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Cost comparison of bolted and welded frame joints

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ABSTRACT

Considering a braced frame structure, a double-sided beam-to-column connection is investigated. The joint is loaded by two equal and opposite bending moments. In the case of a rigid welded joint the beam is designed for a known maximum bending moment. Since the angle deformation of a semi-rigid bolted connection decreases the maximum bending moment, for this version a smaller beam section is selected and, calculating the rotational stiffness, is verified for the same uniformly distributed normal load. Because the rotational stiffness depends on many parameters, the problem can be handled by a numerical example only. The detailed cost calculation, using cost data from Great Britain and South-Africa, shows that, in this case, the bolted version is about 6-7% cheaper than the welded one. This difference is caused partly by the material and partly by fabrication cost differences.

Key words: beam-to-column connections, bolted joints, welded joints, cost analysis, economy of steel structures

1. INTRODUCTION

Beam-to-column connections can be constructed as bolted or welded ones. Welded joints are rigid, while bolted connections are semi-rigid, since the local angle deformations of their structural components cause changes in original angles between the beam and column. This change of angles decreases the bending moment in joints and enables the usage of smaller beam sections, which leads to cost savings. Recent studies have shown that this savings depend on the frame type (braced or unbraced) and can be about 6-20% [1,2,3].

Another aspect is that the site welding of high-rise building frames cannot be performed in such a good quality than in a workshop.

Several studies have been worked out for the optimum design of frames with semi-rigid joints, e.g. [4,5]. The changes of angles influence the frame stability [6].

The aim of the present study is to illustrate the cost differences between the bolted and welded connection by detailed cost analysis in a numerical example.

2. THE STRUCTURAL MODEL

Consider an inner beam-to-column connection of a braced frame (Fig.1). In the case of a rigid welded joint the beam should be designed for the maximum bending moment (Fig.2a)

$$M_0 = \frac{qL^2}{12} . \tag{1}$$

In the case of beams with semi-rigid bolted joints and of moment of inertia I_1 consider a simply supported beam (Fig.2b). The uniformly distributed normal load of intensity q causes an angle deformation at supports

$$\varphi_0 = \frac{qL^3}{24EI_1} , \tag{2}$$

and the bending moments M cause the angle deformation

$$\varphi_1 = \frac{ML}{2EI_1} . \tag{3}$$

The local angle deformation caused by the flexibility of a bolted joint equals the difference of these angles

$$\theta = \varphi_0 - \varphi_1 = \frac{qL^3}{24EI_1} - \frac{ML}{2EI_1} . \tag{4}$$

This local angle deformation can be calculated with the rotational stiffness S_j

$$\theta = \frac{M}{S_j}. \tag{5}$$

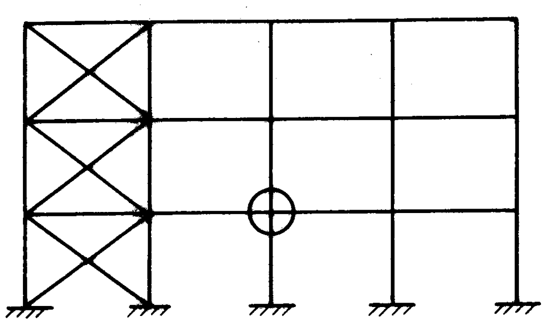

Fig.1. A double-sided beam-to-column connection in a braced frame

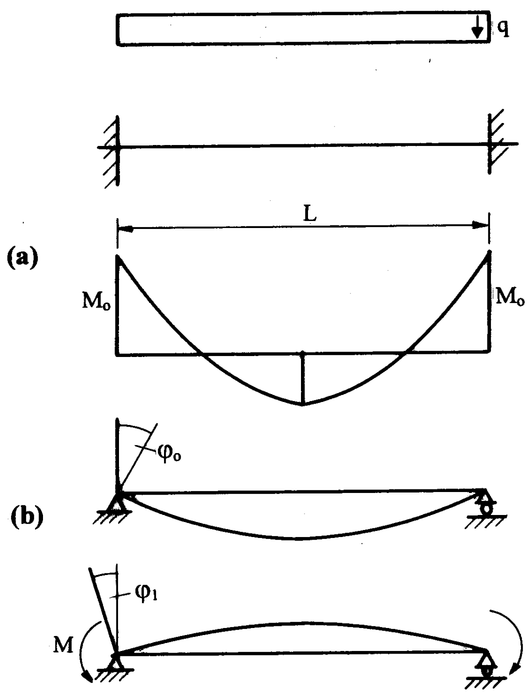

Fig.2. (a) Bending moment diagram for rigid welded beam-to-column connections; (b) angle deformations from the load *q* and from bending moments *M*

Using (4) and (5) the bending moment can be calculated

$$M = \frac{qL^2}{\frac{24EI_1}{S_j L} + 12}. \tag{6}$$

Note that for rigid joints $S_j \to \infty$ and $M = M_0$.

