A SIMPLIFIED MODEL FOR MODELLING FLEXIBLE END-PLATE CONNECTIONS IN FIRE

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Abstract

In this paper a simplified robust 2-noded connection element has been developed for modelling the flexible end-plate connections at elevated temperatures. In this model, the two stage behaviours of flexible end-plate connection are considered. The model has the advantages of both the simple and component-based models. It is computationally efficient and has very good numerical stability under static solver conditions. A total of 14 tests are used to validate the model, demonstrating that this new connection model has the capability to accurately predict the behaviour of the flexible end-plate connections at elevated temperatures. The model can be used to simulate the flexible end-plate connections in real performance-based fire resistance design of steel-framed composite buildings.

Keywords: flexible end-plate connection, component-based model, fire resistance, steel structures.

INTRODUCTION

In recent years considerable research has been done to investigate the performance of steel-framed composite buildings under fire conditions. Current research indicates that the robustness of steel connections is vitally important to the fire resistance of composite buildings. Traditionally, the beam-to-column joints are assumed to be classified as ‘pinned’ or fully ‘rigid’. However, the true behaviour of joints could be classified as ‘semi-rigid’, which is in between the two extremes. A flexible end-plate connection has higher flexibility and larger rotational capacity compared with a flush and extended end-plate connection. Flexible end-plate connection comprises a rectangular plate symmetrically welded to the supported beam web, with the whole assembly then bolted to the supported column flange on site. Because of its simplicity and economy in both fabrication and assembly, flexible end-plate connection is of great popularity in the construction of braced multi-storey steel framed buildings.

Many laboratory tests have been conducted to understand the behaviour of beam-to-column connections at elevated temperatures (Leston-Jones 1997, Al-Jabri et al, 2005). Al-Jabri et al (2005) developed a component-based model to predict the behaviour of flexible end-plate connection under fire conditions. However, the model is only capable of predicting the behaviour of flexible end-plate connection before the beam bottom flange comes into contact with the column flange. In 2008, a series of experimental tests was conducted on the flexible end-plate connections at both ambient and elevated temperatures by Hu el at, (2009), who subsequently developed a component-based model to simulate the response of the flexible end-plate connections at elevated temperatures. In their model, the two stage behaviours of the flexible end-plate connection are taken into account.

The models described so far are component-based models (also known as spring-stiffness models). In these models the connection is regarded as a combination of several basic components. Each individual basic component has its own strength and stiffness characteristics in terms of tension, compression or shear. The overall behaviour of the connection is represented by combining these basic components. The major shortcoming of component-based models is that under a static solver the analysis terminates if one of the components of the connection fails due to numerical singularity. Dynamic solvers need to be used to overcome this problem, but the computational efficiency of the model is significantly reduced.

Huang (2011) has recently developed a robust 2-noded connection element for modelling the bolted flush and extended end-plate connection between steel beam and column at elevated temperatures. The model has good numerical stability under static solver condition. For this reason this model is employed here as the basis for the current development of a simplified model to simulate the behaviour of the flexible end-plate connections under fire conditions. The two stage behaviours of the flexible end-plate connections are considered in the model presented in this paper.

1 DEVELOPMENT OF THE NUMERICAL PROCEDURE

One of the main characteristic of the flexible end-plate connection is that its rotational response comprises two stages, as illustrated in Fig. 1. The first stage is the unimpeded rotation of the connection, and the second stage is when the bottom flange of the beam comes into contact with the flange of the column after sufficient rotation. These two stages comprise the moving of the compression centre of the connection from the end of the endplate to the centre of the beam bottom flange, which leads to an increase of moment resistance and rotational capacity.

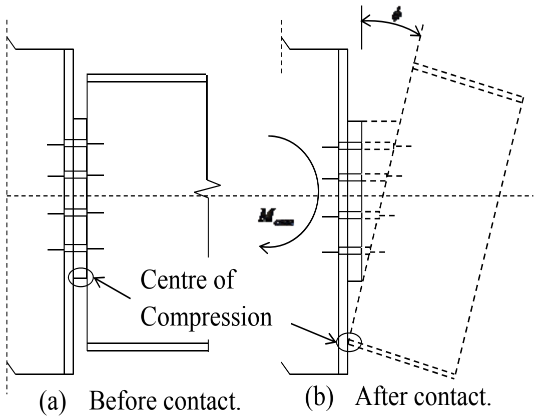

Fig. 1 Movement of centre of compression.

