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Experimental study on the flexural behaviour of demountable steel-concrete composite beams at room temperature and under fire exposure

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ABSTRACT

In a context driven by the urgent need to promote sustainability and circularity in the construction sector, the use of demountable shear connectors in steel-concrete composite beams has a potential to make a significant contribution to reducing the carbon footprint, helping in the management of waste generated in the deconstruction stage through the "Design for Disassembly" concept. To ensure the viability of these shear connectors in practice, it is essential to understand their mechanical behaviour under various loading situations. This paper presents the outcomes of an experimental campaign comprising a series of beam tests conducted under both room and elevated temperature conditions. The results of the tests showed that, during a fire - provided that suitable protection is used for the steel beam -, the shear connectors retain their mechanical contribution and composite action between the steel beam and concrete slab is still active along fire exposure, thereby enhancing the flexural capacity of the beam. The findings from this investigation ensure the structural efficiency of demountable steel-concrete composite beams under both fire and ambient conditions, along with their alignment with the circular economy principles. This study sets the ground to facilitate the practical application of demountable steel-concrete composite beams in the construction sector.

1. Introduction

The management of waste from demolition in construction is one of the challenges to face in order to meet the sustainability requirements. For the purpose of contributing to the circular economy, which aims to reuse waste as raw material, the use of structural components which can be disassembled after its use in construction should be promoted. In line with this concept, a particular typology of steel-concrete composite beams with demountable shear connectors is studied in this paper. This type of composite beam is composed of a concrete slab and a structural steel profile, the connection between these components is materialized by means of a bolted shear connector.

Traditionally, the way to connect these components is through a shear stud welded to the top flange of the steel profile. This construction technique results very efficient in structural terms, however, it does not meet the sustainability requirements, since once the structure has reached its end life and is demolished, the composite beam cannot be reused, since the connection makes it impossible to separate the concrete slab from the steel profile.

In order to solve this problem in composite steel-concrete beams, bolted shear connectors were developed, which allow the separation of the beam components and their subsequent reuse. Demountable shear connectors can be classified as partially demountable or fully demountable, the former refers to structures in which the bolt is embedded in the slab after disassembly, and can be either through-bolted, i.e. surpassing the slab width (Fig. 1).

Fully demountable shear connectors are those in which the bolt can also be completely detached from the system (Fig. 2).

With regard to research on fully demountable shear connectors the company Krupp- Druckenmüller GmbH in Germany developed a connection model similar to Becker's (Fig. 2), which was called Krupp-Montex [13] and was applied in car parks in the 1970s.

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Preloaded through bolts (friction grip bolts) behave differently than conventional welded shear studs in terms of mechanical behaviour. Their load-slip curve can be divided into three clearly differenced stages [14]: a rigid part - until the friction resistance is overcome -, a horizontal plateau representing the slip of the bolt inside the hole, and a nonlinear part due to the bearing deformation of the bolt. In a fire situation, this nonlinear behaviour may be magnified due to the effect of temperature, requiring further investigation to establish the fire performance of demountable composite beams making use of this type of friction bolts.

The research project REDUCE [15], funded by the European Commission's Research Fund for Coal and Steel (RFCS), investigated demountable and reusable steel-concrete composite flooring systems with bolted shear connections. A comprehensive experimental campaign was carried out at the University of Luxembourg in the framework of the referred project, comprising push-out tests and beam tests at room temperature [16,17]. Two types of shear connections were studied: the first type corresponding to a fully demountable shear connector called "Cylinder system" (P3) (Fig. 3a), the second one being a partially demountable connector referred to as "Coupler system" (P15) (Fig. 3b).

The research carried out under the framework of the REDUCE project yielded significant results for the characterization of this novel type of shear connections. The tested bolted connectors demonstrated superior performance in terms of structural efficiency, ease of disassembly, and reusability, compared to other traditional shear connectors. Additionally, an analytical calculation method was developed to determine the plastic moment capacity of steel-concrete composite beams with demountable shear connectors, based on a set of assumptions formulated in accordance with EN 1994–1–1 [18]. However, despite the progress made, the implementation of these results in the construction industry has not yet been achieved, as a detailed characterization of the behavior of such composite beams and their connectors in the fire situation is still required.

Several studies, including those by Wang et al. [19], Zhao et al. [20], Selden et al. [21], Hongyu et al. [22] or Bihina et al. [23], investigated the behavior of steel–concrete composite beams under fire conditions. However, these investigations focused exclusively on composite systems utilizing traditional connectors, that is, shear connectors welded to the top flange of the steel profile.

To increase the knowledge of steel-concrete composite beams with demountable shear connectors, in the subsequent project FIREDUCE, developed by the authors at the Research Institute ICITECH of the Universitat Politècnica de València (UPV), two types of tests were carried out to characterize the performance of these novel bolted shear connectors under fire exposure. The shear connectors used in this project were the same as those used in the previous REDUCE project, as shown in Fig. 3. The first type of tests consisted of a series of push-out tests, which served to characterise the mechanical behaviour of the presented bolted shear connectors at elevated temperature, the results of this testing campaign were published in Mansilla et al. [24]. The second type of tests consisted of a series of four-point bending tests on steel-concrete composite beams using the previously characterized bolted shear connectors (P3 and P15), both carried out at room temperature and under fire conditions.

The aim of this paper is to provide the results obtained in the second part of the experimental campaign of the referred FIREDUCE project for the beam bending tests at room temperature and under fire conditions, in order to study more in depth the performance of the shear connectors at elevated temperature and understand precisely the failure mechanisms that occur in these novel demountable composite beams when subjected to fire.

As previously mentioned, the characterization of the behavior of composite beams with demountable shear connectors under fire conditions completes a comprehensive study that supports the potential implementation of these systems in the construction industry. Nevertheless, cost analysis remains a critical factor when assessing the feasibility of new structural solutions. In this regard, Gîrbacea et al. [25] state in their study that, although the unit cost of a demountable shear connector is significantly higher than that of a traditional shear connector, its impact on the total cost of the composite beam system is minimal, accounting for only approximately a 2 %.

2. Experimental tests on steel-concrete composite beams with demountable shear connectors

2.1. Details of the test specimens

A series of four full scale beam bending tests were carried out within the experimental campaign of the FIREDUCE project: two of them at room temperature and two under fire conditions, each pair being assembled with P3 and P15 demountable shear connectors. The designation and characteristics of the tests performed is given in Table 1.

All the test specimens consisted of an IPE 360 hot rolled profile with S275 nominal steel grade. Additionally, two reinforced concrete slabs of 150 mm thickness by 600 mm width were produced with C40 concrete class to be assembled to the top flange of the steel beam, by means of the described bolted shear connectors.

A detailed view of the connection systems P3 and P15 after attachment to the top flange of the steel beam and the dimensions of the different elements that compose the connection systems can be seen in Fig. 4.

The distance between supports varied according to the conditions of the testing facilities, being 4 m for the beams tested at room temperature and 4.2 m for the beams tested under fire conditions. A schematic view of the geometric configuration of the beam specimens and their supports can be seen in Fig. 5.

A detail of the cross-section of the composite beams tested at room temperature, with the two types of bolted shear connectors studied, is given in Fig. 6.

In particular, for the beams tested under fire conditions, a thermal protection was added, consisting of an outer box materialized with timber panels with a thickness of 18 mm and a mineral wool insulation with a thickness of 30 mm, as shown in Fig. 7. It should be noted that the inner timber blocks shown in this scheme did not contribute to the thermal protection system, but only served as a support for fixing the exterior timber panels.

The choice of this fire protection system is justified from previous studies by the authors, [26] where it was proved by means of numerical simulations that this double protection layer provided sufficient fire endurance, while meeting the sustainability requirements.

The characteristics of the materials used to manufacture the

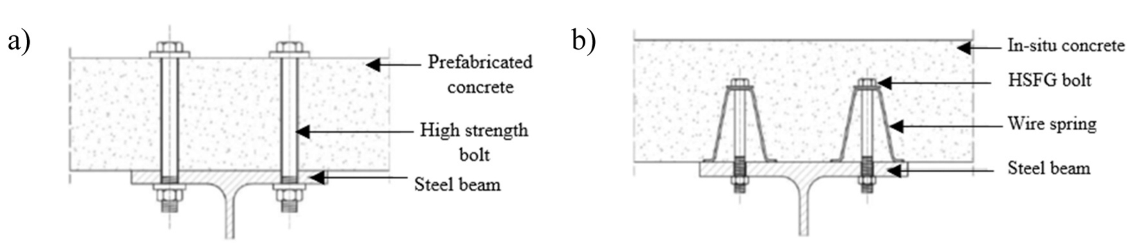


Fig. 1. Partially demountable shear connectors: a) "through" (adapted from Marshall et al. [1]) and b) "lost" (adapted from Dallam [2]).

composite beams (steel profile, bolts and concrete), which were tested at the laboratories of the Research Institute ICITECH, are given in Table 2.

