Structural Engineering and Mechanics, Vol. 10, No. 1 (2000) 1-15
DOI: http://dx.doi.org/10.12989/sem.2000.10.1.001

Effect of direct member loading on space truss behaviour

Ahmed El-Sheikh†

Department of Civil Engineering, University of Dundee, Dundee DD1 4HN, U.K.

Abstract: It is inevitable that every space truss structure would be under some form of direct member loading. At least the structure self weight certainly affects the members directly, and in structures involving top concrete slabs or cladding, their weight is also likely to apply some lateral pressure on the members. In spite of that, direct member loading is usually ignored in space truss designs and assumed to lead only to a negligible effect on truss performance. This study is intended to explore this point and identify the actual effects that can arise from direct member loading, and eventually provide an answer to the question of whether the current design practice is satisfactory or certain modifications would be needed. After presenting two analytical techniques to allow the study of space trusses with laterally loaded members, the paper describes a wide parametric study involving practical-size space trusses with different configurations, aspect ratios, boundary conditions and number of chord panels.

Key words: space trusses; direct loading; behaviour.

1. Introduction

Since the beginning of their commercial use four decades ago, space trusses have been increasingly popular, especially in large open areas with few or no intermediate supports. Over the years, they have become known for their pleasing appearance, lightweight, easy fabrication and rapid erection. Hundreds of successful space truss applications now exist all over the world covering stadiums, public halls, exhibition centres, aeroplane hangers and many other buildings.

The design of space trusses commonly assumes that all loads are applied onto the joints, and that the structure resists these loads through the development of primarily axial forces in its members. However, there are cases where direct member loading is too significant to be ignored. Examples include hybrid systems where the space truss is combined with a cladding in the form of a concrete slab, steel panels or membrane sheets. In these instances, a large portion of the weight of cladding and covering material, and the applied live load and wind pressure would be acting directly on the members.

This paper is focused on the effect of direct member loading on space truss behaviour, and in particular to establish whether this type of loading needs to be explicitly considered in practical space truss designs. The paper starts with a discussion of two analytical techniques on how to consider direct member loading in space truss analysis. A parametric and comparative study is then presented covering a wide spectrum of practical applications with different space truss configurations,

† Lecturer

boundary conditions, aspect ratios and number of chord panels. This study is aimed at identifying the effect of direct loading on space truss behaviour, particularly the strength and flexural stiffness. The results obtained could be beneficial to future space truss designs.

2. Analysis techniques

2.1. First technique (based on the super-structuring method)

The analysis of space trusses with laterally loaded members is conventionally done using matrix methods that divide the loaded members into finite segments and consider member loads as concentrated forces acting on the additional nodes. However, due to the usual great number of members used in space trusses, the division of members leads to a considerable increase in the number of nodes, and hence the number of degrees of freedom involved.

This problem could be overcome by adopting a form of the super-structuring method (Livesley 1975). For instance, for member 1-2 shown in Fig. 1 under direct lateral loads P_A, P_B and P_C applied at the additional nodes A, B and C, the stiffness matrices for the four sub-elements are:

For sub-element 1-A:
$$\begin{Bmatrix} P_1 \\ P_A \end{Bmatrix} = \begin{bmatrix} K_{11}^{(1-A)} & K_{1A}^{(1-A)} \\ K_{1A}^{(1-A)} & K_{AA}^{(1-A)} \end{bmatrix} \cdot \begin{Bmatrix} \delta_1 \\ \delta_A \end{Bmatrix} \tag{1a}$$

For sub-element A-B:
$$\begin{Bmatrix} P_A \\ P_B \end{Bmatrix} = \begin{bmatrix} K_{AA}^{(A-B)} & K_{AB}^{(A-B)} \\ K_{AB}^{(A-B)} & K_{BB}^{(A-B)} \end{bmatrix} \cdot \begin{Bmatrix} \delta_A \\ \delta_B \end{Bmatrix} \tag{1b}$$

For sub-element B-C:
$$\begin{Bmatrix} P_B \\ P_C \end{Bmatrix} = \begin{bmatrix} K_{BB}^{(B-C)} & K_{BC}^{(B-C)} \\ K_{BC}^{(B-C)} & K_{\mathbf{C}}^{(B-C)} \end{bmatrix} \cdot \begin{Bmatrix} \delta_B \\ \delta_C \end{Bmatrix} \tag{1c}$$

For sub-element C-2:
$$\begin{Bmatrix} P_C \\ P_2 \end{Bmatrix} = \begin{bmatrix} K_{\mathbf{C}}^{(C-2)} & K_{C2}^{(C-2)} \\ K_{C2}^{(C-2)} & K_{22}^{(C-2)} \end{bmatrix} \cdot \begin{Bmatrix} \delta_C \\ \delta_2 \end{Bmatrix} \tag{1d}$$

Where P=nodal forces, δ=nodal deformations and K=stiffness matrix elements. The superscripts of K denote the sub-element to which K is related. Eqs. (1) can be re-written in terms of δ as follows:

For sub-element 1-A:
$$\delta_1 = \frac{P_1 - K_{1A}^{(1-A)} \cdot \delta_A}{K_{11}^{(1-A)}}, \text{ and } \quad \delta_A = \frac{P_A - K_{1A}^{(1-A)} \cdot \delta_1}{K_{A}^{(1-A)}} \tag{2a, b}$$

For sub-element A-B:
$$\delta_A = \frac{P_A - K_{AB}^{(A-B)} \cdot \delta_B}{K_{AA}^{(A-B)}}, \text{ and } \quad \delta_B = \frac{P_B - K_{AB}^{(A-B)} \cdot \delta_A}{K_{BB}^{(A-B)}} \tag{2c, d}$$

For sub-element B-C:
$$\delta_B = \frac{P_B - K_{BC}^{(B-C)} \cdot \delta_C}{K_{BB}^{(B-C)}}, \text{ and } \quad \delta_C = \frac{P_C - K_{BC}^{(B-C)} \cdot \delta_B}{K_{\mathbf{C}}^{(B-C)}} \tag{2e, f}$$

For sub-element C-2: $\delta_C = \dfrac{P_C - K_{C2}^{(C-2)} \cdot \delta_2}{K_C^{(C-2)}}$, and $\delta_2 = \dfrac{P_2 - K_{C2}^{(C-2)} \cdot \delta_C}{K_{22}^{(C-2)}}$ (2g, h)

