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Dynamic analysis method for the progressive collapse of long-span spatial grid structures

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Abstract. In the past, the progressive collapse resulting from local failures during accidents has caused many tragedies and loss of life. Although long-span spatial grid structures are characterised by a high degree of static indeterminacy, the sudden failure of key members may lead to a catastrophic progressive collapse. For this reason, it is especially necessary to research the progressive collapse resistance capacity of long-span spatial grid structures. This paper presents an evaluation method of important members and a novel dynamic analysis method for simulating the progressive collapse of long-span spatial grid structures. Engineering cases were analysed to validate these proposed method. These proposed methods were eventually implemented in the progressive collapse analysis of the main stadium for the Universiade Sports Center. The roof of the structure was concluded to have good resistance against progressive collapse. The novel methods provide results close to practice and are especially suitable for the progressive collapse analysis of long-span spatial grid structures.

Keywords: long-span spatial grid structures; progressive collapse; multiple responses; analysis method; construction effect

1. Introduction

Progressive collapse is a disastrous phenomenon in which the failure of some key structural members leads to failure of other members; this in turn leads to the partial or even entire collapse of the structure (ASCE 2010, Rezvani *et al*. 2015, and Zhao *et al*. 2017). Progressive collapse first drew the attention of researchers in 1968 when a gas explosion occurred in a panel-type apartment tower at Ronan Point. In 2001, two jetliners crashed into the World Trade Center (WTC) towers at high speed, and the subsequent progressive collapse caused a large number of casualties and huge economic losses (Usmani *et al*. 2003). In studies based on research from 1968, many results have been obtained (Jiang and Chen 2012, Li and Hao 2013, and Malla *et al*. 2011). Applicable structural design codes have specified some essential requirements for structural integrity and collapse resistance (ASCE 2010 and EN 2010). In addition, a few design guidelines for structures to resist progressive collapse have been published, such as the General Service Administration (GSA 2013) and Unified Facility Criteria (UFC 2013).

So far, most research in this field has focused on the frame system. Little attention has been paid to long-span spatial steel structures, which are widely employed in large-space public buildings (Fulop and Ivanyi 2004, Thai and Kim 2011, and Zhang *et al*. 2008). Although long-span spatial grid structures are characterised by a high degree of static indeterminacy, the sudden failure of key members may lead to catastrophic progressive collapse, and catastrophic events involving such structures are common occurrence (Piroglu and Ozakgul 2016, Biegus and Rykaluk 2009, and Kamari *et al*. 2015). For example, the steel truss of Sultan Mizan Zainal Abidin Stadium in Malaysia collapsed in 2009, the long-span steel roof of Twente Stadium in Holland was destroyed in 2011, and the steel roof of Itaquerao Stadium in Brazil collapsed during the construction process in 2014. For this reason, more research than ever should be focused on the progressive collapse of spatial structures. However, the present analysis methods for the progressive collapse of long-span spatial grid structures are the same as those in frame structures. This results in inevitable singular errors because long-span steel structures have some unique structural characteristics, such as long cantilevers and greater influence of the construction effect. Only by using the correct analysis method can the progressive collapse resistance capacity of structures be understood and appropriate protective measures be taken (Cai *et al*. 2012 and Liu 2013).

A review of the literature related to this topic clearly showed that the progressive collapse behaviour of long-span spatial grid structures has not received adequate attention among scholars in the past. Therefore, the aim of this study was to investigate analysis methods for the progressive collapse of long-span spatial grid structures. For this purpose, a direct design procedure known as the *alternate load path method* (Zhang *et al*. 2014, and Gerasimidis and Sideri 2016) was intensively studied, and new practical analysis methods were developed. The proposed methods were validated through engineering case analyses. All these methods were utilised in a computer simulation of the main stadium for the Universiade Sports

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Center and compared with the design results. The proposed analytical methods were found to provide accurate and reliable models for the progressive collapse of long-span spatial grid structures.

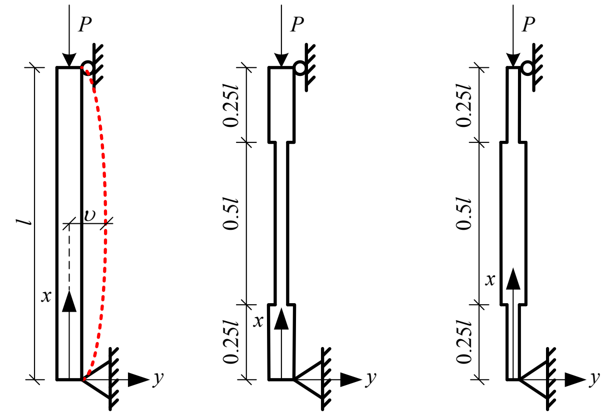
2. Evaluation method of important members

Based on the research conducted during the 1970s, the *alternate load path method* has been recommended as a simplified analysis technique for investigating the potential progressive collapse of a structural design because it focuses on the expansion of local failures instead of the kinds of accidents. Since then, this method has been integrated into the provisions defined by the GSA (2013) and UFC (2013). One of the key tasks of the *alternate load path method* is to determine an important member. Agarwal *et al*. (2001) proposed a method based on stiffness to calculate the importance indices of structural components. Pandey and Barai (1997) presented a method based on sensitivity to measure structural redundancy. Zhang and Liu (2007) constructed a network of energy transfer for structural members and joints based on the energy transfer in frame structures under various loading conditions. Huang *et al*. (2013) provided a simplified evaluation method of structural robustness. Cai *et al*. (2012) discussed the importance of structural components through concept analysis and simplified sensitivity analysis.

