Study on the mechanical properties of thin-layered marbleized dolomite by in situ tests in the Wudongde hydropower station, China

Wei Hu¹, Yun Zheng*¹, Aiqing Wu¹, Shihu Xiong¹, Yihu Zhang¹

1. Key Laboratory of Geotechnical Mechanics and Engineering of the Ministry of Water Resources, Yangtze River Scientific Research Institute, Wuhan 430010, Hubei, China

*Corresponding author: Yun Zheng
Email: Yunzheng@lzb.ac.cn

Abstract: Transverse isotropy is one of the most distinctive features of layered rock, and many previous researchers have studied the layered rock structure using mechanical laboratory tests; however, very little work has been done based on large-scale mechanical in situ tests. In this paper, we studied typical thin-layered strata that are composed of dolomite marble embedded with phyllite in the Wudongde hydropower station of China. To determine the deformation and failure behavior of the thin-layered rock mass during excavation, a set of large-scale cubic in situ samples were prepared. First of all, polyaxial deformation tests with confining stress were carried out on each sample, then an unloading process perpendicular to the layers was undertaken to investigate strength parameters. Based on the experimental results, the anisotropy of the thin-layered rock has been discussed. Because of the weakness of the layered discontinuities, it is more deformable when loading perpendicular to the layers. It shows that the layered discontinuities would open to induce failure when lateral pressure is released that is perpendicular to the layers during the unloading strength tests. The strength parameters from in situ tests are fitted by the conventional Mohr–Coulomb criterion, and the cohesion is about 10.33 MPa, whereas the internal friction angle is about 27.9°, which is slightly less than the strength deduced from the triaxial test in the laboratory.

Keywords: Transverse isotropy; In situ triaxial test; Anisotropy; Wudongde hydropower station
1. Introduction

Varieties of sedimentary and metamorphic rocks, such as limestone, dolomite, sandstone, shale, slate, gneiss, schist, and marble are usually exposed on the surface as an outcrop with layered rock mass. These layered rocks exhibit some preferred orientation of fabric or possess distinct bedding planes, which results in transverse isotropic behavior on the macroscale.

Because of the anisotropy of rock structure, the layered rocks show substantially different mechanical properties from isotropic rocks. Numerous researchers have performed experimental work on mechanical anisotropy since the beginning of the 1960s. Jaeger (1960) conducted a pioneering study on the mechanical anisotropy of transversely isotropic rock, and the experimental results show that the strength varies continuously with $\beta$ (i.e., the angle between the direction of the major principal stress and the layer plane), after that, various transversely isotropic rocks were sampled and examined for uniaxial and triaxial mechanical tests (Pinto, 1970; Attewell and Sandford, 1974; Ramamurthy et al., 1985; Tien and Tsao, 2000; Nasseri et al., 2003).

Based on the experimental research, continuous or discontinuous failure criteria were deduced for predicting the variation of the compression strength of transversely isotropic rocks with varied orientations under various confining pressures. Jaeger (1960) modified the Mohr–Coulomb criterion to obtain two independent failure modes, one is a discontinuous criterion which describes the failure strength of rock material containing a single plane or a system of parallel planes of weakness, and the other describes the failure strength of anisotropic rocks with varying cohesion, but with a constant angle of internal friction. Hoek and Brown (1980) postulated the strength parameters in their well-known failure criterion, varying depending on the direction of the weakness plane for describing the failure of transversely isotropic rock. Nowadays, more and more rock engineering designs and structures are related to the transversely isotropic rocks, and new empirical failure criteria and methodology presented for predicting potential rock failure in engineering construction (Rafii, 2011; Saeidi, 2014; Chen, et al., 2016; Xu, et al., 2017).

