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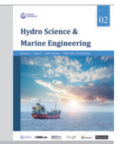

ARTICLE

Progressive Collapse Resistance Analysis of Secondary Shielding Installation Platform for A-type Tank

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ABSTRACT

As a key supporting equipment for the construction of LNG carriers, the installation platform undertakes the support and guarantee of LNG carrier tank internal construction. This paper takes the secondary shielding installation platform of A-type tank as the object of study, the study firstly considers the semi-rigidity of the nodes and the material nonlinearity based on finite element software, and then the residual structure is calculated using static nonlinear method after single truss, two trusses and three trusses are invalid simultaneously. The research results show that the truss with higher components importance coefficient has greater impact on the residual structure when the truss is invalid; After the 2 trusses of installation platform become invalid completely, the further progressive collapse will not occur; When A1-HJ, A2-HJ and A2-HJ are dismantled at the same time, it will lead to the local progressive damage, which can cause the collapse of large-scale structures. The research findings can support the design and use of the installation platform.

1. Introduction

Progressive collapse refers to the local failure of structures under unconventional loads, including explosive explosion, gas explosion, vehicle impact and heavy impact. This damage expands and eventually leads to the collapse of the whole building or causes collapse out of proportion to the initial damage. Since the collapse of Ronan Point in 1968, progressive collapse has attracted the attention of designers at home and broad firstly. Taewam Kim and Jinkoo Kim [1] had analyzed the influencing factors of continuous collapse resistance of steel frame structures, the results show that: The risk of progressive collapse of steel frame structure increases with the decrease of floor number and the increase of beam span; Increasing the span number can significantly improve the anti-progressive collapse performance of the structure; With the increase of design seismic force, the anti-progressive collapse ability of the structure also increases. Hu Xiaobin [2] adopted finite element program LS-DYNA to complete progressive collapse simulation analysis of Multilayer Planar Steel Frame, the results show that: For steel structures, increasing failure strain of materials can improve their ability to resist progressive collapse. Xie Buying described the whole process of frame failure until it completely collapses on the ground by using the concept of Inertia, the relationship between force and acceleration and the principle of collision dy-

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namically. In addition, the general commercial software is used to carry out numerical simulation analysis of the structure in order to give some damage mechanisms or anti-collapse measures.

Secondary shielding installation platform for A-type tank is the new and lightweight scaffold which is used to support the installation engineering of the secondary shield. It is composed of several steel frames connected by connecting beams, and each steel frame is composed of trusses and suspension structures fixed by connecting with hull. At present, hanging structures are mostly used in high-rise buildings. It is it is seldom that large scaffolding construction uses hanging structures. It is difficult to judge whether the remaining structures will collapse continuously once the truss is damaged or invalidated in the installation platform structure. For the installation platform, the progressive collapse of the structure will cause extensive damage to the secondary shield and serious casualties, resulting in incalculable losses. Therefore, the analysis of its anti-progressive collapse has important engineering significance.

2. Analysis Model

2.1 Model Unit Selection

At present, the finite element method is usually used to simulate the progressive collapse process of structures, the accuracy of the analysis model has a great influence on the evaluation results of progressive collapse. The secondary barrier installation platform of A-type tank is mainly composed of frame, connecting beam and expansion beam [3]. The structures are connected with each other by linking pins for EC fasteners. From the point of view of connection mode, the installation platform is a hybrid connection mode. The components of the installation platform, such as the expansion beam structure and the connection beam, are connected by EC fasteners. The connection mode between the components is welding [4]. The internal nodes of components can be simulated by common nodes in finite element calculation, while the EC fastener connections between components can be considered as semi-rigid connections [5]. In the process of modeling using ANSYS Workbench, this paper simulates the connection of nodes between components by spring element, and simulates semi-rigidity of nodes by inserting three-direction non-linear rotating spring [6]. According to *Technical Specification for Safety of Portal Steel Tube Scaffold in Construction* (JGJ128-2000161), the tightening torque of fastener bolts should be 50-60 N.m, and not less than 40 N.m. Therefore, the initial stiffness of fastener connection is taken in the analysis of this chapter when the tightening moment is 60 N. M. The initial stiffness of the joint is 71.27 kN·M/rad at the connection of EC fasteners of the main components based on the research of Luzheng of Zhejiang University.

2.2 Components of Installation Platform

The trapezoidal truss of the installation platform is made of H-section steel with 23 longitudinal sections and 8 transverse sections. Hollow square tubes are used for hanging frames and connecting components. The type of steel used in the structure is Q235B, and the related parameters of the installation platform components are shown in Table 1.

Table 1. Section specification of installation platform

number	Sectional specification	Cross section area/mm^2	Plastic modulus of section/mm^3	Plastic flexural capacity/kN·m	Shear ultimate bearing capacity/kN
1	139×139×10×7	3613	98057.7	23	187.4
2	67×67×3	768	18445.5	4.34	172.8
3	52×52×2	400	7504	1.76	90.0
4	58×58×3	660	13626	3.20	148.5
5	37×37×2	280	3679	0.86	63.0
6	46×46×3	516	8127	1.91	116.1
7	90×48×4	1952	19040	4.47	439.2
8	41×41×2	312	4567	1.07	70.2
9	57×57×3	648	13135.5	3.09	145.8

2.3 Component Importance Evaluation

The installation platform belongs to large steel structure. If the whole structure is dismantled one by one to calculate the importance coefficient of each truss, the workload will be enormous [7]. Platform structure is symmetrical in the direction of captain, so only one half of the model of installation platform is used to evaluate the importance of truss. The steel frames of each truss are divided into three areas: A, B and C. The number of each steel frame is shown in Figure 1.

