Assessment on the Progressive Collapse Resistance of a Long-Span Curved Spatial Grid Structure with Main Trusses

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1. Introduction

Nowadays, the structural collapse accidents occur frequently in China, resulting in huge casualties and economic losses, especially the collapses of public buildings with large pedestrian flow and complex structural forms, such as airport terminal buildings, railway station buildings, etc. The research on the progressive collapse of building structures began from the collapse of Ronan Point Apartment Building in England in 1968, caused by gas pipe explosion. Then, the collapse of Alfred P. Murrah Federal Building in America caused by the terrorist attack of car bomb in 1995 marked the beginning of the second research stage of progressive collapse. The third research stage of progressive collapse began from the 9/11 Incident. The Eurocode 1, British code BS8110, U.S. code GSA2003, and DoD2010 specified the structural robustness, building safety classification, AP method, and tensile strength method, respectively (BS EN 1991-1-2005, 2005). Scalvenzi and Parisi (2021) numerically studied the ability of existing reinforced concrete structure to prevent progressive collapse during structural retrofitting. Manzur et al. (2020) quantitatively evaluated the behavior of typical ready-made garment (RMG) building with low-to-moderate concrete strength, focusing on progressive collapse. Parisi and Scalvenzi (2020) numerically investigated the progressive collapse capacity of gravity-load designed, reinforced concrete (RC) building complying with Eurocode 2, considering both simultaneous and sequential removal of ground-floor columns. Alanani et al. (2020), El-desoqi et al. (2020) and Almusallam et al. (2018) conducted the progressive collapse assessments of precast RC beams and joints. As for the steel structures, Rodriguez et al. (2021), Kiakojouri et al. (2020) and Karimian et al. (2019) conducted progressive collapse assessments of steel frames and plates.
China scholars began to conduct related researches at the third research stage of progressive collapse. Zhang and Liu (2007) studied the evaluation methods of structural vulnerability and component importance under accidental loads. Hu and Qian (2008) researched the dynamic effect of a steel frame structure during progressive collapse process, and summarized the influencing laws of dynamic amplification coefficient. Ye et al. (2006) studied the progressive collapse of a RC frame structure. Li et al. (2007) studied the influences of constraint conditions at column ends on the dynamic responses of frame columns under explosive impact. Shi et al. (2007) studied the influence of initial damage on the collapse process of RC frame under explosion using LS-DYNA software. Li (2011), Cai et al. (2011) and Zhou et al. (2010) studied the design method of structural progressive collapse resistance, failure simulation, and the progressive collapse resistance of practical project, respectively. However, the progressive collapse mechanisms of different structural forms are diverse. The current researches mainly focus on the frame structures and simple large-span structures. For the complex structures, especially the complex large-span spatial structures composed of curved space grids and trusses, the research on the mechanism of progressive collapse resistance is still insufficient.

In this paper, the AP method was used to evaluate the progressive collapse resistance of the terminal building of Zhongchuan Airport in China, which is a long-span spatial grid structure with main trusses. The finite element model was built using MSC. Marc software adopting the fiber model based on material. The improved method of zoned concept judgment was proposed to preliminarily select the initial failure components. The key components were determined through conducting sensitivity analysis, including the columns supporting the roof, columns of RC frame, and web members of main trusses. The key components

![Figure 1](https://example.com/figure1.png)

**Fig. 1.** Global Layout of Roof Structure: (a) Isometric View, (b) Structural Zones of Middle Part, (c) Profile 1-1, (d) Plane Layouts of Roof and Columns (the columns are shown in green color)
were removed individually to evaluate the progressive collapse resistance of the structure. Finally, the parametric analyses of progressive collapse resistance were carried out.

2. Project Overview

The Zhongchuan Airport is located in Lanzhou City, Gansu Province in China, which was opened to traffic in February 4, 2015. The global layout of terminal building is shown in Fig. 1(a), which can be divided into three parts, including the middle part and the corridors on both sides. In this study, the progressive collapse resistance of the middle part shown in Fig. 1(b) was analyzed, which is a large-span curved spatial grid system with main trusses, including Grid parts A ~ D and Trusses I ~ III. The structural form of the single grid unit is a quadrangular cone with a side length of 3 m and a height of 2 m. The grid members and truss members are made of round steel tubes and square steel tubes, respectively. The sectional view and plane layouts of roof and columns are shown in Figs. 1(c) and 1(d), respectively. As can be seen, the middle part is a super long structure with a length of 336 m and a width of 186 m. The length of cantilever segment at the end of roof is up to 24 m. The columns supporting the roof include CFST columns and steel tube columns, without lateral supports between columns.

In addition, a two-story RC frame is included by the middle part, which can be divided into four independent parts, with the plane dimensions of 105 m × 24 m, 288 m × 86 m, 72 m × 24 m, and 68 m × 79 m, respectively. The heights of the first and second floors are 3.8 m and 7.8 m, respectively. The columns include rectangular RC columns and circular CFST columns. The rectangular RC beams are adopted.

