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Optimum seismic design of a 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. A welded box column 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 details. Design constraints relate to interstorey drifts and to stability of column parts and beams loaded by compression and bending. Calculations show that, in this case, the fabrication cost has a little effect on the optimum design, since it forms only a little part of the whole cost. Thus, the minimum weight design gives suitable results.

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

Article Outline

1. Problem formulation

In order to study the effect of seismic loads a relatively simple frame is selected as shown in Figure 1. This is 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 section 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). 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.

2. Calculation of vertical loads

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

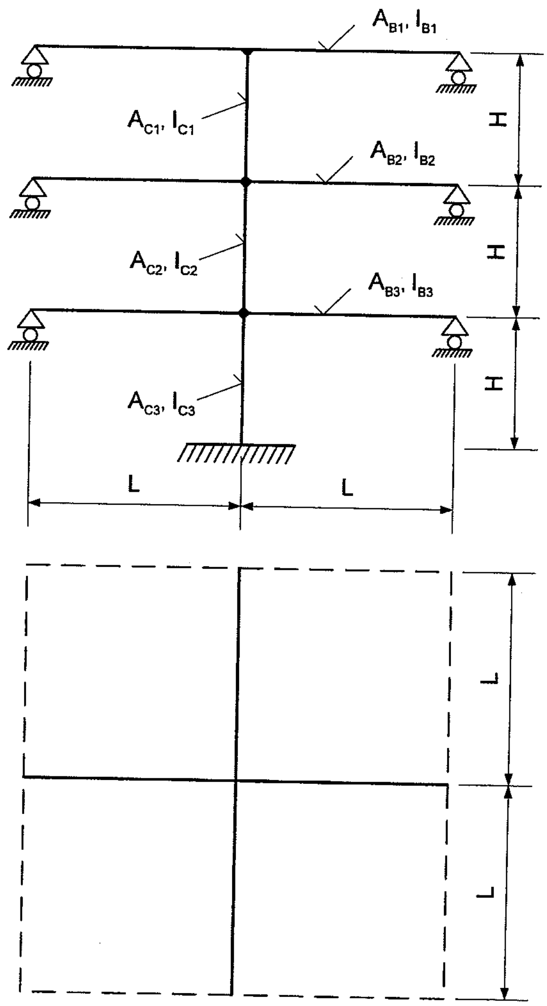

Figure 1. The investigated frame consisting of a column and 4 beams in each storey. The frame is a central part of a building as it is seen in the top view

Permanent load for roof including the structure self weight is $q_1 = 6.00\ \mathrm{kN/m^2}$

Permanent load for floors is $q_2 = q_3 = 8.00\ \mathrm{kN/m^2}$

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:

$$\sum G_k + \gamma_I A_E + \sum \psi Q_k \tag{1}$$

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: $p_1 = (q_1 + 0.3\text{x}2.0)L/2$ (2)

Other storeys $p_2 = p_3 = (q_2 + 0.15\text{x}2.0)L/2$ (3)

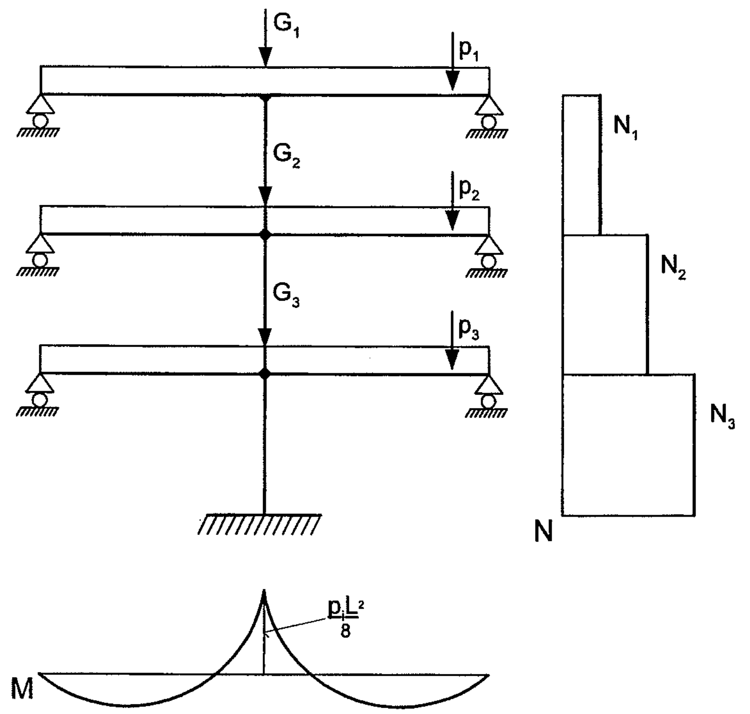

Figure 2. Vertical loads acting on the frame and the diagrams of bending moments (M) and axial forces (N)

Combined vertical loads for column parts:

Top: $G_1 = (q_1 + 0.3\text{x}2.0)\text{x}L^2$ (4)

Other storeys $G_2 = G_3 = (q_2 + 0.3\text{x}2.0)\text{x}L^2$ (5)

These vertical loads and the corresponding M (bending moment) and N (compression force) diagrams are given in Figure 2.

$N_1 = G_1,\ N_2 = G_1 + G_2,\ N_3 = G_1 + G_2 + G_3,$ (6)

$M_1 = p_1L^2/8,\ M_2 = M_3 = p_2L^2/8.$ (7)

3. Calculation of horizontal seismic forces

According to EC8 the seismic base shear force is

$$F_b = S_d(T_1)W \tag{8}$$

where W is the total weight of the building component,

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

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 is

$C_t = 0.085$, thus

$T_1 = 0.085\text{x}H_0^{0.75}$ If H_0=10.8 then T_1=0.5064 s.

for this time the following formula is valid

$$S_d = \alpha S \beta_0 / q \tag{10}$$

For subsoil class C (Table 3.1 of EC8)

S = 1.15, $\beta_0 = 2.5$, $T_B = 0.15$, $T_C = 0.60$, $T_D = 3$.

For the most dangerous Japanese zones $\alpha = 0.40$.

The behaviour factor q (EC8 Table 6.2, Figure 6.1) for moment resistant, unbraced multistorey buildings is

$q = 1.3\text{x}5 = 6.5$

Thus, $S_d = 1.15\text{x}0.4\text{x}2.5/6.5 = 0.1769$.

Horizontal shear force for the top floor is $F_1 = S_dG_1$

For the 2nd floor it is $F_2 = S_dN_2$

and for first floor $F_3 = S_d N_3$.

