The effect of external water pressure on the liner behavior of large cross-section tunnels

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1. Introduction

Groundwater is one of the main challenges associated with stability and safety issues in the construction of mountain tunnels. Groundwater assessment and control during both construction and operation of the mountain tunnels are typically the biggest problems faced by the designer and contractors. In mature karst formations, the risk of tunnel construction is much higher due to interference with ground cavities that are either empty, auriferous or filled with erodible materials (Casagrande et al., 2005; Parise et al., 2008; Waele et al., 2011; Alija et al., 2013; Gao et al., 2014). However, some mountain tunnels have to be constructed in mature karstic systems as they are widely distributed in areas such as the southwest China provinces of Chongqing, Yunnan, Guizhou, Sichuan and Hubei, and many long karst tunnels have been constructed successfully in China.

In general, there are three methods for dealing with groundwater: full sealing, full drainage and blocking with limited drainage. Full sealing is used in tunnels with small cover depth or low groundwater level where the liner takes on the entire groundwater pressure. Blocking with limited drainage is used in mountain tunnels with high groundwater levels, like the Yuanliangshan railway tunnel (Cheng et al., 2014), and underwater tunnels with difficulties in drainage, like the Jiaozhouwan undersea tunnel in Qingdao (Qiu et al., 2014) and the Xiang-an undersea tunnel in Xiamen (Zhang et al., 2014), China. The contractor must always be focused on stability and safety during the excavation (Zhang et al., 1993; Li et al., 2013; Li and Li, 2014). Many grouting and consolidating techniques have been applied successfully in the treatment to decrease the pressure and inflow of groundwater (Tseng et al., 2001; Aksoy, 2008; Zhang et al., 2014; Yesilnacar, 2003). The liner of the tunnel takes on reduced groundwater pressure which is determined by the seepage parameters of the grouting annulus (Dahlo et al., 2003). Based on the estimation of water inflow and reasonable drainage scheme (Li et al., 2009; Hwang and Lu, 2007; Wang et al., 2008), full drainage is generally used whether the grouting consolidation is carried out or not (Yang et al., 2016). The liner with enough drainage capacity is designed for most mountain tunnels while the external water pressure on the structure is neglected (Fu et al., 2007).
In recent years, extreme weather with heavy rainfall has become frequent and has brought new challenges to karst tunnel operation that were not considered in the design. In this case a large amount of rainwater can flow toward the tunnel in a short time from the ground surface through the interconnected passages in the karst formation. If the groundwater flowing to the tunnel exceeds the drainage capacity, it will accumulate behind the liner and cause high external water pressure which can lead to the failure of the liner as shown in Fig. 1.

The Shuangbei twin tunnels, constructed in 2014 and each with length of 4373 m and three lanes, carries an expressway in Chongqing City, China. They are NATM tunnels. The reports of site investigation show that the lithology of the surrounding rocks along the tunnel includes mainly muddy limestone and sandstone. In the limestone formation, karst can easily develop along the tunnel. The length of the karstic zone is about 2127 m, which accounts for nearly half of the tunnel length. The cover depth of the tunnel in the karstic zone ranges from 150 m to 200 m. Groundwater is also widely distributed and has a complex relationship to the ground surface. The evaluated classes of surrounding rocks along the tunnel route range from Grade III–V according to the Chinese Code for Design of Road Tunnel (JTG D70-2004). The initial ground stress was also reported to be comprised of primarily gravity stress; tectonic stress was not apparent. During the construction, grouting with a cement and sodium silicate solution was used to block the groundwater and to fill cavities to ensure the stability and safety of the tunnel liner. The drainage system was designed to drain out the residual groundwater, completely overlooking the external water pressure on the liner. Typical cross sections of the liner for Grade IV are shown in Fig. 2.

During tunnel operation, new interconnected passages in the ground or cavities behind the liner might have formed considering the erosion of the strata due to water. In occasional cases such as extreme heavy rain on the mountain area above the tunnel, high external water pressure develops behind the liner if the groundwater flowing towards the tunnel is beyond the capacity of the drainage system, a case generally not encountered during construction. It is of interest to assess how much external water pressure is allowable and to identify zones of weakness in the liner cross section when cavities are present. This paper describes the development of a new method of applying external water pressure to model tunnels. The method was employed to model tests to characterize the behavior of the liner for the Shuangbei tunnel. The study concentrated on the loads on the liner and ignored the process of excavation in the ground with initial stress. This method can also be used to investigate the behavior of tunnels with waterproofing liner.

