Numerical Modelling of Scale Effect on Mode of Failure and Strength of Offset Rock Joints

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Abstract. The studies on the scale effect on the behavior of non-persistent rock joints are limited and the nature of scale dependency is still not well understood. In this study, the scale effect on failure mechanisms and compressive strength of rock blocks tested previously by the authors was investigated numerically. In the experimental study, two different block sizes, having dimensions of (63.5 x 28 x 20.3) cm and (30.5 x 15.24 x 10) cm, were tested. Samples of rock with non-persistent offset joints were subjected to uniaxial loading. The joint inclination angle was maintained at 22.5° in both cases. Also, degree of persistence was kept constant at 0.3 for all tested blocks. However, the offset angle which connects the inner tips of the joints was changed from 30°-90° with an increment of 15°. In the current study, finite element analyses were performed on the arrangements that were studied experimentally. ANSYS 19 Mechanical APDL was utilized to perform static structural analysis. A linear elastic material was assumed for these analyses, mainly because there were no signs of major material damage prior to fracture. Modulus of elasticity of approximately 10510 MPa and a Poisson’s ratio of 0.25 were assumed. The boundary conditions applied to the finite element model were zero vertical displacement along the bottom edge, and a uniform distribution load on the top surface, the magnitude of which was approximately the measured coalescence load (in MPa) for the joint geometry analyzed. The stress distribution within the block and near the tips of joints in particular was analyzed in terms of maximum principal stresses, shear stresses and factor of safety resulted from maximum tensile stress theory. The FEM results were compared with the experimental results. The results of FEM emphasized the observed experimentally results regarding the decrease in strength as the size of block sample increased and no effect of block size on mode of failure.

1. Introduction
The strength of rock mass is complex due to the fact that many factors are involved in controlling the strength of rock mass under loading. One factor that needs more studies is the scale effect on strength of rock mass with non-persistent joints. The scale effect on rock mass with persistent rock joints has been studied previously by many researchers \cite{1,2} and they indicated the existent of size effect on the strength of rock mass. While, the behavior of non-persistent rock joints at failure under uniaxial or biaxial loading condition is not well understood due to the interaction between the intact rock segments and the joint segments. Many researchers \cite{3,4} have studied the effect of joints orientation and size on the strength of rock mass. The effect of joint orientation on mode of failure and propagation of crack has been studied also by many researchers \cite{5-8} but still no common agreement on the modes of failure has been reached. The interaction between the intact rock and the joints depends on many parameters such as the geometry of the joint and bridge segments, the slope of the
joint segment, the stiffness of the rock among others, [9-15]. Generally, the process of failure begins with the initiation of cracks perpendicular to the joint segment which then becomes parallel to the principal loading axis. These cracks are called wing cracks. Wing cracks are followed by another type of cracks called secondary cracks that initiate from the inner joint segment tips and propagate toward each other and meet across the intact rock segment causing the coalescence of the joint segments leading eventually to failure of the rock. Propagation and coalescence of the secondary cracks are very quick compared to that of the wing cracks.

In the current work numerical modelling was used to study the effect of scale on the strength and more of failure of rock mass with offset nonpersistent joints. Numerical modelling is a powerful tool to understand the crack propagation and mode of failure in jointed rock mass and has been used intensively recently for understanding the behavior in rock mass and jointed rocks as well, [16-18]. Scale effect on the behavior of rock mass containing no-persistent rock joints has been acknowledged by many researchers and more studies are needed.

2. Methods

2.1. Experimental work

This current work is a continuation to the previous work that has been done by the first author and his co-workers [19]. Mughieda et al. [19] have studied the effect of scale on the strength and mode of failure of offset non-persistent rock joints experimentally. In the experimental study; two different block sizes, having dimensions of (63.5 x 28 x 20.3) cm and (30.5 x 15.24 x 10) cm, were tested. Samples of rock with non-persistent offset joints were subjected to uniaxial loading. The joint inclination angle was maintained at 22.5° in both cases. Also, degree of persistence was kept constant at 0.3 for all tested blocks. However, the offset angle which connects the inner tips of the joints was changed from 30°-90° with an increment of 15°.

