Model Test Research on the Mechanical Characteristics of Tunnel Lining Structures under Local High Water Pressure in Concealed Karsts

Meihai Jin¹, Xinrong Liu¹,², Zuliang Zhong¹,²,³*
¹ School of Civil Engineering, Chongqing University, Chongqing, 400045, China.
² National Joint Engineering Research Center of Geohazards Prevention in the Reservoir Areas, Chongqing University, Chongqing, 400045, China.
³ National Engineering Laboratory for Highway Tunnel Construction Technology, Chongqing, 400067, China.
* Corresponding author’s contact: haiou983@cqu.edu.cn
ORCID iD: https://orcid.org/0000-0003-0050-3436

Abstract. Mountainous karst areas contain many hidden karsts around highway tunnels. During the stormy rain season, concealed karsts connected to surface are recharged by surface water. Consequently, the water level of the corresponding caverns rises sharply, posing an imminent threat to the safety of the lining structure of the operating tunnel. This paper uses similar model experiment methods from a highway tunnel in southwestern China to discuss the contact pressure between the lining and surrounding rock, internal lining force, and the change law of the failure mode when the local high water pressure acts on the sidewall. Results showed that at increasing external water pressure, the contact pressure between the lining and surrounding rock in the hydraulic pressure area decreased, whereas the contact pressure at other parts increased. Consequently, the axial force distribution maintained a “horseshoe shape” whose maximum value was proportional to the position of the external water pressure. Moreover, the pressure had a more dramatic influence on the bending moment of the lining that even changed its direction. Furthermore, the right side wall fractured when the water pressure acted on the left side wall, making the left side appear under the load. However, the longitudinal penetrating cracks were the same as the sidewall dislocation during the operation of the tunnel. These findings have specific reference significance for the design of traffic tunnels in karst areas.

Keywords: hidden karst; similar experiment; groundwater; lining structure; failure mode

1. Introduction
Karst landforms are widely distributed in China, accounting for approximately one-third of the country’s land area, and mainly concentrated in southwest China. With the rapid development of transportation infrastructure in China, the number and construction difficulty of tunnel projects are gradually increasing. Many construction disasters have occurred during the construction and operation of tunnels, such as water burst, mud burst, or collapse, which caused significant damage to construction machinery and personnel or operating vehicles[1-6].

Detection space limits a karst tunnel. Although a karst in the front of a tunnel is usually detected during construction, the surrounding hidden cavities are hardly seen. Part of the concealed karst is connected to the surface through deep large karst fissures or corrosion pipelines. During the rainy season or heavy rain, the water level of the cavern rises sharply owing to the replenishment of surface water, which exerts enormous external water pressure on the adjacent cavern walls and lining structures, resulting in cracked or even crushed lining. According to survey statistics in the southwest region where the castells landform is widely distributed, more than half of the tunnels that have been in operation for
more than 10 y are related to hidden karsts\cite{7}. Han (2019) and Guo\cite{8,9} used numerical simulation methods to study the mechanical properties of the Castell tunnel structure under steady water flow. They considered the size and distance between the tunnel, cave, and lining—permeability parameters. The simulation result was compared with a theoretical, analytical solution, and a good agreement was found. Gao, CL et al. (2019)\cite{10} studied the force mechanism of the lining and evolution of water leakage when tunnels with different buried depths were passed through cross faults. They then analyzed the leakage through similar large-scale experiments and numerical simulations. Water volume, vertical displacement, and water pressure on the lining showed that the internal defects of the lining structure and construction quality were the leading causes of water leakage. Moreover, the uneven settlement also had a substantial impact on the cracking of the lining. Zhang et al. (2019)\cite{11} found through field investigation that the primary cause of structural instability was the high water pressure behind the lining. In response to this problem, it was assumed that the water flow rate is stable to determine the water storage volume behind the lining at different periods and convert it into external water pressure. Subsequently, a numerical simulation was conducted to analyze the mechanical response of the lining structure under different water pressures. Wang and Xu\cite{12,13} used a numerical simulation to define various hole shapes around the tunnel lining. They analyzed the characteristics and changes of the internal force of the lining under hoop pressure. To study the rapid pressurization of the lining, Fang et al. (2016)\cite{14} used a device to extract the air inside the tunnel structure. Further, they simulated the effect of external water pressure and achieved almost instantaneous pressurization by quickly removing the internal air. The extraction speed was adjusted according to the specific water pressure growth rate to achieve a natural simulation effect. The experimental results showed that eccentric loading is easy to occur when high water pressure acts on the structure. Conversely, low water pressure reduces the structure’s eccentricity. During the construction of Wuhan Metro Line 6 Qianjincun Station to Maying Road Station, the shield tunnel encountered the problem of the underlying karst cavity when crossing the sand layer. By controlling the size of the cave and tunnel, distance between them, and filling angle under external water pressure, flac3d was used with fluid–solid coupling to conduct a simulation study\cite{15,16}.

