Simple method for modelling hollowcore concrete slabs under fire

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ABSTRACT: Precast prestressed hollow core concrete slabs have been widely used in recent years. However, comprehensive finite element analyses for modelling the structural fire behaviour of this type of slab is too complicated to perform within hours. On the other hand, a simplistic analysis is too crude to capture the thermal expansion across the units. The aim of this paper is to present a simple computational method to be used in design for modelling the structural behaviour of hollow core concrete slabs in fires. The proposed model consists of a grillage system using beam elements to include the thermal expansion in both directions and to simulate the vertical cracking in the flanges, with the topping concrete modelled using shell elements. The simulation outcomes show good agreement with the experimental results.

1 INTRODUCTION

Precast prestressed hollow core concrete [HC] floor system is very popular in New Zealand. It consists of several HC slab units with or without a layer of reinforced concrete [RC] topping spanning among the units. The benefits of using HC floor systems are low onsite labour cost, low self-weight, consistent quality, and economical use of concrete.

The structural behaviour of a HC floor system under fire is complicated. There are many existing studies investigating this behaviour using different approaches, and precise computer models have been developed to improve the understanding of it (Fellinger 2004). However, very detailed finite element analyses of the structural fire behaviour of HC slabs are too time-consuming to apply in the everyday design process. On the other hand, simplistic approaches using simple code rules are insufficient to capture the effects of the support conditions. Consequently, a simple yet sufficiently accurate computational method for designers to model the structural behaviour of HC slabs under fire needs to be developed.

2 BACKGROUND

The behaviour of HC slabs under fire is more complicated than that of solid slabs. The voids cause discontinuity in heat transfer, yet the thermal gradient needs to be addressed correctly to accurately model the temperature induced mechanical strains occurring in the webs. Details of various strains that a concrete slab may experience can be found in Buchanan (2001). The support conditions also have significant influence on the structural behaviour and should be considered in design. The presence of prestressing stress can considerably influence the predicted overall structural performance (Chang 2007), as the HC units have no reinforcing and the resistance to tensile stresses come from the prestressing tendons.

In terms of concrete floor design, there are three methods outlined in the Eurocode 2 (CEN 2002), namely tabulated data, simple calculation methods and advanced calculation methods. The tabulated data from the Eurocode 2, or the NZ Concrete Standard (NZS3101, 1995), does not consider the unique thermal gradient of the HC slabs nor does it take into account the influence of the surrounding structural members. Simple calculation methods cannot accurately predict the thermal gradient, or include the effect of support conditions. Due to the rapid development in computation, with advanced modelling methods, the commercially available finite element analysis [FEA] programs could be used to design HC floor systems based on the fundamental physical behaviour with due consideration to the effects from the surrounding structure. This fits into the category of "advanced calculation methods".

This research uses the non-linear FEA program SAFIR (Franssen et al. 2002), which has been proven to be able to accurately predict the fire behaviour of RC slabs (Lim 2003). A previous study also showed that SAFIR can successfully predict the structural behaviour of *hibond* slabs under fire using a combination of shell and beam elements (Lim 2004), which is the basic idea behind the proposed model in this study.

The grillage analogy has been used for a long time and proved to be reliably accurate in bridge deck designs (Hambly 1991). Grillages by definition have straight longitudinal and transverse beams rigidly connected together, each beam with its own bending and torsional stiffness, and at each junction the deflection and slope is calculated (West 1973, Livesley 1964). This grillage method is adopted in the proposed model.

3 MODEL DESCRIPTION

Before deciding on the final model, various other combinations of shell and beam elements available in SAFIR were tried (Chang 2007), such as using shell elements vertically to simulate the webs or using only beam elements to simulate the entire structure. Shell elements in SAFIR require less discretisation and are suitable for large displacements, but the thermal gradient is one directional and perpendicular to the element. Beam elements are more complex, requiring more computational effort, but can capture the thermal gradient more accurately and allow for prestressing.

The proposed model as shown in

Figure 1 uses beam grillages for the HC units and shell elements for the topping RC slab. The longitudinal and transverse beams are discretised into several fibres as shown in Figure 2. In the transverse beams, only the top and bottom flanges are included. In the grillage, all degrees of freedom except warping of the longitudinal and transverse beams are shared at the intersection points. The topping is modelled using shell elements which join the grillage system at these points and share these degrees of freedom.