As described in the previous section, the above methods for the evaluation of members to be removed are suitable for frame structures. For long-span spatial grid structures, the failure member is often confirmed from the experience of engineers or an analysis of every member. However, even with modern computational power, such computer simulations are still extremely time-consuming and resource-heavy. In addition, there is no uniform standard for identifying important members, which leads to different indicators and evaluation results. In practice, an important member should be consistent for the same analysis object with different methods. Therefore, a simple and convenient numerical approach for evaluating the importance of members needs to be developed.

2.1 Primary scope of the important member

The failure of members in long-span spatial grid structures mainly involves the buckling failure of compression members and strength failure. It is commonly acknowledged that a member with a large stress ratio is of primary importance regarding strength control. For compression members, different initial geometric imperfections are first obtained through eigenvalue buckling analysis. The magnitude of the initial imperfection is $l/300$, where l is the span of the structure (JGJ 7-2010). Because the critical load of the lowest-order imperfection mode is not always the smallest for complex long-span spatial structures and the probability of occurrence decreases with an increasing buckling mode order, the structural nonlinear buckling state can be described by applying the first 10 eigenvalue buckling modes in the model. On the one hand, ordering the eigenvalue buckling modes from low to high reflects the ordering of the stability load capacities for structural members from weak to strong. On the other hand, the structural nonlinear buckling states may not agree with the initial geometric imperfections due to strong structural nonlinearity. Thus, the nonlinear buckling states and first 10 eigenvalue buckling modes are used together to determine the large response area that important members are chosen from. The rationality of this method is illustrated through an axial compression member with hinged ends, as shown in Fig. 1.



(a) Unimpaired (b) Impaired at mid-span (c) Impaired at edges

Fig. 1 Pin-ended column

The elastic buckling load is solved with the Rayleigh–Ritz method (Chen 2013). The deflection of a compression member can be assumed to be

$$y = \upsilon \sin \pi x / l \tag{1}$$

Then, the potential energy of the structure is

$$\Pi = \frac{P^2\upsilon^2}{4EI_1}\left[\left(1+\frac{I_1}{I_2}\right)\frac{l}{2}+\left(\frac{I_1}{I_2}-1\right)\frac{l}{\pi}\right]-\frac{\pi^2 P\upsilon^2}{4l} \tag{2}$$

where

P is the axial load, l is the span of the member and υ is the displacement at mid-span. E is the elastic modulus of the material. I is the moment of inertia for an unimpaired member. Here, I_1 and I_2 are the moments of inertia for the end section and mid-span section, respectively. Based on the principle of stationary potential energy, i.e., $P \neq 0$ and $\upsilon \neq 0$, the buckling load P_{cr} is

$$P_{cr} = \frac{\pi^2 EI_1}{\left[\left(1+\frac{I_1}{I_2}\right)\frac{1}{2}+\left(\frac{I_1}{I_2}-1\right)\frac{1}{\pi}\right]l^2} \tag{3}$$

From Eq. (3), the elastic buckling load for the three kinds of sections can be obtained as

$$\begin{cases} P_{cr,a} = \dfrac{\pi^2 EI}{l^2}, I_1 = I_2 = I \\ P_{cr,b} = 0.550\dfrac{\pi^2 EI}{l^2}, I_1 = 2I_2 = I \\ P_{cr,c} = 0.846\dfrac{\pi^2 EI}{l^2}, 2I_1 = I_2 = I \end{cases} \tag{4}$$

This result shows that the load capacity of a member impaired in the large response area clearly decreases more than that in a small response area, even though the impaired ranges are identical. The mid-span area of the member is more important than the end area of the member. Therefore, it is feasible to determine the members of primary importance by buckling analysis.

2.2 Multiple-response evaluation

Compared with wind loads and seismic action, the vertical load always plays an important role in the design load for long-span spatial grid structures. In addition, fire and collision are infrequent. Therefore, a feasible approach is to analyse the response of the remaining structure under a vertical load and use this to describe the importance of members.

