FIRE PERFORMANCE OF HOLLOWCORE FLOOR SYSTEMS IN NEW ZEALAND

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The 2006 version of the New Zealand Concrete Standard NZS3101 has introduced two new details for connection of precast and prestressed hollowcore floor units to supporting reinforced concrete beams. Although these amended connection details have been experimentally proved to yield better seismic performance of the building, concerns about their performance in fire still remain. A simplified and novel approach to model the two way action of hollowcore floor slabs with topping is established. This paper analytically investigates the fire performance of the precast and prestressed hollowcore floor units connected to the supporting moment resisting reinforced concrete building frames according to the old practice and the improved details recommended in NZS3101 using the finite element tool SAFIR. The simulation results show that rigid connections between the hollowcore floor systems and perimeter beams provide better fire resistance than rotationally flexible connections.

1. INTRODUCTION

1.1 Background

Hollowcore units in New Zealand are 1200mm wide precast, prestressed and voided concrete slabs with various thicknesses and require a reinforced concrete topping to form the hollowcore floor systems [1]. Hollowcore floor systems are widely used in multi-storey buildings in New Zealand. However, because of the complex geometry and a wide range of possible support conditions, the structural behaviour of hollowcore floor systems under fire is too complicated to be considered in normal design. The design for fire resistance of hollowcore concrete floor systems usually follows prescriptive proprietary ratings provided by the manufacturers. These proprietary ratings are based on the concrete cover to the prestressing strands and the equivalent slab thickness which converts the cross section of the hollowcore units into an equivalent thickness of solid slab. These proprietary ratings are not confirmed by fire tests, and they are not representative of real construction because they do not account for the effects of axial restraint at the supports.

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New details for connection of hollowcore floor units to reinforced concrete supporting beams were introduced in the New Zealand Concrete Standard NZS3101:2006 [2] to improve seismic performance. The problems of the connections of hollowcore concrete flooring systems from NZS3101:1995 [3] in earthquakes has been exposed by Matthews [4]. Due to the high fixity of these connections, the rotation of the seating beam caused by the earthquakes transfers directly onto the hollowcore units and causes snapping of the floor slabs. With the excessive fixity in the end and side connections, incompatibility between the slab and the perimeter beams can also cause topping delamination and web splitting in the hollowcore units during earthquakes. Therefore, new seating details are proposed by Lindsay [5] to reduce the fixity at both connections at the ends and the sides, and MacPherson [6] proposed another end connection detail to reduce the possibility of snapping action. The recommended seating connections for hollowcore flooring systems have been updated to the ones proposed by Lindsay and MacPherson in NZS3101:2006, but their influence on fire performance has yet to be determined.

In fire resistance tests, some failures of hollowcore concrete slabs have been observed [7]. However, to the authors' knowledge there have been no known structural failures caused by real fires in buildings constructed with hollowcore floor systems anywhere in the world. This difference is partly because restraint provided by the surrounding structure often reduces the likelihood of premature failures. It has been shown earlier that the fire resistance of other types of floor slabs can be increased if they are continuous over the supports or restrained by the surrounding structure [8, 9], and the effect of axial restraint often surpasses other factors such as steel cover, size and shape of the member, aggregate type, reinforcement type and load intensity [10]. Axial restraints at the boundaries prevent the cracks in concrete from opening, and enable interlocking effects where the shear is transmitted via the friction of the rough crack surface, resulting in an increase of fire resistance [11]. Axial restraints also limit the growth of lateral cracks and reduce the likelihood of slipping of prestressing strands, and consequently enhance the shear capacity of the slab in fire [7].

Failures of hollowcore concrete slabs have been observed in fire resistance tests as mentioned above. Most reported failures in fire tests are flexural failures, but shear and anchorage failures are also common, accounting for about 25% of total failures [7]. Shear and anchorage failures can be caused by the deterioration of strength and stiffness of concrete at elevated temperatures, incompatible thermal strains between the prestressing strands and concrete, as well as high applied load levels.

The aim of this paper is to compare the structural fire behaviour of the hollowcore floor systems following both the old New Zealand Concrete Standard and the new recommendations. The study is conducted using a previously validated simulation method [12].

1.2 Fire designs of hollowcore floor systems in NZ

The New Zealand Building Regulations [13] requires structural members to sustain loads for an adequate time in fire, to allow people to evacuate safely and to allow Fire Service access. One means of compliance is for floor systems to have specified fire resistance in accordance with the Approved Documents to the NZ Building Code [14]. To fulfil this requirement, the floor should maintain the stability, integrity and insulation criteria for the specified fire resistance duration. Stability in fire is difficult to predict for hollowcore concrete slabs. For normal concrete slabs, NZS3101 provides tabulated data and specifies a minimum concrete cover to the strands to achieve a particular fire resistance rating, but these data have not been verified for hollowcore slabs and they do not consider composite construction or the interaction between building elements.

Rather than tabulated data, a more accurate alternative is to use a calculation method for determining fire resistance based on the thermal gradient within the floor slabs. Eurocode 2 [15] provides two calculation methods for carrying out engineering design, which are simplified and advanced calculation methods. Simplified calculation methods consider the loss of strength but ignore the additional mechanical stresses caused by thermal expansion acting along with the boundary conditions. Advanced calculation methods are based upon the fundamental physical behaviour but are difficult to carry out without the aid of computer programs. Because of advances in computational power, finite element programs such as SAFIR [16] are being used not only for research purposes but also for design when advanced calculation methods are chosen.

In this paper the structural fire behaviour of hollowcore slabs is considered for the end and side connections similar to those used by Matthews [4], Lindsay [5] and MacPherson [6]. These connections represent the most common arrangements of hollowcore floor systems used recently in New Zealand. Matthews' connections between floor slab and beams are based on NZS3101:1995 before the introduction of Amendment 3 [17], and Lindsay's and MacPherson's connections follow the recommendations from C18.6.7 in NZS3101:2006 [2]. The connections between the slab and the end beams modelled in this paper are shown in Figure 1, and two alternative connections between the slab and the side beams modelled are shown in Figure 2.

1.3 Membrane action

Previous studies on reinforced concrete slabs show that the load bearing capacity of the slabs is larger than predicted by yield line theory which is based on the flexural strength of the slab [18, 19]. This is due to both the strain hardening effect of the reinforcement and membrane effects. Tensile membrane action occurs under large vertical deflection forming a region of tensile stresses in the middle of the slab and a compression ring at the outer edges of the slab as shown in Figure 3. Tensile membrane action causes the slab to act as a shell, becoming more significant with increasing vertical deflection. Tensile membrane action enhances the load bearing capacity of the slab in ambient conditions as well as in fire [20, 21].