It should be noted that the columns in a building are interior or exterior ones. A simple calculation shows how much horizontal load a column has to support. Even in a big building of m bays by n bays, each column has to support the load of about 25-100 percent of that of total column force (Fig. 3, Table 1).

A plan view of a building with m by n columns. All the spans have equal length.

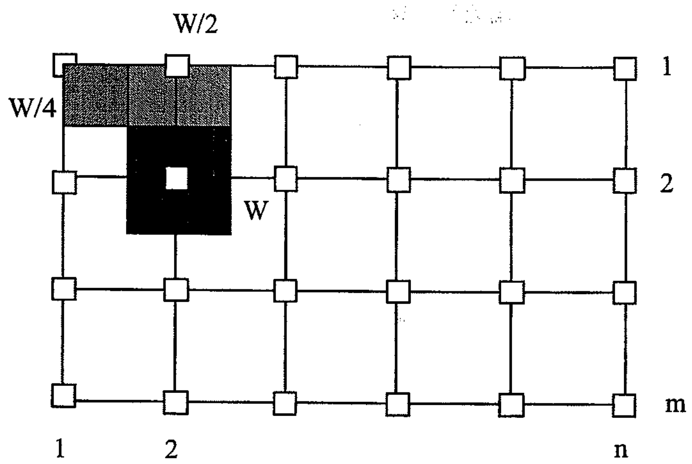

Figure 3. Plan view of a building with m by n columns

The number of columns:

Fully loaded columns = $(m\text{-}2)(n\text{-}2)$

1/2 loaded columns = $2(m\text{-}2)+2(n\text{-}2)$

1/4 loaded columns = 4

The total weight the columns are carrying, ΣW: $\Sigma W = W(m\text{-}2)(n\text{-}2) + W(m\text{-}2) + W(n\text{-}2) + W$

The average weight each column is carrying, w:

$$w = \frac{mn - m - n + 1}{mn} W \tag{11}$$

Horizontal shear forces F_i should be multiplied by w. So F_i=wF_i (i =1,2,3).

Table 1. Average weight each column is carrying for horizontal shear force calculation

m	*n*	*w*	*m*	*N*	*w*	*m*	*n*	*w*
2	2	0.25	3	4	0.50	4	7	0.64
2	3	0.33	3	5	0.53	5	5	0.64
2	4	0.38	3	6	0.56	5	6	0.67
2	5	0.40	3	7	0.57	5	7	0.69
2	6	0.42	4	4	0.56	6	6	0.69
2	7	0.43	4	5	0.60	6	7	0.71
3	3	0.44	4	6	0.63	7	7	0.73

The horizontal seismic shear forces are acting on the floors as it is shown in Figure 4. Since the structure is statically indeterminate, in order to determine the inner forces due to these horizontal forces, an approximate method can be used. In the [3] the method of Ifrim (1984) [4] is used based on the localization of inflection points (Fig. 4). For the top floor $\alpha_1 = 0.65$, for the middle floors $\alpha_2 = 0.5$ and for bottom part of the column $\alpha_3 = 0.4$.

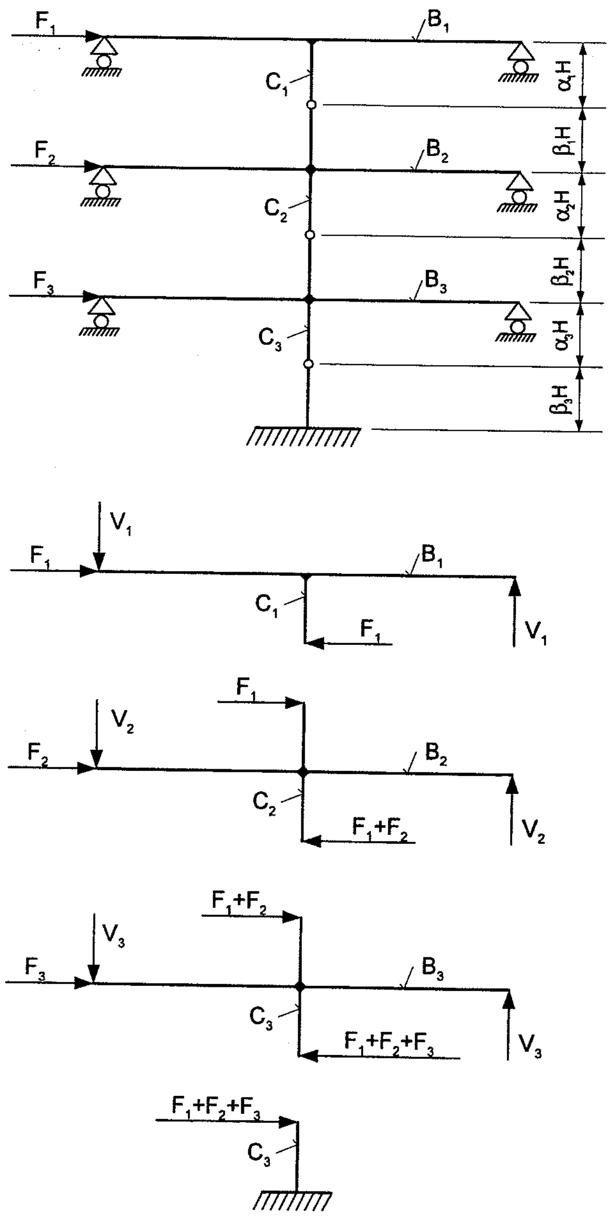

Figure 4. Horizontal seismic forces. The frame is divided to 4 parts by considering inflection points on the column parts

Using this method, the frame can be divided into 4 parts as shown in Figure 4. The vertical reactive forces due to the horizontal seismic forces are as follows:

$$V_1 = 0.65HF_1/(2L),\quad V_2 = H(0.85F_1 + 0.5F_2)/(2L) \tag{12}$$

$$V_3 = H[0.9(F_1+F_2) + 0.4F_3]/(2L). \tag{13}$$

4. Bending moments and axial forces

The bending moment and axial forces acting on beams and column parts, together with the inner forces due to vertical loads are as follows:

Beams: $M_{B1} = V_1L + p_1L^2/8$, $M_{B2} = V_2L + p_2L^2/8$, $M_{B3} = V_3L + p_3L^2/8$ (14)

$N_{B1} = F_1$, $N_{B2} = F_2$, $N_{B3} = F_3$

Column parts: $M_{C1} = 0.65HF_1$, $M_{C2} = 0.5H(F_1 + F_2)$, $M_{C3} = 0.6H(F_1+F_2+F_3)$ (15)

$N_{C1} = N_1$, $N_{C2} = N_2$, $N_{C3} = N_3$.

