Progressive Collapse of Double-Layer Space Trusses

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Summary

The results of the analysis of two square-on square double-layer grid space truss systems are used to compare the collapse behaviour of two physical steel models considering different parameters including loading and member dimensions. It is shown that the collapse behavior of this type of structure involves yielding of tension members and more importantly buckling of compression members. More often the collapse behavior is due to the buckling of compression members which is associated with sudden and catastrophic failure. Using numerical modelling, the progressive collapse behavior of the structure subject to increasing applied load can be traced which is illustrated and discussed. The improvement and benefits from the numerical modelling results are also identified to get a better understanding of the collapse behaviour of double-layer grid space truss systems.

Keywords: double-layer space truss, finite element model, progressive collapse.

1. Introduction

The development of space structures has really played a major role in engineering design, allowing architects and engineers to push the boundaries to design even bigger and more exciting structures. A double-layer grid is a typical example of a prefabricated space structures which has become a popular form of space frame [1]. Applications of double-layer space structures are numerous and include the notorious Hartford Coliseum space roof truss in Connecticut USA. However, the unfortunate catastrophic failure of this particular space roof truss in 1978 resulted in extensive research on the progressive failure of space trusses [2, 3]. Obviously, the aftermath of any building collapse is disastrous and very unfortunate, however, the failure of the Coliseum roof along with other failures in more conventional structures has led structural engineers to realised the importance of progressive collapse in buildings [4].

Fig. 1: Picture of the collapse of the Sultan Mizan Zainal Abidin Stadium , Malaysia [5]

However, although past codes of practice provide design procedures for very long bridges and vulnerable structures, recent codes still lack general applicability to double-layer grids. Apparently, the awareness of such failures has rarely been appraised, as engineering disasters still occur. Consider, for example, the football stadium curved double-layer roof in Terengganu, Malaysia shown in Fig.1 which collapsed one year after completion on 2nd of June 2009. It was reported that the primary cause for the collapse was due to incomplete consideration of the support conditions for the roof structure [6]. Double-layer grids are used to cover large spans with few or even no intermediate supports. This is possible partly because double-layer grid are usually highly statically indeterminate where point loads are distributed widely within the structure [8]. A correctly designed double-layer space truss can usually carry load through its interconnected structural members even after the failure of the first compression member, possibly possessing a reserve of strength beyond its elastic capacity. In general, it is appropriate to consider the elastic load displacement behaviour of the structure until the first compression member fails when assessing the load-carrying capacity of a double-layer space truss. Post-buckling reserves of strength is however not taken into account even though a typical double-layer structure is a highly statically indeterminate structural system. Because the members are fabricated from steel, which is a ductile material, carefully designed double-layer structures may possess reserves of strength in excess of their elastic capacity [8].

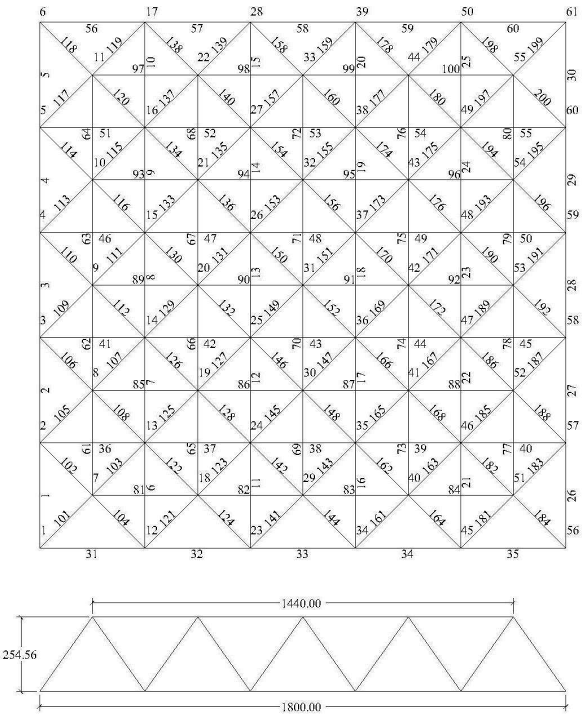
It is important to understand that the collapse behaviour of double-layer space trusses is important, before carrying out the design of the structure. By identifying the ultimate load and failure mode of the truss after a non-linear analysis, designers can design the space truss with a suitable factor of safety between the working and collapse loads. Because a double-layer space truss is usually a highly statically indeterminate structural system, there may be a mistaken belief that a large number of members can fail before a complete collapse of the structure is imminent. However, this is not necessarily true and the failure of just one member can result in the progressive collapse of the entire structure. The failure modes associated with any particular double-layer grid involve both yielding of tension members and, more critically, buckling of compression members. It is also very dependent on the particular type of grid for example square-on-square, square-on-diagonal, diagonal-on-square and also the support conditions.

