# BENCHMARKING FOR THE INCLUSION OF SHEAR STUDS IN FINITE ELEMENT MODELS

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**Abstract**
There are many aspects of shear stud behaviour that may affect a heated steel-concrete composite structure in fire such as stud layout, ductility and strength; heated material behaviour; loss of composite action through the failure of multiple studs; and so on. This paper attempts to understand the role of shear studs on full structural behaviour in fire during both heating and cooling. Predictions of stud behaviour at ambient temperature using numerical models are first compared to experimental work to benchmark the modelling approach. A good correlation is found. This is followed by the analysis of full structural behaviour in fire where shear stud properties are varied parametrically. Individual shear studs are modelled so it is possible to identify which studs fail and at what point in the fire. The results demonstrate the importance of ensuring continued composite action in fire.

**Keywords:** Finite element modelling, connections, shear studs

## INTRODUCTION

Shear connectors in steel-concrete composite construction play a vital role in ensuring both strength and serviceability requirements are met at ambient temperature. Their behaviour and design has been extensively studied. In fire conditions it can expected that shear connectors play a similarly important role, yet their behaviour in fire has received very little attention. This is despite the many studies of composite structures in fire, both experimental and numerical, that have been undertaken over the last 15 years examining almost all other aspects of the behaviour of such structures.

This paper examines how shear connector behaviour may affect the response and strength of heated composite structures by means of a numerical parametric study using Abaqus, the commercial finite element package. Various models were produced which are discussed in detail below; in each case a concrete slab, modelled with 4-noded shell elements, was connected to steel beams, modelled with 2-noded beam elements, to simulate a portion of a steel-concrete composite structure. Steel behaviour was modelled with an elasto-plastic temperature dependent model assuming a von Mises yield criterion. Concrete was modelled using the “damaged-plasticity” model available in Abaqus. Geometric non-linear effects were accounted for. Because of the abrupt changes in stiffness that occur when shear studs fail, it was necessary to use an explicit dynamic solver for all analyses to obtain numerical convergence. This paper explores the effect of the degree of shear connection in fire on structural behaviour. Other parameters are consider by Anderson (2012).

## 1 AMBIENT TEMPERATURE BENCHMARK

In order to gain confidence in later results, an initial model was validated against experimental data. Experimental data on the behaviour of shear-studs in fire is rare so validation was first made against ambient temperature data produced by Chapman and Balakrishnan (1964).

Chapman’s experiments consisted of concrete slab strips attached by shear studs to steel beams. The slabs were loaded uniformly and deflections measured. A span of 5.5 m with simple supports was used. The steel beams were 305 mm deep and 152 mm wide, with a flange thickness of 18 mm and web thickness of 10 mm. The slab was 152 mm deep and 1.22 m wide. It had 4, 8 mm Φ bars top and bottom in the longitudinal direction and 12.7 mm Φ bars in the transverse direction at 152 mm spacing at the top and 305 mm spacing at the bottom. Further details of test arrangements are given in Tab. 1.

Tab. 1. Material properties used for validating numerical models against experimental data; starred values are assumed data; all others are taken from Chapman Balakrishnan (1964)

| **Property** | **Steel** | **Concrete** | **Reinforcement** |
|---|---|---|---|
| Compressive strength (MPa) | 240 | 50 | 500* |
| Tensile strength (MPa) | 240 | 5 | 500* |
| Young's modulus (GPa) | 210 | 26.7 | 210* |
| Strain at peak stress (-) | NA | 0.003 | NA |
| Strain at failure (-) | NA | 0.0045 | NA |
| Poisson's ration | 0.3* | 0.2 | 0.3* |

