Steel and Composite Structures, *Vol. 20, No. 1 (2016) 185-203*
DOI: http://dx.doi.org/10.12989/scs.2016.20.1.185

Performance of shear connectors at elevated temperatures – A review

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(*Received February 26, 2015, Revised May 11, 2015, Accepted July 20, 2015*)

Abstract. Shear connectors are key components to ensure the efficient composite action and satisfactory transfer of shear forces at the steel–concrete interface in composite beams. Under hazardous circumstances, such as fire in a building, the performance of a composite beam significantly relies on the performance of shear connectors. Studies on the behavior of shear connectors subjected to elevated temperatures performed in the last decade are reviewed in this paper. The experimental testing of push-out specimens, the design approaches provided by researchers and different codes, the major failure modes, and the finite element modeling of shear connectors are highlighted. The critical research review showed that the strength of a shear connector decreases proportionally with the increase in temperature. Compared with the volume of work published on shear connectors at ambient temperatures, a few studies on the behavior of shear connectors under fire have been conducted. Several areas where additional research is needed are also identified in this paper.

Keywords: high temperatures; shear connectors; load-slip; composite beam; push-out

1. Introduction

Composite members in which reinforced concrete decks are positioned on steel girders have been utilized for a long time in civil engineering construction. A composite structure requires significant consideration of the design process under hazardous conditions. For example, the behavior of a composite structural system under the condition of fire is considerably different from its behavior at ambient temperature.

The investigation performed on the Broadgate fire and Cardington structure (O'Connor and Martin 1998) has improved the understanding of structural interactions and load distribution that occur in a real building under fire. The Cardington structure was a typical example of current UK steel construction with a concrete-profiled deck floor slab acting compositely with hot-rolled steel beams, which were mostly left unprotected. Tests showed that high temperatures could be sustained; however, the test results on their own are insufficient to provide a full explanation of the mechanics that governs the response of such structures to fire. The interactions among thermal expansion, large deformation effects, material degradation, and 3D effects on a building are the

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http://www.techno-press.org/?journal=scs&subpage=8 ISSN: 1229-9367 (Print), 1598-6233 (Online)

reasons that lead to the complicated behavior of composite members. This complicated behavior can only be understood if high-quality numerical and analytical models are developed and investigated. Several researchers have focused on the behavior of isolated members in composite structures. Sanada *et al*. (2000) studied the structural action in a two-way slab and composite beam structure subjected to compartment fire. Their findings showed that the performance of indeterminate structures under fire is governed by thermal expansion, and local yielding and large deflections minimize the damage to the complete structure. Mäkeläinen and Ma (2000) investigated the thermal and structural performances of a slim floor beam under fire by using numerical analysis programs and applying both International Organization for Standardization (ISO) standard fire and natural fire. Their results showed that the increase in temperature significantly affects the flexural capacity of the beam, and this type of beam significantly depends on shear connection to maintain its performance.

Rose *et al*. (1998) showed that the performance of composite slabs also significantly depends on the performance of shear connection between elements.

Thus, to obtain an error-less structural performance of composite beams, care should be taken to ensure an efficient shear transfer between the steel and concrete parts. The key point in the analysis and optimization of composite structures is therefore the prediction of joint strength and its influence on global stability.

In most composite beams, shear connection is provided by welding a steel member to the upper flange of the steel beam and surrounded in the reinforced concrete as shown in Fig. 1.

These members transfer the forces between the steel girder and connector by shear and between the connector and concrete by bearing (Viest *et al*. 1997). The mechanical interlocking system in the deck profile provides resistance to the vertical separation and horizontal slippage between steel and concrete. A beneficial composite behavior can be achieved by minimizing the displacement of the concrete slab and steel beam at their edge. This composite action is usually assured by the shear connectors.

The behavior of the shear connectors depends on the relationship between the amount of transmitted shear force and the degree of slippage at the edge of the concrete and steel. If this slippage does not occur, both the steel and concrete work in full interaction. In the absence of shear connectors, the members that constitute the composite structure to move freely. Each element