The existing research on the mechanics of transversely isotropic rock is mostly based on laboratory tests and theoretical knowledge. Due to scale limitations, it is difficult to obtain an ideal failure criterion that not only predicts the state of stress at failure but also the failure mode of an anisotropic rock mass. Therefore, a set of large-scale samples (500 mm × 500 mm × 1000 mm), that were rarely adopted in previous related experiments, were prepared in the exploratory pit throughout a thin-layered stratum in the Wudongde hydropower station in China. Referring to the stress change and collapse mode during excavating, a similar loading path was applied to the in situ samples. The deformation and failure behaviors of the large-scale transversely isotropic rock samples have been recorded. It is believed that this work will be very helpful to researchers with similar interests and will be a valuable reference for further research on the mechanics of isotropic rock mass.

2. Layered rock mass in the Wudongde hydropower station

The Wudongde hydropower station is built on the Jinsha River in Southwest China. A concrete double-
curvature arch dam with a height of 270 m was designed with a water storage capacity of $7.408 \times 10^9$ m$^3$, and a total installed capacity of 10200 MW. It will be the fourth largest hydropower station in China after completion.

The underground tunnels for the powerhouse in the right bank are under a medium-level crustal stress condition where the principal stress is about 8–13 MPa, and the direction of maximum horizontal principal stress is nearly NE65°. Part of the underground tunnels will be excavated into thin and extremely thin-layered rock strata denoted as $P_t^4_{2l}$. The lithology of that stratum is gray dolomite marble interbedded with phyllite. The thickness of a single layer ranges from 10 mm to 50 mm (Figure 1), with some extremely thin layers having a thickness of less than 10 mm (Chen et al., 2016).

During the excavating of underground tunnels and the other tunnels that run through the thin-layered strata, the excavated face usually appears with an enormous deformation or even collapses, particularly in the location where the strata have a strike direction parallel to the wall with a high dip angle. Because of the perpendicular stresses released after excavation, the layer planes can be easily bent, buckled, and fractured under an asymmetric load.

To determine the mechanical properties of a thin-layered rock mass under excavation, a test pit was opened on the exploratory pit beside underground tunnels for the powerhouse. The section size of the test pit is 2 m $\times$ 2 m, and the axial direction is 260°. The surrounding rock mass is dolomite marble interbedded with phyllite and shows thin and extremely thin-layered structures.

![Figure 1. Thin-layered rock mass surrounded](image)

3. Laboratory tests

Initially, physical and mechanical tests on dolomite marble interbedded with phyllite samples taken from the test pit were completed in the laboratory. Because of the weakness of the discontinuities, it is difficult to obtain an unbroken core sample by drilling, so we took a block out of the pit wall, then sliced it into a cubic block. The axial length is about 100 mm which is double the section length. Each specimen is composed of three to five layers, and the layer plane is nearly parallel to the axis with an angle $\beta$ less than 10°. The dry density of thin-layered rock is about 2.77 g/cm$^3$, porosity is 1.3%, and the natural
moisture content is 0.41%.

The mechanical tests follow the ISRM standard methods. The uniaxial compression tests were completed in dry and saturated conditions, and the results are shown in Table 1. Comparing the results of dry and saturated samples, it is seen that the strength of the material is greatly affected by water.

Five levels of confining pressure were applied to two groups of triaxial tests (i.e., 2, 4, 6, 8, and 10 MPa). The conventional Mohr–Coulomb criterion was used for characterizing the rock strength, thus the resulting cohesion is about 6.53 MPa, and the internal friction angle is about 55.2°.

The failure modes vary with confining pressure, and under nonconfining or lower confining pressure, the lateral deformation is mainly contributed by the opening of discontinuities, and it shows a tensile-split failure mode (Figure 2a). When the confining pressure was increased the failure mode changed into shearing across the layers (Figure 2c).

