

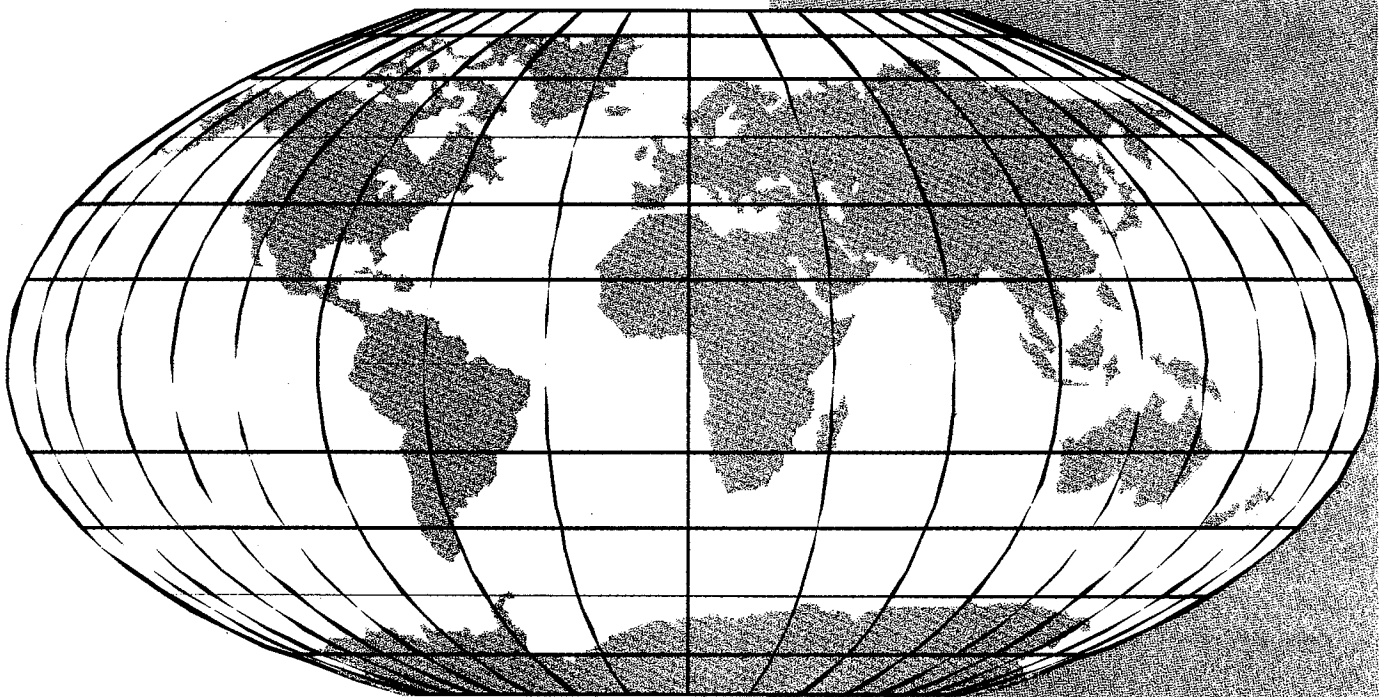
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- engineering properties of the joints
- health and safety and environmental aspects
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- quality assessment
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COST COMPARISON OF BOLTED AND WELDED 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 main aim is to show a detailed calculation of semi-rigid connections. To perform this calculation, cost data from Great Britain and South Africa are used. The example shows differences in costs of bolted and welded versions. This is caused partly by the material and partly by fabrication cost differences.

IIW-Thesaurus keywords: Girders; Columns; Bolts; Nonwelded joints; Welded joints; Steel construction; Costs; Computation; Prices; UK; South Africa; Comparisons; Reference lists.

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 the 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 between the rigid and semi-rigid connections depend on the frame type (braced or unbraced) and can be about 6-20% [1, 2, 3]. It was shown in an extensive literature study made by Weynand [2] that the cost savings are usually larger for braced frames and smaller for unbraced ones.