5. Calculation and constraints on interstorey drifts

Figure 5 shows the structural part B2. The horizontal forces acting on column parts cause a bending moment M. The angle φ due to the bending moment can be calculated using the bending moment diagram M.

$$EI_{B2}\varphi = -\frac{ML}{12} + \frac{ML}{4} = \frac{ML}{6} \tag{16}$$

and the horizontal displacement from this angle is

$$d' = \beta_1 H\varphi = \beta_1 H\frac{ML}{6EI_{B2}} \tag{17}$$

and from the force F_1

$$d'' = \frac{F_1(\beta_1 H)^3}{3EI_{C1}} \tag{18}$$

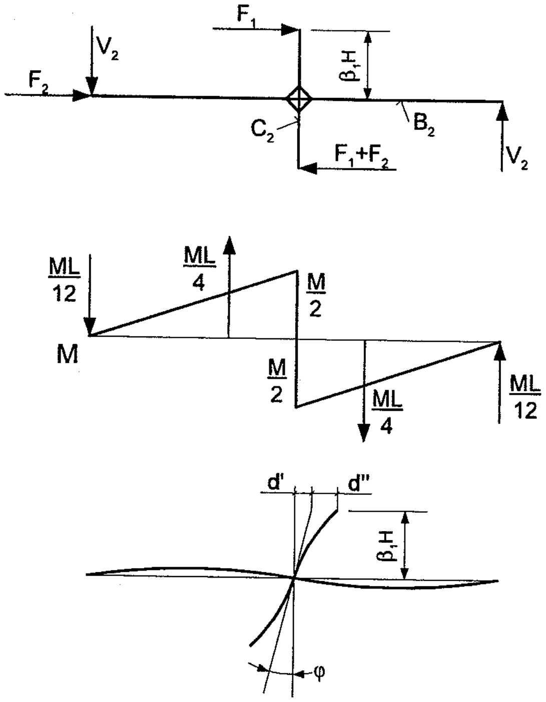

Figure 5. Elastic deformations of the second frame part due to bending moments. The beam deformation causes a horizontal displacement *d'* and the deformation of a column part causes a displacement *d"*

Figure 6 illustrates the elastic deformations of the structural parts. The displacements are as follows (dimensions in N and mm).

$$d_1 = \frac{(F_1 + F_2 + F_3)(\beta_3 H)^3}{3EI_{C3}} \tag{19}$$

$$d_2 = \frac{\alpha_3 H M_3 L}{6EI_{B3}}; \qquad M_3 = \alpha_3 H F_3 + (\alpha_3 + \beta_2) H (F_1 + F_2) \tag{20}$$

$$d_3 = \frac{(F_1 + F_2 + F_3)(\alpha_3 H)^3}{3EI_{C3}} \tag{21}$$

$$d_4 = \frac{\beta_2 H M_3 L}{6EI_{B3}} \tag{22}$$

$$d_5 = \frac{(F_1 + F_2)(\beta_2 H)^3}{3EI_{C2}} = d_7 \tag{23}$$

$$d_6 = \frac{\alpha_2 H M_2 L}{6EI_{B2}}; \qquad M_2 = (\alpha_2 + \beta_1) H F_1 + \alpha_2 H F_2 \tag{24}$$

$$d_8 = \frac{\beta_1 H M_2 L}{6EI_{B2}} \tag{25}$$

$$d_9 = \frac{F_1(\beta_1 H)^3}{3EI_{C1}} \tag{26}$$

$$d_{10} = \frac{(\alpha_1 H)^2 F_1 L}{6EI_{B1}} \tag{27}$$

$$d_{11} = \frac{F_1(\alpha_1 H)^3}{3EI_{C1}} \tag{28}$$

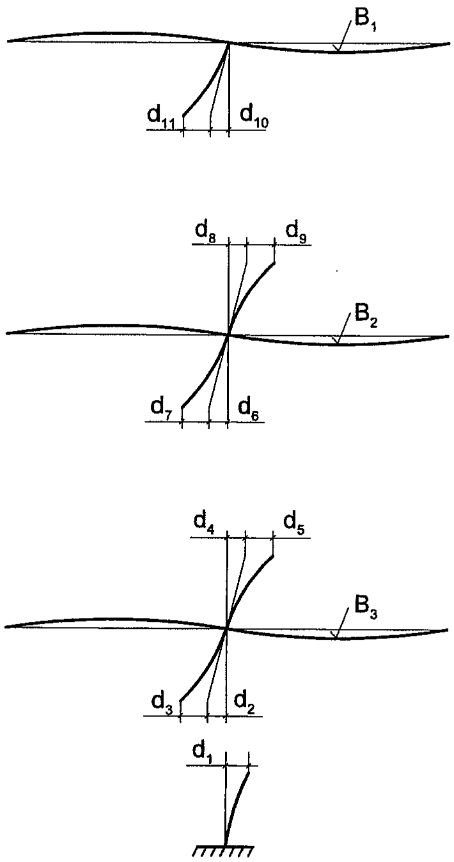

Figure 6. Horizontal displacements of the frame parts for the calculation of interstorey drifts

For the interstorey drift constraint the prescription of EC8 is used. The limiting drift is given as

$$d_r \nu \le 0.005H \tag{29}$$

$$d_r = q\gamma_I d_e \tag{30}$$

The reduction factor for the importance category of III (EC8 1998, Table 4.1) is $\nu = 2$, furthermore $q = 6.5$, $\gamma_I = 1.0$. The constraint on interstorey drift calculated above

$$d_{ei} \le \frac{0.005H}{q\nu} = 1.3846\ \text{mm} \tag{31}$$

It should be noted, that Eurocode 8 (1998) was unrealistically stringent on the storey drift limitation, Japanese rules are not so strict, but the new Final Draft of the EC 8 (2003) has limit similar to the Japanese rule.

Using the above derived displacements the interstorey drift constraints can be formulated as follows:

$$d_{e1} = (d_1 + d_2 + d_3) \le 1.3846 \text{ mm} \tag{32}$$

$$d_{e2} = (d_4 + d_5 + d_6 + d_7) \le 1.3846 \text{ mm} \tag{33}$$

$$d_{e3} = (d_8 + d_9 + d_{10} + d_{11}) \le 1.3846 \text{ mm.} \tag{34}$$

6. Stress constraints for beams and column parts

According to EC3, for simplicity, verifications may be performed in the elastic range only.

