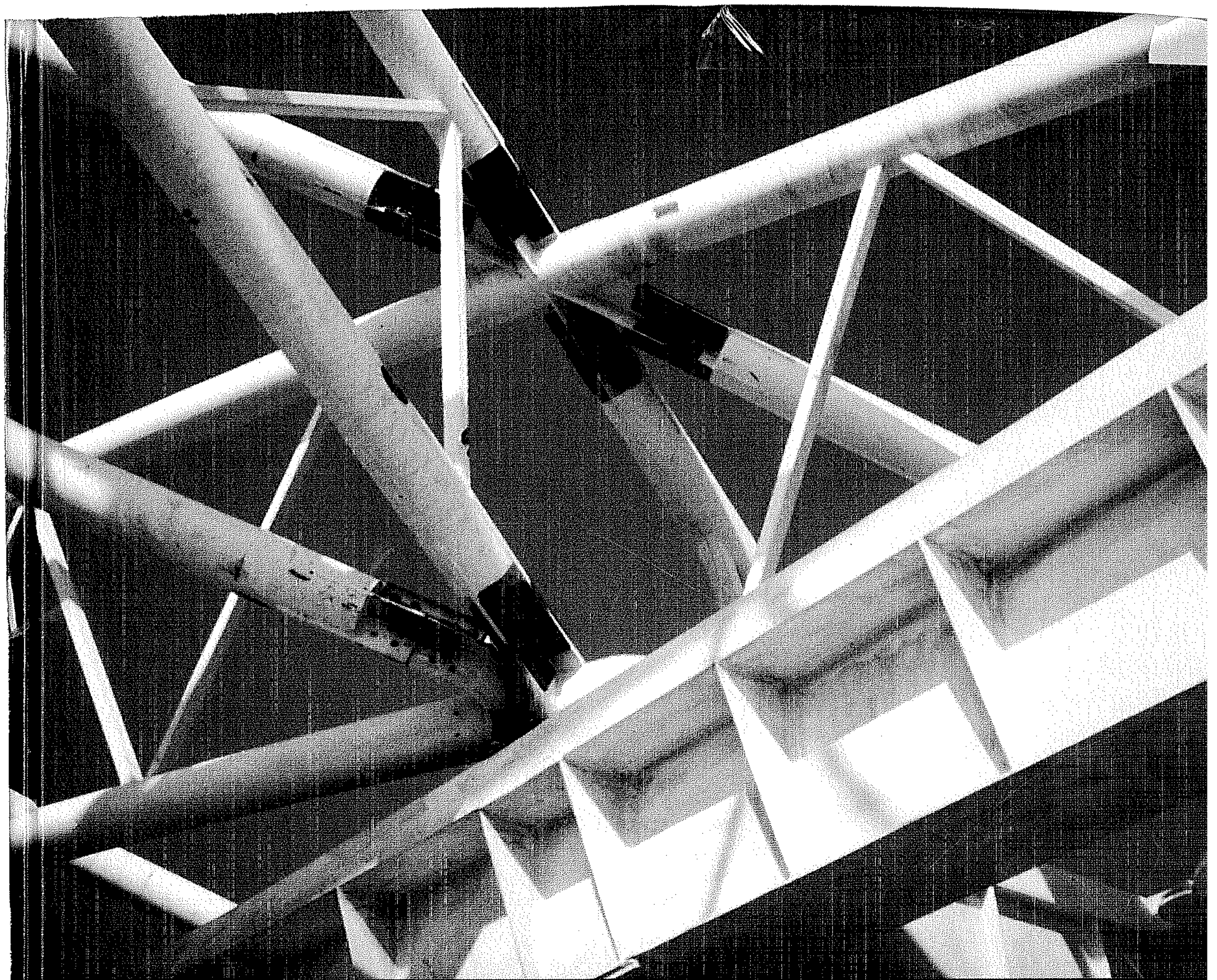

Tubular Structures VII

József Farkas and Károly Jármai, editors

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Cover photo: Tubular roof structure of a new sports hall in Budapest

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Minimum cost design of Vierendeel square hollow section trusses

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ABSTRACT: Members of a Vierendeel girder are predominantly loaded in bending. The unknown dimensions can be calculated using constraints on stress, chord side wall failure, local buckling and effective width based on CIDECT design rules. When the truss height is fixed, then the deflection constraint can also be active for the design of verticals. The optimal number of bays can be determined by calculating the cost of various possible structural versions. In the costs the material and fabrication costs are considered. The optimization procedure is illustrated by a numerical example.

KEYWORDS: Vierendeel trusses, structural hollow sections, welded girders, optimum design, minimum cost design, fabrication costs.

