Research Article

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Bearing properties and influence laws of concrete-filled steel tubular arches for underground mining roadway support

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Abstract: The concrete-filled steel tubular (CFST) arch is a new high-strength support form for a mine roadway in deep/soft rock stratum; however, the bearing characteristics have not been clearly elucidated for scientifically guiding field applications. Numerical simulation tests with 15 schemes shaped as a ‘half circle with two straight legs’ and 10 schemes shaped as a circle were conducted, and the main responses of the numerical model were verified by performing the laboratory tests to evaluate the basic CFST structures and global CFST arches. The bearing and failure behaviors of the CFST arches were studied, and the influence laws, in terms of the arch shape, size and lateral pressure coefficient λ , were further investigated. The results show that the bearing capacity of a circular arch is significantly higher than that of a straight-leg arch under a uniform load. Furthermore, the bearing capacity of the circular arch decreases considerably with the increase in the arch size or λ . In addition, the bearing capacity of a straight-leg arch decreases with the increase in the leg height and arch size; however, it first increases and later decreases with the increase in λ . The failure modes of all the arches correspond to the instability at the extreme point caused by the strength deterioration, except in the case of a circular arch under a uniform pressure, the failure mode of which corresponds to the instability at the branch points. Finally, the recommendations for the field practice are proposed and verified.

Keywords: mine roadway; concrete-filled steel tube; supporting arch; bearing characteristic.

1 Introduction

With the continuous breakthroughs in science and technology in recent years, the problem of a mine roadway support under normal conditions has been gradually solved. However, due to the complexity and diversity of the soft rock roadway conditions (high-stress soft rock, weak cemented soft rock in west China, etc.), the conventional support theories and control technologies still possess several limitations. Roadways often exhibit large long-term deformation, which leads to a lowered production efficiency, increased supporting cost and higher safety risks. Coal is expected to continue serving as the principal energy source for a long time to come, especially in China, where more coal mines are confronted with the issue of soft rock supports as the mining depths continue to increase and the mining strategy more often incorporates Western China [1–3]. Therefore, it is necessary to develop soft rock roadway support theories and promote the enhancement of the supporting technologies to realize the stability of the mine roadways.

Researchers have attempted to employ new concepts, materials and technologies to solve the difficulties of a soft rock roadway support [4–9]. A representative solution is the use of the concrete-filled steel tubular (CFST) arch support. A CFST structure can exploit the material capacities of steel and concrete. CFST arch supports have been sporadically used in trial cases in underground projects such as mine roadways and traffic tunnels in China and overseas since the 1970s. Subsequently, under the increasingly urgent demands for the soft rock roadway support, the CFST arch support has been widely applied and rapidly developed since 2010 [10]. The new support style presents a high bearing capacity and excellent stability. Thus, this ap-

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Figure 1: Failure cases of the mine roadways supported with CFST arches.

proach is presently one of the preferred supporting methods for soft rock roadways [11–15].

For a supporting arch in the field, the mechanical environment is considerably different from that of an ordinary ground structure because the loading effects acting on the supporting arch by the surrounding rocks include not only complicated loads but also special displacement restrictions. As a result, the existing theories of the strength and instability of the CFST structure cannot be directly applied in the design of an underground CFST supporting arch. Therefore, the CFST arch support theory was directly studied by employing laboratory tests, theoretical derivations and numerical simulations when developing the field practices. The strength characteristics of the CFST arch were studied through axial compression tests, pure bending tests and eccentric compression tests [12, 14–19]. Further, several scholars, including the authors of this article, studied the global bearing characteristics of the CFST arches through full sized CFST arch experiments [8, 12, 14, 15]. Based on the full sized CFST arch tests, Zhang *et al.* [8] further revealed the failure mechanism and developed the bearing capacity calculation theory of a CFST supporting arch.

However, these achievements did not provide sufficient and direct guidance for the roadway support practice, and several roadways still experienced critical problems even when supported with high-strength CFST arches, as shown in Figure 1. We believe that the reason for this phenomenon is that the ‘link bridge’ between the CFST support theory and engineering practice has not been established, and some of the existing research achievements cannot be directly used to guide support design. The theoretical calculation can only provide a principle reference

and cannot be directly used for the design because the assumptions and simplicity of the mechanical model and derivation process in the theoretical calculation may lead to a calculation error when applied to the considerably complex practice. Furthermore, it is impossible to conduct a large number of tests considering all the relevant influencing factors, such as the sizes, shapes, cross-section parameters of the arch, and load patterns, due to the high cost of the laboratory tests.

