3.1 Introduction

In this chapter, different forms of space structures are introduced, followed by a discussion of collapse accidents around the world and the collapse mechanism of different types of space structures. At the end of this chapter, a progressive collapse analysis example for a double-layer grid space structure is demonstrated using the programs Abaqus® and SAP2000 (CSI, 2013).

3.2 Major Types of Space Structures

Space structures have been widely used in different types of structures, from long-span to mid-span frames and also short enclosures, closed roofs, floors, exterior walls, and canopies. There are several major types of space structures used in current construction projects, such as double-layer grids, latticed shells, membrane structures, and tensegrity structures.

3.2.1 Double-Layer Grids

Double-layer grids are one of the most popular structures used in current construction practice. They consist of top and bottom square grids with nodal joints connected by diagonal struts. Different configurations of the top and bottom layers can make different grid types. This type of construction resembles a pyramid shape. The steel bars are linked together by the joints to form a uniform roof structure.

3.2.2 Latticed Shells

Latticed shells can be built by either a single-layer or a double-layer grid. For a long-span single-layer grid, the connections are normally
designed as rigid to provide rigidity. However, for double-layer grid, due to its greater redundancy and indeterminacy, the joints can be designed as pinned.

Domes are one of the commonly used lattice shell structures. There are various types, such as ribbed domes, Schwedler domes, three-way grid domes, lamella domes, and geodesic domes.

A barrel vault is another latticed shell structure. It features forms of a cylindrical-shaped shell, which has a surface that can be easily modified due to its zero curvature. The location and type of supports between the members also influence the vault’s structural behaviour.

3.2.3 Tensegrity Systems
As shown in Figure 3.1, tensegrity structures are self-equilibrium systems composed of continuous prestressed cables and individual compression bars. They are one of the most promising solutions for large-span space structures due to their superlight weight. The concept of tensegrity was first conceived by R.B. Fuller (1975), reflecting his idea of “nature relies on continuous tension to embrace islanded compression elements.” D.H. Geiger et al. (1986) made use of Fuller’s thought and designed an innovative “cable dome” in the circular roof structures of the gymnastic and fencing arenas (Figures 3.1 and 3.2) for the Seoul Olympic Games (Geiger et al., 1986).

![Image of a tensegrity structure for the Seoul Olympic Games.](Photo taken by the author.)

**FIGURE 3.1** Interior of the fencing arenas for the Seoul Olympic Games. (Photo taken by the author.)
3.2.4 Membrane Structures
Membrane structures are a type of lightweight space structure. The membrane works together with lattice shell or tensegrity structures (such as the gymnastic and fencing arenas for the Seoul Olympic Games; Geiger et al., 1986), as shown in Figure 3.2. Some are made as inflatable structures.

3.3 Design Guidance for Space Structures to Prevent Disproportionate Collapse

Space structures consist of a large number of structural members. For most types of space structures, due to their structural redundancy, most designers presume that a progressive collapse will not be triggered when the loss of an individual member occurs. However, as it will be introduced in Section 3.4, there have still been a number of space structure collapse incidents that have been reported worldwide.

So far, there are few design codes with detailed requirements for progressive collapse prevention designs of space structures, although several past codes of practice provide design procedures for very long bridges.
3.4 Space Structure Collapse
Incidents around the World

In this section, an introduction of collapse incidents of space structures around the world is made. The cause of the collapse and the failure mechanism of each incident are introduced.

3.4.1 Partial Collapse of Charles de Gaulle Airport Terminal 2E
On May 23, 2004, a portion of Terminal 2E’s ceiling collapsed near Gate E50. The structure was designed with a 300 mm thick curved concrete shell with a span of 26.2 m, which was precast in three parts, as shown in Figure 3.3.

The two sides of the structure were externally strengthened with curved steel tension members, which were propped with struts. The shell rested on two longitudinal support beams that were supported and tied back to the columns. At the location of the failure, there were large openings for access to gangways. The external steelwork and shell were enclosed with glass.