Chapman's test "U3" was used for validation. This test was chosen because it most closely reflects a realistic building design and loading scenario. It used T-studs (a form of shear connector commonly used in current construction) evenly spaced at 216 mm centres. The load-slip behaviour of the studs obtained from push-out tests by Chapman is shown in Fig. 2
The numerical model used for validation is shown in Fig 1. In this model, friction between the steel and concrete was included with a coefficient of friction of 0.5, as recommended in EC4 (2004), while a contact condition was specified that prevented the steel penetrating the concrete but left separation free to occur. Twenty-four pairs of shear studs connecting the steel and concrete were modelled explicitly using individual connector elements. These elements were specified rigid in all direction except parallel to the beam axis where the force-displacement relationship followed that shown in Fig. 2. Chapman's test data showed shear studs failed on average at a load of 120 kN with a deformation of 2.54 mm. This failure condition was included in the numerical model. Pinned-boundary conditions were specified at each end of the beam. These provided rather more axial restraint than was present in reality but it was found that assuming no axial restraint (simple supports) produced poorer correlation with test data. A spring support would have been most accurate but the results obtain (Fig. 3) are sufficiently good for the modelling approach used to represent the shear studs to be considered validated at ambient temperature.

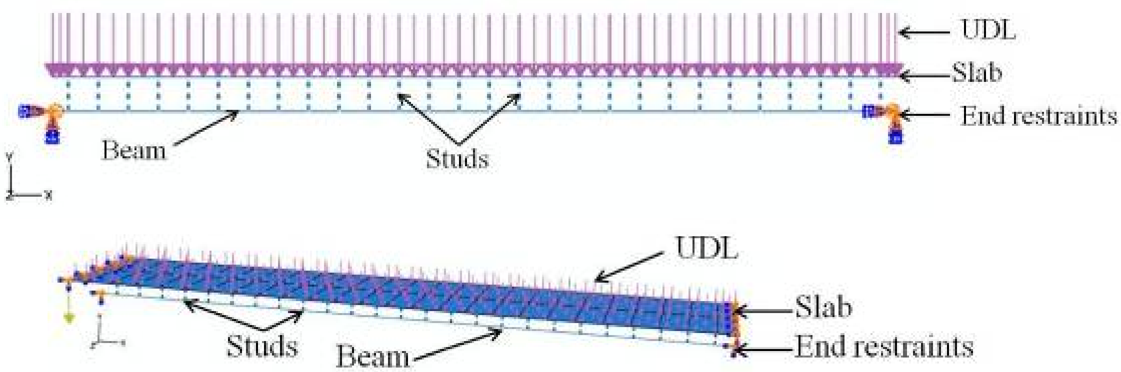



Fig. 1 Numerical model used for benchmarking model

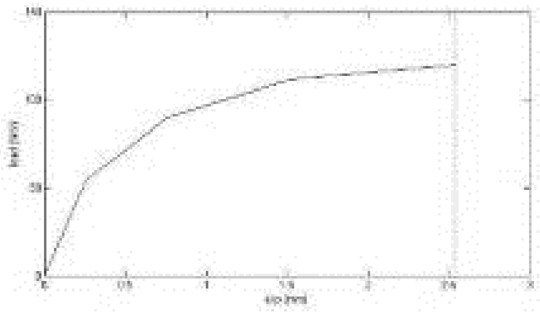


Fig. 2 Shear stud load slip behaviour from Chapman and Balakrishnan (1964)

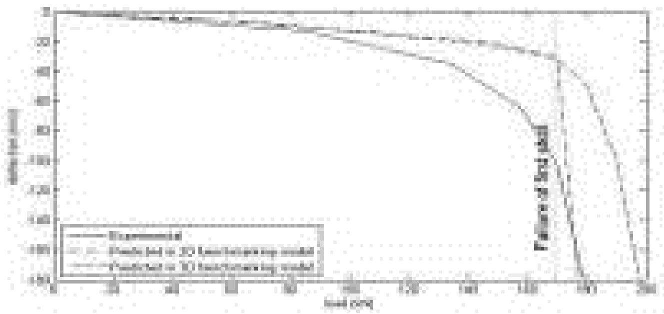


Fig. 3 Load-deflection response of 2D and 3D (discussed here) benchmark models

## 2 NUMERICAL MODEL

With an ambient temperature model validated, a standard model was developed for a parametric study of shear stud behaviour at elevated temperatures. This paper presents only the results from the connectivity study although; other parameters have been considered in Anderson (2012). Full connectivity is defined as the case where the steel or concrete fails before the studs while partial connectivity is the converse. Connectivity therefore relates to the strength of the studs.

