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Comparison of rectangular and square box columns composed from cellular plates with welded and rolled stiffeners

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ABSTRACT

A cantilever column is loaded by a compression force and a bending moment caused by a horizontal force. It can be derived that, in the case of uniaxial bending, the rectangular cross section is more economic than the square one. In the given numerical case, the plate thicknesses are too large for enabling fabrication. Therefore stiffened plates should be used. Thus, the aim of the present study is to elaborate the minimum cost design of a column with rectangular cross-section and cellular plate walls. Cellular plates are constructed from two plates and longitudinal stiffeners welded between them. Previous studies have shown that welded T-stiffeners are more economic than the halved rolled I-section stiffeners, thus, welded T-stiffeners are used.

Stress and horizontal deformation constraints are formulated. In the stress constraint, face plate buckling is taken into account by using effective widths. Local buckling constraint is used for the web of T-stiffeners.

Variables are as follows: heights of welded T-sections, thicknesses of stiffener webs and flanges, number of stiffeners in both directions, main dimensions of the rectangular box section, thicknesses of outer and inner face plates in smaller and larger walls.

The cost function is formulated according to the fabrication sequence and consists of cost of material, welding and painting. The constrained function minimization is performed by using an effective mathematical optimization method.

1. Introduction

Steel columns are widely used for buildings, bridges, as supports of highways etc. The optimum design of such columns has been treated, which constructed from various structural types, such as circular cylindrical unstiffened and stiffened shells and square box sections with walls from stiffened and cellular plates [1]. Bending caused by horizontal force plays an important role in seismic design. A detailed literature survey concerning the cellular plates can be found in [1].

Steinhardt [2] has proposed a design method for box beams with stiffened flange plates using formulae for effective plate width. Nakai et al. [3] have worked out empirical formulae for stiffened box stub-columns subject to combined actions of compression and bending.

Ge et al. [4] and Usami et al. [5] have studied the cyclic behaviour and ductility of stiffened steel box columns used as bridge piers. Longitudinal flat plate stiffeners and diaphragms as well as constant compressive axial force and cyclic lateral loading have been considered. Empirical formulae have been proposed for ultimate strength and ductility capacity.

Other papers about bridge piers can be found in conference proceedings as follows: Yamao et al. [6], Ohga et al. [7] and Hirota et al. [8].

In our previous studies it has been shown that, in the case of uniaxial compression, cellular plates are more economic than a longitudinally stiffened ones (Farkas and Jármai [9]). In a study we have elaborated a minimum cost design of a cellular plate subject to uniaxial compression (Farkas and Jármai [10]).This method is used in the present paper for a square box column constructed from four equal cellular plates.

A column is loaded by a compression force N_F and a bending moment caused by a horizontal force $H_F = 0.1N_F$ shown in Fig. 1. Firstly, the unstiffened rectangular cross section is optimized. It will be derived that, in the case of uniaxial bending, the rectangular cross section is more economic than the square one.

It will be shown that, in the given numerical case, the plate thicknesses are too large for fabrication. Therefore stiffened plates should be used.

Results obtained for square box columns have shown that the cellular plate elements are more economic than the plates stiffened on one side [1].

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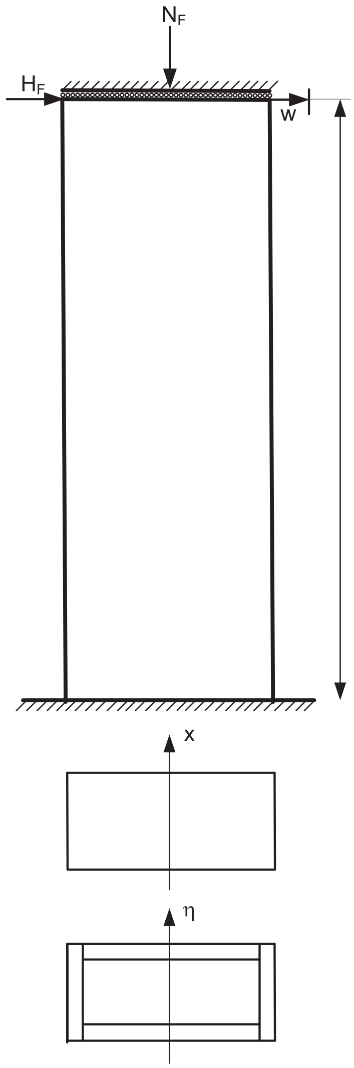

Fig. 1. Box column with walls of unstiffened and cellular plates, the two ends are built-in.

