Scientific paper

Static and Dynamic Performances of Concrete-Filled Braided CFRP Tubular Protective Structures

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Abstract

In this research, concrete-filled braided carbon fibre-reinforced polymer (CFRP) tubular beams and arches were constructed. CFRP tubes and concrete-filled CFRP tubular cylinders were compressed to investigate the constraint effect of the CFRP tube on the concrete. Long and short beams were bent to reveal the deformation styles and the failure modes controlled the flexure and the shear, respectively. All of these experiments on cylinders and beams supply basic data to judge the failure load of the concrete-filled tubular arch. Blast resistance of the concrete-filled tubular arch was investigated by explosion experiments and evaluated by the residual static load-carrying capacity of the exploded arches. Constrained by the CFRP tube, the protective arch has excellent blast resistance. A hybrid compression-bending failure criterion was proposed to estimate the static structural performance, and the prediction is consistent with the experiment.

1. Introduction

Advanced fibre-reinforced polymers (FRPs) have been used to strengthen and retrofit concrete structures (Lu et al. 2006; Lokuge et al. 2019) due to their advantages of low weight-to-strength ratio, short installation period, high corrosion resistance, minimal intervention upon the concrete structure and adaptability to curved surfaces (Sun et al. 2017; Mohammedameen et al. 2019; Cousin et al. 2019). Concrete arches have been widely applied in underground protective constructions, and spalling and tensile cracks are two common damage modes, which can be effectively addressed by the FRP strengthening technique (Tao et al. 2011; Wang et al. 2018a; Skuturna and Valivonis 2016; Wang et al. 2018b).

A large number of studies have applied FRPs to strengthen concrete and masonry arch structures in the last 20 years. Hamed et al. (2015) revealed that FRP application leads to a stable debonding mechanism of the strengthened concrete arch. Caporale et al. (2013; 2014) proposed a numerical procedure to estimate the ultimate load of multi-span masonry arch structures strengthened with externally bonded FRP. In previous studies (Zhang et al. 2015; Wang et al. 2017), the performance of CFRP strengthened concrete arches, as shown in Figs. 1(a) and 1(b), was investigated. The FRP can effectively improve the load-carrying capacity and decrease the structural deformation of the concrete arches (Buchan and Chen 2007; Bertolesi et al. 2016). FRP application could shift the cracks to a segment without FRP strengthening (Foraboschi 2004).

Recent studies also reveal that the FRP strengthening method can also improve the blast resistance of concrete protective structures (Chen et al. 2015; Xie et al. 2014; Chen et al. 2014). Usually, the FRP strips are bonded to the internal surface of the arch and can restrict the development of cracks and spalling. However, delamination from the concrete also weakens the strengthening effect.

How to constrain concrete and prevent spalling and fracture under an explosion has become a hot topic in the design of concrete protective structures. Concrete-filled FRP tubes (CFFTs) is one option. Hadi et al. (2016) revealed that CFFTs perform significantly better than conventional reinforced concrete (RC) structures. The filled concrete will prevent local buckling of the FRP shell (Abouzied and Masmoudi 2017). Furthermore, the FRP shell can enhance the rigidity and improve the concrete strength through ambient constraints. Dagher et al. (2012) proposed a concrete-filled FRP tubular (CFFT) arch structure for bridge construction. The composite arch structure without steel reinforcements exhibits excellent performance. Encased by FRPs, the load-carrying capacity and ductility of the concrete structures are significantly enhanced. The excellent anti-explosive performance of encasing a reinforced concrete member with a glass fibre-reinforced polymer (GFRP) tubular beam

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was verified through a close-in explosion (Qasrawi et al. 2015).

In this research, the static and dynamic performances of concrete-filled tubular structures are investigated to suggest a failure criterion for the tubular protective arch, as shown in Fig. 1(c).

### 2. Manufacture

A description of the symbols used in this article is shown in Table 1.

#### 2.1 CFRP tube

As shown in Fig. 2(a), a three-dimensional (3D) braided textile casing was used to form the tubular shell composed with the epoxy resin. The carbon fibre textile casing was braided by an industrial prototype made up of five concentric crowns of 64 horn gears. As shown in Fig. 2(b), the initial braiding angle $\beta$ is ±45° for all braids. After braiding, the angle is changeable because of the basic quadrilateral braid unit, which allows the 3D braided casing to have a scalable circular cross-section. The carbon yarns used for the 3D braiding casing consist of approximately 12K carbon fibres. The tensile strength and the modulus of the carbon yarns are 3.4 GPa and 230 GPa, respectively.