### 2.1 Geometry

A 6m by 6m slab has been chosen to represent an average room size. A total imposed load of 5 kN/m$^2$ is assumed. Together with the dead load of the slab, a beam size of UB533x210x92 was chosen and a slab depth of 150 mm. Reinforcement bars are provided at 150 centres top and bottom and in both directions, 12 mm Φ bars in the bottom and 8 mm Φ bars at the top.
Shear studs are explicitly included and their spacing is calculated based on the distance between ribs in the profiled steel decking under the concrete slab. This distance can vary between around 150 m – 300 mm and so an arbitrary figure of 200mm was chosen in this case, (Kingspan, 2009). The stud spacing is the assumed to be equal to this distance, (Quiroz, 2009). The studs at each end of the slab were placed at half this distance from the support meaning there are a total of 30 studs in this case. Load-slip behaviour, based on the experimental curve shown in Fig. 2 is defined in the direction of the beam main axis while movement in any other direction is restrained.
Boundary conditions that pinned the ends of the beam and slab were imposed. Symmetry was used at the two edges of the slab parallel to the beam to model continuity.

### 2.2 Heating

A parametric fire was assumed to heat the whole structure. A maximum gas temperature of 895 °C was achieved after 60 minutes and gas temperature returned to ambient after a total time of 190 minutes, 130 minutes after the peak temperature. Heat transfer calculations were carried out to calculate the temperature of the slab at 5 points throughout its depth while the steel beam was assumed to have a uniform temperature. The slab took around 19 hours to return to ambient temperature whereas the beam cooled in about 4 hours.

## 3 PARAMETRIC STUDY

### 3.1 Partial Connectivity

When the strength of an individual shear stud and its spacing along the beam is known, the degree of shear resistance can be calculated. Full shear resistance is not required from the studs and a minimum requirement can also be calculated. The degree of shear resistance can be calculated as follows according to Eurocode 4 as:

$$\eta = \frac{N_c}{N_{c,f}} \tag{1}$$

where $N_c = 0.5nP_{Rd}$ and $N_{c,f} = 0.85A_c F_{cd}$ (2) / (3)

In the above equations $N_c$ is the design value of the compressive normal force in the concrete flange, $N_{c,f}$ is the design value of the compressive normal force in the concrete flange with full shear connection, n is the number of shear studs along the length of the beam, $P_{Rd}$ is the design shear resistance of a shear stud, $A_c$ is the cross sectional area of the concrete and $F_{cd}$ is the characteristic design strength of the concrete. The minimum degree of shear resistance required to meet the Eurocode recommendations can be calculated as:

$$\eta_{\min} = 1 - \left(\frac{355}{f_y}\right)(0.75 - 0.03L_e) \tag{4}$$

where $L_e$ is the effective length and $f_y$ is the yield strength of steel.
In the benchmark model, the capacity of a shear stud was 120 kN. The level of connectivity was increased or decreased in this study to evaluate the effects on time to stud failure and slab deflection. Stud failure forces of 60 kN, 90 kN, 120 kN, 150 kN and 180 kN were chosen for the study. Degree of connectivity and minimum shear requirement is summarised in Tab. 2.