The required section modulus for the beam with bolted ends is

$$W = \frac{M}{f_y / \gamma_{M0}} \tag{7}$$

where f_y = 275 MPa is the yield stress divided by a partial safety factor of $\gamma_{M0} = 1.1$.

The beam section can be obtained by iteration. First select a beam section smaller than that for the welded joint, select a bolted joint, calculate S_j and M, and verify the beam section; then change the beam section, if needed.
For the calculations the publications of The Steel Construction Institute [7,8,9] are used.

3. DESIGN OF THE BEAM WITH WELDED BEAM-TO-COLUMN CONNECTION

For the factored load q = 88 N/mm, we take L = 6 m and $f_y / \gamma_{M0} = 250$ MPa, thus, with (1) M_0 = 264 kNm, the required section modulus is $W_{req} = 264\text{x}10^6/(250\text{x}10^3) = 1056\text{x}10^3$ mm^3 and we select the 406x178x60 UB profile of $W = 1063\text{x}10^3$ mm^3. Note that, according to [10], welded connections without stiffeners on the column web as shown in Fig.4, generally fulfil the requirement for adequate rotational stiffness.

4. CALCULATION OF ROTATIONAL STIFFNESS

For this calculation we use the new Annex J of the Eurocode 3 [11].
The stiffness is calculated from the stiffness coefficients of components k_i as follows

$$S_j = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}}, \tag{8}$$

where $E = 2.1\text{x}10^5$ MPa is the elastic modulus. Since the stiffness depends on many parameters, for our purpose a numerical example is selected.
The calculation is performed for an extended end-plate bolted connection shown in Fig.3. For the beam the 406x178x54 UB and for the column the 305x305x97 UC profile is selected, steel grade of 43, M 20 bolts.
In (8) the lever arm is $z = h_b - t_{fb}$ = 402,6 – 10.9 = 391.7 mm. For the ratio of initial and actual stiffness $\mu = 2$ is used. The moment capacity of this beam-to-column connection is M = 247 kNm [8].

5. CALCULATION OF STIFFNESS COEFFICIENTS k_i

k_1 *for an unstiffened column web panel in shear*. For a double sided joint with equal and opposite moments $k_1 = \infty$.