The degree of shear connection (η) for the tested beams (see Table 1) is calculated according to the formula specified in EN 1994–1–1 [18], which defines η as the ratio between the sum of the design resistance of the shear connectors (N_c) and the maximum force required to fully plastify one of the materials in the composite section ($N_{c,f}$):

$$\eta = \frac{N_c}{N_{c,f}} \tag{2.1}$$

In the case of demountable shear connectors, due to their idealized non-linear behavior, the force transmitted by each connector varies. This has two main implications:

1. The connection will always be considered partial, as full rigidity cannot be achieved.
2. It is necessary to individually determine the force carried by each connector to compute the total resisted force (N_c) and accurately assess the degree of shear connection.

The values of the shear connection degree presented in Table 3 indicate that the composite beams tested experimentally exhibit a partial connection, as the values obtained are clearly below unity. This condition reflects an incomplete interaction between the concrete slab and the steel beam. As a result of the low degree of shear connection, the full load-bearing capacity of the composite section cannot be fully utilized. Therefore, there is a clear potential for structural improvement by increasing the number of connectors along the beam length, which would enhance the shear connection degree and, consequently, improve the overall structural performance of the beam-slab system.

2.2. Manufacturing of the composite beams

The fabrication of the composite beams began with the preparation of the timber formwork for the concrete slab and the positioning of the L-shaped profiles. These profiles contain pre-drilled holes in the positions where the connectors are to be located, and their purpose is to facilitate the slab-beam connection.

For the composite beam specimens FIREDUCE-RT-P3 and FIREDUCE-FR-P3, steel cylinders were welded to the L-profiles and subsequently the top steel plate was welded to them; expanded polystyrene (EPS) foam blocks were positioned to create the cavities in the concrete, these blocks were removed after concrete had hardened. In turn, for the composite beam specimens FIREDUCE-RT-P15 and FIREDUCE-FR-P15, the mechanical coupler was fitted with the bolts to hold it in place during concrete pouring. Once concrete had hardened, the slabs were lifted and placed on top of the steel beam, then the slabs were attached to the steel beam by using M20 high-strength bolts with the corresponding configuration (P3 or P15).

A dynamometric torque wrench was used to apply the desired pretension force to the bolts. An isolated test was performed for each of the connection systems by means of a load cell, the resulting values of the effective pretension force are reported in Mansilla et al. [24]. The applied pretension force was 142 kN for the P3 system and 175 kN for the P15 system, matching the theoretical values obtained by the application of Clause 3.6.1(2) in EN 1993–1–8 [27]. The same calibrated torque and the corresponding pretension force was applied to all the connectors to materialize the composite beams.

The steel beams were strengthened with web stiffeners at the positions of the load application and supports to prevent local buckling (Fig. 8). A detail of the cylinder system (P3) from the top of the slab can be seen in Fig. 9.

The installation of the fire protection system for the two beam specimens to be tested under fire conditions (FIREDUCE-FR-P3 and FIREDUCE-FR-P15) was performed in the following sequence (see details in Fig. 10):

1) auxiliary timber blocks were encased between the steel beam flanges at evenly separated positions along the length of the beam (1 m approximately) (Fig. 10a);
2) mineral wool fastening pins were glued to the lower flange and web of the steel profile (Fig. 10b);
3) the mineral wool panels were attached to the lower flange and web of the steel profile (Fig. 10c);
4) thermocouples were installed at the specified positions of the mineral wool protection and conducted towards the top surface of the slab (Fig. 10d);
5) the external timber panels were attached to the beam contour in a box-shape by fixing them with screws to the auxiliary inner timber blocks (Fig. 10e).

2.3. Room temperature tests

2.3.1. Test setup

A four-point bending test configuration was set up for the room temperature tests. The load was applied by means of a 1000 kN capacity hydraulic jack and distributed at two equidistant points located 700 mm from mid-span of the composite beam. A spreader beam and two additional transversal beams were used for the load application, neoprene layers were conveniently placed under the load application points to avoid local failure of the concrete slab due to the concentrated forces. A general view of the test setup for the room temperature tests can be seen in Fig. 11.

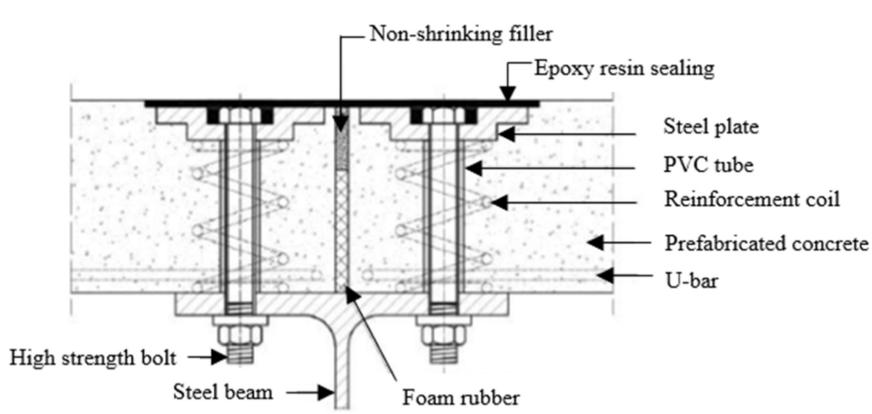


Fig. 2. Fully demountable shear connectors, adapted from Becker [3].Several authors such as Dedic et al. [4], Sedlacek et al. [5], Pavlović [6], Moynihan et al. [7], Lam et al. [8], Rehman et al.[9], Wang et al. [10], Roik et al. [11] and Chen et al. [12] studied partially demountable connectors with the novelty that these systems can be implemented in the precast concrete industry.

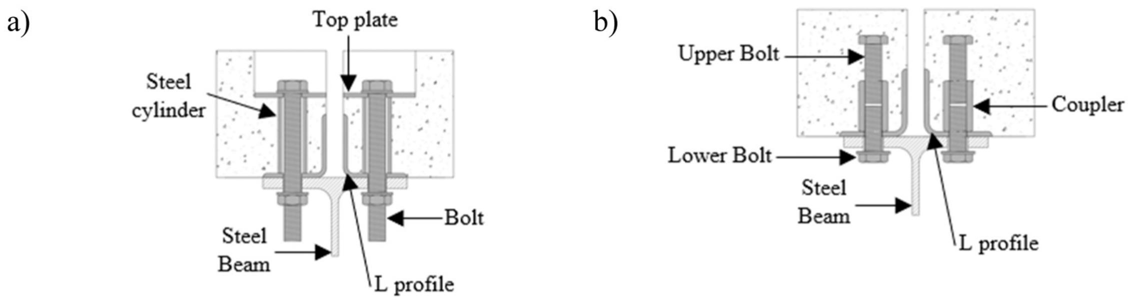


Fig. 3. Demountable shear connectors tested in the research project REDUCE: a) "Cylinder system" (P3) and b) "Coupler system" (P15), adapted from Kozma et al. [14].

Table 1
Summary of the experimental campaign carried out in the FIREDUCE project.

Test ID	Conditions	Shear connector type
FIREDUCE-RT-P3	Room temperature	P3
FIREDUCE-RT-P15		P15
FIREDUCE-FR-P3	Fire exposure	P3
FIREDUCE-FR-P15		P15

2.3.2. Instrumentation

For each composite beam, seven displacement transducers (LVDT) were applied at the locations indicated in Fig. 12, to measure:

– The deflection values of the beam at different locations along its length
– The slip between the concrete slab and steel profile at the beam ends (see detail in Fig. 13)

In addition, three strain gauges were attached to the steel profile at mid-span position to measure the elastic strains, one at the top flange,

