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BEHAVIOUR OF COMPOSITE SLIM FLOOR STRUCTURES IN FIRE

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ABSTRACT In recent years, increasing interest has been shown throughout Europe in developing and designing slim floor systems in steel-framed buildings. This paper presents the fire resistance behaviour of the composite asymmetric slim floor beam as an isolated member and as a part of the frame using numerical analysis methods. Three schemes were investigated, including isolated beams, a plane subframe with semi-rigid beam-to-column connections and a three-dimensional slim-floor frame system. The first scheme aimed to explore the fire resistance of the beams according to standard fire-testing methodology. The objective of the second scheme was to reveal the effect of frame continuity on the fire resistance of the slim floor beam and the mechanical interaction between the frame elements. The third scheme was to preliminarily identify the influence of the composite slab on the beam behaviour in fire. The investigations show that the isolated slim floor beam has a 60-minute standard fire resistance without any additional fire protection, if the load ratio is less than 0.5. As a part of the frame, the beam still keeps its stability even when the temperature of the bottom steel flange of the beam reaches up to 900 °C (90 minutes' ISO fire exposure). The analyzed results indicate that the axial restraints provided by the surrounding parts cause a larger deformation of the beam in the earlier ISO heating phase and, however, a more stable behaviour thereafter. The rotational restraints essentially cause the change in the applied load ratio in fire, which can be quantified using the 'modified load ratio' proposed in this paper.

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INTRODUCTION

In recent years, increasing interest has been shown throughout Europe in developing and designing shallow floor systems in steel-framed buildings. In the shallow floor system, the steel beam is contained within the depth of the pre-cast concrete floor or composite slab with profiled steel decks. This form of construction achieves a minimum depth of building and the flat floor is beneficial because the building services can be run in any direction. The key feature of this system is the steel beam, which can be either rolled or welded sections (see Fig. 1). One of the original slim floor concepts developed in Scandinavia was the 'Thor-beam', which consists of two channel sections welded to a flat plate. The slim floor system using a 'Delta-beam', which consists of a welded beam section and a prefabricated concrete slab, is popular in the Scandinavian Countries (Lu and Mäkeläinen 1996). British Steel, in collaboration with the Steel Construction Institute (UK) developed a Slimflor® beam, which consists of a universal column section welded to a steel plate (Newman 1995; Mullet and Lawson 1993). Recently, interest has been concentrated on the asymmetric hot-rolled steel beam in the UK (Lawson and Mullet 1997), and on the asymmetric welded steel beam in Finland (Ma and Mäkeläinen 1999; Malaska and Mäkeläinen 1999) (see Fig. 2).

Generally, the slim floor beam provides more opportunities for steel in spans of 5 - 9 m. It achieves a slab depth of 300 mm or so, which is much less than in conventional steel construction. This system also has an inherently good fire resistance, due to the partial encasement of the steel beam within the concrete slab. On the other hand, compared with the conventional composite frame system, which has a primary-secondary-beam system, the slim floor frame has a rather precise structural form and therefore offers significant savings in construction cost. In slim floor construction, the slab is sustained directly by the primary beam, forming a part of the composite beam that works together with the steel beam. Between the rows of a single frame, the tie members are employed to link them together to keep the out-of-plane stability of the frame.

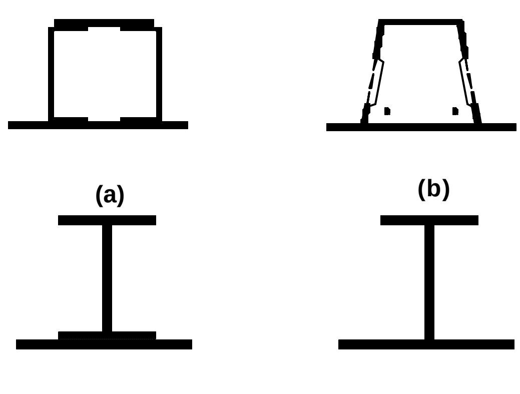

FIG. 1. Different Types of Slim Floor Steel Beams: (a)Thor-Beam; (b)Delta-Beam; (c)Slimflorª Beam; (d) Asymmetric Slim Floor Beam

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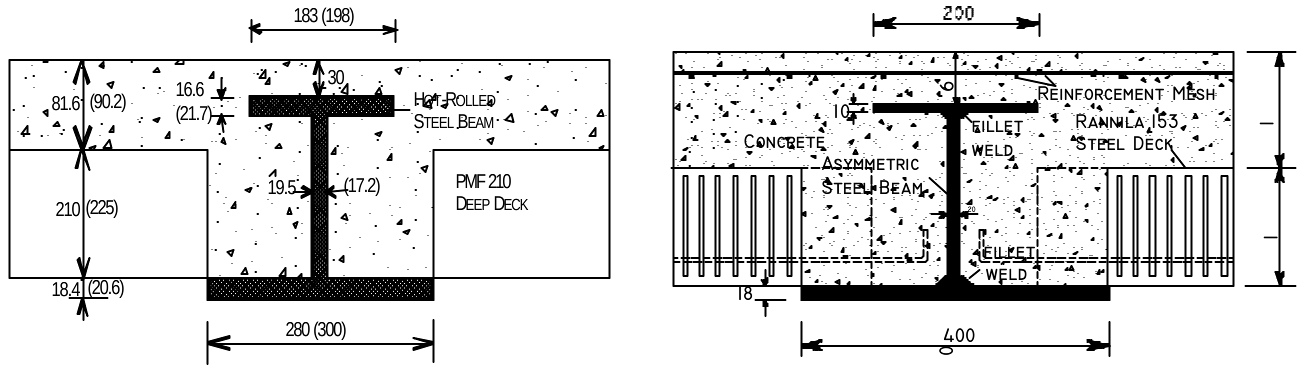


(a) (b)

FIG. 2. Composite Sections of Asymmetric Slim Floor Beams: (a) British 280(300) ASB Beams; (b) Finnish Asymmetric Slim Floor Beam

Traditionally, the fire resistance of constructional steelwork has been investigated by standard fire tests. The additional fire resistance was achieved by applying fire protection to the steel members. Since the late 1980s, fire engineering has rapidly developed, especially in Europe, to the point where buildings are designed with significant built-in fire resistance. 5
Following the investigation of a series of fire events, such as the Broadgate Fire (UK), William Street (Australia) and Washington (USA), as well as the full-scale fire tests in Cardington (UK), it was concluded that the traditional standard fire test on isolated structural components was very conservative. Future research into the structural behaviour of steel-framed buildings subject to fire should consider the structures as a complete entity and not as a collection of isolated members (Johnson 1998; Robinson 1998; Bailey 1997; Newman 1996).