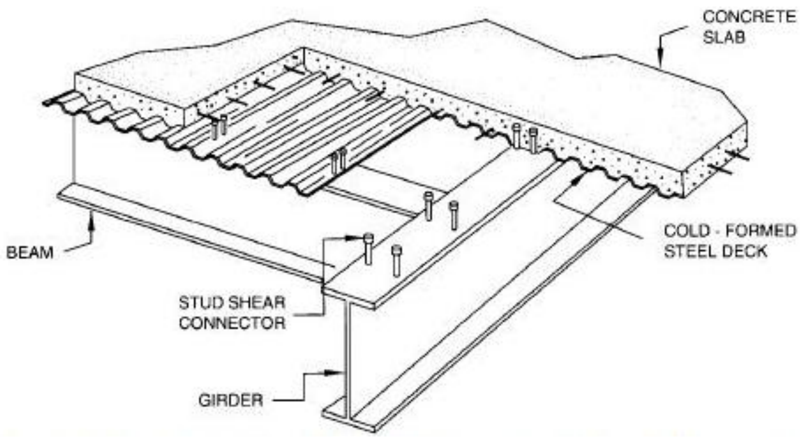

Fig. 1 Typical shear connection in composite structure

acts differently, and a non-composite action is formed. The design strength of a composite beam is typically considered in the full-interaction assumption provided that the number of shear connectors can be calculated. This assumption may not be suitable at all times because it is practically impossible to provide the calculated number of studs, especially when profiled decking is utilized for the concrete slab. The connectors can only be fitted at the profile troughs, which limits the spacing of the studs (Oven *et al*. 1997). Both the steel and concrete are usually urged to exhibit similar deflection with a proportional distribution of the applied force to the flexural stiffness of the component. When the stiffness of the connectors is assumed to be finite, a multifaceted behavior initiates at low stresses, which makes the analysis difficult. The connection experiences some deformation accompanied by an associated movement at the connection edge that results in enhanced shear deformation in the beam. This phenomenon leads to a partial interaction development at the interface of the concrete and steel that provides a relatively good behavior of applied and resisting moments with low constructional expenditures. The use of modified analytical methods may provide a relatively easy solution to this particular design problem. However, studies have shown that any flexibility in the connection may be ignored for beams designed for full connection.

Shear connectors are of many types. According to the distribution of shear forces and the functional dependency between strength and deformation, shear connectors are often categorized as rigid or flexible. For rigid shear connectors, shear forces are resisted through the front side by shearing; in the proximity of ultimate strength, their deformation is insignificant. Strong concentrated stress on the surrounding concrete is produced by this type of connector, which results in either failure of the concrete or failure of the weld. For flexible shear connectors, shear forces are resisted by bending, tension, or shearing at the root. At the connection point of the steel beam, a point where the ultimate strength values are reached, such connectors are subjected to plastic deformation. A complete description and detailed discussion of the mechanism, manufacturing, functioning, advantages and disadvantages, and expressions for the design of various types of shear connectors (e.g., headed studs, Perfobond ribs, T-rib connector, oscillating Perfobond strips, waveform strips, T-connectors, channel connector, non-welded connectors, pyramidal shear connectors, Hilti shear connector, and rectangular-shaped collar connectors) are presented in Shariati *et al*. (2012b). Out of various types of connectors, channel connectors have high load-carrying capacity, utilize a reliable conventional welding system, and have no need for

Fig. 2 Application of channel shear connectors in bridge and building

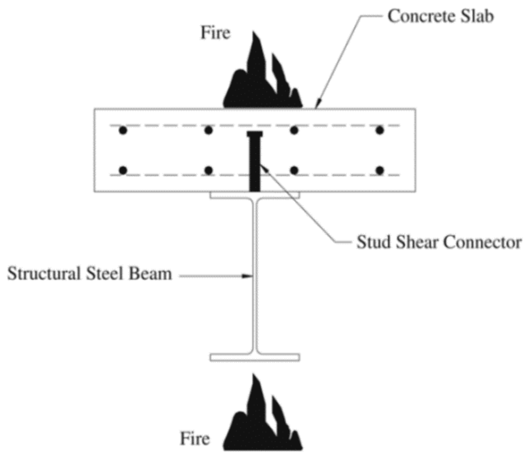

Fig. 3 Cross-sectional diagram of composite steel and concrete beam subjected to fire (Mirza and Uy 2009)

site inspection, such as the bending test for stud connectors. Furthermore, only a few channel shear connectors are needed for a multitude of headed stud shear connectors (Shariati *et al*. 2012c). Channel connectors may provide economic and enhanced performance in many construction applications, such as composite beams and girders in buildings and bridges (Fig. 2) under fatigue and monotonic loading (Maleki and Bagheri 2008a, Maleki and Mahoutian 2009, Shariati *et al*. 2012c), and can be efficiently embedded in fiber-reinforced concrete (Maleki and Mahoutian 2009), plain reinforced, and lightweight concrete (Dallam 1968, Shariati *et al*. 2010, 2011b), solid and metal deck slabs (Hosain and Pashan 2006), and HSC slabs (Shariati *et al*. 2012c).

Shear connectors are subjected to tensile-bending-shear composite stress because of the deformation difference between the steel beam and concrete slab at ambient temperature. The failure mechanism is more complicated at elevated temperatures than at ambient temperatures because of the thermal stress caused by the non-uniform temperature distribution across the composite section caused by variable material properties. The structural resistance of a member decreases with the increase in temperature and significantly relies on the performance of the shear connector. Hence, for a safe design, understanding the behavior and design requirements of shear connectors under fire conditions is essential. In the event of a fire, both steel and concrete members face the fire directly, whereas shear connectors indirectly faces the increase in temperature that is transferred by the steel, as illustrated in Fig. 3.

The structural behavior of shear connectors has been observed by a number of researchers through experimental and numerical investigations. Studies have shown that the structural response of shear connectors is affected mainly by geometrical characteristics, such as the number, height, length, and thickness of the connectors, compressive strength of the concrete, and percentage of the transverse steel reinforcement presented in the concrete slab (Viest 1952). Among several types of connectors, C-shaped connectors require fewer expenses than other connectors (Mohammadhassani *et al*. 2014, Shariati *et al*. 2014, Uenaka *et al*. 2000, Viest 1952, Viest and Siess 1954, Dallam 1968, Zhao and Kruppa 1996, Hosain and Pashan 2006, Maleki and Bagheri 2008a, b, Maleki and Mahoutian 2009, Shariati *et al*. 2010, 2011a, b, 2012a, c, 2013). Despite the availability of research on shear connectors at ambient temperatures, studies on the behavior of shear connectors at elevated temperatures are rare. This paper presents a critical review of the research performed in the last few decades on shear connectors subjected to fire conditions to identify the parameters that govern connector behavior. Shear connectors embedded in different types of concrete slabs and composite beams subjected to elevated temperatures are discussed. Experimental testing methods and associated failure modes are explained. Design approaches recommended by various codes and analytic expressions recommended by different researchers are compared to identify the most workable design solution.