In this study elastic design is considered, because in practice it is used most as indicated by Steenhuis, Weynand and Gresnigt [4]. If frame calculations are made using plastic design, the slenderness ratios for the buckling constraints are smaller, which means a thicker and less economic cross section [5].

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

Several studies have been worked out for the optimum design of frames with semi-rigid joints, e.g. [6, 7]. Simões showed a 12% difference in weight between semi-rigid and fully rigid connections in favour of the semi-rigid ones. The changes of angles influence the frame stability [8].

The aim of the present study is to illustrate the cost differences between the bolted and welded connections 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)

$$M_0 = \frac{qL^2}{12} \quad (1)$$

In the case of beams with semi-rigid bolted joints and 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

$$\varphi_0 = \frac{qL^3}{24EI_1} \quad (2)$$

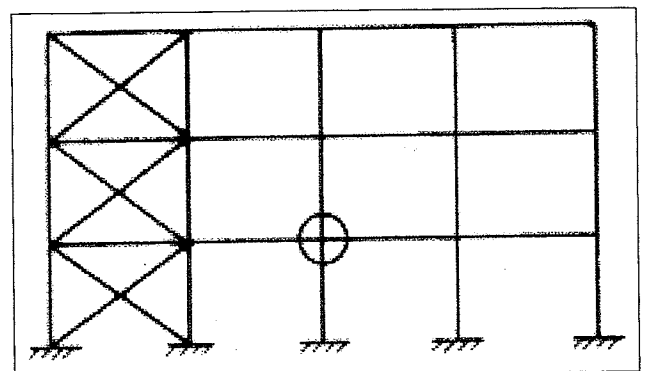


Fig. 1. A double-sided beam-to-column connection in a braced frame.

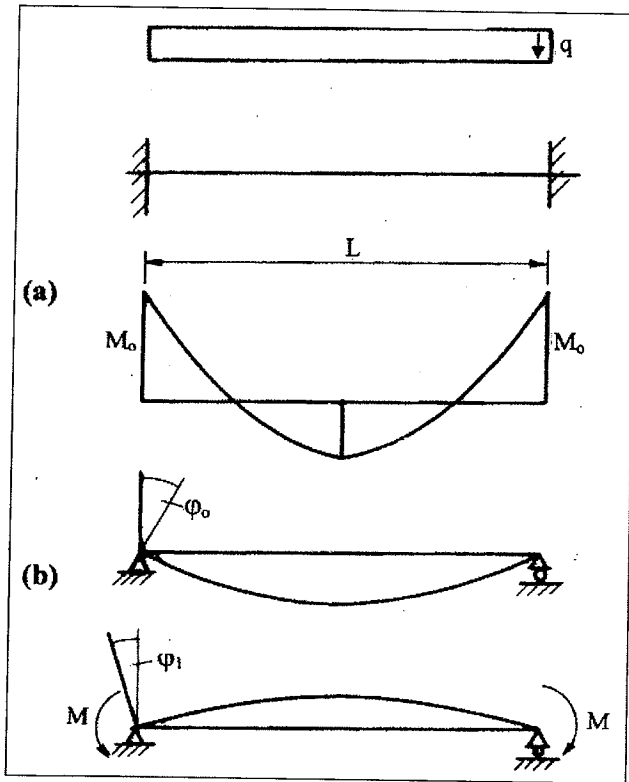


Fig. 2. (a) Bending moment diagram for rigid welded beam-to-column connections; (b) angle deformations from the load q and from bending moments M .

and the bending moments M cause the angle deformation

$$\varphi_1 = \frac{ML}{2EI_1} \quad (3)$$

The local angle deformation caused by the flexibility of a bolted joint is equal to the difference of these angles

$$\theta = \varphi_0 - \varphi_1 = \frac{qL^3}{24EI_1} - \frac{ML}{2EI_1} \quad (4)$$

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

$$\theta = \frac{M}{S_j} \quad (5)$$

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

$$M = \frac{qL^2}{\frac{24EI_1}{S_j L} + 12} \quad (6)$$

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

The required section modulus for the beam with bolted ends is

$$W = \frac{M}{f_y / \gamma_{M0}} \quad (7)$$

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 [9, 10, 11] 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 $W_{req} = 264 \times 10^6 / (250 \times 10^3) = 1056 \times 10^3$ mm³ and we select the 406 × 178 × 60 UB profile of $W = 1063 \times 10^3$ mm³. Note that, according to [12], welded connections without stiffeners on the column web as shown in Fig. 3, 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 [13].