Fabrication of the CFRP tube is not complex, as shown in Fig. 3. First, the rubber mould was inflated and

![Image](image1.png)

**Fig. 1** Different ways to construct composite concrete arches: (a) CFRP bonding and steel anchoring technique to strengthen a reinforced concrete (RC) arch, (b) CFRP bonding and wrapping technique to strengthen a plain concrete arch, and (c) a concrete-filled prefabricated CFRP tubular arch (composite tubular arch).
sprayed with a release agent on its outer surface. Second, one layer of the textile was jacketed onto the surface of the mould. Third, the textile was impregnated with the formulated epoxy resin. Then, the second and third steps were repeated three times, and the tube was cured at room temperature for 24 hours. Finally, the CFRP tube was demoulded from the rubber mould. The fabricated CFRP tube has an outer diameter of 130 mm, and the average fibre fraction in volume, \( \rho_f \), is 21.9\%, which is slightly small.

The equivalent axial elastic modulus of the CFRP tube, \( E_{f\beta} \), and the radial elastic modulus, \( E_{f\theta} \), are given by Hualin and Wei (2006),

\[
E_{\beta} = \rho_f E_f \cos^4 \beta + (1 - \rho_f) E_m
\]

and

\[
E_{\theta} = \rho_f E_f \sin^4 \beta + (1 - \rho_f) E_m
\]

respectively, where \( E_f \) is the elastic modulus of the carbon fibre and equals 230 GPa. \( E_m \) is the elastic modulus of the matrix and equals 2.64 GPa. The axial elastic modulus of the CFRP tube is 14.65 GPa theoretically, which is the same order of magnitude as the elastic modulus of concrete in this research. The radial elastic modulus is still 14.65 GPa theoretically. In this manner, the CFRP tube and concrete materials can co-deform well when stressed.

### 2.2 CFRP tubular arch

As shown in Fig. 1(c), the diameter of the circular cross-section of the CFRP tubular arch is 160 mm. The light hollow CFRP tubular arch can be transited and erected easily. The hollow CFRP tubular arch was filled with concrete. To support and erect the arch, two concrete piers were made to enclose the two arch feet. There are no steel bars within the arch structure.

### 3. Axial compression behaviours

#### 3.1 Axial compression performance of the CFRP tube

Both the diameter and the length of the CFRP tube are 130 mm. The tube thickness is 5 mm. The tube was compressed at a loading rate of 0.2 mm/min. As shown in Fig. 4, the CFRP tube deforms elastically before reaching the peak load of 122.88 kN, and then local buckling initiates from the upper end of the tube, accompanied by a gradually descending deformation curve, as shown in Fig. 4. The strength of the CFRP tube is 86.0 MPa.

#### 3.2 Axial compression performance of the concrete-filled tube

Filled with concrete, the cylinder has a different deformation style, as shown in Fig. 5. The size of the composite cylinder is the same as that of the CFRP tube. Under axial compression, the concrete and the CFRP shell jointly resist the compression and improve the peak load \( P_{\text{max,c}} \) to 593.64 kN, at which point the CFRP tube...
begins to buckle. Gradual crushing of the confined concrete produces a long deformation platform. The mean crushing force (MCF) remains 367.17 kN until the structural failure caused by the CFRP splitting when the deformation reaches 54.70 mm.

An axial compression test was also conducted on the internal concrete cylinder. The peak load is 349.11 kN, and the crushing is brittle, as shown in Fig. 6(a). The compression strength of the concrete is 30.87 MPa. The linear summation of the peak loads of the concrete and the CFRP tube is 471.99 kN, which is much smaller than the peak load of the composite cylinder. The most important aspect of the composite effect is that the member appears plastic stage and the collapse deformation extends to 54.70 mm, close to seven times those of the CFRP tube and the concrete cylinder. The energy absorption of the composite cylinder is 20.76 kJ, over 11 times the summation of those of the CFRP tube and the concrete, which is only 1.85 kJ, as shown in Fig. 6(b).

