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SHEAR PANEL COMPONENT IN THE VICINITY OF BEAM-COLUMN CONNECTIONS IN FIRE

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Abstract. *This research is intended to predict the shear buckling of beam webs in the vicinity of beam-to-column connections in fire. A component-based analytical model, which simulates the behaviour of beam-web shear panels from the elastic stage to failure, has been created for elevated-temperature analysis. ABAQUS models have also been created to validate the component model over a range of geometries. Comparisons between the theoretical and FE models have shown that the proposed component model provides sufficient accuracy to be embodied in due course in global modelling of composite structures in fire. After sufficient validations have been carried out, the new component-based model will be implemented into the software Vulcan as a shear panel element adjacent to the existing connection element.*

1 INTRODUCTION

In fire scenarios, joints are among the key elements which can determine the potential for survival of a steel or composite framed building. The investigation of the "7 World Trade" [1] in the New York City indicated that the total collapse of the whole building was triggered by the fracture of beam-to-column joints, as a result of large thermal expansions of beams. The Cardington Fire Tests [2] indicated that the shear buckling of beams in the vicinity of beam-column joints is very prevalent under fire conditions, as indicated in Figure 1. This phenomenon could have significant effects on adjacent column-face joints. It could increase the transverse drift of a beam, as well as change the force distribution in the joints, leading to changes in their failure patterns. However, no practical research has been carried out to study the shear-buckling behaviour of Class 1 beams at elevated temperatures.

Figure 1. Shear buckling observed after one of the Cardington fire tests [2].

2 DEVELOPMENT OF THE THEORETICAL MODEL

In the proposed theoretical model, for Class 1 or 2 beams, shear response consists of three stages: the elastic, plastic and post-buckling stages. In the elastic stage, it is assumed that no buckling appears in the panel and the principal tensile and compressive stresses are identical. Plate buckling occurs during the plastic stage. After the buckling point, the shear panel enters the post-buckling stage. An example output is shown schematically in Figure 2. In this figure, Point 1 illustrates the end of the pre-buckling elastic stage. Point 2 refers to initial buckling point, and Point 3 represents failure (equivalent plastic strain reaches 0.15). The aim of this model is to produce a tri-linear force-displacement relationship for any shear panel, from initial loading to failure.

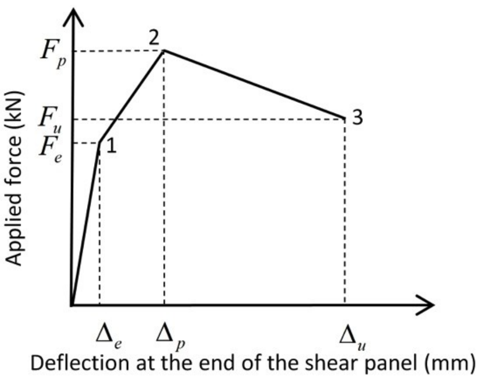

Figure 2. Tri-linear force-deflection relationship of shear panels.

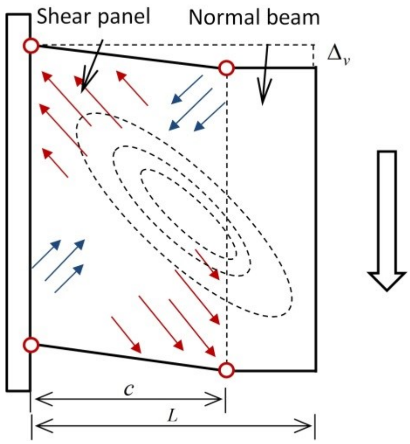

Figure 3. Theoretical model.

The calculation is based on the equality of the internal and external plastic work. The total internal work consists of the energy absorbed by the beam web and the four plastic hinges (indicated by the red circles in Figure 3) formed on the top and bottom flanges. The external work is that done by the movement of the external forces.

$$W_w + W_f = W_e \tag{1}$$

Where W_w is the internal work of the beam web,
W_f is the internal work of the flanges,
W_e is the external work.

It can be seen from Figure 3 that the vertical deflection Δ_v of the shear panel in the post-buckling stage is related to the distance c between the two plastic hinges on the same flange, but not to the total length of the buckle wave on the beam web L. Based on the assumption that failure occurs when equivalent plastic strain within the beam web reaches 0.15, the relationship between Δ_v and c can be found. The calculation procedure is firstly to find the value of c which corresponds to the smallest uniformly distributed load q, based on the principle of conservation of energy. The next step is to find the vertical deflection Δ_v of the shear panel in terms of c.

