EXPERIMENTAL BEHAVIOUR OF FIBRE-REINFORCED CONTINUOUS COMPOSITE SLABS

BRADFORD Mark [1], ABAS Fairul [2], GILBERT Ian [3], FOSTER Stephen [4]

Abstract

A study is described of the behaviour of short-term quasi-static load testing of two-span composite slabs fabricated with conventional deep trapezoidal steel decking and steel fibre reinforced concrete cast in situ. Replacing conventional reinforcement in the negative moment region with fibre reinforcement alleviates the need for on-site steel fixing and ensures that quality is maintained stringently. Eight two-span composite slabs were tested with different levels of fibre dosages (20, 30 and 40 kg/m3), including a slab without reinforcement and one with welded-wire mesh only. Compared to the plain concrete slab and that containing mesh, continuous slabs containing in excess of 20 kg/m3 provide significant improvements in the peak load and in the slip load. Moreover, at service load levels the fibres provide crack control that is of similar effectiveness to that provided by welded wire mesh.

Keywords: Composite slabs; steel-fibre reinforced concrete; partial shear connection; trapezoidal profiles; shear bond.

1. Introduction

Composite steel-concrete flooring slab systems are used widely in contemporary building construction. The understanding and the application of this structural system have been in place for many years. Composite slabs with deep trapezoidal steel decks are gaining a favourable reputation as economic load carrying members and this system is a very efficient form of construction. In a steel-concrete composite slab, the profiled steel sheet acts as the bottom, tensile reinforcement in the positive moment region of the slab. The strength of the composite slab depends, however, on the longitudinal shear resistance at the interface between the concrete slab and the profiled steel sheeting [1]. Current practice uses plain concrete reinforced with conventional reinforcing mesh for early-age shrinkage control and fire resistance purposes.

The use of steel fibres in concrete as an alternative (or supplement) to conventional reinforcement is now a mature technology, after almost 40 years of experience and extensive research being devoted to the technology. The variety of fibres being produced and used by the construction industry is growing steadily, and the range of application of

[1] BRADFORD Mark, CIES, UNSW Australia, UNSW Sydney, NSW 2052, Australia, m.bradford@unsw.edu.au

[2] ABAS Fairul, CIES, UNSW Australia, UNSW Sydney, NSW 2052, Australia, fairul@unsw.edu.au

3 GILBERT Ian, CIES, UNSW Australia, UNSW Sydney, NSW 2052, Australia, i.gilbert@unsw.edu.au

4 FOSTER Stephen, CIES, UNSW Australia, UNSW Sydney, NSW 2052, Australia, s.foster@unsw.edu.au

steel fibre reinforced concrete (SFRC) is expanding. The inclusion of steel fibres in concrete has been shown to improve the post-cracking behaviour of a reinforced concrete member, both in terms of crack control and ductility. Therefore, the viability of combining composite steel-concrete slabs and steel fibres warrants investigation, and is addressed in the present paper.

However, as with all composite slabs, their strength relies heavily on the resistance to vertical separation and to horizontal slippage between the contact surface of the concrete and the profiled steel sheet. Therefore, it is crucial that this strength aspect of a composite slab system is established prior to its application in practice. An understanding of how the steel fibres affect the longitudinal strength of a composite slab system is much needed.

Specifically, this paper describes three groups of short-term static load tests on single-span composite slabs. The aim of the tests was to study the behaviour and the shear-bond strength of composite slabs with steel fibre dosages of 30 kg/m^3 and 60 kg/m^3 in comparison to a control slab without steel fibres. It produces data for strength calibration using the well-known *m*-*k* procedure [2] and for the longitudinal shear bond strength τ_u for the partial shear connection method [1].