The middle part of the terminal building is characterized by large column spacing, high column height, long cantilever length, and single transmission path of vertical force. The residual structure may not have enough backup load path to bear the load after the initial local damage caused by accident, resulting in progressive collapse. Moreover, the ticket hall with dense crowd is located in the middle part. The loss of life and property will be severe if the progressive collapse occurs. Therefore, it is necessary to analyze the progressive collapse resistance of the structure.

3. Finite Element Model

The finite element model was established using the MSC. Marc software. The floor slabs were simulated using the elastic shell element, which can improve the calculation speed compared with solid element and ensure calculation accuracy. The grid members, including chords and webs, were simulated using the spatial link element, which is characterized with two nodes and three degrees of freedom at each node, without consideration of bending moment. The material-based fiber model with high theoretical accuracy was adopted to simulate the column and beam, which can consider the influences of axial force and bending moment on the sectional hysteretic relationship.

3.1 Material-Based Fiber Model

The fiber divisions of RC component, CFST component, and steel pipe are shown in Fig. 2, with assuming that each fiber was only subjected to axial force, and the deformation and stress characteristics of the cross-section were obtained by conducting integral for each fiber.

3.2 Constitutive Relation of Concrete

The calculation accuracy of fiber model depends on the selection of uniaxial hysteretic relationship. The constitutive relations of concrete and steel in the reference (Ye et al., 2006) were adopted. The behaviors of ordinary concrete, RC, and steel tube confined concrete can be simulated by modifying the ultimate compressive strength $\sigma_u$ and compressive strain $\varepsilon_u$ of concrete. In addition, the hysteresis model of compression constitutive relation of concrete is characterized by origin oriented type, the descending segment of constitutive model is a straight line, and the rising segment is shown in Eq. (1):

$$
\sigma = f_c \left[ 2 \left( \frac{\varepsilon}{\varepsilon_y} \right) - \left( \frac{\varepsilon}{\varepsilon_y} \right)^2 \right],
$$

where $\sigma$ and $\varepsilon$ are the stress and strain of concrete, respectively; $f_c$ and $\varepsilon_y$ are the compressive yield strength and strain of concrete, respectively.

Moreover, the strength enhancement of concrete under the restraint of steel rebar or steel pipe was considered. The RC model proposed by Legeron and Paultre (2003) and the CFST
model proposed by Han (2018) were used to calculate the peak compressive strength \( \sigma_{c0} \) and strain \( \varepsilon_{c0} \) of the concrete fibers of material-based fiber models, as well as the ultimate compressive strength \( \sigma_{cu} \) and strain \( \varepsilon_{cu} \).

The tensile constitutive model of concrete proposed by Lu and Jiang (2003) is adopted, as shown in Eq. (2):

\[
\sigma = \begin{cases} 
E_t \varepsilon & (\varepsilon \leq \varepsilon_{t0}) \\
 f_t \exp[-\alpha(\varepsilon - \varepsilon_{t0})] & (\varepsilon > \varepsilon_{t0})
\end{cases}
\]

where \( E_t \) is the elastic modulus of concrete; \( f_t \) and \( \varepsilon_{t0} \) are the tensile yield strength and strain of concrete, respectively. \( \alpha \) is the coefficient related to the fracture energy of concrete, \( \alpha = f_t l / G_f \)

where \( l \) is the characteristic dimension of concrete, \( G_f \) is the fracture energy of concrete. In case of lack of relevant information, \( \alpha = 4,000 \) is recommended by Lu and Jiang (2003).

### 3.3 Constitutive Relation of Steel

The constitutive model of steel rebar is shown in Eq. (3), and the monotonic loading curve of which is composed of three parts, including two straight line segments and one parabola segment. The Bauschinger effect of rebar is reasonably considered in the reloading path, which can reflect the yield, hardening, and softening effects of rebar.

\[
\sigma = \begin{cases} 
E_s \varepsilon & (\varepsilon \leq \varepsilon_y) \\
f_y & (\varepsilon_y < \varepsilon \leq k_1 \varepsilon_y) \\
k_2 \varepsilon_y & (\varepsilon > k_1 \varepsilon_y)
\end{cases}
\]

where \( E_s \) is the elastic modulus of steel rebar; \( f_y \) and \( \varepsilon_y \) are the yield strength and strain of rebar, respectively. \( k_1 \) is the ratio of hardening initial strain to yield strain, taking 4; \( k_2 \) is the ratio of peak strain to yield strain, taking 25; \( k_3 \) is the ratio of ultimate strain to yield strain, taking 40; \( k_4 \) is the ratio of peak stress to yield strength, taking 1.2.