To solve the problem, the research presented in this paper builds upon the existing preliminary research. Numerical simulation tests of the CFST arches were conducted by using the ABAQUS software, considering three designing factors of the supporting arch, namely, the arch shape, arch size, and load distribution patterns. The laboratory tests performed by our own scientific research team [8, 12] were taken as the comparison objects to verify the reliability of the numerical simulation tests. The influence laws of the main factors on the supporting ability of the CFST arch were clarified, which can provide direct guidance for the practical design and help in the preliminary establishment of the ‘bridge’ between the CFST support theory and engineering practice.

2 Numerical simulation test schemes

2.1 Test scheme design

A total of 25 schemes were considered, which were divided into 5 groups with 5 schemes in each group, as presented

Table 1: Numerical simulation test scheme

No.	Arch shape	Arch size/m	Lateral pressure coefficient λ	Test purpose
1–5	A half circular arch with two straight legs	$R=2.575$	1.0	Influence of leg height
		$H=0, 0.5, 1.0, 1.95, 3.0$		
6–10		$R=1.0, 1.5, 2.0, 3.0, 4.0$ $H=0.6R$	1.0	Influence of arch size
11–15	Circular	$R=2.575$	0.5, 0.75, 1.0, 1.25, 1.5	Influence of lateral pressure coefficient
		$H=1.5$		
16–20		$R=1.0, 1.5, 2.0, 3.0, 4.0$	1.0	Influence of arch size
21–25		$R=2.575$	1.0, 1.25, 1.5, 2.0, 2.5	Influence of lateral pressure coefficient

Note: Lateral pressure coefficient $\lambda = q_2/q_1$ (Figure 2b).

Figure 2: Models of the tested CFST supporting arch. (a) straight-leg arch; (b) circular arch; (c) cross-section of the arch.

in Table 1. The arches in Schemes 1–15 were half circular arches with two straight legs (hereafter defined as straight-leg arches, as shown in Figure 2a), and the other schemes involved circular arches (Figure 2b). In the first group, the height of the arch leg H was the variable; the arch radius R was the variable in Schemes 6–10; and the lateral pressure coefficient λ was the variable in Schemes 11–15. The fourth group consisted of Schemes 16–20, in which the arch size (radius R) was the variable. The lateral pressure coefficient λ was the variable in Schemes 21–25.

The cross-section of the test arch was the same as that used in the previous laboratory test [8]. The arch was a cold-formed square steel tube (external side length 150 mm, thickness 8 mm) with C40 strength grade concrete (Figure 2c). The load schemes are shown in Figure 2a and 2b. The arch feet were set as two hinges (the laboratory tested arch feet were also set as hinges) to simulate the interacting conditions of the arch in practice as the feet-lock bolts are installed on the arch feet. In the circular arch simulation schemes, the vertical displacement at the arch crown and the horizontal displacement at the right side were restricted to ensure the computational convergence.

Both in this research work and the previous study, the square cross-section arch was selected instead of the circular cross-section arch for the following reasons [8, 12]: First, the bearing capacity of a square cross-section CFST is similar to or slightly higher than that of a circular cross-section CFST with the same steel area. Second, the square section CFST arch has a larger contact area with the surrounding rock compared with that of the circular one, which reduces the stress concentration and plays a positive role in fostering the supporting ability of the CFST arch. In general, we do not believe that any given cross-section is superior than the other because there still remain differences among the different cross-section CFST arches, such as those corresponding to the production technology, on-site installation processes and economic cost, although the bearing capacity and arch-rock contact relation do not change. The square section form is simply a choice for the roadway support against the circular form, and the constructors can choose any cross-section according to the specific conditions.

Figure 3: Typical results of the strength tests of a steel tube sample and standard concrete cube.