The investigation report of the Ministry of Transportation (Conseil National des Ingenieurs et Scientifiques de France, 2005) found a number of causes for the collapse. The main reason was that the steel dowels supporting the concrete shell were too deeply embedded into it, which caused cracking in the concrete. In turn, this led to a weakening of the roof. The cracks were formed due to high stress caused in

FIGURE 3.3 Partial collapse of Charles de Gaulle Airport Terminal 2E. (From https://upload.wikimedia.org/wikipedia/commons/1/1b/Terminal_2E_CDGCollapse.png. Free licence.)
the construction stage and cycles of stress from differential thermal and moisture movements.

The investigation also found that the structure had little margin for safety in design, and a combination of factors led to the major collapse, including

- High flexibility in the structure under dead load and external actions
- Cracking that may have resulted from insufficient or misplaced reinforcement
- Lack of robustness and redundancy to transfer loads away from a local failure
- High local punching stresses where the struts were seated in the concrete shell
- Weakness of the longitudinal support beam and its horizontal ties to the columns

The above design fault caused progressive collapse between the concrete shell and curved steel tension member and struts.

3.4.2 Snow-Induced Collapse of Double-Layer Grid Space Structure, Hartford Civic Center

Heavy snow is another major cause for the collapse of space structures. O’Rourke and Wikoff (2014) described an investigation into about 500 roof collapse incidents that occurred in the northeastern United States during the winter of 2010–2011. The major reason for these collapses was the snow load exceeded the design load required by the building code or the structural member was designed with a structural capacity that was significantly less than that required by the building code.

One famous collapse incident is that of the Hartford Civic Center Coliseum in 1978, due to the largest snowstorm in a 5-year time period. The snow loading caused excessive deflection of the space frame roof, which causes the final collapse.

Three independent investigations have been done. The space frame construction for the stadium was a double-layer grid. In the conventional design of double-layer grid structures, the centrelines of each member intersect into the same joint to reduce the bending moment. However, the investigation report (Lev Zetlin Associates, 1978) shows that in the case of the Hartford Civic Center’s frame, the top chords intersected at one point and the diagonals at another, which caused bending stresses in the members. In addition, the lateral bracing of the top chords was met through diagonals in the
interior of the frame, but along the edges there was no means to prevent out-of-plane bending (Lev Zetlin Associates, 1978).

A faulty weld connecting the scoreboard to the roof was also noticed. A massive amount of energy would have been caused by the volatile weld release, causing the entire structure to collapse (Feld and Carper, 1997).

It was also noticed that once the roof truss was in place, the construction manager altered the roof material, increasing the dead load by 20% (Feld and Carper, 1997). Therefore, the dead loads were underestimated by more than 20%.

Another investigation showed that the cause of the failure was due to torsional buckling of the compression members, and that members close to the middle of the truss were critically loaded even before live loads were added. This means of failure is usually overlooked as a cause of failure because it is so uncommon (ENR, 1978).

3.4.3 Roof Collapse of Pavilion Constructed in Bucharest
Another collapse example is a pavilion constructed in Bucharest in 1963 (Vlad and Vlad, 2014). The pavilion was a braced dome with a span of 100 m and rise of 0.48 m. The dome collapsed as a result of local snap-through due to an unexpected snow load accumulation on a small area. The local buckling propagated rapidly, and this propagation of deformation caused the dome to collapse.

3.4.4 Roof Collapse of Sultan Mizan Stadium in Terengganu, Malaysia (Support Failure)
The roof of the football stadium in Terengganu, Malaysia, was constructed as a curved double-layer grid. It collapsed 1 year after completion, in 2009. The primary cause for the collapse was incomplete consideration of the support conditions for the roof.

The report from the investigation committee explains the reason for the collapse (Investigation Committee on the Roof Collapse at Stadium Sultan Mizan Zainal Abidin, 2009):

- The design was inadequate; the designer failed to fully take into account the support conditions of the roof structure.
- The complexity and long spans of the roof structure required more detailed consideration in second-order design analysis, which was not carried out.
- The sensitivity of the space frame roof structure required consideration of the support flexibility in the design mode, which was not done.
In the construction stage, the roof was erected poorly, resulting in misaligned geometry; poor workmanship was another reason for collapse.