2. Experimental testing

To perform fire testing of shear connectors, the push-out specimens are pre-loaded at room temperature up to a certain limit of their ultimate load to ensure the accuracy of arrangements, and force is applied across the cross section. In accordance with International Standards Organization specification, ISO 834 (ISO 834-1 1999), after the pre-load process, the specimens are heated to a given temperature, in which the temperature of the stud is measured at 10 mm. Thermal action is

Fig. 4 Typical push out test setup for testing of shear connectors subjected to fire

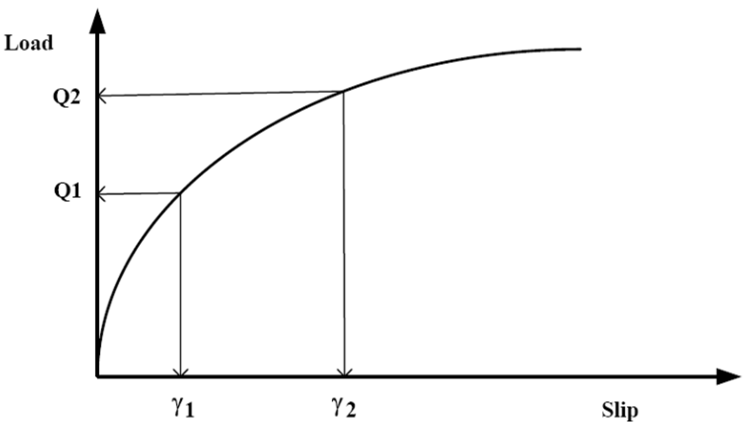

Fig. 5 Typical load-slip relationship from the standard push out specimen

applied up to the failure of the specimens. The specimen is covered with the help of wool in order to prevent the heat loss. Ceremic heaters are usually used to apply the thermal actions to the specimen. The temperature of the stud at 10 mm from the flange of the steel beam is considered for several reasons. First, the temperature distribution of the stud in the longitudinal direction is not uniform. Second, the stress state at the base of the stud is highly critical, and shear failure always occurs at the weld-collar interface. Therefore, the bottom part of the connector governs the shear capacity of the entire stud. Third, for a distance below 10 mm from the flange of the steel beam, the temperature difference between the beam flange and the stud is almost insignificant. Therefore, it is reasonable to adopt the temperature of the stud at 10 mm from the flange of the steel beam as the temperature of the stud. A typical push out test setup at elevated temperature is presented in Fig. 4. A typical load-slip relation from standard push out test is presented in Fig. 5.

2.1 Failure modes

Three types of failure to push-out tests at ambient and elevated temperatures are defined in the literature. These modes are: Fracture of the shear connector (Fig. 6), Concrete failure (Fig. 7) i.e.,

(a) Fractured channel in slab

(b) Fractured channel attached to steel I beam

Fig. 6 Typical fracture of shear connector (Shariati *et al*. 2012c)

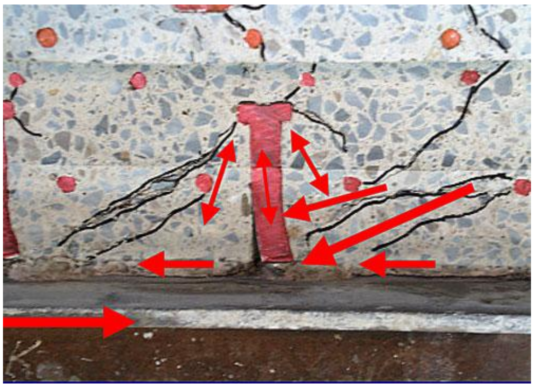

Fig. 7 Concrete failure

concrete crushing-splitting and concrete shear plane failure. The rod of the stud connector experiences shear and flexural stresses, whereas high compressive stress occurs in the concrete element near the connector. A shear connection relies on the material strength and stiffness of the connector as well as on the strength of the concrete placed in front of the connector. Thus, the crushing of the small region adjacent to the stud with a permanent gap behind the stud is the common crushing mode of concrete. In this case, stud failure may be observed, and the compression failure progresses through the thickness of the concrete, forming a conical shape around the stud. In several cases, the temperature of the bottom layers of concrete is considerably higher than that in the other portions. This condition causes the connectors to fail in an overturning mode instead of in a shearing-off mode, which results in reduced load-carrying capacity.

2.2 Experimental research on shear connectors at elevated temperature

Fire resistance testing of construction was formalized over 80 years ago, although testing had been going on prior to that in an unplanned and informal manner. The earliest recorded tests were in the UK, Germany and the USA. The Associated Architects in the UK tested a floor in the 1790s. The Technical High School in Munich tested a column in 1884 and in the Denver Equitable Building in the USA a floor was tested in 1890.

Early tests were carried out in brick huts using wood as a fuel where the floor or wall under test was part of the hut itself. Early testing was very simple, construction was tested and observations made of its behaviour, primarily with reference to collapse and to the transfer of fire to the unexposed side of the wall or floor. The main test station in the UK at Borehamwood was opened in 1935.