The stiffness is calculated from the stiffness coefficients of components k_i as follows

$$S_j = \frac{Ez^2}{\mu \sum \frac{1}{k_i}} \quad (8)$$

where $E = 2.1 \times 10^5$ 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 406 × 178 × 54 UB and for the column the 305 × 305 × 97 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 [10].

5 CALCULATION OF STIFFNESS COEFFICIENTS k_i

5.1 k_1 for an unstiffened column web panel in shear

For a double sided joint with equal and opposite moments $k_1 = \infty$.

5.2 k_2 for an unstiffened column web in compression

$$k_2 = \frac{0.7 b_{eff.c.wc} t_{wc}}{d_c} \quad (9)$$

where

$$b_{eff.c.wc} = t_{fb} + 2\sqrt{2}a_p + 5(t_{fc} + r_c) + 1.5t_p = 10,9 + 2\sqrt{2} \times 7 + 5(15,4 + 15,2) + 30 = 213,7 \text{ mm and } k_2 = 6,0 \text{ mm.}$$

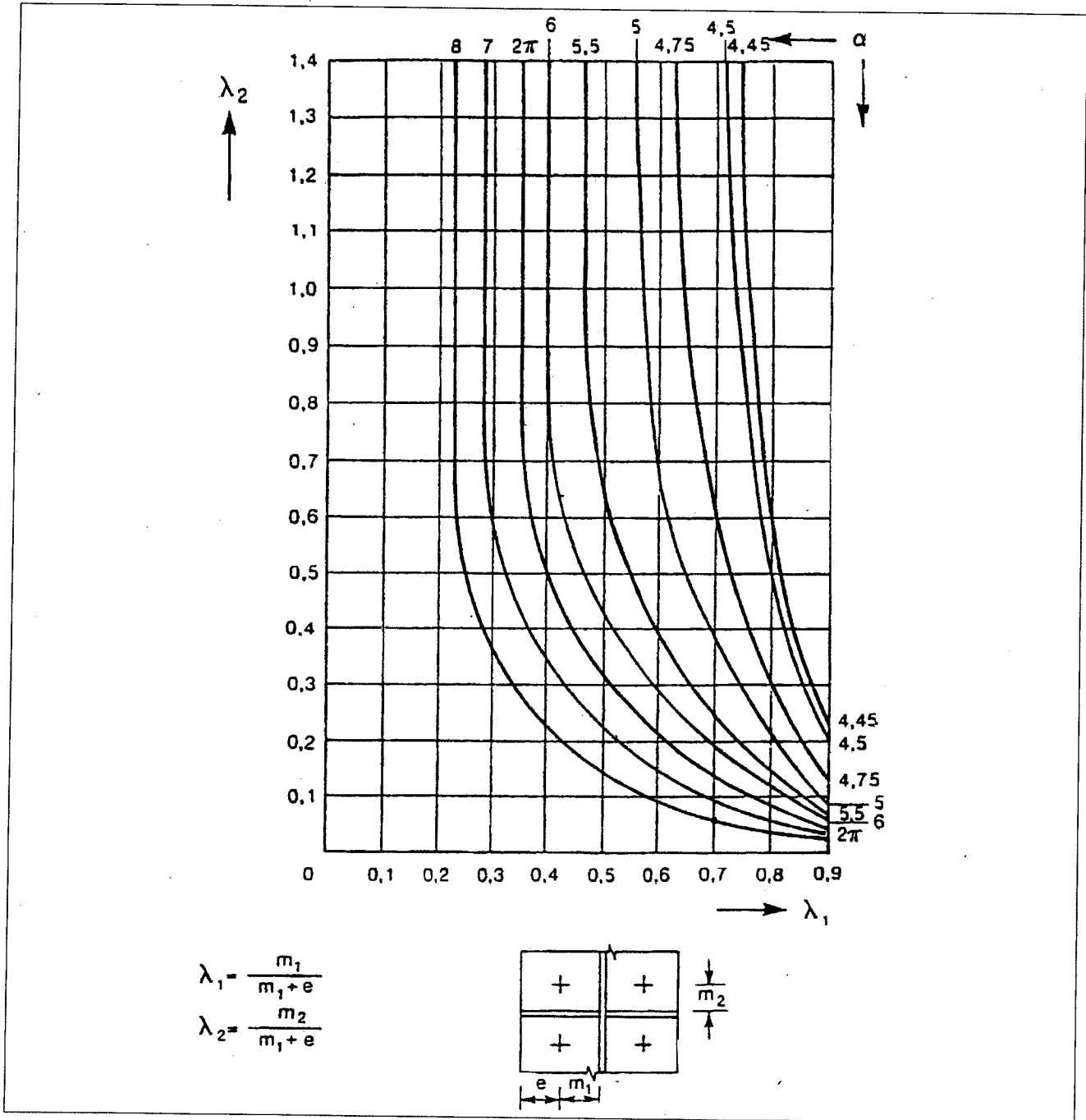


Fig. 4. Values of α .

$$k_{eff,r} = \frac{1}{\sum_r \frac{1}{k_{i,r}}}, \quad (14)$$

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

$$k_{eq} = \frac{\sum_r k_{eff,r} h_r}{z_{eq}}, \quad (15)$$

where

$$z_{eq} = \frac{\sum_r k_{eff,r} h_r^2}{\sum_r k_{eff,r} h_r}. \quad (16)$$

The calculation is summarized in Table 1.

$$\sum_r k_{eff,r} h_r = 1968.93; \quad \sum_r k_{eff,r} h_r^2 = 696435;$$

$$z_{eq} = 353.71; \quad k_{eq} = 5.5665 \text{ mm}.$$

For the bolted connection

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

$$S_j = \frac{2.1 \times 10^5 \times 391.7^2}{2 \times 0.34631} = 4.6519 \times 10^{10} \text{ Nmm}.$$

