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ULTIMATE STRENGTH OF REINFORCED AND STEEL FIBRE REINFORCED CONCRETE (SFRC) CONTINUOUS COMPOSITE SLABS (AN EXPERIMENTAL STUDY)

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ABSTRACT

A comprehensive experimental study was conducted on steel fibre reinforced concrete (SFRC) continuous composite slabs to quantify crack width and ultimate strength resulting from gravity loading. Sixteen full scale slab specimens were cast with different types of interface bond between the concrete slab and steel decking (e.g. greased, unembossed decking or standard decking) and different amounts of reinforcement in concrete (e.g. mesh, steel fibre or normal reinforcing bars). Each slab was continuous over the interior support and had a roller support at each end. All the slabs were tested under increasing load until failure. The mid-span deflection and end slip versus applied load were monitored and the crack sizes were obtained for each slab for different levels of applied load.

It was found that under the serviceability load levels no end slip occurred which indicates the full interaction between steel decking and concrete slab. In terms of crack control in negative bending moment region, application of fibre dosages of 20 kg/m³ and 40 kg/m³ to the unreinforced composite slab had almost similar effects. However, when the steel fibre dosage was increased to 60 kg/m³, a very significant improvement in crack control was achieved as the maximum crack width was often reduced by 50%.

Introduction

Composite flooring systems are widely used in the construction of suspended floors in steel-framed multi-storey buildings around the world because of their relative efficiency compared to many other flooring systems. In this type of floors, the steel decking supports the self-weight of a cast in-situ reinforced or post-tensioned concrete slab and, after the concrete sets, acts as external reinforcement. Embossments on the profiled sheeting provide the necessary shear connection to ensure composite action between the concrete and the steel decking (Ranzi et al. 2013).

The composite action between the steel decking and the hardened concrete is dependent on the transmission of horizontal shear stresses acting on the interface between the concrete slab and the steel decking (Stark 1978, Stark and Brekelmans 1990, Oehlers and Bradford 1995, Johnson 2004). Composite action and the transmission of horizontal shear stresses at the concrete-steel interface are necessary for the steel decking to perform its role as the tension reinforcement for the system. The composite action between

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the concrete and the steel decking is achieved, not only by chemical bonding between the steel decking and the concrete, but also by mechanical interlock between the concrete and the embossments on the profiled steel decking. Further composite action can be attained by attaching shear studs or similar shear devices to provide end anchorage. Resistance against vertical separation is also achieved by the embossments and by adopting a suitable shape for the decking profile.

The steel decking is usually supplied in two-span lengths and negative reinforcement is provided in the top of the slabs over the supports during construction. This makes the composite slab normally continuous. However, it is often beneficial for practising engineers to assume that the slab is simply-supported in ultimate limit states and use linear-elastic analysis. These 'nominally simply-supported' slabs require top longitudinal reinforcement at their support, primarily to control the width of cracks under serviceability actions. Therefore, Standards/Codes introduce a nominal amount of reinforcement in the form of percentage of the cross-section area to be arranged for crack control at the support. Generally, this reinforcement is in the form of mesh or welded wire fabric, and will often be sufficient to cause the shrinkage and temperature stresses to be relieved in small local cracks rather than accumulating over greater distances to form more widely spaced large cracks.

Placing of the mesh reinforcement in composite concrete slabs increases the cost, construction complexity, and erection speed. Therefore, the partial or total replacement of the conventional reinforcement (e.g. rebar, mesh) with other forms of crack control has economical benefits, due to eliminating or reducing labour time in construction. One solution of interest is the use of steel fibre reinforced concrete (SFRC), since it may offer structural performance benefits which can be translated into cost savings. However, there are a number of outstanding issues related to understanding the behaviour of composite slabs for design, especially considering shrinkage, the effects of different amounts of steel fibre reinforced concrete, the performance under severe inelastic loading demands of fire or earthquake.