1 INTRODUCTION

Vierendeel trusses are used because of their aesthetic view and simple fabrication. On the contrary to triangulated trusses with diagonals, Vierendeel truss members should be designed for bending. The calculations show that the axial and shear forces can be neglected. In the CIDECT Design Guide (Packer et al. 1992) a numerical example has been worked out. In the book of Martin and Purkiss (1992) an illustrative numerical example is also treated.

The aim of the present paper is to apply the optimum design for Vierendeel trusses welded from square hollow section (SHS) members. In the cost function the material and fabrication costs are taken into account as in the authors' previous studies (e.g. Farkas & Jármai 1995). Design constraints relate to stresses due to bending, chord side wall failure, effective width, local buckling, deflection and prescription of the minimal height of a truss with parallel chords. The profiles of chords and verticals have the same outside dimensions, so the nodes can be regarded as fully rigid.

The thickness of all chords is t_0 and that of all verticals is t_1, so there are only three unknown profile dimensions. In addition the truss height is determined by service conditions (e.g. a closed footbridge should have a minimal height of 2.8 m). The only unknown to be optimized is the number of bays (spacings between verticals) in the case of a given span length of a simply supported girder. Thus, in an illustrative numerical example the optimal number of bays is sought to minimize the cost.

2 THE COST FUNCTION

The total cost contains the material and fabrication costs

$$K = K_m + K_f = k_m \rho V + k_f \sum_i T_i \tag{1}$$

where k_m (\$/kg) and k_f (\$/min) are the material and fabrication cost factors, respectively, ρ is the material density,

$$V = 2LA_0 + (\varphi + 1)HA_1 \tag{2}$$

where A_0 and A_1 are the cross-section areas of the chords and verticals, respectively, $\varphi = L/a_0$ is the number of bays, L is the span length, H is the truss height. T_i are the fabrication times as follows:

Time for preparation, assembly and tacking

$$T_1 = C_1 \Theta \sqrt{\kappa \rho V}$$

$$C_1 = 1.0 \text{ min/kg}^{0.5} \tag{3}$$

where Θ is a difficulty factor, for Vierendeel trusses it can be taken as 3, κ is the number of assembled structural elements, here $\kappa = \varphi + 3$.

Time for welding

$$T_2 = \sum_i C_{2i} a_{wi}^n L_{wi} \tag{4}$$

where L_{wi} are the weld lengths, here for all welds

$$L_w = 8b(\varphi + 1) \tag{5}$$

$C_2 a_w^n$ are calculated according to the COSTCOMP software worked out by the Welding Institute of Netherlands (Bodt 1990). $a_w = t_1$ (see Fig.1) and $C_2 a_w^n = 0.6 * 10^{-3} t_1^2$ for shielded metal arc welding (SMAW) of 1/2V butt welds and $0.8 * 10^{-3} t_1^2$ for SMAW of fillet welds (for t_1 = 4 - 15 mm), so we calculate with the mean value of

$$C_2 a_w^n = 0.7 * 10^{-3} t_1^2 \tag{6}$$

Time for electrode changing, deslagging and chipping

$$T_3 = \sum_i C_3 i a_{wi}^n L_{wi}.$$

According to Ott and Hubka (1985) C_3 = (0.2-0.4) C_2 so we calculate with C_3 = 0.3 C_2 and

$$T_2 + T_3 = 1.3 \sum_i C_{2i} a_{wi}^n L_{wi} \tag{7}$$

The final form of the cost function to be minimized is as follows

$$\frac{K}{k_m} = \rho V + \frac{k_f}{k_m}\left[C_1 \Theta \sqrt{\kappa \rho V} + 1.3 * 0.7 * 10^{-3} t_1^2 L_w\right] \tag{8}$$

t_1 and L_w in mm, V in mm^3, $\rho = 7.85 * 10^{-6}$ kg/mm^3 (steel). For a wide range of cost factors we select the values of k_m = 0.5 - 1.2 \$/kg (steels) and k_f = 0 - 45 \$/manhour = 0 - 0.75 \$/min, thus k_f/k_m = 0 - 1.5.