2.1 Internal work of beam flanges

In the theoretical model, the four edges of the shear panel are assumed to be rigid. The formation of plastic hinges on the flanges happens soon after the beam enters its plastic stage. Therefore, in the elastic stage, the internal work of the beam flanges has not been taken into consideration. In both the plastic and post-buckling stages, the internal work of the flanges is the work done by rotation of the four plastic hinges on the top and bottom flanges of the beam, as shown in Figure 3. The bending moment capacity of the flange plates, which is the basis of the plastic moment resistance of the four hinges, is

$$M_0 = \frac{1}{4} f_y b_f t_f^2 \tag{2}$$

A reduction factor α is introduced to account for the effect of overall cross-section bending, which induces net axial stress in flanges. The axial stress in turn reduces the bending moment capacity of the flanges. The reduction factor can be expressed as

$$\alpha = 1 - (\frac{\sigma_{t(c)}}{f_y})^2 \tag{3}$$

2.2 Internal work of beam web

It is initially assumed that the four edges of the shear panel are rigid. The panel is composed of tensile strips in the direction at 45° to the horizontal line, and compressive strips perpendicular to the tensile strips, as shown in Figure 3. The stresses within all the tensile strips are the same, as are the stresses within all the compressive strips. From the elastic stage to the initial buckling point, the compressive stresses are assumed to be identical to the tensile stresses. In the post-buckling stage, the compressive strips are considered as struts with three plastic hinges, as shown in Figure 4. It has been assumed that the middle plastic hinge always forms at the mid-length of each strut, although this assumption may lead to an out-of-plane deflection shape, which is slightly different from reality. Based on force equilibrium, as the out-of-plane deflection δ increases, compressive stresses decrease. Both force equilibrium and the geometric relationship determine that the reduced compressive stress σ_c is only related to the deflection level in the compressive strips. The tensile stresses within the shear panel are calculated based on the Huber-von Mises plasticity criterion [3]. The internal work of beam web is the sum of the work done by both the tensile and compressive strips.

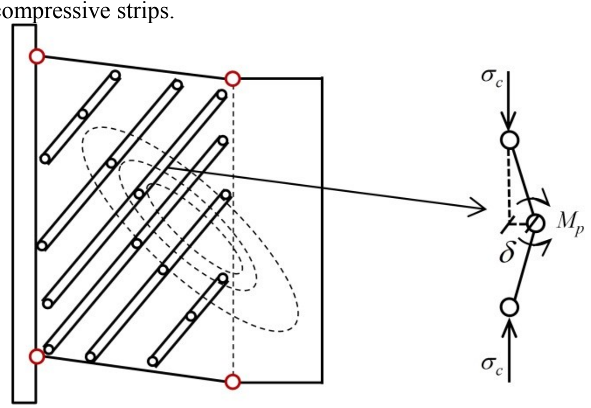

Figure 4. Struts representing compressive strips.

2.3 External work

Assuming that a beam is subject to a uniformly distributed load, the loading condition of the shear panel is as shown in Figure 5. The external work is, therefore, given as

$$W_e = q(l - \frac{1}{2}c)\Delta_v \tag{4}$$

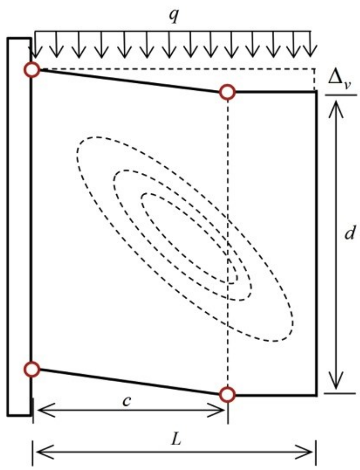

Figure 5. Displacement of the external load.

3 FINITE ELEMENT ANALYSIS USING ABAQUS

The proposed theoretical model has been verified against finite element (FE) modelling. Three-dimensional FE models of Class 1 beams were developed to analyse the shear panel behaviour at elevated temperatures. Temperature was uniformly distributed across the whole beam. The 3D shell element S4R of ABAQUS was adopted. This element is capable of simulating buckling. Riks analysis was used to track the descending load path of the shear panel in the post-buckling stage. A mesh sensitivity analysis was initially conducted, and an element size of 20mnm x 20mm was found to provide optimum accuracy and efficiency. An initial imperfection of amplitude $d/200$ (according to Eurocode 3 Part 1-5 [4]) was adopted. The shape of the initial imperfection was based on the first buckling mode analysis.