Although hollowcore floor systems are usually designed as one-way slabs, in most buildings they have supports parallel to the direction of the units so they can develop two-way action depending on the aspect ratio. In a one-way slab, flexural failure occurs when the applied bending moment exceeds the flexural strength, except that catenary action may prevent collapse of the slab if the supports and connections are strong enough to resist the associated tensile forces. However, if the slab has two-way action, tensile membrane effects can increase the fire resistance as explained above. This study investigates whether two-way tensile membrane action can be established in the topping of hollowcore concrete slabs, subsequently increasing the fire resistance of such floor systems.

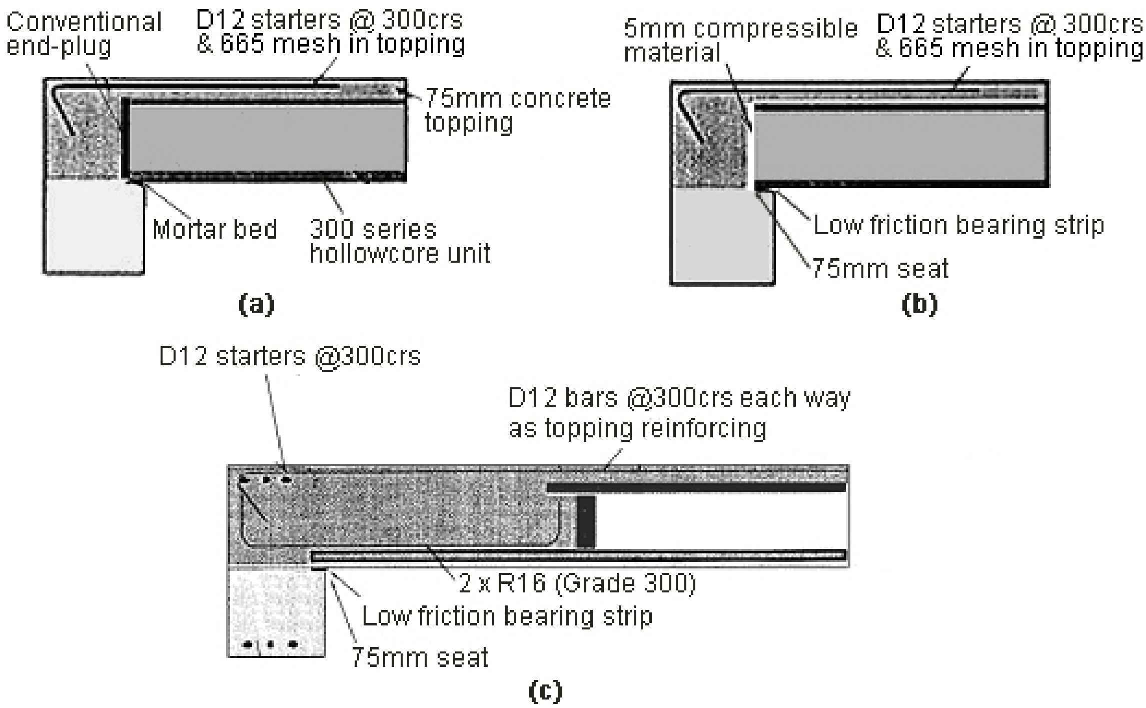

Figure 1. Modelled slab-end beam connections based on (a) Matthews [4] (b) Lindsay [5] (c) MacPherson [6] (not to scale)

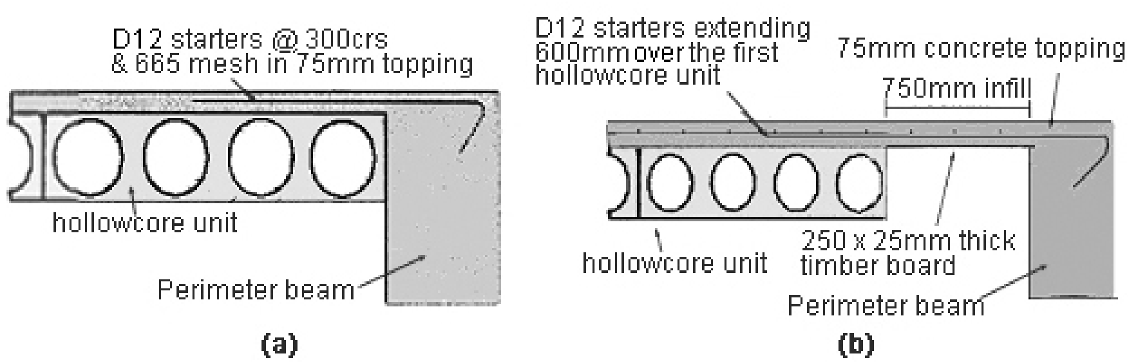

Figure 2. Modelled slab-side beam connections (a) without cast in-situ infill (b) with cast in-situ infill (not to scale)

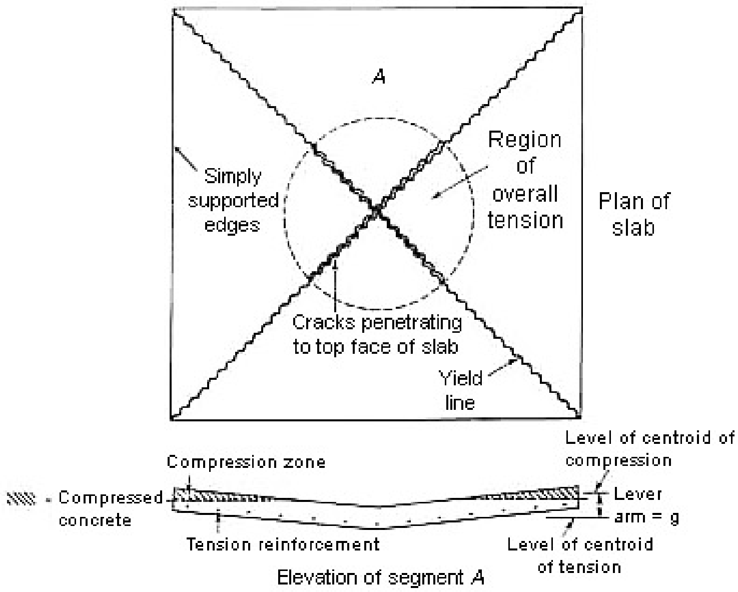

Figure 3. Tensile membrane action in an unrestrained, uniformly loaded slab [18]

1.4 Simulation model

To simulate the structural behaviour of subassemblies with hollowcore floor systems under fire conditions, a new computational model was developed on the platform of SAFIR, which is a non-linear finite element program performing both thermal and structural analyses. SAFIR takes account of thermal and mechanical properties of concrete and steel at elevated temperatures following the Eurocodes [15, 22]. The program is well known in the fire engineering field and has been validated against many experimental results [21, 23].