2.2 Modeling and mechanical parameters

The finite element method software ABAQUS was used as the simulation tool [18, 19]. The C3D8R element was selected for the steel tube and concrete, the “tie restriction” was applied between the inner wall of the steel tube and the core concrete, and the parameters of the steel and concrete were determined through material testing.

2.2.1 Steel parameters

The cold-formed steel tube used in the CFST arch was made of steel sheets through rolling manufacture, and the steel at the corners exhibited reinforcement and hardening. Thus, the cross-section of the square steel tube was distinguished as the corner area or flat area, as shown in Figure 2c.

Adbel-Rahman and Sivakumaran [20] studied this cross-section and presented the following calculation formula to calculate the steel yield strength in the entire corner area:

$$fy1 = \left[0.6 \times \frac{Bc}{(r/t)^m} + 0.4 \right] \cdot fy \quad (1)$$

where

f_y denotes the steel yield strength of the flat area, determined according to the steel tensile test; B_c and m are coefficients,

$$B_c = 3.69(f_u/f_y) - 0.819(f_u/f_y)^2 - 1.79,$$

$$m = 0.192(f_u/f_y) - 0.068.$$

where f_u is the tension strength of the flat area.

Based on the relevant research results [21], the weighted average yield strength formula for the cold-formed square steel tube sections is as given in Eq. (2), and this relation was chosen based on the cold-formed steel de-

sign rules in North America and Australia/New Zealand:

$$fya = Cfy1 + (1 - C)fy \quad (2)$$

where C is the ratio between the corner area and the entire cross-section area.

Figure 3a shows the strength test of a steel sample taken from the flat area of a steel tube. The mechanical parameters of the steel in the flat area, f_p , f_y and f_u , were, respectively, 285 MPa, 351 MPa and 562 MPa, and the elasticity modulus E_s was 2.04 MPa. Subsequently, the weighted average strength of the tested cold-formed steel tube was calculated: The yield strength fya was 409 MPa, the ultimate strength was 594 MPa and the elastic modulus was 2.04 GPa. The abovementioned mechanical parameters were finally applied to the four-stage constitutive model.

2.2.2 Concrete parameters

The plastic damage constitutive model was applied for the concrete because this model can better simulate the plastic performance of the concrete and its rigidity degeneration under alternating stresses. Considering the influence of the restriction effect of the steel tube on the concrete strength, the core concrete stress-strain relationship was modeled as

$$y = \begin{cases} 2x - x^2 & (x \leq 1) \\ \frac{x}{\beta_0(x-1)^{\eta} + x} & (x > 1) \end{cases} \quad (3)$$

where $x = \frac{\varepsilon}{\varepsilon_o}$, $y = \frac{\sigma}{\sigma_o}$, $\varepsilon_o = \varepsilon_c + 800\xi^{0.2} \times 10^{-6}$, $\eta = 1.6 + 1.5/x$, $\varepsilon_c = (1300 + 12.5f'_c) \times 10^{-6}$, and $\beta_0 = \frac{(f'_c)^{0.1}}{1.2\sqrt{1+\xi}}$. ξ is the restriction effect coefficient; $\xi = A_s f_y / A_c f_{ck}$; f_{ck} is the standard value of the axial compressive strength of concrete; A_s is the cross-sectional area of the steel tube; A_c is the cross-sectional area of the core concrete; f'_c is the

Figure 4: Mechanical tests on the basic CFST structures performed via laboratory tests and numerical simulations.

compressive strength of the concrete cylinder; and f_{ck} is the standard value of the axial compressive strength of the concrete. In the above formulas, the unit of the compressive strength of the concrete is MPa. The strength of the test cube of the core concrete, f_{cu} , was determined as 43.2 MPa by using the standard tests shown in Figure 3b.

2.3 Verification of the numerical simulation

2.3.1 Verification of the basic CFST structures through laboratory tests

Axial compression tests for empty short steel tubes and C40 concrete-filled steel tube columns were conducted. The height of the column was 450 mm, the width of the square steel tube section was 150 mm, and its thickness was 8 mm. The grade of the filled concrete was C40. Furthermore, the corresponding numerical simulation tests were conducted with the same structures.