As the sophistication of finite element software has developed in recent years, more engineers have become involved in using finite element simulation to undertake various types of progressive collapse analysis. It is preferable to use finite element simulation to undertake a full non-linear collapse analysis of important space structures because experimental work is both costly and tedious.

Many of the earlier researchers, such as Collins [9] and Parke [8] investigated the collapse behaviour of space structures, both theoretically and experimentally. Parke studied the progressive collapse of double-layer grids and the possibility of improving space truss collapse behavior, while Collins studied strut post buckling and the associated collapse behaviour of double-layer grids. Both of these researchers provided a base on which more in depth studies of double-layer grid collapse behaviour can be developed. The catastrophic collapse of some double-layer grids makes it necessary to model the full non-linear behaviour of the grids in order to determine the true safety factor on the critical loading condition. There are many factors which serve to complicate modelling of the non-linerar response of space structures, for example determining the full non-linear material characteristics and also predicting the redistribution of forces after the progressive failure of compression members [8, 10]. Often, several load effects occurring in a structure need to be carefully appraised in the structural design especially for important long span space truss structures as tragic events concerning their collapse still continue to happen on rare occasions. One way of avoiding this problem, is to model accurately the geometry, boundary conditions, connection and material nonlinearities in the finite element analysis. In this present paper the method used to develop the nonlinear finite element model is able to predict the response of double–layer space truss sensitive to progressive collapse. If the finite element modelling approach adopted proves useful, it will be used in future work to predict the progressive collapse behaviour of double-layer space truss taking into consideration different support conditions and various span to height ratios including automatic removal of one or more highly stressed members.

2. Modelling Approach

Fig.2 shows the plan and elevation of the top-chord, web and bottom-chord members of a double-layer space truss model structure that has been fabricated and tested to collapse as described by Parke [8]. The model of the square-on-square double-layer space structure, used tubular and solid steel bar members with bottom chord plan dimension of 1.8m square. The bottom-chord members are arranged in a 5x5 square grid and the top chord members are in a 4x4 square grid. The model with a depth of 254.56mm allows every member to be the same length. The whole space truss model was simply supported and rested on supporting columns positioned at each of the four corners of the bottom chord. Physical models of this type of double-layer space truss structure had been used by Parke to investigate the possibility of improving space truss collapse behaviour. Two of his models named Model 1 and Model 2, which are considered in this investigation, are identical in terms of spans, member arrangements (Fig.2) and support conditions (Fig.3). However, the loading conditions and the types of some of the web members for the two models are different. Fig.2 also shows the node and member numbers used for Model 1 and Model 2. The three dimensional view of the model structure is shown in Fig.3. Fig.4 and Fig.5 show the type of each member used for Model 1 and Model 2 respectively. The properties of the member types T1, T5 and T6 are given in Fig.6. This investigation simulates the physical model shown in Fig.3 using the general purpose finite element computer package ABAQUS. Beam elements, type B32 which correspond to circular hollow tubes were chosen to model the structural members. Exact characteristics of the physical model given in Parke [8] were used in ABAQUS where the performance of the numerical models were investigated [11].