Table 1: Results of uniaxial compression tests

| Parameters                  | dry   | saturated |
|-----------------------------|-------|-----------|
| UCS (MPa)                   | 49    | 25.9      |
| Tangent modulus (GPa)       | 20.6  | 11.4      |
| Deformation modulus (GPa)   | 12.9  | 6.6       |
| Poisson’s ratio             | 0.26  | 0.28      |

(a) Nonconfining pressure   (b) Lower confining pressure   (c) Higher confining pressure

**Figure 2.** Failure modes under compression

4. Preparation for large-scale in situ polyaxial testing

Large-scale in situ triaxial tests on rock mass were initially performed by Interfels researchers in 1961 (Müller, 1972). They cut large cubic rock stubs for sampling in the exploratory pit of Kurobe Dam (Japan). Each sample was nearly 11 m³ (2.8 m × 2.8 m × 1.4 m). Several flat hydraulic jacks and piston hydraulic jacks were set in the gaps between the samples and pit walls to supply polyaxial compression force (Müller, 1972). Although it is costly with numerous limitations of site condition to conduct in situ tests, nevertheless, it is regarded as the best approach to investigate the mechanical properties of rock masses, especially for jointed rock mass for which it is difficult to get a large undisturbed sample into a
laboratory.

As shown in Figure 3, we used a thin-layered rock sample set in the test pit for triaxial tests in Wudongde. A rock stub was dug out from the ground and burnished to the size of 50 cm × 50 cm × 100 cm. Each face of the sample was fully covered by a steel plate that is 3 cm thick. Each plate has a window with a size of 3 cm × 3 cm in the center so that a steel measuring bar for representing the deformation of each side during loading processes can be fixed in the sample directly. A total of five measuring bars were installed in the four lateral faces and the top face. We added another two plates on the first layer to increase the entire stiffness. It was kept in mind that the applied force should distribute on the sample uniformly. Each lateral side was bounded by two piston hydraulic jacks, and the jacks on each pair of opposite sides were set on the same axis, and all were pumped by the same hydraulic station. Four piston hydraulic jacks were set on the top face to supply an axial force.

Further details of sampling and installation are: (i) clear the broken rocks on the floor of the test pit and burnish a smooth square face as the top of the sample, (ii) dig out the rocks beside two parallel edges, make smooth two of the lateral faces, and install steel plates and jacks, apply 0.2 MPa stress on the two parallel lateral faces for holding the sample steady during the next approach; then dig out the rocks beside the other two parallel edges and do the same operation to complete the installation on the lateral sides, (iii) install the steel plates and jacks on the top of the sample, apply 0.2 MPa axial stress to make sure the apparatus on the sample is fixed tightly, (iv) connect the jacks to the hydraulic stations with pipes, three hydraulic stations control loading processes in three directions independently, and (v) fix the measuring bars in the samples, and connect measuring bar to the displacement sensor.

Figure 3. A schematic diagram and image, showing the in situ triaxial test system in Wudongde

5. Loading processes

A total of six in situ samples (S1 to S6) were prepared and tested individually. In the test pit, the attitude of the in situ layers is about 170°–180° ∠76°–83°, and the layers are nearly parallel to the axis of the test pit with a high dip angle. As shown in Figure 4, the local coordinates are defined as the Z axis, which is perpendicular to the ground, the X axis is parallel to the layers in the horizontal plane, and the Y axis is perpendicular to the layers in horizontal planes. The compression stresses applied on each axis are denoted as $\sigma_z$, $\sigma_x$, and $\sigma_y$, respectively.
First of all, each sample was subjected to triaxial deformation with two confining levels in three directions, and then unloaded to failure under the polyaxial stress condition. Referring to the crustal in situ stress, the higher confining level was 4 MPa (lateral pressure confined), and the axial load was gradually increased from 4 MPa to 8 MPa. In the lower confining pressure condition, the axial load was gradually increased from 0.2 MPa to 4 MPa, and the lateral confining pressure was 0.2 MPa. The total load steps added on an in situ sample are shown in Figure 4, and the details are as follows: (i) Steps 1–6: Deformation test under lower confining pressure conditions. The deformation tests were carried out in all three directions separately (in the order of Y direction, X direction, and then Z direction). In step 1, the normal stress is applied in the Y direction and increased gradually to 4 MPa with several increments, and then the normal stress was gradually unloaded to 0.2 MPa in step 2. During the deformation test in the Y direction, the lateral confining stresses in the X and Z directions remained constant at 0.2 MPa. After completion of the Y direction deformation test, a similar procedure was adopted for the tests in the X direction and then the Z direction. (ii) Step 7: the confining stresses in all three directions were increased to 4 MPa as the initial condition for the higher stress level deformation tests. (iii) Steps 8–13: deformation tests under the higher confining stresses, three deformation tests were carried out according to the similar procedure as in the lower stress condition. (iv) Step 14: unloading the confining stresses in all three directions to 0.2 MPa. (v) Step 15: increasing the confining stresses in all three directions to about 8 MPa. (vi) Step 16: keeping stresses in the X and Y directions at 8 MPa, and increasing the stress in the Z direction at a certain level that is lower than the strength value. (vii) Step 17: keeping stresses constant in the X and Z directions, and gradually unloading the stresses in the Y direction to failure.