By using the values of δ_A, δ_B and δ_C in Eqs. (2c), (2e) and (2g) in Eq. (2a), we obtain:

$$\delta_1 - \frac{K_{1A}^{(1-A)} \cdot K_{AB}^{(A-B)} \cdot K_{BC}^{(B-C)} \cdot K_{C2}^{(C-2)}}{K_{11}^{(1-A)} \cdot K_{AA}^{(A-B)} \cdot K_{BB}^{(B-C)} \cdot K_C^{(C-2)}} \cdot \delta_2$$
$$= \frac{1}{K_{11}^{(1-A)}} \cdot \left[P_1 - \frac{K_{1A}^{(1-A)}}{K_{AA}^{(A-B)}} \cdot \left[P_A - \frac{K_{AB}^{(A-B)}}{K_{BB}^{(B-C)}} \cdot \left[P_B - \frac{K_{BC}^{(B-C)}}{K_C^{(C-2)}} \cdot [P_C] \right]\right]\right] \quad (3)$$

or

$$P_1 - \frac{K_{1A}^{(1-A)}}{K_{AA}^{(A-B)}} \cdot \left[P_A - \frac{K_{AB}^{(A-B)}}{K_{BB}^{(B-C)}} \cdot \left[P_B - \frac{K_{BC}^{(B-C)}}{K_C^{(C-2)}} \cdot [P_C] \right]\right]$$
$$= K_{11}^{(1-A)} \cdot \left(\delta_1 - \frac{K_{1A}^{(1-A)} \cdot K_{AB}^{(A-B)} \cdot K_{BC}^{(B-C)} \cdot K_{C2}^{(C-2)}}{K_{11}^{(1-A)} \cdot K_{AA}^{(A-B)} \cdot K_{BB}^{(B-C)} \cdot K_C^{(C-2)}} \cdot \delta_2 \right) \quad (4)$$

A similar equation can also be obtained for P_2 in the form:

$$P_2 - \frac{K_{C2}^{(C-2)}}{K_C^{(B-C)}} \cdot \left[P_C - \frac{K_{BC}^{(B-C)}}{K_{BB}^{(A-B)}} \cdot \left[P_B - \frac{K_{AB}^{(A-B)}}{K_{AA}^{(1-A)}} \cdot [P_A] \right]\right]$$
$$= K_{22}^{(C-2)} \cdot \left(\delta_2 - \frac{K_{C2}^{(C-2)} \cdot K_{BC}^{(B-C)} \cdot K_{AB}^{(A-B)} \cdot K_{1A}^{(1-A)}}{K_{22}^{(C-2)} \cdot K_C^{(B-C)} \cdot K_{BB}^{(A-B)} \cdot K_{AA}^{(1-A)}} \cdot \delta_1 \right) \quad (5)$$

Eqs. (4) and (5) can be re-written as: $P_1' = k_{11} \cdot \delta_1 + k_{12} \cdot \delta_2$, and $P_2' = k_{21} \cdot \delta_1 + k_{22} \cdot \delta_2$, or simply:

$$\begin{Bmatrix} P_1' \\ P_2' \end{Bmatrix} = \begin{bmatrix} k_{11} & k_{12} \\ k_{21} & k_{22} \end{bmatrix} \cdot \begin{Bmatrix} \delta_1 \\ \delta_2 \end{Bmatrix} \quad (6)$$

Notice that the stiffness matrix in Eq. (6) is symmetric since k_{12} is equal to k_{21}. Eq. (6) can now be used to replace the simple equation (typical for members without direct load):

$$\begin{Bmatrix} P_1 \\ P_2 \end{Bmatrix} = \begin{bmatrix} k_{11}^{(1-2)} & k_{12}^{(1-2)} \\ k_{21}^{(1-2)} & k_{22}^{(1-2)} \end{bmatrix} \cdot \begin{Bmatrix} \delta_1 \\ \delta_2 \end{Bmatrix} \quad (7)$$

in the formation of the structure's overall stiffness matrix. The assembly of Eq. (6) can be made simpler by dividing the member into equal parts as this would lead to identical stiffness matrix elements for the individual parts and equal values for P_A, P_B and P_C for the case with a uniformly distributed member load.

2.2. Second technique (based on building effect of direct loads in stiffness matrix formation)

As an alternative to the above technique, the lateral loads could be incorporated directly in the development of the stiffness matrix for the whole element. A solution along this line and incorporating sinusoidally distributed lateral loads has been developed earlier by Taniguchi *et al.* (1995). A similar solution concerned with the case depicted in Fig. 1 could be developed with relative ease. (Notice that the accuracy of this technique could be improved by increasing the number of member divisions.) In this case, the force deformation relationships are as follows (refer to Fig. 2):