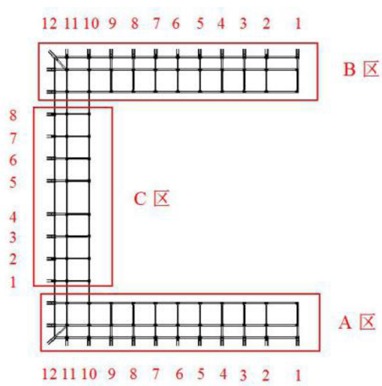

Figure 1. Frame number of Installation platform

For non-frame structures, it is necessary for designers

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to evaluate the importance of components [8]. According to the relevant research results, the main methods of evaluating the importance of components are based stiffness-based, energy-based and strength-based [9]. In this paper, energy-based judgment method is used to evaluate the importance of components, calculate the importance coefficients of each truss, and ultimately determine the key truss. The importance coefficients of each truss are shown in Table 2.

2.4 The Number of Structure

In order to facilitate the follow-up analysis, the members of each steel frame are numbered in the form of steel frame number-member+layer number. For example, the fifth suspender of A1 frame is expressed as A1-DG5, and the fifth walkway beam of A1 frame is expressed as A1-ZD5.The connecting beams between the two frames are numbered in the form of steel frame number-steel frame number-connecting beams+layers. For example, the corridor connecting beams between A1 steel frame and A2 steel frame are named A1-A2-ZDLJ1, A1-A2-ZDLJ2, A1-A2-ZDLJ3, A1-A2-ZDLJ4, A1-A2-ZDLJ5 according to the number of layers. The fifth floor frame connecting beam between A1 steel frame and A2 steel frame near the aisle is named A1-A2-KJLJ5a, and the other side is named A1-A2-KJLJ5b. Since the installation platform is a plane symmetrical structure about A1 steel frame, the A2 steel frame about A1 steel frame is named A-2', and the other components are the same.

3. Analysis of Calculation Results

3.1 Structural Response to Failure of a Truss

3.1.1 Demolition of Truss A1-HJ

After removing the truss A1-HJ, the response of the structure is shown in Figure 2. It can be seen from the Figure that the maximum displacement of the surrounding structure after removing the truss is 29.9 mm (Figure (a)).The axial force of the suspender is greater than other tension members in the same steel frame. Therefore, the suspender is selected to judge the failure of the tension members. The maximum axial force of the adjacent suspender is A2'-DG5, and the axial stress is 122 MPa (Figure (b)).Through the analysis of the axial force of the members connected with the truss, it is found that the suspender in the expansion beam area has relatively large axial force among the steel frames. The failure judgment of A2 steel frame and A2 steel frame suspender in adjacent area of failure truss is made. As shown in Table 3, it shows that the progressive collapse of residual structure will not occur after A1-HJ is completely removed. The maximum shear force of beam section is 60.1kN at the connection of A2 truss and suspender (Figure (c)), which does not reach the failure limit of section. The results show that the shear force of the corridor connection beam is larger than that of the frame connection beam, and the shear ultimate bearing capacity of the section of the corridor connection beam is smaller than that of the frame connection beam. Therefore, the shear failure of the corridor connection beam is judged, as shown in Table 4.The plastic bending capacity of the frame linking beams connected with the dismantled truss is 1.91 kN m, and that of the walkway connection beams is 1.07 kN M. As shown in the bending moment diagram of Figure (d), the maximum bending moment of each connection beam is 1.02 kN m, and the section does not enter the plasticity, so there will be no plastic hinges. The maximum stress of the structure appears at the junction of A2_-HJ and A2_-DG5 (Figure (e)), and the maximum stress is 235.4 MPa.

Table 2. Important coefficient of trusses

Number of removed component	**A1-HJ**	**A2-HJ**	**A3-HJ**	**A4-HJ**	**A5-HJ**	**A6-HJ**	**A7-HJ**	**A8-HJ**
Coefficient of importance	0.1846	0.1057	0.0849	0.0839	0.0839	0.0839	0.0839	0.0832
Number of removed component	A9-HJ	A10-HJ	A11-HJ	A12-HJ	B1-HJ	B2-HJ	B3-HJ	B4-HJ
Coefficient of importance	0.0804	0.0784	0.1167	0.1174	0.1846	0.1057	0.0849	0.0839
Number of removed component	B5-HJ	B6-HJ	B7-HJ	B8-HJ	B9-HJ	B10-HJ	B11-HJ	B12-HJ
Coefficient of importance	0.0839	0.0839	0.0839	0.0832	0.0804	0.0784	0.1167	0.1174
Number of removed component	C1-HJ	C2-HJ	C3-HJ	C4-HJ	C5-HJ	C6-HJ	C7-HJ	C8-HJ
Coefficient of importance	0.0977	0.0813	0.0832	0.1267	0.1267	0.0826	0.0813	0.0977

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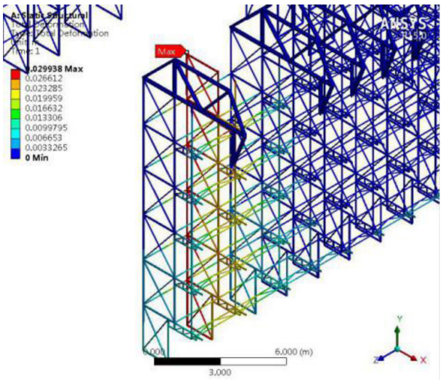
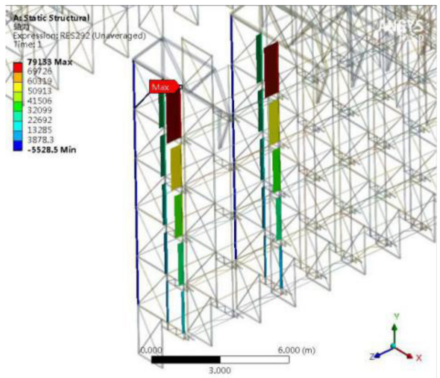
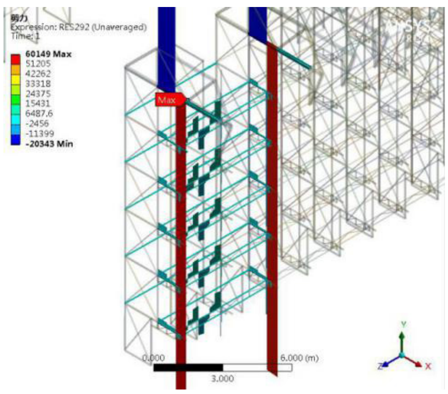