Fig. 1. Liner failure under high external water pressure in Chongqing, China (a) failure at the side wall, (b) failure at the knee.

Fig. 2. Cross section of Shuangbei tunnel (Grade IV, unit: cm).
2. Material preparation

2.1. Scale model similitude

The relationship between a prototype and the corresponding scale model behavior can be represented by theories of scale model similitude. Rocha (1957) described systematic scale modeling for problems in soil mechanics and proposed that the soil constitutive behavior be scaled. He assumed that both the stress and strain held a linear relationship between the model and prototype in a 1 g gravitation field. Moncarz and Krawinkler (1981) considered that the stress and strain affected. Kana et al. (1986) described the application of the Buckingham Pi theorem to scale modeling the dynamic interaction of a pile in clay. Gohl (1991) also used dimensional analysis to derive the functional relationship for scale model similitude as applied to shaking table tests of model piles. In the case of the interaction of the ground and tunnel, the key physical parameters include stress \( \sigma \), strain \( \varepsilon \), elastic modulus \( E \), Poisson ratio \( \mu \), internal friction angle \( \varphi \), cohesion \( c \), gravity \( g \), geometry size \( L \), displacement \( \delta \) and the pressure on the liner \( p \). The solution for these physical quantities of interest can be denoted as

\[
f(\sigma, \varepsilon, E, \mu, \varphi, c, L, \delta, p) = 0 \tag{1}
\]

where gravity \( g \) and geometry size \( L \) are assumed to be the fundamental measures. According to dimensional analysis using the Buckingham Pi theorem, Eq. (1) can be expressed in the form

\[
G(\pi_1, \pi_2, \pi_3, \pi_4, \pi_5, \pi_6, \pi_7, \pi_8) = 0 \tag{2}
\]

where the dimensionless parameters \( \pi_1 = \sigma / gL \), \( \pi_2 = \varepsilon \), \( \pi_3 = E / gL \), \( \pi_4 = \mu \), \( \pi_5 = \varphi \), \( \pi_6 = c / \sqrt{L} \), \( \pi_7 = \delta / L \), \( \pi_8 = p / gL \). In this study, the tests were carried out in a 1 g gravitation field. The similarity ratios of geometry size and gravity were \( C_g = 30 \) and \( C_L = 1 \). According to Eqs. (1) and (2), the similarity ratios of other physical parameters are \( C_\mu = C_\varepsilon = C_\varphi = C_c = C_E = C_p = 30 \), where \( C_\mu \), \( C_\varepsilon \), \( C_\varphi \), \( C_c \), \( C_E \) and \( C_p \) represent the similarity ratios of prototype to model \( \mu, \varepsilon, \varphi, E, \sigma, c \) and \( p \), respectively.

2.2. Model soil preparation

The prototypical surrounding rocks were classified as Grade IV. The mechanical parameters suggested by the Code for Design of Road Tunnel (JTG D70-2004), China are shown in Table 1. According to the similarity ratios, the mechanical parameters of the model soil in the test can be calculated. Hobbs (1966, 1969) and Jeon et al. (2004) proposed a mixture of sand, plaster and water for the scaled model testing when the surrounding rocks have good integrity. Stimpson (1970) also inquired into the properties of granular cemented with different materials such as plaster, clay, cement, oil and rosin. In this paper, the surrounding rocks were simulated by a mixture of barite, river sand, quartz sand, coal fly-ash, viscous oil and rosin with proportions shown in Table 2. The corresponding mechanical parameters of the model soil are shown in Table 1.

2.3. Primary support and secondary liner

The primary support in the prototype was mainly made up of C25 shotcrete and lattice girders. A mixture of plaster, water and diatomite with mix proportion 1:1.2:0.2 was adopted to simulate the C25 shotcrete as its final elastic modulus met the similarity ratio \( C_E \) (see Table 3). The lattice girders were modeled using the iron rods with 2.337 mm diameter and different spacing for three lanes and a parking strip. It was very difficult to find a material that exactly met the similarity relationship of the elastic modulus (E) and dimensions (L) at the same time. Considering that the bending

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**Table 1**

<table>
<thead>
<tr>
<th>Prototype</th>
<th>Model</th>
<th>Similarity ratio</th>
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<tbody>
<tr>
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**Table 2**