The procedure developed by Jamil [10] for preparing open non-persistent joints was used in this research with some modification. The sample preparation mold and set-up is shown in figure 1. A typical sample dimensions are shown in figure 2.

Figure 1. Sample preparation mold and set-up for making open joints [19]
2.2. Description of finite element model

In order to determine the state of stress for uniaxial compressive loading on Rock blocks, finite element analyses were performed on the different arrangements that were studied experimentally. ANSYS 19 Mechanical APDL is utilized to perform static structural analysis. A linear elastic material was assumed for these analyses, mainly because there were no signs of major material damage prior to fracture. Modulus of elasticity of approximately 10510 MPa and a Poisson’s ratio of 0.25 were assumed.

Quadratic eight-noded quadrilateral plain strain element with two degree of freedom per node, and quadratic three-noded constant strain triangular element with two degree of freedom per node were used in the present study. Refinement of mesh with order 3 is implemented around the joint surfaces for better results. The geometric model and the mesh used are shown in figure 3.

Figure 2. Geometry of the specimens and pre-existing cracks [19]

Figure 3. Geometric model and meshing
The boundary conditions applied to the finite element model are zero vertical displacement along the bottom edge, and a uniform distribution load on the top surface, the magnitude of which was approximately the measured coalescence load (in MPa) for the crack geometry analysed. The three-dimensional effect in the experiments is neglected since the model used in the present study was two-dimensional.

The two different block sizes, having dimensions of (63.5 x 28 x 20.3) cm and (30.5 x 15.24 x 10) cm, were modelled at joint inclination angle maintained at 22.5° in both cases. However, the offset angle is modelled for 30°, 45°, 60° and 90°.

3. Results and discussion

3.1. Mode of failure

3.1.1. Slightly offset joints (β = 22.5°, α=30°)

The experimental study showed that failure was due to shear stresses by coalescence of secondary cracks at a point in the rock bridge. Crushed and pulverized materials have been noticed on the failure surface. No scale effect was noticed on mode failure. The failed specimen and sketch of the wing and secondary cracks are shown in figure 4. The shear stress, maximum principal stresses distribution in the tested blocks and the factor of safety based on tensile stress theory are shown in figure 5. The FE analysis also showed that shear and max principal stresses are high and concentrated at the inner tip of the joints causing the propagation of cracks till coalescence and failure for both sizes.

![Figure 4. Crack growth in 22.5°-30° flaws [19]](image)
### 30°-flaw

| Large Block | Shear Stress | Safety Factor maximum tensile stress theory | Maximum Principal stress |
|-------------|--------------|---------------------------------------------|--------------------------|
| ![Shear Stress](image1.png) | ![Safety Factor](image2.png) | ![Maximum Principal Stress](image3.png) |

| Small-Block | Shear Stress | Safety Factor maximum tensile stress theory | Maximum Principal stress |
|-------------|--------------|---------------------------------------------|--------------------------|
| ![Shear Stress](image4.png) | ![Safety Factor](image5.png) | ![Maximum Principal Stress](image6.png) |

**Figure 5.** Shear stress, maximum principal stress and factor of safety based on max. tensile stress theory for 22.5°-30° flaws.

#### 3.1.2. Offset joints (β= 22.5°, α=45°)

In these blocks coalescence occurred through the growth of secondary crack matching through the flaws. The FE analysis emphasized the experimental results and showed that both shear stress and max tensile stress are highly concentrated at the tip of the inner joints and were responsible for the failure of the block. The mode of failure in both sizes was identical. The failed specimen and sketch of the wing and secondary cracks are shown in figure 6. Shear stress distribution, max. principal stress and factor of safety based on max tensile stress theory results are shown in figure 7.