3.1 Material properties

The EC3 [5] stress-strain relationship has been used in the ABAQUS models, as shown in Figure 6. The reduction factors for the proportional limit $f_{p,\theta}$, yield strength $f_{y,\theta}$, and the slope of the linear elastic range $E_{a,\theta}$ were adopted. For all temperatures, the limiting strain $\varepsilon_{t,\theta}$ and the ultimate strain $\varepsilon_{u,\theta}$ are 0.15 and 0.2 respectively. Point 3 (which indicates failure) of the force-deflection relationship (as shown in Figure 2) of the web panel, was identified from ABAQUS at a maximum equivalent plastic strain of 0.15

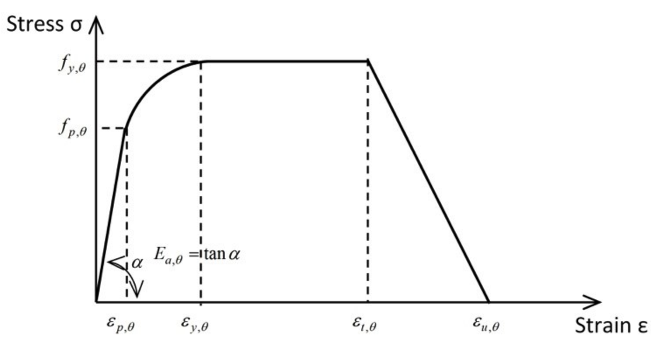

Figure 6. Stress-strain relationship of structural steel at high temperature [5].

3.2 Geometry of the beam

Figure 7 shows the finite element model of an isolated Class 1 beam. The beam length is 3m. The cross section dimension is as shown in Figure 8.

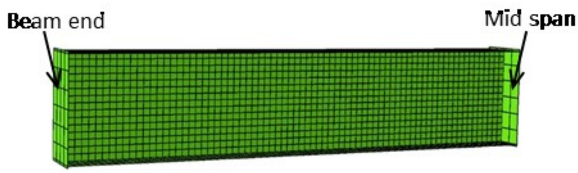

Figure 7. Image of finite element model.

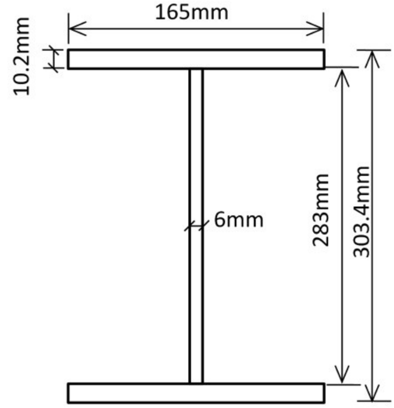

Figure 8. Cross section dimension

3.3 Boundary conditions

Because of its symmetry, only half of the beam was modelled. The beam was assumed to be fixed at both ends. Symmetric boundary conditions should be applied to the mid-span of the beam, so that the mid-span can only move vertically without any rotation. However, at high temperatures, axial force caused by restraint to thermal expansion cannot be neglected. As the influence of axial force has not been included in the proposed theoretical model so far, horizontal movement was allowed at the mid-span. Therefore, one end of the ABAQUS model, which simulates the mid-span of the beam, was allowed to move horizontally and vertically without rotation. Rigid bodies were attached to both ends of the model in order to avoid generating stress concentrations. The boundary conditions were then achieved by applying constraints to the mid-point of each rigid body. The boundary conditions are shown in Figure 9 and Table 1.

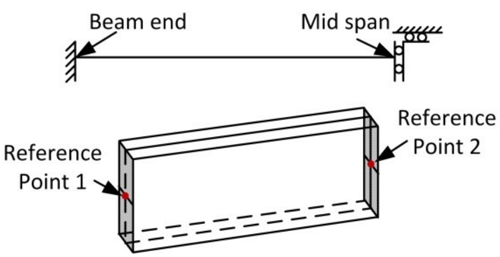

Figure 9. Boundary conditions.

Table 1. Boundary conditions

	Reference point 1	Reference point 2
U1	1	1
U2	1	0
U3	1	0
UR1	1	1
UR2	1	1
UR3	1	1

Note: U1, U2 and U3 are the translational degrees of freedom (DoF) in the *x, y* and *z* directions respectively. UR1, UR2 and UR3 are the rotational DoF in the *x, y* and *z* directions respectively. '0' indicates that a DoF is free; '1' means a DoF is restrained.