<table>
<thead>
<tr>
<th>Materials</th>
<th>River sand</th>
<th>Quartz sand</th>
<th>Coal flyash</th>
<th>Viscous oil</th>
<th>Rosin</th>
<th>Barite</th>
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<tr>
<td>Quality ratio</td>
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<td>1.0</td>
<td>0.75</td>
<td>0.275</td>
<td>0.15</td>
<td>0.25</td>
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**Table 3**

<table>
<thead>
<tr>
<th>Item</th>
<th>Prototype</th>
<th>Model</th>
<th>Similarity ratio</th>
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<td>Shotcrete Material</td>
<td>Concrete,C25</td>
<td>Mixture of plaster</td>
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<td>Compressive strength [MPa]</td>
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<td>0.8</td>
<td></td>
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<tr>
<td>Thickness [cm]</td>
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<tr>
<td></td>
<td>CS2: 28</td>
<td>0.93</td>
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<tr>
<td>Lattice girder Material</td>
<td>3 lanes: 118</td>
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<td></td>
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<tr>
<td></td>
<td>CS2: 122a</td>
<td></td>
<td></td>
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<tr>
<td>Spacing [cm]</td>
<td>80</td>
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<tr>
<td></td>
<td>CS1: 4.89</td>
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<td></td>
<td>CS2: 2.64</td>
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<tr>
<td>E [GPa]</td>
<td>210</td>
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<tr>
<td></td>
<td>1660 for 118</td>
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<td>3400 for 122a</td>
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<td>Concrete Material</td>
<td>Concrete,C30</td>
<td>Mixture of plaster</td>
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<td>Compressive strength [MPa]</td>
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<td>0.75</td>
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<td>Thickness [cm]</td>
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<td></td>
<td>CS2: 90</td>
<td>3.00</td>
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<tr>
<td>Reinforced bar Material</td>
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<tr>
<td>Compressive strength [MPa]</td>
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<td></td>
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</tr>
<tr>
<td></td>
<td>CS1: 225 steel bar</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>CS2: 28 steel bar</td>
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<td></td>
</tr>
<tr>
<td>Spacing [cm]</td>
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capacity was the key factor affecting the behavior of lattice girders, 
the product of the elastic modulus and moment of inertia (EI) was 
met via the similarity ratio (CEI), although CE and CI were not met 
independently.

In practice, the secondary liner is cast with C30 concrete that 
has an elastic modulus = 31 GPa and compressive strength = 22.5 
MPa. The concrete was reinforced with 25 mm- and 28 mm-
diameter steel rebar at a vertical and horizontal spacing of 
200 mm. A mixture of water, plaster and diatomite with a mix pro-
portion of 1.9:1.0:0.4 was used to model the C30 concrete. The 
elastic modulus and compressive strength of the mixture were 
1.1 GPa and 0.75 MPa, respectively. No. 23 (0.61 mm diameter) 
and No. 22 (0.71 mm diameter) iron rod with intervals of 
3.3 cm and 3.364 cm separately were adopted to model the rein-
forced liner CS1 and CS2. Hence in the modeling, the compressive 
strength of the mixture and the $E_A$ of the reinforced iron rod met 
the similitude relationship to the prototype as shown in Table 3.

3. Testing program

3.1. Loading frame and measurement

The model tests were carried out on a loading frame as shown 
in Fig. 3. The frame consisted of the test soil box, foundation, lateral 
cylinders, vertical cylinders, cover plate, reaction frame and control 
system. The dimensions of the soil box are 3.6 m length, 3.6 m 
width and 0.3 m thickness. The box was filled with the model soil 
from the location where the tunnel was to be excavated and 
supported with composite liner. There are 8 lateral cylinders on 
the 4 sides of the testing soil box used to form the transverse stress 
field and apply the transverse loads on the liner. The total stress 
applied in each of the horizontal directions was controlled sepa-
rately to allow anisotropic loading. Four vertical cylinders on the 
cover plate were used to maintain the plain strain conditions in 
the test soil. The surfaces of the foundation and cover plate were 
lubricated to decrease the influence of friction.