The grillage system allows the model to capture the thermal expansion in both lateral directions. The longitudinal beams are used to address the thermal gradient around the voids correctly and include the effect of the prestressing tendons. This prestress effect is accounted for when the stress equilibrium in the cross-section is calculated at the first time step. The transverse beams comprise only the top and bottom flanges and only span within the width of each HC unit, account for the thermal expansion of each unit in the lateral direction, which may affect the structural behaviour of the HC floor system especially when there are restraints on the sides. Therefore, the effects of the restraints on lateral displacements from the surrounding structure can be included. Examples of the thermal gradients in a longitudinal and transverse beam are shown in Figure 2. The transverse beams should only contribute to the transverse displacement, or become effective when the slab consists of multiple units over the width. This is shown later in Figure 4(c) as the vertical displacement for two units is not influenced by the presence of the transverse beam.

The shell element used for the topping provides the membrane action, as the topping is usually used to provide continuity between HC units. This arrangement enables explicit representation of the fact that in many designs the supports at the side are only applied to the topping and not to the HC units.

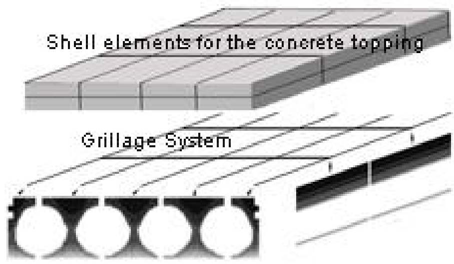
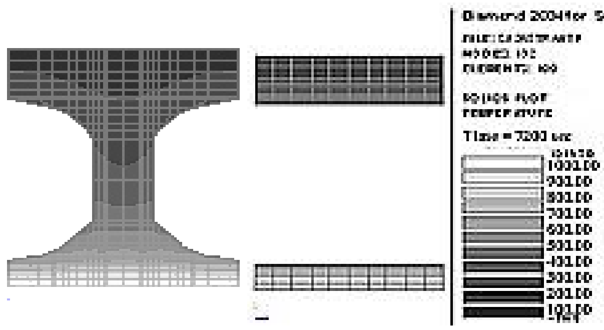


Figure 1. Schematic drawing for modelling of HC floor system



(a) **(b)**

Figure 2. Temperature distribution of SP265 Ergon after 2hr. of ISO fire from (a) longitudinal beam (b) transverse beam

Some details need to be overlooked to reduce the complexity of the model. Shear and anchorage failures as well as bond failures are not included in this model due to the computational effort needed when simulating the entire structure. Shear and anchorage failures have been observed in tests but never in real buildings, as the possibility of this failure mode is reduced when axial restraints are present (Fellinger 2004). Besides, as these are brittle failure modes, in regions requiring earthquake design, these failure modes are generally avoided in structures designed for resisting earthquakes.

The spalling effect is also overlooked. Although spalling has been observed in some tests (Lennon 2003), it is affected by the moisture content and the age of the specimen (Connolly 1997). However, introducing these time factors would make the results too specific and not representative. Besides, Hertz (2003) showed that high strength concrete, which HC units are made of, is almost the same as normal concrete in terms of its vulnerability towards spalling, so no special attention is needed. Currently, no FEA program incorporates this effect on account of the uncertainties and lack of specific experimental data. (Franssen 2005)

4 MODEL VALIDATION

This section shows the comparison between the test data carried out by various institutes and the simulation results.

4.1 *Universities of Ghent & Liège*

Four tests were carried out in the Universities of Ghent and Liège in 1998 focusing on the influence of detailing and of restraint conditions on the shear capacity of HC slabs (Dotreppe & Van Acker 2002). Detailed descriptions and the explanations of the designs are given in Febe(1998a, b). Each test had two 2.4m wide floor (2 HC units) spanning 3m and supported on three beams as shown in Figure 3 (a). Each floor was independent. The floors had self weight of 3kN/m^2, a line load of 100kN in the middle of each of the two spans, and were exposed to 2 hours of ISO fire. Afterwards, extra load was applied to check the remaining load capacity. The parameters studied in the four tests are shown in Table 1. Only half of the floor was simulated (one 1.2m wide floor span of 3m) as shown in Figure 3 (b). The filling of the voids at the ends was included in the model, but the peripheral ties and the detailed anchorage were not.

