# Quantitative assessment of the effect of in-situ stresses on blast-induced damage to rock 

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## ARTICLE INFO

## Keywords:

Blasting
In-situ stress
Damage evolution
Image processing
Quantitative assessment


#### Abstract

In-situ stress significantly affects rock blast damage but there is a paucity of quantitative assessments of damage evolution in rocks affected by confining pressure. The present paper analyses the effect of envelope pressure on blast-induced rock damage through theoretical analysis and numerical simulations. Damage clouds obtained from numerical simulations are processed using image processing techniques. The concept of the damage variable $(\eta)$ is proposed to facilitate the presentation of the image processing results. The damage variable is found to be negatively correlated with the hydrostatic pressure ( $P_{\mathrm{x}}$ ) at the same moment, in equiaxial in-situ stress fields. In contrast, in anisotropic in-situ stress fields, $\eta$ is not negatively correlated with $P_{\mathrm{x}}$ due to the presence of hoop tensile stresses in the rock. The mathematical relationship between $\eta$ and $P_{\mathrm{x}}$ in equiaxial and anisotropic stress fields are established. An anisotropic damage variable $\left(\eta_{\mathrm{k}}\right)$ is introduced to describe the effect of the anisotropy ratio $(K)$ on rock damage, which is found to increase with increasing values of $K$. The sharp increase in $K$ equal to 4 and 5 is explained in terms of the state of the rock stress distribution under static loading. This study provides insights into the effect of in situ stress on rock blast damage and presents new approaches for analyzing and presenting the data.


## 1. Introduction

With the depletion of shallow mineral resources, the mining industry is turning to deep mining to extract all kinds of minerals. Many mines around the world now exceed depths of 2000 m [1]. The TauTona gold mine in South Africa is mined to a depth of $3,900 \mathrm{~m}$ with an in-situ stress level of about 100 MPa [2]. Other deep mines such as the Kloof gold mine, East Rand Proprietary, and Driefontein gold mine have depths exceeding $3,500 \mathrm{~m}$ [3]. In response to the increasing demand for natural resources, the mining depth is increasing yearly to obtain more minerals. Take China's coal mines, for example, increasing by $10 \sim 25 \mathrm{~m}$ annually [4]. The drill and blast (D \& B) technique is a widely used method for mining operations, but there is unique mechanical behavior of rocks under high in situ stress compared with shallow rocks [5,6]. High in-situ stress is becoming one of the main challenges for deep mining blasting excavation [7].

Laboratory experiments are crucial for studying the influence of insitu stress on the dynamic response of the rock during blasting.

Nicholls and Duvall [8] observed that the pre-cracked rocks under static stress fields are most likely to fracture in the direction of the maximum compressive principal stress. Yang et al. [9] found through photoelastic experiments that the in-situ stress perpendicular to the crack would reduce the stress intensity factor at the crack tip, thus hindering crack growth. Zhang et al. [10] investigated the impact of confining pressure on the shape and volume of the crushed zone using model experiments. This effect demonstrates that the crushed zone initially increases and then decreases with increasing confining pressure. He et al. [11] used Digital Image Correlation (DIC) and strain measurement techniques to analyze the dynamic response of rock under active confining pressure and blasting dynamic load. The results revealed that the crack propagation direction is controlled by both, circumferential tensile stress and biaxial preloading ratio. Yue et al. [12] used a digital laser dynamic caustic experiment system to carry out experiments, indicating that the pressure in the vertical direction hinders the crack growth, while pressure parallel to the crack direction promotes it. Also, state that when the principal pressure is at a certain angle to the slit, the main crack

[^0]propagates in the pressure direction. Wang [13] conducted blasting experiments with similar materials and showed that an increase in the maximum principal stress causes the blasting crack to deflect towards the direction of the maximum principal stress, but slows down the growth rate of the blasting crack.

Numerical simulation to study the influence of in-situ stress on rock blasting has become increasingly popular with the development of computer technology. Among the many numerical simulation software, ANSYS/LS-DYNA performs well-suited simulations for the nonlinear dynamic response of structures [14] and has been widely used to study the dynamic response of rock blasting under in-situ stress. Ma and An [15] used the Johnson-Holmquist material model to simulate rock material behavior during a blast and concluded that the blast cracks are aligned in the direction of the prestress axis. This phenomenon becomes more pronounced with increasing prestress. Lu et al. [16] used a coupled SPH-FEM model and found that the radius of the crushed zone increases with increasing in-situ stress. Xie et al. [7] reported that in-situ stresses hinder tensile stresses but enhance compressive stresses in the blasthole direction. Xie et al. [17] studied multi-hole blasting to cut rock under insitu stress conditions and optimized the placement parameters for deep rock cutting and blasting. Yi et al. [18] showed that the early expansion of blast cracks is determined by the dynamic blast load, and the later expansion of cracks is mainly influenced by in-situ stresses as the blast shock wave decays. Tao et al. [19] simulated the dynamic response of rocks under coupled static-dynamic loading and concluded that the increase in stress anisotropy concentrates cracks near the maximum principal stress. Wang et al. [20] investigated the effect of in-situ stress on cyclic blasting damage extension and found that the contribution of cyclic blasting load to rock fragmentation was mainly concentrated in the maximum principal stress direction, while the contribution to rock fragmentation in the direction of the minimum principal stress was not significant.