The reinforced concrete topping slab is simulated using a layer of shell elements to take into account the continuity between the hollowcore units, and each hollowcore unit is simulated using a grillage of 3D beam elements. The longitudinal beams in the grillage can capture the prestressing effect and enable the thermal gradient in the web to be calculated accurately. The transverse beam elements span across the width of the hollowcore units to account for the thermal expansion and thermal bowing across each unit. They also ensure the continuity of the top and bottom flanges across each hollowcore unit. within addition to the top and bottom flanges, the transverse beam cross-section also includes the topping concrete for the thermal analysis but not for structural analysis, in which the topping concrete is simulated using shell elements. This model, shown in Figure 4, has been validated against existing test results. Except for not being able to predict shear failures, the simulation results showed good agreement with the experiments [12].

Some details are overlooked in this model. Shear and anchorage failures, bond failures and vertical tensile stresses in the web are not captured due to the complexity and the computational effort needed when simulating the entire structure. However, a detailed study on the shear and anchorage behaviour of hollowcore concrete slabs has been carried out by Fellinger [7] and findings from that study are included in the design recommendations. Spalling is also not considered as the possibility of spalling depends on the curing period and the age of the building, and currently there are very few finite element structural analysis programs considering spalling due to the uncertainties and the lack of specific experimental data.

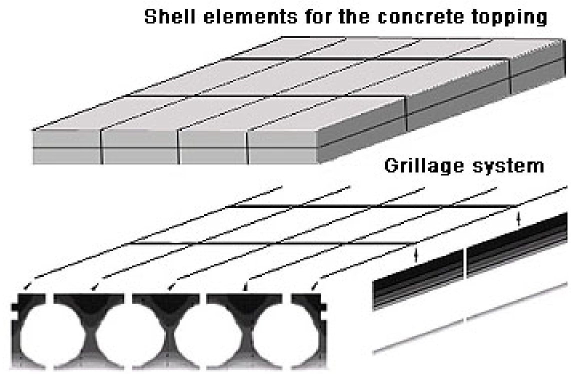

Figure 4. Schematic drawing for modelling of HC floor system

2. SPECIMEN DETAILS

The studied specimen is similar to the 3D frame-slab subassembly recently tested at the University of Canterbury for the seismic performance of hollowcore floor systems [24] but without the central column in the longitudinal direction. Figure 5 illustrates the subassembly of hollowcore slab and reinforced concrete frame. The floor is made from 300mm hollowcore units with a 75mm topping reinforced by a 665-mesh (150x150x5.30mm) at mid-thickness. In this study, five different floor span lengths are investigated ranging from 6.2m to 18.2m. In all cases the modelled slab is 10.2m wide, consisting of eight units if the last unit is adjacent to the side beams as shown in Figure 6(a), or seven units if there is a concrete infill panel between the last unit and the side beams as shown in Figure 6(b). The computer model is 5.1m wide, as symmetry applies.

The end beams are 750mm deep by 400mm wide with three D25 bars at both the top and bottom. The side beams, when present, have the same dimensions as the end beams. There are six columns in the subassembly spaced 5.1m apart along the width of the slab. Each column is 750mm by 750mm square and 3.5m high with end beams connected at mid-height and restraints on displacements at the top and bottom ends.

The total applied load on the 12.2m long slab is 8.0kPa. This includes the self weight of the hollowcore units at 3.2kPa, the weight of the reinforced concrete topping at 1.8kPa, and a reduced live load of 3.0kPa. The reduced live load is the product of the reduction coefficient 0.4 for live load in fire from AS/NZS1170 [25] and the maximum allowable live load for the 12.2m span under the ambient conditions, i.e. 7.6kPa according to the manufacturer [1].