### 3.3 Composite effect

The comparison indicates that the composite cylinder has a higher peak load and much better energy-absorbing performance than the sum of the CFRP tube and the internal concrete cylinder, as shown in Fig. 6(b). The CFRP tubular shell has a great contribution to the high load-carrying capacity of the composite cylinder, especially to the load of the deformation plateau. Two composite effects are revealed. The first is the interaction between the CFRP shell and the core concrete. The core concrete improves the buckling load of the CFRP shell through side supports, while the shell improves the compressive strength of the concrete through the...
three-dimensional compression stress state. The interaction improves the peak load at a magnitude of 25.77%. The second composite effect is from the constraint of the CFRP shell on the core concrete. The buckled shell has little load resistance, but it supplies circumferential constraints to the core concrete, which resist the shearing or splitting collapse of the concrete and maintain the load until the CFRP shell tears. Due to the second composite effect, the deformation plateau is 54.70 mm, and the MCF is 367.17 kN, close to the triaxial compression strength of the core concrete.

A simplified theory is proposed to predict the structural performance of the composite cylinder. Taking the composite cylinder as a whole, the assumed stress-strain relationship of the composite cylinder is as shown in Fig. 7(a). The radial strain of the concrete is

$$\lambda_r = \frac{\sigma_r}{E_{cc}} - \frac{\mu_c}{E_{cc}} (\sigma_r + f_{cc,c})$$

(3)

where $\sigma_r$ represents the confined compressive stress on the core concrete, while $E_{cc}$ and $\mu_c$ represent the elastic modulus and Poisson’s ratio of the core concrete. $f_{cc,c}$ is the strength of the concrete that is constrained by the CFRP tube.

The CFRP tube is considered as a thin-walled cylinder subjected to internal pressure, as illustrated in Fig. 7(b), and the following results are obtained:

$$\sigma_\theta = -\lambda_r E_{f\theta}$$

(5)

where $\sigma_\theta$ represents the tangential tensile stress of the CFRP tube, $r$ is the radius of the cross-section of the core concrete, and $t$ represents the thickness of the CFRP tube. The triaxial compressive strength of the confined concrete $f_{cc,c}$ is suggested as (Cai and Jiao 1984)

$$f_{cc,c} = f_{cc}(1+1.5\frac{\sigma_r}{f_{cc}} + 2\sigma_r / f_{cc})$$

(6)

and by combining Eqs. (3) to (6), the results are given by

$$\sigma_r = \frac{\sqrt{q^2 - 4pn - q^2}}{4p^2}$$

(7)

with

$$p = rE_{cc} + tE_{f\theta}(1 - 3\mu_c),$$

$$q = -1.5tE_{f\theta}H\sqrt{q^2},$$

$$n = -tE_{f\theta}Hf_{cc}$$

(8)

The maximum stress $\sigma_r$ and the elastic modulus $E_s$ of the composite cylinder are given by

$$\sigma_r = \frac{f_{cc,c}A_c + \sigma_\theta A_f}{A_c + A_f}$$

(9)

and

$$E_s = \frac{E_{cc}A_c + E_{f\theta}A_f}{A_c + A_f}$$

(10)

where $A_c$ and $A_f$ represent the cross-sectional areas of the internal concrete cylinder and the external CFRP tube, respectively. $\sigma_{cc}$ represents the maximum compressive stress of the CFRP shell. In this study, $E_{f\theta} = E_{fc} = 14.65$ GPa, $E_{cc} = 30$ GPa, $\mu_c = 0.2$, $f_{cc} = 30.87$ MPa, and $\sigma_{cc} = 85.96$ MPa. The results are $f_{cc,c} = 35.64$ MPa, $\sigma_r = 43.33$ MPa, and $E_s = 27.65$ MPa. The calculated peak load of the cylinder is 578.58 kN, which is close to the tested value in this research.

Because the internal core concrete was tightly
wrapped by the external CFRP tube at the yield stage in the experiment, it is assumed that the concrete can contribute its maximum compressive stress in this stage. The folding of the CFRP tube wall makes it inherently incapable of bearing load. Thus, the yield stress of the composite cylinder has the following expression:

\[
\sigma_p = f_{cc} \frac{A_c}{A_c + A_f}
\]

(11)

In this case, \(\sigma_p = 30.19\) MPa. The yield load of the cylinder is 403.04 kN theoretically.