Dimensions in mm

Fig.2: Plan and elevation of Model 1 and Model 2 [8].

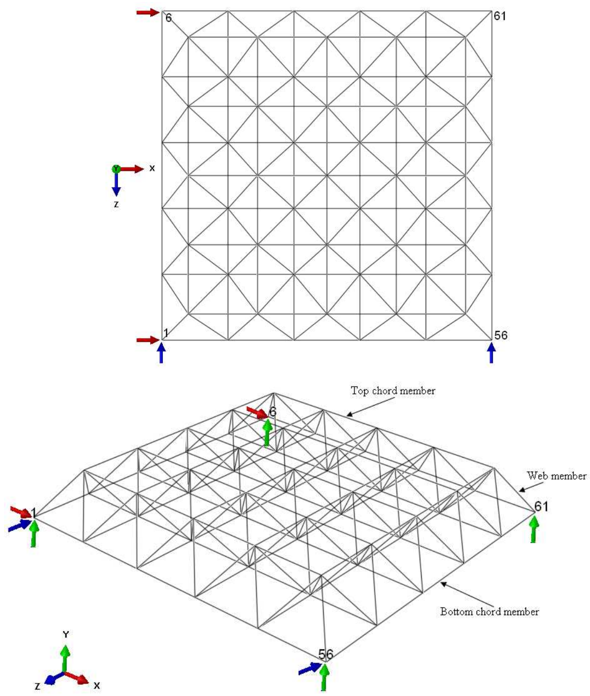


Fig.3: Plan and three dimensional view of Model 1 and Model 2 showing the boundary conditions.

2.2 Formation of the numerical Models 1 and 2

Prior to carrying out the analysis, a three dimensional configuration of the structure as shown in Fig.3, was created using a computer programming language called FORMIAN [12]. It was assumed that the truss members are rigidly connected together at all the intersection points. Hence, this produces 200 beam finite elements that are connected together at 61 nodes as shown in Fig. 2. Once the skeleton layout has been formed in FORMIAN the whole configuration is transferred into ABAQUS together with the other important input data which are necessary for the finite element analysis, such as the sectional and material properties and the loading and boundary conditions of the model. The cross sectional dimensions of the different member types are given in Table 1. Material nonlinear tension behavior is incorporated so that plasticity effects are captured. The material properties determined from individual member tests assigned to each member type are displayed in Fig.6.

2.2.1Boundary conditions

Each of the four corner supports provides constraint along the vertical (y) axis where u_y=0, and free rotation about the three principal axes. However, the support at node 1 is also constrained in the horizontal (x and z) direction where u_x=0 and u_z=0. Supports at node 6 and 56 are also constrained along one of the axes in the horizontal plane, i.e. u_x=0 and u_z=0 respectively. The support at node 61 is unconstrained against translation in the horizontal plane.

2.2.2 Loading conditions

Model 1 and Model 2 have the same configuration and boundary conditions. However, their loading conditions are different. Model 1 is centrally and vertically loaded with one concentrated load at node 31, situated at the center of the top chord. Model 2 is symmetrically and vertically loaded at four points on the top chord i.e. at nodes 19, 21, 43 and 41 (see Fig.2).

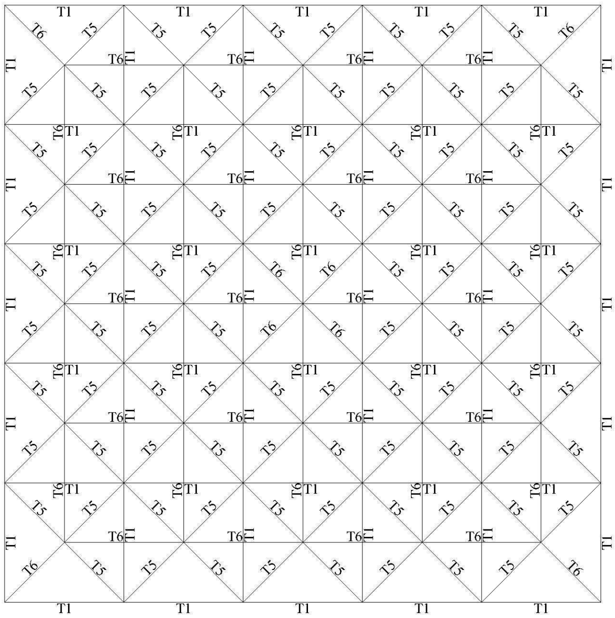


Fig.4: Double-layer space truss with member types T1, T5 and T6 for Model 1[8].