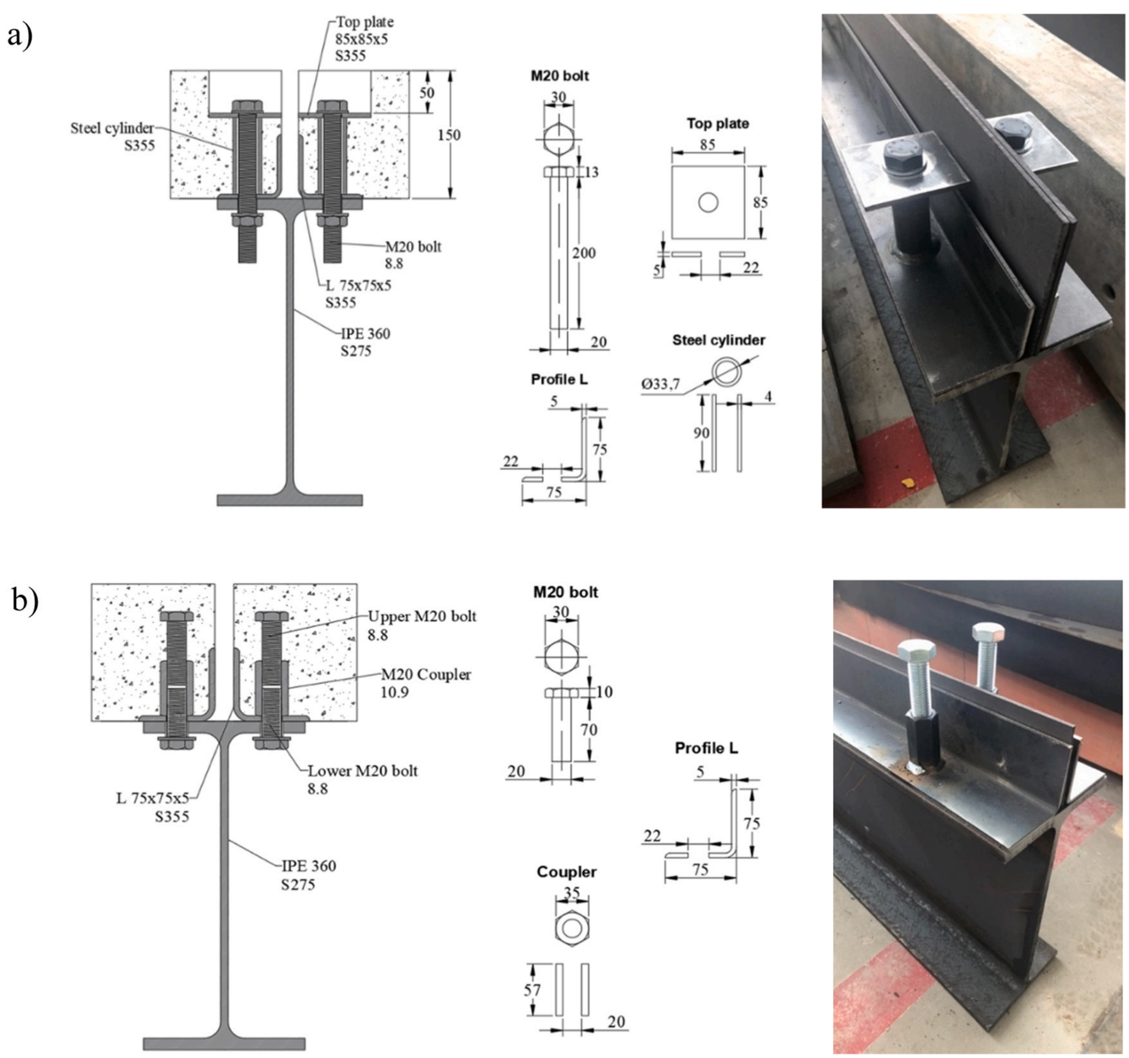


Fig. 4. Details of the connection systems used in the tested beams: a) "Cylinder system" (P3); b) "Coupler system" (P15) (dimensions in mm).

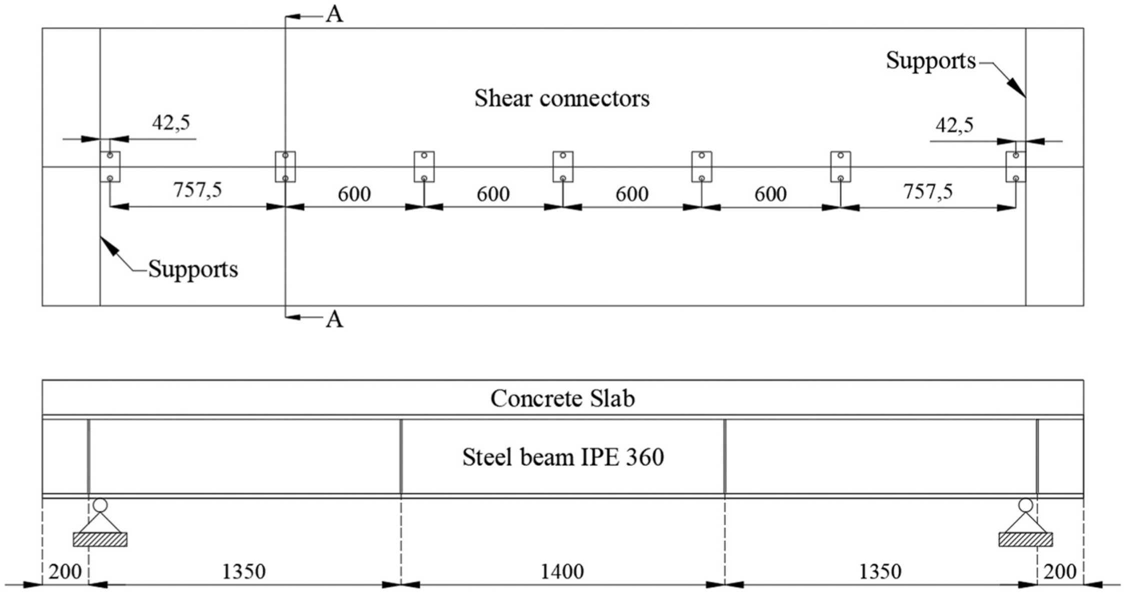


Fig. 5. Geometric configuration of the steel-concrete composite beams tested at room temperature (dimensions in mm).

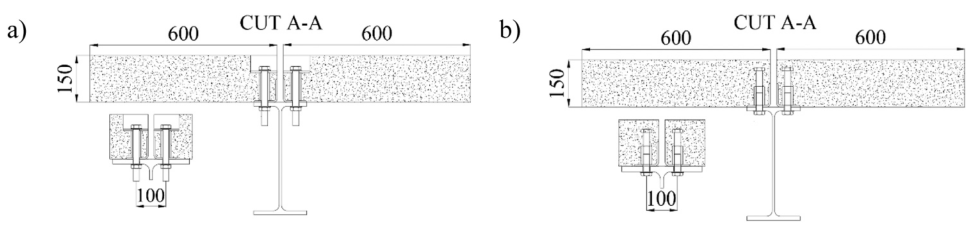


Fig. 6. Cross-section of composite beams for the room temperature tests: a) FIREDUCE-RT-P3; b) FIREDUCE-RT-P15 (dimensions in mm).

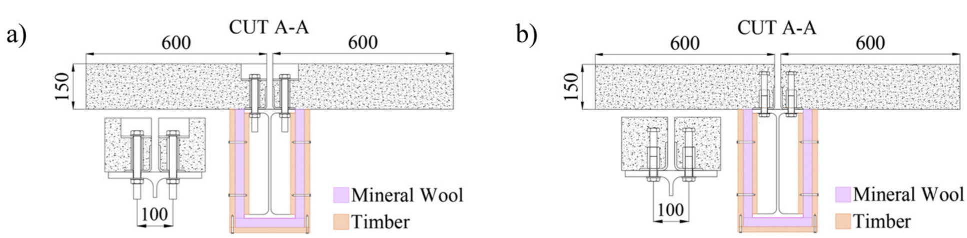


Fig. 7. Cross-section of the composite beams for the fire tests: a) FIREDUCE-FR-P3; b) FIREDUCE-FR-P15 (dimensions in mm).

Table 2
Details of the materials used for the tested composite beams.

		IPE 360	M20 bolts (Grade 8.8)
Steel yield strength (f_y) (MPa)	Nominal	275	640
	Measured	313	626
Steel ultimate tensile strength (f_u) (MPa)	Nominal	430	800
	Measured	443	858
Concrete compressive cylinder strength (f_c) (MPa)	Nominal	30	
	Measured	39.14	

Table 3
Degree of shear connection.

Shear connector type	P3	P15
Degree of shear connection (η)	0.24	0.28

one at mid-height of the web and one at the bottom flange (see detail in Fig. 14).

2.3.3. *Experimental results and analysis*

2.3.3.1. Experimental observations. The state of one of the beams after the room temperature test can be seen in Fig. 15, where it was positioned upside down to proceed to its disassembly. No evidence of shear failure of the bolts was observed. Some of them – the ones located closer to the beam ends – presented a higher level of deformation, but they were still in place with no signs of rupture. In turn, the bolts located at mid-span section did not suffer any plastic deformation and were easily screwed off. Although the demountability concept of the composite beams is conceived for serviceability (i.e. disassembly and reuse after service life), it was observed that the connection system still maintained the capability to be disassembled after ultimate failure, keeping the potential to detach the different parts and recycle the materials. Thus, considering that in service life the beams would not reach such deflection levels and in consequence the bolts would not experience significant

Fig. 8. Detail of steel beam web stiffeners.

Fig. 9. Detail of P3 connectors seen from the top of the slab.

plastic deformations, the ease for disassembly of the proposed system is guaranteed.

