CIRCULAR TUBE COLUMNS IN HIGH STRENGTH STEEL
Economical solutions for building frames

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INTRODUCTION

The steel industry is now able to produce structural members made of high strength steel grades up to S690/S700. However, the use of such members in constructions is still rather limited mainly because it is relatively difficult for a designer to easily identify the projects where there is an economical interest for using high strength steel for such members.

This paper presents economical solutions for building frames using high strength steel (HSS) circular tubes for columns in non-seismic area. Firstly, a global economic comparison between high strength steel and normal steel (NS) circular tube solutions to be used in steel or composite frames is presented with the objective of identifying typologies of buildings in which the use of high strength steel tubes has an economical interest. The comparison is based on an optimum design satisfying the strength, stability and stiffness requirements from Eurocode 3 and 4. It will be demonstrated that the greatest interest is observed for building frames using high strength steel tubes for the columns, with a large number of long spans and a limited number of storeys (as for commercial centres, parkings, airports, etc). Secondly, configurations for double T beam-to-circular column joints and column base joints are proposed for the considered frame typology and design guidelines founded on the component method are proposed for the mechanical characterisation of the proposed configurations

1 DOMAIN OF INTEREST FOR HIGH STRENGTH STEEL

A global comparison between solutions for steel and composite tubular columns using high strength steel (HSS) or normal steel (NS) and submitted to static loading has been carried out [1].The conducted research compared the costs between two columns made of HSS or NS to support the same level of loads. Steel with yield strengths varying from 500 N/mm^2 to 700 N/mm^2 have been considered as HSS while S355 steel has considered as NS. The strength, stability and stiffness conditions according to Part 1-1 of Eurocode 3 and Part 1-1 of Eurocode 4 have been taken into account in the optimum cost design for steel and composite columns. Concerning the analysis of structures made of HSS, the rules of Part 1-12 of Eurocode 3 have been used. Simple columns, columns in braced/un-braced frames and general frames has been investigated. In each case, algorithms of optimisations have been developed, allowing covering most of the design situation which can be met in practice. The following remarks were drawn from this study:

- In many cases, for steel columns, the use of HSS leads to considerable economical profit. As expected, the use of HSS in case of stocky columns provides the greatest advantage while NS is more economic in case of slender columns. Moreover, the interest of using HSS decreases when the eccentricity of the applied axial load increases.

- In braced/non-sway frames made of steel columns, the domain of interest of HSS is the greatest. In the case of unbraced/sway frames, the advantage of using HSS is quite low due to the imposed limitation in terms of horizontal displacements leading to a minimum inertia to be ensured by the column. From the conducted investigations, it can be concluded that the use of HSS for columns in sway frames has no economic interest, except in very few specific situations.

- For the composite columns, very few cases where the use of HSS is economical have been identified.

The interest can be quantitatively estimated from the charts given in [1].

2 CONSIDERED BUILDINGS AND DESIGN RECOMMENDATION

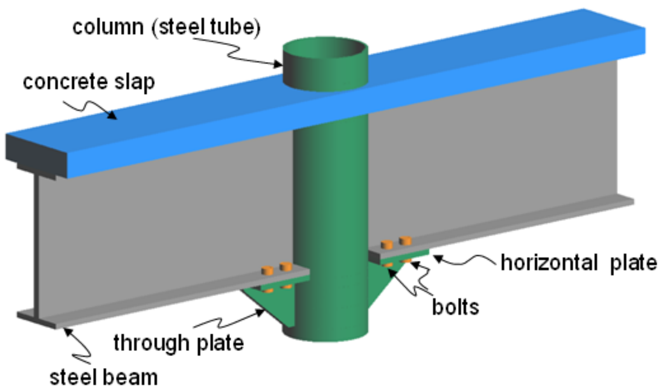
2.1 Building typology

From the remarks presented in Section 1, the typology of steel building where the use of HSS circular tubes for the columns can give an economic interest has been identified: building with a large number of long spans and a limited number of storeys, as for commercial centres, parkings, airports, etc. In these building, the vertical load is preponderant in comparison with the horizontal load. This is due to the geometry of the building, i.e. due to the fact that the height is small in comparison with the horizontal dimensions. So, the columns in these buildings are essentially subjected to compression loads and the influence of the bending moment is not significant. Also, the bending moments at the beam-to-connection are almost generated by the vertical loads.