Twelve pairs of strain gauges were installed on the inner and 
outer surface of the liner as shown in Fig. 4. According to the tested 
strains of the inner surface $e_{in}$ and outer surface $e_{out}$, the internal 
forces of the liner can be calculated by the following equations:

$$N = \frac{1}{2} E(e_{in} + e_{out})bh$$  
$$M = \frac{1}{12} E(e_{in} - e_{out})bh^2$$

where $N$ and $M$ are the thrust force and bending moment, respec-
tively, $b$ is the width of the section (assumed to be a unit length), 
h is the thickness of the liner and $E$ is the elastic modulus. Based 
on the similarity relationship, the thrust force and bending moment 
of the prototype can be calculated correspondingly.

3.2. Simulation of water pressure

External water pressure was simulated by applying the equiva-
 lent hydrostatic load on the liner. Feng et al. (2013) applied the 
equivalent water pressure on the liner of a circular tunnel by ten-
sioning the hoops on the liner’s outer surface. However, the applied
Fig. 4. Layout of stain gauges.

Fig. 5. Apparatus used to simulate external water pressure on model liner.
pressure is related to the arc radius of the hoop. In this case of a non-circular tunnel, this method is not suitable as the radius is not constant. The hoop-tensioning method will result in stress concentration and additional shear force beneath the hoops. In addition, the tension force in the hoops is greatly affected by the deformation of the liner. Consequently, it is hard to maintain steady water pressure in the model test.

In this study, a device was invented to simulate the water pressure on a non-circular tunnel, as shown in Fig. 5. When the inner space of the tunnel is evacuated, the remaining air pressure outside the liner acts on the outer surface like water pressure. Hence, the pressure obtained by this device has no relationship to the shape of the tunnel, and the pressure is more uniform with no stress concentration or additional shear force. The device was comprised of sealing units, an air exhauster, a pressure monitor and a voltage stabilizer. The sealing units include the upper and lower cover plates, pillars and sealing rubber rings. The cover plates were made of Plexiglas with the same shape and dimensions as the tunnel section. The pillars link the upper and lower plates together and are slightly higher than the tunnel. When the inner space is evacuated, the pillars prevent the liner from being affected by the pressure on the cover plates. The gaps between the liner and the cover plates were sealed with greased rubber rings to prevent leakage and loss of pressure. Wire joints were plumbed through the upper cover plate so that the strain sensors can be connected to the data acquisition system. When the air inside the liner was evacuated, an even and steady external pressure acting on the liner could be obtained. In the full vacuum condition, the external pressure on the liner is 1 atmosphere, which represents 300 m of water head in the prototype assuming $C_l = 30$.

### 3.3. Test cases

Eight model tests were performed to simulate CS1 and CS2, each with different cavity scenarios against the liner (see Table 4). The diameter of each cavity was 10 cm, equivalent to D/6, where D is the span of the tunnel. The diameter of the cavity is assumed to be 10 cm for each test case, which is about equal to one-sixth of the span distance. Photographs of each CS1 test setup are shown in Fig. 6. For each test, the surrounding material is assumed to be Grade IV rock.

According to the Chinese Code for Design of Road Tunnel (JTG D70-2004), the designed vertical and lateral loads of Grade IV surrounding rocks on the liner can be calculated to be $p = 158.76$ kPa and $q = 47.63$ kPa for section CS1 and $p = 183.78$ kPa and $q = 55.14$ kPa for section CS2. This loading level is for deep tunnels and has little relationship to cover depth. The loading procedure showed in Fig. 7 during testing was as follows:

1. **Step 1:** Load and consolidate the test ground.
2. **Step 2:** Excavate the tunnel (and the cavity when used) in the test ground and finish the primary support.
3. **Step 3:** Place the precasted liner into the excavated ground with plastic film wrapped around the outer surface.
4. **Step 4:** Load and apply the designed vertical and lateral loads gradually.
5. **Step 5:** Apply the external water pressure $u$ using the evacuation device, increasing the pressure by 1 kpa (that is 0.1 m water head or 3 m water head in prototype) step by step until the liner fractures.