4 VALIDATION AGAINST FINITE ELEMENT MODELLING

The proposed theoretical model has been validated against the ABAQUS models, focusing on the beam-end reaction forces and the mid-span vertical deflections.

In the theoretical model, the beam-end reaction force is calculated on the basis of the design plastic shear resistance according to Eurocode 3 Part 1-1 [6] for Point 1 in Figure 2. In the design formula, the ambient-temperature yield stress has been replaced by the stresses at the proportional limits at elevated temperatures. Up to this point, the mid-span vertical deflection of a beam is assumed to be solely induced by bending moment. Point 2 represents the initiation of web buckling. The internal work done by both the beam web and flanges has been taken into consideration when calculating the beam-end reaction force for Point 2. It is also assumed that plastic hinges have been formed on the flanges at this stage, and the compressive stresses in the beam web have not been decreased due to the effect of buckling. The mid-span deflection for Point 2 is again assumed only to be caused by bending moment. For Point 3, the post-buckling strength reduction is accounted for by reducing the compressive stresses in the compressive strips. At this stage, the mid-span deflection is the summation of the transverse drift of the shear panel and that caused by curvatures due to bending moment.

Figures 10-12 show comparisons of the beam-end reaction forces and the mid-span vertical deflections from the theoretical and ABAQUS models. The length of the example models is 3m, and they were analysed at five different temperatures.

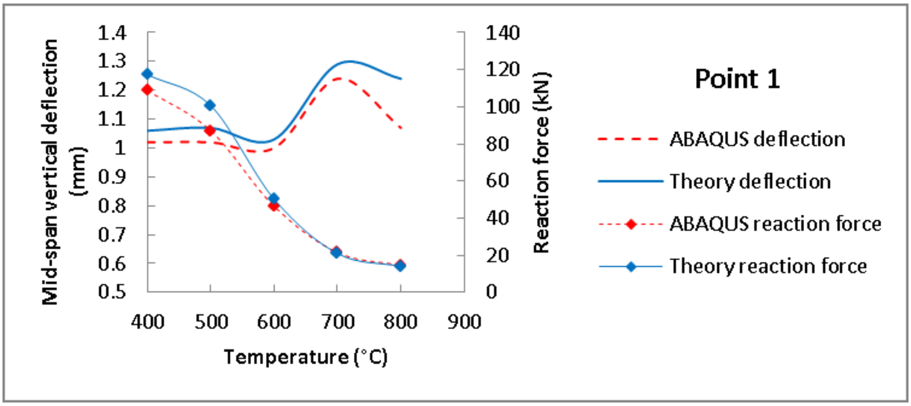

Figure 10. Comparison of mid-span deflections for Point 1 of Figure 2.

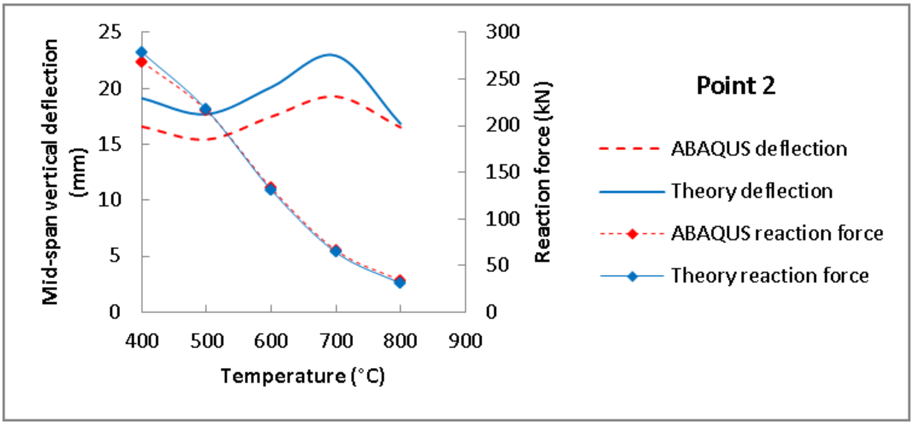

Figure 11. Comparison of mid-span deflections for Point 2 of Figure 2.