Assuming that the volume of the cylinder remains the same, as shown in Fig. 7(c), the axial strain \(\varepsilon_u\) can be expressed by the circumferential strain \(\varepsilon_\theta\) as

\[
\varepsilon_u = 1 - (1 + \varepsilon_\theta)^{-2}
\]

(12)

The ultimate circumferential strain of the textile shell \(\varepsilon_\beta\) is determined by the inclination of the fibres \(\beta\) as

\[
\varepsilon_\beta = 1 - \cos \beta
\]

(14)

and the elastic deformation is neglected. The ultimate compression strain of the composite cylinder \(\varepsilon_{uc}\) is given by

\[
\varepsilon_{uc} = 1 - (2 - \cos \beta)^2
\]

(15)

When \(\beta = 45^\circ\) and \(\varepsilon_{uc} = 0.402\), the crushing displacement of the cylinder is 52.22 mm theoretically, close to the tested value in Fig. 5(a). The maximum axial force that can be withstood by the subsequent arch, \(N_y\), can be obtained by

\[
N_y = \sigma_y A_f + f_{cc} A_c + 1.5 \sqrt{\sigma_y} f_{cc} A_c + 2\sigma_y A_c
\]

(16)

In this case, the radius of the cross-section of the subsequent arch is 80 mm, and the thickness of the CFRP tube is 5 mm. The maximum axial force of the arch is 612.98 kN theoretically.

4. Flexural and shear behaviours of tubular beams

Two composite beams were fabricated for three-point bending tests, as shown in Figs. 8 and 10. The diameter of the circular section is 130 mm. The length of the long beam, B1, is 1040 mm. The short beam, B2, is 390 mm long. The experimental protocols of the two beams are shown in Figs. 8(a) and 10(a), and the loading rate is 1 mm/min. The spans of the two beams are 760 mm and 160 mm, respectively. B1 and B2 have different deformation characteristics and failure modes decided by the span.

B1 has a longer span, and the beam has a typical flexural deformation style, as shown in Fig. 8(c). After elastic deformation, the compressed CFRP shell buckles, and the compressed core concrete flows into plastic crushing. The peak force is 72.6 kN. When the stretched CFRP shell ruptures, the beam completely loses its load-carrying ability. The rupture deflection is 28.3 mm.

According to the plane cross-section assumption, the state of the mid-span cross-section when the CFRP shell is just buckling is analysed, as shown in Fig. 9. The stress on the cross-section can be decomposed into the stresses distributed on the CFRP tube and the core concrete. On the CFRP tube, the tensile force \(F_{ft}\) and the compressive force \(F_{fc}\) are

\[
F_{ft} = \frac{2}{b} \int_0^{\pi/2} r E_{ft} \varepsilon_{\beta} (r + r \cos \alpha - b) d\alpha
\]

(17)

and

\[
F_{fc} = \frac{2}{b} \int_0^{\pi/2} r E_{fc} \varepsilon_{\beta} (b - r \cos \alpha) d\alpha
\]

(18)

respectively, where \(E_{ft} = E_{fc}\) is the equivalent axial tensile elastic modulus of the CFRP tube, \(b\) is the depth of the compression zone, and \(\varepsilon_{\beta}\) is the buckling strain of the CFRP shell. The compressive force in the elastic zone

Fig. 8 (a) Three-point bending experiment of the concrete-filled CFRP tubular B1, (b) flexural failure and (c) load-displacement curve.
and the compressive force in the plastic zone $F_{cc2}$ of the core concrete are given by

$$F_{cc1} = \frac{2}{b} \int_{r-b}^{r+b} F_{cc} E_{cc} \varepsilon_{cc} (x-r+b) \sqrt{(r-t)^2-x^2} \, dx$$

(19)

$$F_{cc2} = \frac{2}{b} \int_{r-b}^{r+b} f_{cc} \sqrt{(r-t)^2-x^2} \, dx$$

(20)

and

$$F_{cc} = F_{cc1} + F_{cc2}$$

(21)

where $\varepsilon_{ccy} = f_{cc} / E_{cc}$ is the elastic strain limit of the core concrete and $F_{cc}$ is the total compressive force of the core concrete. The equilibrium equation of the axial forces of the composite beam B1 at the mid-span is given by

$$F_{cc} + F_{fc} - F_{b} = 0$$

(22)