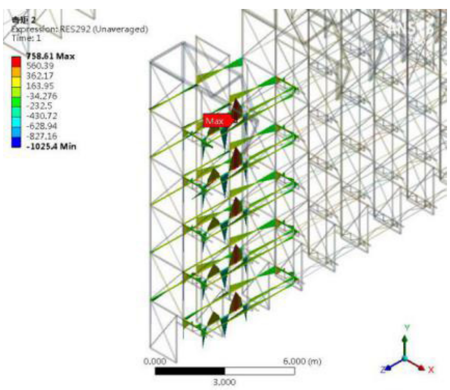
(a) Local structural deformation after removal of truss



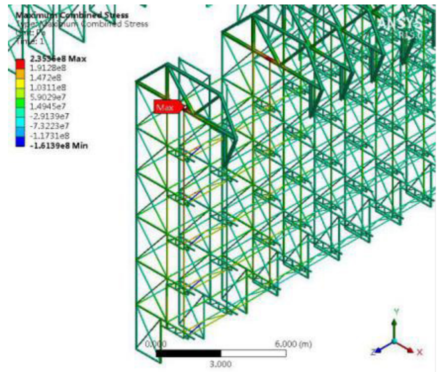
(b) Axis Force of Suspender after Removal of Truss



(c) Shear force of surrounding members after removal of truss



(d) Bending moment of connecting beam after removal of truss



(e) Local structural stress after removal of truss

Figure 2. Local structure response after removing A1-HJ

Table 3. Boom damage judgment after removal of A1-HJ (Based on boom axial deformation)

Number of suspender	Displacement of upper end node of suspender/mm	Displacement of lower end node of suspender/mm	Axial Deformation of Suspender/mm	Deformation limit /mm	Component failure judgment
A2-DG1	-6.62	-6.83	0.21	215	unspoiled
A2-DG2	-6.15	-6.58	0.43	215	unspoiled
A2-DG3	-5.42	-6.07	0.65	215	unspoiled
A2-DG4	-4.37	-5.30	0.93	215	unspoiled
A2-DG5	-2.67	-4.19	1.52	250	unspoiled
A2´-DG1	-6.93	-7.15	0.22	215	unspoiled
A2´-DG2	-6.42	-6.87	0.45	215	unspoiled
A2´-DG3	-5.63	-6.33	0.70	215	unspoiled
A2´-DG4	-4.51	-5.50	0.99	215	unspoiled
A2´-DG5	-2.76	-4.34	1.58	250	unspoiled

Table 4. Beam failure analysis after removal of A1-HJ (Based on beam end shear)

number	Maximum shear force after removal of suspender/kN	Shear ultimate bearing capacity/kN	ratio	Component failure judgment
A1-A2-ZDLJ1	10.1	70.2	0.14	unspoiled
A1-A2-ZDLJ2	10.1	70.2	0.14	unspoiled
A1-A2-ZDLJ3	9.4	70.2	0.13	unspoiled
A1-A2-ZDLJ4	9.8	70.2	0.14	unspoiled
A1-A2-ZDLJ5	9.4	70.2	0.13	unspoiled
A1-A2´-ZDLJ1	6.0	70.2	0.09	unspoiled

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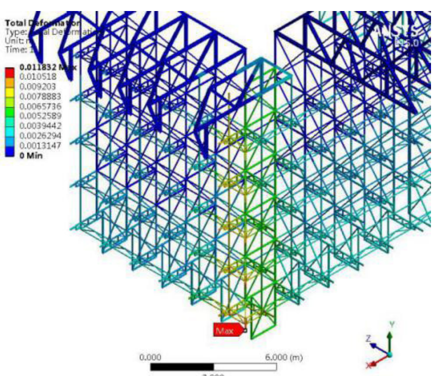
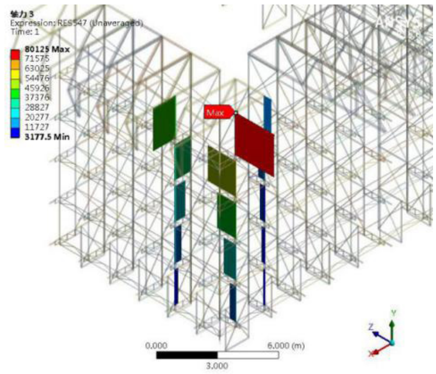
A1-A2´-ZDLJ2	6.2	70.2	0.09	unspoiled
A1-A2´-ZDLJ3	7.2	70.2	0.10	unspoiled
A1-A2´-ZDLJ4	8.6	70.2	0.12	unspoiled
A1-A2´-ZDLJ5	9.8	70.2	0.14	unspoiled
A2-HJ	58.1	187.4	0.31	unspoiled
A2´-HJ	60.1	187.4	0.32	unspoiled