$$M_{z1}=\frac{EI_z}{L}(C_{1z}\theta_{z1}+C_{2z}\theta_{z2}) \tag{8}$$

$$M_{z2}=\frac{EI_z}{L}(C_{2z}\theta_{z1}+C_{1z}\theta_{z2}) \tag{9}$$

$$M_{y1}=\frac{EI_y}{L}(C_{1y}\theta_{y1}+C_{2y}\theta_{y2})+r_y \tag{10}$$

$$M_{y2}=\frac{EI_y}{L}(C_{2y}\theta_{y1}+C_{1y}\theta_{y2})-r_y \tag{11}$$

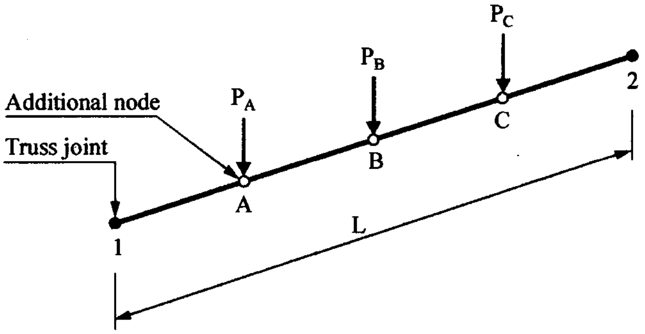


Fig. 1 Member under direct loads, P_A, P_B and P_C

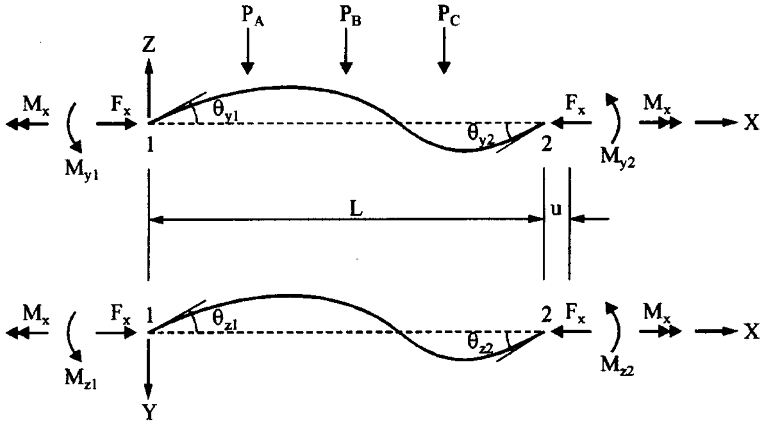


Fig. 2 Member end forces and moments

$$M_x = \left(G + \frac{F_x}{A}\right)\frac{J}{L} \cdot \varphi \qquad \text{(according to Young 1989)} \tag{12}$$

$$F_x = \frac{EA}{L}(u - \mu_z - \mu_y) \tag{13}$$

where:

M_{z1}, M_{z2}, M_{y1}, M_{y2}	= End flexural moments
θ_{z1}, θ_{z2}, θ_{y1}, θ_{y2}	= End flexural rotations
I_z, I_y	= Second moments of area of member cross section
C_{1z}, C_{2z}, C_{1y}, C_{2y}	= Stability functions defined earlier by Livesley (1975)
r_y	= Contribution of lateral loads to M_{y1} and M_{y2}
	= $(3\,P_A+4\,P_B+3\,P_C)\,L/16$ (assuming equal distances $L^{(1\text{-}A)}$, $L^{(A\text{-}B)}$, $L^{(B\text{-}C)}$ and $L^{(C\text{-}2)}$)
M_x, φ	= Torsional moment and twist angle
F_x	= Axial force
u	= Change in member length as determined by the nodal displacements
μ_z, μ_y	= Change in member length due to bowing

The values of parameters μ_z and μ_y were obtained earlier by Saafan (1963) as:

$$\mu_z = b_{z1}\,(\theta_{z1}+\theta_{z2})^2 + b_{z2}\,(\theta_{z1}-\theta_{z2})^2 \tag{14}$$

$$\mu_y = b_{y1}\,(\theta_{y1}+\theta_{y2})^2 + b_{y2}\,(\theta_{y1}-\theta_{y2})^2 + b_{y3} \tag{15}$$

where b_{z1}, b_{z2}, b_{y1} and b_{y2} are the bowing functions first introduced by Saafan (1963), and b_{y3} is the bowing function resulting from the lateral loads. As function b_{y3} is the direct contribution of the lateral load to member bowing, it can be obtained from analysing the member as (assuming $L^{(1\text{-}A)} = L^{(A\text{-}B)} = L^{(B\text{-}C)} = L^{(C\text{-}2)} = L/4$):

$$\begin{aligned} b_{y3} = {} & \frac{L}{4}\left[1 + \left(\frac{(9P_A + 11P_b + 7P_c)L^2}{192EI_y}\right)^2\right]^{1/2} + \frac{L}{4}\left[1 + \left(\frac{(2P_A + 5P_b + 4P_c)L^2}{192EI_y}\right)^2\right]^{1/2} \\ & + \frac{L}{4}\left[1 + \left(\frac{(4P_A + 5P_b + 2P_c)L^2}{192EI_y}\right)^2\right]^{1/2} + \frac{L}{4}\left[1 + \left(\frac{(7P_A + 11P_b + 9P_c)L^2}{192EI_y}\right)^2\right]^{1/2} - L \end{aligned} \tag{16}$$

Further, if the internal loads P_A, P_B and P_C, are equal, Eq. (16) reduces to:

$$b_{y3} = \frac{L}{2}\left[1 + \left(\frac{9\bar{P}L^2}{64EI_y}\right)^2\right]^{1/2} + \frac{L}{2}\left[1 + \left(\frac{11\bar{P}L^2}{192EI_y}\right)^2\right]^{1/2} - L \tag{17}$$

where $\bar{P} = P_A = P_B = P_C$.

The moment-rotation, M-θ, torsional moment-twist angle, M_x-φ, and axial force-shortening, F_x-u, relationships (Eqs. 8 to 13) have been derived in forms similar to those used by Oran (1973), the procedures of whom can now be followed to form the tangent stiffness matrix for the laterally loaded element under consideration. The equilibrium equations relating the incremental member end forces, Δs, to the corresponding incremental end deformations, Δu, can now be written as:

$$\Delta s = t\,.\,\Delta u \tag{18}$$

where Δs^T=(ΔM_{z1}, ΔM_{z2}, ΔM_{y1}, ΔM_{y2}, ΔM_x, $\Delta F_x L$); Δu^T=($\Delta\theta_{z1}$, $\Delta\theta_{z2}$, $\Delta\theta_{y1}$, $\Delta\theta_{y2}$, $\Delta\varphi$, Δu); t = the current stiffness matrix. From a consideration of geometry and equilibrium, the member end forces in local co-ordinates, ΔP (vector 12 × 1) can be derived from Δs as:

$$\Delta P = B \,.\, \Delta s, \text{ or} \tag{19}$$

$$\begin{Bmatrix} \Delta F_{x1} \\ \Delta F_{y1} \\ \Delta F_{z1} \\ \Delta M_{x1} \\ \Delta M_{y1} \\ \Delta M_{z1} \\ \Delta F_{x2} \\ \Delta F_{y2} \\ \Delta F_{z2} \\ \Delta M_{x2} \\ \Delta M_{y2} \\ \Delta M_{z2} \end{Bmatrix} = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & \frac{1}{L} \\ \frac{1}{L} & \frac{1}{L} & 0 & 0 & 0 & 0 \\ 0 & 0 & -\frac{1}{L} & -\frac{1}{L} & 0 & 0 \\ 0 & 0 & 0 & 0 & -1 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & -\frac{1}{L} \\ -\frac{1}{L} & -\frac{1}{L} & 0 & 0 & 0 & 0 \\ 0 & 0 & \frac{1}{L} & \frac{1}{L} & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \end{bmatrix} \cdot \begin{Bmatrix} \Delta M_{z1} \\ \Delta M_{z2} \\ \Delta M_{y1} \\ \Delta M_{y2} \\ \Delta M_x \\ \Delta F_x L \end{Bmatrix} \tag{20}$$