The study of the effect of in situ stress on the dynamic response of rock blasting drew similar conclusions to the aforementioned study. Specifically, blast crack expansion was enhanced in the direction of maximum principal stress, and the crack expansion was suppressed in other directions. However, most existing literature only reports the relationship between in-situ stress and blast cracking qualitatively, with few quantitative analyses of crack extension under the influence of insitu stress. In recent years, some scholars have started to work on the quantitative description of blast cracking. Guo et al. [21] and Zhai et al. [22] quantified experimentally obtained blast cracks using fractal dimensions, while Yue et al. [23] and Zhang et al. [24] proposed the concept of fracture degree based on CDEM simulation software. The magnitude of fracture degree was used to quantitatively assess the damage degree of concrete. Tao et al. [19] used the ImageJ software to process the resultant images derived from ANSYS/LS-DYNA and reached some quantitative useful conclusions. However, there are some disadvantages with this image-processing method. Specifically, ImageJ is weak in processing color-rich images, and it cannot process images in batches automatically. Rock fracture images obtained using the HJC model simulations are available in few colors, which allow the ImageJ software to be used to its full potential. For images of damage obtained using the RHT model, the richness of the colors and the dispersion of the damaged areas make it difficult to capture the damaged areas using ImageJ software. Additionally, the existing studies have only processed images of the final state of blasting, and there is a lack of quantitative analysis of a large number of images throughout the whole rock blasting process. This lack of analysis limits the in-depth understanding of the evolution of the dynamic response of rocks under blast loads. Furthermore, most of the studies at this stage focus on the fracture phenomena of rocks, and there is also a relative lack of research on the mathematical relationship between rock damage and in-situ stress under the action of dynamic blasting loads.

In this study, the stress distribution in rock mass under static loads was determined using theoretical analysis, which forms the theoretical
basis for analyzing the distribution law of rock damage under dynamic blasting load. ANSYS/LS-DYNA numerical simulation software was then used to replicate the dynamic response of rock blasting under static loads. Then, the numerical simulation images were processed using a proposed method for quantitative assessment of blast damage areas based on image processing techniques. Finally, some new useful quantitative conclusions on blast damage patterns under the influence of insitu stress magnitude and anisotropy were obtained through the analysis of image processing results.

## 2. Hoop stress distribution under static loads

### 2.1. Hoop stress distribution at the wall of the blasthole

In engineering structural analysis, when the geometry and forces of a structure have certain characteristics, the space problem can be simplified to a plane problem by appropriate simplification and mechanical abstraction. Since the length of explosive is much larger than its diameter, it can be simplified into a plane strain problem. Fig. 1 shows a single blasthole in an infinite medium subjected to static stress load, assuming an elastic dynamic problem under plane strain conditions. When the dynamic blast loads are not considered, the model can be treated as an infinite plate containing a hollow hole. The plate is loaded by biaxial stresses of magnitudes $P_{\mathrm{x}}$ and $P_{\mathrm{y}}$ in the horizontal and vertical directions, respectively, and the radius of the hole is $a$.

The hoop stresses induced around the blasthole under biaxial stress loading were analyzed, and the complete solution for the stress distribution was obtained [25]:

$$
\left\{\begin{array}{l}
\sigma_{r r}=\frac{1}{2}\left[\left(P_{y}+P_{x}\right)\left(1-\left(\frac{a}{r}\right)^{2}\right)+\left(P_{x}-P_{y}\right)\left(1-4\left(\frac{a}{r}\right)^{2}+3\left(\frac{a}{r}\right)^{4}\right) \cos 2 \theta\right]  \tag{1}\\
\sigma_{\theta \theta}=\frac{1}{2}\left[\left(P_{y}+P_{x}\right)\left(1+\left(\frac{a}{r}\right)^{2}\right)-\left(P_{x}-P_{y}\right)\left(1+3\left(\frac{a}{r}\right)^{4}\right) \cos 2 \theta\right] \\
\tau_{r \theta}=\frac{1}{2}\left(P_{y}-P_{x}\right)\left(1+2\left(\frac{a}{r}\right)^{2}-3\left(\frac{a}{r}\right)^{4}\right) \sin 2 \theta
\end{array}\right.
$$

where $\sigma_{r r}$ is radial stress, $\sigma_{\theta \theta}$ is thoop stress, $\tau_{r \theta}$ is shear stress, $r$ and $\theta$ are the distance and angle from the source of the blast, respectively.

According to Eq. (1), when $r=a$, the magnitudes of $\sigma_{r r}$ and $\tau_{r \theta}$ are zero, and the radial and shear stresses at the wall of the blasthole are free. The hoop stress distribution on the blasthole wall can represent the stress distribution state. Fig. 2 illustrates the distribution of hoop stresses


Fig. 1. Elasticity model under static load.


Fig. 2. Hoop stress distribution on the blasthole wall: (a) in equiaxial in-situ stress fields, and (b) in anisotropic in-situ stress fields.
induced by 10 different in-situ stress conditions on the blasthole wall. The biaxial stress loading conditions are presented in Table 1.

As shown in Fig. 2(a), the hoop stress on the blasthole wall exhibits symmetric compressive stresses under hydrostatic pressure, and the magnitude of hoop stresses increases with the increase of hydrostatic pressure. This indicates that the expansion of blast damage in all directions in the hydrostatic stress field will be hindered to the same extent, and the higher the in-situ stress, the stronger the hindrance. In the anisotropic stress field, the loading stress in the vertical direction $\left(P_{\mathrm{y}}\right)$ is kept constant, and as the loading stress in the horizontal direction $\left(P_{\mathrm{x}}\right)$ gradually increases, the circumferential stress distribution at the wall of the blasthole is no longer symmetric, as shown in Fig. 2(b). The hoop stress appears as compressive stress at $\theta=90^{\circ}$ and $\theta=270^{\circ}$, increases sharply with increasing of $P_{\mathrm{x}}$. At $\theta=0^{\circ}$ and $\theta=180^{\circ}$, the hoop compressive stress gradually decreases with the increase of the value of $P_{\mathrm{x}}$. When $P_{\mathrm{x}}$ reaches 80 MPa , the hoop stress starts to behave as tensile stress. It is not reasonable to assume that the extension of blast damage in the direction of the minimum principal stress will be hindered, while blast damage extension will be promoted in the direction of the maximum principal stress when the hoop stress is expressed as tensile stress.