The beam profile selected for a bolted connection, 406 × 178 × 54 UB has a moment of inertia $I_1 = 18.72 \times 10^7 \text{ mm}^4$. From (6) one obtains $M = 206 \text{ kNm}$, the required section modulus is $W_{1req} = 824 \text{ mm}^3$, the section modulus of the selected profile is $W_1 = 930 \text{ mm}^3$, and it is not necessary to follow the calculation with an iteration.

Check for shear: the actual shear force acting on the joint is $88 \times 3 = 264 \text{ kN}$, the shear capacity of a bolt-

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

row in tension is [10] 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

The costs considered in the calculation such as material, preparation (cutting, deslagging, chipping), welding and drilling costs have direct impact on the dimensions of the structure. Those costs, which impact less significantly, like transportation, surface cleaning, painting and moving elements are not considered here. Neither do we consider that part of the erection costs, which refers to the lifting and positioning of the elements, because these are very similar in both cases (welding, bolting). The other part of the erection, the assembly, in both cases has been considered in the cost calculation.

6.1 Welded connection (Fig. 5)

Material cost: according to [11] for $406 \times 178 \times 60$ UB the specific cost is 23 £/m. We calculate with a half beam length, 3 m, so $K_M = 23 \times 3 = 69.0$ £. (1 £ = 1.43 \$).