The ideal elastoplastic bilinear hysteretic model in MSC.Marc software was used to simulate the constitutive model of steel member, as shown in Eq. (4). The symbol interpretations can be referred to the constitutive model of rebar.

\[
\sigma = \begin{cases} 
E_s \varepsilon, (\varepsilon \leq \varepsilon_y) \\
f_y, (\varepsilon > \varepsilon_y)
\end{cases}
\]

### 3.4 Finite Element Model and Verification

The finite element model established by MSC.Marc software is shown in Fig. 3. The Midas model was established simultaneously for comparison and verification. The total weights of Midas and Marc models are 80030 t and 80160 t, respectively, with a difference ratio of 0.16%, which can be ignored. The comparisons of the first ten natural vibration periods and the first three mode shapes calculated by the two models are shown in Table 1 and Fig. 4, respectively. As can be seen from Table 1, the natural vibration periods of the first three orders calculated by Midas model are 1.3575 s, 1.2745 s and 1.2161 s, respectively; the natural vibration periods of the first three orders calculated by Marc model are 1.2549 s, 1.2083 s and 1.1388 s, respectively. The error ratios of periods are -7.56%, -5.19% and -6.36% respectively, which are within reasonable range. In addition, the periods calculated by Marc and Midas models have the same

![Fig. 3. Finite Element Model Established Using Marc Software: (a) 3D View, (b) Top View](image-url)

### Table 1. Comparison of Natural Vibration Period between Marc Model and Midas Model

| Order | Period (s) | Midas model | Marc model | Error (%) | Order | Period (s) | Midas model | Marc model | Error (%) |
|-------|-----------|-------------|------------|-----------|-------|-----------|-------------|------------|-----------|
| 1     | 1.3575    | 1.2549      | -7.56      |           | 6     | 0.8112    | 0.7634      | -5.89      |           |
| 2     | 1.2745    | 1.2083      | -5.19      |           | 7     | 0.6972    | 0.7189      | 3.11       |           |
| 3     | 1.2161    | 1.1388      | -6.36      |           | 8     | 0.6607    | 0.7163      | 8.42       |           |
| 4     | 0.9858    | 0.9434      | -4.30      |           | 9     | 0.655     | 0.7042      | 7.51       |           |
| 5     | 0.9631    | 0.9091      | -5.61      |           | 10    | 0.6072    | 0.7032      | 15.81      |           |

Note: Error = [(value of Marc model–value of Midas model)/value of Midas model] × 100.
change trend. Moreover, as can be seen from Fig. 4, the mode shapes of the first three orders calculated by Marc and Midas models are the same.

4. Progressive Collapse Analysis Method

4.1 Alternate Load Path Method

The AP method was adopted to conduct the progressive collapse analysis. The subroutine “uactive” in Marc software controlling the life and death of element was used to simulate the instantaneous initial failure of component. The element was judged whether failed or not in the calculation process according to the failure criterion of material. Then, the subroutine “uactive” was used to “kill” the failed element. The failure process of related components caused by the overrun of stress and strain due to initial failure was calculated. Finally, the progressive collapse process of the structure was simulated.

4.2 Collapse Criterion and Failure Criterion

The structural progressive collapse and earthquake are minimum probability events, the earthquake was not considered in the progressive collapse analysis. Similarly, the wind load on the structure is small and randomly varying, which was also not considered in the calculation. The load combination of the structure during progressive collapse calculation is as follows:

\[ S = S_{GK} + 0.5 \sum S_{qik} \]

where \( S_{GK} \) is the standard value of permanent load; \( S_{qik} \) is the standard value of live load, including the snow load and the live loads on roof and floor.

In the past, the indirect methods were generally used as the collapse criterion limited by the complex nonlinear dynamic process of structural collapse, such as the interlayer displacement angle exceeding 1/50. However, the indirect criterion cannot reflect whether the structure really collapses or not. Now, the advanced nonlinear analysis tools have been able to accurately simulate the whole nonlinear dynamic process of structural collapse, including material nonlinearity and geometric nonlinearity. Therefore, the definition of structural collapse, “the structure loses the vertical bearing capacity and cannot maintain the living space to ensure the safety of person”, was taken as structural collapse criterion in this study.

As for the failure criterion of member, the ultimate compressive strain of concrete fiber of the material-based fiber model was calculated according to the RC model proposed by Legeron and Paultre (2003). In addition, the ultimate tensile strain of steel rebar fiber was taken as 0.1. For the spatial link element simulating the grid member, the ultimate tensile strain and ultimate compressive strain were taken as 0.1. The members are allowed to maximize the bearing and deformation capacities, the standard value of material strength was adopted (Lu et al., 2008).