6.1 Stress constraints for welded box column parts

$$\frac{N}{\chi_{yC} A_C f_{y1}} + k_{yy} \frac{M}{W_{yC} f_{y1}} \le 1 \tag{35}$$

$$f_{y1} = f_y / \gamma_{M1} = f_y / 1.1$$

$$\chi_{yC} = \frac{1}{\phi_{yC} + \sqrt{\phi_{yC}^2 - \bar{\lambda}_{yC}^2}} \tag{36}$$

$$\phi_{yC} = 0.5\left[1 + \alpha_C\left(\bar{\lambda}_{yC} - 0.2\right) + \bar{\lambda}_{yC}^2\right] \tag{37}$$

$\alpha_C = 0.34$ for a welded box section.

$$\bar{\lambda}_{yC} = \frac{2K_{yC}H}{r_{yC}\lambda_E}; \quad \lambda_E = \pi\sqrt{\frac{E}{f_y}}; \quad r_{yC} = \sqrt{\frac{I_{yC}}{A_C}} \tag{37}$$

The values of $K_{yC}H$ if H=3600 mm are 2160, 1800 and 2340 mm for bottom, middle and top column part, respectively.

For the calculation of k_{yy} the Method 2 is used:

$$k_{yy} = C_{myC}\left(1 + 0.6\bar{\lambda}_{yC}\frac{N}{\chi_{yC} A_C f_{y1}}\right) \le C_{myC}\left(1 + 0.6\frac{N}{\chi_{yC} A_C f_{y1}}\right) \tag{38}$$

C_{myC} =0.4.

Welded box column parts should be used. For this profile the following formulae are valid:

$$A = 4(b-t)t; I_y = I_z = \frac{2}{3}(b-t)^3 t; \tag{39}$$

$$r_y = r_z = \frac{b-t}{\sqrt{6}}; W_y = W_z = \frac{4}{3}(b-t)^2 t \tag{40}$$

In the optimum design process the cross-sectional areas of column parts A_{ci} are selected as unknowns and the cross-sectional characteristics are expressed by A_{ci} as follows

$$I_{yCi} = \frac{A_{Ci}^2}{24\delta}; W_{yCi} = \frac{4\delta}{3}\left(\frac{A_{Ci}}{2\delta}\right)^{3/2} \tag{41}$$

Knowing A_{ci} the dimensions are the following:

$$b-t = \sqrt{\frac{A_C}{4\delta}}; t = \delta(b-t) \tag{42}$$

For UB I profiles approximate formulae are determined on the basis of tabulated values of available sections (Arbed [5]). In order to calculate with continuous values the geometric characteristics of an UB section (I_y, b, t_f) are approximated by curve-fitting functions as follows: h approximately equals to the first number of the profile name (Table Curve 2D [6]).

$$A_s = 1155.684135 + 0.034090823\, h^2 \tag{43}$$

$$t_f = \sqrt{33.20533808 + 0.0006701288\, h^2} \tag{44}$$

$$I_y = \exp\left[35.73636182 - \frac{156.07351689}{\ln(h)}\right] \text{x} 10^4 \tag{45}$$

$$b = \sqrt{5851.784768098 + 0.01671843845 h^2 \ln(h)} \tag{46}$$

$$t_w = \sqrt{15.62577015376 + 4.358946969 \text{x} 10^{-5} h^2 \ln(h)} \tag{47}$$

$$I_{zb} := 10^4 \cdot e^{\left(14.4133364305 - \frac{153.67541403}{\sqrt{h_b}}\right)} \tag{48}$$

$$I_{tb} := 10^4 \cdot e^{\left(11.623190979 - \frac{168.5142170407}{\sqrt{h_b}}\right)} \tag{49}$$

$$I_{\omega b} := 10^9 \cdot \left(-11.8600732979 + 2.835568539110^{-5} \cdot h_b \cdot h_b \cdot \ln(h_b)\right)^2 \tag{50}$$

$$W_{yb} := e^{\left(25.3497083394 - \frac{111.32333718}{\ln(h_b)}\right)} \cdot 10^3 \tag{51}$$

$$W_{zb} := 10^3 \cdot \left(-2.7526203118234 + h_b \cdot 0.0329915015\right)^2 \tag{52}$$

6.2 Stress constraints for beams of UB profile (I beam)

$$\frac{N}{\chi_{yB} A_B f_{y1}} + k_{yyB} \frac{M}{\chi_{LT} W_{yB} f_{y1}} \le 1 \tag{53}$$

$$\frac{N}{\chi_{zB} A_B f_{y1}} + k_{zy} \frac{M}{\chi_{LT} W_{yB} f_{y1}} \le 1 \tag{54}$$

$$\chi_{yB} = \frac{1}{\phi_{yB} + \sqrt{\phi_{yB}^2 - \bar{\lambda}_{yB}^2}} \tag{55}$$

$$\phi_{yB} = 0.5\left[1 + \alpha_{yB}\left(\bar{\lambda}_{yB} - 0.2\right) + \bar{\lambda}_{yB}^2\right]; \alpha_{yB} = 0.21 \tag{56}$$

$$\bar{\lambda}_{yB} = \frac{K_{yB} L}{r_{yB} \lambda_E}; K_{yB} = 1; r_{yB} = \sqrt{\frac{I_{yB}}{A_B}} \tag{57}$$

$$k_{yyB} = C_{myB}\left(1 + 0.6\bar{\lambda}_{yB} \frac{N}{\chi_{yB} A_B f_{y1}}\right) \le C_{myB}\left(1 + 0.6 \frac{N}{\chi_{yB} A_B f_{y1}}\right) \tag{58}$$

$$\chi_{zB} = \frac{1}{\phi_{zB} + \sqrt{\phi_{zB}^2 - \bar{\lambda}_{zB}^2}} \tag{59}$$

$$\bar{\lambda}_{zB} = \frac{K_{zB} L}{r_{zB} \lambda_E}; K_{zB} = 1; r_{zB} = \sqrt{\frac{I_{zB}}{A_B}}; \tag{60}$$

$$k_{zy} = \left(1 - \frac{0.05\bar{\lambda}_{zB}}{C_{mLT} - 0.25} \frac{N}{\chi_{zB} A_B f_{y1}}\right) \ge \left(1 - \frac{0.05}{C_{mLT} - 0.25} \frac{N}{\chi_{zB} A_B f_{y1}}\right) \tag{61}$$

$C_{myB} = C_{mLT} = 0.4.$

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \tag{62}$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right]; \alpha_{LT} = 0.49 \tag{63}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{yB} f_{y1}}{M_{cr}}} \tag{64}$$

$$M_{cr} = C_1 \frac{\pi^2 E I_{zB}}{L^2} \sqrt{\frac{I_{\omega B}}{I_{zB}} + \frac{L^2 G I_{tB}}{\pi^2 E I_{zB}}}; C_1 = 1.847 \tag{65}$$

$E = 2.1x10^5; G = 0.81x10^5$ MPa.