3.1.2 Demolition of Truss A11-HJ

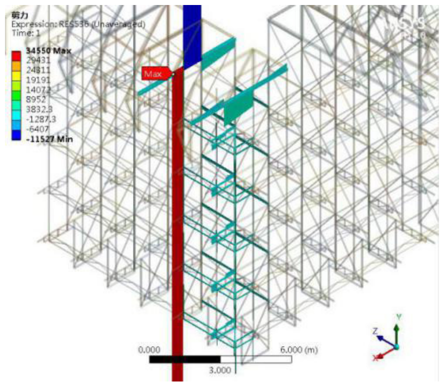
After removing the truss A11-HJ, the response of the structure is shown in Figure 3. The maximum displacement of the surrounding structure after removing the truss is 11.8 mm (Figure (a)). Since A12-HJ links two suspenders, the number suffix of the suspender near the ship's side is added by “a”. The maximum axial force in the adjacent suspender is A12-DG5a (Figure (b). The axial stress is 123.6 MPa. The failure analysis is carried out as shown in Table 5.The plastic bending capacity of the frame connection beams connected with the dismantled truss is 1.91 kN. M. The plastic bending capacity of the walkway connection beams and corner connection beams is 1.07 kN. M. The maximum bending moment of each connection beams is 0.48 kN. m (Figure (d)). The section does not enter into plasticity and plastic hinges will not appear. The maximum stress of the structure occurs in the corner connecting beam, and the maximum stress is 204.9 MPa (Figure (e)).The corner connection beams in A12-HJ plane are numbered A12-JQLJ1~A12-JQLJ5 according to the number of layers. According to Table 6, the maximum shear force of each steel beam section is less than the ultimate shear bearing capacity, and shear failure will not occur. Therefore, progressive collapse of the remaining structure will not occur after A11-HJ is completely invalidated.



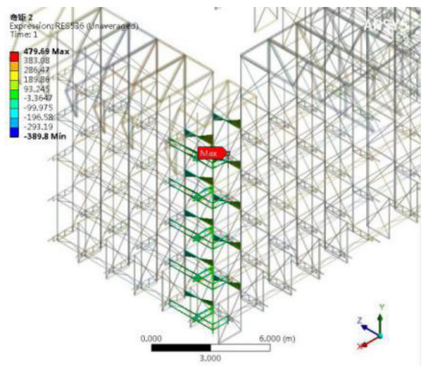
(a) Local structural deformation after removal of truss



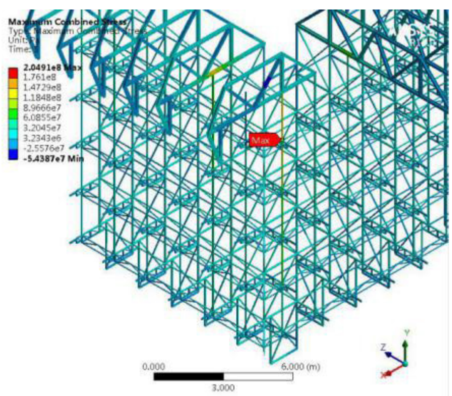
(b) Axis Force of Suspender after Removal of Truss



(c) Shear force of surrounding members after removal of truss



(d) Bending moment of connecting beam after removal of truss



(e) Local structural stress after removal of truss

Figure 3. Local structure response after removing A11-HJ

Table 5. Boom damage judgment after removal of A11-HJ (Based on bom axial deformation)

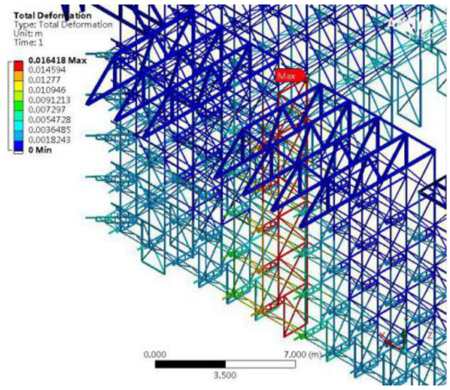
Number of suspender	Displacement of upper end node of suspender/mm	Displacement of lower end node of suspender/mm	Axial deformation of suspender/mm	Deformation limit /mm	Component failure judgment
A12-DG5	-1.99	-3.57	1.58	250	unspoiled

Table 6. Beam failure analysis after removal of A11-HJ (Based on beam end shear)

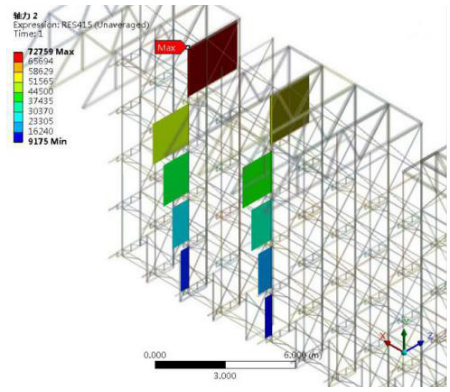
Number	Maximum shear force after removal of suspender/kN	Shear ultimate bearing capacity/kN	ratio	Component failure judgment
A10-A11-ZDLJ1	2.9	70.2	0.04	unspoiled
A10-A11-ZDLJ2	3.2	70.2	0.05	unspoiled
A10-A11-ZDLJ3	4.2	70.2	0.06	unspoiled
A10-A11-ZDLJ4	5.6	70.2	0.08	unspoiled
A10-A11-ZDLJ5	7.1	70.2	0.10	unspoiled
A12-JQLJ1	3.1	70.2	0.04	unspoiled
A12-JQLJ2	3.8	70.2	0.05	unspoiled
A12-JQLJ3	5.3	70.2	0.08	unspoiled
A12-JQLJ4	7.4	70.2	0.11	unspoiled
A12-JQLJ5	9.7	70.2	0.14	unspoiled
A10-HJ	34.5	187.4	0.18	unspoiled
A12-HJ	2.3	187.4	0.01	unspoiled

3.1.3 Demolition of Truss C4-HJ

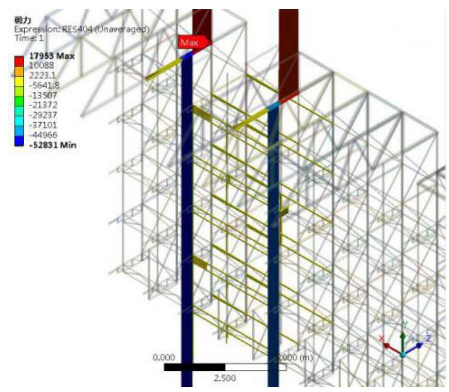
After removing the truss C4-HJ, the response of the structure is shown in Figure 4. The maximum displacement of the surrounding structure after removing the truss is 16.4 mm (Figure (a)). The C3-DG5 has the largest axial force and the axial stress is 112.3 MPa (Figure (b). The failure analysis of C3-DG5 is carried out as shown in Table 7. The plastic bending capacity of the frame connection beam connected with the dismantled truss is 1.91 kN m, and the plastic bending capacity of the corridor connection beam is 1.07 kN m, while the maximum bending moment of each connection beam is 0.41 kN m (Figure (d)). The section does not enter into plasticity and no plastic hinge will appear. The maximum stress occurs at the junction of C3-HJ and C3-DG5, and the maximum stress is 214.5 MPa (Figure (e)). Table 8 shows the results of failure analysis of steel beams after removal of C4-HJ based on shear force. It shows that the maximum shear force of each section of steel beams is less than the ultimate shear bearing capacity, shear failure will not occur, and progressive collapse of remaining structures will not occur after complete failure of C4-HJ.