The aim of this paper is to explore the fire resistance of the composite asymmetric slim floor beam both as an isolated member and as a part of the frame using numerical analysis methods. The effect of frame continuity on the behaviour of a fire-exposed beam, and the moment redistribution and axial force variation among the frame members are investigated. The role of the floor slab in fire is also preliminarily identified. The procedure presented here can 3
also be used in fire engineering design of other steelwork products.

BEHAVIOUR OF ASYMMETRIC SLIM FLOOR BEAMS IN FIRE

Model Description

The modelling of fire-exposed structures includes both the thermal and structural aspects. The modelling of thermal response here is based on the computer program, TACS-FIR (Temperature Analysis of Composite Structures exposed to FIRe) (Ma and Mäkeläinen 1999), which is purposely developed for the temperature analysis of composite structures in fire. The explicit forward difference method was used in this program. The major assumptions used in the analysis are:

- The thermal properties of steel and concrete are taken from Eurocode 4 Part 1.2 ("Eurocode 4" 1993);
- The resultant emissivity for exposed steel is 0.6 and 0.3 for the composite slab with steel decking;
- The convection factor is 25 W/m^{2}K for the exposed side and 8 W/m^{2}K for the unexposed side;
- The interface resistance between the concrete and steel is 50 W/m^{2}K.

The structural response was modelled by the general finite element program, ABAQUS (*ABAQUS/Standard* 1997). The composite slim floor beam was represented by combining a shell element (S8R) and a beam element (B32). The asymmetric steel beam was modelled by 3-node beam elements with 6 degrees of freedom at each node. The concrete slab was modelled using 8-node thick shell elements, also with 6 degrees of freedom at each node. The reinforcements were modelled using REBAR elements encased in the concrete shell elements. In this study, the concrete was considered to be an elastic-plastic material, which has a plastic plateau after reaching the compressive and tensile strength. No descending phase in compression and in tension was taken into account. The tensile strength of the concrete was taken as 10 percent of its compressive strength.

TABLE 1. Material Properties for Structural Modelling

Steel		Concrete	
Yield strength (N/mm^2) (1)	Thermal expansion (°C^{-1}) (2)	Cubic strength (N/mm^2) (3)	Thermal expansion (°C^{-1}) (4)
355	1.4×10^{-5}	30	1.8×10^{-5}

The Newton-Raphson iterative technique was used for the solution process. Both physical and geometrical nonlinearity were included in the modelling. The temperature distribution and variation with time of the composite section were introduced according to the thermal modelling. The temperature-dependent stress-strain curves of the steel and concrete were taken from Eurocode 4 Part 1.2 ("Eurocode 4" 1993). The material properties for the structural modelling are shown in Table 1.

British ASB SLIMDECK Beams

A SLIMDECK system using an asymmetric Slimflor beam (ASB) was developed in UK and two standard fire tests were conducted by the Warrington Fire Research Center (WFRC) (Lawson and Mullet 1997). These tests consisted of two loaded beams spanning 4.5 m, which were heated according to the ISO standard fire curve. The section size is shown in Fig. 2(a) and

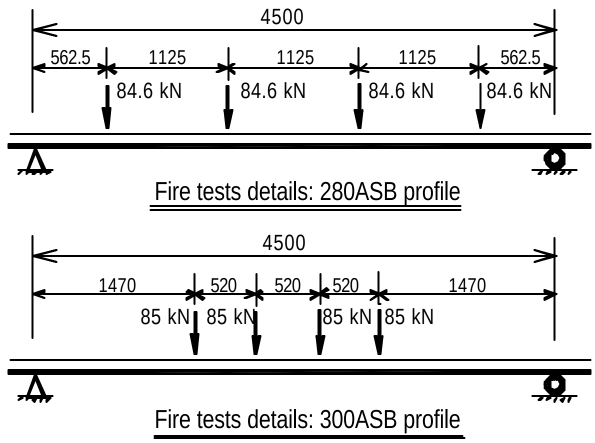


FIG. 3. Load Arrangement of SLIMDECK Beams Tested by WFRC (Lawson and Mullet 1997)

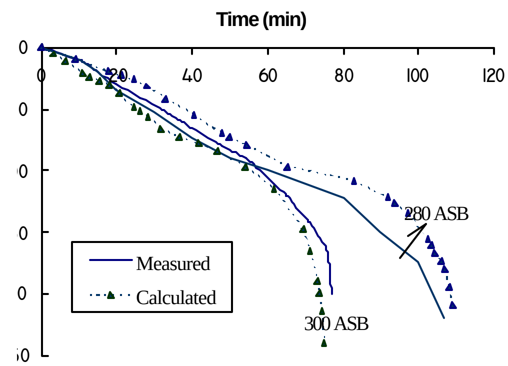


FIG. 4. Mid-Span Vertical Displacement of British SLIMDECK Beams in ISO Fire

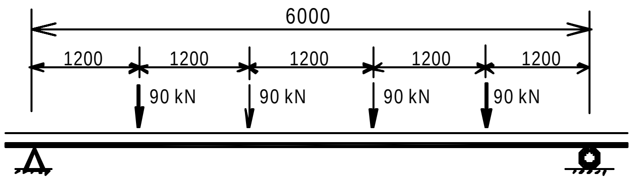
the loads and structural details are shown in Fig. 3. The steel grade is S355 and the concrete grade is NWC C30/25. The load ratio is calculated as 0.36 for the 280 ASB beam and 0.43 for the 300 ASB beam.