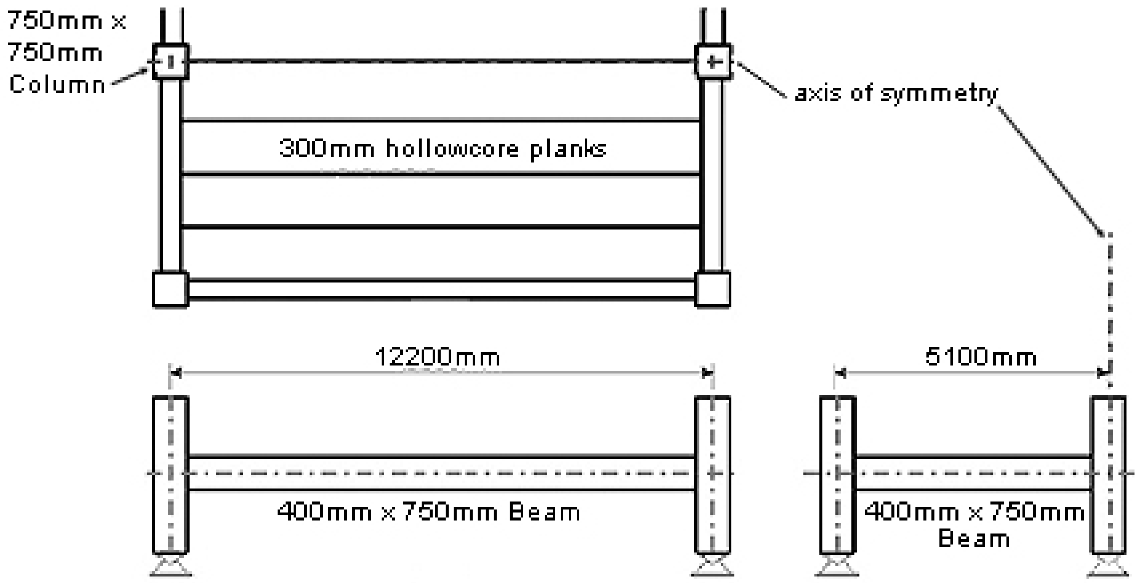

Figure 5. Dimensions and layout of the studied subassembly

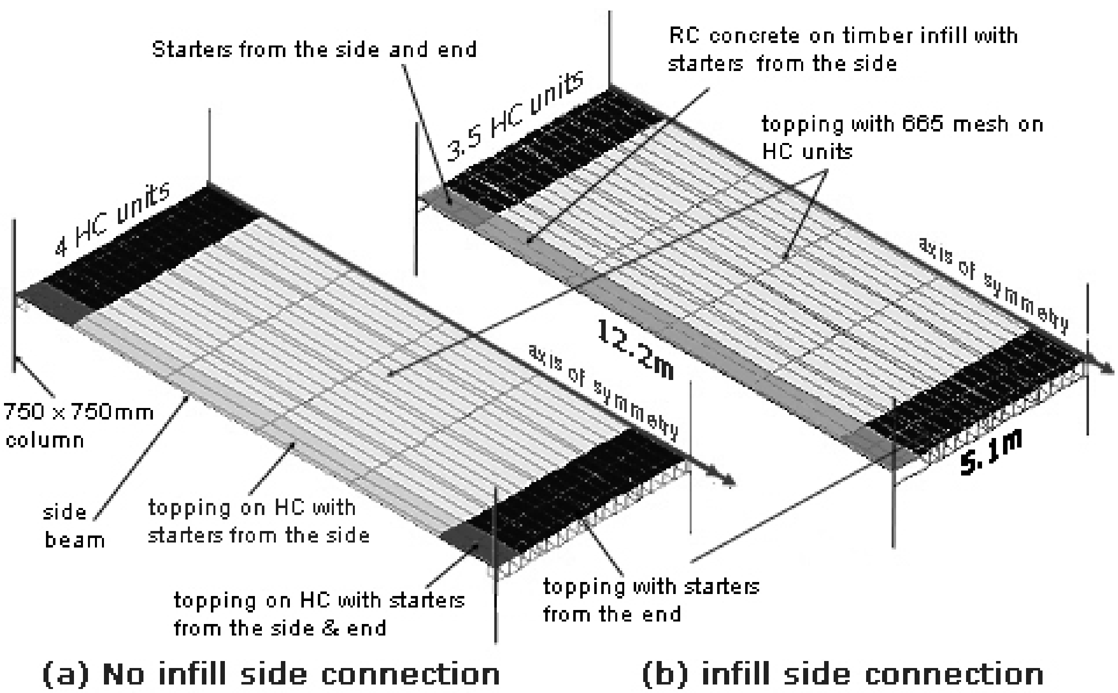

Figure 6. Simulation models used to study the subassembly

2.1 Studied parameters

The first part of the simulation looks into the effects of the end and side connections for a slab span of 12.2m. The three end connections shown in Figure 1 are studied. Three scenarios at the side of the floor are considered, these are the two side connections shown in Figure 2 and a scenario without side beams. In the scenario without side beams, the slab spans one-way only, but the sides of the slab were restrained against lateral expansion in order to simulate the restriction imposed by the surrounding floor slabs. The side beam connection in Figure 2(a) is referred to as the “no infill” side connection, and that in Figure 2(b) as the “infill” side connection. The modelled combinations of different end and side connections are shown in Table 1. The slab and beams are exposed to the ISO fire from below, while the columns are assumed to be fully protected and remain cool. In the cases without side beams, a fire exposure with decay phase is also studied, where the ISO fire is replaced by the fire shown in Figure 7 which follows 60 minutes of the ISO fire and then decays in 625^{o}C per hour based on the ISO834 testing standard [26].

Table 1. Summary of the simulation results of 12.2m long slab with various side and end connections

End connection	Side support	simulation stopping time	Reason for stopping
Matthews	**No infill**	>240min.	Designated end time (No failure)
Lindsay		148min.	Numerical problem
MacPherson		>240min.	Designated end time (No failure)
Matthews	**Infill**	187min.	Numerical problem
Lindsay		60min.	Numerical problem
MacPherson		181min.	Numerical problem
Matthews	**Free**	123 min.	Crushing of topping near the ends
Lindsay		51 min.	Numerical problem
MacPherson		84 min.	Crushing of topping near the ends

The second part of the simulation looks into the effects of the side connections with different span lengths. In all cases, Matthews’ end connection is used to connect the slab to the end beams, and the same three side connections are studied. The slab and beams are exposed to the ISO fire from below, while the columns are assumed to be fully protected and remain cool. The combinations of different side conditions and the span length are shown in Table 2. The load ratios in Table 2 are calculated as shown in Table 3 where the tabulated load capacity is based on the dead loads described above and the live load from the manufacturer’s tabulated data [27], with the load combinations and safety factor for normal design under ambient conditions from AS/NZS1170 [25].

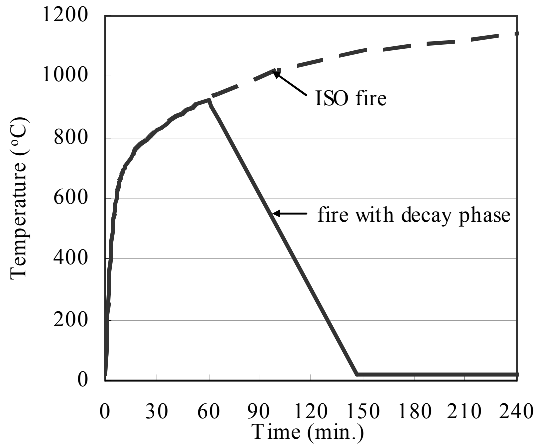

Figure 7. Fire temperature

2.2 Modelling the End Connections

The end connection used in Matthews' test [4] is translated into the SAFIR model as shown in Figure 8. The load path in the end connection is shown in Figure 8(a) and is modelled by three rigid elements connected to each other at the end of each line of beam elements as shown in Figure 8(b). The first rigid element (1), from the bottom of the hollowcore unit to the mid-height of the topping, transfers the vertical load from the end of the hollowcore units to the beam seating point. The second rigid element (2) represents the solid concrete between the seating and the node-line of the beam. The third rigid element (3) connects the shell elements on top of the beam to the node-line of the beam. As the ends of the hollowcore units are in full contact with the supporting beam, no relative displacement is allowed between these two surfaces.