### 2.2. Hoop stress distribution in rocks

Based on Eq. (1), the variation curve of the circumferential stress with the distance to the blasthole wall was plotted (Fig. 3). Fig. 3(a) shows the stress concentration near the blasthole under equiaxial in-situ stress loading. As the distance from the blasthole increases, the magnitude of the hoop stress gradually converges to the biaxially loaded stress value for each case. Fig. 3(b) represents that the magnitude of the hoop stress in the Y-axis direction gradually decreases with the distance from

Table 1
Biaxial stress loading conditions in this study.

| Case | Stress state | $P_{\mathrm{y}} / \mathrm{MPa}$ | $P_{\mathrm{x}} / \mathrm{MPa}$ | $K=P_{\mathrm{x}} / P_{\mathrm{y}}$ |
| :--- | :--- | :--- | :--- | :--- |
| 1 | Equiaxial | 0 | 0 | 1 |
| 2 |  | 20 | 20 | 1 |
| 3 |  | 40 | 40 | 1 |
| 4 |  | 60 | 60 | 1 |
| 5 |  | 80 | 80 | 1 |
| 6 | Anisotropic | 20 | 100 | 1 |
| 7 |  | 20 | 40 | 2 |
| 8 |  | 20 | 60 | 3 |
| 9 | 20 | 100 | 4 |  |
| 10 |  |  | 5 |  |

the blasthole. The magnitude of the hoop stress in the $x$-axis direction shows an opposite trend. In the far field, the magnitude of the hoop stress in the Y -axis direction gradually tends to the value of $P_{\mathrm{x}}$, the magnitude of the hoop stress in the X -axis direction tends to gradually reach the value of $P_{\mathrm{y}}$, and the magnitude of the hoop stress in the other directions is determined by the joint action of $P_{\mathrm{x}}$ and $P_{\mathrm{y}}$. In addition, the magnitude of the hoop compressive stress in the far field in the Y-axis direction is much larger than that in the X -axis direction. For this reason, it can be expected that the blast damage will be mainly distributed in the direction of maximum principal stress (the X-axis direction), and the development of damage in the direction of minimum principal stress (the Y-axis direction) will be more strongly suppressed.

## 3. Numerical simulation of rock damage under in-situ stress

This study establishes a numerical model with a side length of 5 m , and a blasthole with a diameter of 11 cm , and set up with non-reflecting boundaries around it (Fig. 4). A plane strain model is used for numerical investigation in the study, which implies the assumption of infinite velocity of detonation along the borehole and infinite charge length. Ten in-situ stress loading cases shown in Table 1 were numerically investigated to replicate the rock blasting process. To take into consideration the static loading effect of the in-situ stress, the rock is first pre-loaded to obtain an initial stress state. The rock in the initial stress state was then introduced into the display analysis session as the initial condition. During the analysis phase, the explosive was detonated, and the rock is subjected to coupling dynamic-static load. Solid materials are generally simulated using Lagrangian algorithms. However, the blasting process is considered a fluid-solid coupling problem, and therefore, fluid--structure interaction methods are often applied. In this numerical simulation, the arbitrary Lagrangian-Eulerian (ALE) algorithm was employed.

### 3.1. Constitutive model and parameters of rocks

Several constitutive models have been developed to describe the response of rocks under dynamic loads, including the Holomquis-Johnson-Cook (HJC) model [26], the Riedel-Hiermaier-Thoma (RHT) model [27], and the JH series of models [28]. The RHT model is a popular model for simulating the nonlinear response of rocks under dynamic loading [29-31]. The pressure of the RHT model is described by the Mie-Gruneisen form, which includes a polynomial Hugoniot curve and a p- $\alpha$ compaction relation [32]. The p- $\alpha$ compaction model is shown schematically in Fig. 5(a). The model is in an elastic state when the applied pressure is lower than the pore fragmentation pressure. As


Fig. 3. Hoop stress variation with distance from the blasthole: (a) in equiaxial in-situ stress fields, and (b) in anisotropic in-situ stress fields.


Fig. 4. Configuration of the numerical model.
the pores begin to collapse, the bulk stiffness and effective bulk modulus of the material decrease. The variable $\alpha$ controls the porosity of the material, decreasing as the pressure increases, which results in an irreversible loading process. When the pressure reaches the compaction pressure, the material is completely compacted, at which point $\alpha$ equals 1.

The strength of the RHT model is described by three limit surfaces: the elastic yield surface, the failure surface, and the residual strength surface, as shown in Fig. 5(b). The arrow in Fig. 5(b) depicts the typical loading process of the material, which is divided into three stages in total.

Stage 1: Elastic deformation stage. The rock is in the elastic deformation phase when the arrow does not reach the elastic yield surface. the elastic yield surface $Y_{\text {elastic }}$ of the material is determined by the failure surface $Y_{\text {fail }}$, given by Eq. (2).
$Y_{\text {elassic }}=Y_{\text {fail }} \cdot F_{\text {elassic }} \cdot F_{C A P(P)}$
where $Y_{\text {elastic }}$ is the ratio of elastic strength to failure surface strength and
$F_{C A P(P)}$ is the elastic yield surface cap function to limit the elastic bias stress under elastic hydrostatic pressure.