It can be said that the fire resistance test assesses the behaviour of components and structures in the post-flashover stage of a fully developed fire. Techniques for conducting fire resistance tests have not changed significantly in the last 60 years. Standard fire tests are conducted worldwide and are defined by the ISO 834 (ISO 834-1 1999). Standard fire tests in the United Kingdom are defined in BS 476: Parts 20-23: Fire tests on building materials and structures BS 476 (BS 476-20 1987). The first ASTM standard for fire resistance testing, C19 (now E119 (Schwartz and Lie 1985)), was published in 1918. The standard fire curve is prescribed by a series of points rather

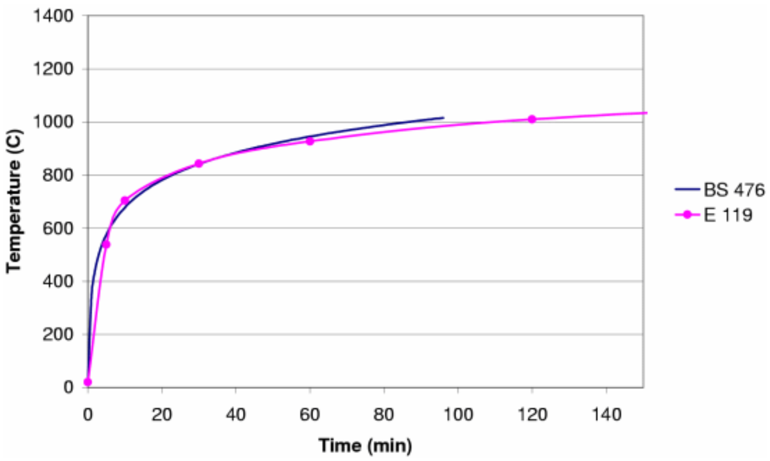

Fig. 8 Standard Temperature-time curves (BS 476 (BS 476-20 1987, ISO 834-1 1999)

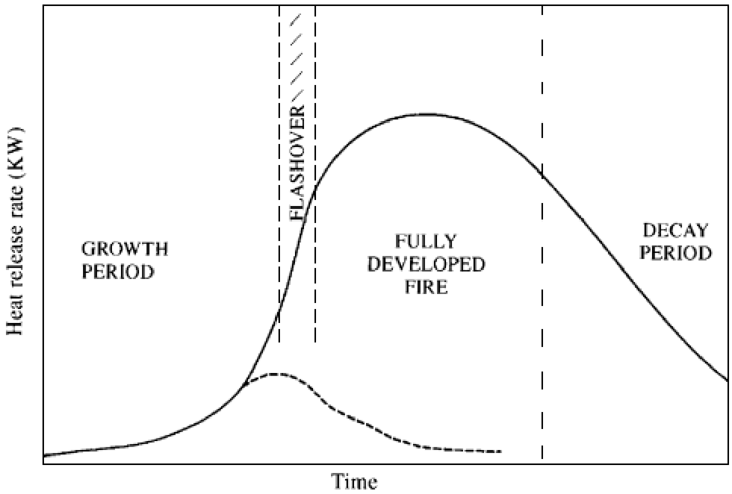

Fig. 9 The growth of natural fire ISO (ISO 834-1 1999)

than an equation, but is almost identical to the British Standard curve. Both the BS temperature time curve and the ASTM curve are illustrated in Fig. 8.

The standard fire curve describes the variation of the temperature of the fire gases within a standard furnace but bears little resemblance to any natural fire curve. It takes no account of the different thermal exposures which result from different compartment geometries, ventilation conditions, fire loads and compartment boundary materials. With the t-equivalence approach the heating effect in a compartment is calculated based on real compartment fire behaviour and that heating is related back to the standard furnace test. However, the energy and mass balance equations for the fire compartment can be used to determine the actual thermal exposure and fire duration. This is known as the natural fire method. This method allows the combustion characteristics of the fire load, the ventilation effects and the thermal properties of the compartment enclosure to be considered. It is the most rigorous means of determining fire duration. This is not related in any way to the standard fire resistance test and represents the real fire duration, once flashover has occurred. Local fires can only be determined by natural fire curves. The growth curve of a natural fire is represented in Fig. 9.

In composite beams, longitudinal shear forces are transferred through shear connectors across the composite edge. The behavior of shear connectors at high temperatures is of great importance in the performance of composite beams exposed to fire. High temperature reduces the mechanical strength of structural elements, particularly in composite structures where all members behave differently according to their material properties.

A few experimental studies that investigate the structural behavior of shear connectors subjected to elevated temperature are available in the literature. Kruppa and Zhao (1995) conducted a series of push-out tests to determine the shear capacity and load–slip relationship of headed studs and angled connectors subjected to standard ISO (ISO 834-1 1999) fire in conjunction with solid and composite slabs with both trapezoidal and re-entrant-profiled steel sheet. All specimens were heated until collapse while the load was constant, and different load levels were considered in the test. The collapse of specimens with flat concrete slab and composite slab with re-entrant-profiled steel sheet was always caused by the shearing off of headed studs

(a) Schematic drawing of the modified push-out test (b) Instrumentation arrangements (c) Locations of thermocouples

Fig. 10 Experimental test setup used by (Choi *et al*. 2009)

only at the flange level. The mode of failure of the specimen with a composite slab with trapezoidal steel sheet was the shearing of studs accompanied by shearing off of the concrete rib. The mode of failure with angle connectors was the shearing off of pins used to fix the connectors in the steel flange.