where B=the instantaneous local equilibrium matrix, see Oran (1973). A similar equation is used to relate member end deformations in local co-ordinates $\Delta\delta$ (vector 12×1) to Δu as:

$$\Delta u = B^T \,.\, \Delta\delta \tag{21}$$

where $\Delta\delta^T$=(Δu_1, Δv_1, Δw_1, $\Delta\theta_{x1}$, $\Delta\theta_{y1}$, $\Delta\theta_{z1}$, Δu_2, Δv_2, Δw_2, $\Delta\theta_{x2}$, $\Delta\theta_{y2}$, $\Delta\theta_{z2}$). Using Eqs. (19) and (21), the member tangent stiffness matrix (12 × 12) relating ΔP to $\Delta\delta$ can then be derived.

This latter technique has clear advantages in reducing computer memory and run time requirements. For these reasons, it has been adopted in the present study using a non-linear finite element computer program (NONTRS) developed and verified by the author in an earlier work (El-Sheikh and McConnel 1989). The output of the program when using the new element with direct lateral loads was compared to corresponding output by ABAQUS (a commercial finite element package) before commencing the parametric study described below. The comparison involved three square-on-square space trusses with the overall sizes 50 × 50 × 3.125 m, 75 × 50 × 3.125 m and 100 × 50 × 3.125 m as shown in Fig. 3. The trusses were corner supported and loaded fully on the top members. In ABAQUS, the top members were loaded with either:

- 3 intermediate point loads each, similar to the technique described in this paper, or
- a uniformly distributed load.

The results from these analyses are plotted against the output of NONTRS in Fig. 4. The comparisons show clearly that the two programs assuming member point loads produced very similar results

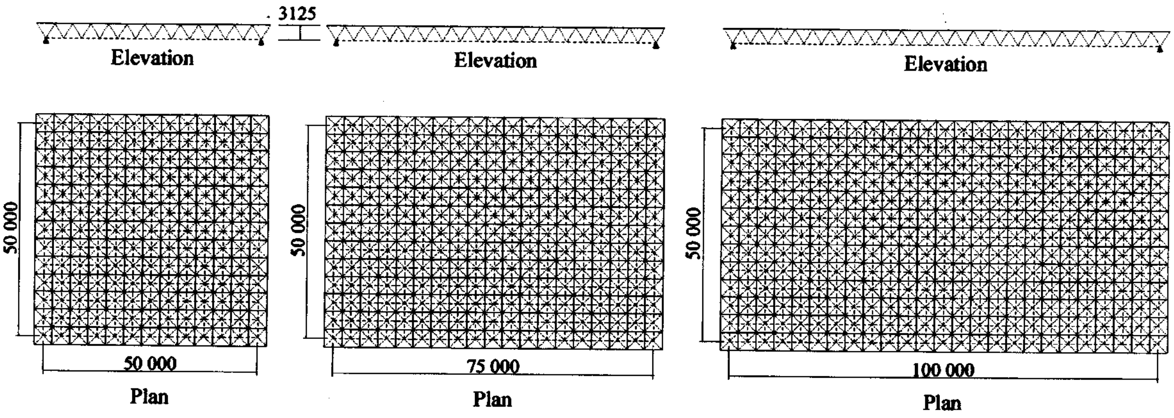


Fig. 3 Space trusses involved in verifying finite element program NONTRS

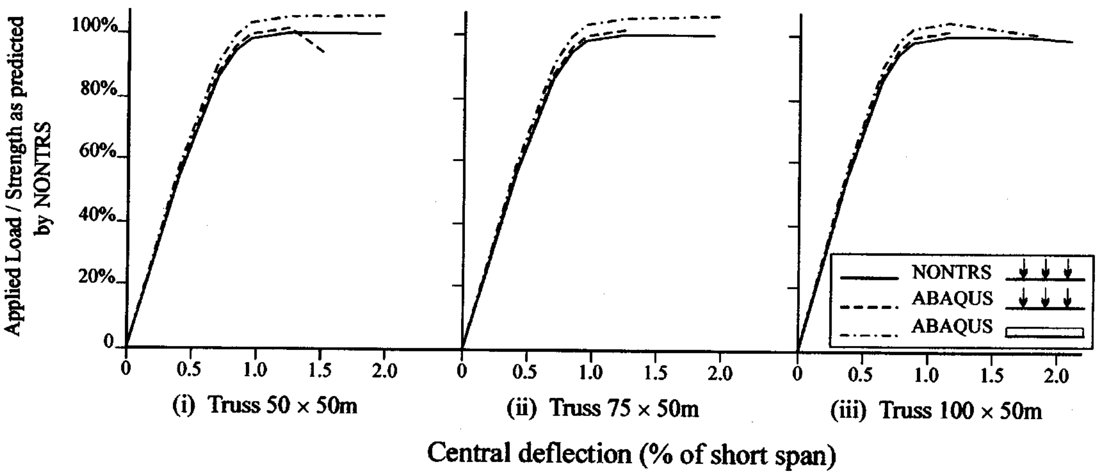


Fig. 4 Assessment of accuracy of NONTRS output

with less than 1% difference in initial stiffness and 2.5% difference in strength. With a UDL on the members, the maximum differences in stiffness and strength between ABAQUS and NONTRS increased to 1.5% and 4%, respectively. These results indicated the ability of the presented second technique in accurately modelling the behaviour of laterally loaded members. They also justified the use of the NONTRS program in the following parametric study on the effect of member lateral loading on the overall behaviour of space trusses.