Fig. 16 shows the state of one of the tested beams after failure. The degradation of the concrete slab at its top surface caused by excessive compressive stresses concentrated near the load application area can be seen in Fig. 16a). In addition, cracks were developed at the bottom surface of the slab, attributable to tensile stresses exceeding the tensile strength of concrete. The failure initiation of the steel profile at mid-span can be noticed, characterized by onset of plastic strains at its web; Fig. 16a) displays indicative plasticization lines that clearly evidence the development of this failure mechanism. Additionally, the rotation of the beam ends can be seen in Fig. 16b) and c).

2.3.3.2. Load-displacement curves. The load-displacement curves given in Fig. 17 plot the evolution of the load supported by the beam along the vertical displacement (i.e. deflection) recorded at the LVDT points. These curves also show if any connector failed during the test, causing a loss of stiffness to the beam. It was observed that the failure of the connectors occurred progressively from the beam ends - where the greatest shear stress occurs - towards the centre of span. In both tests, the composite beams eventually failed due to plastic deformation of the steel profile, but the composite action between the steel beam and concrete slab remained active until the end of the tests.

From the registered load-displacement curves, it was obtained that specimen FIREDUCE-RT-P3 withstood a maximum load of 987.43 kN (Fig. 17a), while specimen FIREDUCE-RT-P15 (Fig. 17b) withstood 1056.63 kN.

In the tested beams, the only different component was the type of connector. As an outcome of the previous push-out tests carried out in the FIREDUCE project [24], it was found that the shear transfer capacity of the P15 connector was higher than that of the P3 connector, reason why specimen FIREDUCE-RT-P15 withstood a greater load at the beam test as compared to specimen FIREDUCE-RT-P3. Nevertheless, while there was a difference in the registered maximum load, it was not significant, so it can be affirmed that both connector types ensure a good structural performance for the studied composite beams, thus facilitating options for their implementation in the industry.

2.3.3.3. Load-slip curves. In addition to the previous load-displacement curves, the slippage of the concrete slab relative to the steel profile was also recorded in the experiments, which occurs in this type of composite beam due to the non-linear behaviour of the demountable shear connectors. The load versus end-slip curves measured at the different LVDTs attached to the beam ends are plotted in Fig. 18.

The records of the experimental load-slip curves are different for each LVDT, which indicates that in the experimental test the beam did not behave symmetrically, i.e. each slab had a different relative slip with regard to the steel beam end. This curve also shows the events of partial loss of stiffness of the beam due to the failure of certain shear connectors.

2.3.3.4. Measured deformations. The plastic deformations at mid-span of the steel profile determine the deformation plane that is generated as the load level increases. Once the yield strength of steel is steel profile reached at the point where a strain gauge is attached, it starts to display wrong readings and therefore no records can be kept after this point.

As displayed in Fig. 19, up to a 50 % of the maximum load the deformations were fully elastic (i.e. linear deformation plane), while once the 50 % of the maximum load was exceeded, the strain gauge placed at the lower flange of the steel profile detached and showed wrong measurements - which have been cleaned for clarity -, revealing the onset of plastic strains.

As it is widely known, in a simply supported steel beam the development of plasticity starts from the bottom fibre of the mid-span section onwards. This justifies that the strain gauge located at the lower flange of the central section was the first one to detach and fail.

2.4. Fire tests

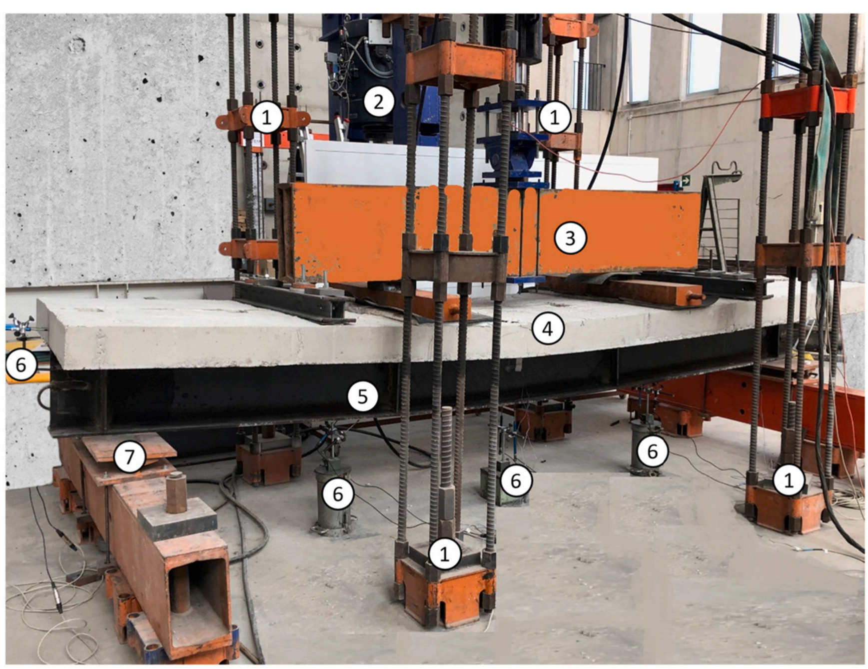
2.4.1. Test setup

The beam fire tests were conducted at the facilities of AFITI, Arganda del Rey, Madrid (Spain). The beams were subjected to a standard fire test on a gas furnace, being placed at the top of the furnace chamber and supported at both ends, simulating a simply supported beam condition. All contact areas between the furnace walls/ceiling and the beam were conveniently sealed with fibre blanket, both inside and outside the furnace, to prevent heat escape. A general view of the fire test setup can be seen in Fig. 20 (inner view) and Fig. 21 (outer view).

2.4.2. Loading protocol

The load application was performed in two steps, following the provisions in EN 1363–1 [28]: in the first step, a percentage (65 %) of the maximum load obtained in the bending test at room temperature was applied by using a hydraulic jack with a load capacity of 1000 kN

Fig. 10. Installation of the mineral wool insulation to protect the steel beam and finishing of the box protection with timber panels.



1	Loading frame	4	Concrete slab	7	End Support
2	Hydraulic jack	5	Steel beam		
3	Spreader beam	6	LVDTs		

Fig. 11. Test setup for the room temperature four-point bending tests.

(see applied load values in Table 4); in the second step, the applied load was maintained constant while the temperature was increased inside the furnace chamber, following the standard ISO-834 fire curve.

The load was applied at two equidistant points located 700 mm from mid-span of the beam, this was done by means of a spreader beam and two additional transversal beams, similarly to the test setup described in

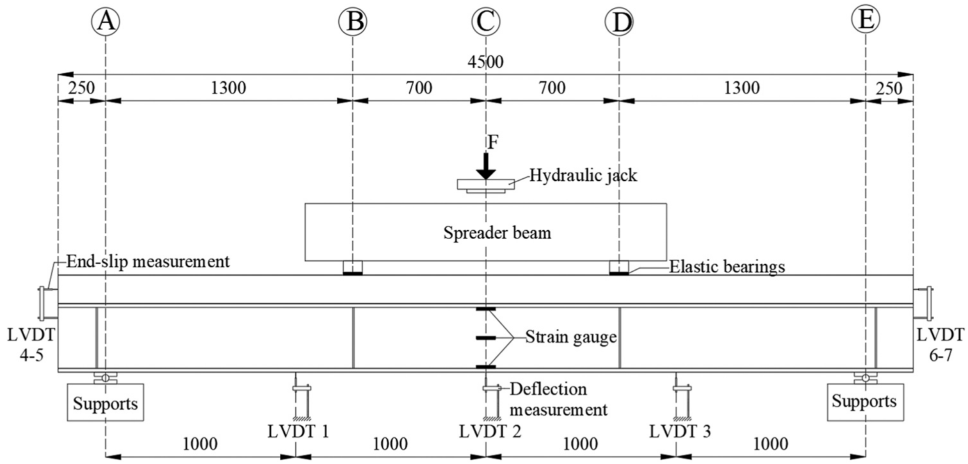


Fig. 12. Schematic view of the instrumentation and load configuration for the room temperature tests (dimensions in mm).

Fig. 13. Location of LVDT 6 and LVDT 7.

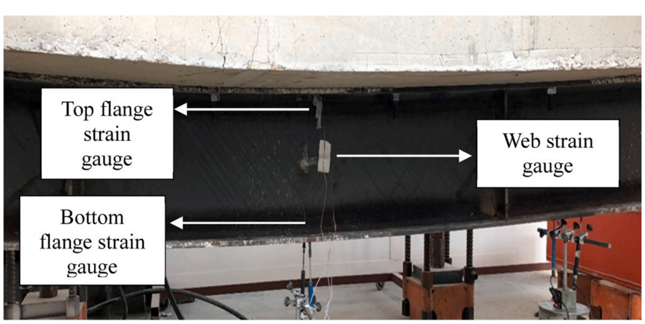


Fig. 14. Location of strain gauges.