4.3 Determination Method of Initial Failure Component

The calculation will be massive if considering the combined initial failure of multiple members for the spatial structures, it is unrealistic to analyze each condition. Therefore, the commonly used method in national codes was adopted, only the single initial failure of one component was considered in each analysis process. Moreover, it is time-consuming and unnecessary to simulate the initial failure of every component because of their different importance to structure. The method of concept judgment plus sensitivity analysis (Legeron and Paultre, 2003) was usually used to select the members having greater impacts on structure after failure, which are called “key components”. Firstly, the relatively important components were preliminarily selected as initial failure components through concept judgment, then the
key components were determined by conducting sensitivity analysis. However, the effectiveness of sensitivity analysis depends on the consistency of structural form. The sensitivity analysis results of different components can only be comparable when the components belong to the same structural form. The spatial forms of different roof parts of the terminal building in this study are different. Therefore, the improved method of zoned concept judgment plus sensitivity analysis was proposed to determine key components. The main steps are as follows:

1. The structure was divided into different areas according to structural systems or spatial forms;
2. The components were divided into different types according to mechanical characteristics;
3. For the different types of components in each area, the preliminary selection of initial failure components was determined by concept judgment;
4. The sensitivity analyses for the preliminarily selected components were conducted, and the key components of each type were determined for each area, which were considered as key components for the whole structure.

### 4.4 Sensitivity Analysis Method

The change of structural response caused by initial failure is defined as sensitivity, which is inversely proportional to structural redundancy (Cai et al., 2011). The sensitivity analysis steps are as follows:

1. The removal of initial failure member was selected as the damage parameter \( \beta \) for the structure;
2. Calculate the sensitivity index \( S_y \) of component \( i \) after applying the damage parameter \( \beta \), namely, the initial failure component \( j \) was removed,
   \[
   S_y = \frac{(\gamma - \gamma')}{\gamma},
   \]
   where \( \gamma \) and \( \gamma' \) are the responses of nodes or elements of the original and damaged structures, respectively.
3. Calculate the importance coefficient \( \alpha' \). When the node displacement is taken as the object of sensitivity analysis, the formula of \( \alpha' \) is as follows:
   \[
   \alpha' = \sum_{j} \left| \frac{S_y}{n} \right|
   \]
   where \( n \) is the total node number of the remaining structure \( j \). In addition, when the overall response of the structure is taken as the object of sensitivity analysis, such as bearing capacity and natural vibration period, the formula of \( \alpha' \) is as follows:
   \[
   \alpha' = \left| S_y \right|
   \]
   where \( S_y \) is the sensitivity index of the whole structure corresponding to damage parameter \( \beta \).

### 5. Results and Discussions of Progressive Collapse Analysis

The components on the vertical load path of the structure were focused on during the selection of key components. Moreover, the single failure of one grid member will not greatly affect the structure because of the high degrees of indeterminacy and redundancy of grid structure, which was ignored during selecting key components. Therefore, the initial failure components of the structure include the columns directly supporting the roof, columns of RC frame, and web members of the main trusses.

#### 5.1 Column Supporting the Roof

##### 5.1.1 Initial Failure Components

The vertical load on the failed column supporting roof will be redistributed to the surrounding columns. The bending moments and shear forces of the grid members close to the failed column will increase and exceed the bearing capacity, resulting in the failures of grid members and even the progressive collapse of the roof. Therefore, the columns supporting the roof should be regarded as key components. During the concept judgment of key columns, the middle structure was divided into three areas as shown in Fig. 5, including Area I, II, and III. The columns with typical positions in each area were preliminarily selected as initial failure columns, including the corner columns, side columns, and inner columns. The cross-sectional sizes and material parameters of the preliminarily selected columns are shown in Table 2, which were numbered as SC1 – SC16, including circular CFST columns and square steel pipe columns.

The sensitivity analyses were carried out for the preliminarily selected columns supporting the roof. The dynamic amplification effect and material nonlinearity were not considered. The object of sensitivity analysis was node displacement. The calculated results of importance coefficients \( \alpha' \) are shown in Table 3. In order to avoid the influence of different roof form in each area, the relatively important columns with the importance coefficients exceeding 1.5 in each area were regarded as the key components, including SC1, SC10, SC11, SC14, and SC15.

##### 5.1.2 Progressive Collapse Analysis Results

The progressive collapse resistance of the structure after removing the key columns SC1, SC10, SC11, SC14, and SC15 individually was studied. The nonlinear dynamic time-history analysis was carried out for the remaining structure. The final deformation of
the structure, distribution of plastic members, and the vertical displacement curve of the node connecting the initial failure column and roof were obtained, as shown in Figs. 6–10.

The final deformation of structure after removing column SC1 is shown in Fig. 6(a). The maximum vertical displacement of the cantilever end close to SC1 is 1.95 m. The members in blue color shown in Fig. 6(b) represent the members entering into plasticity. The vertical displacement curve of the node connecting SC1 and roof is shown in Fig. 6(c). The first 1 s represents the application of initial load on the structure. The column SC1 was removed at 1.01 s to simulate the initial failure. The vertical deformation reaches the maximum at 2.15 s. Then, the structural vibration gradually weakens and finally stabilizes due to the damping effect. The failure of SC1 will not cause the progressive collapse of the structure.