Fig. 4 shows the displacement-time curves of the tested slim floor beam. Both of the tests were terminated when the load-bearing criteria specified in BS 5950: Part 8 were exceeded. In the 280 ASB test, this occurred at a deflection of span/20 after 107 minutes. In the 300 ASB test, this occurred at span/30 after 75 minutes, as the allowable rate of deflection was exceeded before a deflection of span/20 had been reached. The predicted curves by FE analysis are also illustrated in this figure. It can be seen that there is a good correlation between the predicted and the measured curves.

Table 2 summarizes the measured and predicted results. It can be seen that the calculated bottom-flange temperature is very close to the measured value, with only an average difference of less than 30 °C. The predicted fire resistance by FE analysis is also very close to the measured value. However, the predicted fire resistance according to the general rules in Eurocode 4 (numbers in brackets) is lower than the testing time.

TABLE 2. British SLIMDECK Beams: Measured and Predicted Results

Test and Calculated Data	280 ASB			300 ASB		
(1)	Test (2)	Predicted (3)	Predicted (4)	Test (5)	Predicted (6)	Predicted (7)
Applied Moment (kNm)	200			310		
Design Moment Resistance at ULS (kNm)	557			714		
Applied Load Ratio	0.36	0.36	0.5	0.43	0.43	0.5
Fire Resistance (min)	107	109(93)	66(60)	75	74(65)	61(55)
Bottom Flange Temp. at Failure (°C)	869-987	841-1004	650-856	727-861	735-908	675-825



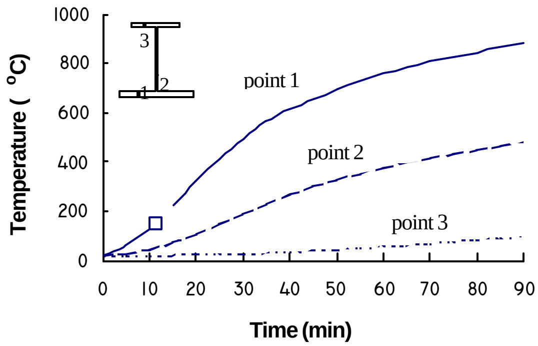


FIG. 5. Load Arrangement of the Finnish Slim Floor Beam

FIG. 6. Temperature Variation of Finnish Beam in ISO Fire

In practicality, the most common load ratio is between 0.5–0.6 when buildings are subject to a fire attack. Because the test load ratios were less than this, a further investigation was carried out to obtain the allowed maximum load ratio for 60 minutes' fire resistance. The FE analysis results indicate that the 60 minutes' fire resistance can be obtained where the load ratio is less than 0.5 (see also Table 2).

Finnish Asymmetric Slim Floor Beam

The Finnish asymmetric slim floor steel beam uses the welded section. The web plate is welded to the bottom and top flange using the submerged arc-welding method. The section shape of the composite beam is shown in Fig. 2(b). The load arrangement of the investigated beam is shown in Fig. 5. This beam is uniformly four-point loaded and simply supported. The applied load ratio is 0.53.

The temperature variation of the steel beam with time is shown in Fig. 6. It can be seen that there exists a large temperature gradient in the beam section due to the encasement of concrete. The average temperature in the bottom flange at 60 minutes is 760 °C and only 50 °C in the upper flange. In this context, the structural response is described against the ISO fire exposure time. Certainly, it is also reasonable for this to be given against the bottom flange temperature. This transformation can be easily made according to Table 3.

TABLE 3. Relationship Between ISO Fire Exposure Time and Temperature of Bottom Flange

ISO Time (min)	0	15	30	45	60	75	90
Bottom Flange Temp. (°C)	20	220	500	650	760	830	890

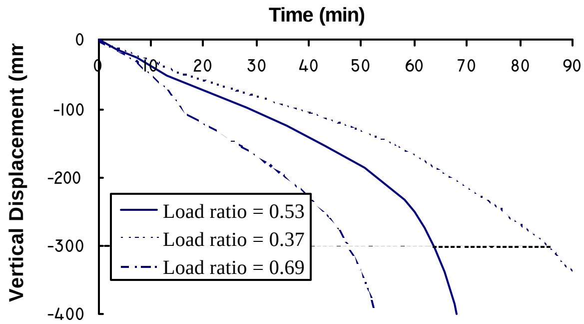


FIG. 7. Mid-Span Vertical Displacement of Finnish Beams in ISO Fire

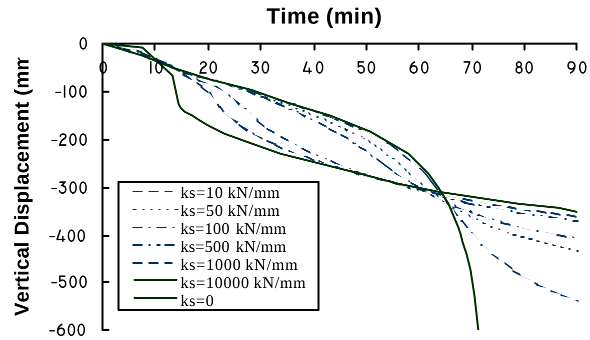


FIG. 8. Effect of Constant Axial Restraints on the Beam Behaviour in Fire

TABLE 4. Analysis Results of Finnish Composite Slim Floor Beam

Numerical Analysis Data (1)	Load Ratio in Analysis 0.37 (2)	0.53 (3)	0.69 (4)
Applied Moment (kNm)	226	324	422
Design Moment Resistance at ULS (kNm)		618	
Moment Resistance at ULS (FE analysis) (kNm)		705	
Predicted Fire Resistance by FE Analysis (min)	86	62	47
Predicted Fire Resistance by Eurocode 4 (min)	80	55	43
Bottom Flange Temperature at Failure (°C)	740-950	660-880	550-740

Fig. 7 shows the vertical mid-span displacement versus the heating time. According to BS 5950: Part 8, the 'failure' point corresponds to the deflection reaching up to 1/20 span length, or to the critical deflection rate at a deflection of span/30 ("BS 5950" 1990). From Fig. 7, it can be seen that 60-minutes' fire resistance can be achieved if the load ratio is less than 0.53. In this figure, the analyzed vertical displacement responses under varying load ratio (0.37 and 0.69 respectively) are also illustrated.

