, as presented in Table 7.

Table 7. Extreme values and positions of the internal force and safety coefficient.

| Working Condition | Maximum Axial Force (Absolute Value) (kN) | Maximum Bending Moment (Absolute Value) (kN·m) | Minimum Safety Coefficient | Position          |
|-------------------|------------------------------------------|-----------------------------------------------|-----------------------------|-------------------|
| Sandstone         | 1259                                     | 65.15                                        | 2.43                        | Arch waist        |
| Mudstone          | 1327                                     | 79.13                                        | 2.35                        | Arch waist        |
| Fault fracture zone | 2251                                     | 104.74                                       | 2.02                        | Right wall foot   |

Table 7 demonstrates that the internal force of the segment during tunneling in the fault fracture zone significantly increases while the safety coefficient decreases. For the specific maximum value alone, the maximum axial force during tunneling in the fault fracture zone increases by 78.8% compared with that in the sandstone stratum and 69.6% compared with that in the mudstone stratum. The maximum bending moment increases by 60.8% and 32.4%, respectively, but the safety coefficient decreases by 20.3% and 14.4%, respectively.

5. Conclusion
This paper takes the well-developed joints in the BBDMP as the study objects to create a dynamic TBM construction model. This model is expected to be used for studying the construction mechanical response of TBM tunneling in the fault fracture zone of the Siwalik rock stratum of the Himalayas. The main conclusions are as follows:
(1) When the single-shield tunneling mode is used in the fault fracture zone, the displacement of the surrounding rock is larger than that in intact rock mass. The displacement distribution is subject to spatial asymmetry mainly because of the asymmetry of the geometric parameters of joints.
(2) In the fault fracture zone, the plastic zone of the surrounding rock is mainly distributed along the geometric plane of two joints. The maximum value appears in the area from the left arch waist to the left arch shoulder and from the right arch waist to the right foot of the wall. The maximum plastic strain reaches 0.1667. Some joints also reach the yield state, with a range of about 0.3 times the diameter of the tunnel. The joints at all the other sites reach the yield state, except for the area around the vault and inverted arch.
(3) The existence of joints in the fault fracture zone leads to the asymmetrical distribution of the internal force in the segment. The minimum safety coefficient appears at the left arch shoulder and the right foot of the wall.
This paper has achieved certain research results. However, some deficiencies still exist, and the existing problems are worth further discussion. For example, (1) for the contact between the segments and the pea gravel, the Tie Contact is adopted in this paper. Due to the lack of unified conclusion on the stress condition and force transmission mechanism of the pea gravel layer, the normal and tangential mechanical behaviors on the contact interface need to be studied further. (2) The ubiquitous joint model in this paper assumes that the rock mass is full of joints. However, there is no quantitative description of the gap between the structural planes, so it is impossible to discuss the impact of rock mass fragmentation on TBM excavation.

Acknowledgments
This research was supported by the Sichuan Science and Technology Program (No. 2019YFG0001), the General Program of the National Natural Science Foundation of China (U1934213), the General Program of the National Natural Science Foundation of China (51878572 and 51678503).