Analogously, after the removal of SC10, only a small number of grid chords enter into plasticity as shown in Fig. 7, which has little impact on the structure and will not cause progressive collapse. As shown in Fig. 8, the initial failure of SC11 has a greater impact on the structure compared with that of SC10. More grid chords enter into plasticity, and there is also no

The Geometric and Material Parameters of the Preliminarily Selected Columns Supporting the Roof

| Area  | Serial number | Column type                  | Sectional size (mm) | Steel material | Concrete material |
|-------|---------------|------------------------------|---------------------|----------------|-------------------|
| Area I| SC1           | Square steel pipe column     | 1,200 × 800 × 25 × 35 | Q345C          | ———              |
|       | SC2           | Circular CFST column         | Φ1,200 × 20          | Q345B          | C60               |
|       | SC3           | Square steel pipe column     | 1,200 × 800 × 25 × 35 | Q345C          | ———              |
|       | SC4           | Circular CFST column         | Φ1,200 × 20          | Q345B          | C60               |
| Area II| SC5          | Square steel pipe column     | 1,000 × 500 × 30 × 30 | Q345C          | ———              |
|       | SC6           | Circular CFST column         | Φ1,200 × 20          | Q345B          | C60               |
|       | SC7           | Circular CFST column         | Φ1,200 × 20          | Q345B          | C60               |
|       | SC8           | Circular CFST column         | Φ1,200 × 20          | Q345B          | C60               |
|       | SC9           | Circular CFST column         | Φ1,200 × 20          | Q345B          | C60               |
|       | SC10          | Circular CFST column         | Φ1,500 × 20          | Q345B          | C60               |
| Area III| SC11         | Circular CFST column         | Φ1,800 × 35          | Q345C          | C60               |
|       | SC12          | Circular CFST column         | Φ2,000 × 40          | Q345C          | C60               |
|       | SC13          | Circular CFST column         | Φ1,800 × 35          | Q345C          | C60               |
|       | SC14          | Circular CFST column         | Φ2,000 × 40          | Q345C          | C60               |
|       | SC15          | Circular CFST column         | Φ1,800 × 35          | Q345B          | C60               |
|       | SC16          | Circular CFST column         | Φ1,500 × 20          | Q345B          | C60               |

The Geometric and Material Parameters of the Preliminarily Selected Columns Supporting the Roof

| Area  | Column supporting the roof | Serial number | Importance coefficient α' |
|-------|-----------------------------|---------------|---------------------------|
| Area I| SC1                          | 5.332         |
|       | SC2                          | 0.575         |
|       | SC3                          | 0.712         |
|       | SC4                          | 0.636         |
| Area II| SC5                          | 0.466         |
|       | SC6                          | 0.419         |
|       | SC7                          | 0.359         |
|       | SC8                          | 0.596         |
|       | SC9                          | 0.389         |
|       | SC10                         | 1.717         |
| Area III| SC11                         | 1.911         |
|       | SC12                         | 1.000         |
|       | SC13                         | 1.408         |
|       | SC14                         | 3.993         |
|       | SC15                         | 2.121         |
|       | SC16                         | 0.824         |
progressive collapse of the structure. As shown in Fig. 9(a), the maximum vertical displacement of the adjacent cantilever end reaches 10 m after the removal of SC14. In addition, the vertical displacement curve of the node connecting SC14 and the roof is shown in Fig. 9(d), with a maximum vertical displacement of 4 m. Most of the grid members around SC14 enter into plasticity. The lower chords included in the red box shown in Fig. 9(b) quit working due to reaching the ultimate tensile strain. Fig. 9(c)

Fig. 7. Structural Response after Removing the Initial Failure Column SC10: (a) Final Deformation, (b) Distribution of Plastic Members (in blue color), (c) Vertical Displacement Curve of the Node Connecting the Roof and SC10

Fig. 8. Structural Response after Removing the Initial Failure Column SC11: (a) Final Deformation, (b) Distribution of Plastic Members (in blue color), (c) Vertical Displacement Curve of the Node Connecting the Roof and SC11

Fig. 9. Structural Response after Removing the Initial Failure Column SC14: (a) Final Deformation, (b) Distribution of Plastic Members (in blue color, top view), (c) Distribution of Failed Lower Chords (included in the light blue circle, side view), (d) Vertical Displacement Curve of the Node Connecting the Roof and SC14
shows the side view of the failed lower chords. The structural
constraints at SC14 are greatly reduced due to the destruction of
a row of lower chords, and the structure is on the verge of
collapse. As shown in Fig. 10, the vertical deformation of
the structure is very small after the removal of SC15, and only a few
members enter into plasticity.

To sum up, the structural responses after the removal of SC14
are the largest, and the structure is on the verge of collapse.
However, the structural responses are small when other columns
are removed, which reflects that the structure has enough backup
load path and strong ability to resist progressive collapse.