3. Parametric study

In order to establish the effect of direct member loading on the behaviour of space trusses, a parametric study was conducted involving 36 double-layer trusses covering wide variations of the following important parameters:

(a) Aspect ratio; 1:1, 1.5:1 or 2:1;

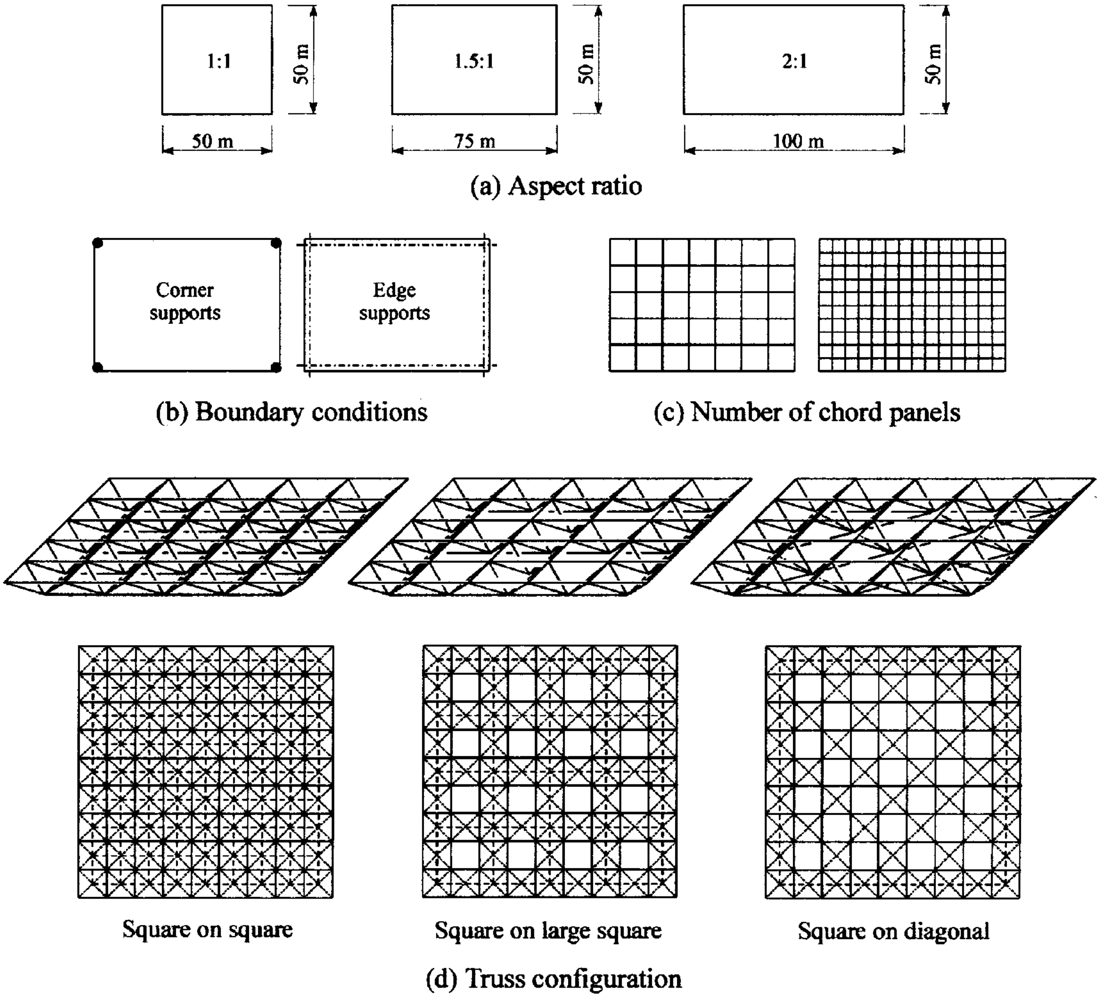


Fig. 5 Parameters considered in parametric study

(b) Boundary conditions; with corner or edge supports;
(c) Number of chord panels; 12 or 20 in the short direction.

Also, three truss configurations; namely the square-on-square (SOS), the square-on-large-square (SOLS) and the square-on-diagonal (SOD), were considered in this work for being among the most commonly used in practice, see Fig. 5. The space trusses had overall sizes of 50 m × 50 m, 75 m × 50 m and 100 m × 50 m. The depth in all cases was 3.125 m, giving a short-span/depth ratio of 16.

The trusses were designed to simulate the conditions of large-span roof structures with a total factored load of 5.0 kN/m^2. Each truss was designed using simple linear finite element analyses to determine the internal forces in all truss members while applying the external loads wholly on the top joints. According to these forces, truss members were sized assuming steel grade S355J2H throughout. Following the design, each truss was analysed five times with the top joints and top members sharing the total load to the ratios: 100%-0%, 75%-25%, 50%-50%, 25%-75% and 0%-

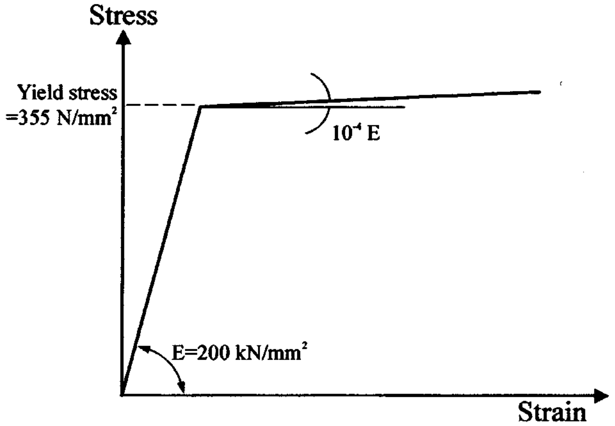


Fig. 6 Material properties considered in numerical analyses

100%. The twelve SOS space trusses included were further analysed with the loads being shared by the bottom joints and bottom members to the same ratios.

All analyses were non-linear and considered both geometric non-linearities (due to buckling, yielding and change of joint coordinates) and material non-linearities (due to yielding and crushing). The material properties of members were as specified in Eurocode 3 (1993) for steel S355J, see Fig. 6. The analyses continued up to and beyond the ultimate load carrying capacity of the trusses. In all cases, the behaviour was traced using Riks method (Riks 1979) that allowed the analysis to continue even after the structures had started to suffer reductions in their load carrying capacity.

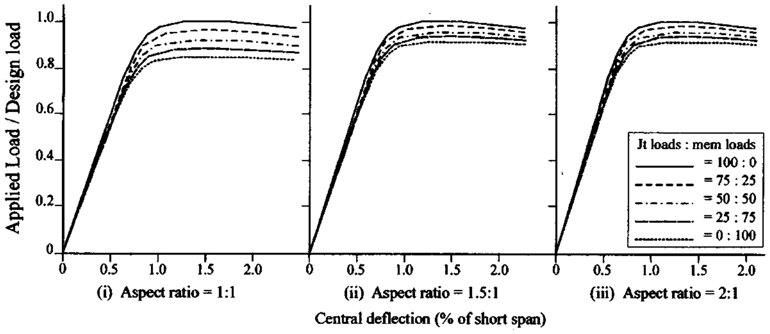
4. Results of the non-linear analyses

In this section, the results of the non-linear numerical analyses carried out on the 36 space trusses considered, are presented. Direct comparisons between the strength and flexural stiffness of trusses with different loading conditions are held and used to extract conclusions on the effect of direct member loading on truss performance.