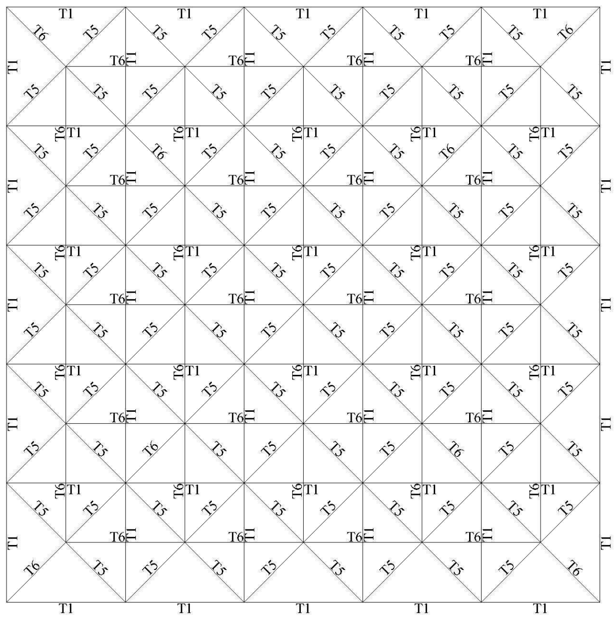


Figure 5: Double-layer space truss with member types T1, T5 and T6 for Model 2 [8].

Table 1: Cross sectional dimensions of member types

Profile	Member Type		
	T1	T5	T6
Shape	Tubular	Tubular	Solid Circular
Diameter (mm)	4.76	9.52	10.00
Thickness (mm)	0.91	0.91	-
Cross Sectional Area (mm^2)	11.00	24.61	78.54

Table 2: Material properties for the different member type in compression

Compression Material Behaviour	Member Type	
	T5	T6
Flexural Buckling Stress	319.24N/ mm^2	312.68N/ mm^2

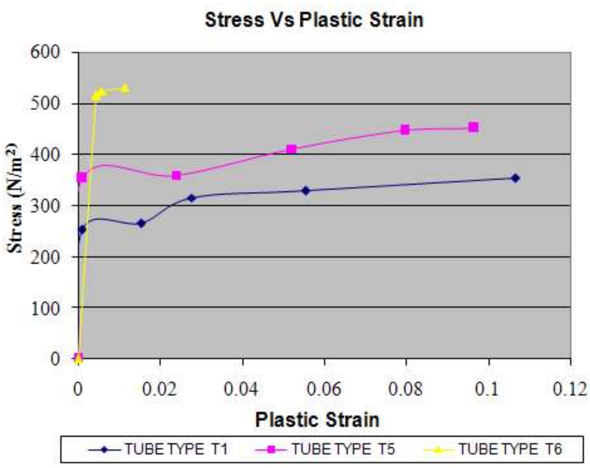
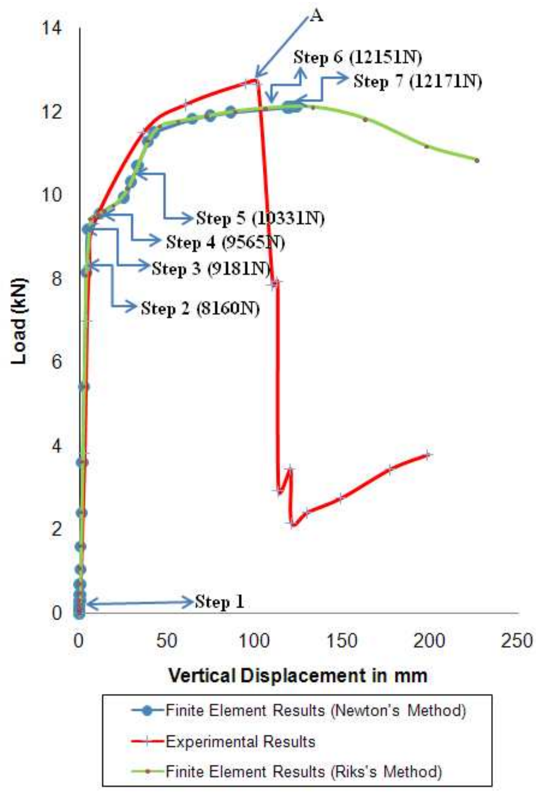