Choi *et al*. (2009) investigated the strength retention properties of headed shear studs with a concrete slab at elevated temperatures using push out test setup as shown in Fig. 10(a)-(c). An alteration in the standard push-out test was implemented by replacing one side of the solid slab with an electric furnace to apply three-sided fire exposure to the specimens.

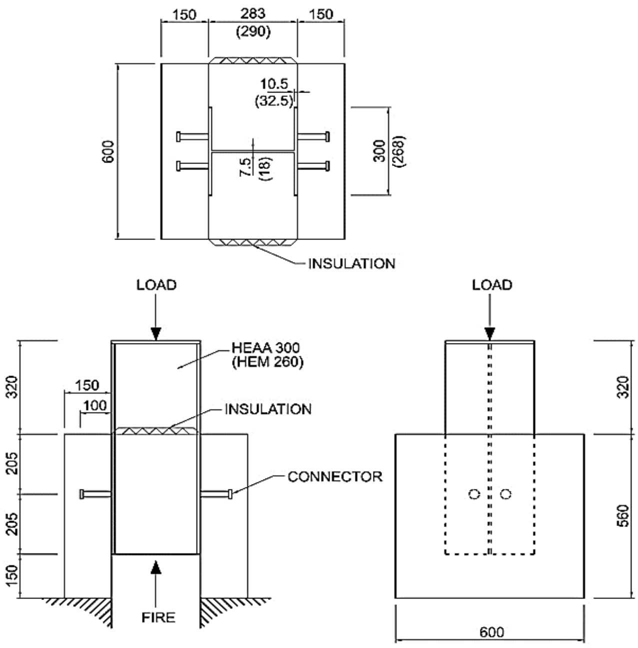

Fig. 11 Details of push out test setup by Quevedo and Silva (2013)

The standard ISO fire test was utilized to heat the specimens for 30 and 60 min. The temperature difference between the 10 and 50 mm reference points was in excess of 200°C within 30 min of the standard fire and 300°C for 60 min. Failure of the connectors occurred because of the shear of the weld-collar interface. To evaluate the residual strength of headed studs, an equation was proposed and verified. However, the lateral movement of the slab base and top of the steel was restrained, and development of uplift forces in the connectors was not considered.

Quevedo and Silva (2013) performed a thermal analysis of connectors through different push-out experiments under fire. The details of push out testing arrangement are illustrated in Fig. 11. The first category consisted of tests at room temperature. The second category was based on incremental temperature, and no load was applied to estimate the effect of thermal elongation on the calculated associated displacements. In the third category, headed studs with a variable diameter, but similar height and flat composite slabs with profiled steel sheets under different load levels were subjected to fire. For the experiments, the steel beam profile and internal sides of the two concrete slabs were directly exposed to the hot gases.

Satoshi *et al*. (2008) performed push-out tests at high temperatures. Two headed stud shear connectors on an H-shaped steel beam were embedded in two types of slab models that represent a reinforced concrete slab and a composite slab with profiled steel decks. The effects of concrete strength, shear loading level, and heating conditions were observed. Results indicated that the most crucial design parameter is the temperature at the bottom of a headed stud shear connector. On the basis of the test results, a new formulation was proposed to calculate with good accuracy the shear capacity per stud by considering the heat-induced degradation of steel–concrete materials.

Rodrigues and Laím (2011) investigated the effects of perforations in Perfobond connectors, the adjustment of reinforced bars passing through connector perforations, and the distance between the connectors under both ambient and elevated temperatures. The governing failure mode was found to be the cracking and crushing of concrete at the bearing face of the connectors. The capacity of the connectors was significantly lower under fire than at room temperature. In another study by Rodrigues and Laím (2014) on the behavior of various types of connectors with perforations, the researchers concluded that the shear strength of connectors considerably depends on their shapes.

Lu *et al*. (2012) presented the use of shear connectors in steel sheeting at different temperatures. Experimental investigations were conducted to estimate the lap shear behavior of shot-nailed and screwed connections at both ambient and elevated temperatures. The governing modes of failure were found to be the failure of the thinner plate and failure of the shear connectors. The limitation of the study was the reduced size of specimens in experimental testing in relation to the size of the electric furnace. Al-Sa’ady (2005) monitored the influence of the earlier heating of the load–slippage behavior of connectors. The prepared concrete slabs were initially heated and then tested through push-out tests. As a result of heating, cracks developed on the concrete surface. During the tests, the stiffness of the connectors was considerably reduced, and an increase in slippage was consequently observed. In terms of stiffness, earlier heating severely damaged the connectors in comparison with the concrete slab.

Chen *et al*. (2012) recently conducted experimental studies through seven push-out tests with headed stud shear connectors at different temperature levels. The furnace for the fire test was obtained by placing two electric heating plates on the sides of the push-out sample. An equation was proposed to estimate the strength of the shear connectors under fire. The proposed equation was compared with the other available equations. The governing failure for the slabs with the corrugation of the deck running parallel to the steel beam was headed stud shear failure while the predominant mode of failure for the profiled slabs with the corrugation of the deck running perpendicular to the steel beam was concrete cracking failure at low temperatures and then turned to the stud shear failure at high temperatures. The governing failure mode at high temperatures was the fracture of the shear connectors. The researchers highlighted that further research is needed to verify the design guidelines of EC4 for a temperature range of 300 ℃ to 400 ℃ when the failure mode develops in the concrete.