Ackermann and Schnell (2008) carried out two different test series on continuous and simply-supported composite slabs considering different fibre dosages and steel decking profiles. The simply-supported slabs were tested to determine the possible rotation of steel fibre reinforced composite slabs in the regions of negative bending. Their results demonstrated that after formation of the first plastic hinge at the mid-span, the slabs attained a good rotation capacity in the order of 10^{-2} radians at peak load in the hogging area and the hogging moment was approximately constant over a wide range of rotation. Similarly in continuous composite slabs after cracking and formation of a plastic hinge at mid-support, sufficient rotation ability was formed at the mid-support and the hogging moment was approximately constant up to the failure of the slab, whereas the sagging moments were increased as the load increased. The hogging moments were redistributed to the mid-span until the sagging plastic moment capacity was reached. This was mainly as a result of crack distribution ability of the steel fibre reinforced concrete.

Abas et.al (2013) tested eight double-span composite slabs fabricated with deep trapezoidal decking and steel fibre reinforced concrete to failure and reported the effects of varying the steel fibre dosage on the cracking behaviour at the negative moment region; on the redistribution of bending moments; on the end slip between the steel decking and the concrete slab; and on the load carrying capacity of the slabs. Compared to the plain concrete composite slab and the slab containing SL62 welded wire mesh in the negative moment region over the interior support (without fibre), the slabs containing steel fibres in excess of 20 kg/m^3 provided significant improvements in the slip load and the peak load capacity. Under typical service load levels the fibres provided crack control that was of similar effectiveness to that provided by the SL62 mesh. In addition, the deflections at both the peak load and the slip load were greater for the slabs containing fibres.

Gholamhoseini et al. (2013) tested three continuous composite concrete slabs for failure and evaluated the longitudinal shear failure mode of each slab. The slabs were lightly reinforced only over the mid-support. They also modelled the slabs in a non-linear finite element software package using an interface element to model the contact between the steel decking and concrete slab.

Scope of Research

Despite the aforementioned research works, there is still limited knowledge on the short-term behaviour of SFRC composite slabs. In particular, extensive further research work is required to clearly understand the beneficial effects of the steel fibres on the rotational capacity of composite slabs over the area of hogging moments and redistribution of internal actions, as well as on the bond-slip characteristics in continuous composite slabs and to quantify these effects for the design codes. More investigations are required in terms of the effects of steel fibres on the load carrying capacity of continuous composite slabs with or without negative reinforcement over the supports.

This paper illustrates the results of an experimental study on the short-term behaviour of continuous composite slabs with different types and amounts of reinforcement. Sixteen two-span composite slabs were cast and tested to failure to study their ultimate performance. The effects of different parameters (e.g. amount of steel fibre, steel decking surface condition, presence of reinforcing bars or mesh) have been studied and reported.

Experimental Program

Sixteen two-span continuous composite slabs were constructed using trapezoidal steel decking ComFlor 80 that is widely used in New Zealand. The composite slabs were cast with the profiled steel decking as permanent formwork. The steel decking was cleaned thoroughly before placing the concrete. All the slabs were covered with wet hessian and plastic sheets within four hours of casting and moist cured for 14 days after casting the concrete as shown in Fig. 1.



Figure 1. Composite Slabs after Concrete Casting.

The slabs were 150 mm thick, 1.2 m wide and 6.3 m long consisting of two spans of 3.0 m plus an 150 mm overhang from each exterior support in short-term test. Each slab was continuous over the interior support and was simply-supported on a roller at each of the two exterior supports. The cross-section of the composite slabs is shown in Fig. 2. The details of the composite slabs are summarised in Table 1. One type of profiled steel sheeting was used in all the slabs except for L1 and L2 slabs which had no embossments on their surface. The steel sheeting with no embossments was provided by the manufacturer specifically for this research work. Figure 2 also shows the cross sectional dimensions of the steel sheet profile. The decking was formed from 0.9 mm thick grade G500 zinc coated steel sheeting produced according to AS 1397-2011.