Table 4 summarizes the relevant results from the modelling. In this table, the design moment capacity of the beam at ultimate limit state (ULS) at ambient temperature was calculated with the material safety factor specified by Eurocode 4 Part 1.1 ("Eurocode 4" 1992). Nevertheless, since fire is regarded as an accidental action in the Eurocodes, the moment resistance in fire was calculated using the characteristic values of the material properties. The effective width of the concrete slab was taken as 1/8 span length in calculating the sectional moment capacity, which was proposed by Lawson and Mullet (1997).

The bending-moment capacity method was used to investigate the contributions of the beam section parts to the fire resistance. The beam cross-section is divided into five parts: bottom steel flange, top steel flange, lower part of web, upper part of web and compression part of the concrete slab cross-section. Here, the lower part of web means the part of web below the plastic neutral axis (p.n.a), and the upper part of web is that part above the plastic neutral axis. In Table 5, the contributions of the beam section parts to the total moment capacity at 60 minutes are compared with those at ambient temperatures. Due to the elevated temperature, the moment capacity contribution of the bottom flange is reduced to 31% from the original value of 44% at ambient temperature. The lower part of the web contributes 53% of the total moment capacity at 60 minutes fire while this value is 6% at ambient temperature. It can be seen that the web contributes a major part to the total moment capacity in fire while the bottom flange has a major contribution at ambient temperature. The top flange has a slight contribution to the moment capacity both at ambient and at elevated temperatures because of its position close to the p.n.a. At 60 minutes under ISO fire, the concrete above the top steel flange still has a very low temperature (≤70 °C) and the full strength can be expected. The contribution by this part to the total moment capacity is approximately 14% at 60 minutes' fire and 30% at ambient temperature.

TABLE 5. Moment Capacity Contributions for the Section Parts (at 60 minutes under ISO Fire) (Numbers in Brackets are at Ambient Temperature)

Section Parts (1)	Temperature (°C) (2)	Centroid Distance from p.n.a. (mm) (3)	Axial Resistance (kN) (4)	Moment Resistance (kNm) (5)	Percentage of Total Moment Capacity (%) (6)
Bottom Flange	650~870	235 (118)	399 (2324)	94 (274)	31 (44)
Lower Part of Web	85 ~650	115 (54.5)	1411 (702)	162 (38)	53 (6)
Upper Part of Web	65~85	2 (60.5)	28 (783)	-- (48)	-- (8)
Top Flange	45 ~65	9 (116)	710 (646)	7 (75)	2 (12)
Concrete Slab	≤100	37 (106)	1110 (1730)	41 (183)	14 (30)
Total Moment Capacity (kNm)				304 (618)	

Effect of Axial Restraints

Fig. 8 shows the effect of axial restraints on the behaviour of the beam in fire. For simplicity, the spring is assumed to be elastic and well protected to keep a constant stiffness in fire. Two issues can be seen from the results:

- Before a certain time (say, 60 minutes in this example), the increasing axial restraint stiffness causes the larger mid-span vertical displacement.
- After that, the displacement becomes more stable and the post-behaviour improves with the increasing axial stiffness.

This phenomenon is caused by the secondary P-Δ effect and the catenary action at large displacement. During the early ISO fire phase, a significant axial compressive force in the beam occurs due to the rapid thermal expansion. The stiffer the axial restraint, the bigger the axial force. Meanwhile, the significant thermal bowing caused by the large temperature gradient within the beam section enlarges the beam deflection. However, with the increasing deflection, the catenary action begins to take effect, which leads to a rather stable deformation behaviour afterwards.

It can also be seen that the fire resistance time happens to be almost the same for the investigated axial stiffness at a displacement of span/20. However, it is evident that the beam with an axial spring of higher stiffness has more stable deformation behaviour versus the ISO fire exposure time. As we know, the failure criterion for isolated members in the standard fire tests is defined in terms of displacement, and is set at values which prevent damage to the furnace during testing ("BS 5950" 1990). Generally, there is no conflict with the philosophy in the Eurocodes for isolated members at this point. In the Eurocodes, fire is regarded as an accidental action and the fire engineering design criterion is to keep the structural stability and integrity during and after fire attack. However, the conflicts are obvious when the members are considered a part of the structures, and it is therefore arguable whether the displacement criterion is still valid when the members are considered a part of the building structures.

MECHANICAL INTERACTION BETWEEN SLIM FLOOR SUBFRAME ELEMENTS

Analyzed Subframe and Loads

The analyzed one-bay subframe is shown in Fig. 9. The span is 6 m and the level height is 3 m. The beam was the Finnish composite slim floor beam. The concrete slab was 1.5 m wide and 147 mm thick. The yield strength of the steel was 355 N/mm^2 and the concrete grade was NWC 30/25. The anti-crack reinforcement mesh A142 was incorporated into the concrete slab and the yield strength was 235 N/mm^2. The universal steel columns were filled between the steel flanges by the aerated concrete blocks, which were considered to be non-structural. The composite beam was connected with the columns via the steel beam.

The fire was located in the lower level of the frame and followed the ISO standard curve. The beam was heated from the lower side and the columns in the lower level were assumed to be heated on all sides. The temperature distribution and variation with time were calculated using computer program, TACS-FIR. The columns in the upper level were not attacked by fire.