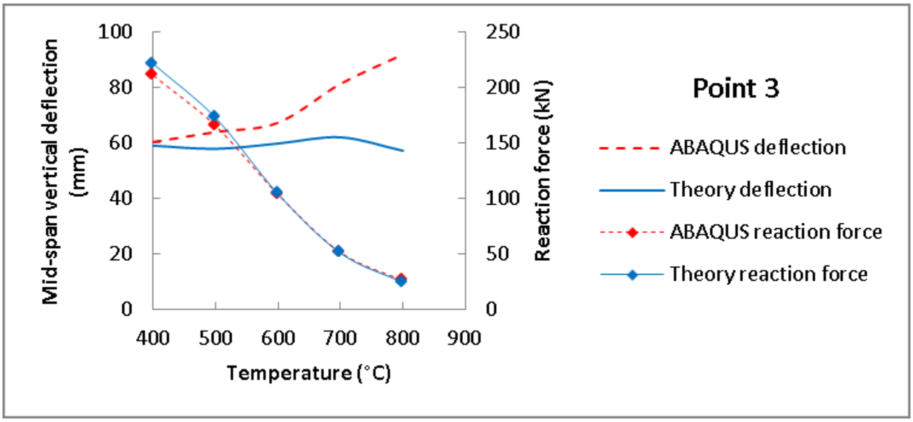

Figure 12. Comparison of mid-span deflections for Point 3 of Figure 2.

As shown in Figures 10-12, the beam-end reaction forces given by the theoretical model compare well with those of the ABAQUS model at all three stages. The comparison of mid-span deflections is acceptable for Points 1 and 2. The results from the theoretical model are always on the safe side. For Point 3, the deflections compare reasonably well for temperatures below 600°C. However, as temperature increases, the proposed theoretical model tends to underestimate the deflections at the post-buckling stage . There may be two reasons for this phenomenon. The first reason is that the structural system at high temperatures may have changed. Taking the mid-span vertical deflection due to bending moment as an example, the theoretical model assumes that a plastic hinge has formed at each end of the beam, as shown in Figure 13(a). However, as temperature increases, a larger portion of the beam end would become plastic, as captured by the FE modelling (Figure 13(b)). Therefore, the theoretical model may underestimate the vertical deflection caused by beam bending. As the theoretical component-based model will be implemented into the software *Vulcan*, this miscalculation of deflection will automatically be overcome, because *Vulcan* considers non-linear beam bending behaviour instead of the simple hand-calculation used in the proposed theoretical model. In addition, some of the assumptions of the theoretical model, such as the directions of strips and the positions of the plastic hinges on the compression strips, may influence the results. This will be investigated after implementing the model into *Vulcan*.

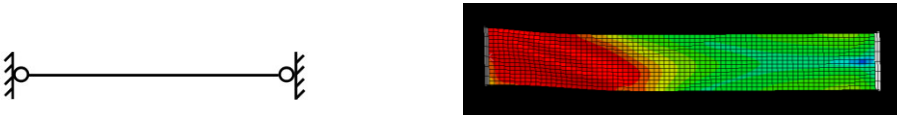
Figure 13. Beam model considering deflection caused by bending moment (a) theoretical model (b) Contours of Von mises stress in ABAQUS model.

5 COMPONENT-BASED MODEL IN *VULCAN*

Vulcan is a three-dimensional non-linear analysis program, which is capable of modelling the global 3-dimensional behaviour of composite steel-framed buildings under fire conditions. Based on the proposed theoretical model, a shear panel element will be created in *Vulcan,* adjacent to the existing component-based connection element [7], as shown in Figure 14.

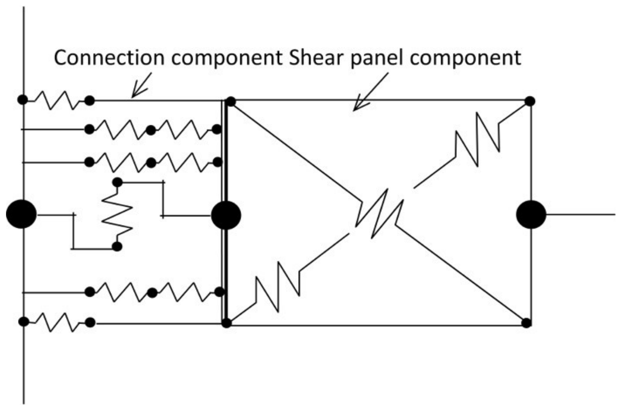

Figure 14. Component-based model including connection and shear panel.

6 CONCLUSION

A component-based theoretical model has been created to predict the shear capacity and vertical deflection of shear panels for Class 1 beams, up to failure at high temperatures. This theoretical model has been validated against FE models. Comparing the two models, the simplified theoretical one generally performs well. However, there is some discrepancy in mid-span deflection at the failure stage for temperatures higher than 600°C, which requires some further investigation. The theoretical component-based model will be implemented in the software *Vulcan*, and in due course to be embodied in global modelling of composite structures in fire.

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