<table>
<thead>
<tr>
<th>Liner for CS1</th>
<th>Test no.</th>
<th>Cavity location</th>
<th>Liner for CS2</th>
<th>Test no.</th>
<th>Cavity location</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1-1</td>
<td>None</td>
<td>CS2-1</td>
<td>None</td>
<td>CS1-2</td>
<td>Crown</td>
</tr>
<tr>
<td>CS1-2</td>
<td>Crown</td>
<td>CS2-2</td>
<td>Crown</td>
<td>CS1-3</td>
<td>Side wall</td>
</tr>
<tr>
<td>CS1-3</td>
<td>Side wall</td>
<td>CS2-3</td>
<td>Side wall</td>
<td>CS1-4</td>
<td>Invert</td>
</tr>
<tr>
<td>CS1-4</td>
<td>Invert</td>
<td>CS2-4</td>
<td>Invert</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

![Fig. 6. Test cases for the liner of CS1](image-url)
4. Test results

4.1. Thrust force

The observed thrust force around the liner at various loading stages for each CS1 and CS2 test is presented in Figs. 8 and 9. Beginning with CS1-1 (no cavity outside the liner), the application of the design load without external pore pressure \( (p = 159 \text{ kPa}, q = 47.6 \text{ kPa}, u = 0) \) results in the maximum thrust force \( (T_{\text{max}} = 639 \text{ kN}) \) occurring beneath the invert and otherwise gradually decreasing around the liner. The minimum thrust force \( (T_{\text{min}} = 180 \text{ kN}) \) occurs at the spring line. The observed thrust force is not completely symmetrical in that the right side invert and right side spring line exhibit higher and lower thrust, respectively, than these locations on the left side. This asymmetry is the primary cause of the observed degree of nonuniform thrust \( (i.e., T_{\text{max}} / T_{\text{min}} = 3.5) \). Some deviation from symmetry is to be expected in complex large-scale experimental testing.

As the air inside the model tunnel is evacuated maintaining \( p = 159 \text{ kPa} \) and \( q = 47.6 \text{ kPa} \), the external air pressure begins to act on the liner like external water pressure. Consequently, the thrust force increases significantly. When the external water pressure increases to 150 kPa \( (i.e., 5 \text{ kPa in the model}) \), \( T_{\text{max}} \) and \( T_{\text{min}} \) increases to 1590 kN and 567 kN, respectively. In comparison to the externally applied geostatic vertical and lateral loads that have a similar magnitude, the increment of thrust force increase due to 150 kPa external pore pressure is significant, particularly in that \( T_{\text{min}} \) grew by a factor of nearly four. The location of \( T_{\text{max}} \) also shifts to the invert corners where the radius of curvature is the smallest. The degree of asymmetry is also improved. \( T_{\text{max}} / T_{\text{min}} \) decreased to 2.8, implying that the nonuniformity of the thrust force was slightly relieved. Hence, a little external water pressure contributes to reduce the nonuniformity of the thrust force in the liner.

Nevertheless, as the external water pressure increases to 240 kPa \( (8 \text{ kPa in the model}) \), \( T_{\text{max}} / T_{\text{min}} \) increases again to 3.51. The results of test CS1-1 also shows that each monitoring point has a different increasing rate of thrust force with respect to the external water pressure. The thrust force of point #8 (the knee of the liner) has the biggest increasing rate, while the thrust force at point #6 (between the spring line and the shoulder) has the smallest one. These different increasing rates strengthen the nonuniformity of the thrust force especially when the external water pressure is high. When the water pressure was greater than 90 kPa, the maximum thrust forces always occurred at the knees of the liner while the minimum always occurred between the right shoulder and the spring line.

In the liner at the parking strip (CS2), the designed vertical and lateral loads are \( p = 183.78 \text{ kPa} \) and \( q = 55.14 \text{ kPa} \). When there is no cavity behind the liner (Test CS2-1), the \( T_{\text{max}} \) and \( T_{\text{min}} \) is 806.4 kN.
Fig. 8. Thrust force distribution in the liner (kN).
Fig. 9. Thrust force of the liner with respect to water pressure [kN].
and 389.3 kN separately which occurs near the right invert and left shoulder when the designed loads applied. The thrust force of each point increases with the external water pressure at different rates. The thrust force of point #8 (the knee of the liner) has the greatest increasing rate, while the thrust force of point #5 (the shoulder of the liner) has the smallest one. As the water pressure increases to 150 kPa (5 kPa in the model), the $T_{\text{max}}/T_{\text{min}}$ increases slightly to 2.09. While the water pressure increases to 240 kPa and 360 kPa, the $T_{\text{max}}/T_{\text{min}}$ is 2.56 and 2.83 respectively. Hence, the higher external water pressure makes the thrust force of the liner more nonuniform. It is also shown that when the water pressure is bigger than 180 kPa, the maximum and minimum thrust forces always occurs at the right knee and right shoulder of the liner.