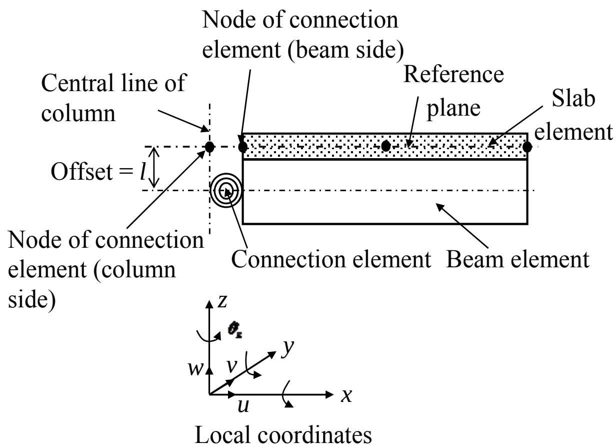

Fig. 2 Two-noded connection element.

These two stages are both taken into account in the model developed in this paper. The proposed numerical procedure is based on the main frame of Huang's two-noded connection element (Huang 2011) with the further developments to analyse the behaviour of the flexible end-plate connections at elevated temperatures. In Huang's original model, the connection is regarded as a two-noded element which has no physical length (see Fig. 2). Each node has six degrees of freedom, three translational degrees of freedom $u, v, w,$ and three rotational degrees of freedom $\theta_x, \theta_y, \theta_z$. Only the in-plane behaviour of the connection element is considered. In order to develop a numerical procedure for determining the bending moment characteristic of the flexible end-plate connection in fire, the connection is divided into tension and compression zones. The developed models for determining the tension, compression and moment resistances of the flexible end-plate connection are based on the component-based method. The moment-rotation characteristic of the connection element is represented as a multi-linear curve at each temperature level (see Fig. 3). In the figure, $S_{j,\text{int}}$ is the initial rotational stiffness of the connection element; ϕ_{Xd} is the rotation when moment of the connection reaches the moment resistance $M_{j,Rd}$; $\phi_{Contact}$ is the rotation when the bottom flange of beam contacts to the flange of column after sufficient rotation; $M_{Contact}$ is the bending moment of the connection when contact happens; ϕ_{Cd} is the maximum rotation of the connection. The rotation ϕ_{Id}, ϕ_{Xd}, $\phi_{Contact}$ are calculated as follows:

$$\phi_{Id} = \frac{2M_{j,Rd}}{3S_{j,\text{int}}} \tag{1}$$

$$\phi_{Xd} = \frac{2M_{j,Rd}}{S_{j,\text{int}}} \tag{2}$$

$$\phi_{Contact} = \frac{t_p}{0.5 * D_{beam} - d_{beam,plt}} \tag{3}$$

where t_p is the thickness of flexible end-plate, D_{beam} is the depth of beam, $d_{beam,plt}$ is the distance from the bottom flange of the beam to the end of endplate. Because the relative rotation between flexible endplate and the beam is small compared to the geometry of the connection, the value of rotation when contact occurs is calculated directly relate to the geometry of the flexible end-plate and the beam.

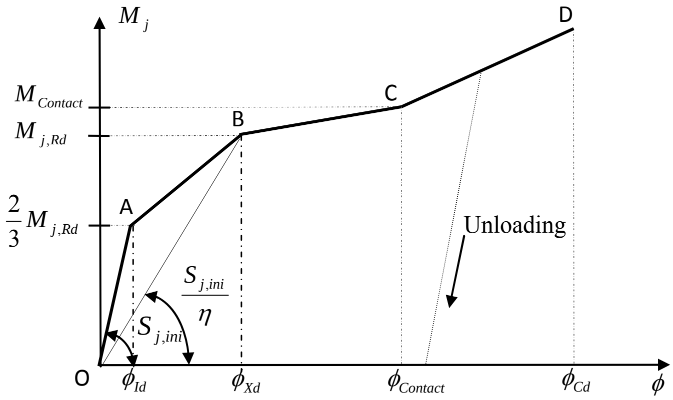

Fig. 3 Multi-linear moment-rotation curve used for the flexible end-plate connection.