References
[1] Dammnyr O, Nilsen B and Gollegger J 2017 Feasibility of tunnel boring through weakness zones in deep Norwegian subsea tunnels Tunn. Undergr. Space Technol. 69 133-146
[2] Paltrinieri E, Sandrone F and Zhao J 2016 Analysis and estimation of gripper TBM performances in highly fractured and faulted rocks Tunn. Undergr. Space Technol. 52 44-61
[3] Gattinoni P, Pizzarotti EM and Scesi L 2016 Geomechanical characterisation of fault rocks in tunnelling: The Brenner Base Tunnel (Northern Italy) Tunn. Undergr. Space Technol. 51 250-257
[4] Huang Z, Zeng W and Zhao K 2019 Experimental investigation of the variations in hydraulic properties of a fault zone in Western Shandong, China J. Hydrol. 574 822-835
[5] Kang Y, Liu Q, Xi H and Gong G 2018 Improved compound support system for coal mine tunnels in densely faulted zones: A case study of China's Huainain coal field Eng Geol 240 10-20
[6] Manouchehrian A and Cai M 2018 Numerical modeling of rockburst near fault zones in deep tunnels Tunn. Undergr. Space Technol. 80 164-180
[7] Zhang N, Shen JS, Zhou A and Arulrajah A 2018 Tunneling induced geohazards in mylonitic rock faults with rich groundwater: A case study in Guangzhou Tunn. Undergr. Space Technol. 74 262-272
[8] Paltrinieri E, Sandrone F, Dudt JP and Zhao J 2016 Probabilistic simulations of TBM tunnelling in highly fractured and faulted rocks Tunn. Undergr. Space Technol. 57 183-194
[9] Dalgûc S 2003 Tunneling in fault zones, Tuzla tunnel, Turkey Tunn. Undergr. Space Technol. 18 453-465
[10] Home L 2016 Hard rock TBM tunneling in challenging ground: Developments and lessons learned from the field Tunn. Undergr. Space Technol. 57 27-32
[11] Abdollahi MS, Najafi M, Bafghi AY and Marji MF 2019 A 3D numerical model to determine suitable reinforcement strategies for passing TBM through a fault zone, a case study: Safaroud water transmission tunnel, Iran Tunn. Undergr. Space Technol. 88 186-199
[12] Bayati M and Khademi Hamidi J 2017 A case study on TBM tunnelling in fault zones and lessons learned from ground improvement Tunn. Undergr. Space Technol. 63 162-170
[13] Hao YH and Azzam R 2005 The plastic zones and displacements around underground openings in rock masses containing a fault Tunn. Undergr. Space Technol. 20 49-61
[14] Kahrman S and Alber M 2008 Triaxial strength of a fault breccia of weak rocks in a strong matrix Bull. Eng. Geol. Environ. 67 435-441
[15] Zhao K, Janutolo M, Barla G and Chen G 2014 3D simulation of TBM excavation in brittle rock associated with fault zones: The Brenner Exploratory Tunnel case Eng Geol 181 93-111
[16] Shrestha PK and Panthi KK 2014 Groundwater effect on faulted rock mass: An evaluation of Modi Khola Pressure Tunnel in the Nepal Himalaya Rock Mech Rock Eng 47 1021-1035
[17] Zhang Z, Chen F, Li N, Swoboda G and Liu N 2017 Influence of fault on the surrounding rock stability of a tunnel: Location and thickness Tunn. Undergr. Space Technol. 61 1-11
[18] Kun M and Onargan T 2013 Influence of the fault zone in shallow tunneling: A case study of Izmir Metro Tunnel Tunn. Undergr. Space Technol. 33 34-45
[19] Baziar MH, Nabizadeh A, Mehrabi R, Lee CJ and Hung WY 2016 Evaluation of underground tunnel response to reverse fault rupture using numerical approach Soil Dyn. Earthq. Eng. 83 1-17
[20] Gao CL, Zhou ZQ, Yang WM, Lin CJ, Li LP and Wang J 2019 Model test and numerical simulation research of water leakage in operating tunnels passing through intersecting faults Tunn. Undergr. Space Technol. 94 103134
[21] Sainsbury BL and Sainsbury DP 2017 Practical use of the ubiquitous-joint constitutive model for the simulation of anisotropic rock masses Rock Mech Rock Eng 50 1507-1528
[22] Wang TT and Huang TH 2009 A constitutive model for the deformation of a rock mass containing sets of ubiquitous joints Int. J. Rock Mech. Min. Sci. 46 521-530
[23] Ismael M and Konietzky H 2019 Constitutive model for inherent anisotropic rocks: Ubiquitous joint model based on the Hoek-Brown failure criterion Comput Geotech 105 99-109
[24] Das AJ, Mandal PK, Bhattacharjee R, Tiwari S, Kushwaha A and Roy LB 2017 Evaluation of stability of underground workings for exploitation of an inclined coal seam by the ubiquitous joint model Int. J. Rock Mech. Min. Sci. 93 101-114
[25] Oden JT, Martins JAC 1985 Models and computational methods for dynamic friction phenomena Comput Methods Appl Mech Eng 52 527-634