The abovementioned studies mainly discuss the influence of uniform external water pressure, lining cavity influence, fluid–solid coupling, and other working conditions on the stability of the tunnel lining structure. However, few studies exist on the failure mechanism of the lining structure under local high water pressure in hidden karsts. Herein, the lining structure of the Tongxi Tunnel in southwestern China is cracked through external water pressure during operation. A model test method is proposed to study the impact of the local external water pressure on the tunnel vault or sidewall, mechanical characteristics, and failure modes of the tunnel lining structure to provide a reference for the design of tunnel structure in karst areas.

2. Project overview and site investigation of lining disasters

2.1. Engineering geology
The Tongxi Tunnel is 2080-m long with complicated engineering geological conditions located in the Tonggu Township of Youyang County, Chongqing City. The tunnel site is the middle mountain landform of the dissolution peak cluster depression. The fractures in the area are relatively developed, i.e., generally tensile fractures, with an opening width ranging from 1 to 35 cm. However, no other folds and fracture structures are observed, and the geological structure is simple. According to a geological engineering survey, the stratum in the tunnel site is noncomplex, mainly limestone. The starting section of the tunnel is shale. As shown in Fig. 1, the tunnel’s limestone primarily comprises thin to medium-thick layered limestone and dolomitic limestone. Cracks and karsts develop on the surface of the mountain through which the tunnel passes. Abundant karst water and complete underground pipelines are present at the deeper part of the mountain. Further, hydrogeological diseases such as water inrush and mud outbursts existing in the tunnel site have highly unfavorable effects on the tunnel.
2.2. Hydrogeology
Karsts generally develop in the tunnel site area where there is abundant groundwater. The karst depressions form at the top west area of the mountains. The planar shape is a long strip, with a bottom width of 40–70 m, a depth of 20–75 m, and a length of more than 5,000 m. The tunnel site is adjacent to the right bank of the middle and upper reaches of the Tongxi River with an undeveloped surface water system. The rainy season ranges from April to October, accounting for 83% of the annual rainfall. During the rain, the dry ditch surface gully becomes flooded, characterized by rapid rise and fall, thereby affecting the water stability of the tunnel in operation. Conversely, the water pressure is low during the dry season at normal tunnel operation. In times of water surge, the surface water flows downward through the cracks in the rock mass, and the water pressure acts on the local structure of the lining, leading to multiple hydraulic-induced dislocations, cracks, and water spray disasters in the tunnel.

2.3. Investigation on the cracks of tunnel lining structure
The Youyang Expressway Tongxi Tunnel site area developed karsts during heavy downpour in the rainy season. Consequently, the tunnel’s secondary lining cracked through the impact of the external water pressure, which necessitated the cracks to be closed and repaired. After opening the tunnel in the middle and late June 2016, a heavy rainstorm occurred at the site of the Youyang Project resulting in many longitudinal cracks. Moreover, an interesting water leakage phenomenon was discovered in the tunnel. Field investigation and testing showed a large number of longitudinal cracks in the ZK12 + 930–ZK13 + 060 section on the right wall of the left line of the Tongxi Tunnel during operation. Further, the width of the longitudinal cracks at ZK12 + 990 was large and misaligned, as displayed in Fig. 2.