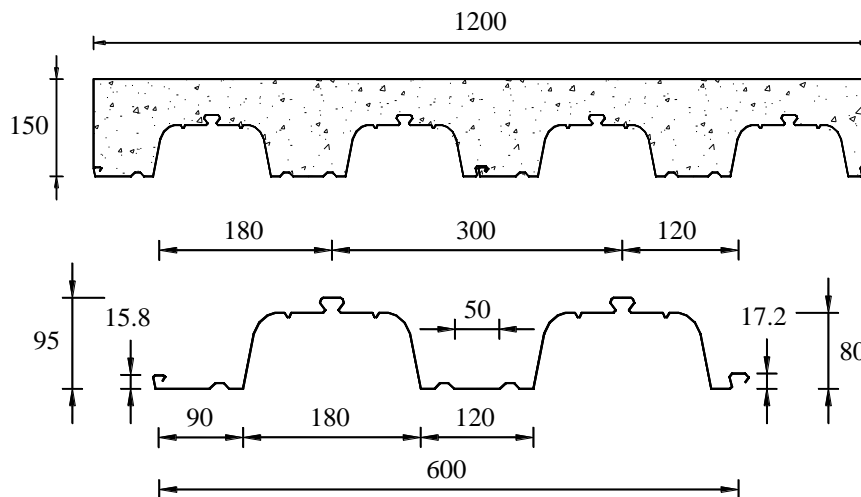


Figure 2. Cross-sectional Dimensions of the Composite Slabs and Steel Decking Profile ComFlor 80.

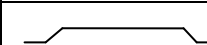
Table 1. Details of Composite Slabs

Slab	Decking surface condition		Reinforcement type	Bar diameter (mm)	Reinforcement area (mm ² /m)	Cast No.
	Embossment	Greased				
L1	N	N	-	-	-	I
L2	N	Y	-	-	-	I
L3	Y	Y	-	-	-	I
L4	Y	N	-	-	-	I
L5	Y	N	Fibre (20 kg/m ³)	-	-	I
L6	Y	N	Fibre (40 kg/m ³)	-	-	II
L7	Y	N	Fibre (60 kg/m ³)	-	-	IV
L8	Y	N	Mesh 665 (with deformed bars)	5.3	147.1	II
L9	Y	N	Mesh 661 (with deformed bars)	7.5	294.5	II
L10	Y	N	Mesh SE62 (with plain bars)	6.1	147.0	II
L11	Y	N	Mesh SE92 (with plain bars)	9.0	318.0	II
L12	Y	N	HD 12@260	12	565.5	III
L13	Y	N	HD 10@260	10	392.7	III
L14	Y	N	Mesh SE62+HD 12@260 L=1500 at Mid-support	12	712.5	III
L15	Y	N	Fibre (40 kg/m ³)+ HD 12@260 L=1500 at Mid-support	12	565.5	II
L16	Y	N	Fibre (20 kg/m ³)+ HD 12@260 L=1500 at Mid-support	12	565.5	I

Slabs L1, L2, L3 and L4 were constructed to study the effect of steel decking in restraining the shrinkage induced stresses and deformations. That study is beyond of the scope of this paper and its results are presented elsewhere. Conventional plain concrete (without steel fibres or mesh) was used to cast these slabs. Steel decking in slabs L1 and L2 did not have embossments to eliminate any mechanical bond. The surface of the steel sheeting in slabs L2 and L3 was greased to eliminate the chemical bond (adhesion). In slab L4, both mechanical and chemical bond was provided.

In order to study the effect of reinforcement amount and type, different types of reinforcement were used in the slabs. For slabs L5, L6 and L7, SFRC with 20 kg/m³, 40 kg/m³ and 60 kg/m³ were used, respectively. One type of steel fibre, Dramix RC-80/60-BN, was used in the tests. The fibres were high strength, hooked-end cold drawn produced by Bekaert. The dimensions and nominal material properties of the steel fibres (provided by the supplier) are given in Table 2.

Table 2. Nominal Properties of Steel Fibre

Length l_f (mm)	Diameter d_f (mm)	Aspect Ratio l_f/d_f	Ultimate Strength f_{uf} (MPa)	Shape
60	0.75	80	1050	

Mesh reinforcement was used in slabs L8, L9, L10 and L11. Class L (low ductility) mesh consisting of deformed bars with a minimum yield stress of 485 MPa and a minimum uniform elongation of 1.5% was considered in slabs L8 and L9. For slabs L10 and L11, ductile mesh (Class E) using plain bars with a minimum yield stress of 500 MPa and a minimum uniform elongation of 10% was used. High strength reinforcing bars with diameters of 12 mm and 10 mm were placed in slabs L12 and L13, respectively. In slab L14 continuous mesh SE62 (similar to slab L10) was provided together with additional 1.5 m long reinforcement over the interior support. Slabs L15 and L16 had the same additional reinforcement as slab

L14 over the interior support but, instead of reinforcing mesh, steel fibres with dosages of 20 kg/m³ and 40 kg/m³ were used, respectively. The minimum concrete cover to any rebar or mesh was 25 mm.