Tab. 2 Shear connectivity provided based on strength of studs

| **Required degree of shear connection** |
|---|
| 43% |

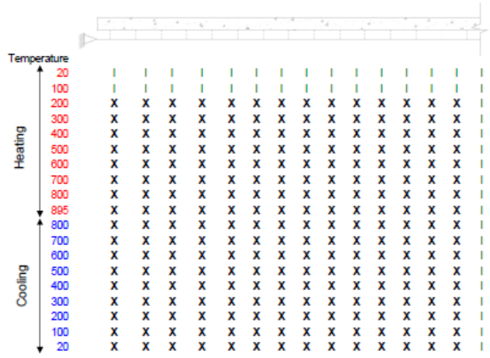
| **Stud Shear strength** | 60kN | 90kN | 120kN | 150kN | 180kN |
|---|---|---|---|---|---|
| **Actual degree of shear connection** | 35% | 52% | 69% | 86% | 103% |

### 3.2 Results

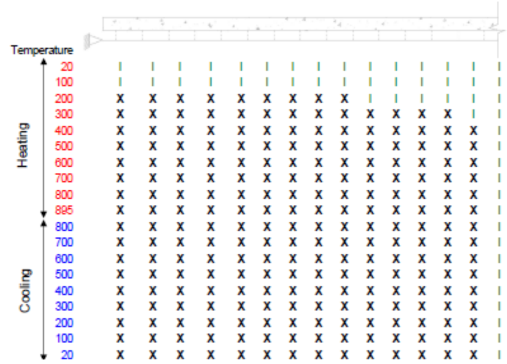
Fig. 4 show the failure temperatures of each shear stud over half the length of the beam for the five analyses. At the top of the diagrams is the layout of the slab and beam assembly where the shear studs are explicitly indicated by a dashed line. On the left hand side is the gas temperature throughout the analysis and adjacent to that the status of each shear stud is given: a line '|' denotes that the shear stud is still intact and a cross, 'x' indicates failure.

The following conclusions can be drawn from these figures:

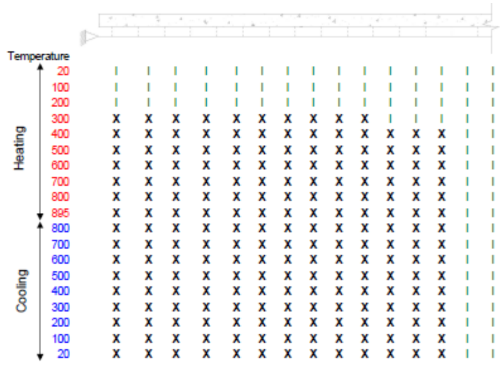
- As the level of connectivity increases, the shear stud failure temperature increases for the studs nearer the centre. This is because the studs near the centre of the beam carry less force and therefore will be the last to fail.
- With increasing connectivity, fewer studs fail over the duration of the analyses
- Most studs that fail do so in heating however there are a few that fail in cooling in the 180 kN model.
- The stud to the far right of each figure is 100 mm from the middle of the beam and as the analysis is symmetrical and ideal, the middle shear stud should not be subject to a significant shear force throughout the analysis. This is highlighted in the fact that this stud does not fail in any of the analyses.



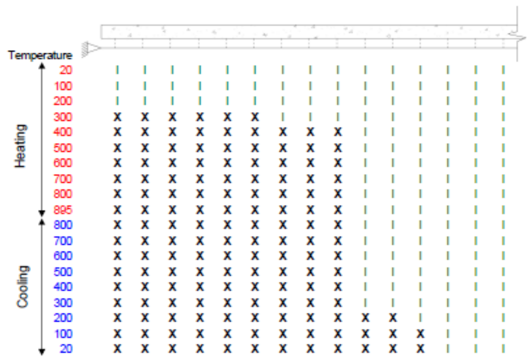

(a) 60 kN and 90 kN




(b) 120 kN




(c) 150 kN




(d) 180 kN

Fig. 4 Failure temperatures of shear studs for different stud capacities

Stud strength also has a marked effect on beam and slab deflection as can be seen from Fig. 5. The beam mid-span deflections increase as the stud strength decreases.