The response of a structure includes the response of members (e.g., stress ratio of the member and node displacement) and the total response (e.g., load capacity, strain energy, and natural frequency). For the former, the remaining members with a large response can reflect the importance of the removed member. It is assumed that the response of members is a random variable obeying an approximately normal distribution because long-span spatial grid structures have many members. Then, the overall mean value μ and standard difference σ are estimated based on the response samples of members under normal service conditions and when a member is removed, respectively. Thus, calculating the 0.05-upper quantile η is easy under both conditions

$$\mu = \frac{1}{n}\sum_{i=1}^{n}\eta_i \tag{5}$$

$$\sigma = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(\eta_i - \mu)^2} \tag{6}$$

$$\eta = \mu + 1.645\sigma \tag{7}$$

$$\alpha_k^r = \pm(\eta_o - \eta_k)/\eta_o \tag{8}$$

where

η_i is the response of member i and n is the number of members under normal service conditions or when member k has been removed. η_o and η_k are the 0.05-upper quantiles under the normal service condition and when member k has been removed. α_k^r is the importance coefficient of member k based on the structural response r. If the increased response is detrimental to the structure, Eq. (8) returns a negative value. Conversely, Eq. (8) returns a positive value for the opposite case.

For the overall response, the importance coefficient of member k is equal to the sensitivity index of removing member k

$$\alpha_k^r = \pm(\gamma_o - \gamma_k)/\gamma_o \tag{9}$$

where

γ_o and γ_k are the overall responses under the normal service condition and when member k is removed, respectively.

At present, several evaluation methods for important elements have been implemented, such as those based on the stiffness, strength, energy, sensitivity, and concept.

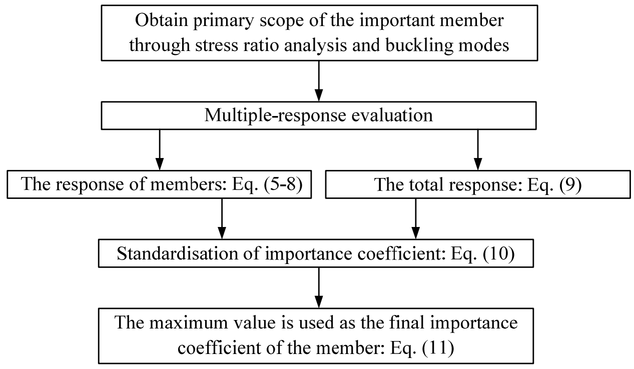


Fig. 2 Basic analysis procedure of the evaluation method of important members

However, these conventional methods only consider a single response of structures in progressive collapse analysis, and this may ignore some important elements because each response can only reflect some structural characteristics. For this reason, it is necessary to evaluate the importance of members according to multiple structural responses.

Different structural responses will clearly lead to different importance coefficients. In order to consider various structural responses, the maximum value is used as the final importance coefficient of the member. Because the order of magnitude for each importance coefficient may be different, standardisation is needed.

$$\alpha_{k,s}^r = \frac{\alpha_k^r}{\left(\alpha_1^r, \alpha_2^r, \cdots, \alpha_t^r\right)_{\max}} \tag{10}$$

$$\alpha_k^F = \max \alpha_{k,s}^r \Big|_{r=1,2,\cdots,m} \tag{11}$$

Here,

$\alpha_{k,s}^r$ is the standardised importance coefficient, t is the number of primary important members, α_k^F is the final importance coefficient, and m is the number of structural responses.

In this paper, the above method is called the *multiple-response evaluation method based on the primary scope* and the basic analysis procedure is proposed in Fig. 2. The results of this method can include all adverse conditions and correctly reflect the true important member.

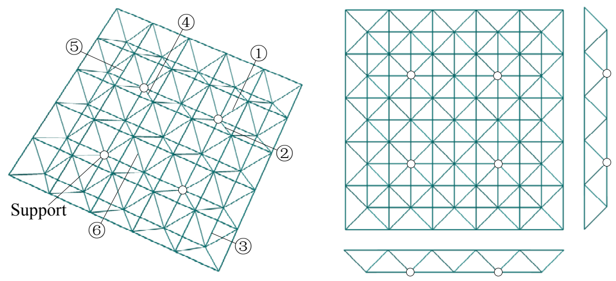
2.3 Verification

In order to validate the proposed method, two simple examples are illustrated here.

2.3.1 First case: Square pyramid grid structure

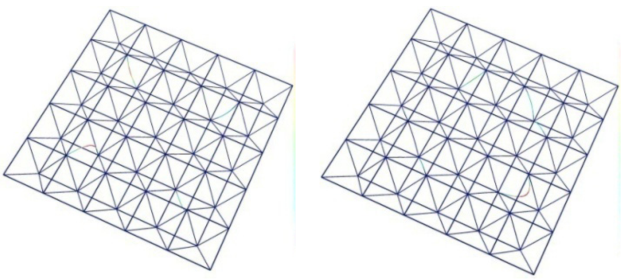
A square pyramid grid structure with four hinge supports constituted of units of quadrangular pyramids was considered. The dimensions of the architectural plane were 15 m × 15 m with a height of 1.5 m, as shown in Fig. 3. The sections of the members were circular. The upper chords, lower chords, and web members had dimensions of Φ68 mm × 6 mm, Φ63.5 mm × 4.5 mm, and Φ50 mm × 2.5 mm, respectively.