Combining Eqs. (17) through (22), $b = 44.2$ mm. In this case, the maximum bending moment of the composite beam, $M_y = 13.79$ kN·m. When the CFRP shell buckles, the bending moments contributed by the CFRP tube, $M_f$, and by the core concrete, $M_c$, are given by

$$M_f = \frac{2}{b} \int_{r-b}^{r+b} E_f \varepsilon_{fa} (r + r \cos \alpha - b)^2 \, d\alpha$$

(23)

$$M_c = \frac{2}{b} \int_{r-b}^{r+b} E_c \varepsilon_{cc} (b - r \cos \alpha)^3 \, d\alpha$$

(24)

In the three-point bending test of the composite beam B1, when the CFRP shell buckles, the load value is almost the same as that when the CFRP shell ruptures. Assuming that the ultimate bending moment of the composite beam $M_y$ is equal to the bending moment of the composite beam when the CFRP shell buckles, it is given by

Fig. 9 (a) The strain distribution of the mid-span cross-section, (b) stress distribution on the CFRP tube and (c) stress distribution on the core concrete.
When $b = 44.2$ mm, $r = 65$ mm. The ultimate bending moment of the composite beam is 11.84 kN·m theoretically, close to the experimental value. When $r = 80$ mm, $t = 5$ mm, the depth of the compression zone is 49.76 mm, and the maximum bending moment of the composite beam is 20.88 kN·m.

B2 has a shorter span, and the beam has a typical shearing deformation style, as shown in Fig. 10. After elastic deformation, the shear zone forms from the left support to the concentrated loading. When the CFRP shell ruptures by the shearing force, the beam completely loses its load-carrying ability. The peak load is 302.9 kN, which is much larger than that of B1.

It can be concluded from the experiment that the CFRP tube plays an important role in constraining the internal concrete, making the concrete-filled CFRP tube a good structural unit. The maximum shear force, $Q_y$, is given by

$$Q_y = \frac{P_{\text{max,s}}}{2}$$

$P_{\text{max,s}}$ is the ultimate load of the composite beam with a small span in the form of shear failure under the three-point bending test and equals 302.9 kN.

5. Static and dynamic behaviours of a concrete-filled tubular CFRP arch

5.1 Dynamic structural responses

A concrete-filled CFRP tubular arch was fabricated to resist the explosion, as shown in Fig. 11. The radius of the circular arch is 1260 mm, and the opening angle of the arch is approximately 104°. Two concrete blocks were used to prevent movement of the two piers. The constraint at the arch feet can be simplified as a pin-joint. Four close-in explosions were imprinted on the tubular arch in sequence, as listed in Table 2. In the four explosions, the weight of the TNT charge changes from 1 kg, 2 kg, and 4 kg to 4 kg. The standoff distance from the apex of the arch to the TNT block is 1.2 m for the first three explosions and 0.6 m for the last one. Then, the scaled distance ranges from 1.2 m/kg$^{1/3}$ to 0.38 m/kg$^{1/3}$ in this research.

To record the dynamic displacements along the arch,
three LVDTs, L1, L2, and L3, were placed under the arch, as shown in Fig. 11. One is located at the mid-span, and the other two are located at the haunch of the arch. The sampling frequency of the LVDTs is 20 K/s. Under the 1st and the 2nd experiments, the arch has little damage, as listed in Table 2. In the 3rd explosion, cracks appear on the two piers, as shown in Fig. 12(a). In the 4th explosion, the scaled distance is reduced to 0.38 m/kg \(^{1/3}\), and the standoff distance is only 0.6 m. The CFRP shell facing the explosion at the apex was burned off, and the concrete pier at right side was crushed, as shown in Fig. 12(b).

The dynamic displacement curves are displayed in Fig. 13. The maximum and residual displacements of the first three explosions are listed in Table 3. The negative data represent downward displacements. Under explosion, the arch first moves downwardly and then rebounds upward. Dynamic displacements went downward first and then rebounded upward. Because the arch was not fixed on the ground, the rebound was much larger than the flexural displacement. Because the TNT charge is not massive, the CFRP shell is not damaged. The core concrete may produce damage as the residual deformation at LVDT2 reaches 3.5 mm in the 2nd explosion. Free vibration information is also clearly displayed in the 4th explosion. After the shock wave, the periodic and decaying vibration lasts almost one second. The fundamental free vibration frequency is 13.8 Hz.