The stiffeners can be made of halved rolled I-sections (UB profiles are used) or by welded T-sections. Advantages of welded T-sections are that their dimensions (mainly the web thickness) can be freely varied. The economy of welded T-stiffeners depends on local buckling strength caused by the stress state (compression or bending).

Thus, the aim of the present study is to elaborate the minimum cost design of a column with rectangular and square cross-sections and cellular plate walls. We have considered the welded structure with initial imperfections according to the standards [11,12]. Dented structures have not been considered [13]. That is another problem.

2. Numerical data

The factored compression force is $N_F = 10^8$ [N], the height of the column is $a_0 = 15$ m, the steel yield stress is $f_y = 355$ MPa, the Young-modulus is $E = 2.1 \times 10^5$ MPa.

3. Minimum cross-sectional area design of a rectangular unstiffened box section

The cross-sectional area is expressed as

$$A = ht_w + 2bt_f \tag{1}$$

h is the height of the web, b is the width of the flange, t_w and t_f the thicknesses of the box section.

Local buckling of plate elements can be avoid by using the constraints on plate slendernesses, where β and δ are the limit slenderness values for the web and the flange.

$$\frac{h}{t_w/2} \leqslant \beta, \quad \frac{b}{t_f} \leqslant \delta \tag{2}$$

According to Eurocode 3 [12]

$$1/\delta = 42\varepsilon, \quad \varepsilon = \sqrt{235/f_y}, \quad 1/\delta = 34 \tag{3}$$

The value of β depends on the stress distribution (Fig. 2). The stress constraint is formulated as

$$\sigma = \frac{N_F}{A} + \frac{H_F a_0}{2W_x} = \sigma_1 + \sigma_2 \leqslant f_y \tag{4}$$

Taking the constraints on limiting plate slenderness as active from Eq. (2), since the largest slendernesses give the smallest objective function of the column, the moment of inertia is as follows:

$$I_x = \frac{h^3 t_w}{12} + 2bt_f\left(\frac{h}{2}\right)^2 = \frac{\beta h^4}{6} + \frac{\delta b^2 h}{2} \tag{5}$$

The section modulus is the following:

$$W_x = \frac{I_x}{h/2} = \frac{\beta h^3}{3} + \delta b^2 h \tag{6}$$

$$\text{For} \quad \psi = \frac{\sigma_1 - \sigma_2}{\sigma_1 + \sigma_2} \geqslant -1 \tag{7}$$

$$\frac{1}{\beta} = \frac{42\varepsilon}{0.67 + 0.33\psi} \tag{8}$$

Eqs. (7) and (8) give the limiting plate slenderness β in the case of different edge stresses according to Eurocode 3 [12] Table 5.2. σ_1 σ_2 are defined in Eq. (4).

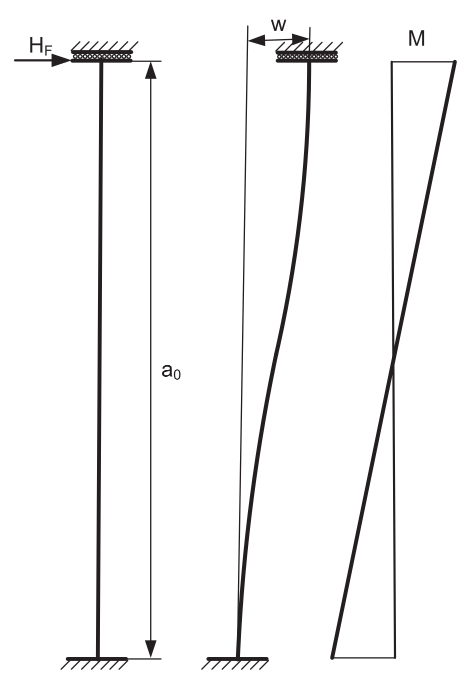

Fig. 2. Deformation and bending moment distribution of the column caused by the horizontal force.