First, eigenvalue buckling analysis was performed by applying unit vertical concentrated loads on the nodes of the



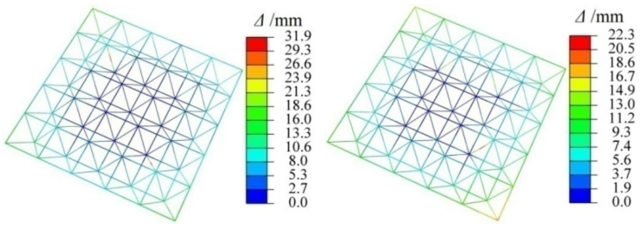
(a) Axonometric drawing (b) Three-view drawing

Fig. 3 Square pyramid grid structure



(a) First buckling mode (b) Ninth buckling mode

Fig. 4 Typical buckling modes of the square pyramid grid structure



(a) First buckling mode is applied (b) Ninth buckling mode is applied

Fig. 5 Displacement contour of the limit state for the square pyramid grid structure

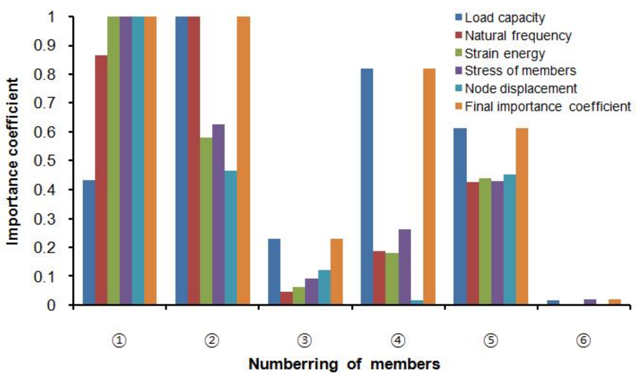


Fig. 6 Results of importance coefficients

upper chords. Because of the symmetry of the structure, some buckling modes were repeated (such as the first 4 buckling modes). In addition, the first and ninth buckling modes (Fig. 4) could represent the rest of the first 10 eigenvalue buckling modes by the distributions of the buckled members. The structural nonlinear buckling states were obtained by applying these two imperfections in the model individually, as shown in Fig. 5.

This analysis showed that the web members connected to the hinge supports had a larger response. These members were selected as the preliminary important members for the multiple response analysis. Therefore, members 1 and 2 were selected as preliminary important members considering the symmetry of the structure, and member 3 was used to verify the correctness of the proposed method. Because the structural nonlinear buckling states in Fig. 5 are in accordance with the initial geometric imperfections, there was no need to add an important member. Moreover, members 4 and 5, which had large stress ratios, were selected as preliminary important members, and member 6, which had a small stress ratio, is selected for comparison. Fig. 3 shows how the members were numbered.

Then, the multiple-response analysis was performed by using the above preliminary important members. The results were standardised and are as shown in Fig. 6.

Based on the obtained results, the importance coefficients of members 1, 2, 4, and 5 were greater than those of the comparison members 3 and 6. Moreover, there was a great difference between the results of the strain energy and load capacity. Therefore, it is possible to ignore some important elements by using single-response analysis. Members 1, 2, and 4 were chosen as the final important members of the square pyramid grid structure. Fig. 7 presents the comparisons of importance coefficients between this paper and Cai *et al.* (2012). Based on the results for two different methods, the comparable calculated values were nearly identical and member 1 was chosen as the important member because of the stress of members.

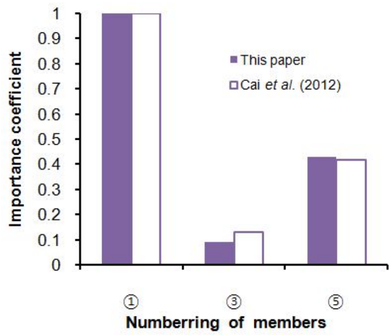


Fig. 7 Comparison of importance coefficients

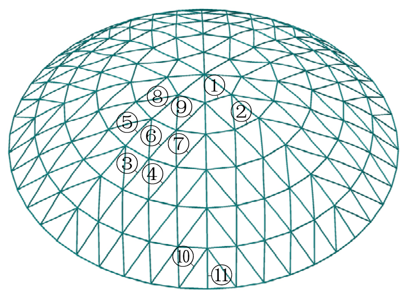


Fig. 8 Kiewitt8 single-layer reticulated dome

Therefore, the computational results in this paper matched well with the *sensitivity analysis method based on concept* (Cai *et al*. 2012). However, note that the upper chords near the support are not the important members with the proposed method, and Cai *et al*. (2012) did not compare the importance between members. Based on the above studies, the *multiple-response evaluation method based on the primary scope* is a feasible approach to evaluating the importance of members because of its accuracy and speed.

2.3.2 Second case: Single-layer reticulated dome

Fig. 8 shows the Kiewitt8 single-layer reticulated dome used for analysis. The span of the structure was 40 m, and the ratio of the rise to span was 1/5. The main members and members in the circumferential direction had dimensions of Φ121 mm × 3.5 mm, and the section of the diagonal members had dimensions of Φ114 mm × 3 mm. All of the connection nodes were rigid, and equivalent vertical loads were applied to each connection node.