Short-Term Test Setup

All test specimens were lifted and positioned in a four-point bending loading test setup with shear span of span/3 = 1m at each span. Lifting and moving the slabs was done with caution by means of a lifting frame which was specifically designed to avoid or minimise any unexpected damage and to maintain existing geometry and support conditions. A schematic photo of the test setup is shown in Fig. 3.



Figure 3. Failure Test Setup View.

During the test of the slabs to failure, central deflection in both spans, deflections under the loading points, and end slip at both ends were measured using linear variable displacement transducers (LVDTs). The load was applied by a hydraulic jack of capacity 500 kN and distributed to the top surface of each slab at each loading point using a spreader beam. The load was applied in a displacement control manner at a rate of 0.3 mm/min. Crack widths in the hogging moment region, over the middle support, were measured manually by means of a microscope with an accuracy of 0.01 mm.

Test Results

Material Properties

The compressive strength, elastic modulus and splitting tensile strength of concrete were measured on three standard 100 mm diameter x 200 mm long cylinders. All the strength tests were carried on both plain and fibre reinforced concrete samples and the mean results are presented in Table 3.

Table 3. Properties of Plain Concrete and Fibre Reinforced Concrete

Material Property	Cast No. I		Cast No. II		Cast No. III	Cast No. IV	
	Plain	With Fibre (20 kg/m ³)	Plain	With Fibre (40 kg/m ³)	Plain	Plain	With Fibre (60 kg/m ³)
f_c (MPa)	34.8	36.2	34.6	35.8	37.1	34.6	33.8
E_c (GPa)	33.4	32.8	32.6	33.4	32.2	30.4	31.9
f_t (MPa)	3.3	4.1	4.6	4.3	3.8	3.8	3.9

Additionally, the average elastic modulus E_s and the yield stress f_{yp} of the steel decking were also measured on coupons cut from the decking. The samples were cut from the slabs after the failure tests and were from the end part of slabs that were not under high tensile strains during the test. The average of the measured values of yield stress and elastic modulus of the steel decking were $f_{yp} = 635$ MPa and $E_s = 220$ GPa, respectively. Similarly, standard tests were carried out on three samples to measure the elastic modulus, the yield stress, the ultimate strength and the ultimate strain of steel mesh and reinforcing bars and the results are presented in Table 4. The yield strain in Table 4 was calculated as: $\epsilon_y = f_y / E_s$.

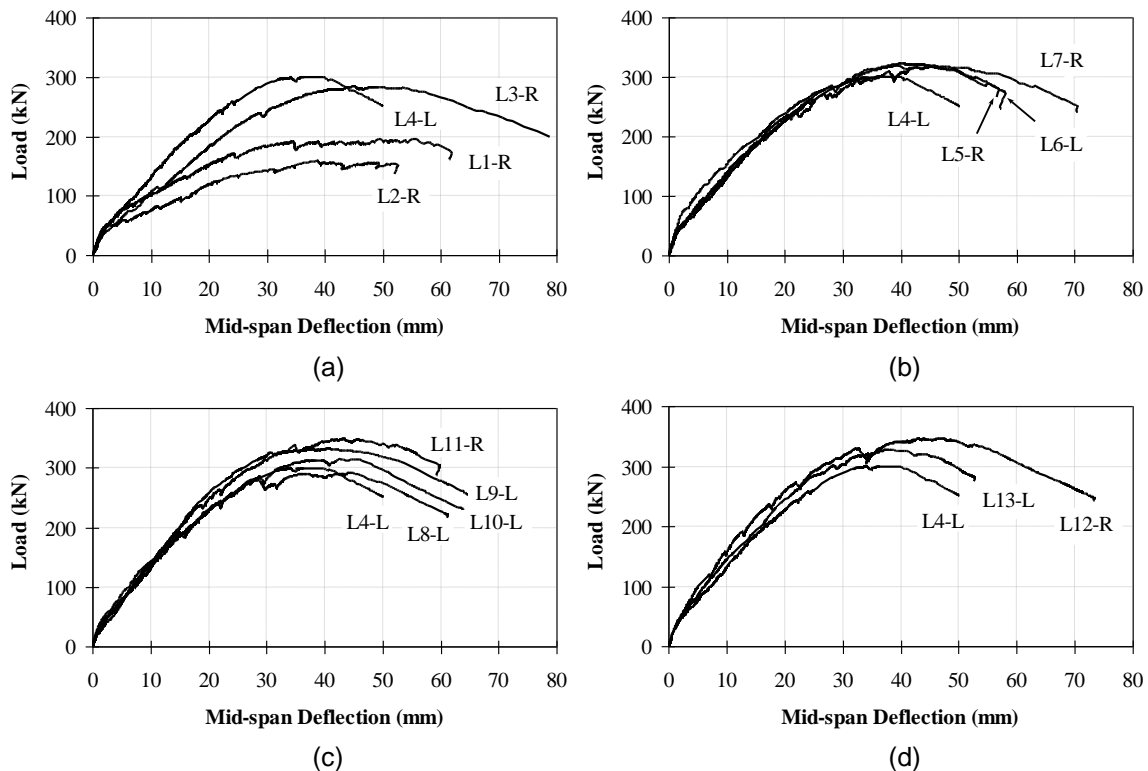
Table 4. Material Properties of Mesh and Rebar