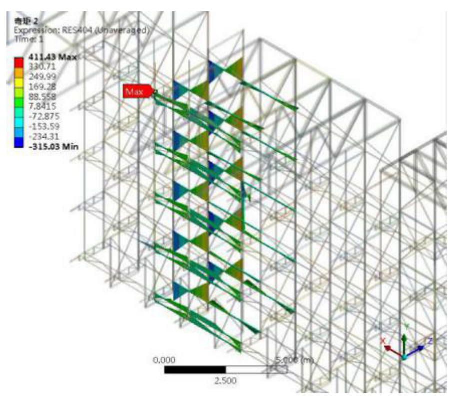
(a) Local structural deformation after removal of truss



(b) Axis force of suspender after removal of truss



(c) Shear force of surrounding members after removal of truss



(d) Bending moment of connecting beam after removal of truss

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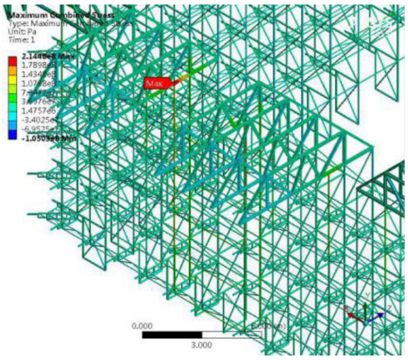

(e) Local structural stress after removal of truss

Figure 4. Local structure response after removing C4-HJ

Table 7. Boom damage judgment after removal of C4-HJ (Based on boom axial deformation)

Number of suspender	Displacement of upper end node of suspender/mm	Displacement of lower end node of suspender/mm	Axial deformation of suspender/mm	Deformation limit /mm	Component failure judgment
C3-DG1	-5.99	-6.17	0.18	215	unspoiled
C3-DG2	-5.57	-5.95	0.38	215	unspoiled
C3-DG3	-4.92	-5.50	0.58	215	unspoiled
C3-DG4	-3.96	-4.80	0.84	215	unspoiled
C3-DG5	-2.41	-3.80	1.39	250	unspoiled
C5-DG1	-5.08	-5.24	0.16	215	unspoiled
C5-DG2	-4.72	-5.05	0.33	215	unspoiled
C5-DG3	-4.16	-4.66	0.50	215	unspoiled
C5-DG4	-3.34	-4.07	0.73	215	unspoiled
C5-DG5	-2.06	-3.22	1.16	250	unspoiled

Table 8. Beam failure analysis after removal of C4-HJ (Based on beam end shear)

Number	Maximum shear force after removal of suspender/kN	Shear ultimate bearing capacity/kN	ratio	Component failure judgment
C3-C4-ZDLJ1	6.0	70.2	0.08	unspoiled
C3-C4-ZDLJ2	6.5	70.2	0.09	unspoiled
C3-C4-ZDLJ3	7.4	70.2	0.11	unspoiled
C3-C4-ZDLJ4	8.5	70.2	0.12	unspoiled
C3-C4-ZDLJ5	9.7	70.2	0.14	unspoiled
C4-C5-ZDLJ1	4.9	70.2	0.07	unspoiled
C4-C5-ZDLJ2	5.5	70.2	0.08	unspoiled
C4-C5-ZDLJ3	6.3	70.2	0.09	unspoiled
C4-C5-ZDLJ4	7.5	70.2	0.11	unspoiled
C4-C5-ZDLJ5	8.4	70.2	0.12	unspoiled
C3-HJ	52.8	187.4	0.28	unspoiled
C5-HJ	44.8	187.4	0.24	unspoiled

According to the above analysis, it can be found that only single truss failures, the remaining structure will not appear progressive damage, indicating that the installation platform structure has good anti-progressive collapse performance. Table 9 gives a comparison of the responses of the remaining structures after removing the three trusses at different locations. It is found that the importance coefficient of the trusses is large, the response of the remaining structures is more significant after removing them.

Table 9. Response of residual structure after removal of trusses

Number of truss	Coefficient of Importance	Maximum displacement of structure/mm	Maximum stress of structure/MPa	Maximum ratio of shear force to bearing capacity of beams	Maximum elongation of adjacent suspenders/mm
A1-HJ	0.1846	29.9	235.4	0.32	1.58
A11-HJ	0.1167	11.8	204.9	0.18	1.58
C4-HJ	0.1267	16.4	214.5	0.28	1.39

3.2 Structural Response to Simultaneous Failure of Two Adjacent Trusses

A1-HJ has the greatest importance, and it is located on the symmetrical plane of the captain direction of the installation platform. Therefore, the structural response of the truss A1-HJ and A2-HJ after simultaneous failure is analyzed, as shown in Figure 5. The maximum displacement of the local structure around the demolished member is 51.5mm (Figure (a)).The largest axial force of the suspender near the dismantled truss is A3-DG5, and the axial stress reaches 171 MPa (Figure (b)).The suspenders of A3 and A2 steel frames are judged. As shown in Table 10, all suspenders will not be damaged. The plastic bending capacity of frame connection beams is 1910N m, the cross-section plastic bending capacity of walkway connection beams is 1070N m, and the maximum bending moment of the connection beams around the dismantled truss is 604.8N m (Figure (d)). No plastic hinges will appear at the end of the beams. The maximum stress of the platform structure is 328.7 MPa (Figure (e)), which appears at the end of A3-A2-ZDLJ5.Table 11 shows the results of failure analysis of the beam after removing A1-HJ and A2-HJ according to the shear force at the end of the beam. It shows that the maximum shear force of the cross-section of the connecting beams and the walkway beams is less than the ultimate shear bearing capacity, and there will be no shear failure, so there will be no progressive collapse of the remaining structures after the truss A1-HJ and A2-HJ fail completely.

