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# Optimum seismic design of a multi-storey steel frame

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**Abstract**

An interior three-storey frame structure with a column and 4 beams in each floor is investigated. The vertical and horizontal (seismic) forces, normal forces and bending moments as well as elastic interstorey drifts are calculated. The welded box columns and rolled I-section beams are designed for minimum weight and cost. The beam-to-column connections are selected from a number of structural versions improved for seismic resistance. The fabrication costs are calculated in detail. Design constraints relate to interstorey drifts and to the stability of column parts and beams loaded by compression and bending. Calculations show that, after a connection type is selected, the fabrication cost has little effect on the optimum design, since it varies proportionally with the mass according to the present calculating method. Thus, the minimum weight design gives suitable results.


*Keywords:* Steel frames; Optimization; Minimum cost design; Earthquake-resistant design; Beam-to-column connections; Fabrication cost calculation; Stability

## 1. Problem formulation

In order to study the effect of seismic loads, a relatively simple frame is selected as shown in Fig. 1. This is a simplified model of a central part of a three-storey building frame structure. The frame is unbraced and horizontal displacements can occur due to horizontal seismic forces. The column parts are constructed from welded square box sections and the beams have a rolled universal beam (UB) profile. The frame is subject to vertical permanent and live loads as well as to horizontal seismic forces (Figs. 1, 2 and 4). In the fishbone model the beam ends are considered to be built up for vertical loads and pinned for horizontal ones. The problem is to find suitable column and beam profiles, which fulfil the design constraints and minimize the objective function. The beams and column parts are subject to bending and compression, thus, stress constraints should be formulated for 3 beam and 3 column profiles according to Eurocode 3 (2002) (EC3) [1]. The seismic forces and interstorey drifts are calculated according to Eurocode 8 (1998, 2003) (EC8) [2]. Constraints on interstorey drifts are also formulated. The calculation of drifts and stability are based on the linear elastic behaviour of the structure. This is the most popular method of analysis recommended in Eurocode 8. However, the structure and the parts of it have to meet the ductility requirements inherent to the behaviour factor adopted. One of the important requirements is the overstrength requirements for beam-to-column connections: the plastic strength of the connections should be large enough to allow formation of plastic hinges at the beam ends. The connection design was determined on the basis of the plastic analysis and experimental evidence.

## 2. Calculation of vertical loads

We use slightly modified data of Design (1995) [3] in which the seismic-resistant design of a 5-storey residential building frame is detailed.

Permanent load for roof including the structure self weight is $q_1 = 5.5\ \mathrm{kN/m^2}$.

Permanent load for floors is $q_2 = q_3 = 5.0\ \mathrm{kN/m^2}$.

Live load for roof and floors is $2.0\ \mathrm{kN/m^2}$.

According to EC8 the combination of seismic action with other actions should be performed in the following way:

$$\sum G_k + \gamma_I A_E + \sum \psi Q_k, \qquad (1)$$

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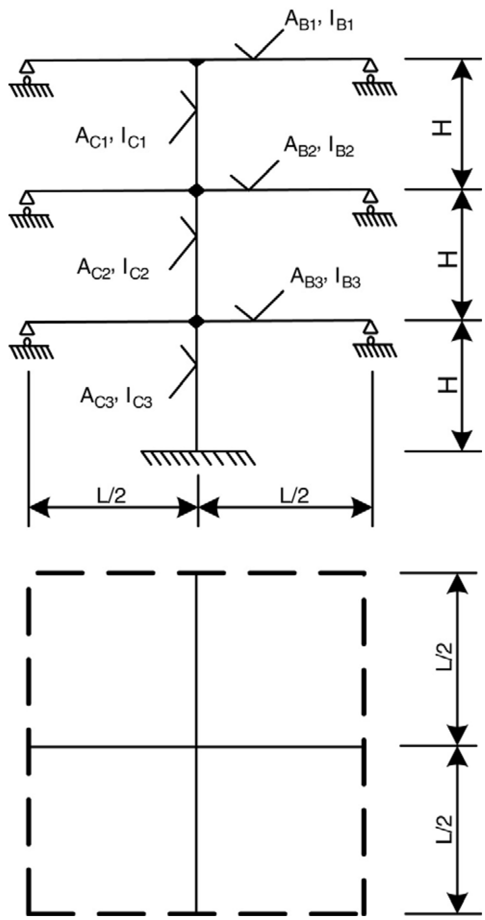


Fig. 1. The investigated frame under horizontal loads consisting of a column and 4 beams in each storey. The frame is a central part of a building as is seen in the top view.

where $G_k$ are the permanent actions, $A_E$ is the earthquake action, $Q_k$ are the variable (live) actions and $\psi = \varphi\psi_{21}$ is the combination coefficient, where for each storey $\psi_{21} = 0.3$, for top storey (roof) $\varphi = 1$ and for other storeys $\varphi = 0.5$. For the combination of vertical and seismic load actions the EC8 rule is used, in which the importance factor for ordinary buildings not belonging to the other categories (EC8 Table 4.3) is $\gamma_I = 1$ (Class II).