Material	E_s (GPa)	f_y (MPa)	f_u (MPa)	ϵ_y (%)	ϵ_u (%)	f_u / f_y	ϵ_u / ϵ_y
Mesh 665	193	656	714	0.34	3.4	1.09	10.0
Mesh 661	193	623	668	0.32	3.5	1.07	10.9
Mesh SE62	187	502	620	0.27	10.4	1.24	38.5
Mesh SE92	192	569	648	0.30	5.9	1.14	19.7
Rebar HD 12	178	503	618	0.28	14.2	1.23	50.7
Rebar HD 10	200	635	675	0.32	10.7	1.06	33.4

Short-Term Test Results

At first loading all the slabs were quite stiff and had a linear elastic stiffness until an applied load of about $P_{cr} = 60$ kN (as shown in Fig. 4). Flexural cracks then formed in the interior support region and propagated into the cross-section. Significant post-cracking strength was observed in all slabs. However, the ultimate load capacity was not controlled by either compression in concrete or tension in steel decking, but by the interface slip between the two materials. Failure was considered to have occurred when one of the spans had deflected excessively, and hence the load had dropped significantly (by more than 15%) below the ultimate load capacity.

Figure 4 shows the applied load versus mid-span deflection of the test slabs. As expected, the lowest peak load was for slab L2 which did not have either chemical or mechanical bond. Slab L4 with normal concrete and steel decking and no reinforcement was used as a reference composite slab to evaluate the effect of any reinforcement in the other test specimens. The peak load for slab L4 was 299.5 kN.