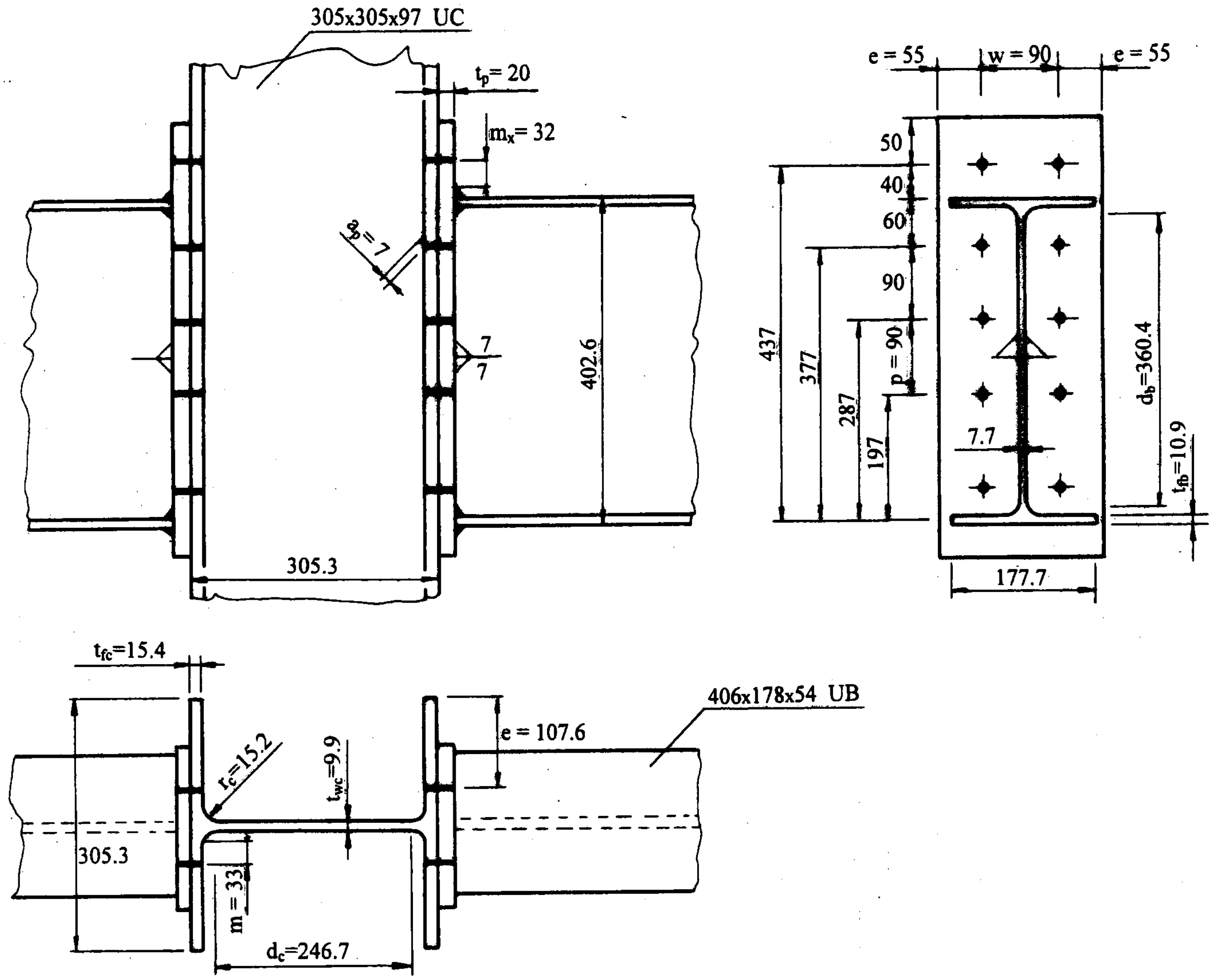

Fig.3. A double-sided bolted beam-to-column connection with extended end-plates

k_2 for an unstiffened column web in compression

$$k_2 = \frac{0.7 b_{eff.C.WC} t_{WC}}{d_C}, \tag{9}$$

where

$$b_{eff.C.WC} = t_{fb} + 2\sqrt{2}a_p + 5\left(t_{fC} + r_C\right) + 1.5t_P = 10.9 + 2\sqrt{2}x7 + 5\left(15.4 + 15.2\right) + 30 = 213.7\ \text{mm}$$

and $k_2 = 6.0$ mm.

For a column web in tension

$$k_3 = \frac{0.7 b_{eff.t.WC}.t_{WC}}{d_C}. \tag{10}$$

This coefficient should be calculated for each bolt-row in the tension zone.

For the outside end row:

$b_{eff.t.WC}$ is the smallest value of $2\pi m; \pi m + e_1; 4m + 1.25e_1; 2m + 0.625e + e_1; e_1 + 0.5p$

m = 33, e = 107.6, e_1 = 50, p =100, thus, we calculate with 100, k_{31} = 2.81 mm.

For an inner row:

$b_{eff.t.WC}$ is the smallest value of $2\pi m; 4m + 1.25e; 2p_2; p_2$, we calculate with p_2 = 90 mm, k_{32} = k_{33} = k_{34} = 2.53 mm.

For a column flange in bending

$$k_4 = \frac{0.85 l_{eff} t_{fC}^3}{m^3}, \tag{11}$$

$l_{eff} = b_{eff.t.WC}; t_{fC} = 15.4; m = 33$, for the outside end row k_{41} = 8.64, for inner rows k_{42} = k_{43} = k_{44} = 7.77 m.