A wavy buckling appeared on the empty steel tube (Figure 4a), while a waist drum shaped deformation occurred in the CFST specimens (Figure 4b). The deformation type and characteristics of the short column, as obtained from the numerical test, were basically the same as those obtained using the laboratory tests. Figure 4c shows the axial compression load-strain curves of the short column specimens; it can be noted that the curve shapes of the numerical simulation and laboratory test were nearly identical. In addition, the deviations of the ultimate bearing ca-

capacity were 0.1% and 1.15% for the empty specimens and CFST specimens, respectively. The compression-bending tests on CFST columns with a length of 1400 mm and the same section were conducted at different eccentricity ratios, as shown in Figure 4d. The numerical simulation results were consistent with the laboratory test, and the obtained load axial strain curve (Figure 4e) indicated that the two curves were nearly identical in terms of the curve shape and peak value. The above comparison results indicate that the selected simulation method and parameters are feasible for simulating the CFST structures.

2.3.2 Verification of the CFST arch through laboratory tests

Figure 5a illustrates the deformation shapes of a straight-leg CFST arch with a square cross-section (150 mm width and 8 mm thickness with C40 concrete) under uniform loads, both in the laboratory and numerical simulation tests (Scheme 4 in Table 1). Figure 5b shows the deformation curve of the arch at typical sections I and II (the outward deformation is positive, and the inward deformation is negative). The load-displacement curves in the numerical simulation can be divided into three stages: the elastic stage (OA), in which the load and displacement are in a linear relationship and the displacement increases slowly; the elastic-plastic stage (AB), in which the curve slope decreases; and finally, the plastic stage (BC), in which the curve stays horizontal and the load no longer increases.

Figure 5: Full size tests of the CFST arches.

Table 2: Numerical simulation results of Schemes 1–15

Scheme No.	Arch size/m	Lateral pressure coefficient λ	Position of the dangerous section	Load / kN/m		
				Point A	Point B/ Bearing capacity	Maximum
1	$R=2.575$ $H=0$	1.0	800 mm upwards toward the leg top-end	386.2	409.9	412.0
2	$R=2.575$ $H=0.5$		leg top-end	407.2	446.2	451.6
3	$R=2.575$ $H=1.0$		leg top-end	289.5	298.3	303.0
4	$R=2.575$ $H=1.95$		500 mm lower on the leg top-end	79.3	97.6	103.1
5	$R=2.575$ $H=3.0$		1000 mm downward from the leg top-end	41.2	46.6	47.3
6	$R=1.0$ $H=0.6$	1.0	50 mm lower on the leg top-end	1018.1	1196.1	1243.5
7	$R=1.5$ $H=0.9$		150 mm lower on the leg top-end	418.6	485.7	495.2
8	$R=2.0$ $H=1.2$		200 mm lower on the leg top-end	244.8	248.1	253.3
9	$R=3.0$ $H=1.8$		450 mm lower on the leg top-end	88.6	94.7	99.4
10	$R=4.0$ $H=2.4$		500 mm lower on the leg top-end	43.5	48.0	48.6
11	$R=2.575$ $H=1.5$	0.5	911 mm up the leg top-end	111.4	123.6	126.8
12		0.75	400 mm lower on the leg top-end	238.3	253.5	257.3
13		1.0	350 mm lower on the leg top-end	129.2	140.4	141.9
14		1.25	150 mm lower on the leg top-end	75.1	87.2	103.7
15		1.5	150 mm lower on the leg top-end	56.7	67.0	89.6

Points A, B and C are considered as the critical loading moments, as described in the following text. The load at Point B is defined as the bearing capacity of the arch, specifically 103.1 kN/m. At point B, the arch legs bent inward, the lateral displacement reached approximately 78 mm, and the roof deformed outward, after which the arch deformation became more intense and unstable. The deformation of the

numerical arch, as shown in Figure 9b, is similar to that of the laboratory testing. In summary, the arch deformation form in the numerical simulations is basically consistent with that of the laboratory tests, and the deviation of the arch bearing capacity from the laboratory test results is lower than 5%. The applied numerical simulation method and parameters are thus reliable.

Figure 6: Typical results of Schemes 1–5 with different leg heights. (a)–(c) Stress contours and deformed shapes of the arches; (d) Load-radial displacement curves; (e) Characteristic loads influenced by the leg heights; (f) Axial stress at the dangerous section, influenced by the leg height; (g) Ratio of the axial stress at the inner and outer sides of the dangerous section, influenced by the leg height.