3. Analytic approach at elevated temperature

The resistance of a structure against fire has been articulated by evaluating the time successfully resisted by the structure under standard furnace testing within a recognized time–temperature regime (BS 476-20 1987, ISO 834-1 1999). The American Institute of Steel Construction (AISC) (1993, 2005), and the National Earthquake Hazards Reduction Program Seismic Provisions (NEHPR 2000) provide provisions for the design of composite members with a detailed design of shear connectors. BS 5950 (Code 1990) provides further guidance on the resistance of headed stud connectors surrounded in a concrete slab. The capacity table is based on a linear relationship between the strength of the stud and concrete cube established empirically by Menzies (1971) from regression analyses of experimental results obtained from standard push-out tests. In AISC (2005), the nominal shear of a stud shear connector is governed by

$$Q_{sc=}0.5\, A_{sc}\sqrt{f'_c E_c} < A_{sc}F_u \tag{1}$$

Where Q_{sc} is the nominal unfactored design strength calculated using AISC, A_{sc} is the cross-section area of headed stud shear connector, f'_c is the compressive cylinder strength of concrete, E_c is the initial Young's modulus of concrete, and F_u is the specified minimum ultimate tensile strength of the headed stud shear connector.

EN 1992-1-2 (2005) suggests the following formulae to determine the ultimate resistance of the connectors, $P_{R,}$ at ambient temperature, which is taken the lesser of

$$P_R = 0.29 \propto d^2\sqrt{f_{ck}E_c} \tag{2}$$

and

$$P_R = 0.8 f_u \frac{\pi d^2}{4} \tag{3}$$

3.1 Proposed equations by researchers

The equations proposed by several researchers to calculate the degradation of shear resistance with temperature are listed below.

3.1.1 Equations in EN 1994-1-2 (2005)

In 1995, Kruppa and Zhao (1995) performed push-out experiments to determine the shear capacity and load–slip relationship of headed studs and angled connectors subjected to standard ISO fire. On the basis of their experimental results, empirical equations to calculate the shear resistance of shear connectors exposed to elevated temperatures were achieved and applied in Eurocode EN 1994-1-2 (2005). These formulae are as follows

$$Q_d = \frac{0.8 f_u \pi \frac{d^2}{4}}{\gamma_v} \tag{4}$$

$$Q_d = \frac{0.29 \alpha d^2 \sqrt{E_c f_{ck}}}{\gamma_v} \tag{5}$$

$$dT = 0.8\, k_{u\theta\, d} \qquad \text{with } d \text{ as obtained from Eq. (4)} \tag{6}$$

$$dT = 0.8\, k_{c\theta\, d} \qquad \text{with } d \text{ as obtained from Eq. (5)} \tag{7}$$

3.1.2 Equations by Choi et al. (2009)

Consideration of the acceptable connector ductility in the composite plastic design is available in the form of 5 mm slip. The following equation was proposed by Choi *et al*. (2009) to evaluate the shear strength of a stud at elevated temperatures, in which the ultimate capacity of a stud at a slip of 5 mm was calculated.

$$Q_{uT} = f_u k_{u\theta} \pi \frac{d^2}{4} \tag{8}$$

3.1.3 Equations by Mirza (Mirza and Uy 2009)

Mirza and Uy (2009) reported that the ultimate shear load associated with different temperatures at 4 mm slip for shear connectors surrounded in the solid and profiled slabs could be determined from Eqs. (9) and (10), respectively.

$$Q_{uT} = -0.0003T^2 + 0.0525\mathrm{T} + 186.67 \qquad \text{for} \qquad 0°\mathrm{C} \leq T \leq 600°\mathrm{C} \tag{9}$$

$$Q_{uT} = 4 \times 10^{-7} T^3 - 0.0004\mathrm{T}^2 + 0.034\mathrm{T} + 38.721 \qquad \text{for} \qquad 0°\mathrm{C} \leq T \leq 600°\mathrm{C} \tag{10}$$

Eqs. (6) to (8) were based on empirical equations to determine the shear resistance of a stud at ambient temperature, whereas Eqs. (9) and (10) were directly derived from the experimental results with curve fitting. Although the equations to calculate the shear resistance of studs at room temperature were also developed from empirical methods, Eqs. (6) to (8) appear to be more reasonable because the degradation of the shear resistance of the stud is caused by the degradation of the steel and concrete. Numerous existing research achievements on the behavior of studs at room temperature could be used. In this study, the above equations were compared with the test results achieved after push-out specimens and the calculation formula specified in Eurocode EN 1994-1-2 (1994) to estimate the shear strength at elevated temperatures. EC 4 predicted the shear strength of stud shear connectors at elevated temperatures with acceptable accuracy.

4. Finite element modeling

FE modeling is an important tool in investigating shear connector performance given the lack of experimental results and can be utilized to conduct extensive parametric studies. In FE analysis, a 2D numerical model is prepared to apply the thermal action to the specimens. The resulting temperature profiles, degradation, and histories are then used in another FE model based on mechanical loading. A specimen is progressively loaded up to the ultimate load to estimate the strength of the composite structure. This approach may result in several errors because the contact between the shear connector and concrete slab enables heat transfer in a 3D model. Thus, a 2D model may not be capable of transferring heat in a required manner, which may further disturb the mechanical loading model. The review of FE analysis shows that the strength of a shear connector becomes delicate under fire. Profiled steel sheeting slabs are more resistant under fire than solid slabs.