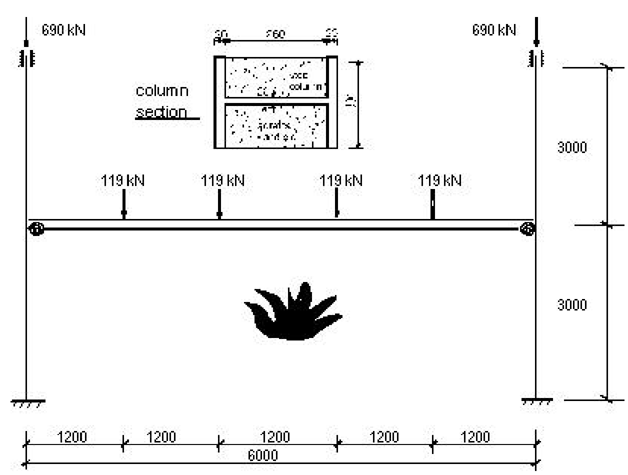


FIG. 9. Analyzed One-Bay Subframe and Loads

The applied load ratio for the beam was 0.69 in the case of the pinned beam-to-column connection. The columns were also loaded to simulate a realistic situation. The applied axial force was 690 kN and the according load ratio for the lower columns was around 0.2.

Deformation Behaviour of Frame Beam and Modified Load Ratio

The subframe was studied under the same load value with different connection stiffness. Three cases were studied: pinned steel beam-to-column connection, semi-rigid connections with rotational stiffnesses being 600 and 2 000 kNm/rad, respectively. Fig. 10 shows the deformation behaviour of the beam with different rotational stiffness in fire.

Compared with the isolated simply supported beam, the beam in the subframe with the pinned beam-to-column connection has more stable behaviour during fire. In the earlier phase, the beam in the frame has a larger deflection than the isolated beam. This is caused by the axial restraint. Since the thermal expansion is restrained by the column, a compressive axial force is initiated in the beam. Apparently, the second-order P-Δ effect enlarges the deflection of the beam. However, the behaviour of the beam becomes more stable with heating time.

With the increasing rotational connection stiffness, the vertical deflection of the beam becomes smaller. In essence, the semi-rigid connection changes the practical load ratio applied to the beam. For the semi-rigid beam-to-column connection, the formulae of the load ratio calculation for the simple beam should be modified. According to the definition of load ratio, the following equation is suggested:

$$R = \frac{M_{static}}{M_{P+} + min(M_{P-}, M_{con})} \tag{1}$$

Where M_{static} is the isostatic (free) bending moment in the beam, M_{p+} and M_{p-} are the plastic moments of the sagging section and hogging section, respectively, and M_{con} is the minimum value of the plastic bending moment of the connection and the applied moment to the connection at the limit rotation θ_{lim}. For a beam in the frame, M_{con} can be given by

$$M_{con} = \frac{1}{2}(M_{con,left} + M_{con,right}) \tag{2}$$

where $M_{con,\ left}$ and $M_{con,\ right}$ are the bending moment capacity of the connections in the left end and in the right end of the beam, respectively.

The feature in Eq. 1 is that it accounts for the realistic rotational restraints in beam-ends when the beam is a part of a structure. It also covers the cases for pinned (M_{con}=0) and fixed (M_{con}>M_{p-}) end restraints. Using Eq. 1, the modified load ratio of the investigated beam is shown in Table 6. In the study, M_{con} means the applied moment on the connection at the limit state at ambient temperature, which was obtained here by FE analysis. In the practical design, M_{con} and θ_{lim} can be calculated according to the design codes when the connection details are given.

TABLE 6. Modified Load Ratio for Semi-Rigid Connection

Analyzed Data (1)	Rotational stiffness of connection (kNm/rad) 0(pinned) (2)	600 (3)	2000 (4)
Isostatic bending moment	427	427	427
Sagging plastic moment M_{p+}	618	618	618
Hogging plastic moment M_{p-}	345	345	345
Connection moment M_{con}	0	84	178
Modified load ratio	0.69	0.60	0.53

Fig. 10 also shows the mid-span displacement of the beam (with pinned connection) using the modified load ratio. The solid lines in the figure represent the displacement of the beam with specific connection conditions, and the dashed lines represent the displacement of the beam with a pinned connection under the modified load by Eq. 1. It can be seen that there is a good similarity between these two results. This similarity indicates that the deformation behaviour of the beam with rotational restraints in fire is similar to that with pinned restraints under the modified load. The effect of rotational restraints on the beam fire resistance is consequently quantified. This method provides an easy way to calculate the fire resistance of the beam which has the rotational restraints in the beam-ends.

Moment Redistribution and Axial Force Variation

The stiffness of the frame component decreases during fire because the elastic modulus of structural steel and concrete reduce significantly at elevated temperatures. In most cases, the fire is limited within one or several rooms or on one level such that only a relatively small part of the whole structure is attacked by fire with the according deterioration of stiffness. This local deterioration of component stiffness changes the original stiffness ratio and results in the moment redistribution in the structural components. Another notable factor that affects the moment and force in the structural components is the thermal expansion of the heated structural components.

It is important to recognize the variation of moment and axial force in the frame components during fire. It is helpful to understand the deformation behaviour and failure mechanism of a fire-exposed structure. Moreover, the design details can therefore be enhanced and the potential premature failure of elements may be avoided. Fig. 11(a) shows the moment variations of a semi-rigid joint in fire. In this study, the rotational stiffness of the beam-to-column connection is 600 kNm/rad. It can be seen that the end moments of the upper and lower columns increase rapidly, by up to around three times the original values in the first 15 minutes, before starting to decline, while the beam-end moment continues to increase during fire.