Stage 2: Plastic hardening stage. The material is in the plastic deformation phase when it is between the yield and failure surfaces. The behavior of the material in this phase is described by strain-hardening characteristics. The failure surface is defined as a function of the pressure $P$, the Lode angle $\theta$ and the strain rate $\dot{\varepsilon}$ :
$Y_{\text {fail }}=Y_{T X C(P)} \cdot R_{3(\theta)} \cdot F_{R A T E(\hat{\varepsilon})}$
where, $Y_{T X C}=f_{c}\left|A\left(P^{*}-P_{\text {spall }}^{*} F_{\text {RATE }}\right)^{N}\right|, f_{c}$ is the uniaxial compressive strength, $A$ is the failure surface constant, $N$ is the failure surface index, $P^{*}$ is the normalized pressure according to $f_{c}, P_{\text {spall }}^{*}$ is defined as $P^{*}\left(f_{t} / f_{c}\right)$, and $R_{3(\theta)}$ is the third invariant of the partial stress tensor.

Stage 3: Softening damage stage. Further, the material retains a certain residual strength after failure, and the residual strength surface is denoted as $Y_{\text {resid }}^{*}=B \cdot P^{* M} . B$ is the residual strength surface constant. $M$ is the residual strength surface index. The material behavior between the failure surface and the residual surface is described by the damage characteristics. Damage is accumulated by the equation:
$D=\sum \frac{\Delta \varepsilon_{P l}}{\varepsilon_{P}^{\text {filure }}}$
where, $\varepsilon_{P}^{\text {failure }}=D_{1}\left(P^{*}-P_{\text {spall }}\right)^{D_{2}} \geqslant \varepsilon_{f}^{\mathrm{min}}, D_{1}$ and $D_{2}$ are the damage constants. $\varepsilon_{f}^{\min }$ is the minimum failure strain. When $D=0$, there is no accumulated damage to the material. When $D=1$, the residual surface is reached, and the material is completely damaged.

The strain rate effect is an important characteristic of dynamic load that distinguishes it from the action of static load. The RHT model incorporates the strain rate effect, and the dependence of strain rate on strength can be expressed by the following equation:
$F_{r}\left(\dot{\varepsilon}_{p}\right)=\left\{\begin{array}{c}\left(\dot{\varepsilon}_{p} / \dot{\varepsilon}_{0}^{c}\right)^{\beta_{c}} \mathrm{P} \geqslant f_{c} / 3 \\ \frac{P+f_{t} / 3}{f_{c} / 3+f_{t} / 3}\left(\dot{\varepsilon}_{p} / \dot{\varepsilon}_{0}^{t}\right)^{\beta_{c}}-\frac{P-f_{c} / 3}{f_{c} / 3+f_{t} / 3}\left(\dot{\varepsilon}_{p} / \dot{\varepsilon}_{0}^{c}\right)^{\beta_{t}}-f_{t} / 3<\mathrm{P}<f_{c} / 3 \\ \left(\dot{\varepsilon}_{p} / \dot{\varepsilon}_{0}^{t}\right)^{\beta_{c}} \mathrm{P} \leqslant-f_{t} / 3\end{array}\right.$


Fig. 5. RHT model: (a) Schematical description of the p- $\alpha$ equation of state, and (b) Stress limit surfaces and loading scenario in the RHT model.
where $F_{r}\left(\dot{\varepsilon}_{p}\right)$ is the strain rate strength factor, $\dot{\varepsilon}_{p}$ is the strain rate, $\dot{\varepsilon}_{0}^{c}$ is the reference strain rate under compression, $\dot{\varepsilon}_{0}^{c}=3.0 \times 10^{-5} s^{-1}, \dot{\varepsilon}_{0}^{t}$ is the reference strain rate under tension, $\dot{\varepsilon}_{0}^{t}=3.0 \times 10^{-6} s^{-1}, f_{t}$ is the tensile strength, $\beta_{c}$ is the material constant in compression, $\beta_{t}$ is the tensile material constant.

In this paper, parameters of the granite calibrated by Li [33] are used and listed in Table 2.

### 3.2. Jones-Wilkins-Lee (JWL) EOS

In LS-DYNA, the Jones-Wilkens-Lee (JWL) equation of state is used to describe the relationship between pressure, volume, and energy of a high explosive product. The equation of state (EOS) is given in the following equation [34]:
$P=A\left(1-\frac{\omega}{R_{1} V}\right) \mathrm{e}^{-R_{1} V}+B\left(1-\frac{\omega}{R_{2} V}\right)+\frac{\omega E}{V}$
where, $P$ is the detonation pressure, $A, B, R_{1}, R_{2}, \omega$ are the explosive characteristics parameters, $E$ is the internal energy, $V$ is the relative volume. PETN explosives were used in this calculation. The explosive parameters were confirmed by Ayman [35], and the parameters are reproduced in Table 3.

### 3.3. Verification of the performance of simulation parameters without insitu stress

Fig. 6(a) shows the numerical model used for rock blasting without

Table 2
RHT model parameters for granite.