The fabrication cost is calculated as follows:

$$K_F = K_C = K_F = \left(\Theta_w \sqrt{\kappa \rho V} + 1.3 \sum_i \alpha_{p_i} C_{w_i} a_w^n L_{w_i} \right) \quad (17)$$

where K_C is the cost of cutting. According to [11] for a half beam it is $25/2$ £ = 12.5 £, $k_F = 25$ £/hr = 0.42 £/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), $\rho V = 60.1 \times 3 = 180.3$ kg is the mass of the half beam, the factor 1.3 expresses the effect of additional works (electrode change, chipping, deslagging), α_p is a coefficient expressing the effect of welding position, according to [11]. 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 [5, 14] according to COSTCOMP data [15, 16] for downhand welding position, a_w (mm) is the weld size. The first term in parentheses expresses the time of assembly and tacking: $2\sqrt{2 \times 180.3} = 38$ min.

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

Welds for two flanges, $a_w = 12.8$, $L_w = 2 \times 177.9$, single bevel butt welds, SMAW (shielded metal arc welding)

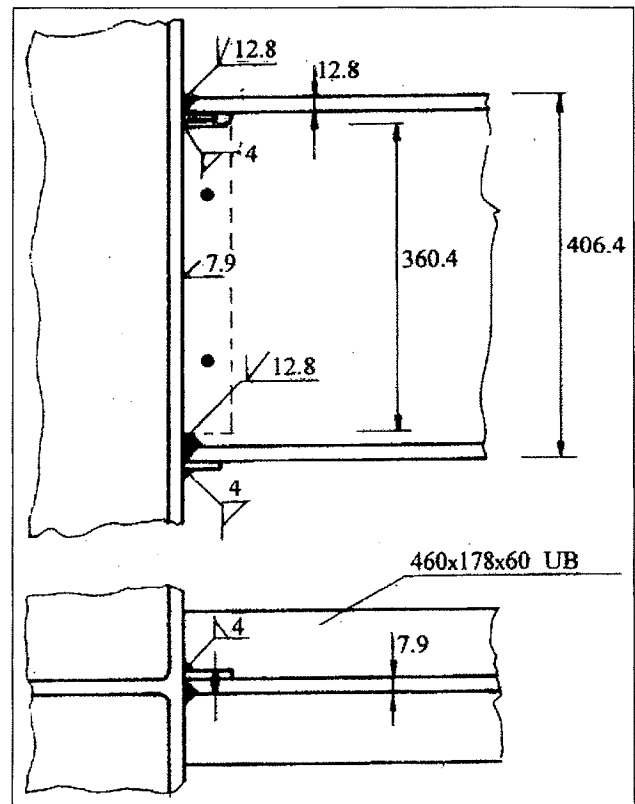


Fig. 5. Fully welded connection.

$$1.3 \times 0.5214 \times 10^{-3} \times 12.8^2 \times 2 \times 177.9 = 40.0 \text{ min.}$$

Weld for web, $a_w = 7.9$, $L_w = 360.4$, vertical position, single bevel butt weld, SMAW

$$2 \times 1.3 \times 0.5214 \times 10^{-3} \times 7.9^2 \times 360.4 = 30.5 \text{ min}$$

Welding of backing strips for two flange welds, overhead position, $a_w = 4$, $L_w = 2 \times 177.9$, SMAW fillet welds

$$3 \times 1.3 \times 0.7889 \times 10^{-3} \times 4^2 \times 2 \times 177.9 = 17.5 \text{ min.}$$

Welding of backing strip for web weld, vertical position, SMAW fillet weld, $a_w = 4$, $L_w = 360.4$

$$2 \times 1.3 \times 0.7889 \times 10^{-3} \times 4^2 \times 360.4 = 12.0 \text{ min.}$$

Altogether $K_F = 58 + 12.5 = 70.5$ £ and total cost is $K_M + K_F = 69 + 70.5 = 139.5$ £ = 200 \$.

6.2 Bolted connection

Material cost for $406 \times 178 \times 54$ UB is $21 \times 3 = 63.0$ £.

According to [11] the cutting cost for a half beam is $25/2 = 12.5$ £.