Fig. 11(b) shows the change in the beam axial force in fire. In the first 15 minutes, the axial force increases rapidly due to thermal expansion. After that, the axial force begins to

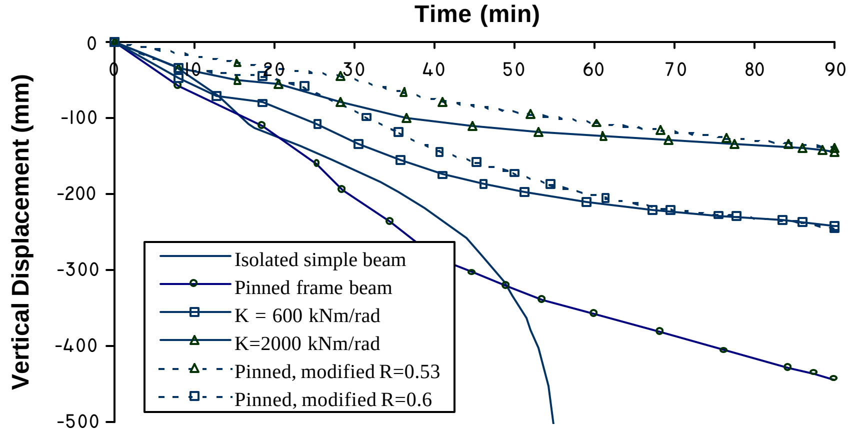


FIG. 10. Mid-Span Displacement of Beam with Semi-Rigid Beam-to-Column Connection

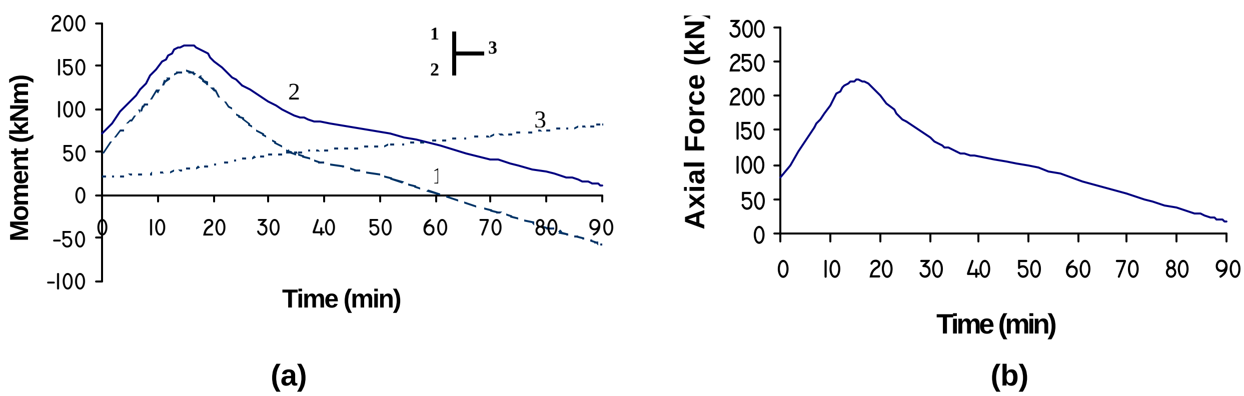


FIG. 11. (a) Moment Variation of Beam and Columns; (b) Axial Force Variation of Frame Beam

decrease. This decrease may be caused by the reduction in axial restraint stiffness with time (softening of columns), the reduction in the axial beam stiffness and the large displacement in the beam (catenary action). Up to 90 minutes, the axial force in the beam becomes very small.

Comparing Figs. 11(a) and (b), it can be seen that the variation in column-end moments during fire is very similar to that of the axial force in the beam. It can be surmised that the change in the column-end moments are mainly caused by the axial force in the beam, or the thermal expansion of the beam. The large push-force by the thermal expansion of the beam initiates the significant moment variation in the column-ends. Another factor affecting the moment variation is the thermal bowing in the beam. As mentioned before, for the slim floor beam, a large temperature gradient occurs in the beam section in fire. The beam tends to bend towards the fire-exposed side due to the greater expansion of the lower part of the beam. Because of the flexural continuity at the beam-ends, this deformation is resisted and the end moments are developed in the beam-ends. The third factor contributing to the moment variation is the variation in the stiffness ratio among the structural members during fire. This variation might lead to a moment redistribution.

Although the axial restraints are somewhat advantageous to the stability of a fire-exposed beam, the large push-force due to the thermal expansion is detrimental to the stability of the columns. The P-Δ effect can reduce the fire resistance of the column, especially the side-columns. Moreover, the large axial force in the beam, in combination with the hogging moment, is prone to cause local buckling in the beam-ends. This buckling will further lead to a larger vertical displacement of the beam. Nevertheless, this will not occur for the slim floor beam because most parts of the steel beam are encased within concrete.

STRUCTURAL RESPONSE OF A THREE-DIMENSIONAL SLIM FLOOR FRAME

Frame and Loads

The analyzed composite steel-frame building was designed to resemble one part of a typical office development. The frame covered an area of 9×12 meters with a level height of 3 m (see Fig. 12). There were two equally spaced bays of 6 m along the length of the building, and of 4.5 m across the width of the frame. The structure is assumed to be a braced frame. The beams were designed as pin-connected with the steel column via the steel beam, acting together with the floor slab. The overall minimum depth of the slab was 147 mm. The Finnish slim floor beam was used for both the central beams and edge beams (see Fig. 2(b)). The composite floor system consisted of the Ranila 153 steel deck with normal-weight concrete of grade C30/25 and A142 anti-crack mesh. The hogging reinforcement is ϕ16 c/c 150. The universal column of grade 355 was protected by aerated concrete blocks in between the steel flanges. The uniform design load on the floor was 2.5 kN/m^2 in fire and the according load ratio was 0.15. The beams were four-point loaded and the nominal load ratio was 0.69. The frame was connected together by the tie members across the width.

The fire followed the ISO standard curve and was set in the compartment of 1~2 axis and A~C axis. The temperature distribution of the heated columns (A1, B1, C1 and A2, B2, C2) was similar to that in the previous section.

Four types of elements were used in this modelling:

- Beam element (B32), to represent the steel beams and columns;
- Shell elements (S8R), to represent the continuous floor slab;
- Truss elements (T3D2), to represent the tie members;
- Joint (or Spring) element, to represent the steel beam-to-column connection;

Both physical nonlinearity and geometric nonlinearity were taken into account in this study.