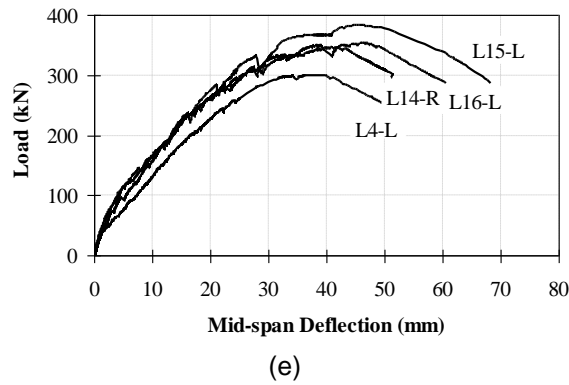


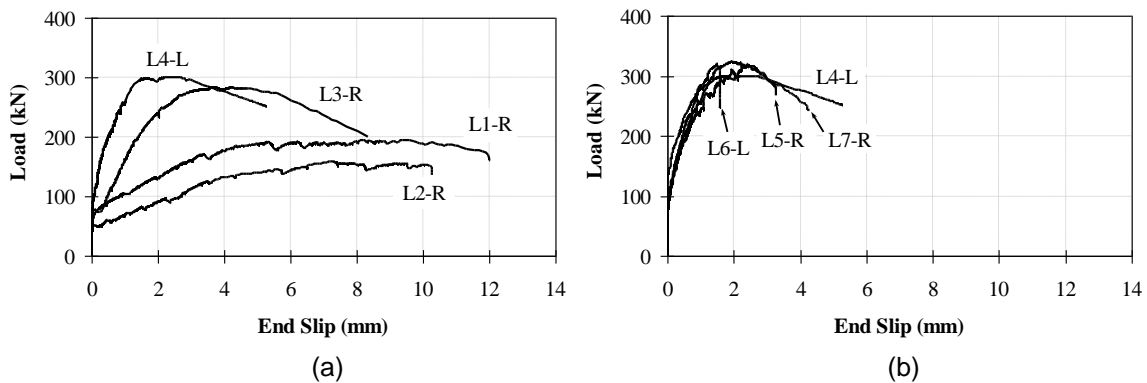
Figure 4. Mid-span Deflection vs Applied Load.

Composite slabs with different dosages of fibre (i.e. 20, 40 and 60 kg/m³) performed very similarly in terms of ultimate strength and displacements. The peak load values in slabs L5, L6 and L7 were 318.9 kN, 319.3 kN, 323.7 kN, respectively. Compared to the slab L4 that had no reinforcement, the ultimate strength in slabs L5, L6, L7 was increased by about 7% whereas the load causing the end-slip of 0.1 mm ($P_{(0.1mm)}$) in slab L5 (with 20 kg/m³ fibre) and slab L6 (with 40 kg/m³ fibre) was increased by about 5% and 10%, respectively. However, when the steel fibre increased to 60 kg/m³ in slab L7, $P_{(0.1mm)}$ was increased significantly by about 42% compared to that in slab L4.

In the mesh-reinforced slabs, the peak load increased with reinforcement ratio. The peak load values in slabs L8, L9 and L10 were 290.0 kN, 332.3 kN, 314.6 kN, respectively. Although slabs L8 and L10 had same percentage of reinforcement, there was 24.6 kN difference in their peak load. This difference can be attributed to the different surface conditions of reinforcements and early slip between the reinforcement and surrounding concrete.

The longitudinal shear failure mode determines the post-slip strength and behaviour of composite slabs. According to Eurocode 4, the longitudinal shear behaviour of a simply-supported slab may be considered to be ductile if the failure load (P_u) exceeds the load causing a recorded end slip of 0.1 mm ($P_{(0.1mm)}$) by more than 10%. Although this definition is presented for simply-supported slabs only, there is not a specific definition regarding the behaviour of continuous composite slabs. If this definition is also assumed to be applicable to continuous composite slabs, all slabs failed in a ductile manner. Significant post-slip strength was achieved in all slabs as the ratio of $P_u / P_{(0.1mm)}$ in all slabs was greater than 2.1.

Graphs of end slip versus applied load are shown in Fig. 5. In slab L3 with mechanical bond only and slabs L4-L16 with mechanical and chemical bonds, the ultimate load capacity was reached when the end slip of 1.5-3.7 mm was occurred. In slabs L3-L16, the presence of surface bond and reinforcement (as fibre or reinforcing bars or both) increased the ultimate load capacity of slabs to at least 80% more than that of slab L2 that had no reinforcement of any type or surface bonding.