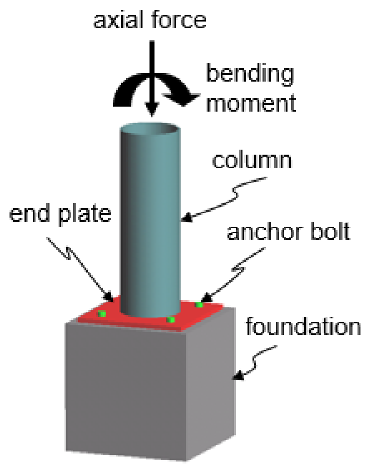
As far as the beam-to-column joints are concerned, the composite joint configuration shown in *Fig.1a* is proposed. In this joint configuration, one through-plate is welded to the column. On this plate two horizontal plates (each side of the column) are welded through fillet welds. The lower flanges of the steel beams are then connected to the horizontal plates with bolts while the upper flanges are connected to the concrete slab though shear connections. The outside part of the through-plate may be rectangular or triangular shapes. The rebars within the slab each side of the column act as a tensile component of the joint. Form the structural point of view, the joint is considered as a hinge during the construction phase (i.e. when the concrete is not yet cured) while, during the exploitation phase, the joint may be considered as a semi-rigid and partial strength composite joint. As in most practical cases, profiled steel sheets are recommended to use in the composite slab, so that during the concrete casting time the lateral stability of the steel beams can be assumed by these steel sheets.

With respect to the column base, it is proposed that the column bases should made of one end-plate welded to the column and connected to the concrete foundation through four anchor bolts (*Fig.1b*).

The suggested joint configurations can be considered as economic solutions in terms of used materials, fabrication and construction procedure on site.



a) beam-to-column joint



b) column base

Fig. 1: proposed beam-to-column joint and column base

2.2 Design recommendation

Global analysis

As the behaviour of the beam-to-column joint during the construction and exploitation phases is different, the global analysis of the considered frames should adopt the following assumptions:

- During the construction phase: the behaviour of beam-to-column joints can be considered as hinges. The column base behaviour may be considered as semi-rigid and partial strength. An elastic analysis for simple steel frames may be adopted.

- During the exploitation phase: both beam-to-column joints and column base has to be considered as semi-rigid and partial strength joints. Elastic or plastic analyses for semi-continuous composite frames can be applied.

Beam and column design

The design of the steel columns and the steel beams during the construction phase are covered by Eurocodes 3, part 1.1 for NS and Eurocode 3, part 1.12 for HSS, while the design of the composite beam during the exploitation phase is covered by Eurocode 4, part 1.1.

The bending moment at the joints during the exploitation phase may be used as a parameter to optimize the beam section. Indeed, the steel beam must resist to the construction load as simply supported as the joints are considered as hinges (*Fig. 2*) while, during the exploitation phase, the composite beam must resist to the exploitation load with possibilities of redistribution from the semi-rigid joints (*Fig. 2*).

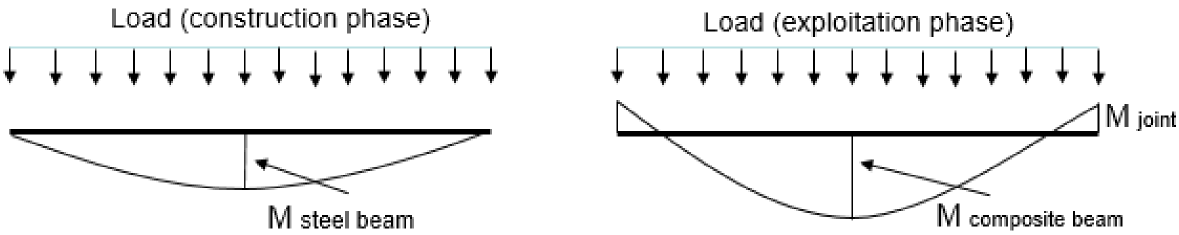


Fig. 2. Internal forces to design the beam

Beam-to-column design

As mentioned above, it may be considered that the joint is only subjected to hogging moment. By using the component method concept, the proposed joint can be decomposed into the following components:

- longitudinal slab reinforcement in tension;
- bolts in shear (bolt shank in shear and involved plates in bearing);
- welds in shear (through-plate to column and horizontal plates to through-plate welds);
- through-plate and column under diagonal compression (Fig. 3).