| Parameter | Value | Parameter | Value | Parameter | Value |
| :--- | :--- | :--- | :---: | :--- | :--- |
| $\rho_{0}\left(\mathrm{~kg} \cdot \mathrm{~m}^{-3}\right)$ | 2700 | $A_{1}(\mathrm{GPa})$ | 86.71 | $\beta_{\mathrm{t}}$ | 0.0144 |
| $G(\mathrm{GPa})$ | 24.17 | $A_{2}(\mathrm{GPa})$ | 145.67 | $G_{\mathrm{c}}^{*}$ | 0.4 |
| $f_{\mathrm{c}}(\mathrm{GPa})$ | 0.119 | $A_{3}(\mathrm{GPa})$ | 89.03 | $G_{\mathrm{t}}^{*}$ | 0.7 |
| $N_{1}$ | 0.56 | $Q_{0}$ | 0.64 | $X I$ | 0.48 |
| $\beta_{\mathrm{c}}$ | 0.0106 | $B_{\mathrm{q}}$ | 0.0105 | $D_{1}$ | 0.042 |
| $B_{0}$ | 1.68 | $A$ | 1.6 | $D_{2}$ | 1.0 |
| $B_{1}$ | 1.68 | $N_{\mathrm{p}}$ | 4.0 | $P_{\text {crush }}(\mathrm{GPa})$ | 0.04 |
| $\alpha_{0}$ | 1.1 | $P_{\text {comp }}(\mathrm{GPa})$ | 5.5 | $A_{\mathrm{f}}$ | 1.62 |
| $T_{1}(\mathrm{GPa})$ | 86.71 | $\dot{\varepsilon}_{0}^{\mathrm{c}}\left(\mathrm{ms}^{-1}\right)$ | $3.0 \mathrm{E}-8$ | $N_{\mathrm{f}}$ | 0.6 |
| $T_{2}(\mathrm{GPa})$ | 0 | $\dot{\varepsilon}_{0}^{\mathrm{t}}\left(\mathrm{ms}^{-1}\right)$ | $3.0 \mathrm{E}-9$ | $\varepsilon_{\mathrm{p}}^{\mathrm{m}}$ | 0.012 |
| $F_{\mathrm{t}}^{*}$ | 0.1 | $\dot{\varepsilon}^{\mathrm{c}}\left(\mathrm{ms}^{-1}\right)$ | 3.0 E 22 |  |  |
| $F_{\mathrm{s}}^{*}$ | 0.38 | $\dot{\varepsilon}^{\mathrm{t}}\left(\mathrm{ms}^{-1}\right)$ | 3.0 E 22 |  |  |

in-situ stress effects. The rock parameters used for in the simulations are presented in Table 2, while the explosives parameters are shown in Table 3. To maintain consistency with Banadaki's experiments [36], the model boundary was not set as a reflection-free boundary. In the RHT model, the LS-DYNA commercial software uses rock damage distribution to simulate blast-induced crack extension. Fig. 6(b) illustrates the blast shock wave strength was greater than the compressive strength of the rock, resulting in a crush zone near the blasthole. Fig. 6(c) shows that a cracked area is created by the tensile component of the stress wave. As the stress wave propagated towards the model boundary, the compressive stress wave is reflected at the free surface, producing a tensile wave. When the rock is subjected to tensile stress waves, it incurred tensile damage, with caused circumferential cracking and spalling (Fig. 6(d)). The final state of the rock after blasting was divided into three zones, as shown in Fig. 6(e). This division corresponds to the pattern of the crack distribution observed in the granite experiments conducted by Banadaki (Fig. 6(f)) [36], indicating that the simulation parameters are reliable.

### 3.4. Results of numerical simulation

The damage distribution of the rock at different moments under the combined effect of blast load and in-situ stress is illustrated in Fig. 7 and Fig. 8. Fig. 7 shows the damage pattern of the rock in equiaxial in-situ stress fields, while Fig. 8 displays the damage pattern of the rock in the anisotropic in-situ stress fields.

In Fig. 7(a), the variation of rock damage with time is presented in the absence of in-situ stress. During the initial stage of the explosion, the blast shock wave level is much larger than the dynamic compressive strength of the rock, resulting in severe radial compression damage in the near zone of the detonation of the rock (Fig. 7(a)). As time passes, the shock wave energy gradually dissipates and decays into a stress wave. Although, the strength of the stress wave is not sufficient to cause radial crushing damage to the rock, the dynamic tensile strength of the rock is much less than its compressive strength, the tensile stress induced by the stress wave leads to tensile damage to the rock [37]. Radial tensile damages continue to increase in the rock as the simulation proceeds to $800 \mu \mathrm{~s}$, and their extent as well with the propagation of the stress wave until the stress wave is not powerful enough to cause damage to the rock Fig. 7(a). The final state of the rock is shown in the $2000 \mu$ s image, where the damage has stopped expanding Fig. 7(a).

As described in Section 2.1, the hoop stresses on the blasthole wall in equiaxial in-situ stress fields are compressive, which impedes blast crack extension (see Fig. 2(a)). Fig. 7(b), Fig. 7(c), Fig. 7(d), Fig. 7(e), and Fig. 7(f), demonstrate a significant decrease in the extent of radial damage in the rock with increasing in-situ stress. An increase in

Table 3
Parameters of JWL EOS for the PETN.

| $\rho_{0}\left(\mathrm{~kg} \cdot \mathrm{~m}^{-3}\right)$ | $D\left(\mathrm{~m} \cdot \mathrm{~s}^{-1}\right)$ | $P_{C J}(\mathrm{GPa})$ | $\mathrm{A}(\mathrm{GPa})$ | $B(\mathrm{GPa})$ | $R_{1}$ | $R_{2}$ | $\omega$ | $E_{0}(\mathrm{GPa})$ |
| :--- | :--- | :--- | :--- | :--- | :--- | :--- | :--- | :--- |
| 1500 | 7450 | 22 | 625 | 23.3 | 5.25 | 1.6 | 0.28 |  |

 failure mode of the model; (f) final pattern of granite blast cracking obtained by test [36].
equiaxial in-situ stress leads to a rise in the magnitude of the hoop compressive stresses in the blasthole wall and the rock, further inhibiting blast cracking. A comparison of the $2000 \mu$ s moment damage images in Fig. 7 (a) and Fig. 7 (f) reveals a reduction in the number of radial major tensile damage cracks from 16 to 8 and inhibition of radial branching development.

Fig. 8 shows the distribution of rock blast damage caused by anisotropic in-situ stress fields. As the value of $K\left(K=P_{\mathrm{x}} / P_{\mathrm{y}}\right)$ increases, the deflection of the major radial tensile damage towards the direction of the maximum principal stress becomes more pronounced.