6.3 Local buckling constraint for welded box column profiles

According to EC3 [1]:

$$b_i / t_i \leq 33\varepsilon; \varepsilon = \sqrt{235 / f_y} . \tag{66}$$

7. The objective function

In the first design phase we use the structural volume as an objective function:

$$V = H\left(A_{C1} + A_{C2} + A_{C3}\right) + 2L\left(A_{B1} + A_{B2} + A_{B3}\right) . \tag{67}$$

A refined objective function can be the material cost. A final objective function will be the total cost including material and fabrication costs also for beam-to-column connections.

8. Beam-to-column connections

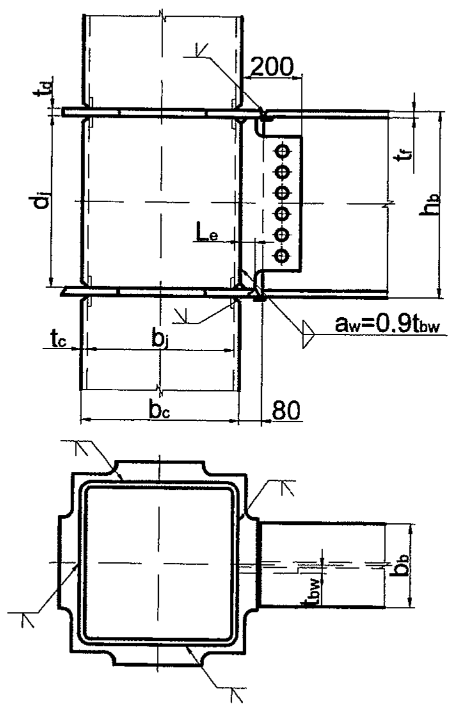

Figure 7. A beam-to-column connection improved for seismic resistance

A lot of connections have been investigated, tested and evaluated by the research team of Kurobane [7,8,9,10,11,12,13,14]. From the improved connections four types have been selected and their cost has been analyzed by Shinde [15,16]. The cheapest one is shown in Figure 7.

Details of the connections in Figures 7 and 9 assume square hollow section columns. The improved field-welded connection in Figure 7 is constructed from two through diaphragms and a shear plate welded by fillet welds to the column flange and to the diaphragms by fillet welds of length L_e. The beam flanges are welded to the diaphragms by butt welds with backing bars. One row of bolts connects the shear plate to the beam web.

Figures 7, 8 and 9 show three different connection versions each improved for seismic resistance. Versions of Figures 8 and 9 use a complete bolted connection of the beam applying a short stub beam instead of a shear tab in version of Figure 7. Version of Figure 8 does not use through diaphragms between the column parts, only two inside diaphragms, so the cutting planes of a column are reduced from two to one. In this case the column widths of the two column parts should be equal.

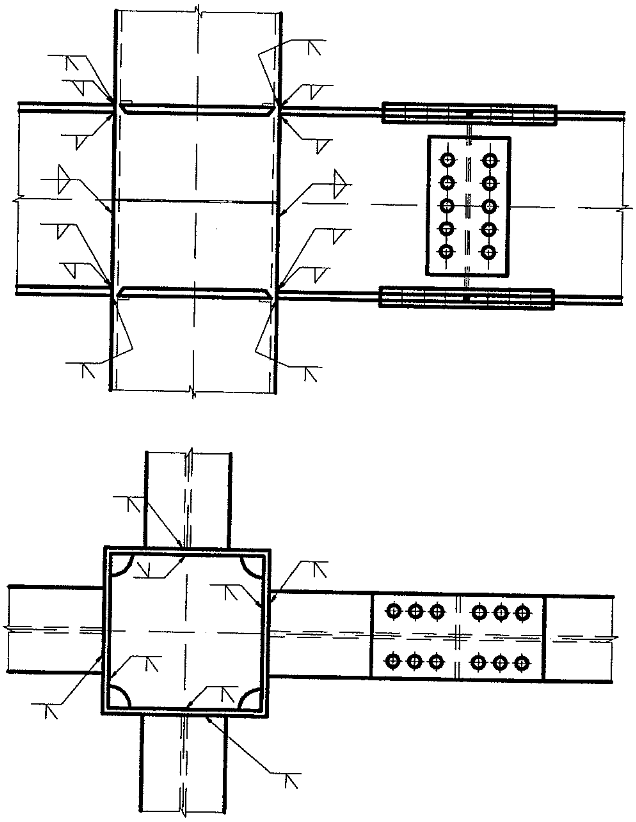

Figure 8. Another beam-to-column connection improved for seismic resistance

It is clear without any detailed cost calculations that the version in Figure 7 is the cheapest. Version shown in Figure 8 uses much more bolt holes, thus, its cost is higher because of the higher cost of drilling bolt holes. The welding cost of all the three versions is approximately the same. Version of Figure 9 needs similar bolt holes drilling cost than that of Figure 8. Thus, according to this approximate cost comparison, the cheapest version of Figure 7 is selected in the present study for detailed cost calculation.

8.1 The connection strength

The strength of the connection is calculated by the following formulae [11].

The ultimate flexural strength at the column face is

$$M_f = M_{fu} + M_{wu} \tag{68}$$

where M_{fu} is the ultimate moment carried by the welded joints between the beam flange and the diaphragms

$$M_{fu} = b_b t_f (h_b - t_f) f_u \tag{69}$$

and M_{wu} is the ultimate moment carried by the fillet welds between the shear plate and column flange as well as the diaphragms