Fig. 6: Tensile Stress Vs Strainbehaviour for Member Type T1, T5 and T6 [8].



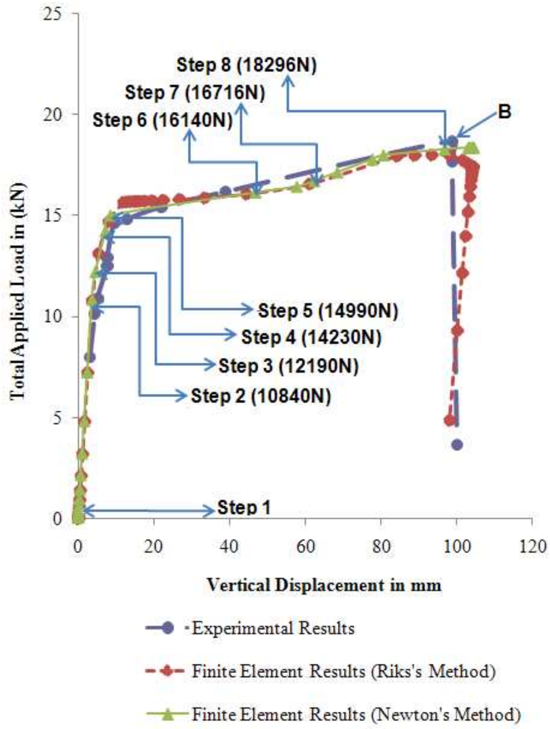


Fig.7: Load Vs central node displacement measured at node 31 for Model 1.

Fig.8: Load Vs vertical displacement for node number 25 for load applied at four top chord nodes (19, 21, 24 and 43) of Model 2.

3.0 Finite Element Study of the Double-Layer Space Truss

3.1 Discussion related to Model 1

The stress ratio for a tension member is taken as the member stress divided by the yield stress and for a compression member is taken as member stress divided by critical flexural buckling stress. When Model 1 is subject to an impose load of 8160N (see Step 2 in Fig.7) Member 13, 18, 43 and 48 are the most highly stressed bottom chord tension members with a stress ratio of 1.00 and Member 62, 63, 78, 82, 83, 98 and 99 are the most highly stress compression members with a stress ratio of 0.14. Fig.7 shows the load displacement response of node 31 obtained experimentally by Parke [7] and also obtained theoretically using the finite element method. The load displacement behaviors of both responses have a similar pattern and are closely related up to a value of a displacement of about 100mm. Both responses exhibit a ductile post-elastic load displacement behaviour. As the imposed load was increased to 9181N (at Step 3 in Fig.7) Member 8, 23, 38 and 53 yielded since the stress ratio value is equal to 1.00 (See Table 3). Yielding was then followed in Member 3, 28, 33 and 58 since these member now have a stress ratio 1.00 (See Table 3) at the corresponding load of 9565N (at Step 4). As the load increased to 10331N (at Step 5) and then to 12151N (at Step 6) Member 2, 4, 27, 29, 32, 34, 57 and 59 and then Member 7, 9, 22, 24, 37, 39, 52 and 54 yielded respectively. Yielding continues in the bottom chord members until the imposed load reaches a value of 12171 N (at Step 7) where the top chord compression Member 70, 71, 90 and 91 become unstable and buckle as indicated in Table 3 where their stress ratio is equal to 1.00. Table 3 demonstrates the collapse behaviour of the model. The yielding of the bottom layer in tension allows a slow and graceful collapses with load being maintained as the stress ratios of the yielded members exceed 1.00. It can be seen from Fig.7 that the experimental results show that the first catastrophic failure occurs when bottom chord Member 43 ruptured and had a complete tensile failure. The tensile failure or rupture of material type T1 was found to have a strain value of 0.10688 as shown in Fig.6. However, in the finite element analysis the strain in Member 43 at the