Section 2.3.1 for the room temperature tests.

The beam, which was simply supported at its end-bearings, progressively lost stiffness and deflected due to the degradation of the mechanical properties experienced by the materials (i.e. steel and concrete) when subjected to the high temperatures reached inside the furnace chamber.

The criteria given in EN 1363–1 [28] were applied at the fire tests to determine the failure time, which is established by a limiting displacement value or a maximum deformation rate. The values obtained by applying these criteria were 86.47 mm and 3.84 mm/min, respectively.

Fig. 15. State of one of the composite beams after room temperature test.

2.4.3. Instrumentation

For each composite beam, 18 type-K thermocouples and one displacement transducer (LVDT) were applied to measure:

– The evolution of temperatures across the section
– The deflection of the beam at mid-span

The temperature evolution across the composite beam section was measured at three different locations along the beam, as given in Fig. 22; the exact positions of the thermocouples on each section with their respective designation are presented in Fig. 23. Additionally, a displacement transducer was attached to the hydraulic jack, placed outside of the furnace chamber, to measure the vertical deflection of the beam at mid-span.

2.4.4. Experimental results and analysis

2.4.4.1. Experimental observations. The state of the beams after the fire tests can be seen in Fig. 24. It is noticeable to observe that the timber panels were completely burnt out and fell off due to pyrolysis (which occurred during the first 20 min of fire exposure) and only the insulating mineral wool panels were visible when the tests were terminated and the furnace was opened.

The overall failure of the composite beams was due to bending, which was accelerated in the last stage of the fire tests due to the detachment of the mineral wool fire protection panels, causing the steel profile to be directly exposed to the fire. This issue was particularly

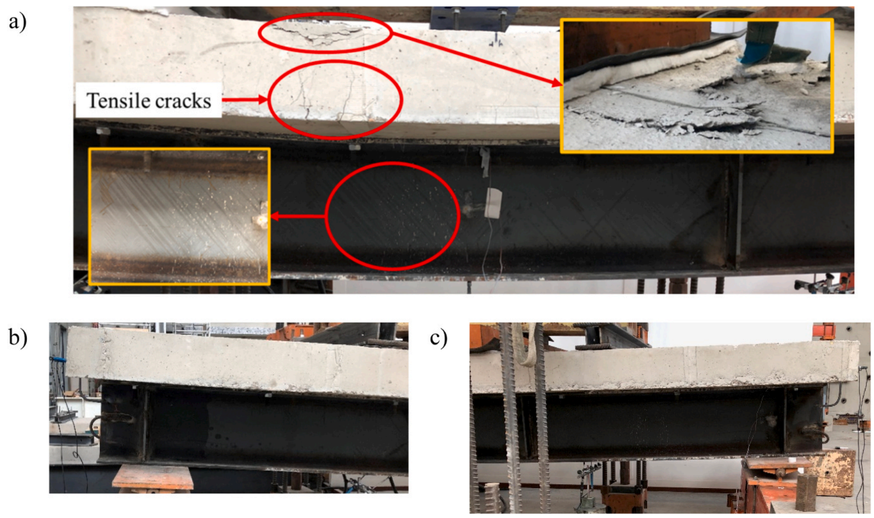


Fig. 16. Observed degradation mechanisms: a) crushing of concrete at the top surface of the slab, tensile cracks at the bottom surface of the concrete slab and plastic strains development at the web of the steel profile; b) and c) rotation of the beam ends.

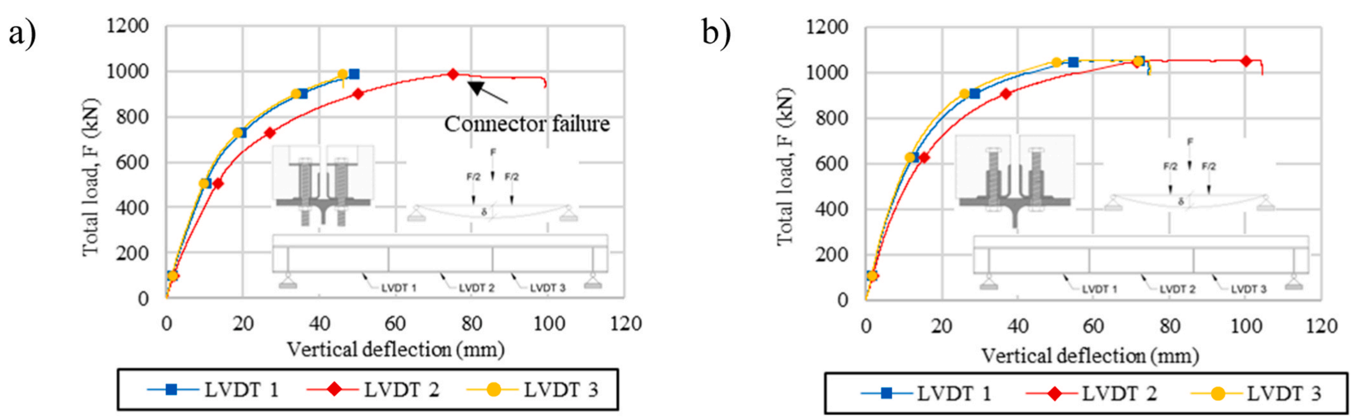


Fig. 17. Load-displacement curves: a) FIREDUCE-RT-P3; b) FIREDUCE-RT-P15.

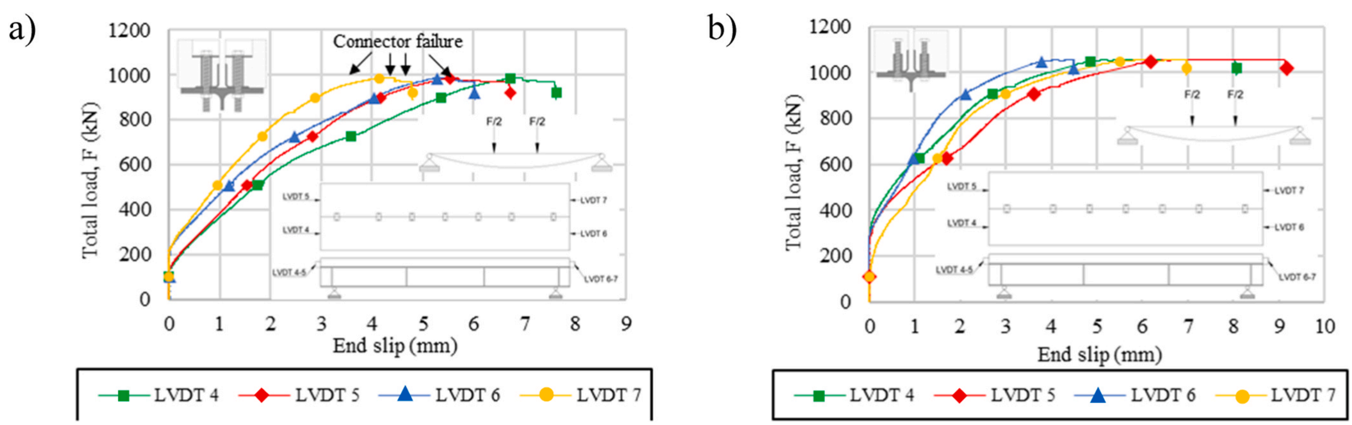


Fig. 18. Load-slip curves: a) FIREDUCE-RT-P3; b) FIREDUCE-RT-P15.

evident in the case of test FIREDUCE-FR-P15, which suffered a significant increase of the temperatures registered at the steel beam in Section 3 after 100 min of fire exposure, experiencing a sudden runaway failure.

It should be pointed out that, although the demountability concept of the composite beams is conceived for serviceability and not for a ultimate state such as the fire situation (accidental load), the steel beam and slab still had the potential to be detached after ultimate failure and the materials conveniently recycled.

2.4.4.2. Temperature evolution. It was observed from the thermocouple measurements that the temperatures were similar across the three monitored sections of the beam (Sections 1, 2 and 3), leading to the conclusion that there were no significant thermal variations along its length.