The static analysis conducted in Section 2.2, explains that, under anisotropic in-situ stress conditions (Fig. 2(b)), maximum compressive stresses occur in the direction of $\theta=90^{\circ}$ and $\theta=270^{\circ}$ on the blasthole walls, inhibiting the crack expansion in this direction. Conversely, crack extension is more likely to occur in the direction of $\theta=0^{\circ}$ and $\theta=180^{\circ}$. These theoretical findings explain the results of the numerical simulations in Fig. 8 very well. Furthermore, all the crush zones in Fig. 8 are elliptical, with the long axis of the ellipse aligned with the direction of the maximum principal stress. As the value of $K$ increases, the long axis of the ellipse grows, and the short axis shortens. These results are consistent with those reported in the literature $[8,10]$.

Note that by rotating the rock damage image in Fig. 8 by $90^{\circ}$ in the instantaneous clockwise direction, a map of the rock damage under the influence of different magnitudes of vertical ground stress can be obtained. This is because when the absolute value of the difference
between $P_{\mathrm{x}}$ and $P_{\mathrm{y}}$ is equal, the circumferential stress distribution curve on the wall of the gun hole is symmetrical.

## 4. Quantitative analysis of numerical simulation results

### 4.1. Image processing techniques

The post-processing software LS-PrePost of ANSYS/LS-DYNA does not provide access to macroscopic damage ranges. The amount of damage to the rock in LS-PrePost is presented on an elemental basis, which poses difficulties for the quantitative assessment of the macroscopic damage characteristics of the rock model. Quantitative analysis is essential for a deep understanding of the effects of in-situ stress on rock blasting damage. In this paper, a method for quantitative assessment of blast damage areas based on image processing techniques is proposed. This method allows for the precise extraction of the target area (damage area) from the numerically simulated image, which is then used as the basis for rapid computational analysis of the target area. Previous studies used numerical simulations with CDEM software by Zhang et al. [24]. The fracture degree, defined as the ratio of the number of fractured contact elements to the total number of contact elements, was introduced to quantitatively describe the damage of rock materials [24]. A new concept called the damage variable, which is similar to the fracture degree, to describe the characteristics of the ANSYS/LS-DYNA numerical simulation software is proposed in this paper. The damage variable is


Fig. 7. Rock damage distribution in equiaxial in-situ stress fields.
denoted by $\eta, \eta$ is given by Eq. (7):
$\eta=S_{t} \times 100 \backslash \% / S$
where, $S_{t}$ is the damaged area (sum of damage pixels) in the numerical
simulation, $S$ is the total area of the numerical model (total number of pixel points in the picture). The magnitude of the $\eta$ value reflects the extent of the damage. The image processing technique consists of four steps, as shown in Fig. 9.


Fig. 8. Rock damage distribution in anisotropic in-situ stress fields.


Fig. 9. Steps in image processing technology.

Step1: Substract the white edges of the images from the numerical simulations. The purpose of this step is to avoid the white blank areas affecting the accuracy of the calculation results.

Step 2: The original Red-Green-Blue (RGB) digital images are converted into grey-scale images. The purpose of this step is to prepare for a binary image.

The digital image that has been converted to a greyscale format contains an $\mathrm{M} \times \mathrm{N}$ matrix of grey values (pixels) of luminance information, expressed by Eq. (8) [38].
$F(M, N)=\left[\begin{array}{ccc}f(1,1) & \cdots & f(1, N) \\ \vdots & \ddots & \vdots \\ f(M, 1) & \cdots & f(M, N)\end{array}\right]$
where, $F(M, N)$ is an $\mathrm{M} \times \mathrm{N}$ matrix of gray values (pixel). $f(i, j)$ is the gray value of a pixel, and $f(i, j)=(R \times 30+G \times 59+B \times 11+50) / 100, R$, $G$, and $B$ are RGB values.

Step 3: Based on the obtained grey-scale histograms, a threshold is selected using the Otsu method (global thresholding algorithm) to
binarise the grey-scale images.
In some simple images where the target regions and the background are distinctly different, the selection of the threshold value is very simple. The grey scale threshold of an image can be selected at the bottom of the valley between the two peaks representing the target regions and the background in the grey scale histogram [39]. However, for the rock damage images covered in this paper, different levels of damage are represented by different colors in the image, and each image contains a large and complex range of colors. The grey scale thresholds selected directly from the grey scale histogram are inaccurate, which will cause significant errors in the calculation results. The Otsu method offers a solution to the problem. The Otsu method is an automatic threshold selection method that relies on the maximum between-class variance between the background and target regions [40].

The total number of pixels in a greyscale image is given by equation Eq. (9), where $g_{\max }$ is the maximum grey scale value in the image:
$N=\sum_{i=1}^{g_{\text {max }}} n_{i}$
$n_{(g)}$ is the total number of pixels in the image with a grey value of $g . P_{g}$ is the proportion of pixels in the image with a grey value $g$. The relationship between $P_{g}$ and $n_{(g)}$ is expressed by Eq. (10).
$P_{g}=\frac{n_{(g)}}{N}$
In addition, the probability distribution of $P_{g}$ is given by Eq. (11):
$\sum_{i=1}^{g_{\text {max }}} P_{g}=1$
According to the principles of the Otsu method for selecting thresholds, the threshold $T_{0}$ is given by Eq. (12).
$T_{0}=\underset{1 \leqslant g \leqslant g_{\text {max }}}{\operatorname{argmax}}\left\{\sigma_{B}^{2}(g)\right\}$
where $\sigma_{B}^{2}(g)$ is the maximum between-class variance.
Step 4: The pixels of the binarised image only have grey values of 0 and 255. After the binarization process, the image is traversed, and the pixels with a grey value of 255 are added up to obtain the total number of pixels in the target region. The total number of pixels is defined as the area of the target region $S_{t}$. Finally, by inputting $S_{t}$ into the Eq. (7), the final value of $\eta$ can be obtained.