Table 3: Stress ratios for the members of Model 1

Finite Element Analysis	Member Numbers					
	Tension Members (bottom chord)					Compression members (top chord)
	13, 18,43,48	8, 23, 38,53	3, 28, 33,58	2,4,27,29, 32, 34, 57,59	7,9, 22, 24, 37, 39, 52, 54	70, 71, 90, 91
Step 2	**1.00**	0.80	0.81	0.76	0.51	0.11
Step 3	**1.00**	**1.00**	0.93	0.87	0.61	0.15
Step 4	**1.02**	**1.01**	**1.00**	0.93	0.63	0.21
Step 5	**1.13**	**1.10**	**1.08**	**1.00**	0.68	0.36
Step 6	**1.37**	**1.34**	**1.32**	**1.01**	**1.00**	0.99
Step 7	**1.38**	**1.35**	**1.33**	**1.01**	**1.00**	**1.00**

Table 5(a): Stress Ratios for members of Model 2.

Finite Element Steps	Member Numbers				
	Tension Members (bottom chord)				
	3, 28, 33, 58	2, 4, 32, 34, 27, 29, 57, 59	7,8,9,22,23,24,37,38,39,52,53	1, 4, 5, 30, 31, 35, 56, 60	12, 13, 14, 17, 18, 19, 42, 43, 44, 47, 48, 49
Step 2	**1.00**	0.97	0.70	0.72	0.38
Step 3	**1.00**	**1.00**	0.89	0.81	0.45
Step 4	**1.00**	**1.00**	**1.00**	0.95	0.73
Step 5	**1.00**	**1.00**	**1.00**	**1.00**	0.88
Step 6	**1.04**	**1.04**	**1.04**	**1.08**	**1.03**
Step 7	**1.08**	**1.09**	**1.14**	**1.14**	**1.06**
Step 8	**1.18**	**1.20**	**1.19**	**1.29**	**1.19**

Table 4: Value in percentage of member strain divided by strain at rupture for Member 43, Type T1.

Finite Element Analysis	Axial Strain	Strain at rupture for type T1	Percentage
Step 2	0.00084	0.10688	0.8%
Step 3	0.00144		1.4%
Step 4	0.00232		2.2%
Step 5	0.01848		17.3%
Step 6	0.04886		45.8%
Step 7	0.10414		97.6%

Table 5(b): Stress Ratios for members of Model 2.

Finite Element Steps	Members	
	Tension Members (web)	Compression members (web)
	103,120, 182, 197	101,118, 184,199
Step 2	0.28	0.17
Step 3	0.38	0.20
Step 4	0.57	0.24
Step 5	0.64	0.25
Step 6	0.95	0.63
Step 7	**1.02**	0.71
Step 8	**1.02**	**1.20**

failure load (Step 7) was given as 0.10414 as shown in Table 3.7. This is within the range of experimental error for the tensile test response data used in the finite element analysis because the failing stress is 97.6% (Table 4) of that obtained from the tensile test results.