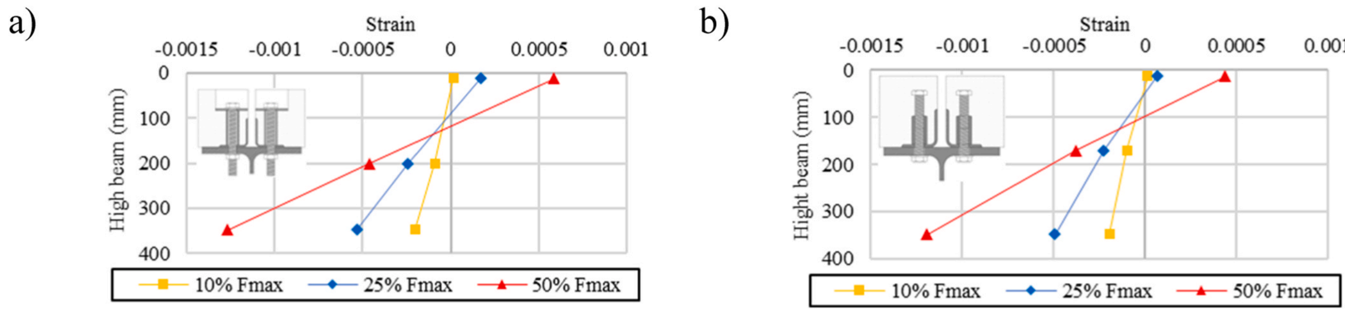
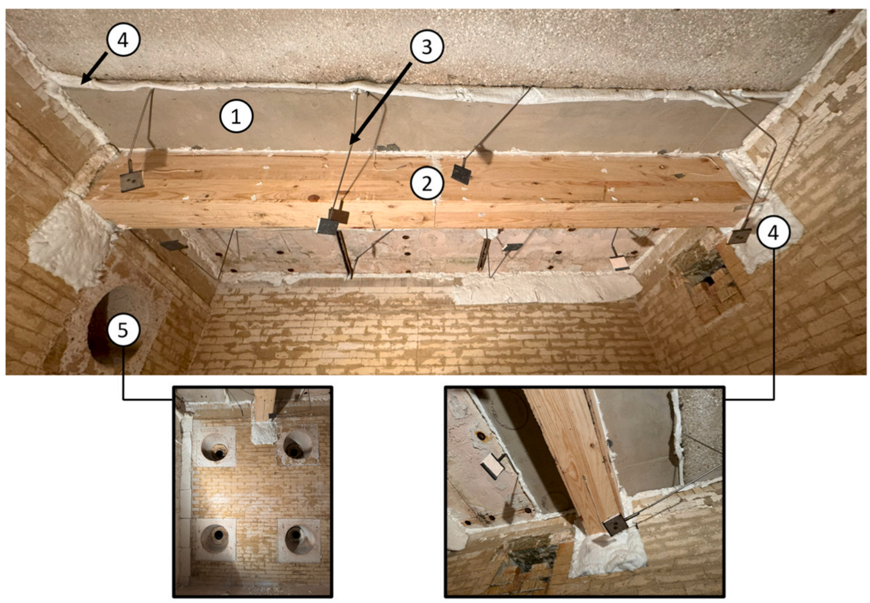


Fig. 19. Elastic deformations across the steel profile depth: a) FIREDUCE-RT-P3; b) FIREDUCE-RT-P15.



1	Concrete slab	4	Fibre blanket
2	Protected beam	5	Gas burners
3	Plate thermocouples		

Fig. 20. Fire test setup (inner view of the gas furnace).

1	Loading frame	4	Fibre blanket
2	Hydraulic jack	5	Type-K thermocouples
3	Spreader beam	6	Access gate to the furnace chamber

Fig. 21. Fire test setup (outer view of the gas furnace).

Table 4
Values of the load applied in the fire tests.

Test ID	Ultimate load, room temperature test (kN)	Applied load, fire test (kN)	Load level (%)
FIREDUCE-FR-P3	987.43	645.04	65
FIREDUCE-FR-P15	1056.63	683	65

Note that the applied load level ($\eta_{fi} = 0.65$) is a commonly assumed value in fire design, as per Clause 4.7(2) of EN 1993–1–2 [29].

The evolution of the thermocouple measurements at the timber panels, mineral wool and concrete slab can be seen in Fig. 25. Since the observed differences between the readings of TC-TL and MWU-MWL were negligible, only the measurements of TC and MWL are plotted in the figures for timber and mineral wool, respectively.

A it can be seen in these figures, a significant temperature difference was found between the relevant points of timber and mineral wool during the first 20 min of fire exposure, approximately (time when the timber boards had been completely burnt out due to charring, which agrees well with the charring rates given in EN 1995–1–2). Beyond that point, a sudden increase occurred at the mineral wool thermocouple measurements and both registered temperatures at the timber boards and mineral wool insulation started to evolve parallel. This indicates that the timber boards only acted as a first fire protection layer delaying the temperature rise of the section during the initial heating period upon charring, and from that point onward, the only remaining fire protection layer was the mineral wool insulation.

In turn, for the concrete slab, due to the low thermal conductivity of concrete, a significant temperature gradient was measured through its depth, in views of the measurements of thermocouples HS and HI installed at the top and bottom side of the slab, respectively.

Similarly to what was observed with the other materials analyzed, the steel profile exhibited a homogeneous temperature development across the three sections studied. The measurements obtained from the thermocouples located at positions TF-TF', BF-BF' and WU-WL of the steel profile showed almost identical results. For this reason, only the average values of the results of the thermocouple measurements at the relevant points of the steel section are presented in Fig. 26.

As it can be seen, the temperatures at the bottom flange and web of the steel profile were almost identical, however, the top flange had a significantly lower temperature, this was due to the presence of the concrete slab, causing a temperature gradient across the steel profile.

It should be noted that the plotted results of the experimentally measured temperatures correspond to the average temperatures obtained for the three thermocouple sections located along the beam for specimen FIREDUCE-FR-P3. However, for specimen FIREDUCE-FR-P15, the plotted temperatures correspond to Section 3, which was the one that eventually triggered the beam failure due to its sudden heating after the detachment of the mineral wool panels, as was commented before.

Fig. 27 illustrates the temperature gradient developed along the full height of the cross-section of the steel profile for the tested specimens, specifically for the configurations FIREDUCE-FR-P3 (a) and FIREDUCE-FR-P15 (b). This thermal gradient represents the temperature variation from the bottom flange to the top flange of the steel profile for a specific

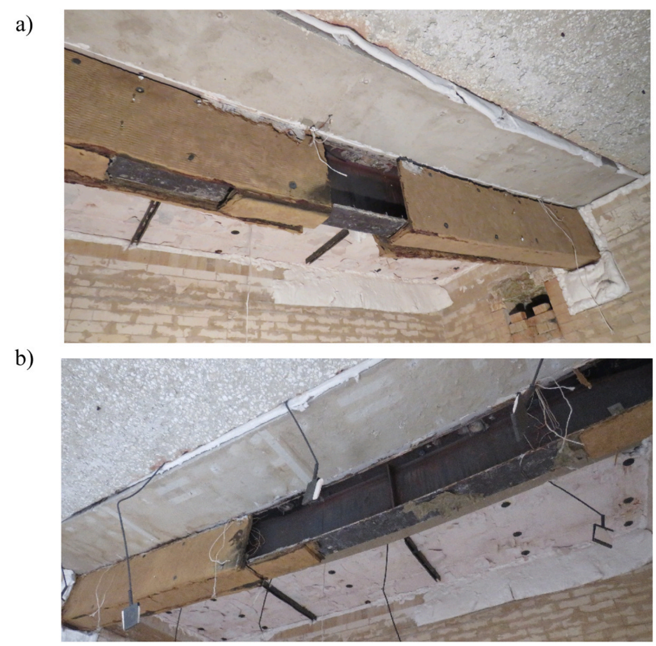


Fig. 24. State of the beams after the fire tests: a) FIREDUCE-FR-P3; b) FIREDUCE-FR-P15.

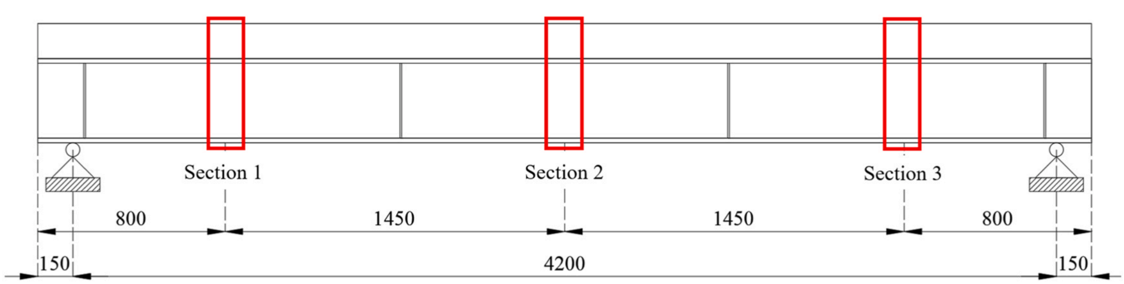


Fig. 22. Sections where the thermocouples were installed at the fire tests (dimensions in mm).