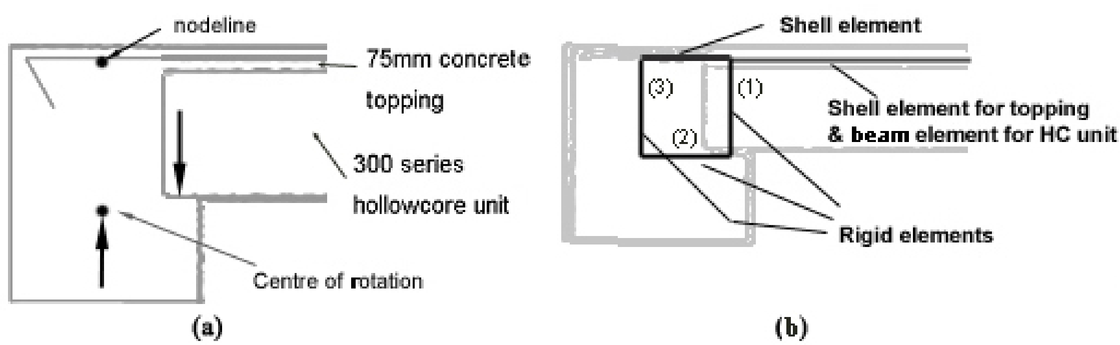

Figure 8. Modelling the end connection details from Matthews' tests (a) Node-line of the model in SAFIR and force paths in the connection (b) Simulation scheme in SAFIR

The modelling scheme of the end connection tested by Lindsay [5] following C18.6.7(a) of NZS3101:2006 is shown in Figure 9, which also uses three rigid elements at the ends of every line of beam elements representing the hollowcore units. The significant difference between Lindsay's and Matthews' end connection details is the soft packing at the ends of the hollowcore units which allows sliding displacement until the two surfaces press against each other. This connection is modelled in the same way as the Matthews' detail except for the junction between rigid elements (1) and (2), where horizontal sliding is permitted, as shown in Figure 9(b). To be strictly correct, the displacement of the bottom of the hollowcore units moving towards the face of the end beams should be limited to the size of the gap, but this limitation cannot be represented using SAFIR and is ignored in the simulations [27].

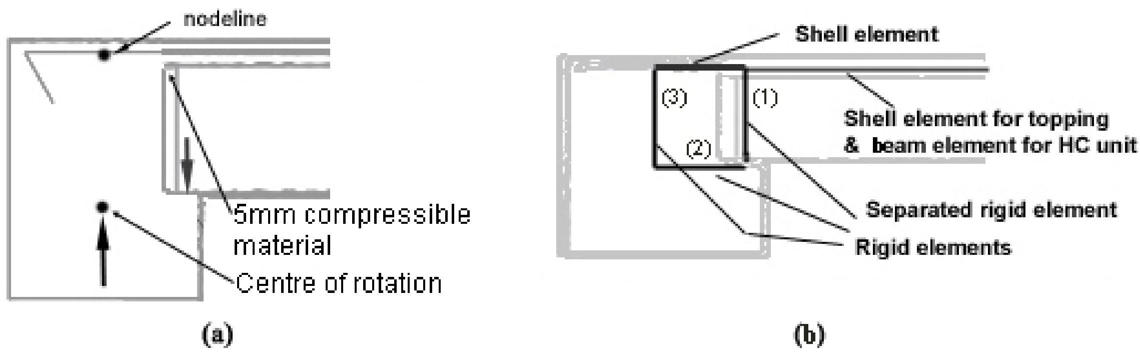

Figure 9. Modelling the end connection details from Lindsay's tests (a) Node-line of the model in SAFIR and force paths in the connection (b) Simulation scheme in SAFIR

In the end connection used by MacPherson [4] following C18.6.7(b) of NZS3101:2006, the reinforced concrete cell filling is included in the beam grillage which represents the hollowcore units. The force path in the end connection is the same as in the connection used by Matthews, as shown in Figure 8(b).

3. SIMULATION RESULTS

3.1 Effect of the end connections, no decay phase

The simulation results from studying the effects of the end and side connections are shown in Table 1. It can be seen that many of the simulation came to a stop due to numerical problems. Although SAFIR could not determine the failure mode of the slabs in fire during the simulation, the comparison of vertical displacement shows that Lindsay's end connection is the least favourable when considering the fire resistance of the subassembly.

Figure 10 compares the maximum vertical displacement at the centre of the slab. The slabs with Matthews' and MacPherson's end connections behave similarly to each other and show a similar trend of maximum vertical displacement. The slight difference between them is caused by the core-filling which only exists in the MacPherson's end connection. The floor slab with Lindsay's end connection has more deflection and fails earlier than the slabs with other types of end connection because it cannot benefit from arch action, due to the lack of rotational restraints at the ends. This is confirmed by further investigation on the stress distribution within the cross section of the hollowcore slabs where significantly less compressive force near the ends is found in the slab with Lindsay's end connection than with other types of end connection. Nevertheless, in reality, because the gap between the end of the hollowcore slab and the supporting beams could eventually be closed, some arch action could still be achieved but at a time much later than in the slabs with Matthews' or MacPherson's connections.

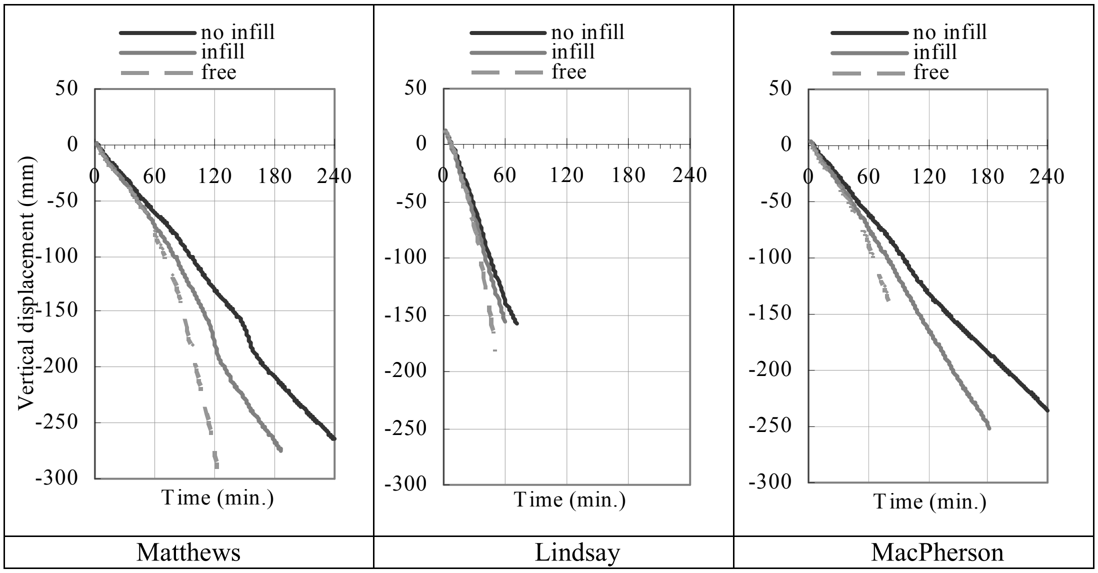

Figure 10. Maximum vertical displacement of the 12.2m long slabs with applied load of 8.0kPa and various side connections, no decay phase