Based on comprehensive numerical parametrical studies, the proposed moment-rotation characteristic of the flexible end-plate connection can be expressed as:

$$M_j = \begin{cases} S_{j,\text{int}}\phi & \phi \le \phi_{Id} \\ \dfrac{M_{j,Rd}}{3(\phi_{Xd} - \phi_{Id})} \times (\phi - \phi_{Id}) + \dfrac{2}{3}M_{j,Rd} & \phi_{Id} \le \phi \le \phi_{Xd} \\ 0.065 \times S_{j,\text{int}} \times (\phi - \phi_{Xd}) + M_{j,Rd} & \phi_{Xd} \le \phi \le \phi_{Contact} \\ \begin{pmatrix} 0.065 \times S_{j,\text{int}} \times (\phi_{Contact} - \phi_{Xd}) + M_{j,Rd} \\ + 0.15 \times S_{j,\text{int},II} \times (\phi - \phi_{Contact}) \end{pmatrix} & \phi_{Contact} \le \phi \le \phi_{Cd} \end{cases} \tag{4}$$

where $S_{j,\text{int},II}$ is the initial stiffness of the connection for the second stage. If the rotation of connection ϕ is larger than ϕ_{Cd}, it is assumed that the connection is broken, hence the bending moment M_j is assumed to be zero.

The initial stiffness of the connection $S_{j,\text{int}}$ is calculated based on the model developed by Al-Jabri et al (2005), as shown below:

$$\frac{1}{S_{j,\text{int}}} = \frac{1}{K_{eqt}z^2} + \frac{1}{K_c z^2} \tag{5}$$

where K_{eqt} is the equivalent tension stiffness, K_c is the compression stiffness, z is the equivalent level arm.

When the connection has more than one bolt row in tension, the level arm z is the distance from the centre of compression to the equivalent tension bolt row, which can be expressed as (Al-Jabri et al, 2005):

$$z = \frac{\sum_r (K_{tt,r} h_r^2)}{\sum_r (K_{tt,r} h_r)} \tag{6}$$

where $K_{tt,r}$ is the tension stiffness of each individual bolt row r, r is bolt row number, h_r is distance from bolt row r to the centre of compression. $K_{tt,r}$ is given as the combination of the stiffness of three components as follows (Hu el at, 2009):

$$\frac{1}{K_{tt,r}} = \frac{1}{K_{plt}} + \frac{1}{K_{weld}} + \frac{1}{N_{bt} K_{bt}} \tag{7}$$

where K_{plt}, K_{weld}, K_{bt} are the stiffness for the T-stub assembly, the weld and the bolt respectively. N_{bt} is the number of bolts in tension at a given bolt row. The detail formulation for calculating the stiffness of these three basic components can be found in the Reference (Hu el at, 2009).

In the tension zone, equivalent tension stiffness K_{eqt} represents the overall initial stiffness when the connection has more than one tension bolt row. This equivalent tension stiffness can be given from the following expression (Al-Jabri et al, 2005):

$$K_{eqt} = \frac{\sum_r (K_{tt,r} h_r)}{z} \tag{8}$$

When rotation of the connection $\phi < \phi_{Contact}$ the centre of compression is at the bottom end of endplate. However, when $\phi_{Contact} \le \phi \le \phi_{Cd}$, the bottom flange of the beam comes into contact with the column flange. At this stage, the centre of compression shifts to the centre of beam bottom flange, which leads to the increase of level arm z. Hence the initial stiffness $S_{j,\text{int},II}$ for the second stage is increased.