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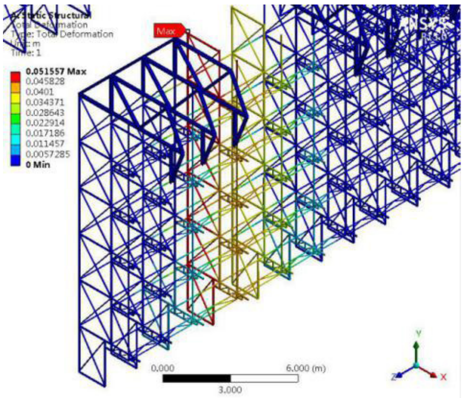
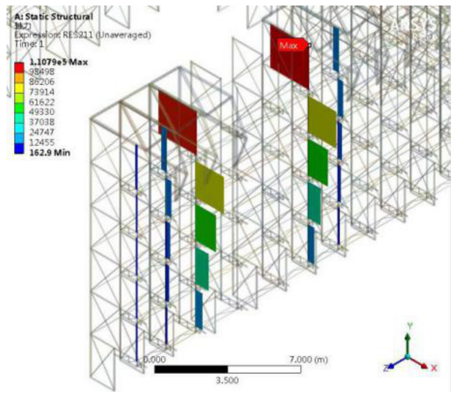
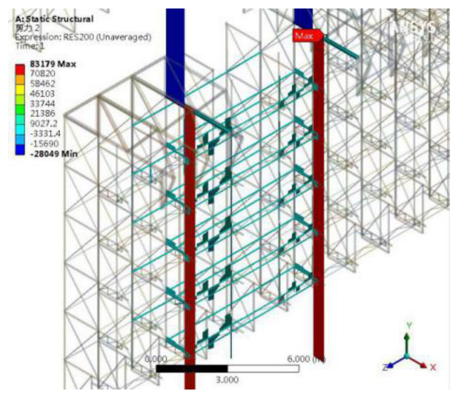


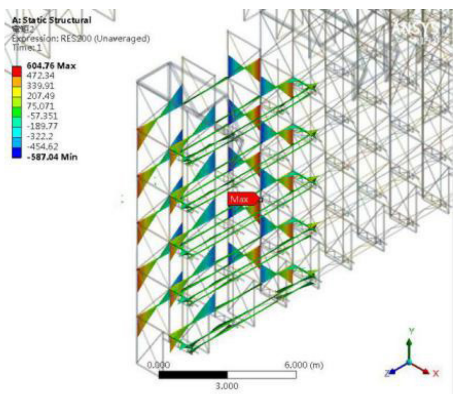
(a) Local structural deformation after removal of truss



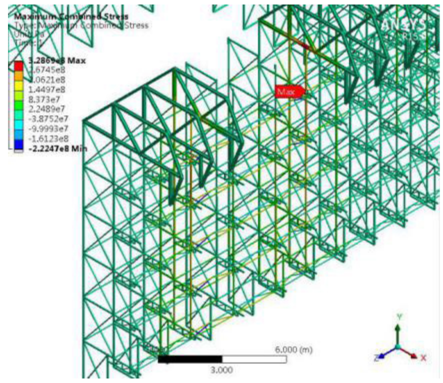
(b) Axis force of suspender after removal of truss



(c) Shear force of surrounding members after removal of truss



(d) Bending moment of connecting beam after removal of truss



(e) Local structural stress after removal of truss

Figure 5. Local structure response after removing A1-HJ, A2-HJ

Table 10. Boom damage judgment after removal of A1-HJ, A2-HJ (Based on boom axial deformation)

Number of suspender	Displacement of upper end node of suspender/mm	Displacement of lower end node of suspender/mm	Axial deformation of suspender/mm	Deformation limit /mm	Component failure judgment
A3´-DG1	-9.93	-10.25	0.32	215	unspoiled
A3´-DG2	-9.23	-9.87	0.64	215	unspoiled
A3´-DG3	-8.11	-9.11	1.00	215	unspoiled
A3´-DG4	-6.51	-7.93	1.42	215	unspoiled
A3´-DG5	-3.99	-6.29	2.30	250	unspoiled
A2-DG1	-8.20	-8.47	0.27	215	unspoiled
A2-DG2	-7.61	-8.15	0.54	215	unspoiled
A2-DG3	-6.69	-7.52	0.83	215	unspoiled
A2-DG4	-5.38	-6.55	1.17	215	unspoiled
A2-DG5	-3.29	-5.18	1.89	250	unspoiled

Table 11. Beam failure analysis after removal of A1-HJ, A2-HJ (Based on beam end shear)

Number	Maximum shear force after removal of suspender/kN	Shear ultimate bearing capacity/kN	ratio	Component failure judgment
A3´-A2´-ZDLJ5	14.9	70.2	0.078	unspoiled
A3´-A2´-ZDLJ4	13.2	70.2	0.078	unspoiled
A3´-A2´-ZDLJ3	11.3	70.2	0.084	unspoiled
A3´-A2´-ZDLJ2	9.3	70.2	0.098	unspoiled
A3´-A2´-ZDLJ1	8.5	70.2	0.137	unspoiled
A2-A1-ZDLJ5	12.6	70.2	0.179	unspoiled
A2-A1-ZDLJ4	11.5	70.2	0.164	unspoiled