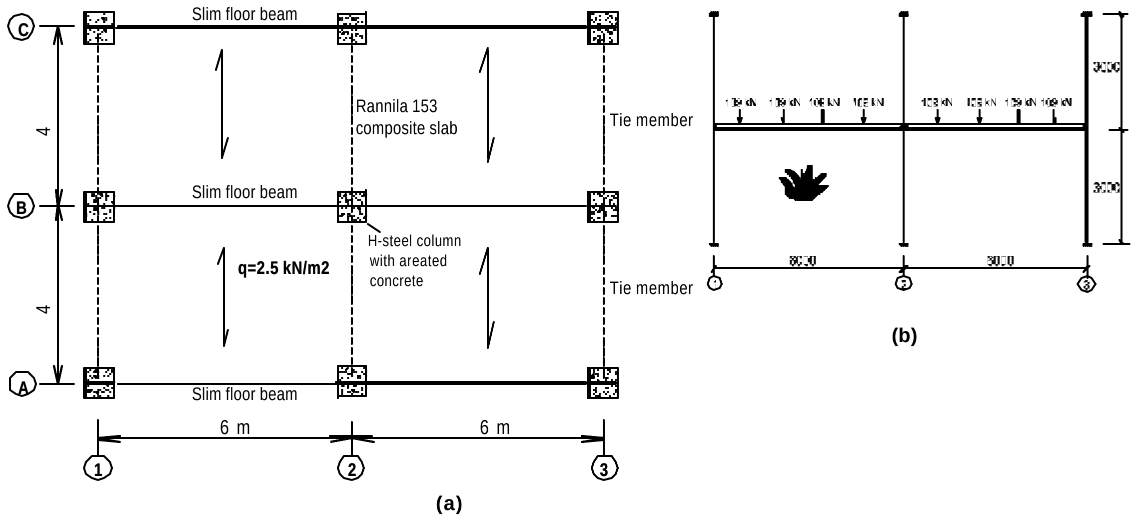


FIG. 12. Analyzed Frame: (a) Layout; (b) Elevation and Loads

Structural Response of the Floor System

The analyzed deformation mode at 90 minutes of ISO standard fire is shown in Fig. 13. The vertical deflection profile of the composite floor in 1~2 and A~C axes at 90 minutes is shown in Fig. 14. The maximum deflection in the slab at 90 minutes is 318 mm and that in the central beam (B and 1~2 axes) is 302 mm. The floor still maintained the stability at 90 minutes' ISO heating. From Fig. 14(a), it can be seen that the maximum relative displacement between the beam and concrete floor slab is small.

Fig. 14(b) shows the displacement of the composite beam with the heating time. The isolated simply supported beams with a load ratio of 0.69 and 0.54 (modified according to Eq. 1, see Table 7) are also represented in this figure. It can be seen that the beam in the frame exhibits far better behaviour in fire than the isolated beam. When applying the modified load on the isolated simply supported beam, the deformation behaviour is close to that of the beam in the frame. However, the beam in the frame maintains stability even after 60 minutes, and no run-away point occurs. It can also be concluded that the effect of the composite slab is not significant in this example, because the little restraints were provided by the surrounding structures.

TABLE 7. Modified Load Ratio of Frame Beam

Calculation of Modified Load Ratio	
Free Bending Moment M_{static} (kNm)	442
Sagging Design Moment Capacity M_p^+ (kNm)	618
Hogging Design Moment Capacity M_p^- (kNm)	399
Modified Load Ratio (by Eq. 1)	0.54