Table 1 Studied parameters in 1998 tests in Universities of Ghent and Liège

Parameters	**Test 1**	**Test 2**	**Test 3**	**Test 4**
HC section (type)	SP200 Ergon	VS20 Echo	VS20 Echo	SP265 Ergon
Height of HC section	200mm	200mm	200mm	265mm
RC topping (1 floor)	50mm	none	none	30mm

Test 4 has previously been simulated using SAFIR with a different approach (Dotreppe 2004). The differences between the old and new models are the presence of the transverse beam and the location of dividing the units. The new method recognises that the bottom flange is more likely to split than the web. Therefore, it is more reasonable to divide the units at the thinnest point of the flange as shown in Figure 2 (a) rather than at the middle of the web. The previous study showed that the heat transfer through the cavities in SAFIR is properly calculated and the thermal gradient given by the simulation was similar to the experimental results. Hence, the thermal gradient in the new model must also be accurate.

In all the tests, the compressive strength of the concrete in the HC units was around 45MPa, and the strands strength was 1.85GPa. The results from simulations of Tests 1 and 4 are shown in Figure 4, 5 and 6. The level of prestressing was not known and was assumed to be 75% of the strand strength. From Figure 4(a) it has shown that fire performance of the slab is affected by the level of prestressing, but is not very sensitive to that, unless the prestressing is really low. Therefore even if the assumption was a bit different from the actual value, the result should be similar to that from the real prestressing level.

The rebar strength was claimed to be 500MPa, however, no test was done to obtain the actual strength. Figure 4(b) showed the effect of having two different rebar strengths, and the results showed that even if the rebar strength was 100MPa smaller than the stated value, the difference to the overall performance was negligible.

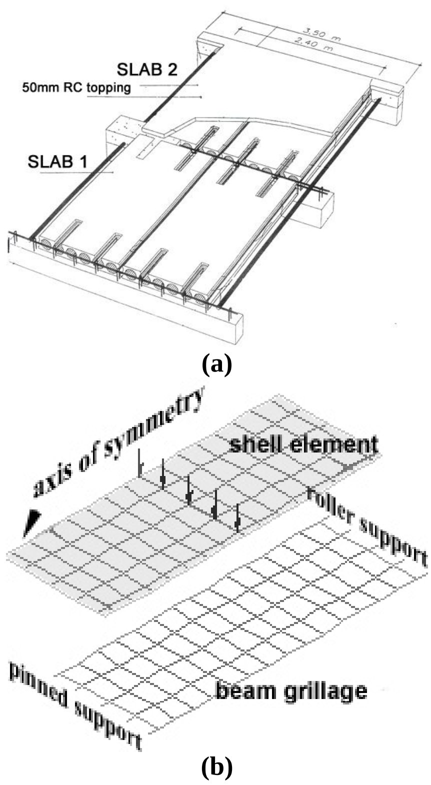
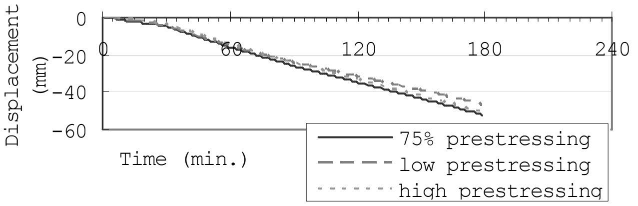
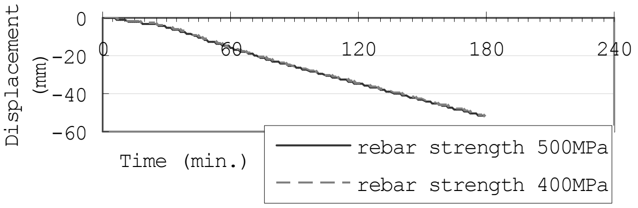


Figure 3. (a) Detailing of Test 1 in 1998 SSTC tests (FeBe 1998a); (b) Simulation model

In Test 1, the simulation with no rotational restraints at the ends predicted that the slab had more than 3 hours of fire resistance, which reduces to 1 hour if there are full rotational restraints. However, in reality the support condition is one of partial rotational restraint, and the slab withstood 83min. of the fire. Figure 5(a) shows that the simulation result was not very close to the experimental data from Febe (1998b). The maximum difference between the maximum deflections obtained from simulation and the test data was 10mm. During the experiment, shear cracking was observed 7 minutes from the start of the experiment, and vertical cracking was observed at 12 minutes (Febe 1998b). This explained the rapid increase in midspan deflection in the experiment at the early stage of the fire. Nevertheless, the simulation model could not predict the shear displacement or failure.