The important members were chosen from one in sixteen structures because of symmetry. First, the initial geometric imperfection was determined by eigenvalue buckling analysis. Because the distributions of the buckled members in the overall buckling mode could include that in the local buckling modes, the first buckling mode (Fig. 9) could represent the rest of the first 10 eigenvalue buckling modes. The structural nonlinear buckling states were obtained by applying this imperfection in the model, as shown in Fig. 10.

From this analysis, members 3–6 were selected as preliminary important members based on the most representative first buckling mode. Members 7–9 also buckled because of nonlinearity (Fig. 10). Members 1 and 2, which had a large stress ratio, were also selected as preliminary important members, and members 10 and 11 were selected as comparison. Fig. 8 shows how the members were numbered.

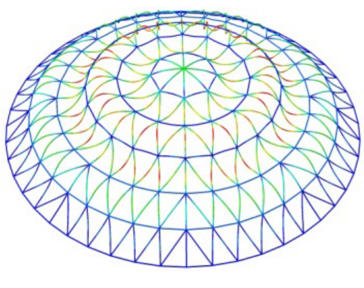


Fig. 9 First characteristic buckling mode

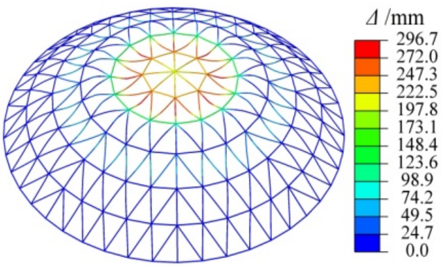


Fig. 10 Displacement contour of the limit state for the structure

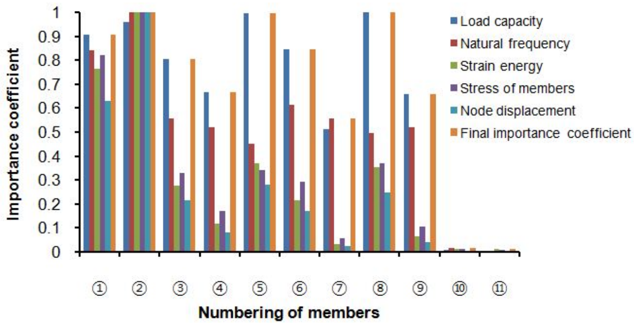


Fig. 11 Results of importance coefficients

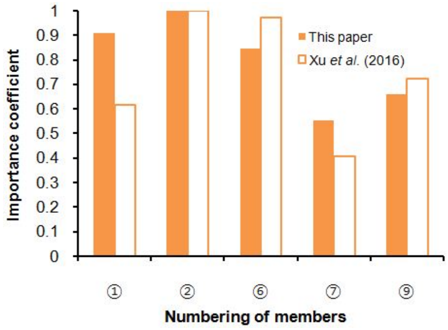


Fig. 12 Comparison of importance coefficients

Then, the multiple-response analysis was performed based on the above preliminary important members. The results were standardised and are as shown in Fig. 11.

These results show that the importance coefficients of members 1–9 were greater than those of the comparison members. Moreover, members 5 and 8 can be ignored simply by using single-response analysis. Therefore, the *multiple-response evaluation method based on the primary scope* can be used to evaluate the final importance of members because of its accuracy and speed. Members 1–3, 5, 6, and 8 were chosen as the final important members of the Kiewitt8 single-layer reticulated dome. Fig. 12 presents the comparisons of importance coefficients between this paper and Xu *et al*. (2016). Based on the results for two different methods, the comparable calculated values were nearly identical and members 2 and 6 were chosen as the important members. Therefore, the computational results in this paper matched well with the reference Xu *et al*. (2016). However, Members 3, 5 and 8 were ignored because of neglecting the response of load capacity in the above-mentioned reference. But as similar with this paper, Zhao *et al*. (2017) selected this type of main member as the important member. In addition, note that member 1 was the important member with the proposed method, and the method of Xu *et al*. (2016) was not comprehensive. Therefore, the *multiple-response evaluation method based on the primary scope* can provide an efficient and accurate way for evaluating the importance of members.

3. Analysis method of progressive collapse

Before the important member selected by the *multiple-*

response evaluation method based on the primary scope is removed, the structure maintains the state of initial equilibrium. In order to correctly evaluate the progressive collapse behaviour of the structure, the state of initial equilibrium should be considered. At present, theoretical analysis, finite element analysis, and experimental research on the progressive collapse of long-span spatial grid structures are based on the design conditions and frame structure. However, it is broadly accepted that different construction procedures lead to different internal force and deformation distributions, and there is a great difference in mechanical behaviour between a one-time load on the design model and the step-by-step construction process for long-span spatial grid structures. Therefore, the progressive collapse behaviour of long-span spatial grid structures needs to be analysed by considering the construction effect.

3.1 Equivalent load unloading method considering the construction effect

The *element birth and death of node rectification* algorithm developed by Tian and Hao (2015) is suitable for the construction simulation of long-span spatial grid structures. Based on this method, the *equivalent load unloading method considering the construction effect* is proposed here. The basic principles of this proposed method are as follows:

(1) The static internal force P_i of important member i for every construction stage and the final internal force P_0 after construction are given by using the method called *element birth and death of node rectification*.