The tension capacity of each individual bolt row is calculated based on the component-based model proposed by Hu el at (2009). In the current model, the connection is considered as a combination of three basic components, which are the bolt, weld and a T-stub assembly comprising the endplate and the beam web. Each component has its own elastic stiffness and tension resistance. For the T-stub assembly, there are three different failure mechanisms. According to the experimental tests conducted by Hu el at (2009), the failure mechanism for the flexible end-plate connections is mainly the second failure mode. The second failure mechanism is that the T-stub suffers complete yielding, which has a first plastic hinge forming at the flange-to-web intersection and a second plastic hinge occurring in the bolt line. Therefore, in this model, the stiffness and the tension resistance capacity of the T-stub assembly is calculated based on the second failure mode.

For each individual tension bolt row, its tension resistance $F_{tens,r,bolt}$ can be determined as the minimum value of tension resistance of three basic components as below:

$$F_{tens,r,bolt} = \min(F_{plt}, F_{weld}, F_{bt}) \tag{9}$$

where F_{plt} , F_{weld} , F_{bt} are the tension capacity for the T-stub assembly, the weld and the bolt respectively. The detail formulation for determining them can be found in the Reference (Hu el at, 2009). For the compression zone, the rotational stiffness is taken as the value of stiffness of column web, which is calculated based on a simplified model proposed by Block et al (2007).

The moment resistance of a flexible end-plate connection can then be calculated as:

$$M_{j,Rd} = \sum_{r} h_r F_{tens,r,bolt} \tag{10}$$

where $F_{tens,r,bolt}$ is the tension resistance of bolt row r .

In this paper, steel material properties, such as yield strength; ultimate tensile strength and Young’s module, are temperature dependent. It is assumed that the material degradation of bolts at elevated temperatures is the same for the beam, column and endplate and the material model specified in Eurocode 3 Part 1.2 (2005) is adopted.

2 VALIDATIONS

In order to validate the model presented above 12 tests conducted by Hu et al (2009) have been used, 3 of these were at ambient temperature and 9 at elevated temperatures. In the validation the measured material properties and temperature distribution within the connections are used as input data for the model. The connections tested by Hu et al comprised of a 305x165x40UB beam connected to a 254x254x89UC column, with three M20 Grade 8.8 bolts. The thickness of partial endplate is 10mm. During the tests, the force was applied with inclined angle (θ) to the axis of the connected beam. Three angles, θ =35°, 45° and 55°, were employed. These three different angles represent three different combinations of vertical shear, axial tension. Three different temperatures of 450°C, 550°C and 650°C were also employed. According to the test data the two stage behaviours of the partial end-plate connection only occurred in the three ambient temperature tests. In the 9 tests at elevated temperatures the endplate ruptured before the bottom flange of the beam contacted the column.

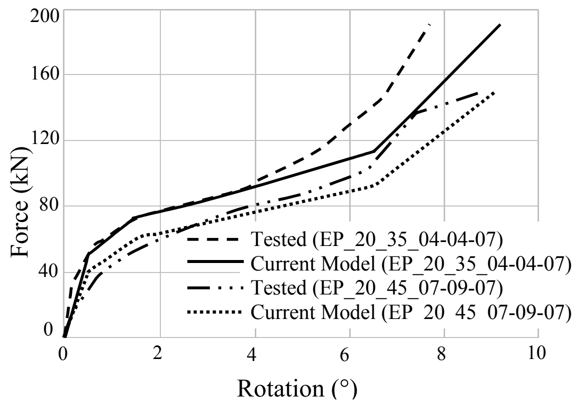

Fig. 4 Comparison results for Tests EP_20_35-04-04-07 and EP_20_45_07-09-07(Hu et al, 2009).

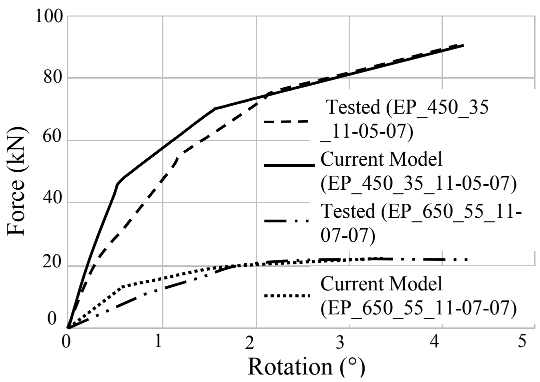

Fig. 5 Comparison results for Tests EP_450_35_11-05-07 and EP_650_55-11-07-07(Hu et al, 2009).