(a)



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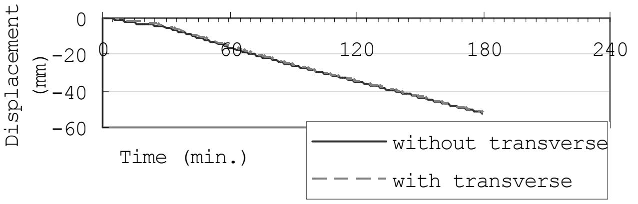


(c)

Figure 4. Simulation results from Test 1 (a) different prestressing level (b) different rebar strength (c) with or without transverse beams

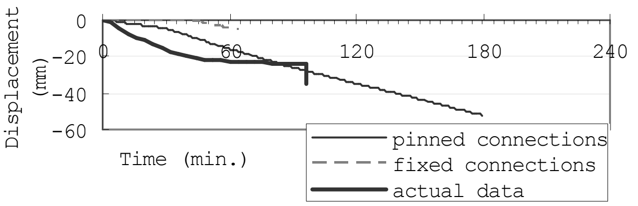


Figure 5. Simulation results from Test 1 with different support condition against actual result

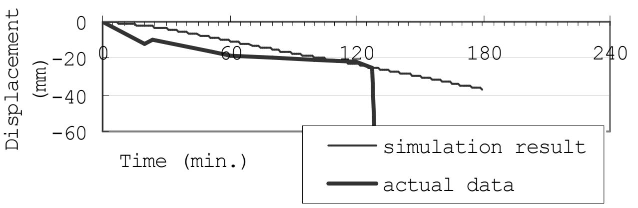


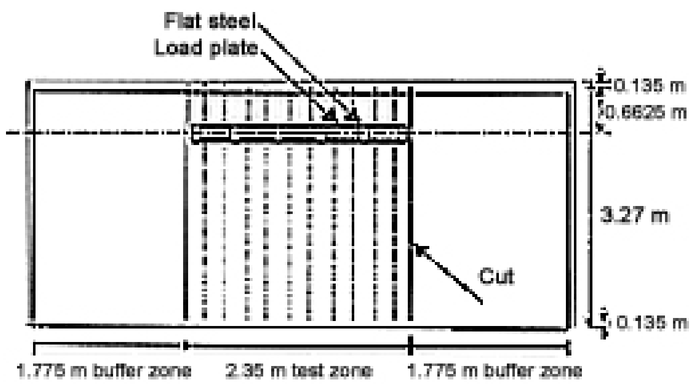
Figure 6. Simulation results of Test 4

In Test 4, the HC unit is changed from SP200 Ergon with circular voids to SP265 Ergon with oval shaped voids, and the number of voids changed from six to five, but the applied load and other mechanical values were the same as in Test 1. The experimental result in Figure 6 showed that the slab could sustain up to 2 hours of ISO fire. The failure from the actual data was caused when the fire was stopped after 2 hours and more loading was added at the midspan to check the capacity. There was no shear failure or substantial shear displacement during the fire test. Therefore the simulation result matched the test data reasonably well up to 120 minutes. The maximum difference between the two deflections was around 5mm.

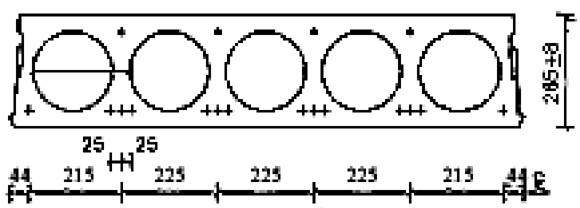
4.2 *Betonelement-Foreningen, BEF*

Three fire tests with HC slabs were conducted by BEF in 2005 (BEF 2005). The purpose of these tests was to confirm if after exposing to the ISO fire for 60 minutes, the HC slabs could still resist at least 65% of the ultimate design shear capacity in cold condition derived from the DS411 Danish Standard (1999). Therefore, the applied shear force in these tests was higher than expected in a normal fire design. In these tests the fire curve followed only 60 minutes of the ISO fire and then stopped, and the tests continued for a further 60 to 90min. with the constant applied load. The tested specimens were 265mm thick, with no topping, spanning 3.27m as shown in Figure 7.

