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A component-based model for innovative prefabricated beam-to-hybrid tubular column connections

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ABSTRACT

This paper presents the results of a component-based model developed for an innovative type of connection, recently proposed by the authors, for beam-to-hybrid fabricated column (HFC) connections. The active constitutive components of the connection are identified and their relevant resistance and flexural stiffness are presented. The moment-rotation curve of the joint is reproduced using the assembly of these components. The failure point of the connection is also predicted using the failure modes observed in the experimental tests. The results obtained from the developed component-based model are then compared to those provided by a three-dimensional (3-D) finite element (FE) model and experimental tests. It is shown that the component-based model shows good accuracy in the prediction of the initial stiffness, rotation capacity and failure of the connection.

1. Introduction

Of the different types of closed section columns, hybrid fabricated columns (HFCs) are a reliable alternative to conventional tubular box columns. HFCs (part 7 in Fig. 1) are composed of structural mild steel plates, which may be flat or corrugated, welded to hollow thin-walled tubes at two opposite edges. The concept of HFC was first introduced by Aoki [1]; however, since then it has been studied by many researchers [2–13]. HFC columns exhibit higher load-bearing capacity, and better post-buckling strength and energy absorption compared to the equivalent tubular columns [2–4]. Javidan et al. [2] have demonstrated that the capacity of hybrid fabricated columns is significantly higher than the corresponding conventional welded box column. For instance, when ultra-high strength tubes with external diameter of 76.1 mm and wall thickness of 3.2 mm are added to the corners of a 2-meter 210 mm × 210 mm × 3 mm welded box column, its axial capacity increases at least 10 times. The ductility of this HFC section is also 200% more than the welded box column. An HFC with high strength or ultra-high strength corner tubes can carry two to three times more axial load compared to an HFC with mild steel tubes, respectively [7]. Thus, considering the high capacity to weight ratio and economic benefits [2], the application of these columns in high-rise buildings sounds reasonable and feasible. The main obstacle to the broad usage of these columns in construction has been the issue of a lack of a robust connection, for which the authors of the present paper have recently proposed a solution (Fig. 1) that can be effectively used along with HFCs [14,15]. In spite of the complex look of the connection components, the manufacturing of the components is rather easy and quick. In the experimental tests conducted on the connection, the parts were machine cut from mild steel (Grade 250). However, the recommended manufacturing process for large-scale production is "casting". Having this perspective in mind, different components of the connection have been designed in a way that poses no difficulties in the casting process. Avoiding complex shapes or cavities, not having sharp edges or very thin features are among these measures. Therefore, achieving a consistent and production tolerance is possible. The design philosophy behind this connection is 'weak beam-strong column' which aims at shifting the failure from the column and connection towards the beam ends such that the connection is the latest component which may fail. In these research studies, the behaviour of an innovative modular connection under monotonic loading has been studied experimentally and numerically using finite element (FE) analysis. The advantage of using this connection over conventional connections has been numerically studied before by the authors in Ref. [15]. It was demonstrated that the conventional connections, namely flush and extended end-plate and reverse channel connections, cannot utilise the superior capacity of the corner tubes in HFCs and behave as a pinned or very close to a pinned connection according to Eurocode 3 classification. However, this innovative connection was able to take full advantage of the HFC capacity, and perform as a full-strength, semi-rigid or rigid connection