(2) The important member is removed, while is P_i applied on the remaining structure in reverse. That means that the failure of the important member is equivalent to the unloading process over time of the final internal force P_0.

Fig. 13 illustrates the time–history curve of the equivalent load. The common *equivalent load unloading method* is shown by the dashed line, which indicates that the final internal force of the important member is P_D under a one-time loading. The polygonal line 0–3 represents the change in internal force during the construction process, and the horizontal line 3–4 is the time of stability before the important member is removed. According to this curve, the dynamic response of the structure is divided into three stages. First, forced vibration of the structure is produced because of the time-varying construction load and equivalent load ($0 \le t < t_0$). Then, the important member is removed ($t_0 \le t < t_0 + t_p$). After that, the remaining structure stays in the stage of free vibration ($t \ge t_0 + t_p$).

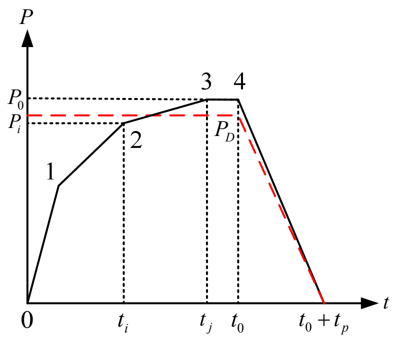


Fig. 13 Time–history curve of the equivalent load

In order to validate the nonlinear dynamic analysis method, a single degree of freedom example is illustrated in this paper, as shown in Fig. 14. The process of free vibration was simulated by the support was removed immediately. In order to simplify the calculation, the damping ratio and the plastic property of the material were not considered. The basic equation of the particle was then obtained as follow

$$\Delta(t) = F / K(\cos\omega t - 1) \tag{12}$$

where

F is the vertical load of the particle. K and ω are the stiffness and circular frequency, respectively.

The computational results by the *equivalent load unloading method* were compared with the theoretical data, as shown in Fig. 15. From this analysis, it is observed that the computational results matched well with the theoretical data. Therefore, the numerical analysis in this paper can obtain reliable results.

3.2 Verification

In order to validate the proposed method, two simple examples are presented: a hexagonal star-type grid structure (span: 12.0 m, rise: 1.0 m) and extended hexagonal star-type grid structure (span: 17.5 m, rise: 2.2 m), as shown in Fig. 16. All members had a section of Φ102 mm × 3.5 mm, and a vertical concentrated load of 9 kN was applied to each node. The stress and deformation of the two structures when members 1 and 4 were each removed were analysed by using the conventional analysis algorithm (i.e., *equivalent load unloading method*) and proposed algorithm (*equivalent*

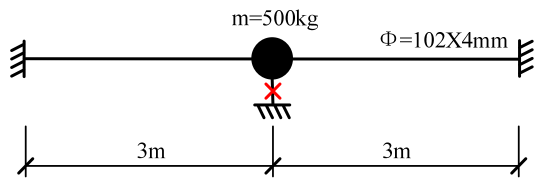


Fig. 14 Single degree of freedom model

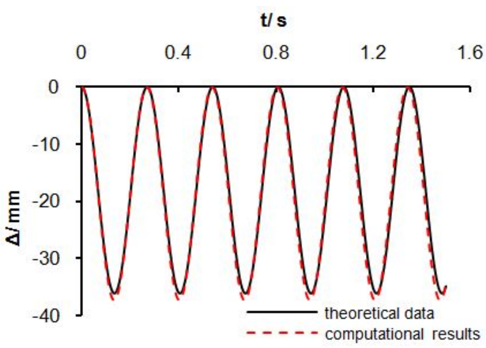
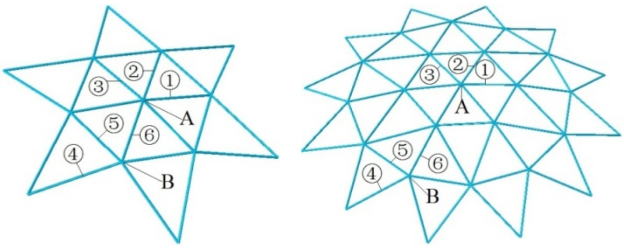


Fig. 15 Dynamic analysis results

Table 1 Analysis results

Structural form		Member 1 is removed				Member 4 is removed			
		f /Hz	Δ_A /mm	$\sigma_{②}$ /(N/mm^2)	$\sigma_{③}$ /(N/mm^2)	f /Hz	Δ_B /mm	$\sigma_{⑤}$ /(N/mm^2)	$\sigma_{⑥}$ /(N/mm^2)
Hexagonal star-type rid structure	Y	15.99	13.19	185.26	85.02	18.76	14.78	140.63	138.74
	N	16.11	15.17	90.36	60.48	18.93	14.49	90.43	32.98
Extended hexagonal star-type grid structure	Y	13.41	17.37	134.91	123.08	20.28	5.46	49.90	84.29
	N	13.19	13.10	93.59	53.38	20.46	6.90	56.80	33.75