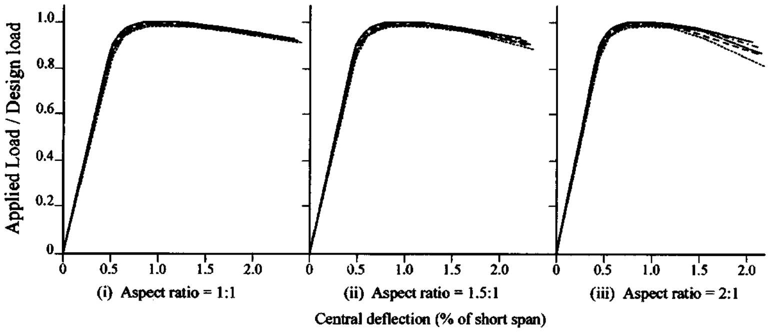
Fig. 7 shows the behaviour of the twelve square on square (SOS) space trusses designed, under an increasing uniformly distributed loading to failure. As explained above, each truss was studied under five loading conditions involving different distributions of the total load between the top joints and the top chord members. The strength and flexural stiffness losses associated with applying portions of the load directly on the top members are also listed in Table 1.

The results presented demonstrate a significant effect on truss performance caused by direct member loading in all cases with corner supports. As the load was shifted gradually from the top joints to the top members, the strength and flexural stiffness deteriorated by up to 14.9% and 8.0%, respectively. In contrast, edge-supported space trusses were much less sensitive to direct member loading with the maximum effects on strength and stiffness being 1.9% and 3.0%, respectively. Note also that in all cases considered, changes in truss aspect ratio led to only minor variations in truss sensitivity to direct member loading.

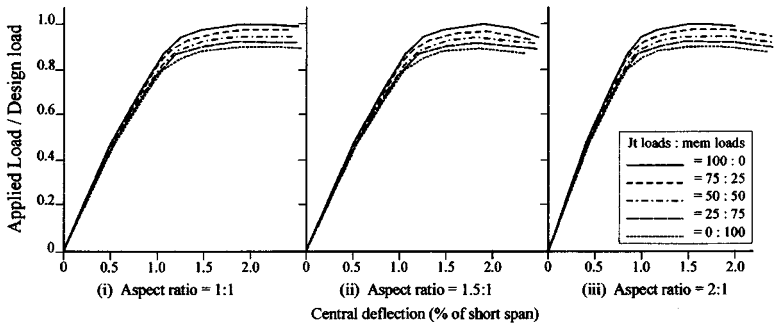
The behaviour of SOS trusses was repeated by corresponding SOLS and SOD trusses as shown in Figs. 8 and 9 and Tables 2 and 3. In this case, the maximum losses in strength and flexural stiffness



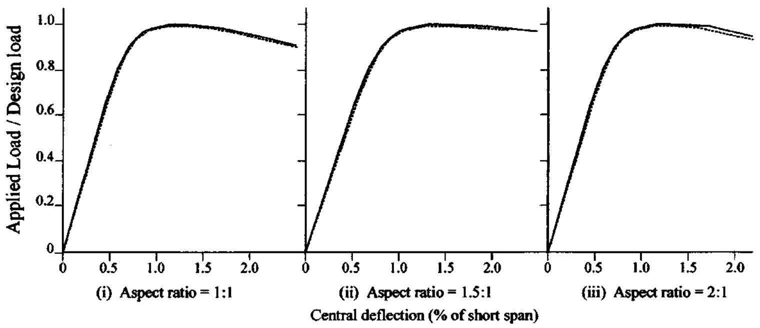
(a) Corner-supportred trusses with 12 panels in the short direction



(b) Edge-supportred trusses with 12 panels in the short direction



(c) Corner-supportred trusses with 20 panels in the short direction



(d) Edge-supportred trusses with 20 panels in the short direction

Fig. 7 Behaviour of square-on-square space trusses

Table 1 Effect of member direct loading on performance of square-on-square space trusses

Aspect ratio	No of panels	Supports	Strength losses				Stiffness losses			
			Member load share =				Member load share =			
			25%	50%	75%	100%	25%	50%	75%	100%
1:1	12	Corner	03.8%	08.0%	11.6%	14.8%	01.9%	03.9%	06.0%	07.9%
		Edge	00.5%	00.8%	01.3%	01.7%	00.6%	01.3%	01.9%	02.9%
	20	Corner	02.8%	05.6%	07.7%	10.0%	01.5%	02.9%	04.7%	06.1%
		Edge	00.3%	00.4%	00.5%	00.6%	00.2%	00.5%	00.7%	01.2%
1.5:1	12	Corner	04.0%	08.1%	11.4%	14.5%	01.8%	03.6%	05.6%	07.8%
		Edge	00.6%	00.9%	01.4%	01.9%	00.7%	01.5%	02.2%	03.0%
	20	Corner	02.7%	05.4%	07.7%	10.2%	01.6%	03.2%	04.9%	06.4%
		Edge	00.2%	00.3%	00.5%	00.7%	00.0%	00.2%	00.5%	00.8%
2:1	12	Corner	03.7%	07.8%	11.4%	14.9%	02.1%	04.0%	06.1%	08.0%
		Edge	00.6%	00.9%	01.3%	01.6%	00.7%	01.4%	02.0%	03.0%
	20	Corner	02.8%	05.7%	07.8%	09.8%	01.7%	03.3%	05.1%	06.6%
		Edge	00.2%	00.4%	00.6%	00.7%	00.1%	00.2%	00.4%	00.7%