| Explosion | L1 Maximum | L1 Residual | L2 Maximum | L2 Residual | L3 Maximum | L3 Residual |
|-----------|------------|-------------|------------|-------------|------------|-------------|
| 1st       | 1.5/-0.9   | -0.5        | 1.1/-1.4   | -0.2        | 3.2/-0.9   | -0.1        |
| 2nd       | 3.0/-4.5   | 0.3         | -7.7       | -3.5        | 6.3/-3.1   | -0.3        |
| 3rd       |            |             | 7.6/-1.4   | 0.5         |            |             |

5.2 Damage evaluation through residual static load capacity

The damage of the arch is difficult to judge from the dynamic displacement curves. The arch has no spalling. Cracks in the concrete are not visible. The residual load-carrying capacity of the exploded composite arch may be a good choice. As shown in Fig. 14(a), the arch is subjected to a concentrated load at the vault. The arch is placed on a steel frame, which restricts the lateral displacement of the feet, and the arch is pin-jointed. The arch is monotonically compressed to collapse at a loading rate of 1 mm/min.

The first failure is caused by the tensile rupture of the CFRP shell and appears at the vault where the bending moment is the maximum. The rupture is abrupt with significant cracking of concrete. Following this failure at the vault, the arch remains stable, acting effectively from a two-hinged arch to a three-hinged one.

As the loading continues, the arch produces great deflection while the load changes slightly. The internal forces of the arch are redistributed, and the location of the maximum bending moment shifts to one of the shoulders. Once the outer FRP shell at the shoulder ruptures, a four-hinge collapse mechanism forms, and the arch is destroyed and broken into three segments.

For the exploded arch, the first peak load is 126.6 kN, and the ultimate load is 119.7 kN. For the reference arch, the first peak load and the ultimate load are 202.3 kN and 196.5 kN.
171.7 kN, respectively, as shown in Fig. 14(b). The peak load of the exploded arch is 62.6% of that of the reference arch, and the ratio of the ultimate load is 69.7%.

Because the damaged arch had repeatedly been subjected to explosions, the external CFRP tube and the internal core concrete may be damaged, especially at the vault. In other positions of the arch, there are some subtle damages, which may not be visible but weaken the arch. Through residual load-carrying capacity, the damage degree of the exploded arch can be evaluated quantitatively.

It is also found that the load-carrying capacity of the damaged arch structure is still considerable, indicating that the CFRP tubular arch filled with concrete has relatively excellent blast resistance.

5.3 Failure criteria of the tubular arch

(1) The first failure load

The structural model of the two-hinged arch is shown in Fig. 15(a). The arch is simply supported at both ends and compressed by a concentrated load $P$ at the vault. According to the force method, the model can be turned into a determinate structure with a horizontal reaction ($X_1$) at the foot. Using the force method, $X_1$ can be computed in terms of $P$ as follows:

$$X_1 = \frac{PR^2A(\sin^2\theta + 2\cos\theta - \theta\sin2\theta - 2\cos^2\theta)}{R^2A(2\theta + 4\cos^2\theta - \sin 2\theta) + (2\theta + \sin 2\theta)I} \tag{27}$$

and

Fig. 13 Dynamic displacements of the composite arch under (a) the first explosion, (b) the second explosion, and (c) the third explosion, and (d) displacement at L1 under the second explosion.

Fig. 14 (a) Experimental protocol and (b) load-deflection curve of the composite arch.
\[ \eta = \frac{R^2 A \left( \sin^2 \theta + 2 \cos \theta - \theta \sin 2 \theta - 2 \cos^2 \theta \right)}{R^2 A \left( 2 \theta + 4 \cos^2 \theta - 3 \sin 2 \theta \right) + \left( 2 \theta + 2 \theta \right) I} \]  

(28)

where \( 2 \theta \) is the central angle of the arch. \( A \) and \( I \) are the area and the second moment of area of the cross-section. The horizontal reaction is equal to \( X_1 \). The axial force \( N_1 \), bending moment \( M_1 \), and shear force \( Q_1 \) of the cross-section at the vault can be expressed by

\[ N_1 = X_1 = P \eta \]  

(29)

\[ M_1 = PR \left[ \frac{1}{2} \sin \theta - \eta (1 - \cos \theta) \right] \]  

(30)

and

\[ Q_1 = P / 2 \]  

(31)

respectively.