Cavities behind the liner change the thrust force distribution greatly. If the cavity is located above the crown (Test CS1-2), the thrust force near the crown decreases dramatically and the minimum thrust force 42.1 kN occurs on the left shoulder. If the cavity is located behind the side wall (Test CS1-3), the thrust force has the maximum 960.9 kN existing on the right knee. The left side has much lower thrust force than the right side except the spring line, where the thrust force increases minimally. Compared with the case of no cavity behind the liner, $T_{\text{max}}/T_{\text{min}}$ has the maximum value 11.1 when the cavity is located above the crown and the minimum value 2.31 when the cavity is located under the invert (Test CS1-4). The thrust force at each point increases with the external water pressure at a different rate for both CS1 and CS2. When the cavity is located above the crown (Test CS1-2 and CS2-2), the shoulders (point #3 or point #5) and the crown (point #4) have the minimum thrust force and increasing rate while the invert (point #10) has the maximum increasing rate. When the cavity is located behind the side wall (Test CS1-3 and CS2-3), the thrust force and its increasing rate at the left knee (point #12) are apparently bigger than those at the other points. When the cavity is located under the invert (Test CS1-4 and CS2-4), the thrust force and its increasing rate at the invert (point #10) drop significantly. It is evident that the cavity changes the thrust force of the liner nearby.

The maximum and the minimum $T_{\text{max}}/T_{\text{min}}$ of all the test cases are shown in Table 5. It can be concluded that the external water pressure increases, the thrust forces increase, and the difference between the maximum and minimum thrust forces also increases.

### Table 5

<table>
<thead>
<tr>
<th>Liner type</th>
<th>Test no.</th>
<th>Cavity location</th>
<th>Maximum $T_{\text{max}}/T_{\text{min}}$ (water pressure [kPa])</th>
<th>Minimum $T_{\text{max}}/T_{\text{min}}$ (water pressure [kPa])</th>
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<tr>
<td>CS1</td>
<td>CS1-1</td>
<td>None</td>
<td>4.43 (0)</td>
<td>2.68 (270)</td>
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<td></td>
<td>CS1-2</td>
<td>Crown</td>
<td>12.00 (30)</td>
<td>2.19 (240)</td>
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<tr>
<td></td>
<td>CS1-3</td>
<td>Side wall</td>
<td>4.43 (0)</td>
<td>2.68 (270)</td>
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<tr>
<td></td>
<td>CS1-4</td>
<td>Invert</td>
<td>3.8 (0)</td>
<td>1.83 (180)</td>
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<tr>
<td>CS2</td>
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<td>None</td>
<td>2.82 (360)</td>
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<td>CS2-2</td>
<td>Crown</td>
<td>12.26 (0)</td>
<td>3.00 (360)</td>
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<td>CS2-3</td>
<td>Side wall</td>
<td>3.07 (390)</td>
<td>2.61 (0)</td>
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<td></td>
<td>CS2-4</td>
<td>Invert</td>
<td>5.44 (0)</td>
<td>1.79 (330)</td>
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</table>

**Fig. 10.** Bending moment distribution in the liner (kN·m).
pressure helps to decrease the nonuniformity of the thrust force. If the cavity is located above the crown, the liner has the maximum nonuniformity of the thrust force when the water pressure is zero or very low.

4.2. Bending moment

Figs. 10 and 11 illustrate the bending moment of the liner as a function of geostatic and external water pressure for all CS1 and CS2.
Fig. 12. Eccentricity of the liner under external water pressure.
CS2 test cases. All plots show positive bending moment (i.e., bending inwards) at the crown and the middle of invert and negative bending moment (i.e., bending outwards) at the spring line and knees, with 0 bending moment near the shoulder and the fringe of the invert.

When there is no cavity behind the liner for CS1 (Test CS1-1), as the external water pressure increases from 0 kPa to 150 kPa and 240 kPa, the maximum positive bending moment increases from 281.8 kN·m to 380 kN·m and 455.4 kN·m and the maximum negative bending moment increases from –313.3 kN·m to –768.7 kN·m and –1010.3 kN·m. With the increasing of external water pressure, the positive bending moment at the crown and invert grows slowly while the negative bending moment at the spring line and the knee grows rapidly. The bending moment distribution in the liner of CS2 (Test CS2-1) is similar to that of CS1. The maximum positive bending moment occurred at the crown or invert while the maximum negative bending moment always appears at the knee where the bending moment is much higher than that near the spring line. As the external water pressure increases from 0 kPa to 150 kPa and 360 kPa, the maximum positive bending moment increases from 356.4 kN·m to 413.1 kN·m and 548.1 kN·m while the maximum negative bending moment increases from –514.3 kN·m to –954.4 kN·m and –1815.9 kN·m. The bending moment shows small increases in the crown and invert and bigger increases at the spring line and knee with the increasing external water pressure. As for the liner of both CS1 and CS2, the bending moment distributions show some symmetry and stability with the increasing of external water pressure.