### 4.2. Quantitative analysis of rock damage

One image of damage was captured every $100 \mu \mathrm{~s}$, starting from $0 \mu \mathrm{~s}$ mark. A total of 21 images were captured for each calculation. The damage variables in each of the numerical simulation plots are calculated using image processing techniques, and the variation curves of damage variables are plotted with increasing time.

### 4.2.1. Variation of damage variables in equiaxial in-situ stress fields

Fig. 10 illustrates the dynamics of the damage varies over time, which shows the three main stages in the evolution of damage variables under equiaxial loading. The first stage (Zone 1) ranges from $0 \mu$ s to 400 $\mu$ s it is dominated by the damage caused by the blast shock wave, as the rock is compressed during this phase and the damage variables increase rapidly. The second stage, between $400-\mu \mathrm{s}-1400 \mu \mathrm{~s}$ (Zone 2), is dominated by blast stress waves, as radial cracking continues to expand under the tangential component of the blast stress wave. The growth rate of the damage variable is slower than in the first stage. The third stage ranges from $1400 \mu$ s to $2000 \mu$ s (Zone 3), at which point the blast stress wave can no longer cause extensive damage to the rock. With the end of the blast, the damage zone in the rock no longer increases, and the growth of the damage variable tends to stagnate. In addition, there is an observable suppressive effect of in-situ stress on all three stages of the


Fig. 10. Evolution of damage variables in equiaxial in-situ stress fields.
evolution of the rock damage variable. That is, the higher the in-situ stress, the smaller the damage variable in all three phases.

When the blasting process is completed at the 2000th $\mu \mathrm{s}$, the damage distribution can be considered as the final damage state of the model. To establish a mathematical relationship between the damage variables and the magnitude of the in-situ stress in the final state of the model, the damage variables for different equal biaxial loading are summarized in Fig. 11. The damage variable is inversely proportional to the hydrostatic pressure, as reflected in Fig. 11, and a good linear relationship was found after fitting. In this paper, the damage variable related to the equiaxial pressure $P_{\mathrm{x}}\left(P_{\mathrm{y}}\right)$ can be expressed by Eq. (13):
$\eta=20.243-0.931 P_{\mathrm{x}}$

### 4.2.2. Variation of damage variables in anisotropic in-situ stress fields

Fig. 12 illustrates the time course of rock damage evolution in anisotropic stress fields. The time-course curve trends for the rock damage variables shown in Fig. 12 are consistent with those in Fig. 11 and can be divided into three phases. The phases correspond to the compressional damage phase caused by the dominant blast shock wave (Zone 1), the rapid expansion of radial cracks caused by the dominant


Fig. 11. Relationship between damage variable and equiaxial static stress.


Fig. 12. Evolution of damage variables in anisotropic in-situ stress fields.
blast stress wave (Zone 2), and the final stage of the blast (Zone 3). An interesting phenomenon emerges from the zoomed-in view in Fig. 12. The magnitude of the rock damage variables $\eta$ in Cases 9 and 10 exceeds those in Cases 7 and 8, ranging from $400 \mu \mathrm{~s}$ to $800 \mu \mathrm{~s}$. After $800 \mu \mathrm{~s}$, the rock damage variables $\eta$ in Cases 9 and 10 become progressively smaller than in Cases 7 and 8.

This phenomenon occurs because the maximum hoop tensile stresses in the blasthole wall of Cases 9 and 10 occur in the horizontal direction (see Fig. 2(b)), which contributes to blast crack extension and results in a rapid increase in blast damage variables. As one moves further away from the hole, the annular stress gradually changes from tensile to compressive (see Fig. 3(b)), which changes the effect on blast damage from a facilitator to a hindrance, resulting in a progressively smaller damage.

Different degrees of anisotropy were stablished by fixing $P_{\mathrm{y}}$ equal to 20 MPa and making $P_{\mathrm{x}}$ equal to $20 \mathrm{MPa}, 40 \mathrm{MPa}, 60 \mathrm{MPa}, 80 \mathrm{MPa}$, and 100 MPa , respectively. The damage variables for the final damage state of the rock in different anisotropic in-situ stress fields were calculated and plotted (Fig. 13). Meanwhile, Fig. 13 suggests that as the horizontal stress $P_{\mathrm{x}}$ (or anisotropy ratio) is raised, the damage variable is reduced. By fitting the data points, it was found that when $P_{\mathrm{y}}=20 \mathrm{MPa}, P_{\mathrm{x}}$ and


Fig. 13. Relationship between damage variable and anisotropic static stress.
the damage variable $\eta$ can be described by the following equation:

$$
\begin{equation*}
\eta=18.336+0.16271 P_{\mathrm{x}}-0.09929 P_{\mathrm{x}}^{2} \tag{14}
\end{equation*}
$$

### 4.2.3. Effect of anisotropy ratio on damage variables

The effect of anisotropy ratios on rock damage distribution is significant. Blasting damage is approximately symmetrically distributed in equiaxial in-situ stress fields (Fig. 7). In contrast, blast damage in anisotropic stress fields is mainly distributed in the direction of the maximum principal stress, with less damage distribution in the direction of the minimum principal stress (Fig. 8). This conclusion has been confirmed in previous studies [17-20,41,42]. However, the quantitative description of the difference between blast damage in the direction of maximum principal stress and minimum principal stress is rarely reported. In addition, the quantitative assessment of the change in rock damage with increasing in-situ stress anisotropy has also been reported only in a few previous studies. Yue et al. [23] set up 10 regions in the model to describe the degree of fracture in the loading direction and in the vertical direction of loading. However, the 10 zones set did not cover the entirety of the model, and no conclusions were drawn on the global nature of in-situ stress damage to the rock. Thanks to the efficiency of image processing techniques, a global assessment of rock damage was achieved in this paper. As shown in Fig. 14, the numerical model is cut into four triangular regions. To quantify the effect of changes in the value of the anisotropy ratio $K\left(K=P_{\mathrm{x}} / P_{\mathrm{y}}\right)$ on the damage distribution, the anisotropic damage variables $\eta_{\mathrm{k}}$ is presented, $\eta_{\mathrm{k}}$ defined by Eq. (15):
$\eta_{\mathrm{k}}=S_{\mathrm{x}} \times 100 \backslash \% / S_{\mathrm{y}}$
where $S_{\mathrm{x}}$ is the sum of the areas of damage appearing in zones II and IV and $S_{\mathrm{y}}$ is the sum of the areas of damage appearing in zones I and III. The four zones in Fig. 14 cover the entire model, so the results of the study can reflect how the anisotropy ratio affects the model as a whole.