**Figure 6.** Crack growth in 22.5°-45° flaws [19]
3.1.3. Offset joints (β = 22.5°, α = 60°)

The FE analysis showed that shear and tensile stresses are both responsible for the failure of the blocks through the propagation of the secondary cracks along the rock bridge. This result explained the mode of failure obtained from the experimental study. In both block sizes the failure initiated by wing cracks propagated from the external flaw tips only, extended to a certain distance from the tips of the joints and stopped. Coalescence occurred through the growth of secondary crack matching through the flaws. The failed specimen and sketch of the wing and secondary cracks are shown in figure 8. Shear stress, maximum principal stress and factor of safety based on max. tensile stress theory distributions are shown in figure 9.

![Figure 7. Shear stress, maximum principal stress and factor of safety based on max. tensile stress theory for 22.5°-45° flaws.](image)

| 45°-flaw | Shear Stress | Safety Factor-maximum tensile stress theory | Maximum Principal stress |
|----------|--------------|---------------------------------------------|-------------------------|
| Large-Block |
| ![Image](image) |
| ![Image](image) |
| ![Image](image) |
| Small-Block |
| ![Image](image) |
| ![Image](image) |
| ![Image](image) |

![Figure 8. Crack growth in 22.5°-60° flaws [19]](image)
3.1.4. Offset joints (β = 22.5°, α = 90°)
In these blocks failure occurred due to coalescence of wing cracks which are tension cracks. The FE analysis showed that that the area in between the inner tips of the joints were subjected to high value of tensile stresses which were responsible for the failure of the sample. The failed specimen and sketch of the wing and secondary cracks are shown in figure 10. Shear stress, maximum principal stress and factor of safety based on max. tensile stress theory are shown in figure 11.

Figure 9. Shear stress, maximum principal stress and factor of safety based on max. tensile stress theory for 22.5°-60° flaws.

Figure 10. Crack growth in 22.5°-90° flaws [19]
80°-flaw

Shear Stress

Safety Factor-maximum tensile stress theory

Maximum Principal stress

| Large-block | Shear Stress | Safety Factor-maximum tensile stress theory | Maximum Principal stress |
|-------------|--------------|--------------------------------------------|--------------------------|
| 30º         | Tensile + Shear failure | Tensile + Shear failure | Tensile failure |
| 45º         | Tensile + Shear failure | Tensile + Shear failure | Tensile failure |
| 60º         | Tensile failure | Tensile failure | Tensile failure |
| 90º         | Tensile failure | Tensile failure | Tensile failure |

| Small-block | Shear Stress | Safety Factor-maximum tensile stress theory | Maximum Principal stress |
|-------------|--------------|--------------------------------------------|--------------------------|
| 30º         | Tensile + Shear failure | Tensile + Shear failure | Tensile failure |
| 45º         | Tensile + Shear failure | Tensile + Shear failure | Tensile failure |
| 60º         | Tensile failure | Tensile failure | Tensile failure |

Figure 11. Shear stress, maximum principal stress and factor of safety based on max. tensile stress theory for 22.5º-90º flaws

The results of mode of failure of all tested samples are summarized in table 1.

Table 1. Mode of block failure with different bridge inclinations

| Large sample | 30º | 45º | 60º | 90º |
|--------------|-----|-----|-----|-----|
| Tensile + Shear failure | Tensile + Shear failure | Tensile failure | Tensile failure |

| Small sample | 30º | 45º | 60º | 90º |
|--------------|-----|-----|-----|-----|
| Tensile + Shear failure | Tensile + Shear failure | Tensile failure | Tensile failure |

4. Conclusion

The FE results were compared with the experimental results. The results of FE emphasized the observed experimentally results as no effect of block size on mode of failure. Also, the FE analysis explained the experimentally observed modes of failures. Mode of failure of rock bridge varies from shear to tensile failure as the bridge inclination increases. For slightly offset joint (α = 30º or 45º) the failure was due to combined shear and tension failure while it was purely tension for overlapped joints (α = 90º).
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