In addition, the typical structural responses after the removals
of SC1 and SC14 are shown in Table 4. As can be seen from
Table 3 above, the importance coefficient $\alpha$ of SC1 is greater
than that of SC14, which are 5.332 and 3.993, respectively.
However, the structural responses after the initial failure of SC14
are greater than those of SC1 according to the results in Table 4.
Namely, the column SC14 is more important to the progressive
collapse resistance of the structure compared with SC1, which
reflects that the actual importance of component in different area
is not positively correlated with the importance coefficient $\alpha$. Therefore, the complex structure should be divided into different
des before conducting sensitivity analysis. Moreover, the proposed
method of zoned concept judgment plus sensitivity analysis in
this study can effectively avoid the omission of key components.

5.2 Column of RC Frame

5.2.1 Initial Failure Components

The columns of the two-story RC frame include RC columns
and CFST columns, which play different roles in the vertical force
transmission path because of their different locations. For the initial
failure conditions of the RC columns, even if the frame beams and
upper floor supported by a failed column are damaged and lose the
abilities to bear vertical loads, the scope of damage is only limited to
the part where the failed RC column is located. There is no
significant impact on the columns directly supporting the roof,
Therefore, the failures of RC columns of the frame was not

| Serial number of initial failure column | Importance coefficient of column | Final vertical displacement of the top node of column (m) | Final number of yield component | Final number of failed component |
|----------------------------------------|----------------------------------|---------------------------------------------------------|---------------------------------|---------------------------------|
| SC1                                    | 5.332                            | 1.01                                                    | 55                              | 0                               |
| SC14                                   | 3.993                            | 3.96                                                    | 590                             | 16                              |

Table 5. Preliminarily Selected CFST Columns of the Frame

| Area     | Serial number | Failure position of the CFST column | Area     | Serial number | Failure location of the CFST column |
|----------|---------------|-------------------------------------|----------|---------------|-----------------------------------|
| Area I   | FC2           | First floor part                    | Area II  | FC8-2         | Second floor part                 |
|          | FC4-1         | First floor part                    |          | FC9           | First and second floor parts      |
|          | FC4-2         | Second floor part                   |          | FC10          | First and second floor parts      |
| Area II  | FC6-1         | First floor part                    | Area III | FC14          | First and second floor parts      |
|          | FC6-2         | Second floor part                   |          | FC15-1        | First floor part                  |
|          | FC7-1         | First floor part                    |          | FC15-2        | Second floor part                 |
|          | FC7-2         | Second floor part                   |          | FC16-1        | First floor part                  |
|          | FC8-1         | First floor part                    |          | FC16-2        | Second floor part                 |

Note: The frame structure at column FC2 only has the first floor; the first and second floors at columns FC9, FC10, and FC14 are interconnected. The CFST columns SC1, SC3, SC5, SC11, SC12, and SC13 in Fig. 5 do not support the frame structure.
Table 6. Important Coefficients of the Preliminarily Selected CFST Columns of the Frame

| Area   | CFST column of the frame | Serial number | Importance coefficient $\alpha_j$ |
|--------|--------------------------|---------------|----------------------------------|
| Area I | FC2                      |               | 0.415                            |
|        | FC4-1                    |               | 0.376                            |
|        | FC4-2                    |               | 0.428                            |
| Area II| FC6-1                    |               | 0.450                            |
|        | FC6-2                    |               | 0.379                            |
|        | FC7-1                    |               | 1.278                            |
|        | FC7-2                    |               | 0.928                            |
|        | FC8-1                    |               | 1.366                            |
|        | FC8-2                    |               | 0.834                            |
|        | FC9                      |               | 0.563                            |
|        | FC10                     |               | 1.684                            |
| Area III| FC14                    |               | 6.313                            |
|        | FC15-1                   |               | 2.194                            |
|        | FC15-2                   |               | 1.244                            |
|        | FC16-1                   |               | 0.619                            |
|        | FC16-2                   |               | 0.387                            |

5.2. Progressive Collapse Analysis Results

After the initial failure of CFST column of the frame, the column above the slab directly supporting the roof will be only supported by the frame beams. The column supporting the roof might lose the ability to bear vertical loads due to the loss of support if the frame beams and slab failed. Moreover, the vertical force transmission path of the grid structure will change correspondingly, which is equivalent to the failure of the column directly supporting the roof. Namely, the influence of CFST column failure is more significant and may lead to the occurrence of progressive collapse, which should be regarded as key component. The preliminarily selected CFST columns of the frame are shown in Table 5, which are characterized by the same locations as the preliminarily selected columns supporting the roof.

The sensitivity analyses were carried out for the preliminarily selected CFST columns of the frame. The calculated results of important coefficients $\alpha_j$ are shown in Table 6. The relatively important columns in each area with the importance coefficients exceeding 1.5 were regarded as key components, including FC10, FC14, and FC15-1.