3 DESIGN CONSTRAINTS

Constraint on chord side wall failure. According to Packer et al (1992)

$$M_{max} \le M^* = 0.5 f_y t_0 (b_0 + 5t_0)^2 \tag{9}$$

Instead of the yield stress f_y we calculate with $f_{y1} = f_y / \gamma_{M1}$ where, according to the Eurocode 3 (EC3) (1992) $\gamma_{M1} = 1.1$. Furthermore, we use a reduction factor 0.9 for the effect of roundings of SHS profiles, we calculate with the mean width $b = b_0 - t_0$ and introduce the notation $\delta_S = b / t_0$. Thus, Eq.(9) can be written in the form

$$M^* = 0.45 f_{y1} b^3 (\delta_S + 4)^2 / \delta_S^3 \tag{10}$$

According to Packer et al (1992), to calculate with rigid joints, we take the limiting local plate slenderness of the chord connecting face

$$\delta_{SoL} = (b / t_0)_L = 16 \tag{11}$$

Thus $M_{max} \le M^* = 0.043945 f_{y1} b^3$ (12)

Constraint on maximal elastic stress in verticals due to bending

$$M_{max} / W_x \le f_{y1} \tag{13}$$

where the elastic section modulus of a SHS profile, considering also a reducing factor of 0.9 for roundings and a limiting plate slenderness for verticals δ_{S1L}

$$W_x = 0.9 * 4(b - t_1)^2 t_1 / 3 = 1.2 b^3 (\delta_{S1L} - 1)^2 / \delta_{S1L}^3 \tag{14}$$

The local buckling constraint for verticals is given by CIDECT rules (Packer et al 1992)

$$\delta_{S1L} = (b / t_1)_L = 1.1 \sqrt{E / f_y} \tag{15}$$

For $E = 2.1*10^5$ and $f_y = 355$ MPa it is $\delta_{SIL} = 26$, thus, Eq.(13) can be written as

$$M_{max} \leq 0.042672 f_{y1} b^3 \tag{16}$$

It can be seen that Eq.(16) gives larger b-values than Eq.(12), thus, we calculate b from Eq.(16), then t_1 using Eq.(15) and t_0 with Eq.(11).Furthermore, the effective width ratio should be 1

$$\frac{b_e}{b_1} = \frac{10t_0}{(b/t_0)t_1} \geq 1 \tag{17}$$

or $$t_0/t_1 \geq b/(10t_0) \tag{18}$$

4 AN ILLUSTRATIVE NUMERICAL EXAMPLE

The factored load of $F = 50$ kN is uniformly distributed along the upper nodes (Fig.1), L=30 m, steel Fe 510, f_y = 355 MPa, f_{y1} = 355/1.1 = 323 MPa. It can be seen from Fig.1. that the maximum bending moment acts in the verticals second from the outside post. The maximum bending moments and the selected profiles are given in Table 1.

The deflection constraint is defined by

$$w_{max} \leq w^* \tag{19}$$

where w^* is the allowable deflection, according to EC3 $w^* = L/300$ and w_{max} should be calculated without multiplying the loads by load factors. The maximum deflection can be calculated by using the well-known principle of virtual work.
For a structure subject to bending

$$w_{max} = \sum_i \int \frac{Mmds}{EI_i} \tag{20}$$

For a member with moment diagrams shown in Fig.1. it is

$$\int_0^{a_0} Mmds = M_0 m_0 a_0 / 3 \tag{21}$$

The results of calculations are summarized in Table 2.

$$w_{max} = \frac{F_0 L^3}{C_7 E I_0}\left(C_8 + C_9 \frac{I_0 H}{I_1 L}\right) \tag{22}$$

where $F_0 = F/\gamma$ is the load without safety factor $\gamma = 1.3$. Constants C_7, C_8 and C_9 are given in

Table 1. Maximum bending moments and selected profiles for different numbers of bays

φ	8	10	12	14
M_{max}	$FL/42.67$	$FL/50.00$	$FL/57.60$	$FL/65.33$
b (mm) Eq.(16)	137	130	124	119
$t_1 = b/26$ (mm)	5.3	5.0	4.8	4.6
$t_0 = b/16$ (mm)	8.5	8.1	7.7	7.4
verticals b_1*t_1	140*6.3	125*5	120*5	110*5
chords b_0*t_0	140*10	125*8.8	120*8	110*8
modified verticals	140*6.3	140*6.3	120*5	120*5
chords	140*10	140*10	120*8	120*8
A_1 (mm^2)	3230	3230	2210	2210
A_0 (mm^2)	4770	4770	3360	3360

Table 2. I_0 and I_1 are the moments of inertia of chord and vertical profiles.

Size limitation for the truss height H. It can be seen from Eq.(22) that the maximum deflection depends on the height so that a decreasing of H decreases the w_{max}. If a minimum height is prescribed, then the limited deflection can be realized only by changing I_1 and I_0. In our numerical example the height limitation is

$$H \geq H_{min} = 2.8 \text{ m} \tag{23}$$

We calculate the deflections with the moments of inertia of profiles determined using constraints of Eqs (16), (15),(11) and (18) and then we modify the profile of verticals (I_1) to fulfill the deflection constraint Eq.(19). The modified sections are given in Table 1. The maximum deflections calculated with the first and modified sections are summarized in Table 2. The results of cost calculations are shown in Table 3. It can be seen that the optimum number of bays is 12.