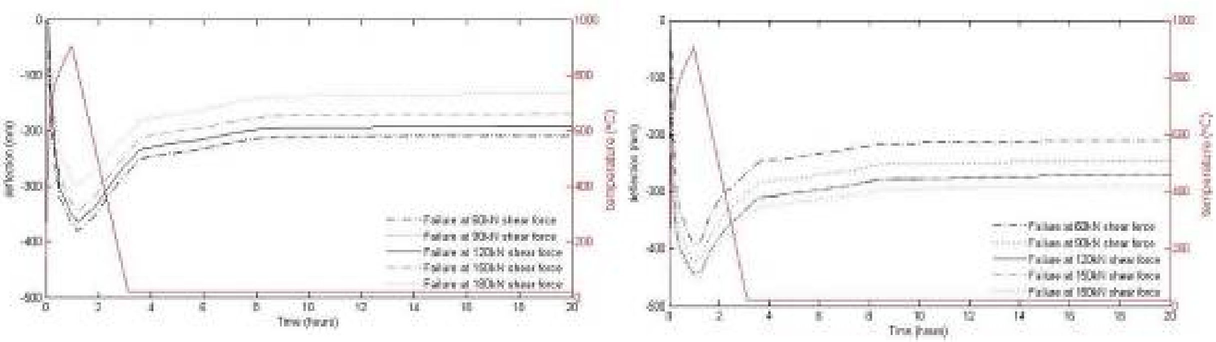
- In the model with the highest level of connectivity, with a stud failure force 180 kN, the maximum deflection during heating is 300 mm.
- In the model with the lowest level of connectivity, where the stud failure force is 60 kN, the maximum deflection during heating is 380 mm.
- The maximum deflection for the model with the lowest connectivity is therefore 25 % higher than that for the model with the highest level of connectivity.
- In cooling, the 180 kN failure model has a residual deflection of 135 mm.
- This is compared to 210 mm for the 60 kN model.
- The residual deflection for the model with the weakest studs is therefore larger by 55 %.

As the structure begins to heat, deflections increase and the slip between the slab and beams also increases. As studs begin to fail at the edges of the beams, where the largest shear forces are, larger rotations are possible and therefore in the models where more shear studs fail, i.e. those with lower stud strength, there will be a larger mid-span deflection. Again, the opposite pattern is seen in the slab deflections: as the stud strength increases, so do the slab edge mid-span deflections, Fig. 5.

- In the model with the highest level of connectivity, where the stud fails at 180 kN, the maximum deflection during heating is 450 mm.
- In the model with the lowest level of connectivity, the maximum deflection during heating is 380 mm.
    - The maximum deflection for the 180 kN failure force model is therefore 20 % larger than that for the 60 kN failure force model.
- In cooling, the 180 kN failure force model has a residual deflection of 295 mm.
- This is compared to 210 mm for the 60 kN failure force model.

o The residual deflection for the model with the highest level of connectivity is therefore 40 % larger than that with the least connectivity.

By considering the relative expansion rates of the beam and slab this can be explained. In each model, the total thermal expansion of both the beam and slab will be the same. As the failure force of the studs increases, the slab deflection over the beam decreases, as explained above. For the total thermal expansion of the concrete to be the same in all models, this will require the concrete expansion to be more prevalent elsewhere in the model. It is therefore evident as a vertical deflection at the slab edges.



(a) beam midspan (b) slab midpoint

Fig. 5 Deflections at the mid-point of the slab of beam for different shear stud strengths

## 4 CONCLUSIONS

This study shows that shear studs do effect the predicted response of composite structures in fire. Depending on the design criteria, it may that the degree of shear connectivity appropriate at ambient temperature is not sufficient for high temperature. Therefore, consideration should be given to checking shear stud adequacy at in structural fire design, rather than, as is currently common, simply assuming shear studes will be adequate in the fire limit state.


## 5 ACKNOWLEDGEMENTS

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