For the end-plate in bending

$$k_5 = \frac{0.85 l_{eff.P} t_P^3}{m^3}. \tag{12}$$

For the outside end row $l_{eff.P}$ is the smallest value from

$2\pi m_x; \pi m_x + w; \pi m_x + 2e; 4m_x + 1.25e_x; e + 2m_x + 0.625e_x; 0.5w + 2m_x + 0.625e_x$

With values of m_x = 32, w = 90, e_x = 50, e = 50, t_p = 20, m = 33, $l_{eff.P}$ = 140.4, k_{51} = 29.1 mm.

For the first inner row $l_{eff.P}$ is the smallest value from

$2\pi m; \alpha m; \pi m + p; 0.5p + \alpha m - (2m + 0.625e)$; α should be determined from a diagram (Fig.J.27 in [10]) in function of $\lambda_1 = \dfrac{m}{m+e}, \lambda_2 \dfrac{m_2}{m+e}$. With m = 33, p = 90, e = 50, m_2 = 32 and $\alpha = 6.5$

$l_{eff.P}$ = 162.25 mm, and k_{52} = 30.70 mm.

For other inner rows the smallest from $2\pi m; 4m + 1.25e; 2p; p$ is p = 90 and k_{53} = k_{54} = 17.03 mm.

For a single bolt-row in tension

$$k_{10} = \frac{1.6 A_S}{L_b}, \tag{13}$$

A_S is the tensile stress area of the bolt, L_b is the elongation length i.e. total thickness of material and washers + half the sum of the height of the bolt head and the height of the nut. For a M 20 bolt it is A_S = 314 mm^2, L_b = 35.4 + 6 + (17+12.5)/2 = 56.15 mm and k_{10} = 8.95 mm.

From k_3, k_4, k_5 and k_{10} an effective coefficient should be calculated for each bolt-row

$$k_{eff.r} = \frac{1}{\sum_r \frac{1}{k_{i.r}}}, \tag{14}$$

where i = 3, 4, 5, 10. To consider the distance of a bolt-row from the centre of compression, an equivalent stiffness coefficient is calculated

$$k_{eq} = \frac{\sum_r k_{eff.r} h_r}{z_{eq}}, \tag{15}$$

where

$$z_{eq} = \frac{\sum_r k_{eff.r} h_r^2}{\sum_r k_{eff.r} h_r}. \tag{16}$$

The calculation is summarized in Table 1.

Table 1. Calculation of the equivalent stiffness for 4 bolt-rows

Bolt-row r	k_{3i}	k_{4i}	k_{5i}	k_{10i}	$k_{eff.r}$	h_r	$k_{eff.r}h_r$	$k_{eff.r}h_r^2$
1	2.81	8.64	29.1	8.95	1.61894	437	707.98	309167
2	2.53	7.77	30.70	8.95	1.49642	377	564.15	212685
3	2.53	7.77	17.03	8.95	1.44071	287	413.48	118670
4	2.53	7.77	17.03	8.95	1.44071	197	283.82	55913

$$\sum_r k_{eff.r} h_r = 1968.93; \sum_r k_{eff.r} h_r^2 = 696435; z_{eq} = 353.71; k_{eq} = 5.5665 \text{ mm.}$$

For the bolted connection

$$\sum_i \frac{1}{k_i} = \frac{1}{k_2} + \frac{1}{k_{eq}} = 0.34631 \text{ mm}^{-1},$$

$$S_j = \frac{2.1x10^5 x 391.7^2}{2x0.34631} = 4.6519x10^{10} \text{ Nmm.}$$

The beam profile selected for a bolted connection, 406x178x54 UB has a moment of inertia I_1 = 18.72x10^7 mm^4. From (6) one obtains M = 206 kNm, the required section modulus is W_{1req} = 824 mm^3, the section modulus of the selected profile is W_1 = 930 mm^3, and it is not necessary to follow the calculation with an iteration.

Check for shear: the actual shear force acting on the joint is 88x3 = 264 kN, the shear capacity of a bolt-row in tension is [8] 74 kN and of the row dedicated to shear it is 184 kN, together 480 kN, which is more than required.

6. COST CALCULATIONS USING THE STEEL CONSTRUCTION INSTITUTE (GREAT BRITAIN) SUGGESTIONS

Welded connection (Fig.4)

Material cost: according to [9] for 406x178x60 UB the specific cost is Ł 23/m. We calculate with a half beam length, 3 m, so K_M = 23x3 = 69.0 Ł. (1 Ł = 1.43 $).