(2) The second failure load

Following the failure at the vault, the arch acts effectively as a three-hinged member, as shown in Fig. 15(b). The fourth hinge will appear randomly at one of the shoulders of the arch. Before the second failure, the axial force \( N_2 \), the bending moment \( M_2 \), and shear force \( Q_2 \) of the cross-section at the shoulder can be expressed as

\[ N_2 = \frac{P}{2 \sin(\theta/2)} \]  

(32)

\[ M_2 = \frac{PR(1 - \cos(\theta/2))}{2\sin(\theta/2)} \]  

(33)

and

\[ Q_2 = 0 \]  

(34)

The stress state of the cross-section of the vault is similar to that of the CFRP tubular concrete cylinder under eccentric compression, as shown in Fig. 16(a). When the axial force is small, the failure of the cylinder is tension-controlled, as shown in Fig. 16(b). When the axial force is great enough, the failure of the cylinder is compression-controlled. With the increase of the axial force, the failure of the cylinder transitions from flexural failure to compression failure. Figure 16(b) depicts the envelope diagram of \((N_u, M_u)\) of the reinforced concrete cylinder under eccentric compression. The overall trend of the failure envelope capacity of the CFRP tubular concrete arch under concentrated load is consistent with the envelope diagram in Fig. 16(b).

The relationship between \( M_y \) and \( N_y \) is given by

\[ a_0 M / M_y + (N / N_y - a_1) y^2 = a_2 \]  

(35)

where \( a_0, a_1, \) and \( a_2 \) are three coefficients to be determined. In the \((N/N_y, M/M_y)\) coordinate system, the envelope is known to pass through the coordinates \((0, 1)\) and \((1, 0)\), so only one more point needs to be determined.

In this paper, the coordinate of the third point is de-
terminated by \((N_1/N_y, M_1/M_y)\) when double-hinged arches fail and \((N_2/N_y, M_2/M_y)\) when triple-hinged arches fail. After solving, we get Eqs. (36) and (37). Here, we can also consider that \(M/M_y\) is maximized at \(N/N_y = 5/18\) and \(N/N_y = 8/25\). The failure criterion curves for the CFRP tubular concrete arch are simplified as

\[
\frac{4}{9} \frac{M}{M_y} \left( \frac{N}{N_y} - \frac{5}{18} \right)^2 - \frac{169}{324} = 0 \quad \text{(36)}
\]

and

\[
\frac{9}{25} \frac{M}{M_y} + \left( \frac{N}{N_y} - \frac{8}{25} \right)^2 - \frac{289}{625} = 0 \quad \text{(37)}
\]

The failure lines are shown in Fig. 17. The tested axial force and bending moment at the failure time of the arch are depicted in Fig. 17. Wang et al. (2019) made two types of components, i.e., composite arches composed of three layers of CFRP braided pipes and one layer of CFRP braided pipe. The relevant test data are also marked in Fig. 17. The first and second failure loads calculated from the failure criterion are listed in Table 4. The calculated first failure load of composite arches composed of three layers of CFRP braided pipes is 11.4% lower than that of the experimental value, and the load of composite arches composed of one layer of CFRP braided pipe is 24.5% higher than that of the experimental value. For the second failure load, the theoretical value is only 4.9% higher for the composite arch with three layers of CFRP braided pipes and is only 7.0% higher for the composite arch with one layer of CFRP braided pipe. The error of theoretical calculations is acceptable.

### 6. Conclusions

The proposed novel composite tubular technique is an efficient way to fabricate CFRP tubes, concrete-filled CFRP tubes, and CFRP tubular composite arches. The experiments verified the excellent static and dynamic structural behaviours of those composite structures. This study specifically concludes the following:

1. The benefit of CFRP confinement is that the load-carrying capacity of the CFRP tubular composite cylinder was significantly improved compared with the concrete cylinder and the CFRP tube. The CFRP tubular composite beams have shown excellent mechanical properties in three-point bending tests.

2. The CFRP tube plays an important blast protective role in the composite tubular arch when subjected to the explosion. Its effect in controlling the concrete’s cracking and improving structural stability is remarkable.

3. Though the composite arch structure had been subjected to close-in explosions, the load-carrying capacity of the damaged arch is still considerable, the composite arch is suitable for use in protective structures.

4. The simplified theoretical analysis method can effectively evaluate the failure behaviours of composite arches, but there are still some factors that have not been considered.

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