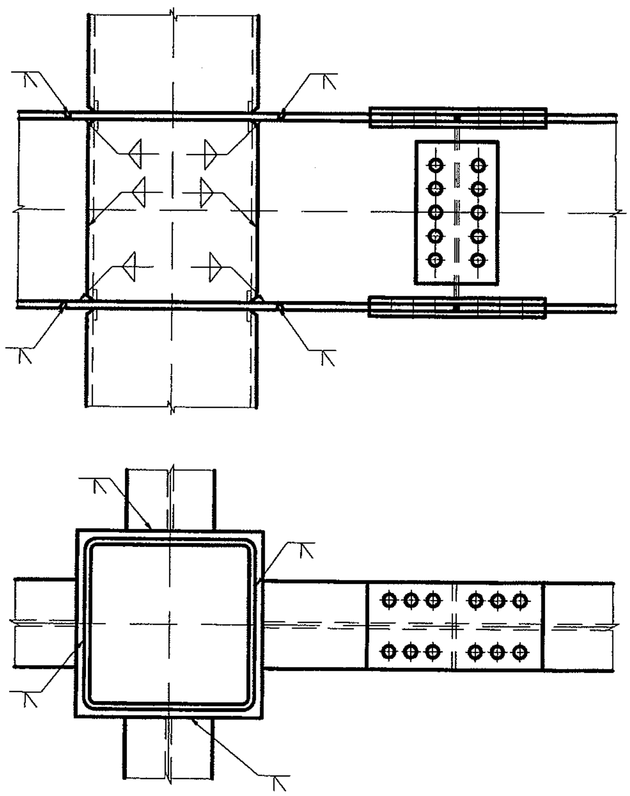

Figure 9. Another beam-to-column connection improved for seismic resistance

$$M_{wu} = mt_{bw}\frac{\left(h_b - 2t_f\right)^2}{4}f_y + L_e\frac{t_{bw}\left(h_b - 2t_f\right)}{\sqrt{3}}f_u \tag{70}$$

where m is the dimensionless moment capacity of the welded web joint expressed as

$$m = 4\frac{t_c}{d_j}\sqrt{\frac{b_j f_{yc}}{t_{bw} f_{yb}}}; m \le 1.0 \tag{71}$$

f_{yb} and f_{yc} are the yield stresses of beam and column steel material, respectively.
The overstrength criterion for the connection is formulated by

$$M_f \ge \alpha M_{pb} \tag{72}$$

where M_{pb} is the plastic moment of the beam and the value of

$\alpha \ge 1.3$

is recommended.

8.2 The cost function of the frame including the cost of connections

For the purpose of optimization we need the cost in function of variables, i.e. dimensions of columns and beams (b_{ci}, t_{ci}, h_{bi}, b_{bi}, t_{fi}, i=1,2,3).

8.2.1. *Material cost*

$$K_M = K_{Mcolumn} + K_{Mbeams} \tag{73}$$

$$K_{Mcolumn} = 1.05x1.08x7.85x10^{-6}H\sum_{i=1}^{3}4t_{ci}\left(b_{ci} - t_{ci}\right) \tag{74}$$

$$K_{Mbeams} = 1.05x0.67x7.85x10^{-6}x4L\sum_{1}^{3}A_{bi} \tag{75}$$

where the factor of 1.05 expresses the 5% material loss, the material cost factors for columns of square box section (108 ¥/kg = 1.08 \$/kg) and for beams of rolled I-section (67¥/kg=0.67\$/kg) are used in Japan. Furthermore the steel density is 7850 kg/m^3 = 7.85x10^{-6} kg/mm^3.
The material costs for plates (diaphragms and shear plates) are neglected.

8.2.2. *Cost of design, assembly and inspection*

According to the Japanese calculation method, the cost of design, assembly and inspection is related to the structural mass as follows:

$k_{drawingn}$ = 1.55 hour/tonne, $k_{assembly}$ = 5.91 h/t, $k_{inspection}$ = 1.80 h/t, together 7.71 h/t.

The fabrication cost factor is k_F = 3125 ¥/h = 31.25 \$/h.

$$K_{D,A,I} = 31.25x7.71x10^{-3}\left(\frac{K_{Mcolumn}}{1.08} + \frac{K_{Mbeams}}{0.67}\right) \quad (76)$$

$$K_{D,A,I} = 2.8600x10^{-5}\sum_1^3 t_{ci}\left(b_{ci} - t_{ci}\right) + 4.7648x10^{-5}\sum_1^3 A_{bi} \quad (77)$$

8.2.3. *Cost of cutting*

The Japanese calculation uses times for programming of numerical control machine. Therefore this calculation results in very high values. Instead of these times we use times for manual cutting with acetylen gas according to speed data of ESAB [17].

Cutting of column parts

For cutting with acetylen gas, according to ESAB [17], quality I, for an average thickness of t_c = 25 mm, the cutting speed is 450 mm/min = 27000 mm/h. We use a beveling factor of 1.1, then the cost of cutting of column parts is

$$K_{C1} = \frac{1.1x31.25}{2.7x10^4}16\sum_1^3 b_{ci} = 2.0370x10^{-2}\sum_1^3 b_{ci} \quad (78)$$

Cutting of beams

$$K_{C2} = \frac{1.1x31.25}{2.7x10^4}2x4\sum_1^3\left(2b_{bi} + h_{bi} - 2t_{fi}\right) = 1.0185x10^{-2}\sum_1^3\left(2b_{bi} + h_{bi} - 2t_{fi}\right) \quad (79)$$

Cutting of diaphragms

$$K_{C3} = \frac{1.1x31.25}{2.7x10^4}8\sum_1^3\left(b_{ci} + 160\right) = 1.0185x10^{-2}\sum_1^3\left(b_{ci} + 160\right) \quad (80)$$

Cutting of shear plates

Cutting speed for thickness of 20 mm is 480 mm/min = 28800 mm/h.

$$K_{C4} = \frac{31.25}{2.88x10^4}4x2\sum_1^3\left(h_{bi} - 2t_{fi} + 200\right) = 0.8680x10^{-2}\sum_1^3\left(h_{bi} - 2t_{fi} + 200\right) \quad (81)$$

Numerically controlled drilling of bolt holes according to Japanese calculation:

Quantity: 72 holes in shear plates and 72 holes in beams

$K_{C5} = 31.25x72(0.0225 + 0.038) = 136.1\$.$

8.2.4. *Cost of welding according to the Japanese calculation*

The Japanese calculation uses welding times in function of equivalent ratios. These ratios are given in function of weld size in tabulated form. For optimization it is better to use these ratios as direct function of weld size. Using a curve fitting technique one can obtain approximate expressions of equivalent ratios for different weld types.