When the cavity is located above the crown, as the contact force between the ground and liner disappears in the cavity, the range of the positive bending moment decreases (Test CS1-2) or shrinks greatly (Test CS2-2) at the crown under zero external water pressure. With the increase of water pressure, the value and range of positive bending moment at the crown expand, and the maximum negative bending moment always appears at the knees and increases with the water pressure greatly as well. When the cavity is outside of the left side wall (Test CS1-3 and CS2-3), the invert usually has the maximum positive bending moment and an increasing rate with respect to water pressure, and the left side wall and knee have the maximum negative bending moment or increasing rate with respect to water pressure. If the cavity is located below the invert (Test CS1-4 and CS2-4), the invert has the maximum positive bending moment and increasing rate with respect to water pressure and the left or right knee have the maximum positive bending moment and increasing rate with respect to water pressure. Hence, in the case of a cavity behind the liner, the maximum negative bending moment generally appears at the knees or the side wall if the cavity is located behind it, while the maximum positive bending moment usually forms at the invert.

### 4.3. Eccentricity

Eccentricity, which is the ratio of bending moment to thrust force, is one of the key factors that determine the capacity of the liner. When there is no cavity behind liner, the eccentricity of the liner of CS1 (Test CS1-1) shows the biggest value at the spring line without external water pressure. When external water pressure changes from 0 kPa to 150 kPa, the eccentricity at the crown, right spring line and left invert reduces significantly. However, as external water pressure increases from 150 kPa to 240 kPa, the eccentricity of the liner changes minimally. The eccentricity of the liner of CS2 (Test CS2-1) has a similar shape to that of CS1; the biggest eccentricity appears at the knee without external water pressure. Hence, when water pressure is applied, the maximum eccentricities decreases dramatically and becomes more even. This means that lower external water pressure helps to reduce the eccentricity while higher external water pressure cannot reduce it further.

The cavity also affects the eccentricity distribution of the liner under external water pressure as shown in Fig. 12. When the cavity is located above the crown (Test CS1-2 and CS2-2), maximum eccentricities are located at the crown, the left and right knee and the invert, and reduce due to the external water pressure. When the cavity is outside of the left side wall, the liner for both CS1 and CS2 (Test CS1-3 and CS2-3) has the maximum eccentricity near the cavity. If the cavity is located below the invert, the maximum eccentricity occurs at the invert (Test CS1-4) or at the knee (Test CS2-4), which is even bigger than that without external water pressure. It can be concluded that the cavity might reduce the bearing capacity of the liner nearby although the water pressure helps to reduce the nonuniformity of the eccentricity.

### 4.4. Fracturing of the liner

The fracturing of liner can be recorded by observing the air tightness of the sealed liner. In the test, when the air pressure in the tunnel space is 0.2 kPa higher or lower than the target value, the air exhauster starts or stops working. Hence, the interval period of air exhauster working represents the air tightness of the sealing units. The leakage velocity of the sealed liner is affected not only by the air pressure difference between the inner and outer space of the tunnel but also by the fractures of the liner. Generally, when permeable fractures form, the air leakage increases and the air exhauster’s interval period decreases significantly. As the air pressure difference increases and the fractures grow (i.e., increasing of numbers, depth, width and length), the air exhauster’s interval period becomes shorter and shorter until it cannot maintain the expected air pressure difference even if it works continuously. At that point, the testing stopped. The recorded interval period of the working of the air exhauster for each testing case is shown in Fig. 13. It can be seen that there is an apparent drop when the permeable fractures formed. In CS1, the critical external water pressure is 330 kPa (Test CS1-1), 300 kPa (Test CS1-2 and CS1-4) and 240 kPa (Test CS1-3). The final applied maximum pressure is 450 kPa, 390 kPa, 360 kPa and 420 kPa for Test CS1-1 to CS1-4 respectively. The results of the air tightness tests for CS2 are similar to those for CS1. It can be concluded that the cavities behind the liner decrease the bearing capacity under external water pressure.