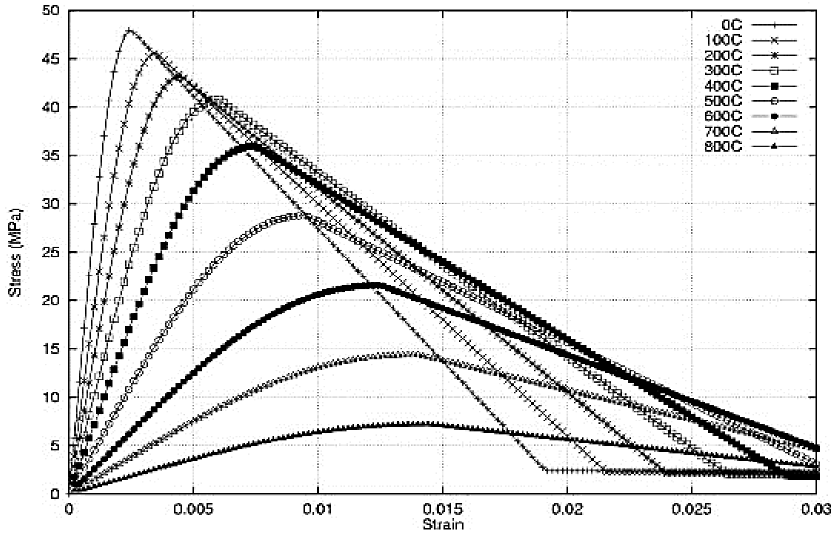

Fig. 12 Compressive concrete material behaviour in EC2 Part 1.2 (EN 1992:1-2 2004)

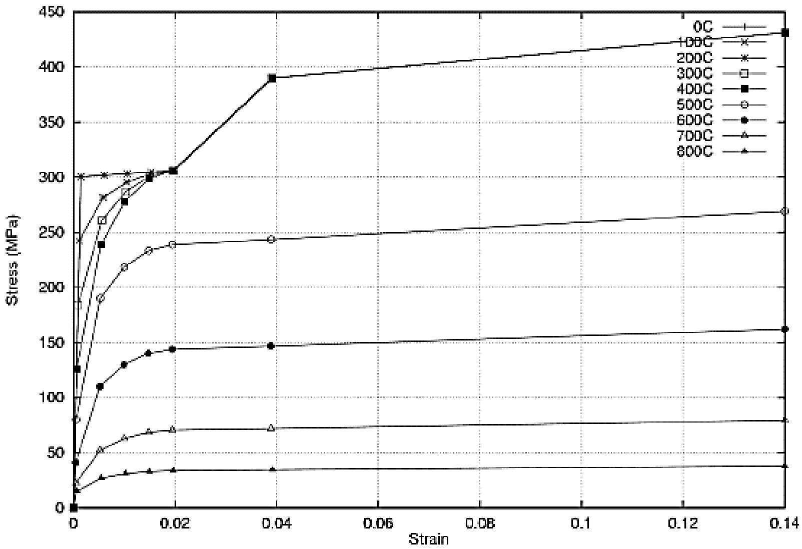

Fig. 13 Steel material behaviour in EC3 Part 1.2 (EN 1993:1-2 2005)

Most of the plasticity in the concrete and shear connector occurs at the bottom of the shear connector in the loading direction. Most FE analyses presented in literature could capture these plastic actions under shear loading of the composite beam. After only 15 min of fire (Schwartz and Lie 1985), the shear connector loses around 50% of its capacity. The friction between two different materials in a composite structure affects the results at low values of slip. However, as the slip increases, some separation occurs between the beam and deck, and the friction does not affect the results significantly.

Regarding the FE modeling described by the codes, the stress-strain material definitions in EC2 Part 1.2 (2004) and EC3 Part 1.2 (2005) were assumed for concrete and steel respectively. The steel material model is elasto-platic and includes enhancement from strain hardening above 400℃. Both sets of properties include degradation with increasing temperature and are illustrated in Figs. 12 and 13.

Quevedo and Silva (2013) performed FE modeling of shear connectors at elevated temperatures. Two simple models were prepared. The sensitivity of the calibrated model to various connector diameters and heights was also assessed. Furthermore, different alternatives on the level of concrete temperatures to be considered were evaluated. The parametric numerical studies demonstrate that the connector height, the concrete compressive strength and the level in which the concrete temperature is considered, have a great influence on the resistance, specifically when concrete failure prevails against stud failure. The results were compared with the equations provided in existing design codes. The findings suggested that under specific design situations, within the scope of EC 4 (2005), the current formulation could overestimate the resistance of the stud when subjected to the temperature above 400℃.

Mirza and Uy (2009) analyzed solid and profiled trapezoidal slabs subjected to fire by modeling a 2D thermal model and a 3D mechanical model. Three-dimensional solid elements were used to model the push-out specimens. For both the concrete slab and structural steel beam, a 3D solid 8-node element was used to improve the rate of convergence. A second-order 3D 30-node quadratic brick element was adopted for shear connectors. A 4-node doubly curved thin shell element was used for the profiled steel sheeting with 6 degrees of freedom. A 2-node linear 3D truss element for steel reinforcement was adopted where the axial direction was released. The analysis did not focus on the slip between the reinforcing bar and concrete. Failure occurred because of the concrete cracking and crushing prior to the fracture of the shear connectors close to the weld collar. The results showed that the area of the concrete under elevated temperature significantly governed the failure.

Ranzi and Bradford (2007) analyzed composite beams with partial interaction under fire with an analytic expression. Examples were provided to illustrate the influence of connector stiffness on the capability to resist the applied load, which caused the first yield of the cross-section under fire. A simplified integrated method that is able to consider material degradation and thermal influence was introduced. The proposed expression slightly underestimated deformation under fire for high levels of shear connection.