Step 4 of Section 4.1 was improved to apply to the new image processing. This is done by giving traversal boundaries of each image according to the range shown in Fig. 14 and thus obtaining the damaged area of $S_{\mathrm{x}}$ and $S_{\mathrm{y}}$. Finally, the value of $\eta_{\mathrm{k}}$ for the anisotropic damage variable is calculated according to Eq. (15). The trends over time for each of the four anisotropic damage variables, obtained using image processing techniques, are shown in Fig. 15.

It is observed in Fig. 15 that the value of $\eta_{\mathrm{k}}$ rises with $K$ at the same moment in time. In addition, the curves in Case $7(K=2)$ and Case 8 $(K=3)$ are flat and have smaller values. The curves for Case $9(K=4)$ and Case $10(K=5)$ increase sharply at around $400 \mu$ s and remain at large values thereafter. The results of the theoretical analysis in Section 1. can explain the dramatic increase in the $\eta_{\mathrm{k}}$-curves of Case $9(K=4)$ and Case $10(K=5)$. When $K$ equals 4 or 5 , the hoop stress in the


Fig. 14. Division in the numerical model.


Fig. 15. Time course curves for each anisotropic damage variable.
horizontal direction of the blasthole wall is tensile, and the initial blast damage is created first in the direction of the maximum principal stress. The existing cracks have a direct effect on the further extension of blast cracks, which provides good conditions for the rapid development of blast damage in that direction. When $K$ equals 2 or 3 , there is no such situation. It is speculated that this is consistent with the cause of the anomalous change in the damage variable $\eta$ at the $400 \mu$ s time in the enlarged plot of Fig. 12 in Section 4.2.2.

## 5. Conclusions

This paper investigates the effects of in-situ stress on rock blast damage using theoretical analysis and numerical simulation methods. Although numerical simulations have been widely used in the study of the dynamic response of deep rock, previous studies have lacked a quantitative description of model damage phenomena. Therefore, in this paper, a processing of images from ANSYS/LS-DYNA numerical simulation results were processed using image processing techniques to quantitatively assess the distribution of rock blast damage.
(1) The analysis of rock stresses under static loading was carried out, and the distribution of stresses on the borehole wall as well as in the far field was determined. The analysis demonstrates that under isotropic loading, the hoop compressive stresses are symmetrically distributed on the borehole wall, and the magnitude of the far-field stresses converges to the magnitude of the hydrostatic pressure. Under anisotropic loading, the maximum compressive stress on the blasthole wall occurs in the direction of the minimum principal stress, with the far-field stress tending towards $P_{\mathrm{y}}$ in the x-axis direction and $P_{\mathrm{x}}$ in the y-axis direction.
(2) The damage variable was introduced, and the time-course curve of the damage variable was plotted using image processing techniques to analyze the damage cloud at different moments. The dynamic evolution of rock blasting damage under in-situ stress was categorized into three stages on the curves trend. In equiaxial in-situ stress fields, the evolution of blast damage is suppressed throughout due to the effect of in-situ stress. In anisotropic stress fields, the tensile stress in the direction of the maximum principal stress causes the damage variable deviate from the being smaller at a given time with higher in-situ stresses.
(3) In the final state, the damage variable $\eta$ and the hydrostatic pressure $P_{\mathrm{x}}\left(P_{\mathrm{y}}\right)$ can be fitted using the equation: $\eta=$ $20.243-0.931 P_{\mathrm{x}}$. In an anisotropic stress field, when $P_{\mathrm{y}}$ is 20

MPa , the relationship between the damage variable $\eta$ and $P_{\mathrm{x}}$ in the final state can be represented by $\eta=18.336+0.16271 P_{\mathrm{x}}-0.09929 P_{\mathrm{x}}^{2}$.
(4) To overcome the challenge of quantitatively describing the effect of the anisotropy ratio $K$ on damage distribution, the anisotropic damage variable $\eta_{\mathrm{k}}$ was introduced. The analysis demonstrates that $\eta_{\mathrm{k}}$ is positively correlated with $K$ at the same moment. When the hoop stress in the direction of the maximum principal stress is tensile, rock damage is promoted, leading to a sharp increase in $\eta_{\mathrm{k}}$.

## CRediT authorship contribution statement

Guodong Qiao: Conceptualization, Investigation, Software, Writing - original draft, Visualization. Zegong Liu: Resources, Supervision, Funding acquisition. Changping Yi: Writing - review \& editing, Methodology. Kui Gao: Software, Validation. Gaoyuan Xuan: Formal analysis.

## Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

## Data availability

No data was used for the research described in the article.

## Acknowledgments

This work was supported by the Graduate Research Project of Anhui Education Department (YJS20210396), National Natural Science Foundation of China (52074013), and Natural Science Foundation of Anhui Province (2208085ME125), which are gratefully appreciated. Thanks to Carlota Rodriguez San Miguel for proofreading the article.

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