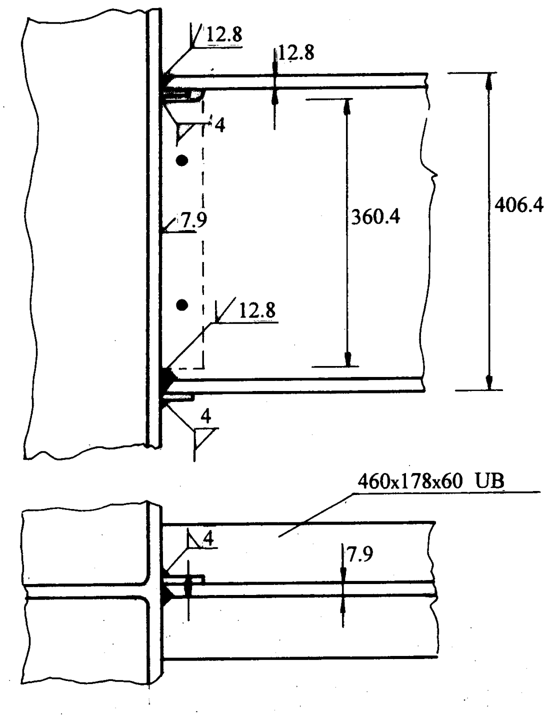

Fig.4. Fully welded connection

The fabrication cost is calculated as follows:

$$K_F = K_C + k_F\left(\Theta_W \sqrt{\kappa\rho V} + 1.3\sum_i \alpha_{Pi} C_{Wi} a_{Wi}^n L_{Wi}\right) \tag{17}$$

where K_C is the cost of cutting, according to [9] for a half beam it is Ł 25/2 = Ł 12.5, k_F = 25 Ł/hr = 0.42 Ł/min = 0.60 $/min is the fabrication cost factor, $\Theta_W = 2$ is the complexity factor for welding, $\kappa = 2$ is the number of structural parts to be assembled (in this case we calculate with the beam and a backing strip for welding of web), ρV = 60.1x3 = 180.3 kg is the mass of half beam, the factor of 1.3 expresses the effect of additional works (electrode changing, chipping, deslagging), α_P is a coefficient expressing the effect of welding position, according to [9] for downhand welding it is 1, for vertical welding 2, for overhead welding 3. L_W (mm) is the weld length, $C_W a_W^n$ is given for different types of welds and for different welding technologies in [12,13] according to

COSTCOMP data [14,15] for downhand welding position, a_W (mm) is the weld size. The first term in parentheses expresses the time of assembly and tacking: $2\sqrt{2x180.3} = 38$ min.

The parts of the second term in parentheses are as follows.

Welds for two flanges, a_W = 12.8, L_W = 2x177.9, single bevel butt welds, SMAW (shielded metal arc welding)

$1.3x0.5214x10^{-3}x12.8^2x2x177.9 = 40.0$ min

Weld for web, a_W = 7.9, L_W = 360.4, vertical position, single bevel butt weld, SMAW

$2x1.3x0.5214x10^{-3}x7.9^2x360.4 = 30.5$ min

Welding of backing strips for two flange welds, overhead position, a_W = 4, L_W = 2x177.9, SMAW fillet welds

$3x1.3x0.7889x10^{-3}x4^2x2x177.9 = 17.5$ min

Welding of backing strip for web weld, vertical position, SMAW fillet weld, a_W = 4, L_W = 360.4

$2x1.3x0.7889x10^{-3}x4^2x360.4 = 12.0$ min.

Altogether K_F = 58 Ł = 83 $ and total cost is $K_M + K_F$ = 69+83 = 152 Ł = 182 $.

Bolted connection

Material cost for 406x178x54 UB is 21x3 = 63.0 Ł.

According to [9] the cutting cost for a half beam is 25/2 = 12.5 Ł.

In [9] cost is given for three fabrication categories: low, medium and high. Low is for flush end-plate, medium is for extended end-plate and high is for haunched end-plate. We use an extended end-plate. Calculating with medium cost including welding of the end-plate to the beam and bolting implies a cost of 87/2 = 43.5 Ł for a beam end.

The total cost is 119.0 Ł =170 $.

Comparison

The bolted version is 7% cheaper than the welded one. This difference is caused partly by the material and partly by fabrication cost difference.