![Figure 1. Stratigraphic structure of the study area](image1)

(a) Cavity morphology  (b) Sidewall staggered
To identify the cracks in the ZK12 + 930–ZK13 + 060 section of the tunnel, a crack depth sounder and a width gauge were used to observe and record the tunnel’s appearance. According to the observation results, 39 cracks were present in the detection section. Thirty-one were longitudinal cracks, accounting for 79.5% of the total number; one circular crack accounted for 2.6%; seven oblique cracks accounted for 17.9%. From the crack width meter and depth sounder results, 12 cracks of width greater than 1 mm were present of which the widest crack was 3.4 mm. For the longitudinal crack located on the left 17-m long wall of pile number K12 + 994, the crack depth was 142 mm, whereas the diagonal crack on the left wall of K12 + 983, which appeared to be staggered, was 4 m in length and 3 mm in width.

3. Similar model test

3.1. Similarity ratio and similar materials

The focus of this experimental study is the impact of local high water pressure at different locations on the tunnel lining structure. Although the elevation in water pressure in real cases is a gradual process over time, we simplified this process into a static problem owing to the limitation of the experimental conditions. According to the similarity theorem, the elastic modulus and length are selected as two similar fundamental constants, and the similarity ratio is considered as 1:50. Based on a previous tunnel similar model test experience, 400 mesh barite powder (barium sulfate), 40–70 mesh quartz sand, ordinary gypsum, gypsum retarder, water, glycerin, and PC32.5 white cement were selected as similar materials for model surrounding rock and lining structure. From field data, the surrounding rock of the tunnel is mostly limestone in the karst area, and the lining adopts a C30 plain concrete structure. Through similar material tests, material ratios for surrounding rock and lining structure were finally chosen, as shown in Tables 1 and 2, respectively.

| Table 1. Main physical parameters of prototype surrounding rock and similar materials |
|-----------------------------|-----------------|-----------------|----------------|-----------------|----------------|
| Type                        | Bulk density    | Elastic modulus | Poisson’s ratio | Cohesion        | Friction angle  |
|                            | (kN/m³)         | (GPa)           |                 | (kPa)           | (°)            |
| Surrounding rock            | 22              | 2.4             | 0.3             | 500             | 39             |
| Similar material            | 24              | 0.069           | 0.28            | 29              | 37             |
| Similar material ratio      |                 |                 |                 |                 |                |
|                            | quartz sand: barite powder: plaster: water = 0.81:1:0.41:0.33 |
Table 2. Physical parameter table for secondary lining prototype and similar model

| Type                  | Bulk density (kN/m³) | Elastic modulus (GPa) | Poisson's ratio | Cohesion (kPa) | Friction angle (°) |
|-----------------------|----------------------|-----------------------|-----------------|----------------|-------------------|
| Secondary lining      | 24.9                 | 31                    | 0.3             | 2000           | 41.8              |
| Target value          | 24.9                 | 0.62                  | 0.3             | 61.3           | 41.8              |
| Test value            | 23                   | 0.86                  | 0.2             | 50             | 55                |
| Similar material ratio|                      |                       |                 |                |                   |
|                       | White cement: quartz sand: barite powder: plaster: water = 1.27:2.26:1:2.3:1.87 |

3.2. Test device

The test adopts a self-made model frame comprising three parts: model box, reaction frame, and loading device. The model box’s left, right, and rear sides are all made of a 5-mm steel plate, and the front side is made of tempered glass for observation and data collection. The reaction frame is an auxiliary device for applying initial ground stress and simulating high water pressure load. It is welded using angle steel and I-beam. The appearance and design dimensions of the model box and self-made reaction frame are shown in Fig. 3.