Due to space limitations only five tests are presented here for illustration. Fig. 4 shows a comparison between the results of two ambient tests. Tests of EP_20_35_04-04-07 and EP_20_45_07-09-07 were conducted with the force applied at an angle of 35° and 45°, respectively. In these figures it can be seen that the proposed model provides good agreement with the test data, suggesting that the model predicts the two stage behaviours of the connection well. Fig. 5 and Fig. 6 illustrate three comparisons between predicted and measured connection rotations at elevated temperatures. The tests of EP_450_35_11-05-07, EP_550_35_15-05-07 and EP_650_55-11-07-07 were conducted at 450°C (θ = 35°), 550°C (θ= 35°) and 650°C (θ = 55°), respectively. It is evident that the predicted results by the current model agree well with the tests data.

Two other fire tests conducted by Al-Jabri et al (2005) are also used for the validations. The tested connections consist of two 356x171x51UB beams symmetrically connected to a 254x254x89UC column with M20 Grade 8.8 bolts. The thickness of the flexible endplate is 8mm. The tests were conducted under a constant load with increased temperatures. Two different load levels of 8.2 kNm and 16.5 kNm were applied to the same connection. Fig. 7 shows the comparison of predicted and tested connection rotations for Test 1. It can be seen that the predictions of the current model are in good agreement with the test data.

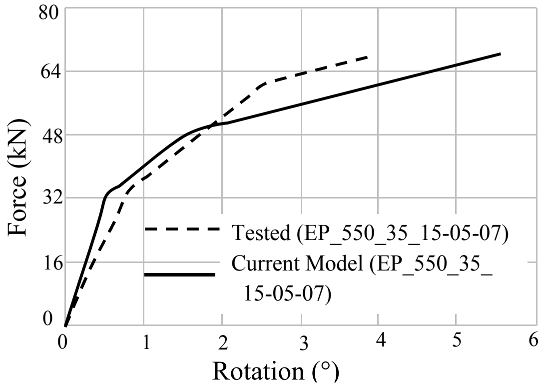

Fig. 6 Comparison results for test EP_550_35_15-05-07 (Hu et al, 2009).

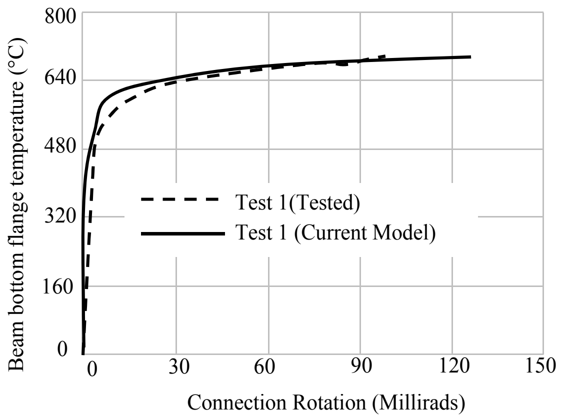

Fig. 7 Comparison results for test 1(Al-Jabri et al, 2005).

3 CONCLUSIONS

In this paper, a simplified model has been developed for modelling flexible end-plate connections at elevated temperature. The two stage behaviours of the flexible end-plate connection have been explicitly considered in this new model, and the capacities of tension, compression, bending, and the initial stiffness of each connection are all calculated based on the component-based approach. The developed model has very good numerical stability under static solver and it is very computationally efficient. A total of 14 tests was used to validate this model. The validation results show very good correlation between the predicted and tested results at both ambient and elevated temperatures. Hence, it is evident that this new connection model has the capability to accurately predict the behaviour of the flexible end-plate connections at elevated temperatures. The model can be used to simulate the flexible end-plate connections in real performance-based fire resistance design of steel framed composite buildings.

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