* Note: Y represents the *equivalent load unloading method considering the construction effect*;
N represents the *equivalent load unloading method*



(a) Hexagonal star-type grid structure (b) Extended hexagonal star-type grid structure

Fig. 16 Member numbers

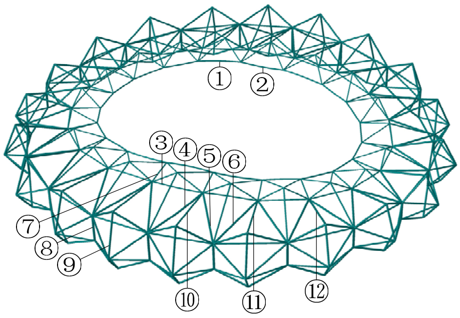
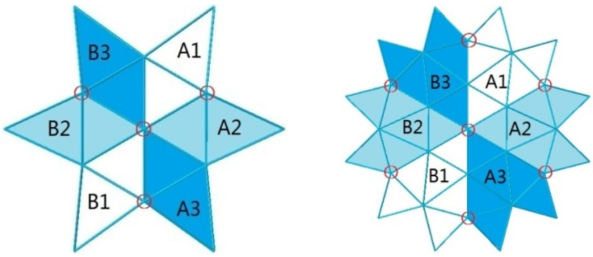


Fig. 18 Structural system



(a) Hexagonal star-type grid structure (b) Extended hexagonal star-type grid structure

Fig. 17 Partition of structural units

load unloading method considering the construction effect). To obtain the correct results, the parameter t_p was set to 0.01 s, which is feasible for considering geometric nonlinearity in progressive collapse analysis. Table 1 compares the results for the above two algorithms. The members are numbered as shown in Fig. 16. To consider the construction effect, the temporary supports were set up at the outer ring nodes and top of the two structures, as shown in Fig. 17. Construction areas A and B were installed in the clockwise direction at the same time. Then, the temporary supports were dismantled after installation was completed.

Compared with the conventional method, the maximum stress of the structure obtained with the *equivalent load unloading method considering the construction effect* increased by approximately 50% and even quadrupled. Consequently, the improved method was verified to be more reliable because the actual initial state and construction effect must be considered.

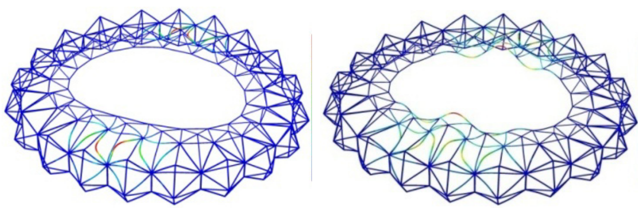
4. Engineering application

4.1 Project profile

The roof of the main stadium for the Universiade Sports Center has a single-layer folded-plane latticed shell structure system. The structural system comprises twenty units with similar shapes, and the dimensions of the architectural plane are 274 m × 289 m, as shown in Fig. 18. The sections of the main members are circular, and the diameters are from 700 to 1400 mm. The materials of the main members are Q390 and Q420.

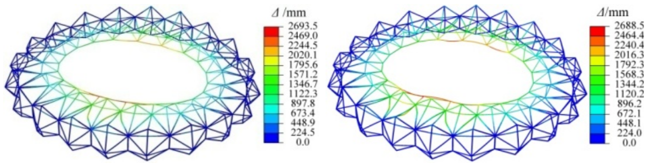
4.2 Evaluation of important members

The *multiple-response evaluation method based on the primary scope* was applied to the structure to determine the distribution of important members. These important members were chosen from a quarter of the structure because of symmetry. Because the first and third buckling modes (Fig. 19) could represent the rest of the first 10 eigenvalue buckling modes by the distributions of the



(a) First buckling mode (b) Third buckling mode

Fig. 19 Typical buckling modes of the structure



(a) First buckling mode is applied (b) Third buckling mode is applied

Fig. 20 Displacement contour of the limit state for the structure

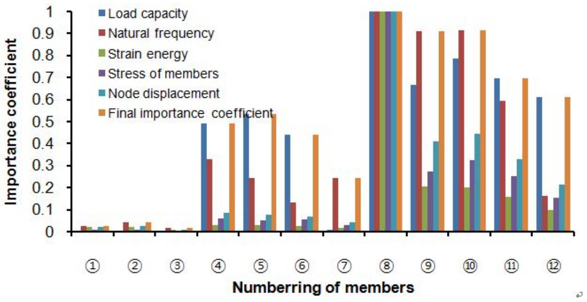


Fig. 21 Results of importance coefficients

buckled members, the structural nonlinear buckling states were obtained by applying these two imperfections to the model individually, as shown in Fig. 20. Members 1–6 were selected as the preliminary important members based on Fig. 19. No important members needed to be added because the nonlinear buckling states were in accordance with the initial imperfections. Note that, although the cantilever members of the inner ring were liable to buckle, the deformation and balance mode clearly would not change because the above members would deform by bending. Hence, these members would not buckling. Members 7–12, which had a larger stress ratio, were also chosen as preliminary important members. Fig. 18 shows how the members were numbered.