On February 20, 2013, the stadium collapsed again while undergoing reconstruction work. Two-thirds of the old structure (137 m) collapsed, followed by the collapse of steel pillars. The collapse was due to the removal of the middle framework.

From these two collapses in one stadium, it can be seen that the support failure was one of the major reasons for the collapse of the space structure. A modelling analysis considering the support failure is demonstrated later in this chapter.

### 3.5 Collapse Mechanism of Space Structures

As introduced in Section 3.1, there are different types of space structures. Therefore, the collapse mechanism varies depending on their structural form. In this section, the collapse mechanism for different types of space structures will be explained.

#### 3.5.1 Collapse Mechanism for Double-Layer Grid

A double-layer grid space structure is one of the conventional long-span structures. Due to its large statical indeterminacy and redundancy of structural members, in design practice, it is normally considered that progressive collapse will not be triggered when the loss of an individual member occurs. However, in the research presented by Murtha-Smith (1988), an analysis was performed on hypothetical space trusses and showed that progressive collapse could occur following the loss of just one of several potentially critical members when the structures were subject to full service loading.

The collapse of the Hartford Coliseum showed that progressive collapse could also occur following the loss of some critical members when the structures are overloaded by a gravity load, such as excessive snow loading due to severe weather.

In addition to snow, strong wind was also found to be a reason for the collapse of a building; therefore, in real design practice, a wind tunnel test is normally required for long-span space structure designs.

Based on the above discussion, in the design, an extra safety margin should be made for structural members to resist extra loading, therefore to prevent progressive collapse due to abnormal gravity loads.
3.5.2 Collapse Mechanism of Single-Layer Space Structures

For certain types of space structures, such as the single-layer lattice shell, stability is important, as buckling may initiate the collapse of the structure. Structures such as single-layer braced domes are prone to progressive collapse due to propagation of local instability initiated by member or node instability. Research from Abedi and Parke (1996) found that, for single-layer braced domes, the dynamic snap-through is associated with inertial effects and large localized deformation and can propagate to lead to collapse.

The reason is that for all the space structures introduced in this chapter, a single-layer space frame exhibits greater sensitivity to buckling than a double-layer structure. It was also found that a shallow shell, such as a dome, is more prone to overall buckling than a cylindrical shell. And global buckling may be triggered if certain critical members fail. An example is a roof collapse of the pavilion constructed in Bucharest.

To help readers fully understand the buckling behaviour of the lattice shell, it is worthwhile to introduce the major types of buckling here. In design practice, there are three major types of buckling that need to be checked: member buckling, local buckling of certain members, and global buckling.

Member buckling is when the individual member becomes unstable; it includes the overall buckling of the members and local buckling of the flanges or webs. The corresponding design formulas are given in worldwide guidances such as Eurocode 3 (European Committee for Standardization, 2005). The theories behind these design formulas are Euler’s buckling theory and the Perry–Robertson equations. A designer can check the stability of individual structural members accordingly.

Local buckling consists of a snap-through buckling of a group of structural members in a local area, which often takes place at joints. The local buckling of a space frame is prone to happen in single-layer structures such as single-layer lattice shells. The type of buckling collapse of a space frame is greatly influenced by the curvature and thickness of the structure and the manner of supporting and loading. It is apt to occur when $t/R$ is small, where $t$ is the thickness of the structure and $R$ is the radius of curvature.

Global buckling refers to a relatively large area of the space frame becoming unstable. It is often triggered by local buckling. Therefore, global buckling analysis of the whole structure should also be performed in the design to prevent progressive collapse.
3.5.3 Collapse Mechanism of Tensegrity Structures

Different than conventional space structures, tensegrity structures have a unique feature in their collapse mechanism due to their self-equilibrium system. Research from Kahla and Moussa (2002), Abedi and Shekastehband (2008), and Shekastehband et al. (2011) shows that the behaviour of members has a dominant effect on the overall collapse behaviour of space structures.