Three load levels, 65%, 75% and 80% of the ultimate shear capacity (91.6kN/m) were used in the test. The dead weight of the HC slabs is 3.65kN/m^2 including joint castings.



(a)



(b)

Figure 7. (a)Test layout (b) HC dimension from BEF (2005)

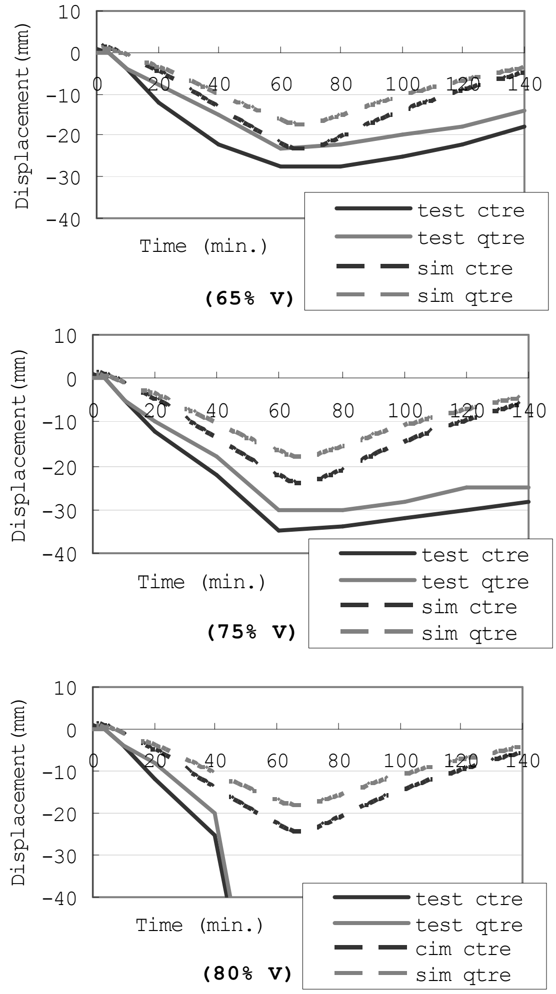


Figure 8. Comparison between the vertical displacements at midspan (ctre) or at the location of the point load (qtre) from simulation (sim) and test (test) results

The test results showed that the HC slabs have good shear resistance under elevated temperature. The comparison between the analytical prediction and experimental results in Figure 8 shows that the vertical displacements calculated from the simulation are close to the test data. The difference between the vertical deflection at the centre and the side of the unit in the simulations is almost the same as that from the tests. The difference between the predictions and the test data is presumed caused by shear deformation. In the case with an applied load equal to 80% of the slab's shear capacity, the slab had a shear failure after 45mininutes of ISO fire exposure. The model could not foresee the shear failure and therefore it continued to provide results after this time. The large shear force was not captured in the analysis, and the model underestimated the deflection. Nevertheless, in normal practice such high level of shear force is never designed for.

4.3 *Danish Institute of Fire Technology, DIFT*

The last set of data is from the three fire tests carried out by DIFT in 1998 on HC slabs (Andersen & Lauridsen 1999). Three different hollowcore slab sections were tested with 185mm (SP18), 220mm (SP22) and 270mm (SP27) thickness. The self-weights were 2.75kN/m^2, 3.10kN/m^2 and 3.55kN/m^2 respectively. The slabs were one way pin-supported spanning 6.2m and had no topping as shown in Figure 9(a). Four equal line loads were applied to give a total load of 135.4kN, 135.4kN and 112.1kN respectively. The characteristic concrete strength of the HC unit was 2MPa in tension and 54MPa in compression. The mechanical properties and prestressing condition of the strands in each case are shown in Table 2. The grillage system used to model these cases is shown in Figure 9(b).