**Figure 4.** A schematic diagram and images of the assumed local coordinates

**Figure 5.** Loading path in the triaxial test
6. Deformation properties of thin-layered rock

6.1 Deformation behavior under confined stress

Two confining stress levels were applied to the six in situ samples. The samples were prepared in a similar manner, which showed identical layered orientation. The stress–strain curves of a typical sample (S5) are shown in Figure 6, and the deformation behavior is explained as: (i) Nonlinear deformation: the stress–strain curve of the loading direction develops as concave up the curve in the lower confining state, and this nonlinearity decreases under the higher confining stress condition. It is well known that the unloading process behaves as a nonlinear deformation for most rock materials, which is also true for the thin-layered rock used in this study. During the low confined deformation test, the strain rebounded to some extent in the initial unloading steps, and then changed a lot at the end of the unloading process, (ii) anisotropic deformation: according to the defined local coordinates, it is nearly parallel to the layers that were loaded in the X and Z directions, and it is perpendicular to the layers that were loaded in the Y direction. Referring to Figure 6a and Figure 6b, the compressed areas and stress conditions in the Y and X directions are the same, however, the normal strain of loading perpendicular to the layers (Y direction) is greater than the normal strain of loading parallel to the layers (X direction), and the difference is more significant under a higher confined state as shown in Figure 6d and Figure 6e.

Figure 6. Stress–strain curve of the S5 sample during two confining stress levels in three directions
6.2 Parameters from deformation test

The total strain in a load step includes the elastic strain and the inelastic strain. It is worth estimating the elastic strain from a loaded and unloaded cycle, and the resilient modulus can stand for elastic modulus and is simplified as \(E_i\) \((i = x, y, z)\). Considering the nonlinearity of the stress–strain curves in a loading step, it is adequate to adopt the slope of the straight section for estimating the deformation modulus, and the deformation modulus can be simplified as \(E_i\) \((i = x, y, z)\), as well as Poisson’s ratio can be estimated by the slope of the straight line in the curve of normal strain versus lateral strain, and Poisson’s ratio is denoted as \(\nu_{ij}\). Figure 7 shows a schematic pattern for estimating the related parameters above, and all data have been summarized in Tables 2 and 3.

![Figure 7. The schematic pattern for calculation of parameters](image)

(a) Deformation modulus (b) Poisson’s ratio

Table 2: Modulus from deformation tests

| Modulus | \(E_{xx}/\text{GPa}\) | \(E_{yy}/\text{GPa}\) | \(E_{zz}/\text{GPa}\) | \(E_{xy}/\text{GPa}\) | \(E_{xz}/\text{GPa}\) | \(E_{yz}/\text{GPa}\) |
|---------|-------------------|-------------------|-------------------|-------------------|-------------------|-------------------|
| Confining | 0.2 | 4 | 0.2 | 4 | 0.2 | 4 | 0.2 | 4 | 0.2 | 4 | 0.2 | 4 |
| NO. | | | | | | | | | | | | |
| S1 | 2.07 | -- | 3.35 | -- | 1.68 | -- | 4.67 | -- | 8.93 | -- | 10.77 | -- |
| S2 | 0.35 | 0.84 | 0.48 | 4.07 | 1.42 | -- | 1.24 | -- | 2.37 | -- | 11.44 | -- |
| S3 | 2.09 | 7.31 | 5.88 | 25.47 | 5.76 | 21.93 | 8.93 | -- | -- | 15.44 | 42.09 |
| S4 | 1.58 | 4.83 | 2.03 | 13.36 | 1.49 | 26.42 | 2.34 | 26.17 | 3.71 | -- | 8.39 | 46.30 |
| S5 | 1.63 | 4.13 | 2.87 | 14.18 | 1.36 | 16.30 | 2.76 | 31.08 | 5.10 | 37.30 | 7.46 | 45.89 |
| S6 | 4.13 | 7.62 | 2.82 | 8.97 | 4.17 | 30.39 | 8.55 | -- | 6.07 | 37.50 | 17.70 | 55.14 |
| Stats AVG | 1.98 | 4.95 | 2.91 | 13.21 | 2.65 | 23.76 | 4.75 | 28.63 | 5.24 | 37.40 | 11.87 | 47.36 |