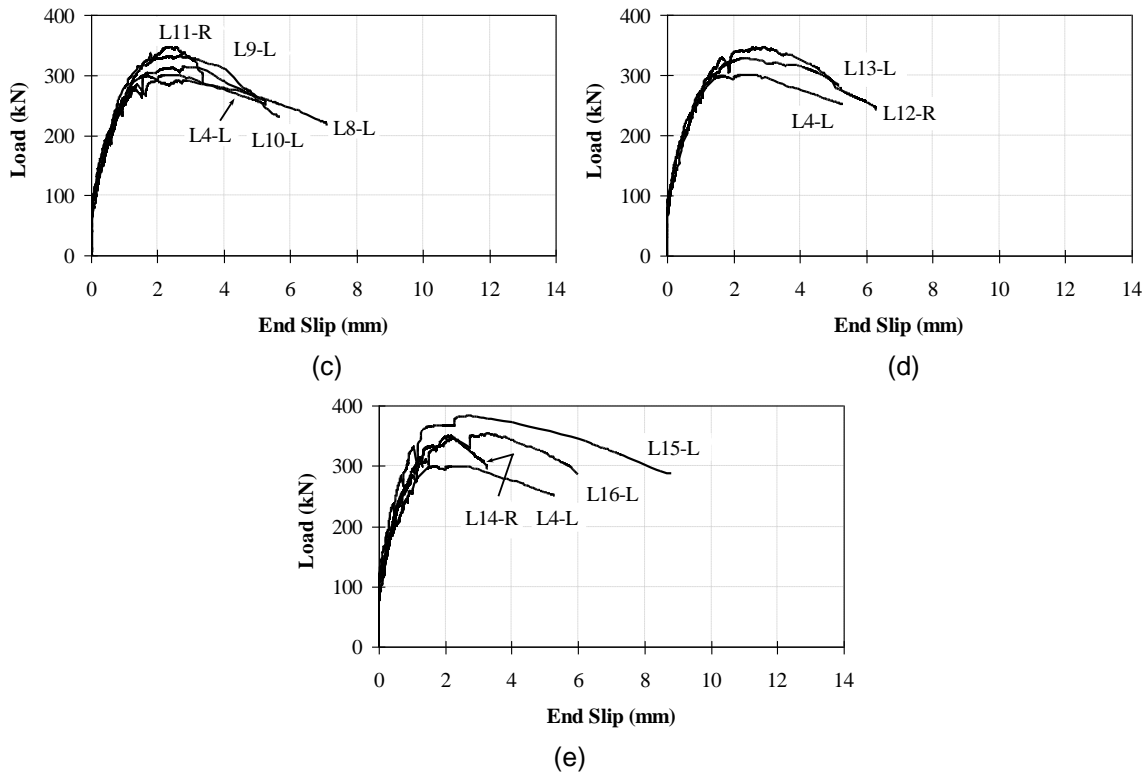
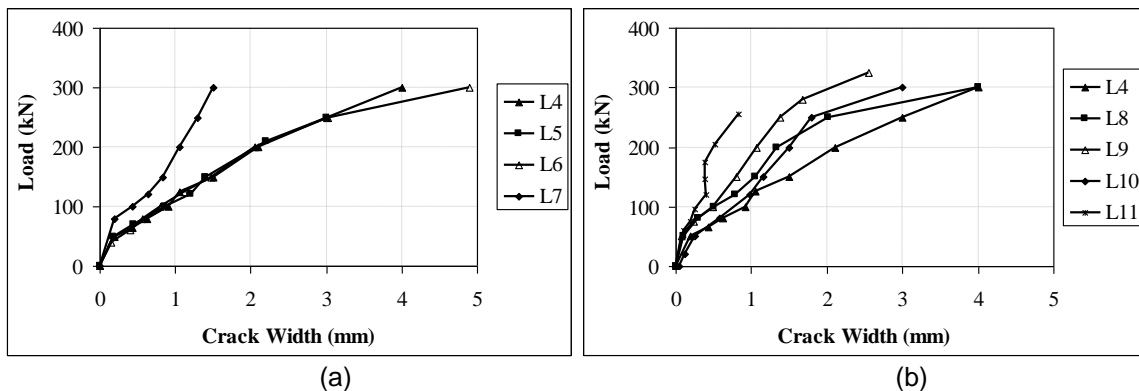


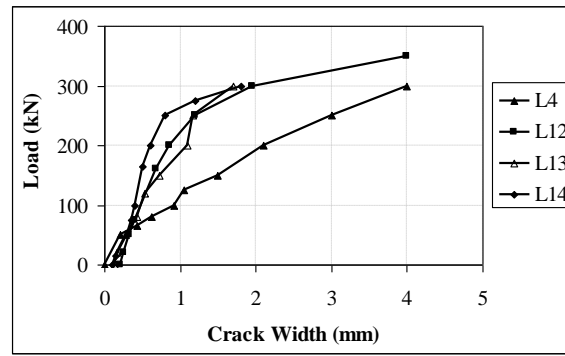
Figure 5. End-slip vs Applied Load.