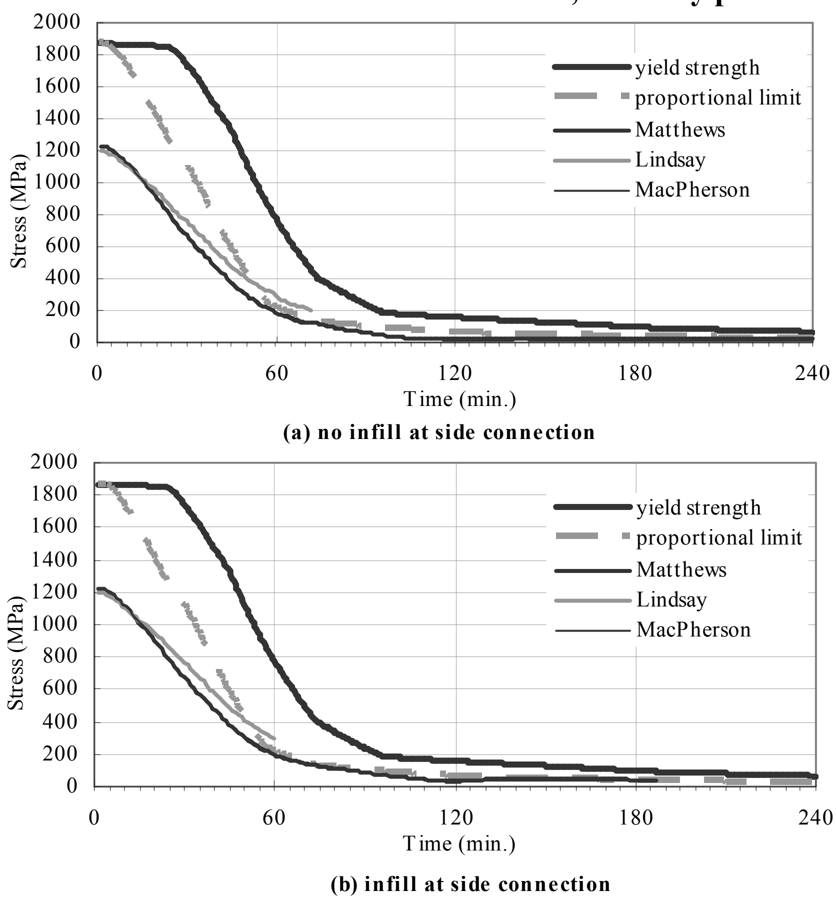

Figure 11. Stress in the prestressing strand at the centre of the slab, no decay phase

Figure 10 shows that the slab with Lindsay's end connection deflects upwards at the start of the fire further than with the other two types of end connection. This is because with Lindsay's end connection, the slab cannot be benefit from the resistance provided by the end beams from the very beginning against the bending of the slab induced by the prestressing effect.

Figure 11 shows the stress in the strand in the unit near the centre of the slab where it is at its maximum for the three end connections. The temperature-dependent yield strength and proportional limit of the prestressing strands according to Eurocode 2 [15] are also shown in Figure 11, which are the same under the ambient temperature but decrease at different rates in elevated temperatures. Figure 11 shows that the stress in the prestressing strands sometimes exceeded the proportional limit, but the strands did not yield throughout the simulation. Figure 11 also shows that the strand stresses with Matthews' and MacPherson's end connections are very similar, while the strand stresses tend to be slightly higher in the slab with Lindsay's end connection.

3.2 Effect of the End Connections, With Decay Phase

During cooling of the fire with a decay case shown in Figure 7, the bottom surface of the hollowcore slab cools, so the maximum deflections shown in Figure 12 are much less than the final deflections for no decay phase, shown in Figure 10. It can be seen that Lindsay's end connection results in much larger deflection during the fire but a return to zero deflection after the fire. The corresponding forces in the prestressing strands are shown in Figure 13 where the magnitude of the final tensile force depends on the end connection detail. During the simulation the prestressing strand reached a maximum temperature of 535^{o}C, which causes a permanent loss of 35% of the original yield strength and proportional limit after cooling [28], as shown Figure 13. Although the stress within the strands is higher with Matthews' and MacPherson's end connection than with Lindsay's, it is never high enough to cause the strand to yield.

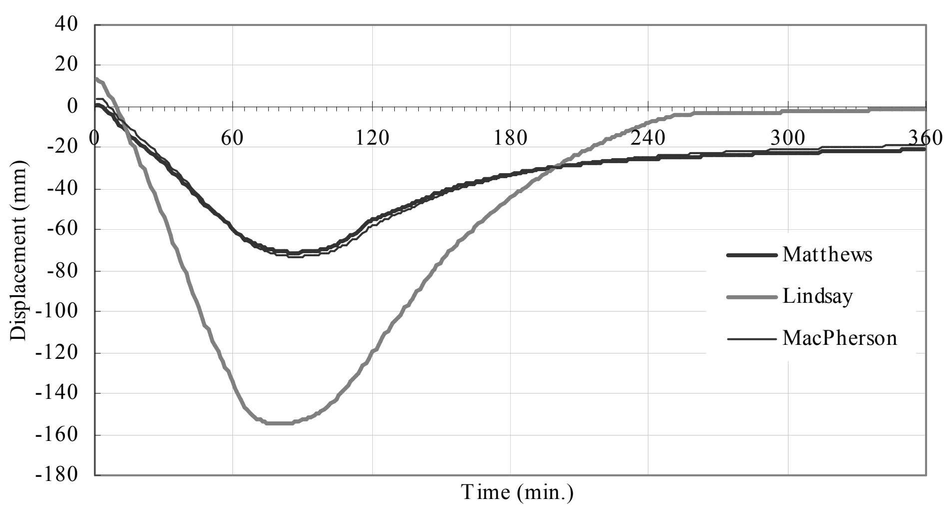

Figure 12. Maximum vertical displacement of the 12.2m long slabs exposed to fire with decay phase