In tensegrity structures, a member may suddenly fail in tension (cables) or compression (struts). These may be due to the snap-through of struts under compression and cable ruptures under tension. Similar to single-layer domes, an initial failure of a small portion of the structure has the potential to propagate to other parts of the structure and may ultimately cause overall collapse.

In fact, member failure has a dynamic effect on the behaviours of the whole system, as it releases kinetic energy in a local region of the structure. Therefore, it is important to account for dynamic effects in the analysis, especially the redistribution of member forces and inertia forces caused by the member failure, when evaluating the response of these structures under the member failure phenomenon (Shekastehband et al., 2012; Shekastehband and Abedi, 2013).

3.5.4 Support Failure

From the incidents introduced in Section 3.2, it can be seen that support failure is one of the key reasons for the collapse of all types of space structures. Improper construction methods (such as in Mizan Stadium in Terengganu), heavy earthquakes, and foundation settlement will all cause support failure. Therefore, in the design, an engineer should be able to check the support collapse potential of the space structures with support failure. Detailed analysis using Abaqus® is shown in Section 3.6.

3.6 Progressive Collapse Analysis of Double-Layer Grid Space Structure Using Abaqus®

As mentioned in Section 3.4.2, an excessive gravity load may cause the collapse of a double-layer grid space structure. Therefore, in this section, how to perform a progressive collapse analysis of the double-layer grid space structure is demonstrated using the commercial programs Abaqus® and SAP2000 (CSI, 2013).
3.6.1 Prototype Space Structure
In research by Fu and Parke (2015), a double-layer grid space structure was modelled as the prototype. It was a conventional square grid, 27 m long on each side, and consisted of 324 square pyramids.

These kinds of systems are sometimes formed by continuous top and bottom chords, with pinned diagonal struts forming the web members. However, the normal construction of these structural types is to use individual tubular members, spanning from node to node for the top and bottom chords, and additional tubular members for the diagonal web members. All of the members are generally considered to be pinned. Therefore, in the simulation, pin connections are modelled for all the members.

The height of the grid was 1.5 m. The whole structure was vertically supported at selected perimeter nodes in the locations shown in Figure 3.4. The support-to-support span was 9 m, giving a span-to-depth ratio of 6.

3.6.2 Setting Up a 3D Model
The geometry of the space structure is sophisticated. It is more efficient to set the three-dimensional (3D) model in software such as Rhino or AutoCAD, and then import it into an analysis program such as SAP2000 or Abaqus®. In this analysis, a 3D model was set up first in SAP2000, as shown in Figure 3.4. This is because

![FIGURE 3.4 Double-layer grid model in SAP2000. (SAP2000 screenshot reprinted with permission of CSI.)](image)

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SAP2000 is a design-oriented program with design code incorporated. Therefore, the model was first analysed and designed under normal loading conditions using SAP2000 to make sure no member was overstressed and that no overall buckling and local buckling of the structural members were observed. To simplify the analysis and design, $60.3 \times 5$ CHHFs—60 mm diameter pipe sections with a 5 mm wall thickness—were chosen for all the members. The yield stress of the chosen steel was 355 N/mm².

As shown in Figure 3.5, after analysis in SAP2000, the 3D model can be imported into Abaqus® CAE. However, similar to Chapter 2, the analysis is conducted using INP files here.

In the proposed model, all of the top and bottom chord and diagonal members were modelled using *BEAM elements. The material properties of all the structural steel components were modelled using elastic–plastic material behaviour from Abaqus® incorporating material nonlinearity. The elastic part of the stress–strain curve was defined with the *ELASTIC option, and the values $2.06 \times 10^5$ N/mm² for Young’s modulus and 0.3 for Poisson’s ratio were used. The plastic part of the stress–strain curve was defined with the *PLASTIC option. Steel grade S355 was used for all the structural steel. Engineering stresses and strains, including the yield and ultimate strength, were obtained from BS 5990 (BSI, 2001) and converted into true stresses and strains with the appropriate input format for Abaqus®.