![Figure 3. Self-made reaction frame and model box schematic](image)

To simulate the effect of high water pressure on the tunnel lining structure and obtain the tunnel lining structure’s internal force and deformation characteristics under the combined action of water pressure and confining pressure, the loading device is divided into surrounding rock and hydraulic pressures. The surrounding rock pressure loading device is applied step by step using hydraulic jacks. Before loading, a high-strength steel plate with a thickness of 2.5 cm is placed on the upper part of the model to ensure that the surrounding rock mass is evenly compressed.

The hydraulic loading device uses a separate hydraulic jack to apply a specific area of local load at a predetermined point behind the lining. The load applied by the jack gradually simulates the effect of high water pressure on the lining (shown in Fig. 4). The device comprises four parts: a separate hydraulic jack, an oil pressure gauge, a pressure sensor, and a fixing device. The jack has an accuracy of 0.1 kPa. Its specific structure and working principle are as follows: high-strength steel bars with a diameter of 20 mm are welded to the telescopic end with a high-precision hydraulic gauge jack, and soft rubber is pasted using high-strength glue at the places where the steel bar is in contact with the lining. The pressure sensor is then glued on the other end of the rubber, and the jack is finally fixed to the predetermined position of the reaction frame through the iron absorption stone (shown in Fig. 5). During the test, the

![Figure 4. Hydraulic loading device schematic](image)

![Figure 5. Reaction frame schematic](image)
soft rubber played the role of protecting the pressure sensor and reducing the excessive hardness of the steel bar. The sensor is used to accurately collect the load size of the jack’s step-by-step loading, and it is easy to control.

Figure 4. Simulated hydraulic device schematic

Figure 5. Diagram of the simulated water pressure point contact

3.3. Lining structure production
The lining model is based on the actual tunnel engineering design drawing. Therefore, the cross-section size meets the similarity ratio of 1:50. The final designed mold size is 38-cm high with the cross-section width and height of 26 and 18.5 cm, respectively. The mold adopts three half molds of the inner cylinder and upper and lower walls. The material uses a high-strength aluminum plate and is fixed with bolts. After the model is poured, the concrete vibrating platform is used to vibrate and compact, and after the initial setting time is reached, the mold is removed for maintenance. The production process of the entire lining model is shown in Fig. 6.

3.4. Measuring point layout
According to the different positions of the tunnel section vault in the experiment, arch waist, arch foot, sidewall, invert arch, and strain gauges were arranged at eight key points. The arrangements were symmetrically inward and outward along the model ring. To study the changes in the longitudinal mechanical properties of the lining structure, three sections were arranged from the inside to the outside of the tunnel, and a miniature earth pressure sensor was placed close to the lining at the corresponding position of eight points on each lining section. The internal force value at each secondary lining position was obtained by measuring the strain on the inner and outer surfaces of the lining. The layout of the strain gauges inside and outside the lining structure is shown in Fig. 7.
3.5. Loading process
The content of this experiment mainly considers the hydraulic pressure influence of the tunnel lining on the internal force of the lining structure. From the project site situation, the research focuses on studying the change in the internal force of the lining when the water pressure is concentrated on the vault. At an interval of 8 kPa and stoppage time of 5 to 8 min for observation, the water pressure is loaded step by step. After the lining and surrounding rock deformation are stabilized, the next loading level is conducted until the lining structure is damaged and the loading is stopped.