5.2.2 Progressive Collapse Analysis Results

The key CFST columns FC10, FC14, and FC15-1 were instantaneously removed individually using the AP method. The final structural deformation and distribution of plastic members are obtained as shown in Fig. 11. Where, the columns included in the light blue ellipses in Figs. 11(a), 11(c), and 11(e) represent the initial failure columns. The members in blue color in Figs. 11(b), 11(d), and 11(f) represent the members entering into plasticity.

As shown in Figs. 11(a) − 11(b), the vertical deformation of the structure after the removal of FC10 is very small, only the ends of few beams and columns enter into plasticity, and the roof members do not yield. As shown in Figs. 11(c) − 11(d), the structural vertical deformation is also very small after the removal of FC14. Only a few of beams and columns yield, the other parts do not enter into plasticity, and there is also no collapse. As shown in Figs. 11(e) − 11(f), the vertical displacements of the upper columns supporting the roof are larger compared with the adjacent CFST columns of the frame after removing FC15-1, but the absolute values of displacements are small. The structure does not collapse and no member enters into plasticity.

Fig. 11. Final Deformation and Distribution of Plastic Members after Removing the Initial Failure CFST Column of the Frame: (a,b) FC10, (c,d) FC14, (e,f) FC15-1
5.3 Web Member of Main Truss

5.3.1 Initial Failure Components
The main trusses are located at the junctions of adjacent grid structures with different heights, which are important parts to transfer the vertical loads of the upper grid layer to CFST columns supporting the lower grid layer. Therefore, the web members of the main trusses should be considered as key components. The three main trusses I – III have certain similarity, and the web members can be divided into ordinary web members and the web members directly connected with CFST columns. Therefore, the two representative web members of each truss were selected for progressive collapse analysis as shown in Fig. 12, including the web member in the middle of each truss (Rods I-1, II-1, and III-1) and the web member connected with the middle CFST column (Rods I-2, II-2, and III-2).

5.3.2 Progressive Collapse Analysis Results
The final structural deformation and distribution of plastic members are shown in Fig. 13. Where, the members included in the light blue ellipses represent the initial failure web members. As can be seen from the calculated results of Trusses I and II shown in Figs. 13(a) – 13(h), the removals of the web members directly connected with CFST columns, including Rods I-2 and II-2, have greater impacts on the structure compared with the ordinary web members Rods I-1 and II-1. Only a small number of members enter into plasticity, and the structure does not collapse. As shown in Figs. 13(i) – 13(l), the removals of Rods III-1 and III-2 of Truss III have little effects on the structure, no member enters into plasticity, and the structure does not collapse.

Conclusions can be drawn from the above analysis, the structural responses are the largest when the columns supporting the roof are selected as initial failure components, otherwise the structural responses are small. Namely, the main cause of progressive collapse of the structure is the failure of the column supporting the roof, which will results in significant change in the supporting mode of grid structure, including the decrease of support points and increase of span. The grid members in the damaged area may yield or even fail due to the sharp change of stress, which will lead to the redistribution of internal force and failures of other members, and finally lead to the progressive collapse of the structure.
6. Parametric Analysis of Progressive Collapse Resistance

According to the above calculated results, most chord members of the grid structure close to the initial failure components will yield, while the web members of grid structure rarely yield. In order to study the influences of the resistances of chord and web members on the progressive collapse resistance of the structure, the cross-sectional sizes of the chord and web members of grid structure were selected as independent variables. The amplification factors of member sectional area were 1.05, 1.10, 1.15, 1.20, 1.30, 1.40, and 1.50, respectively. Because the structural responses...
after the removal of CFST column SC14 supporting the roof are
the largest, and the structure is on the verge of collapse, SC14
was selected as the initial failure component of parametric
analysis. The range for adjusting the cross-sectional sizes of
chords and webs is shown in Fig. 14, which exactly includes all
the grid members entered into plasticity or failed when the
initial failure component was SC14. In addition, the vertical
displacements of the four nodes supported by column SC14
were selected as the dependent variables, including the nodes
numbered as 6715, 6644, 1916, and 6652 in the Marc model, as
well as the vertical displacement of node 8297 between SC14
and the roof.

The parametric analysis results are shown in Tables 7 and 8,
including the node displacements and the numbers of yield
and failure members, respectively. The vertical displacements of
the structure after the removal of column SC14 can be effectively
reduced through increasing the cross-sectional sizes of chord
members, as well as the numbers of plastic and failed members.
While increasing the cross-sectional sizes of web members has
little effects on the changes of structural responses. Namely,
increasing the cross-sectional area of chord member is more
effective to improve the progressive collapse resistance of the
structure compared with web member. In addition, it should be
noted that the number of failed members is reduced to 50%
compared with the original structure when the chord cross-sectional

sizes were enlarged to 1.15 times those of original chords. Then,
the number of failed members is 0 when the amplification factor
was 1.2, namely the initial failure of column SC14 will not cause
further damages to other parts of the structure. Therefore, the
conclusions and advices guiding the structural optimization
design can be obtained through the parametric analysis. For the
large-span grid structure does not meet the requirements of
progressive collapse resistance, it is suggested that the cross-
sectional sizes of chord members can be adjusted on the basis of
meeting design requirements and codes. For the terminal building in
this study, the suggested amplification factor of chord cross-
sectional area is 1.2.