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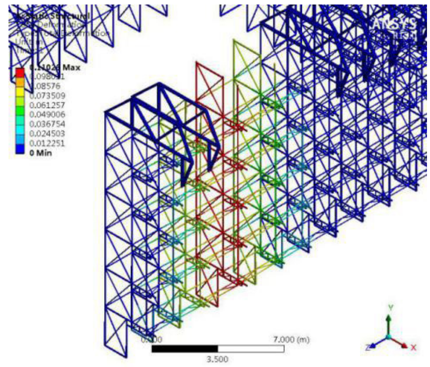
A2-A1-ZDLJ3	9.8	70.2	0.140	unspoiled
A2-A1-ZDLJ2	8.6	70.2	0.123	unspoiled
A2-A1-ZDLJ1	8.4	70.2	0.120	unspoiled
A3´-ZD5	16.3	148.5	0.110	unspoiled
A3´-ZD4	15.3	148.5	0.103	unspoiled
A3´-ZD3	13.2	148.5	0.089	unspoiled
A3´-ZD2	10.4	148.5	0.070	unspoiled
A3´-ZD1	7.3	148.5	0.049	unspoiled
A2-ZD5	13.6	148.5	0.092	unspoiled
A2-ZD4	12.7	148.5	0.086	unspoiled
A2-ZD3	10.8	148.5	0.073	unspoiled
A2-ZD2	8.5	148.5	0.057	unspoiled
A2-ZD1	6.0	148.5	0.041	unspoiled
A3´-HJ	86.5	187.4	0.46	unspoiled
A2-HJ	71.4	187.4	0.38	unspoiled

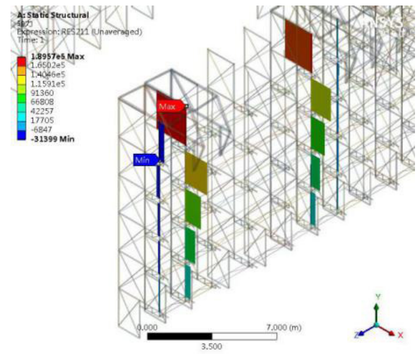
The sum of A1-HJ and A2-HJ importance coefficients is the largest in the adjacent two trusses. According to the conclusion of section 2.1, it can be concluded that the adjacent two trusses fail and the remaining structure will not collapse continuously.

3.3 Structural Response of Three Trusses to Simultaneous Failure

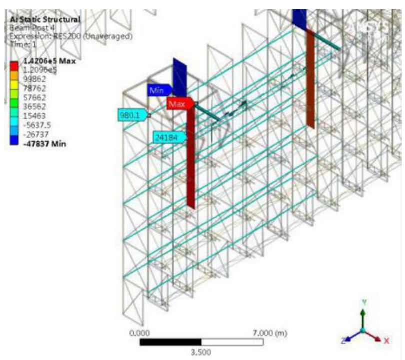
A1-HJ is of the greatest importance and it is located on the symmetrical plane in the direction of the captain of the installation platform. Therefore, the structural responses of three trusses in this area after simultaneous failure are analyzed, as shown in Figure 6. The steel frame area of hanging layer A1 has a large displacement, reaching 110 mm (Figure (a)).At the lower chord of A3'-HJ, the maximum shear force is 142 kN(Figure (c). The maximum shear force of the corridor connection beam is 24.2 kN. The ultimate shear force of the frame connection beam is 70.2 kN. The ultimate shear force of the frame connection beam is 0.98 kN. The ultimate shear force of the frame connection beam is 116.1 kN. No shear failure occurs. The maximum positive bending moment is 1.3kN m (Figure (d)) and the plastic bending capacity is 1.07kN m, which has entered the plasticity. The maximum negative bending moment occurs at the end of A3'-ZD5, which is 1.87kN M. The plastic bending capacity of the cross-section of the walkway beam is 3.2kN m, and bending failure will not occur. The maximum bending moment is 0.97 kN m and the plastic bending capacity is 1.91 kN m, so there will be no bending failure. The maximum stress of the structure is 576 MPa (Figure (e). It occurs at the end of A3-A2-ZDLJ5, far exceeding the material failure stress of 450 MPa. At the same time, the material failure stress is also exceeded by the lower chord of A3'-HJ and A3-HJ, the end of A3'-A2'-ZDLJ5 and the end of A3-A2-ZDLJ4.



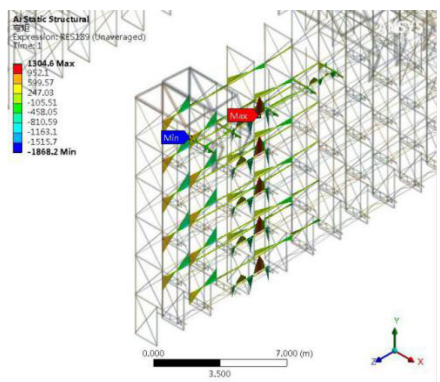
(a) Local structural deformation after removal of truss



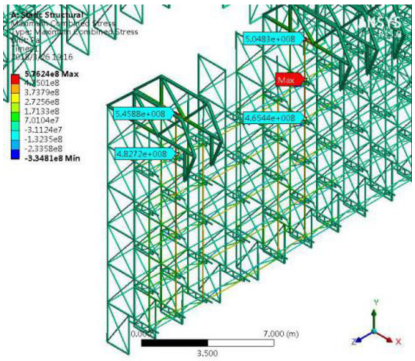
(b) Axis force of suspender after removal of truss



(c) Shear force of surrounding members after removal of truss



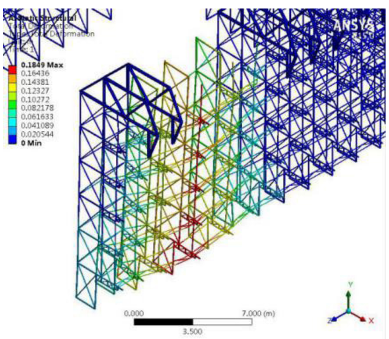
(d) Bending moment of connecting beam after removal of truss