Welding of through-diaphragms in shop with a robot using single-bevel CJP (complete joint penetration) welds with angle of 35°, a root gap of 7 mm and backing strips. The equivalent ratio for this weld type is

$q_1 = (a_1 + b_1 t_f)^2$, $a_1 = 0.437541$, $b_1 = 0.147718$,
the required specific time is 0.0026 h/m = 2.6x10^{-6} h/mm

$$K_{W1} = 31.25x2.6x10^{-6}x16\sum_1^3 b_{ci}\left(a_1 + b_1 t_{fi}\right)^2 = 1.300x10^{-2}\sum_1^3 b_{ci}\left(a_1 + b_1 t_{fi}\right)^2 \qquad (82)$$

Manual field welding of single-bevel welds with backing bars for beam flanges to diaphragms. Root gap of 7 mm and weld angle of 35° is used. The required specific time is 0.074 h/m = 7.4x10^{-5} h/mm, field factor is 1.5, the equivalent ratio can be approximated as

$q_2 = \left(a_2 + b_2 t_f\right)^2; a_2 = 1.107405, b_2 = 0.132698$, t_f in mm.

$$K_{W2} = 1.5x31.25x7.4x10^{-5}x4x2\sum_1^3 b_{bi}\left(a_2 + b_2 t_{fi}\right)^2 \qquad (83)$$

$$K_{W2} = 0.02775\sum_1^3 b_{bi}\left(a_2 + b_2 t_{fi}\right)^2 \qquad (84)$$

Manual shop welding of shear tabs with double fillet welds. The equivalent ratio is approximated by
$q_3 = a_3 + b_3 s^2; a_3 = 0.0041975, b_3 = 0.027771$

s is the perpendicular side size of the fillet weld in mm, $s = 1.22 t_{bw}$.

$$K_{W3} = 31.25x7.4x10^{-5}\sum_1^3\left(a_3 + b_3 s_i^2\right)\left[8\left(h_{bi} - 2t_{fi}\right) + 2x2x60x4x3\right] \qquad (85)$$

$$K_{W3} = 2.312x10^{-3}\sum_1^3\left(a_3 + b_3 s_i^2\right)\left[8\left(h_{bi} - 2t_{fi}\right) + 2880\right]. \qquad (86)$$

9. The mathematical method, optimization and results

9.1. Particle swarm optimization

A new and promising optimization technique is introduced, the particle swarm optimization (PSO). In this evolutionary technique the social behaviour of birds is mimicked. The technique is modified in order to be efficient in technical applications. It calculates discrete optima, uses dynamic inertia reduction and craziness at some particles [18].

A number of scientists have created computer simulations of various interpretations of the movement of organisms in a bird flock or fish school (Millonas [19]). The Particle Swarm Optimization (PSO) algorithm was first introduced by Kennedy [20]. The algorithm models the exploration of a problem space by a population of individuals; the success of each individual influences their searches and those of their peers. In our implementation of the PSO, the social behaviour of birds is mimicked. Individual birds exchange information about their position, velocity and fitness, and the behaviour of the flock is then influenced to increase the probability of migration to regions of high fitness (Kennedy & Eberhard [21]).

Particle swarm optimization has roots in two main component methodologies. Perhaps more obvious are its ties to artificial life in general, and to bird flocking, fish schooling, and swarming theory in particular. It is also related, however, to evolutionary computation, and has ties to both genetic algorithms and evolutionary programming. Particle Swarm optimizers are similar to genetic algorithms in that they have some kind of fitness measure and they start with a population of potential solutions, (none of which are likely to be optimal) and attempt to generate a population containing fitter members.

In theory at least, individual members of the school can profit from the discoveries and previous experience of all other members of the school during the search for food. This advantage can become decisive, outweighing the disadvantages of competition for food items, whenever the resource is unpredictably distributed in patches. Social sharing of information among conspeciates offers an evolutionary advantage: this hypothesis was fundamental to the development of particle swarm optimization.

Millonas [19] developed his models for applications in artificial life, and articulated five basic principles of swarm intelligence. The first one is the proximity principle: the population should be able to carry out simple space and time computations. The second one is the quality principle: the population should be able to respond to quality factors in the environment. The third one is the principle of diverse response: the population should not commit its activities along excessively narrow channels. The fourth one is the principle of stability: the population should not change its mode of behaviour every time the environment changes. The fifth one is the principle of adaptability: the population must be able to change its behaviour mode when it is worth the computational price.

Basic to the paradigm are n-dimensional space calculations carried out over a series of time steps. The population is responding to the quality factors pBest and gBest (gBest is the overall best value, pBest is the best value for a particle). The allocation of responses between pBest and gBest ensures a diversity of response. The population changes its state (mode of behaviour) only when gBest changes, thus adhering to the principle of stability. The population is adaptive because it does change when gBest changes.

The method is derivative free, and by its very nature the method is able to locate the global optimum of an objective function. Constrained problems can simply be accommodated using penalty methods.

Lately, the PSO was successfully applied to the optimum shape and size design of structures by Fourie & Groenwold [20]. An operator, namely craziness, was re-introduced, together with the use of dynamic varying maximum velocities and inertia. An attempt was also made to optimize the parameters associated with the various operators in the case of generally constrained non-linear mathematical programming problems.

9.2. Numerical example and results

Data of the calculated frame are as follows:

To show the effect of the beam length, three values of L are used: $L = 4, 5, 6$ m. $H = 3.6$ m

The interstorey drift limit is as follows (see Eq. 31)

$$d_{ei} \le \frac{0.005H}{q\nu} = \frac{0.005x3600}{6.5x2} = 1.3846\,\text{mm}, \; i = 1,2,3$$

The average weight each column is carrying, w:

$$w = \frac{mn - m - n + 1}{mn} W$$

In our example we chose $m = 4$ and $n = 6$. In this case $w = 0.63$.

Table 2 shows the results for $H = 3.6$ m and different span length L.

Check the connection strength

M_{fu2} is the ultimate moment carried by the welded joints between the beam flange and the diaphragm on floor 2 according to Eq. (69).

The optimum sizes of the beam are as follows

$b_{b2} = 317.92$, $t_{f2} = 24.35$, $t_2 = 30$ mm, at the connection $t_{bw} = 20$, $L_e = 60$ mm.

$M_{fu2} = b_{b2}t_{f2}(h_{b2} - t_{f2})f_u$=2.48x10^9 Nmm

Eq. (70) $M_{wu2} = 9.58\text{x}10^8$ Nmm

Eq.(71) $m_2 = 0.844$

Eq. (72) $\alpha\ M_{p2} = 2.913\text{x}10^9$ Nmm

$\alpha\ M_{p2} < M_{fu2} + M_{wu2} = 3.438\text{x}10^9$ OK.