Similarly, the parametric analysis for the influence of axial
compression ratios of CFST columns supporting the roof on
the structure was carried out. The cross-sectional sizes of
CFST columns around the initial failure column SC14 were
reduced in equal proportion, and the axial compression ratios
of the columns were enlarged to 1 – 9 times those of original
columns. The structural response after the removal of SC14
shows a slight increasing trend with the increases of axial
compression ratios of adjacent columns. The differences of
vertical displacements between the adjusted and original structures
are less than 10%. Therefore, the change of axial compression
ratio of column has little impact on the progressive collapse
resistance of the structure.

| Amplification factor of sectional area of member | Maximum vertical displacement after adjusting the sectional area of chord member | Maximum vertical displacement after adjusting the sectional area of web member |
|-----------------------------------------------|---------------------------------|---------------------------------|
| Node 8297          | Node 6715          | Node 6644          | Node 1916          | Node 6652          | Node 8297          | Node 6715          | Node 6644          | Node 1916          | Node 6652          |
| 1                 | 3.958             | 2.764             | 2.409             | 1.995             | 6.722             | 3.958             | 2.764             | 2.409             | 1.995             | 6.722             |
| 1.05              | 3.695             | 2.572             | 2.245             | 1.851             | 6.276             | 3.943             | 2.752             | 2.402             | 1.986             | 6.693             |
| 1.1               | 3.324             | 2.288             | 2.013             | 1.675             | 5.643             | 3.924             | 2.735             | 2.388             | 1.973             | 6.662             |
| 1.15              | 2.893             | 1.957             | 1.746             | 1.473             | 4.913             | 3.890             | 2.712             | 2.371             | 1.959             | 6.591             |
| 1.2               | 2.182             | 1.420             | 1.303             | 1.182             | 3.728             | 3.900             | 2.718             | 2.375             | 1.960             | 6.622             |
| 1.3               | 1.820             | 1.181             | 1.085             | 0.989             | 3.113             | 3.884             | 2.705             | 2.364             | 1.949             | 6.594             |
| 1.4               | 1.512             | 0.976             | 0.898             | 0.826             | 2.604             | 3.866             | 2.687             | 2.351             | 1.938             | 6.576             |
| 1.5               | 1.281             | 0.812             | 0.763             | 0.705             | 2.186             | 3.847             | 2.672             | 2.342             | 1.928             | 6.532             |

| Amplification factor of sectional area of member | After adjusting the sectional area of chord member | After adjusting the sectional area of web member |
|-----------------------------------------------|---------------------------------|---------------------------------|
| Number of plastic member | Number of failed member | Number of plastic member | Number of failed member |
| 1                 | 590 | 16 | 590 | 16 |
| 1.05              | 543 | 15 | 589 | 13 |
| 1.1               | 462 | 9 | 604 | 29 |
| 1.15              | 418 | 8 | 581 | 11 |
| 1.2               | 307 | 0 | 594 | 27 |
| 1.3               | 246 | 0 | 597 | 30 |
| 1.4               | 205 | 0 | 577 | 17 |
| 1.5               | 184 | 0 | 569 | 15 |
7. Conclusions

In this study, we focused on the progressive collapse resistance of the terminal building of Lanzhou Zhongchuan Airport in China, which is a large-span curved spatial grid structure with main trusses. The significant contributions of this study are summarized as follows:

1. The proposed improved method of zoned concept judgment plus sensitivity analysis can effectively avoid the omission of key members. The structure has enough backup load transmission path, which can effectively prevent the progressive collapse after initial failure. However, the structural response after the removal of CFST column SC14 supporting the front middle part of the roof is very obvious, with a maximum vertical displacement of 10 m at the cantilever end. Most of the grid chords supported by SC14 yield, and the structure is in the state of near collapse. The reason is that the column spacing at cantilever end is large. Therefore, the columns directly supporting the roof should be increased appropriately for this kind of large-span spatial structure.

2. The research results can provide references for the structural optimization design and safety control of the similar long-span spatial structures. The resistance levels of the upper and lower chords should be increased by 20% for the structure in this study, including the tensile strength and cross-sectional area. However, the changes of axial compression ratios of the columns supporting the roof have little effects on the progressive collapse resistance of the structure.

3. During the subsequent research, the effects of various unexpected loads should be considered, including the progressive damage processes of components during collapse, the falling processes of damaged components and the impacts on structure, which are the lessons learned from this paper.

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