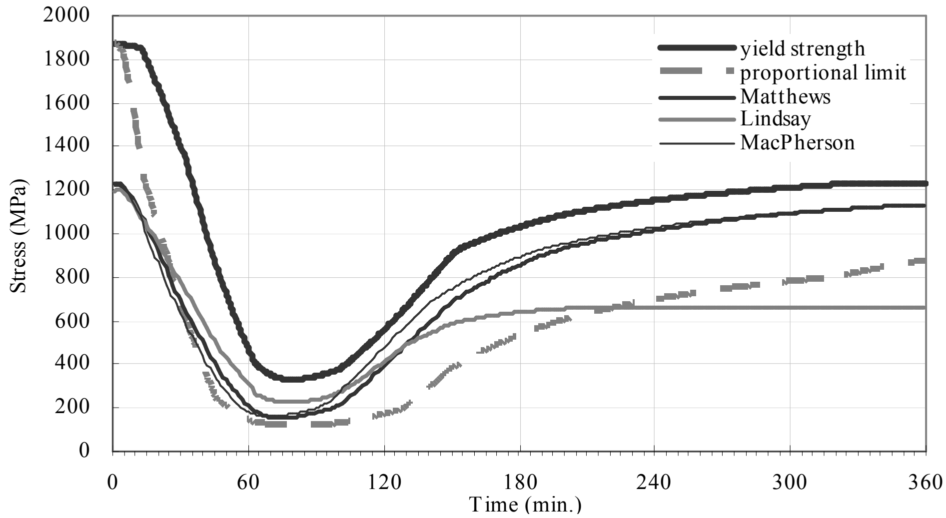

Figure 13. Stress in the prestressing strand at the centre of the 12.2m long slab exposed to fire with decay phase

3.3 Effect of the Side Connections

Among the nine cases shown in Figure 10, the slabs with "no infill" side connection have the smallest deflection and the fire resistance is increased to more than twice of that of the subassemblies without side beams. This is because of the development of two-way behaviour as tensile membrane action is established in the topping concrete. The "infill" lines on Figure 10 indicate that two-way behaviour is less effective with the cast in-situ infill at the side of the slab. This may be because the concrete infill allows larger deflection of the hollowcore unit closest to the side beams.

The maximum deflections and failure times of the hollowcore flooring systems in the ISO fire are affected by the span lengths and load ratio as shown in Figures 14 and 15, which compare the deflections of slabs with various span lengths and different side connections under 40% and 60% load ratio respectively. The load ratio is the ratio of the expected loads on the structure, or element, during a fire to the loads that would cause collapse at normal ambient temperatures. The loads on the slabs are shown in Table 2. Both Figures 14 and 15 indicate that the side connection with no concrete infill gives better fire resistance than with infill, moreover, both side connections are better than the cases without side beams. This finding is more obvious with increasing span length. In all the simulations, the side beams, if present, have 10m spacing. If this spacing increases it can be expected that the slab would behave closer to the case without side beams.

Comparing Figure 14 to Figure 15, the slabs without side beams or with concrete infill next to the side beams show decreasing fire resistance with increasing load ratio, while the slabs with no infill at the side connections maintain their high fire resistance.

The findings above show that the two-way behaviour becomes more important with increasing span length or load ratio. In practice the fire resistance of the slab can be increased by providing side beams or by adding extra "fire emergency beams" to slabs which have large number of hollowcore units side by side. The extent of this increase depends on the spacing of the “fire emergency beams” and the fixity between floor slab and the beams.

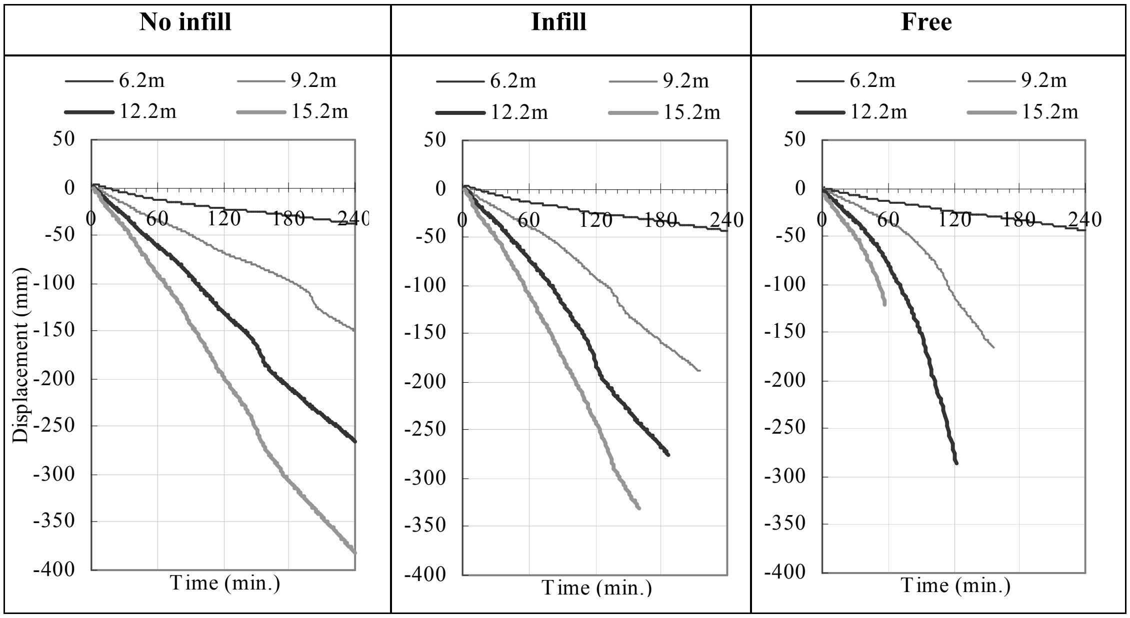

Figure 14. Maximum displacement at the midspan of the slabs with Matthews’ end connection, various span lengths and load ratio of 40%, no decay phase