Combined vertical loads for beams (we consider that beams are in two directions)

Roof: $p_1 = (q_1 + 0.3 \times 2.0)L/2,$ (2)

Other storeys $p_2 = p_3 = (q_2 + 0.15 \times 2.0)L/2.$ (3)

Combined vertical loads for column parts:

Top: $G_1 = (q_1 + 0.3 \times 2.0) \times L^2$ (4)

Other storeys $G_2 = G_3 = (q_2 + 0.3 \times 2.0) \times L^2.$ (5)

These vertical loads and the corresponding $M$ (bending moment) and $N$ (compression force) diagrams are given as follows:

$$N_1 = G_1, \quad N_2 = G_1 + G_2, \quad N_3 = G_1 + G_2 + G_3, \tag{6}$$

$$M_1 = p_1 L^2/12, \quad M_2 = M_3 = p_2 L^2/12. \tag{7}$$

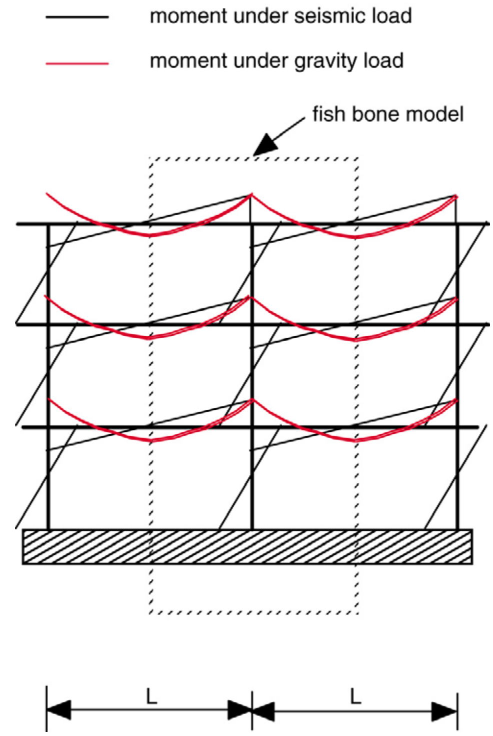


Fig. 2. Vertical and seismic loads acting on the frame and the diagrams of bending moments.

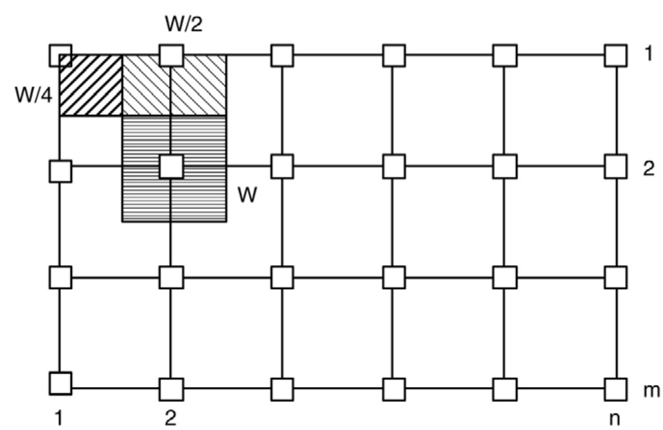


Fig. 3. Plan view of a building with *m* by *n* columns.

## 3. Calculation of horizontal seismic forces

According to EC8 the seismic base shear force is

$$F_b = S_d(T_1)\, m\lambda, \tag{8}$$

where $m$ is the total mass of the building, $\lambda$ is the correction factor, which is equal to 0.85 since $T_1 < 2T_C$ (see below)

$$T_1 = C_t H_0^{0.75}, \tag{9}$$

and the height of the building is $H_0 = 3H$. If $H = 3.6$ m then $H_0 = 10.8$ m.

For moment resistant space steel frame $C_t = 0.085$, thus

$$T_1 = 0.085 \times H_0^{0.75}.$$

If $H_0 = 10.8$ then $T_1 = 0.5064$ s.