4. Model test results and discussion

4.1. Contact pressure of lining-surrounding rock
Figure 8 shows the change in the distribution of lining-surrounding rock contact pressure under different water pressures when water pressure acts on the lining vault. As is evident from the change in the contact pressure distribution diagram in Fig. 7, when the earth pressure only acts on the lining structure,
contact pressure behind the tunnel lining gravitates toward a symmetrical distribution. The maximum contact pressure appears at the left and right arch foots at −29.0 and −28.5 kN, respectively, whereas the minimum occurs at the inverted arch with a value of −8.3 kN. However, when the hydraulic load acts on the lining structure, the contact pressure distribution changes. When the water pressure increases from 0 to 44 kPa, the vault contact pressure decreases from −19.6 kN to a minimum value of −3.2 kN, a 83.67% reduction. The contact pressure of the remaining measuring points mostly increases with the increase in water pressure. The above analysis shows that the lining in the hydraulic pressure area is prone to uneven deformation and detaches from the surrounding rock, resulting in a decrease in contact pressure. Consequently, the contact between the remaining regions and surrounding rock is closer, and the contact pressure increases.

![Figure 8](image1)

**Figure 8.** Variation in contact pressure distribution in surrounding rock lining

4.2. Lining structure axial force

Figure 9 shows the axial force distribution at each characteristic point of the lining. At no water pressure outside the lining, the axial force distribution of the lining structure is symmetrically distributed in a “horseshoe shape” with the axial force gradually increasing from top to bottom. Consequently, the axial force in the arch waist and sidewall areas enlarges. For example, the maximum axial force of the right arch foot is −5268.1 N, and the minimum axial force of the invert is −2567.9 N. When the water load on the lining increases, the axial force of the lining is still distributed in a horseshoe shape, but the axial force value increases with the increasing water load, except for a slight decrease in the invert. Further, the closer the horizontal direction is to the water pressure loading area, the faster is the increment in the axial force. For example, when the water pressure increases from 0 to 44 kPa, the maximum rise of the vault is 20.05%, arch waist is 17.19% (left) and 10.62% (right), and minimum sidewall is 7.86% (left) and 4.98% (right).

![Figure 9](image2)

**Figure 9.** Changes in axial force distribution of tunnel lining

From the curve diagram of the change in the axial force at each characteristic point of the lining with the water pressure (Fig. 10), it can be observed that the axial force at each distinctive point increases linearly with the increase in the water pressure. According to the analysis of the increase rate in the axial force change with every 8 kPa increase in the water pressure, the axial force increase rate at the dome was the fastest and lowest at the invert with the average values of 106 N/8 kPa, and 16 N/8 kPa, respectively. The difference between the maximum and minimum axial forces under different water pressures consistently widened, reaching the highest value at 4562 N.
4.3. Lining structure bending moment

Figure 11 shows the distribution of bending moments at each characteristic point of the lining. When there is no water pressure, the bending moments’ distribution at each point of the lining was butterfly shaped. At no vault water pressure, the invert of the lining is the point of maximum positive bending moment, which is 9.67 N·m. When the water pressure is gradually loaded to 44 kPa, the bending moment of the inverted arch slowly increases and that at the arch waist slowly decreases, showing a tendency to turn into a negative bending moment. The maximum positive bending moment moderately shifts from the invert to the vault area, reaching 14.01 N·m at a corresponding vault bending moment increase by 107%. However, the bending moment of the left wall only increases by 6.55%. Intuitively, the rise in water load has the most obvious influence on the bending moment in the load acting area.

4.4. Safety factor

Figure 12 shows the variations in the safety factor at each measuring point on the structure when the vault is loaded. It is evident from Fig. 12 that when the water pressure is loaded on the vault, the overall safety factor of the structure diminishes as the water pressure increases. Moreover, the vault area experienced the largest amplitude, whereas the sidewall area showed the smallest decline. Therefore, it can be judged that instability first occurs in the vault area when the tunnel vault is loaded with water, and the sidewall is the least affected.
Figure 12. Variation curve of safety factor under water pressure

4.5. Destruction form of lining structure
Figure 13 shows the failure form of the lining model. At 36 kPa water pressure, two cracks developed along the depth of the tunnel, with the first appearance at the vault and the other at the left footwall. At increasing water pressure, the depth of the cracks gradually widened from the inside to outside, resulting in penetration. Finally, when the water pressure was nearly 44 kPa, the width of the cracks at the vault continuously expanded. Moreover, circular shear cracks broke out at the position of the load, and thus the block collapsed. The maximum diameter of the lining penetration caused by the fall was 5 cm.