3 Straight-leg arches

3.1 Main results

Table 2 presents the results of the straight-leg arches (Schemes 1–15). In the table, ‘high-risk section’ is the cross-section corresponding to the highest steel stress on the arch at the critical loading moment B (Point B on the load-displacement curve); ‘Load’ is the total load on the arch

divided by the complete length of the arch axis line, unit: kN/m; and ‘Bearing capacity’ is the load on the arch at loading moment B, unit: kN/m.

3.2 Influence laws of the leg height

Figure 6 shows the typical results of Schemes 1–5 with different leg heights.

- (1) The arch crown in Schemes 2–5 deformed outwardly and the legs bent inward, which is contrary to the state in Scheme 1 ($H=0$ m). The leg height within a certain range thus influences the deformation model of the arch.
- (2) The load-displacement curves in each scheme basically exhibit the same shape (Figure 6d), all of which can be divided into three stages, similar to those in Scheme 4, as described in Section 1.3. In addition, a higher arch leg corresponds to a flatter linear section of the curve.
- (3) Figure 6e shows the critical load curves influenced by the leg height. The elastic critical loads (Point A) are slightly smaller than the elastoplastic critical loads (Point B), which is basically the same as the extreme load. The bearing capacity curve presents an early peak followed by a decrease, and the bearing capacity of the arch with a leg height of 0 m is smaller than that of the arch with a leg height of 0.5 m. Furthermore, the bearing capacity decreases as the height of the arch leg increases when the height of the arch leg is greater than 0.5 m; a longer leg corresponds to a lower influence.
- (4) As the height of the arch leg increases, the position of the dangerous section of the arch gradually moves downward from the leg top-end.
- (5) The steel stress at the dangerous section of the arch at the critical loading moments A and B is shown in Figures 6f and 6g. The signs of the stress values on the inner and outer sides on the dangerous section are opposite, which indicates that the dangerous section is subjected to a compression-bending inner force. When the arch leg height is larger than 0.5 m, the absolute value of the stress ratio increases as the arch leg becomes longer, which indicates that the bending moment acting on the dangerous section increases as the arch leg becomes longer. The steel stress exceeds the yield strength at critical loading moment A and ranged from $-3300\mu\epsilon$ – $6000\mu\epsilon$ at critical loading moment B. This result indicates that the failure modes of the tested arches correspond to the instability at the extreme point caused by the strength deterioration.
- (6) Overall, the arch leg height (0–3 m) has an influence on the arch deformation form but not on the arch failure mode.

3.3 Influence laws of the arch size

Figure 7 shows the main results for Schemes 6–10 with different arch sizes.

- (1) All the deformation forms of the tested arches ($R=1.0$ – 4.0 m) are the same, the arch legs are bent inward, and the crown is deformed outward.
- (2) The forms of the load-radial displacement curve of each scheme are basically the same and can be divided into three stages, which is the same as that for Scheme 4 described in Section 1.3. A larger arch size corresponds to a smaller slope of the linear stage (OA) of the curve.
- (3) Figure 7e shows the critical load curves influenced by the arch size. The bearing capacity constantly decreases as the arch size increases, which is reflected in the form of an approximately inverse function. A smaller arch size corresponds to a larger influence.
- (4) The position of the dangerous section gradually deviates downward from the leg top-end as the arch size increases.
- (5) The steel stress values at the dangerous section on the arch at critical loading moments A and B are shown in Figures 7f and 7g. The stress on the outer side of the dangerous section is compressive, and that on the inner side is tensile, which indicates that the dangerous section is subjected to a compression-bending load with a large eccentricity. The absolute value of the stress ratio in Figure 7g increases as the arch size increases, and the ratios in both schemes with $R=3$ m and $R=4$ m are larger than 1, which indicates that a larger arch size corresponds to a larger bending moment acting on the dangerous section. The steel stress exceeds the yield strength at critical loading moment A and ranges from $-3200\mu\epsilon$ to $-4200\mu\epsilon$ (except for in Scheme 6) at critical loading moment B. This finding indicates that the failure modes of the tested arches ($R=1.0$ – 4.0 m) correspond to the instability at the extreme point caused by the strength deterioration.