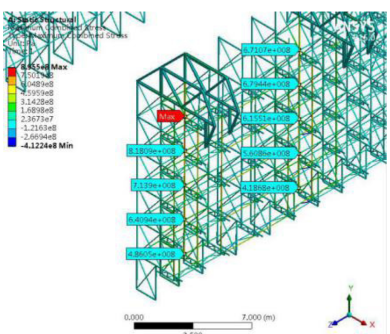
(e) Local structural stress after removal of truss

Figure 6. Local structure response after removing A1-HJ, A2-HJ, A2´-HJ

Due to the end failure of A3’-A2’-ZDLJ5, A3-A2-ZDLJ4, A3-A2-ZDLJ5 and the end failure of the connection between A3-HJ and A3’-HJ lower chord and suspender, the finite element corresponding to this location needs to be "killed" in the software. The remaining structures are further calculated and analyzed, as shown in Figure 7. A large displacement occurred in A1 steel frame area, the maximum displacement was 184.9 mm (Figure (a)). As shown in the stress diagram (b), except that the end of A3-ZD1 does not exceed the material damage stress limit, material damage occurs at the end of A3 steel frame and other aisle beams of A3’ steel frame, and material damage occurs at the hangers connected with the aisle beams.



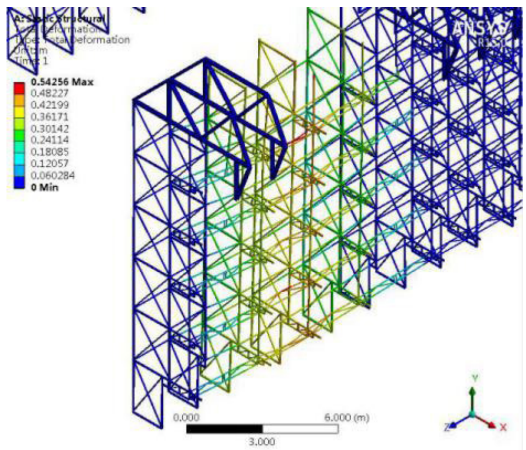
(a) Local structural deformation after removal of failure members



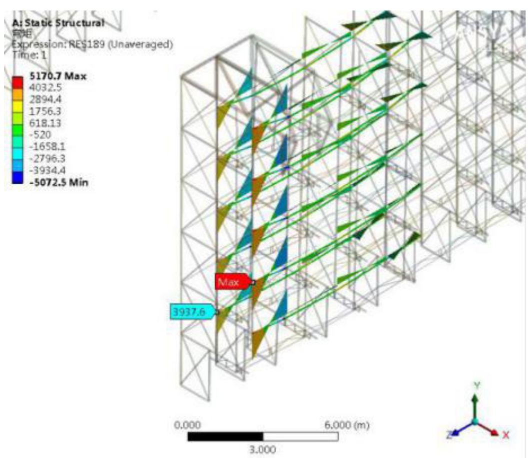
(b) Local structural stress after removal of failure components

Figure 7. Local structural response after removing the invalid component

The damage element is killed and the residual structure is analyzed. The calculation results are shown in Figure 8. The maximum displacement of the dismantled truss area is 542.5mm (Figure (a)); the minimum bending moment of the frame connection beam between A3’ and A2’ is 3.9kN.m (Figure (b)), forming plastic hinges. As shown in Table 12, the plastic hinge rotation angles at the ends of each frame connection beam exceed the limit of 6.0 plastic hinge angle of the steel beam. It can be judged that the end sections of each frame connection beam between A3’ and A2’ is destroyed. Large-scale collapse and destruction occurred in the region. Therefore, when A1-HJ, A2-HJ and A2’-HJ are demolished at the same time, progressive damage will be caused, leading to the collapse of large-scale structures. In the process of using the structure, it is necessary to pay attention to the truss in this area to prevent the simultaneous failure of A1-HJ, A2-HJ and A2’-HJ, and to avoid unnecessary losses.



(a) Local structural deformation after removal of failure members



(b) Bending moment of connecting beam after removing failure members

Figure 8. Local structural response after removing the invalid component

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Table 12. The angle of frame connection beam between A3´ and A2´

Number	Displacement Difference of Beam End Joints/mm	Beam length/ mm	Corner of beam end/°	Deformation limit/°
A3´-A2´-KJLJ5a	246	2000	7.1	6
A3´-A2´-KJLJ5b	286	2000	8.2	6
A3´-A2´-KJLJ4a	231	2000	6.6	6
A3´-A2´-KJLJ4b	280	2000	8.0	6
A3´-A2´-KJLJ3a	230	2000	6.6	6
A3´-A2´-KJLJ3b	270	2000	7.8	6
A3´-A2´-KJLJ2a	227	2000	6.5	6
A3´-A2´-KJLJ2b	275	2000	7.9	6
A3´-A2´-KJLJ1a	256	2000	7.4	6
A3´-A2´-KJLJ1b	304	2000	8.7	6

4. Conclusion

This paper simulates the response of the remaining structure after the failure of some trusses of the secondary shielding installation platform for A-type tank, and evaluates the anti-progressive collapse ability of the installation platform structure. The conclusions are as follows:

(1) The failure of three trusses with larger importance coefficient is simulated separately. It is found that the importance coefficient of components is larger, the influence of failure on the residual structure is greater, and the failure analysis of two trusses is also guided.

(2) No further progressive damage will occur after the two trusses of the installation platform completely are invalid.

(3) When A1-HJ, A2-HJ and A2'-HJ are demolished at the same time, progressive damage will be caused locally, which can lead to the collapse of large-scale structures. To prevent the failure of the suspenders ZD-1 and ZD-2 at the same time and avoid unnecessary losses, it is necessary to pay attention to the suspenders of this area.

(4) The vertical bearing components of the installation platform all bear tension which do not need to consider the instability, and the spatial arrangement of the structure is relatively flexible. Therefore, the installation platform is a relatively promising structural form.

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