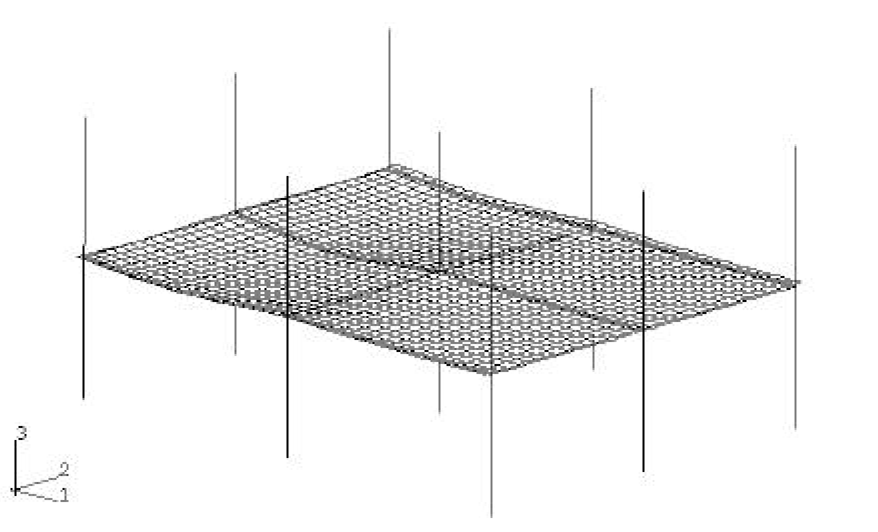


FIG. 13. Deformation Mode of Frame at 90 min under ISO Fire

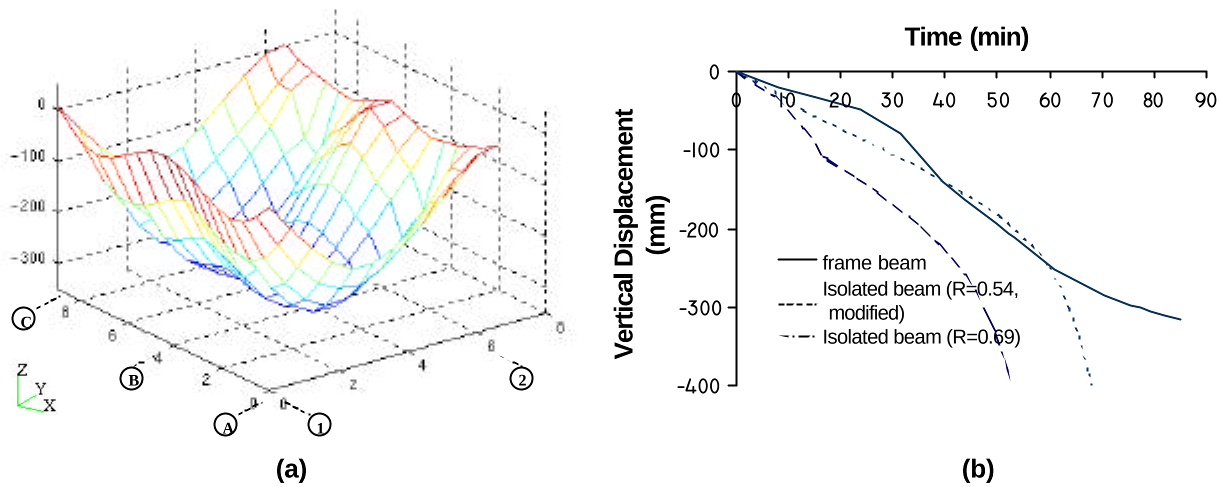


FIG. 14. Deformation of Floor System (at 90 min): (a) Floor Slab; (b) Beam

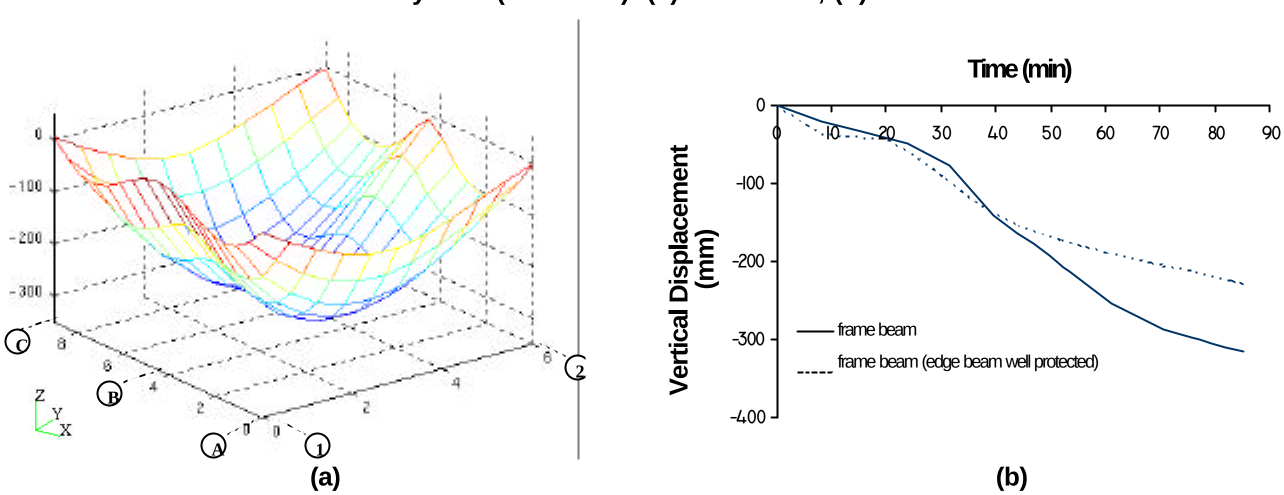


FIG. 15. Deformation of Floor System (at 90 min, Edge Beam Well-Protected): (a) Floor Slab; (b) Beam

Effect of Edge Beams

In practicality, the lower flanges of the edge beams are often well protected by the surrounding walls. When the edge beams are well protected, more restraints are provided for the composite slab. Fig. 15(a) shows the obtained deflection profile of the composite floor at 90 minutes' heating, where the edge beams were protected. The maximum vertical displacement of the floor is 230 mm, which is 88 mm lower than when the edge beams are left unprotected. Fig. 15(b) compares the mid-span displacement of the beam (B and 1~2 axes) with and without the protected edge beams. It can be seen that the protected edge beams reduce the vertical displacement of the beam (B and 1~2 axes). At 90 minutes, the reduced displacement is up to 83 mm. This effect may be attributed to the composite slab. Since the additional vertical restraint provided by the edge beams is negligible, the only factor is the tensile membrane action in the slab. Due to the edge beams being protected, the slab can utilize the tensile membrane action at large displacement to maintain the floor's stability.

CONCLUSIONS

The fire-resistant behaviour of the composite asymmetric slim floor beam both as an isolated member and as a part of a frame with surrounding restraints was investigated using numerical methods.

The investigated simply supported asymmetric slim floor beam has a 60-minute fire resistance if the load ratio is less than 0.5, without any additional fire protection. The steel web plays an important role in the fire resistance of the beam, as the bottom steel flange does at ambient temperature. The investigation also shows that the axial restraint causes a larger vertical displacement of the beam in the early fire phase and a more stable behaviour in the later phase. The behaviour of the beam in the frame exposed to fire is significantly more complicated than that of the isolated beam. The effect of frame continuity can be identified as the axial and rotational restraints. The effect of rotational restraints can be quantified using the modified load ratio (Eq. 1). The influence of the axial restraints on the beam is difficult to quantify. However, it appears that the axial restraints keep the beam standing for up to 90 minutes under ISO standard fire in this investigation (Fig. 10). The moment variation in the side-columns is largely due to the thermal expansion of the beam. This might be detrimental to the columns due to the significant P-Δ effect. Thermal bowing and material softening (stiffness degradation) are the other factors which contribute to the moment variation in the beam.

The behaviour of the composite slim floor system exposed to fire was also investigated. The effect of the concrete slab on the beam deformation in fire was preliminarily identified. With increasing fire exposure, the surrounding restraints and the large displacement make the tensile membrane action take effect. The investigated slim floor frame still maintained its stability under a modified load ratio of 0.54 up to 90 minutes' ISO fire exposure.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

R = load ratio, defined as a ratio between the load on the beam in fire conditions to the failure load under normal conditions;
M_{con} = bending moment capacity of connection at ambient temperature (kNm);
$M_{con,left}$ = bending moment capacity of left connection at ambient temperature (kNm);
$M_{con,right}$ = bending moment capacity of right connection at ambient temperature (kNm);
M_{p+} = plastic moment capacity of the sagging beam section (kNm);
M_{p-} = plastic moment capacity of the hogging beam section (kNm);
M_{static} = isostatic (free) bending moment in the beam (kNm);
M_u = plastic bending moment capacity of composite cross-section (kNm);
T = temperature (°C);
θ_{lim} = design value of limited rotation of beam-to-column connection (rad).