The maximum crack widths were measured over the interior support during the failure loading test in slabs L4-L14 and the results are shown in Fig. 6. Crack width values for slabs L4, L5 and L6 were almost identical for the same level of loads. In other words, no improvement was gained in terms of crack control by inclusion of 20 and 40 kg/m³ of steel fibres compared to the plain concrete. On the other hand, crack width control in slab L7 with 60 kg/m³ of steel fibres was significantly improved and the maximum crack width was often reduced by 50%, as shown in Fig. 6a.

In slabs with mesh reinforcement, maximum crack widths were decreased as the steel ratio was increased. The extent of cracking in slab L7 (with 60 kg/m³ of steel fibre) was marginally greater than the best degree of crack control in all slabs reinforced with mesh reinforcement that was in slab L11. The cracked surface of slab L6 over the interior support with crack width of 4.9 mm at the end of the test is shown in Fig. 7.

In mesh-reinforced composite slabs, slab L9 with reinforcement ratio of 0.42% had the most effective cracking control. Slab L8 cracked at lower load level compared to slab L10 with reinforcement ratio of 0.21%. The crack width values at higher load levels were closer in slabs L8 and L10. In slabs with ductile reinforcing bars (L12 and L13) and slab L14 with continuous ductile mesh and additional reinforcing bars at the interior support, the crack width was significantly less than that in slab L4.





(c)

Figure 6. Crack Width vs Applied Load.

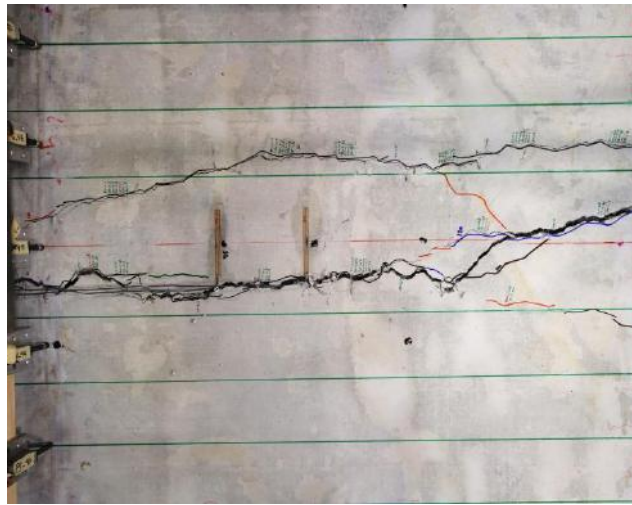


Figure 7. Final Cracked Surface of Slab L6 over the Interior Support.

Summary and Conclusions

This paper describes a comprehensive experimental study on sixteen full-scale continuous two span composite slabs with steel decking and width of 1.2 m and total length of 6.3 m. Each slab had different types and amount of reinforcement. From four-point bending failure test of composite slabs, the following conclusions were drawn:

- 1- No end slip occurred under the serviceability load levels indicating full interaction between the steel decking and concrete slab.
- 2- The failure mode in all slabs was interface slip at the ends. The behaviour was ductile according to Eurocode 4.
- 3- Significant post-cracking and post-slip strength gain in all slabs were observed as the ratio of P_u / P_{cr} and $P_u / P_{(0.1mm)}$ in all slabs was greater than 2.6 and 2.1, respectively.
- 4- Compared to the slab with plain concrete, the slabs that contained 20 kg/m^3 and 40 kg/m^3 of steel fibre had a slightly higher slip load (in the order of 5% and 10%, respectively). On the other hand, when the fibre dosage increased to 60 kg/m^3 the slip load was 42% greater than that of unreinforced composite slab.
- 5- Composite slabs with different dosages of steel fibre performed very similarly in terms of ultimate strength with their average being only 7% greater than that of an unreinforced composite slab.
- 6- Compared to the slab with plain concrete, the average of deflections at peak load of SFRC composite slabs was 16% higher.
- 7- SFRC composite slabs with fibre dosages of 20 kg/m^3 and 40 kg/m^3 performed almost identically to the unreinforced composite slab, in terms of crack control in negative bending moment region.

8- Application of steel fibres for crack control became effective for slab with SFRC with 60 kg/m^3 of fibre, which showed a very significant improvement in crack control as the maximum crack width was often reduced by 50%. The crack control performance was better than that of the mesh-reinforced slabs.

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