Table 2. Optimum values of the three welded box columns and the three UB type beams

L (mm)	b_{c1}/t_{c1} (mm)	b_{c2}/t_{c2} (mm)	b_{c3}/t_{c3} (mm)	h_{b1} (mm)	h_{b2} (mm)	h_{b3} (mm)	Cost ($)
4000	400/13	530/17	550/18	533	686	762	10785
5000	490/16	690/21	640/21	533	838	838	16274
6000	**490/24**	**800/30**	**700/29**	**686**	**914**	**914**	**26015**

We have checked the plastic hinges. In the model of EC8 (Figure 6.1) it was assumed that plastic hinges are created at the beam ends, i.e. the plastic static moment of the beams should be smaller than that of columns.

$$W_{pl.column} = 1.5b_i^2t_i > W_{pl.y.beam} \tag{87}$$

On all levels Eq. (87) is fulfilled, the plastic hinges were created at beam ends.

10 Conclusion

Using a relatively simple frame model it is shown how to apply the optimum design system for the case of seismic loads. The cost function to be minimized is formulated on the basis of detailed cost calculations, including the fabrication cost of beam-to-column connections. The connection type is selected from three seismic resistant types by cost comparison. For the constrained cost function minimization the Particle Swarm algorithm is used. The optimum beam and column dimensions are determined for three values of beam length.

In most cases the interstorey drift constraint is active. In some cases the stability is also active. Due to the high material cost and the cost calculation method that the design, inspection and erection costs are proportional to the weight, the mass minima do not differ from the cost minima.

Columns on the second level are a little bit larger that of the ground floor due to the interstorey drift limits. It is possible to use similar columns on the two floors, but it will increase the total cost. Arbed UB profile height is limited to 914 mm. For larger spans other profiles are more suitable.

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References

1. Eurocode 3:2002. prEN 1993-1-1: 2002. *Design of steel structures.* Part 1-1: General structural rules. Brussels, CEN.
2. Eurocode 8. 1998. *Design provisions for earthquake resistance of structures*. Part 1-1: General rules – Seismic actions and general requirements for structures. Part 1-2: General rules – General rules for buildings. Part 1-3: General rules – Specific rules for various materials and elements. Section 3: Specific rules for steel buildings. Brussels, CEN.

   Eurocode 8. 2003. *Design of structures for earthquake resistance* - Part 1: General rules, seismic actions and rules for buildings, Final Draft, December, 215 p. Brussels, CEN.
3. *Design of structures in seismic zones.* 1995. Eurocode 8. Worked examples. Eds Lungu,D., Mazzolani,F. & Savidis,S. Tempus CME 001198/95 Project: Implementing of Structural Eurocodes in Romanian Civil Engineering Standards. Bridgeman Ltd, Timisoara, Romania.
4. Ifrim,M. *Dinamica structurilor si ingineria seismica.* Ed. 2. Editura didactica si pedagogica, Bucuresti, 1984.
5. *Profil Arbed Sales program*, Structural shapes. Arcelor Long Commercial, 2001.
6. TableCurve 2D: Users' manual, Systat Software Inc., 2003.
7. Kurobane,Y., Azuma,K. & Ogawa,K. 1997. *Brittle fracture in steel building frames* – Comparative study on Northridge and Kobe earthquake damage. IIW Doc. XV-946-97.
8. Kurobane,Y. *Improvement of I beam-to-RHS column moment connections for avoidance of brittle fracture.* Tubular Structures VIII. Proc. 8th International Symposium on Tubular Structures, Singapore. Eds Choo,Y.S & Van der Vegte, G.J. Balkema, Rotterdam-Brookfield, 1998.

9. Kurobane, Y., Makino, Y., Miura, K., Tokutome, Y. and Tanaka, M.: *Testing of new RHS column-to-beam connections with U-shaped welded joints*, Tubular Structures IX, R. Puthli and S. Herion eds, Balkema, Rotterdam, the Netherlands, pp. 493-502, 2001

10. Kurobane, Y., Azuma, K. and Makino, Y.: *Fully restrained beam-to-RHS column connections with improved details*, Proceedings, 10th International Symposium on Tubular Structures, Madrid, Spain, pp. 439-446, 2003.

11. Kurobane,Y., Packer,J.A., Wardenier,J. and Yeomans,N.F.: *Design guide for structural hollow section column connections.* CIDECT Design Guide No.9. Verlag TÜV Rheinland Köln,Germany (in press).

12. Azuma, K., Kurobane, Y. and Makino, Y.: *Cyclic testing of beam-to-column connections with weld defects and assessment of safety of numerical modeled connections from brittle fracture*, Engineering Structures, Vol. 22, No. 12, 2000, pp. 1596-1608

13. Miura, K., Makino, Y. Kurobane, Y., Tanaka, M., Tokutome, Y., and Vegte, van der, G.J., *Testing of beam-to-column connections without weld access holes.* Int. Journal of Offshore and Polar Engineering, Vol. 12, No. 3, pp.229-235, 2002

14. Obukuro,Y., Makino, Y., Vegte, van der, G.J., Kurobane, Y., Azuma, K., and Shinde, H., *Testing of beam-to-RHS column field connections using F14T high strength bolts*, 7th Int. Conf. on Steel & Space Structures, Singapore, pp.351-358, 2002

15. Shinde, H., *Experimental study into steel beam-to-column connections with improved details at beam ends*, M.S. Thesis, Sojo University, Kumamoto, Japan, Feb. 2004 (in Japanese).

16. Shinde, H., Kurobane, Y, Azuma, K., Makino, Y. and Obukuro, Y.: *Additional full-scale testing of beam-to-column connections with improvements in welded joints*, Proceedings, 13th International Offshore and Polar Polar Engineering Conference, Honolulu, Hawaii, USA, pp. 243-249, 2003

17. ESAB: *Cutting and consumption table.* Cutting speed according to DIN 2310. 2003.

18. Farkas,J. & Jármai,K.: *Economic design of metal structures.* Rotterdam, Millpress, 2003.

19. Millonas,M.M.: *Swarms, phase transitions, and collective intelligence.* In Langton,C.G. Ed., *Artificial Life III.* Addison Wesley, Reading, MA. 1994.

20. Kennedy,J.: *The particle swarm: social adaptation of knowledge*, Proceedings of the International Conference on Evolutionary Computation, IEEE, Piscataway NJ, 1977, 303-308.

21. Kennedy,J. & Eberhart,R.C.: *Particle swarm optimization*, Proceedings of the 1995 IEEE International *Conference on Neural Networks*, Perth, Australia, IEEE Service Center, Piscataway, NJ, Vol. 4. 1995, 1942-1948.

22. Fourie,P.C. & Groenwold,A.A.: *Particle swarm in size and shape optimisation*, International *Workshop on Multidisciplinary Design Optimization*, 7-10, Aug. 2000, Pretoria, South Africa, Proceedings 97-106.