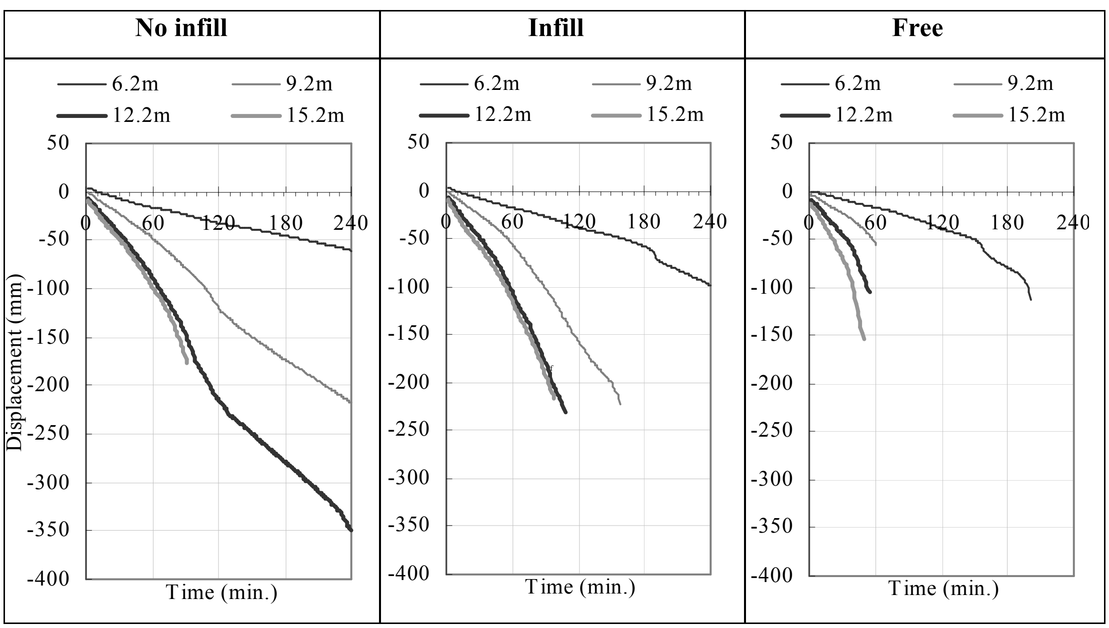

Figure 15. Maximum displacement at the midspan of the slabs with Matthews’ end connection, various span lengths and load ratio of 60%, no decay phase

Table 2. Summary of the simulation results of slabs with Matthews' end connection and various span lengths

L (m)	Total load (kPa)	Load ratio	Side support	Simulation stopping time	Reason for stopping
6.2	18.3	40%	No infill	>240min.	Designated end time
			Infill	>240min.	Designated end time
			Free	>240min.	Designated end time
9.2	12.3		No infill	>240min.	Designated end time
			Infill	214min.	Numerical problem
			Free	156min.	High compression in the topping near the ends
12.2	8.0		No infill	>240min.	Designated end time
			Infill	187min.	Numerical problem
			Free	123min.	High compression in the topping near the ends
15.2	5.2		No infill	>240min.	Designated end time
			Infill	160min.	High compression in the infill
			Free	58min.	Numerical problem
6.2	28.2	60%	No infill	>240min.	Designated end time
			Infill	>240min.	Designated end time
			Free	200min.	High compression in the topping near the ends
9.2	19.0		No infill	>240min.	Designated end time
			Infill	158min.	Numerical problem
			Free	61min.	Numerical problem
12.2	12.4		No infill	>240min.	Designated end time
			Infill	108min.	High compression in the topping near the ends
			Free	54min.	High compression in the topping near the ends
15.2	8.0		No infill	92min.	High tension in the topping near the ends
			Infill	98min.	High compression in the infill
			Free	49min.	Numerical problem
9.2	8.0	25%	No infill	>240min.	Designated end time
			Infill	>240min.	Designated end time
			Free	203min.	High compression in the topping near the ends
12.2	5.2		No infill	210min.	Numerical problem
			Infill	221min.	Numerical problem
			Free	140min.	High compression in the topping near the ends
18.2	8.0	100%	No infill	67min.	High tension in the topping near the ends
			Infill	67min.	Numerical problem
			Free	28min.	Numerical problem

Table 3. Load ratios for different span lengths

Span length	6.2m	9.2m	12.2m	15.2m	18.2m
Imposed load	8.0kPa				
Tabulated load capacity	46.8kPa	31.4kPa	20.7kPa	13.2kPa	8.1kPa
Load ratio	16%	25%	40%	60%	100%

4. CONCLUSIONS AND RECOMMENDATIONS

In terms of the prescriptive connections between the hollowcore units and the end beams, the end connection detail from C18.6.7(b) of NZS3101:2006 (i.e. the end connection used by MacPherson) is recommended as it provides better fire resistance than the other end details.

The simulation shows that any kind of gap between the end of the hollowcore units and the end beams will reduce the axial restraint, and hence give larger deflection during fire and decrease the fire resistance. Fellinger [7] pointed out that hollowcore concrete flooring systems without axial restraint are also more likely to have shear and anchorage failures in the early stages of the fire.

The end connection design for the hollowcore floor system from C18.6.7(a) of NZS3101:2006 (i.e. the end connection used by Lindsay) may not be good for the overall structural performance of the floor system in fire. This conclusion is based on a model which ignores the possibility of axial restraint after closure of the gap between the hollowcore units and the end beams, so a more sophisticated model is needed for a more definite conclusion to be drawn.

In terms of the connections between the hollowcore units and the parallel side beams, a rigid side connection with the hollowcore units placed immediately adjacent to the side beams (i.e. design before the Amendment 3 in NZS3101:1995) has better fire resistance than a flexible side connection with infill concrete, and this effect is more significant with long spans or high load ratios.

Two-way behaviour and membrane action can increase the fire resistance, and their benefit is more significant with increasing span lengths or increasing load ratios. Therefore, side beams or fire emergency beams which reduce the width of the slab can be used to improve the fire resistance. The improvement can be maximised by fixing the slab to the beams.

Overall, hollowcore floor systems with the end and side connections designed prior to the introduction of Amendment 3 of NZS3101:1995 perform better in fire than those designed according to the newer standard, and the worst combination from a fire perspective appears to be the subassembly with the end connection based on C18.6.7(a) of NZS3101:2006 and the flexible side connection with infill concrete, because the flexible side connection cannot transfer sufficient load to the side beams, and arch action along the span fails to develop.

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