5 CONCLUSIONS

The common width of chords and verticals can be calculated from the constraint on maximal elastic stress due to bending in the verticals second from outside posts. The thickness of verticals is obtained from the constraint on local buckling and on effective width.

The thickness of chords is calculated from the rule $b/t = 16$ so that the nodes can be treated as fully rigid ones. The final thickness of verticals is determined from constraints on deflection and on minimal truss height.

The optimal number of bays can be determined on the basis of cost calculations.

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Table 2. Maximum deflections for various numbers of bays calculated with Eq.(22) with the data of H=2.8 m, L=30 m, $E = 2.1*10^5$ MPa and $F_0 = F/1.3 = 38462$ N

φ	8	10	12	14
C_7	$3*32^2*16$	$3*40^2*20$	$3*48^2*24$	$3*56^2*28$
C_8	4	5	6	7
C_9	55	89	131	181
I_0*10^{-4} (mm^4)	1268	775	677	505
I_1*10^{-4} (mm^4)	941	544	478	361
w_{max} (mm)	87	112	103	114
modified I_0*10^{-4} and	1268	1268	677	677
I_1*10^{-4}	941	941	478	478
modified w_{max}	87	66	103	86

Table 3. Volumes and costs for different numbers of bays

φ	8	10	12	14
$V*10^{-6}$ (mm^3) Eq.(2)	338	322	282	297
L_w (mm) Eq.(5)	10080	11000	12480	13200
K/k_m (kg) Eq.(8) for				
k_f/k_m=1.5	4233	4516	3460	3654

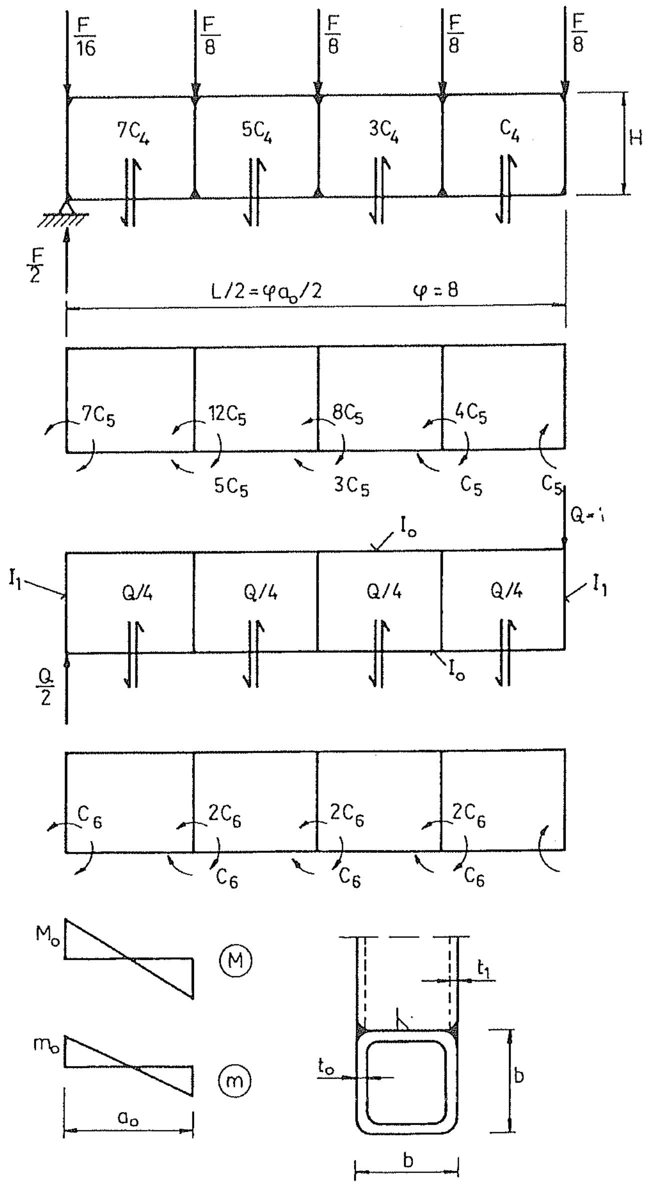

Fig. 1. Half Vierendeel truss in the case of 8 bays. Shear forces and bending moments in lower nodes due to the external vertical load F and due to the virtual load $Q = 1$ for the calculation of maximum deflection. $C_4 = F/32$, $C_5 = FL/(32*16)$, $C_6 = QL/(4*16)$.

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