3.4 Influence laws of the lateral pressure coefficient

Figure 8 shows the main results of Schemes 6–10 with the different arch sizes.

- (1) Arches with lateral pressure coefficients from 0.75 to 1.5 share the same deformation form; the arch crown deforms outward, and the legs bend inward, which

Figure 7: Typical results of Schemes 6–10 with different arch sizes. (a)–(c) Stress contours and deformed shapes of the arches; (d) Load–radial displacement curves; (e) Characteristic loads influenced by the arch size; (f) Axial stress at the dangerous section, influenced by the arch size; (g) Ratio of the axial stress at the inner and outer sides of the dangerous section, influenced by the arch size.

is in contrast to the trend for the arches with a lateral pressure coefficient of 0.5.

- (2) The load–radial displacement curves of each scheme have basically the same form, and all of them can be divided into three stages, which is the same as the curve defined in Section 1.3.
- (3) Figure 8e shows the critical load curves influenced by the lateral pressure coefficient. The arch bear-

ing capacity first increases and later decreases as the pressure coefficient increases, and this capacity reaches the maximum value when the lateral pressure coefficient reaches 0.75.

- (4) The dangerous section is located above the leg top-end when the lateral pressure coefficient is 0.5. This section gradually moves downward as the coeffi-

Figure 8: Typical results for Schemes 11–15 with the different lateral pressure coefficients. (a)–(c) Stress contours and deformed shapes of the arches; (d) Load–radial displacement curves; (e) Characteristic loads influenced by the lateral pressure coefficient; (f) Axial stress at the dangerous section, influenced by the lateral pressure coefficient; (g) Ratio of the axial stress at the inner and outer sides of the dangerous section, influenced by the lateral pressure coefficient.

cient increases, and this section moves below the leg top-end when the coefficient reaches 0.75.

- (5) The steel stress values at the dangerous section of the arch at critical loading moments A and B are shown in Figures 8f and 8g. All the stress ratios are approximately -1 , and the stress signs on the inner and outer sides of the dangerous section are opposite, indicating that the dangerous section is subjected to a compression-bending load with a large eccentricity. The steel stress exceeds the yield

strength at critical loading moment A and ranges from $-2700\mu\epsilon$ to $-3900\mu\epsilon$ (except in Scheme 11) at critical loading moment B. The abovementioned results indicate that the deformation form changes as the lateral pressure coefficient changes from 0.5 to 0.75. However, the failure mode does not change, and all the failure modes correspond to the instability at the extreme point caused by the deterioration in the section strength.

Table 3: Numerical simulation results for Schemes 16–25

Scheme No.	Arch size R/m	Lateral pressure coefficient λ	Position of the dangerous section	Load/kN/m		
				Point A	Point B/ Bearing capacity	Maximum
16	1.0	1.0	Crown and bottom	1857.4	1944.1	1945.5
17	1.5			1431.8	1431.8	1464
18	2.0			986.6	1072.0	1081
19	3.0			364.6	399.7	416.9
20	4.0			193.6	213.1	217.4
21	2.575	1.0	Crown and bottom	793.0	837.7	843
22		1.25		127.4	140.1	141.3
23		1.5		78.7	87.3	88.5
24		2.0		43.8	50.6	51.6
25		2.5		28.8	35.8	37.8

4 Circular arches

4.1 Main results

Table 3 presents the results of the circular arches (Schemes 16–25).

4.2 Influence laws of the arch size

Figure 9 shows the main results for Schemes 16–20 with the different arch sizes.

- (1) The arch deformation form changes remarkably as the arch size changes. Except the case of Scheme $R=1.0$, the roofs of all the other arches exhibit an inward deformation.
- (2) The forms of the load–radial displacement curve in each scheme are basically the same, and they can all be divided into three stages, which is the same trend as the curves in Scheme 4, as described in Section 1.3. One difference pertaining to the semicircular straight leg schemes is that the distance between Points A and B is considerably smaller, which indicates the failure instantaneity.
- (3) Figure 9e shows the critical load curves influenced by the arch size. The arch bearing capacity decreases remarkably as the arch size increases, which corresponds to an approximate inverse function style.
- (4) The stress values of the crown and bottom sections are, respectively, similar and higher than those for the other sections, and they are both dangerous sections for the arch.