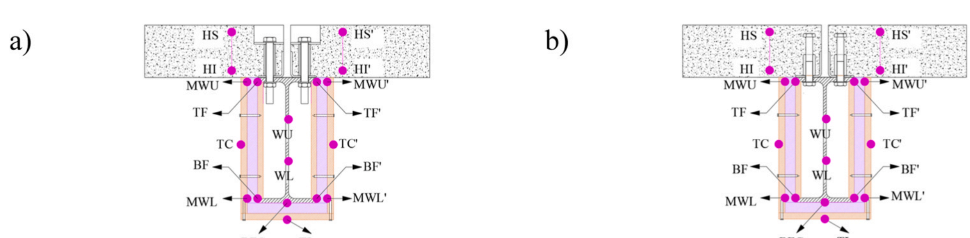


Fig. 23. Location of thermocouples in the cross-section: a) FIREDUCE-FR-P3; b) FIREDUCE-FR-P15.

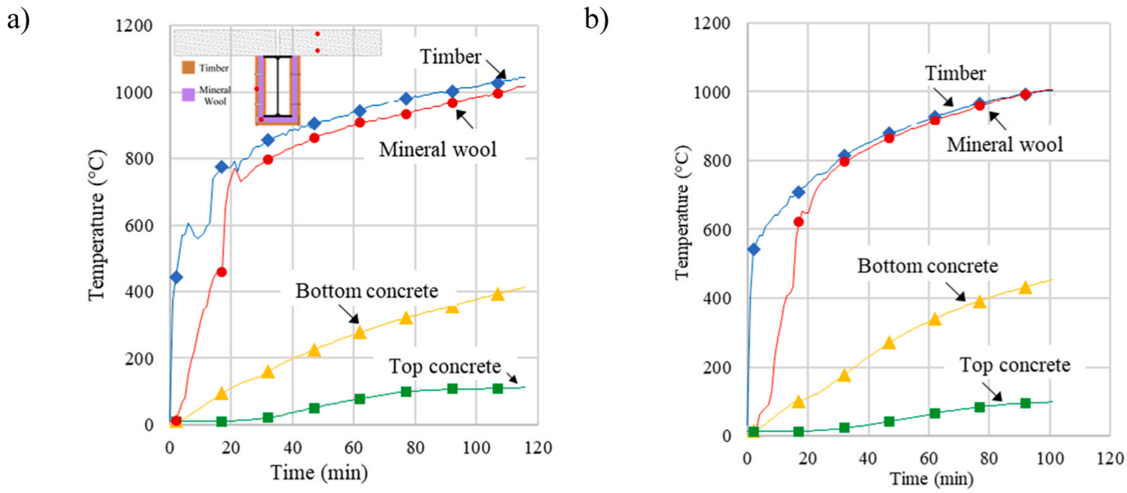


Fig. 25. Temperature-time curves for timber, mineral wool and concrete: a) FIREDUCE-FR-P3; b) FIREDUCE FR-P15.

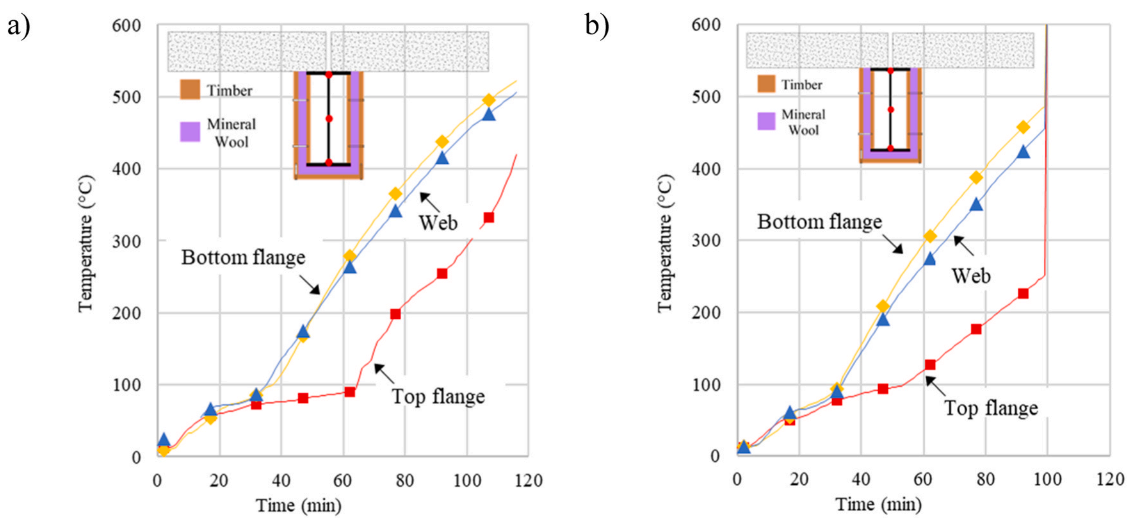


Fig. 26. Temperature-time curves for the steel beam: a) FIREDUCE-FR-P3; b) FIREDUCE FR-P15.

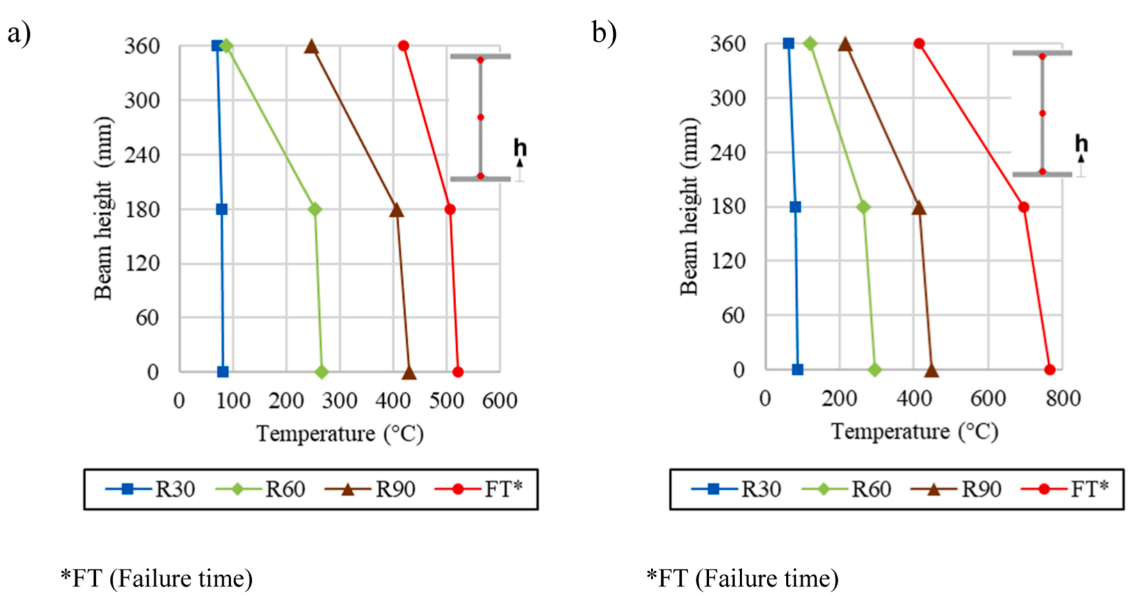


Fig. 27. Temperature gradient across the steel beam height: a) FIREDUCE-FR-P3; b) FIREDUCE-FR-P15.

fire exposure time. The temperature gradient at the standard fire times have been plotted (R30, R60 and R90), together with the one registered at the failure time (FT).

The results show that, for the tested beams, the higher temperatures are recorded at the bottom flange of the steel profile, while the top flange exhibits significantly lower temperatures. This thermal behavior is primarily attributed to the presence of the concrete slab on top of the steel beam, which, as previously mentioned, acts as a thermal barrier by delaying heat transfer along the web of the steel profile. Due to its considerable thermal inertia and relatively low thermal conductivity as compared to steel, the concrete slab slows down the rate of temperature increase at the upper portion of the beam, thereby creating a pronounced thermal gradient across the composite section.

2.4.4.3. Displacement-time curve. The recorded vertical displacement (measured at the LVDT attached to the hydraulic jack) along the fire exposure time is given in Fig. 28. As can be seen, the evolution of the displacement followed a controlled path, until after a certain point the displacement rate suddenly increased, leading to a runaway failure of the beams.

The criteria given in EN 1363–1 [28] were applied to determine the failure of the beam specimens. In the case of test FIREDUCE-FR-P3, the failure was due to the maximum displacement reached, while for test FIREDUCE-FR-P15, the criterion that determined failure was the deformation rate. In consequence, the obtained failure times for specimens FIREDUCE-FR-P3 and FIREDUCE-FR-P15 were 115 and 102 min, respectively.

It was confirmed that both test specimens outperformed the 90-minute fire resistance (being close to 120 min in the case of FIREDUCE-FR-P3 beam test), suggesting that the applied protection methodology for the composite beams is suitable for the most common situations in multi-storey buildings, meeting the requirements for a standard fire resistance R90.