3.6.3 Load Combinations
For nonlinear dynamic analysis, the GSA guideline (2003) has the load combination requirement of dead load plus 0.25 of the live load.
However, with reference to the collapse incident of the Hartford Civic Center, to make the analysis more conservative, the full live load was used, so the load combination used in the analysis was 1.0 dead + 1.0 live (live load is taken as 1 kN/m² in the analysis).

3.6.4 Major Abaqus® Command Used in the Simulation
Similar to Chapter 2, the INP file consists of several main parts. Readers can refer to Chapter 2 for detailed examples.

3.6.5 Member Removals
The members to be removed were forcibly removed by instantaneously deleting them. Several removal scenarios were selected; they are shown in Table 3.1.

3.6.5.1 Case 1 In the first analysis, as shown in Figure 3.5, one web member, which was at the centre of the grid, was removed. However, as shown in Figure 3.6, no obvious dynamic response was observed.

<table>
<thead>
<tr>
<th>Case 1</th>
<th>Removal of a structural member at centre</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 2</td>
<td>Removal of a square pyramid at centre</td>
</tr>
<tr>
<td>Case 3</td>
<td>Centre support failure</td>
</tr>
</tbody>
</table>

**FIGURE 3.6** Response of axial force in the top chord near the removed member for case 1.
It can be seen that due to the redundancy of the structure, a single member failure does not have a significant effect on the structure.

3.6.5.2 Case 2 In case 2, all of the web members in one square pyramid, located at the centre of the structure, were removed. Figure 3.7 shows the contour plot of vertical displacement after the central pyramid removal. The response of the axial force in the top chord near the removed central pyramid is shown in Figure 3.8. A dynamic response was observed. It should be noted that the first second consists of the static step; the static load (live + dead) was applied in this analysis step, and the axial force increased from 0 to the maximum force. After the first second, the dynamic procedure started, whereupon the structural members were removed. The response of the axial force of a diagonal strut is shown in Figure 3.9.

A design check was made after the analysis. The tensile capacity of each structural member was 308 kN, the buckling load for the top and bottom chords was 215 kN, and the buckling load for the diagonal struts was 144 kN. This indicated that no further member failure occurred after the removal of the square central pyramid.

3.6.5.3 Case 3 In this analysis, in the middle of one edge, support A was removed. The removal was done by deleting several members connected to the support. This was also to simulate the support failures as they occurred in Sultan Mizan Stadium. The removed

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**FIGURE 3.7** Contour plot of vertical displacement (one central pyramid removal). (Abaqus® screenshot reprinted with permission from Dassault Systèmes.)
FIGURE 3.8 Response of axial force in the Top chord near the removed central pyramid (case 2). (Abaqus® screenshot reprinted with permission from Dassault Systèmes.)

FIGURE 3.9 Response of axial force in the diagonal strut near the removed central pyramid (case 2). (Abaqus® screenshot reprinted with permission from Dassault Systèmes.)
members and the location of support A are shown in Figure 3.10, which also shows the distribution of the vertical deflection in the structure after the removal of support.

Figure 3.11 shows the stress contour after removal of the support. It can be seen that after removal of support A, some structural members close to the adjacent support (B) become overstressed. This is because most of the loads carried by support A were redistributed into the remaining support, primarily those supports at locations B and caused the overstresses in the members close to support B.

Figure 3.12 shows the axial force in the bottom chord near support B. It can be seen that after removal of the support, the bottom
chord buckled as the axial force exceeded the buckling capacity, which is 215 kN for the relevant members.

### 3.6.5.4 Progressive Collapse Potential Check

Based on the above analysis, we can also note that due to the high redundancy of the structure—for a space grid, supporting a normal live load—removal of a single structural member is unlikely to trigger the collapse of the whole structure. However, under an abnormal live load condition, such as a very heavy snow load, a progressive collapse of the structure can be triggered, as it may cause the failure of several members, which will increase the possibility of a progressive collapse.

However, great attention also needs to be paid to support failures, because when a support fails, the load is redistributed to the adjacent supports, which are likely to cause further member failures and trigger a progressive collapse. Detailed analysis results can be found in Fu and Parke (2015).

### References


ENR. 1978. Collapsed space truss roof had a combination of flaws. ENR, June 22.