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Nomenclature

A_s	Thread area of bolt
A_{vc}	Shear area of the column
b_1	Width of bottom corner part
$b_{eff,ta}$	Effective width of angle in bending
b_{wx}	Thickness of web extension
D	Flexural rigidity of tube
D_{tb}	Tube external diameter
d_0	Diameter of bolt hole
d_{M16}	Diameter of M16 bolt
d_b	Bolt diameter
E	Modulus of elasticity
e_b	Distance of bolt line to free edge in the direction of applied load
e_p	Distance from bolt line to the free edge of T-stub
e_1	Distance from bolt line to the free edge of angle
e_2	Distance between bolt holes in top/seat angle
F_{Rd}^i	Component plastic resistance
F_{Rd}^{Tr}	Plastic resistance at bolt row r
F_{Total}	Total external force
$F_{at,Rd}$	Resistance of angle leg in tension
$F_{bs,Rd}$	Resistance of bolt row in shear
$F_{bt,Rd}$	Resistance of a single bolt
$F_{ctc,Rd}$	Resistance of corner tube in compression
$F_{ctt,Rd}$	Resistance of corner tube in tension
$F_{cw,Rd}$	Resistance of column web
$F_{pb,Rd}$	Resistance of plate in bending
$F_{tab,Rd}$	Resistance of angle in bending
$F_{wat,Rd}$	Resistance of web angle in tension
$F_{wxt,Rd}$	Resistance of web extension in bending
f	Summation of thickness and fillet size of web angle
f_{ub}	Ultimate strength of bolt
f_{up}	Ultimate strength of plate
f_{uw}	Ultimate strength of weld
f_{uwa}	Ultimate strength of web angle
f_{uwx}	Ultimate strength of web extension
f_{ya}	Yield strength of angle
f_{yb}	Design shear stress of the bolt
f_{ycw}	Yield strength of the column web
f_{ytb}	Yield strength of the tube material
g	Gap between the beam end and face of column/connection
g_1	Distance from the bolt hole centre to the face of beam on the outstanding leg of angle
g_a	Gauge length of the angle
H_1	Column height
H_{ep}	Height of bottom corner part
h_r	Component lever arm
h_{wx}	Height of web extension
h_t	Tension lever arm
I	Second moment of inertia of the angle leg
I_{wx}	Second moment of inertia of the cross section of web extension
K1, K2,…, K37	Component reference number in the model (Fig. 5)
K^c	Overall equivalent stiffness of the compression zone
K^t	Overall equivalent stiffness of the tension zone
k^i,k_j^i	Component extensional deformability
k_{at}	Stiffness of angle leg in tension
k_{bs}	Stiffness of bolt row in shear
k_{bt}	Stiffness of bolt in tension
k_{ctc}	Stiffness of corner tube in compression
k_{ctt}	Stiffness of corner tube in tension
k_{cw}	Stiffness of column web
k_{pb}	Stiffness of plate in bending
k_r^T	Total extensional deformability of row number r
k_{tab}	Stiffness of angle in bending
k_{wat}	Stiffness of web angle in tension
k_{wxb}	Stiffness of web extension in bending
L	Effective length measured along the mid-line of angle leg along the leg
L_a	Full length of the outstanding leg of angle
L_b	Grip length of bolt
L_{ta}	Length of top/seat angle
$L_{eff,a}$	Effective length of angle leg
$L_{eff,p}$	Effective length of plate
L_{cw}	Column faceplate width
L_{wa}	Length of the web angle along the length of column
L_{wx}	Length of the web extension
$M_{bp,Rd}$	Flexural resistance of plate
$M_{j,Rd}$	Flexural resistance of the joint
$M_{tab,Rd}$	Plastic moment of the angle leg
M_{Total}	Total external moment
M_u	Ultimate moment of connection
m_p	Distance from bolt line to the corner of T-stub
m_{ta}	Characteristic length of angle
n	Number of components in row r
n_b	Number of bolts
n_c	Number of bolt rows in compression zone
n_s	Number of shear planes passing through the bolt
n_t	Number of bolt rows in tension zone
P	Force
p_b	Spacing between bolts
r_w	Column faceplate weld radius
T_1	Column plate thickness
T_2	Tube thickness
t_a	Thickness of angle leg
t_{ep}	Thickness of bottom corner part
t_p	Plate thickness
t_{pb}	Thickness of the plates subjected to bolt shear force
t_{ta}	Angle thickness
t_{wa}	Thickness of web angle
$S_{j,ini}$	Initial rotational stiffness of the joint
V_{cws}	Design shear resistance of the panel zone
W_1	Width of column measured between tube axes
W_2	Depth of column measured between tube axes
w	Distance between internal bolt lines of angle
w_p	Vertical distance between bolt lines
Δ_{sh}	Deformation capacity of angle
α	Angle of engagement of corner tube in compression
γ_{M0}	Partial safety factor for design shear resistance of panel zone
γ_{M5}	Partial safety factor for resistance of corner tube in tension
γ_{Mb}	Partial safety factor for resistance of bolt in tension
ε_u	Ultimate tensile strain of material
θ_{Cd}	Rotation capacity of connection

which makes them a suitable choice for moment resisting frames. Additionally, unlike the conventional connections, this connection was able to rotate more than 0.04 rad which is required for the connections used in seismic applications [16]. The FE model, which was verified against the data extracted from the experimental tests, can accurately predict the overall behaviour of the connection [14]. However, the FE model is a full three-dimensional (3-D) representation of the joint, which is computationally expensive.

The behaviour of a connection is generally reflected in its moment-rotation (M-θ) curve (Fig. 2) by providing essential parameters such as initial stiffness ($S_{j,ini}$), moment resistance ($M_{j,Rd}$), and rotation capacity (θ_{Cd}). Many researchers try to relate these parameters to the mechanical