Figure 13. Failure mode of a specimen under arch load

5. Conclusion
In this study, similar experimental researches were conducted to analyze the structural mechanics and failure modes of a hidden karst tunnel subjected to local high water pressures. Based on the analysis of the experimental results, the following conclusions were obtained.

1) When water exerts pressure on a particular lining area, its contact pressure area decreases with increasing water pressure. Conversely, the contact pressure of the other regions increases accordingly. Therefore, attention should be paid to the local uneven deformation that may occur in the area.

2) The position and size of the external water pressure do not affect the general distribution of the axial force of the lining. The value of the axial force is proportional to the size of the water pressure, but the maximum value and position of the axial force are affected by the area of action.

3) When local water pressure acts on the lining structure, the bending moment in the acting area changes to the direction of the positive bending moment. Therefore, paying attention to the water load in the negative bending moment area is particularly necessary. The lining is likely to change from internal compression to internal tension.
(4) When the water pressure acts on the vault, the lining structure starts to crack from the sidewall, then a circular shear crack appears at the load acting position, and finally, the lining breaks.

**Acknowledgment**

This work is supported by the Natural Science Foundation Project of Chongqing (Grant No. cstc2013jcyjA30005) and Fund of National Engineering Laboratory for Highway Tunnel Construction Technology (No. NELFHT201901).

**References**

[1] Zhou ZQ, Li SC, Li LP, Shi SS and Xu ZH 2015. *Geomech. Eng.* 8 (5) 631-647.
[2] Li SC, Zhou ZQ, Li LP, Xu ZH, Zhang QQ and Shi S S, 2013b. *Tunn. Undergr. Sp. Tech.* 38, 50-58.
[3] Alp M and A. Apaydin 2019 *Tunnelling and Underground Space Technology* 89 157-169.
[4] Peng L J 2014 *Adv. Mater. Res.* 1004-1005, 1444-1449.
[5] Wang HT, Wang LG, Li SC, Wang Q, Liu P and Li XJ 2019 *Engineering Failure Analysis* 98 215-227.
[6] Huang Y, Fu ZM, Chen J, ZhouZF and Wang JG 2015 *Tunnelling and Underground Space Technology* 48 58-66.
[7] Zou YL, He C, Zhou, Y, Zhang, Z and Fu JK 2013 *J. Highway. Transp. Res. Dev.* 30 (1), 86-101 (in Chinese).
[8] Han HY, Liu HY, Chan A and Mcmanus T 2019 *Sādhanā* 44(8).
[9] Guo R, Zhang MY, Xie HM, He C, Fang Y and Wang SM 2019 *Tunnelling and Underground Space Technology* 91.
[10] Gao CL, Zhou ZQ, Yang WM, Lin CJ, Lia LP and Wang J 2019 *Tunnelling and Underground Space Technology* 94.
[11] Zhang L, Feng K, Gou C, He C, Liang K and Zhang HH 2019 *Tunnelling and Underground Space Technology* 92.
[12] Wang XY, Tan ZS, Wang MS, Zhang M and Ming HF 2008 *Tunnelling and Underground Space Technology* 23(5) 552-560.
[13] Xu ZH, Wang XT, Li SC, Gao B, Shi SS and Xu XJ 2019 *KSCE Journal of Civil Engineering* 23(6) 2772-2783.
[14] Fang Y, Guo JN, Grasmick J and Mooney M 2016 *Tunnelling and Underground Space Technology* 60 80-95.
[15] Weng XL, Sun YF, Zhang YW, Niu HS, Liu X and Dong, YL 2019 *Tunnelling and Underground Space Technology* 90 208-219.
[16] Zhao YD, Zhang YX, Yang JS and Sun R 2019. *Geotechnical and Geological Engineering* 37(5) 4613-4625.