Table 3: Poisson’s ratio from deformation tests

| Poisson’s ratio \(\nu_{ij}\) | \(\nu_{xy}\) | \(\nu_{xz}\) | \(\nu_{yz}\) | \(\nu_{xy}\) | \(\nu_{yz}\) | \(\nu_{xz}\) |
|-------------------|--------|--------|--------|--------|--------|--------|
| Confining | 0.2 | 4 | 0.2 | 4 | 0.2 | 4 | 0.2 | 4 | 0.2 | 4 | 0.2 | 4 |
| NO. | | | | | | | | | | | | |
| S1 | 0.19 | -- | 0.27 | -- | 0.14 | -- | 2.52 | -- | -- | -- | -- | 0.27 |
| S2 | 0.92 | 0.44 | 0.20 | 0.05 | 0.24 | 0.06 | 2.29 | 0.74 | 0.22 | 0.35 | 0.34 | -- |
| S3 | 0.04 | 0.10 | 0.13 | 0.08 | 0.76 | 0.10 | 2.97 | 0.46 | -- | -- | -- | -- |
| S4 | 0.91 | 0.42 | 0.20 | 0.19 | 0.27 | 0.19 | 2.32 | 0.68 | -- | -- | 0.76 | -- |
| S5 | 0.48 | 0.30 | 0.10 | 0.31 | 0.37 | -- | 2.47 | 0.84 | 3.16 | -- | 0.57 | -- |
Referring to the average values of modulus in the three directions (Table 2), both confining states have shown anisotropy such that the modulus that is parallel to the layers is greater than the one perpendicular to the layers. The confining stress would close the cracks in the rock mass, so that the modulus could increase with the increasing confining stress. The modulus increased obviously in the three directions, yet the effect of confinement is different perpendicular and parallel to the layers. Comparing the average of $E_{dy}$ and $E_{dx}$ in two confining states, the increase rate of $E_{dy}$ (i.e., the average $E_{dy}$ in the higher confining state divided by the average $E_{dy}$ in the lower confined state) is 2.5 and the increase rate of $E_{dx}$ (i.e., the average $E_{dx}$ in the higher confined state divided by the average $E_{dx}$ in the lower confined state) is 4.54.

### 6.4 Lateral deformation of layered rock

Referring to Figure 5, the lateral deformation induced by the normal load is quite different between loading parallel to the layers and loading perpendicular to the layers. As shown in Figure 5b, the lateral strain is greater than the normal strain when loading is parallel to the layers, and the calculated Poisson ratio is greater than 1. In general, Poisson’s ratio of most intact and homogeneous solid material is smaller than 0.5. Hence, when loading on the thin-layered rock mass is parallel to the layers, the layer planes would probably open to result in a large lateral deformation. Poisson’s ratio calculated in this paper just represents the effect of lateral deformation in the jointed rock mass. The lateral deformation decreases with the increase in confining stress as well as with Poisson’s ratio.

### 7. Failure under unloading

As previously mentioned, the failure tests under unloading conditions were carried out after the deformation tests. Initially, there was an increase in axial stress in the $Z$ direction to a certain value that is lower than the strength under 8 MPa confined pressure, then the stress was kept constant in the $Z$ and $X$ directions, and then the confining pressure in the $Y$ direction (i.e., perpendicular to the layers) was gradually released from 8 MPa. The unloading paths were designed to simulate the stress changes of the surrounded rock mass during excavating.