7. COST CALCULATIONS USING SOUTH AFRICAN DATA

Quotes obtained from South African Suppliers and Manufacturers are listed in Table 2. The exchange rate is 13 Rand (R) $\approx$ 1 USD.

Table 2. South African cost data

Item	Units	Quantity (Rand)	Source
UB 406 x 178 x 60	R/m	337.22	Alert Steel [16]
UB 406 x 178 x 54	R/m	303.56	Alert Steel
UC 305 x 305vx97	R/m	559.02	Alert Steel
Plate 2.5 X 1.2 x 20	R	2533.51	Alert Steel
Flat bar for back strips 40 x 5	R/m	8.04	Alert Steel
Total Overhead, Labour, Consumables & Power	R/h	180	Spencer Erling SAISC [17]
Cost of cutting plates : material cost		1.08	Alert steel
Drilling of M20 holes	R/hole	5	Jan Brand UP [18]
8.8grade M20bolts	R each	6.97	Screw Man [19]

Welded connection

Using the material cost indicated, the cost of half a beam (3m) 406 X 178 x 60 UB is R809.338 and of the back strip R2.31 giving a total material cost of R811.64.

With the welding times calculated by $\Theta_w \sqrt{\kappa \rho V} + 1.3 \Sigma \alpha_{pi} C_{wi}{}^n L_{wi}$ = 2((38 + 40 + 30.5+17.5+12) =138 min and the cost factor of R3/min, the welding cost equates to R414. Thus the total cost of the welded joint is R1225 ($94).

Bolted connection

For the bolted connection the material cost consists of (3)x(R303.56) =R728.54 for the 406x178x54UB, R2026.81/30 = R67.56 for the endplate of 500x200x20 of which 30 can be cut from a plate and R69.70 for 10 M20 bolts at R6.97 each.

Fabrication cost is contributed by cutting of the plate which constitutes roughly 8% of material cost, i.e. R5.40, drilling of 10m holes at R5 each and welding of the endplate to the beam. For single bevel butt welding of the flanges the welding time is given by 1.3x0.5214x10^{-3}x10.9x10.9x2x177.9 = 28.6 min. The time for the single bevel butt weld for the web is 1.3x0.51214x10^{-3}x7.7x7.7x360.4. = 14.5 min. The time for assembly is 2x$\sqrt{2x54x3}$ =36 min. At a rate of R3/min the total welding cost is 3x79.1=R237.31. The total cost of a welded joint thus amounts to R1158 ($89).

Comparison

The bolted connection is about 6% cheaper than the welded one. The material cost is however 7% more expensive and manufacturing is 6% cheaper than for the welded connection. Whereas material cost constitutes 75% of the total cost of the bolted joint and fabrication cost 25%, with the welded connection material cost is 66% of total cost and fabrication cost is 34%. Welded connection are thus dependent on labour and overhead rates.

Further comments

The manufacturer [20] pointed out the fact that erection of a structure with welded connections would be difficult and that the bolted connection would be preferred anyway. His recommendation is also that the end plates should be laser cut which would put the cost of a endplate at R80-90, material included. Labour and overhead costs are calculated at R50 to 60/hour when high production low schooled labour is used. The rate when qualified labour has to be used is R130 to R150. The ratio of vertical to overhead and welding times to flat welding he estimated at 2 to 3.

8. CONCLUSIONS

In Table 3 the summary of costs is given in $.

Table 3. Summary of costs in $

Joint	Cost of	Great Britain	South Africa
welded	material	99	62
	fabrication	83	32
	total	182	94
bolted	material	90	67
	fabrication	80	22
	total	170	89

The detailed cost calculations show that, in this numerical example, according to Great Britain data, the bolted connection is 7% cheaper than the welded one. This difference has two components: the difference between material costs and the difference between fabrication costs.

According to the South African data, the bolted connection is about 6% cheaper than the welded one. The material cost is however 7% more expensive and manufacturing is 6% cheaper than for the welded connection.

It can be concluded that the bolted connections are more economic than the fully welded ones. Since the rotational stiffness of semi-rigid bolted connections is smaller than that of welded ones, the maximum bending moment in a braced frame structure is smaller and the beam section can be smaller. The difference between the fabrication costs is significant as well. The disadvantage of bolted connections is the very complicated calculation of rotational stiffness. This causes difficulties in the optimum design of frames with semi-rigid beam-to-column connections.

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