The results obtained from the fire tests indicate that, up to 100 min, the displacement-time curves exhibited a nearly identical behavior for both tested beams. However, after the detachment of the mineral wool protection in the FIREDUCE-FR-P15 test, a significant increase of the displacement rate was observed. This phenomenon prevented this beam specimen from continuing its displacement evolution upon failure in a similar manner to specimen FIREDUCE-FR-P3. Should this detachment not have occurred, it could have been concluded that both beams exhibit a very similar behavior, providing equivalent fire endurance, and could potentially achieve standard fire resistance times of up to 120 min.

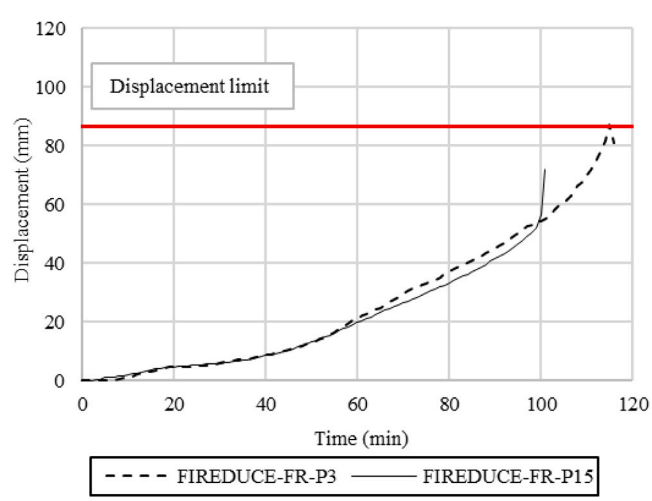


Fig. 28. Displacement-time curves recorded at the fire tests.

2.5. Discussion

Upon completion of the fire tests, the experimental results were analyzed. Through analytical calculations in accordance with EN 1994–1–2 [30], the fire-resistant moment ($M_{fi,Rd}$) of the composite beams was determined. The findings demonstrate that, due to prolonged exposure to elevated temperatures, there is a progressive degradation of the mechanical capacity of the composite beams, resulting in a reduction of $M_{fi,Rd}$ over time.

The analytical determination of $M_{fi,Rd}$ requires, among other parameters, the calculation of the shear resistance of the connectors (P_{Rd}). The design provisions in EN 1994–1–2 [30] provide a specific formulation for this purpose, which is valid only for traditional connectors (i.e. headed studs). However, since this study involves the use of demountable connectors, a modification to the design procedure is needed to account for their different mechanical behaviour. Specifically, the method proposed by Kozma [31] is adopted, which enables the estimation of P_{Rd} for this type of flexible connectors.

The degradation curve of the fire-resistant moment ($M_{fi,Rd}$), obtained theoretically by applying the principles of EN 1994–1–2 [30], is shown in Fig. 29. This curve makes it possible to identify the theoretical failure time as the intersection point between $M_{fi,Rd}$ and $M_{fi,Ed}$. As long as the fire-resistant moment ($M_{fi,Rd}$) remains greater than the applied moment under fire conditions ($M_{fi,Ed}$), the composite beam retains its load-bearing capacity. When $M_{fi,Rd}$ falls below $M_{fi,Ed}$, failure occurs. For the example shown in this figure, corresponding to beam specimen FIREDUCE-FR-P15, the theoretical failure time (93 min) is compared to the experimental failure time, which was recorded at 102 min (Fig. 28).

As it can be seen, the theoretical value obtained by applying the calculation method in EN 1994–1–2 [30] results more conservative than the experimental result, but gives a good estimate of the failure time.

Although this is a promising result, which validates the application of the current provisions in EN 1994–1–2 [30] to other types of non-traditional shear connectors as the ones used in this research, further analysis should be performed based on a wider range of experimental results and complementary parametric studies.

3. Conclusions and future work

In this paper, the experimental results of a series of bending tests on steel-concrete composite beams were presented, which served to analyse the behaviour of a particular type of demountable shear connectors under fire conditions and at room temperature. Two types of demountable bolted shear connectors were studied: the so-called cylinder system and the coupler system. A total of four experimental tests were carried out, two at room temperature and two under fire exposure inside a gas furnace, each pair of tests analysing the behaviour of a connector type.

The main conclusions drawn from this investigation are as follows:

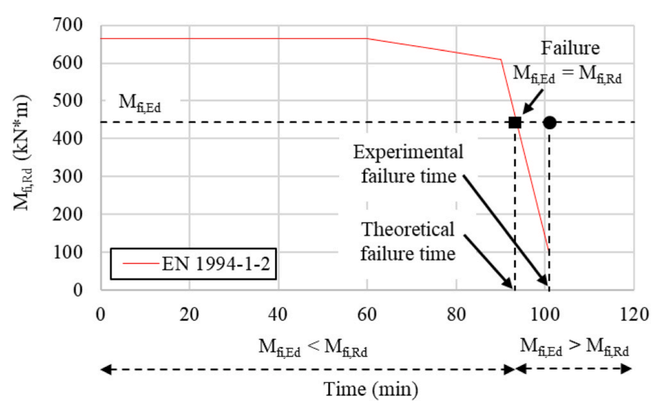


Fig. 29. Degradation of the resistant moment in the fire situation ($M_{fi,Rd}$).

- The experimental tests performed at room temperature showed that when the applied load is such that it leads to the maximum shear capacity of the connectors, there is a partial loss of stiffness of the composite beam, however, the rupture of the connectors occurs progressively from the beam ends - where the greatest shear stress occurs - towards mid-span. In addition, due to the nonlinear behaviour of the shear connectors, a relative slippage between the concrete slab and the steel beam occurs.
- The mechanical performance provided by the demountable bolted connectors demonstrated strong structural capacity. A similar behavior was observed when the two types of demountable shear connectors were compared, in terms of structural performance, deflections, and deformations under service conditions. Consequently, two viable options are provided to the construction industry, allowing practitioners to select the connector type that best meets the design requirements.
- Despite the lack of specific regulations and design guidance for determining the fire-resistant bending moment in composite beams using this type of shear connectors, analytical methods already exist for traditional shear studs that enable equivalent calculations. To this end, the design provisions in EN 1994–1–2 were tentatively applied to predict the failure time of these composite beams with flexible connections under fire conditions, showing a good approximation.
- The experimental fire tests presented in this research showed that it is relevant to consider adequate passive fire protection material to ensure significant fire resistance of the composite beams. The fire protection system implemented in the tested beams demonstrated high effectiveness, easily surpassing 90 min of fire endurance. Therefore, the proposed structural system can be suitable for applications that require to meet a standard R90, as it is commonly prescribed for multi-storey buildings.
- According to the results obtained in the fire tests, the composite beams were close to achieve 120 min fire resistance, although they did not reach to this value. To optimize the fire performance of the composite beams, an adjustment of the fire protection dimensions would be recommended, which might be achieved through further parametric studies based on numerical models, to be developed in the framework of this investigation. Based on these future studies, the applicability of the proposed fire protection system will be potentially extended to a wider range of building typologies.

The complete characterization of the fire performance of the studied composite beams is highly demanded by the construction industry for the implementation of the demountable shear connection system in real practice to facilitate the “design for disassembly” concept, in response to the circular economy needs.

Thus, it is essential to continue this research, by evaluating the suitable parametric variations through numerical modelling. Additionally, it is crucial to develop an analytical methodology that allows designers to easily determine the flexural capacity of the studied composite beams with demountable shear connectors under fire conditions, as there is currently no specific regulation addressing this aspect.

CRediT authorship contribution statement

P.S. Mora: Conceptualization, Methodology, Investigation, Validation, Formal analysis, Writing- Original draft preparation. **A. Espinós:** Conceptualization, Methodology, Investigation, Validation, Formal analysis, Writing- Original draft preparation. **C. Odenbreit:** Conceptualization, Supervision, Resources. **M.L. Romero:** Conceptualization, Methodology, Investigation, Supervision, Writing- Review & Editing, Funding acquisition, Project administration.

Declaration of Competing Interest

The authors declare the following financial interests/personal relationships which may be considered as potential competing interests: Manuel L. Romero reports financial support, administrative support, article publishing charges, equipment, drugs, or supplies, and travel were provided by State Agency of Research. Manuel L. Romero reports a relationship with Engineering Structures, Elsevier that includes: board membership. Prof. Manuel L. Romero is an Associate Editor of this journal. Dr. Ana Espinós is a Guest Editor of the Special Issue “Advances in Structural Fire Safety” from this journal. In accordance with the editorial policy, both authors were blinded to the entire peer review process If there are other authors, they declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data Availability

Data will be made available on request.

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