Methods for the characterizations of the “longitudinal slab reinforcement in tension” component can be found in the literature, e.g [2], while the “bolts in shear”, “plate in bearing” and “weld in shear” components are coved in Eurocode-3, part 1-8.

The “through-plate and column in diagonal compression” component is not yet directly covered in the current codes and literatures. An analytical method for the characterisation of this component has been developed and the obtained results have been validated through comparisons to numerical and experimental results. The details about this development are available in [3]. The design procedure for verifying the resistance of the through-plate components may be summarized as follow.

Step 1: determination of the applied loads on the connection, V_{Ed} and F_{Ed}. V_{Ed} is equal to the design value of the shear force at the beam end while F_{Ed} is equal to the ratio of the design bending moment at the connection to the height of the joint. The height of the joint is considered as the distance between the gravity centre of the rebars and the upper side of the through plate.

Step 2: calculation of the load direction, $\alpha = \arctan\dfrac{V_{Ed}}{F_{Ed}}$ (Fig. 3) and of the load parameters, q_s and q_i, by using Eq.(1).

$$q_s = \frac{4F_{Ed}}{h} - \frac{V_{Ed}(4b+2c)}{h^2}\,;\; q_i = \frac{V_{Ed}(4b+2c)}{h^2} - \frac{2F_{Ed}}{h} \tag{1}$$

Step 3: computation of the coefficients μ_1 and μ_2 from *Table 1* in which a linear interpolation can be applied for intermediate points.

Step 4: verification of the resistance of the through-plate by using the following expressions, Eqs.(2) and (3) for the outside and the inside parts respectively:

$$\frac{V_{Ed}}{t(b-c)} \leq \kappa\mu_1 \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2 / \gamma_M \quad (2)$$

$$\max\left(\left|\frac{4F_{Ed}}{th} - \frac{V_{Ed}(4b+2c)}{th^2}\right|, \left|\frac{V_{Ed}(4b+2c)}{th^2} - \frac{2F_{Ed}}{th}\right|\right) \leq \mu_2 \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{h}\right)^2 / \gamma_M \quad (3)$$

with γ_M, the partial safety factor (a value of 1.25 is recommended); κ, a coefficient taking into account of the shape of the outside plate (κ= 1.0 for a rectangular form and κ = 0.9 for a triangular form). The other parameters are defined in Fig. 3.

b is the width of the outside part;
c is the gap between the horizontal plate and the column face;
D is the outside diameter of the column;
E is Young modulus of the through-plate material;
F_{Ed} is the design value of the horizontal component of the load;
f_y is the yield strength of steel used for the through-plate;
h is the height of the through-plate;
t is the thickness of the through-plate;
V_{Ed} is the design value of the vertical component of the load;
α is the load direction;
ν is Poisson ratio of the through-plate material.