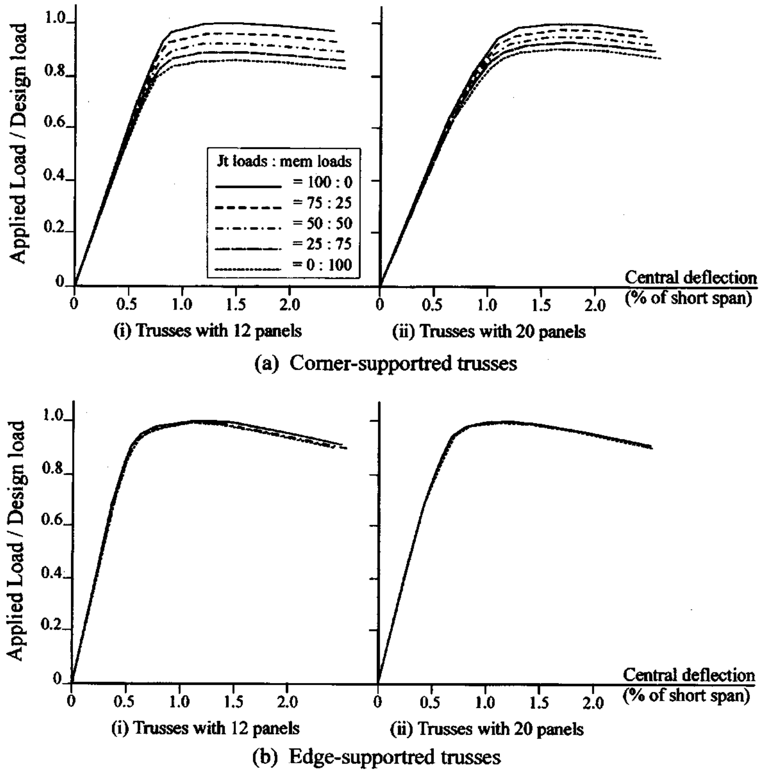


Fig. 8 Behaviour of square-on-large-square space trusses

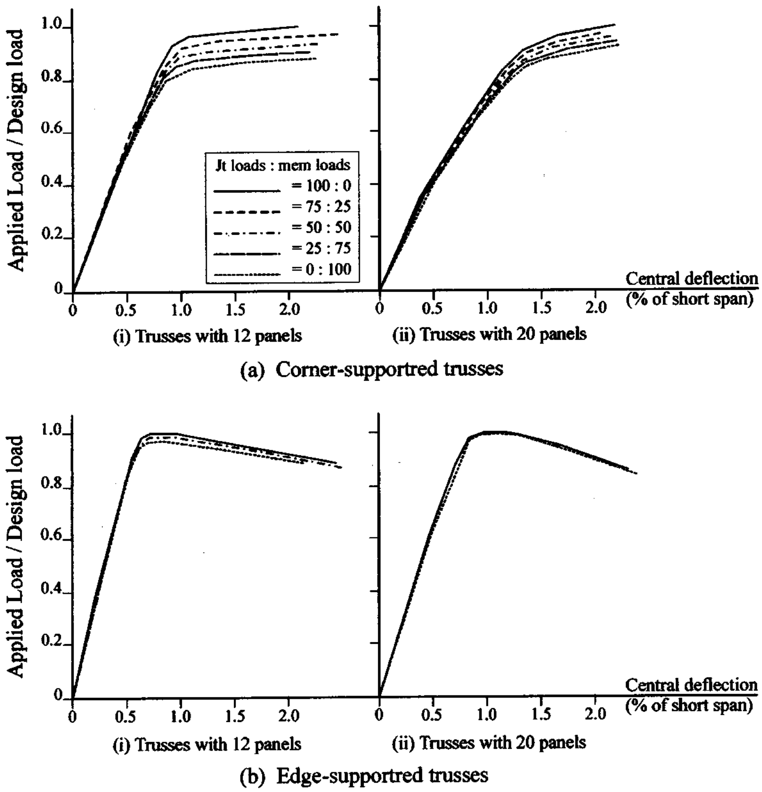


Fig. 9 Behaviour of square-on-diagonal space trusses

Table 2 Effect of member direct loading on performance of square-on-large-square space trusses

Aspect ratio	No of panels	Supports	Strength losses: Member load share = 25%	50%	75%	100%	Stiffness losses: Member load share = 25%	50%	75%	100%
1:1	12	Corner	03.7%	07.5%	10.9%	14.0%	01.8%	03.5%	05.1%	06.7%
		Edge	00.3%	00.6%	00.8%	01.0%	00.7%	01.4%	02.1%	02.4%
	20	Corner	02.6%	05.1%	07.4%	09.7%	01.6%	02.9%	04.4%	05.6%
		Edge	00.1%	00.3%	00.4%	00.6%	00.2%	00.5%	00.7%	01.0%
1.5:1	12	Corner	03.5%	07.2%	10.5%	13.4%	02.0%	03.9%	04.4%	06.3%
		Edge	00.2%	00.5%	00.7%	00.9%	00.5%	01.1%	02.1%	02.6%
	20	Corner	02.5%	05.2%	07.6%	09.9%	01.7%	03.0%	04.5%	05.8%
		Edge	00.0%	00.1%	00.2%	00.3%	00.0%	00.2%	00.6%	00.9%
2:1	12	Corner	03.6%	07.5%	11.1%	14.1%	02.1%	03.8%	04.8%	06.5%
		Edge	00.3%	00.5%	00.8%	01.1%	00.5%	01.5%	01.9%	02.5%
	20	Corner	02.4%	05.2%	07.4%	09.6%	01.2%	02.6%	04.2%	05.5%
		Edge	00.1%	00.2%	00.5%	00.8%	00.1%	00.3%	00.6%	01.2%

Table 3 Effect of member direct loading on performance of square-on-diagonal space trusses

Aspect ratio	No of panels	Supports	Strength losses				Stiffness losses			
			Member load share =				Member load share =			
			25%	50%	75%	100%	25%	50%	75%	100%
1:1	12	Corner	03.1%	06.4%	09.4%	12.3%	02.5%	04.9%	07.1%	09.2%
		Edge	00.8%	00.9%	01.9%	02.9%	00.4%	01.1%	01.8%	02.2%
	20	Corner	02.3%	04.4%	06.5%	08.4%	01.9%	03.5%	05.3%	06.9%
		Edge	00.1%	00.1%	00.2%	00.3%	00.0%	00.2%	00.5%	00.7%
1.5:1	12	Corner	02.9%	05.9%	09.2%	11.8%	02.2%	04.6%	06.8%	08.9%
		Edge	00.9%	01.2%	01.8%	02.6%	00.5%	01.3%	01.9%	02.3%
	20	Corner	02.3%	04.1%	05.9%	07.8%	01.7%	03.4%	05.2%	06.2%
		Edge	00.0%	00.0%	00.1%	00.2%	00.0%	00.0%	00.3%	00.6%
2:1	12	Corner	03.2%	06.1%	08.9%	12.4%	02.3%	05.0%	07.2%	09.4%
		Edge	00.7%	01.0%	01.9%	02.7%	00.3%	01.1%	01.7%	02.3%
	20	Corner	02.4%	04.6%	06.6%	08.3%	01.6%	03.6%	05.6%	07.3%
		Edge	00.2%	00.2%	00.3%	00.4%	00.1%	00.3%	00.6%	00.9%