In [11] 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 $87 / 2 = 43.5$ £ for a beam end.

The total cost is 119.0 £ = 170 \$.

6.3 Comparison

The bolted version is 17% 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.

7.1 Welded connection

Using the material cost indicated, the cost of half a beam (3 m) 406 \times 78 \times 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 $\Theta_w \sqrt{k\rho V} + 1.3 \sum \alpha_{pi} C_{wi}^n L_{wi} = 2 (38 + 40 + 30.5 + 17.5 + 12) = 138$ min and the cost factor of 3 R/min, the welding cost equates to R 414. Thus the total cost of the welded joint is R 1225 (94 \$).

7.2 Bolted connection

For the bolted connection the material cost consists of $(3) \times (R 303.56) = R 728.54$ for the 406 \times 178 \times 54UB, $R 2026.81/30 = R 67.56$ for the endplate of 500 \times 200 \times 20 of which 30 can be cut from a plate and R 69.70 for 10 M20 bolts at R 6.97 each.

Fabrication cost is contributed by cutting of the plate, which constitutes roughly 8% of material cost, i.e. R5.40, drilling of 10mm holes at R5 each and welding of the end-plate to the beam. For single bevel butt welding of the flanges the welding time is given by $1.3 \times 0.5214 \times 10^{-3} \times 10.9 \times 10.9 \times 2 \times 177.9 = 28.6$ min. The time for the single bevel butt weld for the web is $1.3 \times 0.5214 \times 10^{-3} \times 7.7 \times 7.7 \times 360.4 = 14.5$ min. The time for assembly is $2 \times \sqrt{2 \times 54 \times 3} = 36$ min. At a rate of 3 R/min the total welding cost is $3 \times 79.1 = R237.31$. The total cost of a bolted joint thus amounts to R 1158 (89 \$).

7.3 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%.

8 CONCLUSIONS

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

The detailed cost calculations show that, in this numerical example, according to British data, the bolted connection is 17% 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. The calculation is

Table 2. South African cost data.

Item	Units	SA costs Dollar value	Reference
UB 406 \times 178 \times 60	amount/m	25.94	[17]
UB 406 \times 178 \times 54	amount/m	23.35	[17]
UC 305 \times 305v \times 97	amount/m	43	[17]
Plate 2.5 \times 1.2 \times 20	amount	194.89	[17]
Flat bar for back strips 40 \times 5	amount/m	0.62	[17]
Total Overhead, Labour, Consumables & Power: welding	amount/h	12.30 - 15.40	[18]
Wages for low skilled labourers	amount/h	3.85	[21]
Wages for high skill labourers	amount/h	10.00 - 15.00	[21]
Vertical: horizontal welding time		2:1	[18]
Overhead: horizontal welding time		3:1	[18]
Welding of truss columns and plates			
6 mm down fillet (factory conditions)	m/h/man	4	[18]
6mm down fillet (on site)	m/h/man	2	[18]
Cost of cutting plates:		0.08	[17]
Material cost (Alert Steel)			
Flame cutting (Shutcor)	amount/h	13.85	[22]
Drilling of M20 holes	amount/hole	0.23 - 0.54	[22, 23]
8.8grade M20 bolts	amount each	0.54	[20]
Mounting on site	amount/h	6.15 - 11.54	[19]

Table 3. Summary of costs in USD.

Joint	Item	Great Britain	%	South Africa	%
Welded	Material	99	49	62	66
	Manufacturing	101	51	32	34
	Total	200		94	
Bolted	Material	90	53	67	75
	Manufacturing	80	47	22	25
	Total	170		89	
Difference Bolted – welded	Material	-9	-10	5	7
	Manufacturing	-21	-26	-10	-45
	Total	-30	-17	-5	-6

very sensitive to the given data concerning the manufacturing times. These data are different in various companies and countries as well. The scatter can be relatively large between solutions, but making the calculation for a given frame using actual time and cost data, one can get the result and can choose the type of connection.

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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