Then the multiple-response analysis was performed based on the above preliminary important members. The results were standardised and are as shown in Fig. 21.

Based on this analysis, members 8–11 were chosen as the final important members for a quarter of the structure. Fig. 22 presents the comparisons of importance coefficients between this paper and Zhang *et al*. (2011). The results indicated that the important members obtained with this proposed method were nearly identical to that in the design of the practical project (Zhang *et al*. 2011) when only considering buckling analysis (the importance coefficients of members 4-6 were greater than those of the members 1-3). Moreover, the important members obtained with this proposed method are more comprehensive and accurate. Because all of unfavourable responses are included, the *multiple-response evaluation method based on the primary scope* is a feasible approach to evaluating the importance of members of the main stadium for the Universiade Sports Center.

4.3 Progressive collapse analysis

As an application of the *equivalent load unloading method considering the construction effect*, the progressive collapse behaviour of the main stadium was estimated based on the important members obtained from the *multiple-response evaluation method based on the primary scope*. The mechanical behaviour of the structure was analysed when members 8, 9, 10, and 11 were individually removed. The analysis with the proposed method considered the dead loads as well as live loads (DL+0.25LL). To obtain the correct results, the parameter t_p was set to 0.01s, which is less than 1/10 of the inherent period of remaining structure and is feasible for considering geometric nonlinearity in progressive collapse analysis. Fig. 23 plots the stress distributions of the structure when the maximum stress reached its peak value.

The results of the proposed method showed that the structure stayed in the elastic condition when members 9–11 were individually removed. When member 8 was removed, the edges of individual members entered the plastic state, but few exceeded the yield stress. Three ring beams (i.e., bottom of the shoulder, bottom of the crown, and inner ring (Tian and Hao 2015)) connect the whole structure and clearly prevent collapse. In addition, vibration was induced in part of the structure. The maximum vertical displacements when members 8, 9, 10, and 11were each removed were 1667, 1036, 1122, and 924 mm, respectively. The displacement–span ratios were 1/173, 1/279, 1/258, and 1/313 respectively. Thus, the roof of the main stadium for

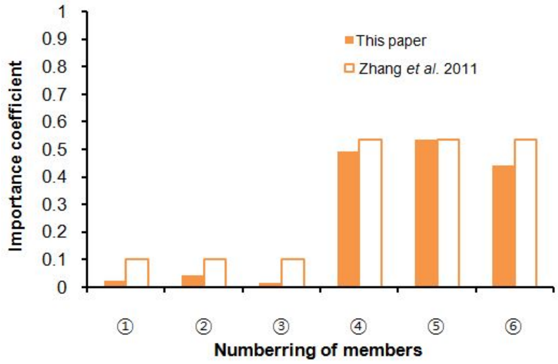
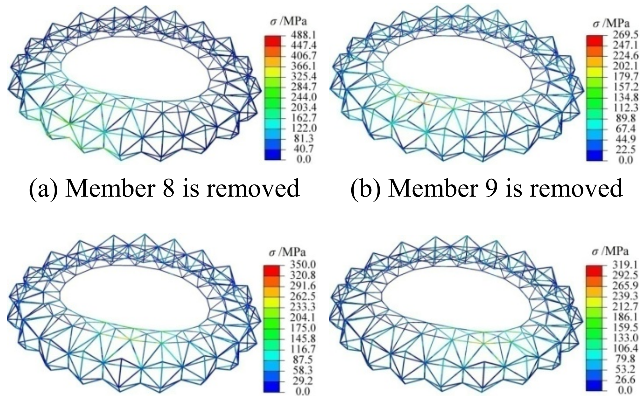


Fig. 22 Comparison of importance coefficients



(a) Member 8 is removed (b) Member 9 is removed

(c) Member 10 is removed (d) The member 11 is removed

Fig. 23 Stress distribution of the structure the Universiade Sports Center was demonstrated to have good resistance against progressive collapse. This conclusion was also proposed in the reference Zhang *et al*. (2011).

Based on these results, the proposed method can assess the progressive collapse resistance capacity of long-span spatial grid structures reliably with high computational efficiency.

5. Conclusions

A direct design procedure known as the *alternate load path method* was intensively studied, and new practical analysis methods were developed. These proposed methods were validated through engineering case analyses, and all of these methods were utilised in the computer simulation of the main stadium for the Universiade Sports Center. The following conclusions were drawn:

(1) The *multiple-response evaluation method based on the primary scope* can include all adverse conditions and correctly reflect the true important members. It is suitable for evaluating the important members of long-span spatial grid structures.
(2) The *equivalent load unloading method considering the construction effect* can assess the progressive collapse resistance capacity of long-span spatial grid structures reliably and rationally because it considers the actual initial state.
(3) Based on the results for the proposed methods, the roof of the main stadium for the Universiade Sports Center has good resistance against progressive collapse.

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