- (5) The comparison between Schemes 18 ($R=2.0$ m) and 8 ($R=2.0$ m, $H=1.2$ m) indicates that for a CFST arch with the same section under a uniform pressure, the bearing capacity of the circular arch is approximately 4.3 times that of a straight-leg arch.
- (6) The steel stress values at the dangerous section of an arch at critical loading moments A and B are shown in Figures 9f and 9g. The stress on the inner and outer sides of the dangerous section are all compressive stresses, which indicates that the critical section is subjected to a compression-bending inner force with a lower eccentricity, which is different from the state for the straight-leg arch. The axial stress of the steel tube in the crown section at critical loading moment A decreases rapidly as the arch radius increases, and the stress values in Schemes 19 and 20 are considerably lower than the steel yield strength, which is still in the elastic state. The above-mentioned results indicate that the arch failure form of a circular arch can be recognized as the instability at the branch point; Schemes 16–18 exhibit elastoplastic or plastic instability, and Schemes 19–20 exhibit elastic instability.

4.3 Influence law of the lateral pressure coefficient

Figure 10 shows the main results for Schemes 21–25 with the different lateral pressure coefficients.

- (1) The deformation forms of each arch with a lateral pressure coefficient larger than 1 are basically the

Figure 9: Typical results for Schemes 16–20 with the different arch sizes. (a)–(c) Stress contours and deformed shapes of the arches; (d) Load–radial displacement curves; (e) Characteristic loads influenced by the arch size; (f) Axial stress at the dangerous section, influenced by the arch size; (g) Ratio of the axial stress at the inner and outer sides of the dangerous section, influenced by the arch size.

same, and the shape corresponds to a vertical ellipse.

- (2) The forms of the load–radial displacement curve of each scheme are nearly identical and can be divided into three stages, which is the same as that for Scheme 4, as described in Section 1.3. The distance between Points A and B in Scheme 21 ($\lambda = 1$) is considerably smaller than those of the other schemes.
- (3) Figure 10e shows the critical load curves influenced by the lateral pressure coefficient. The comparison indicates a small difference in the load values at

points A, B and C. The arch bearing capacity decreases as the lateral pressure coefficient increases, and the bearing capacity when the lateral pressure coefficient is not 1 is considerably smaller than those of the arches under a uniform pressure.

- (4) The stress values of the crown and bottom sections are, respectively, similar and higher than those for the other sections, and they are both dangerous sections for the arch.
- (5) The steel stress values of the dangerous section of an arch at critical loading moments A and B are shown

Figure 10: Typical results for Schemes 21–25 with the different lateral pressure coefficients. (a)–(c) Stress contours and deformed shapes of the arches; (d) Load-displacement curves; (e) Characteristic loads influenced by the lateral pressure coefficient; (f) Axial stress at the dangerous section, influenced by the lateral pressure coefficient; (g) Ratio of the axial stress at the inner and outer sides of the dangerous section, influenced by the lateral pressure coefficient.

in Figures 10f and 10g. Except in the case of Scheme 21, the stresses on the outer and inner sides of the dangerous section are all tensile and compressive stresses, respectively, which indicates that the dangerous sections are subjected to the compression-bending inner forces with a large eccentricity, which is similar to the case for a straight leg arch and considerably different from the case of a circular arch under a uniform pressure. The steel stress is similar to the yield strength when the arch reaches the

elastic limit, which indicates that the steel on the compressed side of the dangerous section yields before the arch reaches the elastic limit. At Point B, the axial strain on the steel tube in the above position ranges from $-3100\mu\epsilon$ to $-6900\mu\epsilon$. The abovementioned analysis results indicate that the arch failure modes of the circular arch correspond to the instability at the extreme point caused by the deterioration of the section strength under a load when the lateral pressure coefficient is not 1.

Figure 11: Field practice of the CFST arch support with the arch locking bolts.

5 Summary of the failure modes and recommendations for field practice

The failure modes of the CFST arches in the above tests can be summarized as follows.