#### 7.1 Failure mode

A general failure mode of the six in situ samples is shown in Figure 8. During the unloading process perpendicular to the layers, the layer planes separated gradually from the edge to the center (Figure 8), and part of the planes slide and separate from the sample, then cracked under the effect of bending and compression forces. The failure mode is similar to the failure of underground thin-layered strata of a wall rock mass.
7.2 Strength parameters

The stress–strain curve of S5 during the unloading process is shown in Figure 9 as an example. The $\sigma_z$ remains constant as the maximum principal $\sigma_1$, and $\sigma_z$ decreases as the minimum principal $\sigma_3$ to the failure state. The strength parameters in terms of $\sigma_1$ and $\sigma_3$ are summarized in Table 4.

Figure 8. The failure mode of in situ samples under unloading condition

Figure 9. Stress–strain curves of S5 under the unloading process

| Sample | $\sigma_1$ (MPa) | $\sigma_3$ (MPa) |
|--------|-----------------|-----------------|
| S1     | 48.88           | 5.73            |
| S2     | 35.38           | 1.3             |
| S3     | 45.67           | 2.79            |
| S4     | 47.6            | 5.54            |
| S5     | 49.49           | 4.85            |
| S6     | 55.83           | 7.66            |

Table 4: Strength parameters from triaxial tests

The conventional Mohr–Coulomb criterion is used for characterizing the rock strength, and the criterion can be linearly fitted in terms of $\sigma_1$ and $\sigma_3$:

$$\sigma_i = F\sigma_3 + R,$$

where $R$ is equal to the uniaxial compressive strength (actually the uniaxial compressive strength is always smaller than $R$), and $F$ is the influence coefficient of the effect of confining pressures on the
triaxial compressive strength. The relationships among the cohesion $C$, friction factor $f$ and $F, R$ can be expressed as follows:

$$
\begin{align*}
    f &= \frac{F - 1}{2\sqrt{F}} \\
    C &= \frac{R}{2\sqrt{F}}
\end{align*}
$$

The strength data in Table 4 are linearly fitted as shown in Figure 10, the parameter $F$ is 2.76 and parameter $R$ is 34.32 MPa, and so the calculated cohesion is 10.33 MPa and friction angle $\phi$ is 27.9°.

![Figure 10. The fitting curve between $\sigma_1$ and $\sigma_3$](image)

8. Conclusions

Transverse isotropy is one of the most distinctive features of layered rock, and many previous researchers have studied the layered rock structure using a mechanical laboratory test; however, very little work has been done based on large-scale mechanical in situ tests. In this paper, we studied typical thin-layered strata that are composed of dolomite marble embedded with phyllite in the Wudongde hydropower station of China. According to the deformation and failure behavior of the thin-layered rock mass during excavation, a set of large-scale cubic in situ samples were prepared.

Laboratory and in situ mechanical tests show that the mechanical anisotropy is distinctly reflected by the thin-layered rock masses in the Wudongde hydropower station. The deformation properties are directly related to the loading direction, and it is more deformable when the load is applied perpendicular to the layers. Because of the weakness of the layered discontinuities, it also shows larger lateral deformation when loading is parallel to the layers. Our results show that the modulus increased with the increased confining pressure, and the increased degree is higher when loading is parallel to the layers.

Compared with the results from mechanical tests in the labs, the modulus and strength parameters that are obtained from the in situ tests are smaller. The cohesion is about 10.33 MPa, and the internal friction angle is about 27.9°, which are calculated by the conventional Mohr–Coulomb criterion using the unloading strength parameters. The failure modes of all the in situ samples by unloading tests are nearly the same, abstracted as layered discontinuities opening and layer planes sliding. It is believed that this work will be very helpful to researchers with similar interests and will be a valuable reference for
further research on the mechanics of isotropic rock mass.

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