The features and sequence of the visible fractures were recorded as shown in Fig. 14 although the initial micro fractures in the liner could not be identified in the testing. Under external water pressure, the fractures develop first at the knees of the liner,
Fig. 14. Fracturing of the liner under external water pressure.
then at the invert and lastly at the crown. At the knees, the fractures are located on the external face of the liner where there is negative bending moment (i.e., bending outwards). This implies that visual inspection would not have been able to detect these cracks forming. At the invert and crown, the fractures are located on the inner face where there is positive bending moment (i.e., bending inwards).

In case of no cavity behind the liner (Test CS1-1 and CS2-1), two longitudinal through fractures form at both the left and the right knees. The first one is quite deep: 1.8 cm for CS1 and 2.1 cm for CS2, which accounts for about 80% (CS1) and 70% (CS2) of the thickness of the liner. The second one forms later and is not as deep. One longitudinal through fracture with an average depth of 1 cm, which accounts for about 40% of the liner thickness, forms on the inner face of the invert. One transversal fracture with the length of 9–15 cm also forms in the middle of the invert. One longitudinal through fracture with a smaller depth of 0.6–0.8 cm, which accounts for about 25% of the liner thickness, forms on the inner face of the crown. As for CS2 (Test CS2-1), the last longitudinal through fracture appears on the inner surface of the crown.

When the cavity is located above the crown (Test CS1-2 and CS2-2), the number and depth of the fractures at the knees of the liner decrease. Compared with the case of no cavity, the depth of the fracture increases and a branch fracture appears (Test CS1-2) at the crown and more longitudinal fractures appear at the invert. When the cavity is located outside of the left side wall, the number and depth of the fractures at both the left and right knee increase. The fractures at the invert are also more prevalent compared to those at the crown. If the cavity is located below the invert, the number of longitudinal through fractures increase and some branch fractures appear at the invert as well. Another difference between these cases is that the scale of the transversal fractures enlarges significantly.

5. Conclusion

For most mountain tunnels in karst areas, the liner is designed with enough drainage capacity to ignore the external water pressure on the structure. However, occasional extreme heavy rain events can result in a large amount of groundwater flowing towards the tunnel in a short time, which may exceed the drainage capacity of the liner and cause high external water pressure. It is therefore essential to assess how much external water pressure is allowable and where the weaknesses in the liner are that need to be cared for to ensure the safety of the tunnel. To model this on a laboratory scale, the key challenge is how to exactly simulate the external water pressure distribution. This is particularly challenging when modeling non-circular tunnels. In this study, an apparatus was developed to apply the appropriate external water pressure by evacuating the inner space of the tunnel and applying external air pressure to the liner to act as the water pressure. Compared with the traditional method proposed by Feng et al. (2013), the pressure obtained by this device has no relationship to the shape of the tunnel. From the model tests of the liner behavior for large tunnel cross sections in this study, the following conclusions can be made:

- The application of external water pressure at the tunnel liner results in an increased thrust force and bending moment, while the nonuniformity of the thrust force is reduced.
- The maximum thrust force typically appears at the knee and side walls, while the vault and shoulder exhibit the least thrust force. The maximum negative bending moment (bending outwards) forms at the knees or side walls and maximum positive bending moment (bending inwards) formed at the invert.
- The presence of external water pressure helps to reduce the eccentricity of the liner. However, the eccentricity is not further reduced if the external pressure is increased. The maximum eccentricity generally occurs at the knees or side walls of the tunnel.
- Cavities behind the liner have significant influence on the internal force distribution of the liner, including when external water pressure is present. The presence of a cavity significantly decreases the bearing capacity of the liner and accelerates the formation of fractures nearby. Generally, a cavity located behind the sidewall yields the lowest bearing capacity of the liner.
- As the external water pressure increases, fractures appear first at the knee, followed by the invert and lastly at the vault of the tunnel liner.

The results go to show that it is critical to estimate the bearing capacity of the tunnel liner under external water pressure as the thrust forces, bending moments and eccentricity can significantly change. Using the test apparatus and methodology described in this paper, this simpler and more accurate means of applying external water pressure on a laboratory scale can be applied to both circular and non-circular tunnels to assess the changes that result from external water pressure.

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