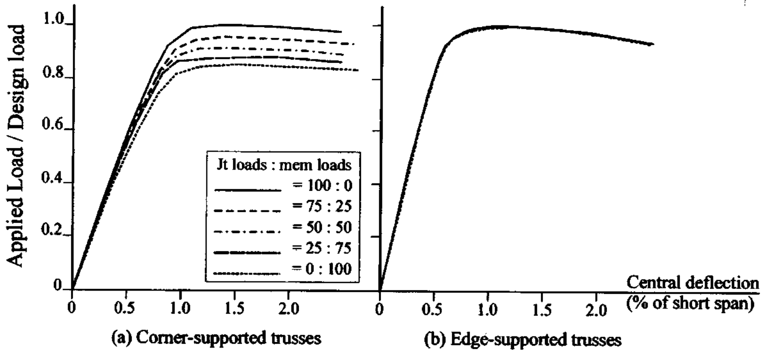


Fig. 10 Behaviour of 1:1 square-on-square space trusses with 12 panels in each direction under bottom chord loading

were 14.1% and 6.7% in corner-supported SOLS trusses, 1.1% and 2.6% in edge-supported SOLS trusses, 12.4% and 9.4% in corner-supported SOD trusses, and 2.9% and 2.3% in SOD trusses with edge supports. Notice that due to the similar behaviour of trusses with different aspect ratios (as can be seen in Fig. 7 and Tables 1 to 3), the results presented in Figs. 8 and 9 were limited to cases with 1:1 aspect ratio.

Finally, when the SOS space trusses considered in this work were analysed under direct loading on the bottom chord members, similar behaviour to that exhibited in Fig. 7 was recorded. A selection of the results obtained is presented in Fig. 10.

5. Discussion of results

The results presented in this paper reveal a number of important common trends. These are discussed in detail in this section, with the main conclusions briefly presented in the next section.

- Direct member loading, which causes member bowing and hence a reduction in member buckling-strength and axial stiffness, leads in most cases to adverse effects on overall truss strength and flexural stiffness.
- Corner-supported space trusses are in general more sensitive to direct member loading than trusses on edge supports. This can be attributed to the many more load paths available with edge supports, a feature which has been found earlier to make space trusses tolerant to potentially damaging effects such as local failures and member imperfections (El-Sheikh 1994, 1995).
- Space trusses with different configurations responded similarly to direct loading on top chord members. This was caused by the facts that the top chord remained unchanged in all configurations considered, and all trusses were designed with the same degree of dependence on the top chord integrity.
- The strength and stiffness losses associated with direct member loading were consistently reduced with more chord panels (and hence shorter members). It appears that as the members became shorter, the effect of direct loading in increasing the members bowing and affecting their buckling strength reduced, leading to less overall effects on truss performance.
- The response of space trusses to bottom member loading is similar to that found under top member loading. In both cases, direct loading developed flexural moments in the loaded chord members, and this is believed to be responsible for the lower strength exhibited by both the loaded members and the truss.

6. Conclusions

Two techniques are presented in this paper to study the behaviour of structures (including space trusses) with some direct member loading. While the first technique is based on the super-structuring method, the second involves modifying the element tangent stiffness matrix by including internally the effect of direct lateral loading. The second technique, while being more complicated, is thought to be the better of the two for its ability to reduce computer run time and memory requirements. It was therefore adopted in the present study to assess the effect of direct member loading on space truss performance. The study involved a parametric comparative analysis of 36 space trusses with different configurations, boundary conditions, aspect ratios and number of chord panels. From the results obtained, the following conclusions could be drawn:

(1) The effect of direct member loading on corner-supported space trusses ought to be considered at the design stage especially if the direct member loads constitute a considerable share of the total truss load.

(2) In contrast, space trusses on edge supports are much less sensitive to direct member loading, and therefore their design could be safely simplified by considering all loads to be applied at the joints.

(3) Space trusses with the SOS, SOLS and SOD configurations, and with different aspect ratios, respond similarly to direct member loading.

(4) On the other hand, with more chord panels, the effect of direct loading in increasing the member bowing and reducing their axial strength becomes less significant, leading to smaller reductions in truss overall strength and stiffness.

References

El-Sheikh, A.I. and McConnel, R.E. (1989), "A Finite Element Approach", Technical Report CUED/D Struct/TR128, Dept of Engineering, Cambridge Univ.

El-Sheikh, A.I. (1994), "Sensitivity of composite and non-composite space trusses to member loss", *Int. J. Space Structures*, **9**(2), 107-119.

El-Sheikh, A.I. (1995), "Sensitivity of space trusses to member geometric imperfections", *Int. J. Space Structures*, **10**(2), 89-98.

European Committee for Standardisation, CEN/TC 250 Committee (1993), Eurocode 3: Design of steel structures.

Livesley, R.K. (1975), *Matrix Methods of Structural Analysis*, Pergamon Press, Oxford.

Oran, C. (1973), "Tangent stiffness in space frames", *J. Struct. Div., ASCE*, **99**(6), 987-1001.

Riks, E. (1979), "An incremental approach to the solution of snapping and buckling problems", *Int. J. Solids and Structures*, **15**, 524-551.

Saafan, S.A. (1963), "Nonlinear behaviour of structural plane frames", *J. Struct. Div., ASCE*, **89**(4), 557-579.

Taniguchi, Y., Saka, T. and Tanaka, H. (1995), "Nonlinear behaviour of space frame structures with laterally loaded members", *Int. Symposium on Spatial Structures*, Italy, 661-668.

Young, W.C. (1989), *Roark's Formulas for Stress and Strain*, McGraw-Hill Book Company, London.