Table 2. Material properties of strands in DIFT 1999 test

Section type	**SP18**	**SP22**	**SP27**
Strands (per unit)	8 of Φ9.3 mm		4 of Φ12.5 mm, 4 of Φ15.2 mm
Yield strength	1880 N/mm^2		1840 N/mm^2
Mechanical prestressing	62kN/strand		110 kN/strand for Φ12.5mm, 150 kN/strand for Φ15.2mm
Modulus of elasticity	198 kN/mm^2		

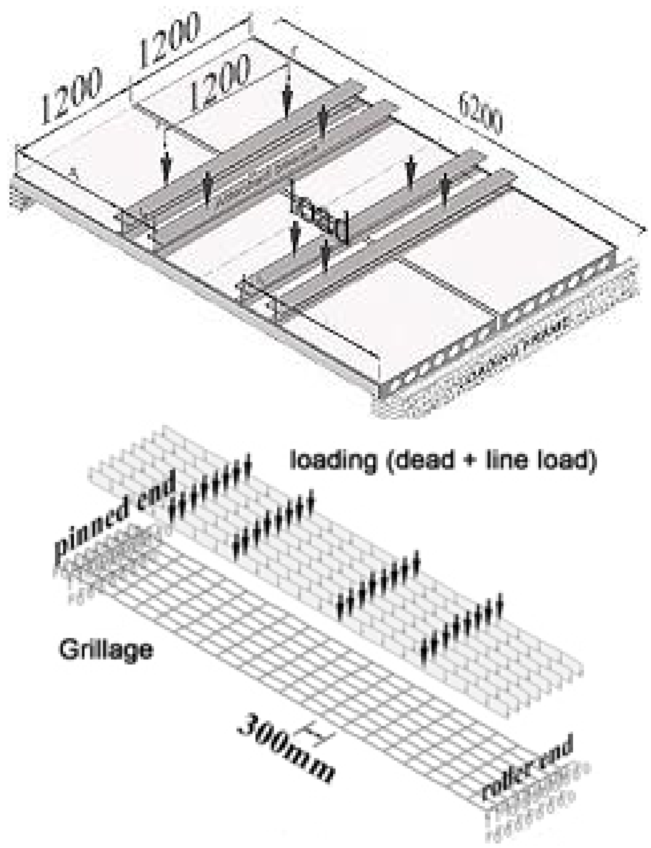


Figure 9. (a) Layout of the DIFT test; (b) SAFIR model

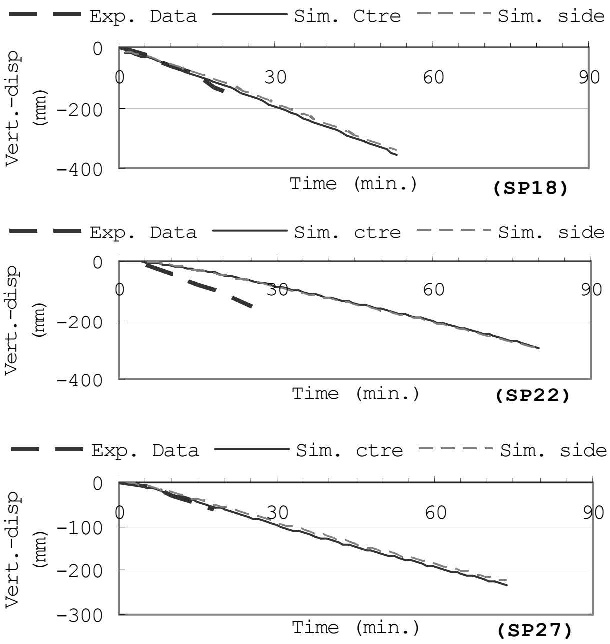


Figure 10. Comparison of midspan vertical displacement from simulation (centre, side) with that from the experiments (centre) of SP18, SP22 and SP27

The results of the simulation are shown in Figure 10. In the experiments all three specimens had shear failures, which again cannot be predicted in the simulation. Nevertheless, the vertical deflection from the model showed a good agreement with the experimental data in SP18 and SP27 cases, but underestimates the deflection in SP22.

5 CONCLUSIONS

A new modelling scheme is proposed to simulate the behaviour of HC floor system under fire. The new scheme uses a grillage system to model the HC unit, and a layer of shell elements to model the RC topping slab. The advantage of the new scheme is that it recognises the effects of thermal expansion in the transverse direction, and it also can model the membrane action through the topping layer.

The new model can predict the fire performance of HC slabs well, on the condition that no shear failure or significant shear displacements are present. It is expected that this new model could work better in actual building design than in the simulating test results, because shear failure or displacement is likely to be significantly reduced with the presence of axial restraints.

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