3.2 Discussion related to Model 2

From the finite element analysis results it was observed that the stress ratios for model 2 at the end of the linear elastic range corresponding to a total load on the structure of 10840N (at Step 2 in Fig.8). At Step2 in Fig.8 members 3, 28, 33 and 58 are the most heavily stressed bottom chord tension members having a stress ratio of 1.00, indicating that the forces in the members are just sufficient to cause the members to yield. Whereas, the top chord compression members 62, 63, 98, 99 etc, are the most heavily stressed top chord compression members with a stress ratio of 0.19, which is considered to be very low or under stressed when compared with the value for tension members. However, the four corner web compression members 101, 118, 184 and 199 have a stress ratio of 0.11. Fig.8 shows the load displacement behavior of Model 2 obtained theoretically using the finite element method and experimentally by Parke. The graphs show close agreement between the theoretical and experimental displacement over the linear range and the early portion of the non linear range. The nonlinear range is associated with the strain hardening of the yielding tension members. As the imposed load is increased to 12190N (at Step 3 in Fig.8) members 2, 4, 32, 34 etc. began to yield. This is shown by the stress ratio value of 1.00 as given in Table 5(a) & (b). The next group of members to yield are members 7, 8, 9, 22, 23, 24, 37, 38, 39, 52 and 53 when the structure was subject to a total load of 14230N (Step 4). At 14990N (Step 5) members 1, 5, 31, 35 etc. started to yield, followed by members 12, 13, 14, 18, 37, 43, 47 and 48 which yielded when the load reached 16140N (Step 6). When the load reached 16716N (Step 7), the web members 103, 120, 182 and 197 yielded. Finally when the load on the structure reached 18296N (Step 8), the corner web members 101, 118, 184 and 199 buckled in compression. Also, point B on the experimental curve signifies the failure of Model 2 caused by buckling of the corner web compression member 118 when Model 2 was subjected to a total load of 18598N.

3.3 Discussion Related to Riks's Method

In order to simulate the nonlinear behavior, two different approaches were used namely the Newton's Method [13] and Riks's Method [14]. In this case study Newton's method was able to predict the collapse behavior for both models as shown in the experimental results up to where the first member failed but unable to capture the post buckling behaviour. In an attempt to capture the post buckling behavior the authors used Riks's Method. In applying Riks's Method, it was observed that the numerical simulation for Model 2 (see Fig.8) was in very good agreement with the experimental results however, not such a close agreement was obtained for Model 1. Therefore further work is needed by the authors in order to successfully capture the post buckling behaviour for Model 1 possibly by investigating in greater detail the material nonlinearity of the model steel members.

The non-linear behavior of space truss structures involves both geometric and material nonlinearity which require careful modeling in order to predict member buckling and/or complete collapse. Using the Newton's Method, the increase in loading and the corresponding decrease in stiffness can only be followed provided the structural stiffness does not decrease to zero However if zero or negative stiffnesses occur then the load-displacement response can be followed using Riks's Method which allows the simulation of the post ultimate behavior in the response of the space truss structure. Fig.7 and Fig.8 show the results obtained for Models 1 and 2 respectively using Riks's Method to follow both the softening and possible snap through behavior [13, 14, 15].

4.0 Conclusion

This research showed that insight into how sensitive a double-layer space truss is to progressive collapse is most important. The numerical simulation presented was undertaken in order to verify the finite element analysis with existing experimental results. Good agreement between the experimental and finite element results has been obtained. The proposed methods give a similar prediction of the space truss collapse process up to the point where it signifies the first failure in both models. This demonstrates that a relatively simple approach can be used to model the collapse behaviour where only tensile yielding results in a ductile structural behaviour. However where buckling of compression members is the primary cause of collapse the load-displacement response can be brittle and it is therefore necessary to simulate the entire collapse pattern beyond point A and B (see Fig.7 and Fig.8) using Riks's Method. This is important so that the full phenomena of progressive collapse can be understood clearly. In addition, where the collapse is triggered by the sudden buckling of a compression member careful consideration must also be given to assess the level of member imperfection to be used in the analysis as this may determine the ultimate capacity of the structure. It is hoped to investigate further the above mentioned points in subsequent work.

5.0 Acknowledgement

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