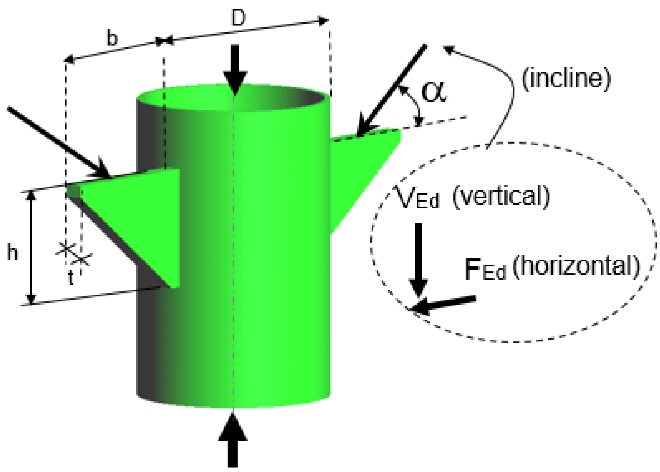


Fig.3. Notations for the through-plate component characterisation

Table 1. Buckling stress factors μ_1 and μ_2

Buckling stress factor μ_1

Geometries		Load direction (in degree)				
h/b	t/b	α=90	α=60	α=45	α=30	α=15
0,60	0,050	0,169	0,159	0,172	0,150	0,063
	0,075	0,103	0,089	0,087	0,084	0,040
	0,100	0,076	0,061	0,059	0,053	0,030
	0,125	0,066	0,048	0,043	0,038	0,021
	0,150	0,055	0,040	0,034	0,030	0,019
0,80	0,050	0,246	0,252	0,249	0,165	0,072
	0,075	0,147	0,134	0,131	0,108	0,047
	0,100	0,103	0,090	0,084	0,068	0,034
	0,125	0,080	0,067	0,062	0,049	0,026
	0,150	0,070	0,054	0,049	0,038	0,023
1,00	0,050	0,315	0,325	0,275	0,170	0,079
	0,075	0,182	0,176	0,164	0,120	0,053
	0,100	0,126	0,112	0,099	0,078	0,037
	0,125	0,091	0,081	0,074	0,059	0,029
	0,150	0,074	0,064	0,058	0,046	0,026
1,20	0,050	0,376	0,386	0,279	0,174	0,084
	0,075	0,204	0,204	0,170	0,123	0,057
	0,100	0,132	0,126	0,114	0,083	0,042
	0,125	0,096	0,089	0,081	0,064	0,033
	0,150	0,077	0,069	0,062	0,052	0,029
1,40	0,050	0,419	0,428	0,287	0,177	0,088
	0,075	0,222	0,254	0,179	0,125	0,058
	0,100	0,139	0,137	0,119	0,084	0,047
	0,125	0,099	0,094	0,086	0,066	0,035
	0,150	0,079	0,071	0,065	0,055	0,030

(*) type 1: q_s/q_i= -1; type 2: q_s/q_i= 0; q_s/q_i= 1 where q_s and q_i are given in Eq.(1).

Buckling stress factor μ_2

Geometries		Load type (*)		
D/h	t/h	1	2	3
1,00	0,050	1,123	1,079	0,748
	0,075	0,499	0,480	0,331
	0,100	0,281	0,270	0,187
	0,125	0,180	0,173	0,119
	0,150	0,125	0,120	0,083
1,50	0,050	1,147	0,984	0,748
	0,075	0,499	0,480	0,334
	0,100	0,281	0,270	0,187
	0,125	0,180	0,173	0,119
	0,150	0,125	0,120	0,083
2,00	0,050	1,096	0,892	0,641
	0,075	0,499	0,476	0,325
	0,100	0,281	0,270	0,187
	0,125	0,198	0,173	0,119
	0,150	0,125	0,120	0,083
2,50	0,050	1,045	0,847	0,514
	0,075	0,499	0,445	0,312
	0,100	0,281	0,270	0,187
	0,125	0,180	0,173	0,119
	0,150	0,125	0,120	0,083
3,00	0,050	0,993	0,662	0,386
	0,075	0,499	0,415	0,290
	0,100	0,281	0,263	0,184
	0,125	0,180	0,173	0,119
	0,150	0,125	0,120	0,083
3,50	0,050	0,869	0,414	0,232
	0,075	0,475	0,304	0,176
	0,100	0,281	0,217	0,129
	0,125	0,180	0,156	0,096
	0,150	0,125	0,115	0,072

Column base design

The application of the component method to the column base has been presented in many documents, e.g. [4, 5]. The two main components are the "concrete in compression" and the "end-plate in bending and bolt in tension". How to calculate the concrete in compression component for the column bases with circular tubular column have been presented in many literature, e.g. [6, 7] and they are not reported herein. On the other hand, the investigation on the "end-plate in bending and bolt in tension" component is aimed to determine the yielding mechanisms of the end-plate in bending and the bolts in tension. Beside the classic mechanisms that can be found in many literatures (*Fig. 4a* and *4b*), a new mechanism (*Fig. 4c*) is proposed and developed. The details about the developments are presented in [8].

The tension in the bolts may be computed using *Table 2*. The prying forces are not considered for the studied mechanism, as this effect is not observed in many experimental tests due to the important elongation of the anchor bolts (when they are not preloaded).

Table 2. Determination of the tension in the bolt, F_b (for one bolt)

Yield pattern in tension zone	Failure mode	Bolt force F_b (one bolt)
Circular (*Fig. 4a*)	Mode I- thin plate	$F_{b1} = 4\pi m_p$
Perpendicular (*Fig. 4b*)	Mode I – thin plate	$F_{b2} = \dfrac{b}{h - r - e_h} m_p$
Incline (*Fig. 4c*)	Mode I- thin plate	$F_{b3} = \dfrac{(\theta_{12} l_{12} + 0.5\theta_{23} l_{23}) m_p}{h + r - e_h}$
	Mode III– thick plate	$F_{b4} = B_p$

m_p is the unit plastic moment of the end plate ($m_p = 0.25 t_p^2 f_y$) with t_p and f_y the thickness and the yield strength of the end plate, respectively; B_p is the resistant plastic load per bolt: $B_p = A_s f_{yb}$ with A_s, the net tensile area of the bolt shank; $\theta_{12}, \theta_{23}, l_{12}, l_{23}$ are determined through Eqs. (4), (5), (6).