5.0.1 Straight-leg arch

All the failure modes of the straight-leg arches correspond to the instability at the extreme point caused by the strength deterioration of the dangerous sections. Because the test parameters (150×8 -C40, $R=1.0$ – 4.0 m, $H=0$ – 3.0 m, and evenly distributed load or lateral pressure coefficient 0.5 – 1.5) basically correspond to the parameters of the field practice, we can conclude that the failure modes of the straight-leg arches are all due to instability at the extreme point caused by the strength deterioration.

5.0.2 Circular arch

The failure modes of the circular arches (150×8 -C40) when the lateral pressure coefficient is not 1 all correspond to the instability at the extreme point caused by the strength deterioration of the critical sections. The failure mode of a

circular arch under a uniform load is typically the instability at the branch point; however, the failure mode is elastic instability when the arch radius is larger than 2 m, and elastoplastic or plastic instability serve as the failure modes in other cases.

Based on the abovementioned results, the following suggestions are proposed for the optimization of the arch support design.

- (1) The height of the straight wall should be reduced as much as possible for the straight-wall semicircular roadways.
- (2) The arch locking rock bolts should be set on the dangerous sections (generally near the leg top-end) for the straight-wall semicircular roadways to prevent local failure, thus ensuring the global bearing capacity of the arches.
- (3) Because the rock roadways with high stresses or a weak cementation are always subjected to a large surrounding rock pressure, the cross-section shape of the roadway should be first optimized according to the roadway demands and ground stress conditions. The circular shape, three-core arch shape and curved-leg semicircular shape are preferable under the above conditions.
- (4) For the circular soft rock roadways under an approximate water pressure, more attention should be focused on the global stability than on the local strength of the supporting arch.

The Zhaolou coal mine is a typical 1000 m deep mine in China. The objective roadway of this mine was excavated, and it was composed primarily of mudstone with an inferior rock quality; this roadway was further weakened by a fault. The roadway was originally supported with a U-steel arch + rock bolts + shotcrete support system, and it experienced a critical deformation failure. To solve this problem, the following CFST support scheme was designed: the width of the supporting arch was 5 m, and the height was 4.3 m; the height of the leg was 1.8 m, the row spacing was 1 m, and the arch cross-section parameters were the same as those in the simulation tests. The preceding studies indicated that an arch with high legs has a high risk of leg bending failure. Therefore, the arch locking bolts were set at the leg top-end point of the arch in this scheme, as shown in Figures 11a and 11b. Figures 11c and 11d show the roadway 1.5 y after the excavation and support, and the monitoring data shows that the average deformation of the roadway in the new arch supporting system after 157 days was 15.7 mm, which is only 28% that of the original supported roadway.

6 Conclusions

- (1) The bearing capacity of a straight-leg arch decreases as the arch leg height (except for $H=0$) and arch size increase. A smaller arch leg height corresponds to a larger arch size and larger influence. The arch bearing capacity first increases and later decreases as the lateral pressure coefficient increases, and this capacity reaches its maximum value when the coefficient reaches 0.75. The arch failure modes of the straight-leg arches with the commonly used parameters ($H=0-3.0$ m, $R=1.0-4.0$ m) and lateral pressure coefficient ($\lambda=0.5-1.5$) correspond to the instability at the extreme point caused by the deterioration of the strength, and these modes are not influenced by the arch leg height, arch size or lateral pressure coefficient.
- (2) The failure modes of the circular CFST arches when the lateral pressure coefficient is not 1 all correspond to the instability at the extreme point caused by the section strength damage, while the failure modes under a uniform pressure are recognized as the instability at the branch points. In addition, the failure modes correspond to the elastic instability in schemes with a radius larger than 2 m, and the modes correspond to the elastoplastic or plastic instability in the other cases. The bearing capacity decreases remarkably as the arch size increases, and

it decreases as the lateral pressure coefficient increases.

- (3) The bearing capacity of the circular CFST arches (Scheme 18) is approximately 4.3 times that of a straight-leg arch (Scheme 8). The bearing capacity of a circular arch is higher than that of a straight-leg arch with the same section under a uniform pressure.
- (4) Several recommendations for the field practice were proposed, and a field application in a typical kilometer depth roadway was performed. The monitoring data indicated that the average deformation of the roadway supported with the CFST arch system is only 28% that of the original U-steel scheme.

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