Benedetti and Mangoni (2007) proposed an extended method of Fourier series calculations to analyze composite beams subjected to fire. The finite stiffness of the connectors were assumed, and special consideration was given to quantify the overall influence of temperature on the degradation of material properties of the member and stress distribution across the composite interface. This formula covers all changes along the beam axis and calculates deflection in a closed form.

Fahrni and Tofiq (2012) proposed a nonlinear 3D FE model to analyze composite steel–

concrete beams subjected to BS476 fire test. An eight-noded isoparametric brick element was used to model the reinforced concrete slab for structural analysis and the solid slab and steel for thermal analysis, whereas the shear connectors were modeled by using truss (spar) elements and nonlinear spring elements to resist slip and uplift the separation between the steel beam and concrete slab. The number of shear connectors highly influenced the beam failure. The fire resistance of the composite beam decreased significantly by decreasing the number of connectors. Majdi *et al*. (2014) modeled the concrete slab by using 3D solid elements, whereas all steel parts were modeled by using 2D shell elements because of their small thicknesses. A continuous hat channel (furring channel) was modeled as the shear connector and FE analysis of new composite floors having cold-formed steel and concrete slab was performed.

Lu *et al*. (2013) developed FE models to quantify the resistance of shot-nailed connections to the applied forces at both room and elevated temperatures. The protuberance feature, residual stresses, and deformation of thin sheet developed during the driving process into the subsequent heating and the lap shear loading processes were considered. Three types of failure, namely, bearing, shear, and net-section tension failure of the thin sheet, were observed. The failure of the nail shank was not considered in the FE modeling.

Wang (2011) performed a numerical investigation of the behavior of shear connectors using thermal and mechanical FE models. The temperature of the bottom layers of the concrete was considerably higher than that of the other portions, which caused the connectors to fail in an overturning mode instead of in a shearing-off mode; hence, load-carrying capacity was reduced. Huang *et al*. (1999) modeled a 3D procedure to analyze shear connectors under fire. In the statically determinate elements, the local failure of the connectors abruptly changed the behavior at high temperatures. An analytical expression was also introduced on the basis of Kruppa and Zhao's experimental result. However, limited study was conducted, and the effect of the combined axial and shear deformations on the failure of the shear studs was ignored in the proposed formulation.

5. Future recommendations

For future work, the effect of using a shear stud without threads and nuts welded directly to the steel parts should be studied, and these two cases should be compared. The effects of the compressive strength of concrete and the effects of impact and fatigue loading under fire on the behavior of the connectors should also be studied. Further research on shear connectors with different heights should be conducted to develop a general equation that could predict the shear capacity of connectors with different heights at elevated temperatures.

With respect to the evaluation of the overall behavior of composite beams, which indirectly affects the performance of shear connectors, we noticed that the strength of composite beams subjected to fire is estimated with reference to the time elapsed for a structure to resist temperature and mechanical loading under fire exposure, at which some prescribed form of limiting behavior occurs.

The lack of design guidance on relevant standards, promotes a performance-based design of such structures. In performance-based design, this limiting behavior may be defined either as real structural collapse or as a failure of integrity that would allow fire-spread to occur but is usually defined in terms of a deflection limit. Performance-based design may provide a good opportunity to estimate the behavior of individual components, including shear connectors. Instead of using separate FE modeling for thermal action and mechanical loading, the development of a combined model may accurately predict the behavior of shear connectors under fire.

Channel connectors have high load-carrying capacity, utilize a reliable conventional welding system, and have no need for site inspection, such as the bending test for stud connectors. Furthermore, only a few channel shear connectors are needed for a multitude of headed stud shear connectors. Channel connectors have provided economic and enhanced performance in many construction applications, such as composite beams and girders in buildings and bridges under fatigue and monotonic loading and can be efficiently embedded in fiber-reinforced concrete, plain reinforced, and lightweight concrete, solid and metal deck slabs and HSC slabs. Future research on fire behavior of channel connectors may prove an economical and reliable fire safety design and application of channel connectors in bridges and buildings.

6. Conclusions

The following conclusions can be drawn from the review of studies on the behavior of shear connectors under elevated temperatures.

(1) The shear strength of a connector is considerably reduced with the increase in temperature at a comparatively higher rate than that in concrete.
(2) Three types of failure of push-out specimens are defined in the literature. These modes are fracture of the shear connector, crushing or splitting of the concrete, and concrete shear plane failure.
(3) The degradation in the structural properties of a composite structure under fire changes the flexural and extensional stiffness of the beam and slab. This variation modifies the equilibrium along the beam axis and the transfer of shear forces through the connection.
(4) The calculation formula specified in Eurocode EN 1994-1-2 to calculate shear strength at elevated temperatures predicts the shear strength of stud shear connectors at elevated temperatures with acceptable accuracy.
(5) The literature revealed that out of several types of available shear connectors, headed stud shear connectors are more reliable in the case of fire. The use of headed stud connectors may result in minimum property loss when applied to the buildings where there is high risk of fire such as storage buildings with combustible materials and goods.

Acknowledgments

The study presented herein was made possible by the University of Malaya Higher Impact Research (HIR) Grant: UM.C/HIR/MOH/ENG/57 and UMRG grant (project RP004C-13AET). The authors would like to acknowledge the support.

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