$$\theta_{12} = \frac{h + r - e_h}{\sqrt{(h - e_h)^2 + (b - e_b)^2} - r} \; ; \; \theta_{23} = \frac{2\tan\beta(1 + 1/\sin\alpha)}{\tan\alpha\tan\beta + \tan\beta/\cos\alpha - 1/\sin\alpha - 1} \tag{4}$$

$$l_{12} = l_{12,1} + l_{12,2} \text{ with } \left| \begin{array}{l} l_{12,1} = (b - r\cos\alpha)/\sin\alpha \text{ if } r(\tan\alpha + 1/\cos\alpha) \geq b \\ l_{12,1} = r(\tan\alpha + 1/\cos\alpha) \text{ if } r(\tan\alpha + 1/\cos\alpha) < b \\ l_{12,2} = r/\tan\alpha \text{ if } r/\sin\alpha \leq h \\ l_{12,2} = (h - r\sin\alpha)/\cos\alpha \text{ if } r/\sin\alpha > h \end{array} \right. ; \tag{5}$$

$$l_{23} = \left| \begin{array}{l} 0 \text{ if } r/\sin\alpha \geq h \\ h - r\sin\alpha \text{ if } \begin{cases} r/\sin\alpha \leq h \\ r(\tan\alpha + 1/\cos\alpha)\tan\beta \geq h + r \end{cases} \\ r(\tan\alpha + 1/\cos\alpha)\tan\beta - r(1 + 1/\sin\alpha) \text{ if } \begin{cases} r/\sin\alpha < h \\ r(\tan\alpha + 1/\cos\alpha)\tan\beta < h + r \end{cases} \end{array} \right. , \tag{6}$$

with $\alpha = \arctan\left(\frac{h-e_h}{b-e_b}\right)$ and $\beta = \arctan\left(\frac{h+r-e_h}{r(\tan\alpha + 1/\cos\alpha - (b-e_b))}\right)$; r is the outside diameter of the tube including the weld: $r = r_0 + 2(0{,}8a\sqrt{2})$ where r_o is the outside radius of the tube; the notations are shown in *Fig. 4d*.

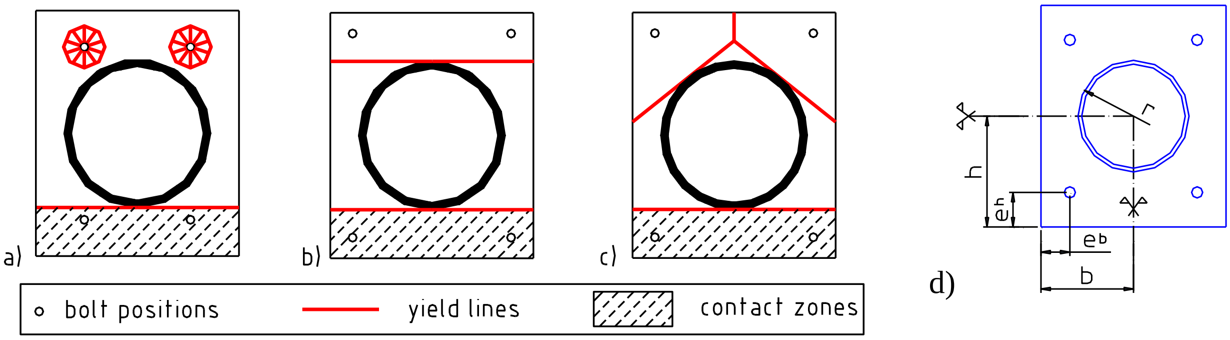


Fig. 4. Considered mechanism for the base plate and dimensional notations

3 CONCLUDING REMARKS

A structural typology for building, in non-seismic area, with details regarding the structural members and where the use of HSS circular tubes for the columns has an economic interest is proposed in this paper. The design procedure for the considered structure, from the global frame analysis to the structural elements/joints calculations, are presented. In particular, analytical models for some new components, the "through-plate and column under diagonal compression" (met in the proposed beam-to-column joint) and the "end-plate in bending and bolts in tension" (met in the proposed column base) are provided.

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