2. Test setup

Single-span composite slabs fabricated using deep -deck) trapezoidal steel sh and steel fibre reinforced concrete were cast and loaded by static and cyclic load patterns up to failure. Three (3) groups of specimens were constructed from a ready-mix concrete containing steel-fibres with dosages of 0 kg/m^3, 30 kg/m^3 and 60 kg/m^3 of end hooked fibres high strength steel (Dramix RC80/60 BN). In total, 17 single-span composite slab specimens were cast and moist cured for a period of 28 days and then loaded to failure. The specimens were divided into 3 major groups: 5 slabs in Group 1; 6 in Group 2 and 6 in Group 3, according to the dosage of steel fibres. Each specimen was 3.5 m long (3.3 m between supports) and the average depth of all slabs was 140 mm. The concrete properties, including compressive strength, tensile strength, modulus of elasticity and fracture energy, were measured on companion specimens for every slab tested. An elevation and cross-section of a typical slab is shown in Fig. 1. For each specimen, the deflection at mid-span, the crack widths near the loading points and slip between the decking and the concrete at both ends of the slab were recorded at each load increment from zero to failure of the specimen. Also recorded continuously throughout the tests were the applied loads and the reactions at each support.

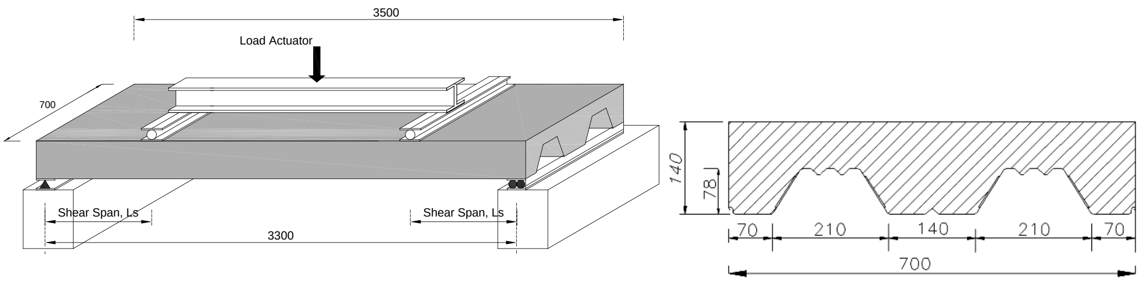

Fig. 1: Dimensions and sections

Tab. 1: Details of test specimens.

Slab #	*t*	Shear span	Fibre dosage	Loading	f_c
Group 1 – Plain concrete slab					
S1-A-0	145	825	0	static	50
S2-A-0	142	825	0	static + cyclic	50
S1-B-0	143	420	0	static	50
S2-B-0	145	420	0	static + cyclic	50
S3-B-0	143	420	0	static + cyclic	50
Group 2 – 30 kg/m3 steel fibres					
S1-A-30	142	825	30·67	static	45·3
S2-A-30	146	825	30·67	static + cyclic	45·3
S3-A-30	142	825	30·67	static + cyclic	45·3
S1-B-30	142	420	30·67	static	45·3
S2-B-30	146	420	30·67	static + cyclic	45·3
S2-B-30	148	420	30·67	static + cyclic	45·3
Group 3 – 60 kg/m3 steel fibres					
S1-A-60	142	825	64·62	static	52·3
S2-A-60	143	825	64·62	static + cyclic	52·3
S3-A-60	145	825	64·62	static + cyclic	52·3
S2-B-60	142	420	64·62	cyclic	52·3
S2-B-60	144	420	64·62	static + cyclic	52·3
S2-B-60	147	420	64·62	static + cyclic	52·3

Fig. 2: Formwork and slab immediately after casting.

Fig. 2 shows the spreader beam arrangement. Load was applied at the third-span points in both spans. The initial load, comprising of the self-weight of the slab, the weight of the spreader beams and the weight of all packing used in each test, was measured at the beginning of the test as the sum of the reactions measured by the load cells at each support.

60 mm long hooked end steel fibres (RC80/60BN) were used in the test specimens. The properties of the steel fibres are given in Table 2. The steel fibres were mixed to the required dosages in the back of the concrete truck in the laboratory. The quantities of the

measured steel fibre added to the concrete for each test specimen were calculated and the exact dosage was calculated after mixing using standard wash-out tests.

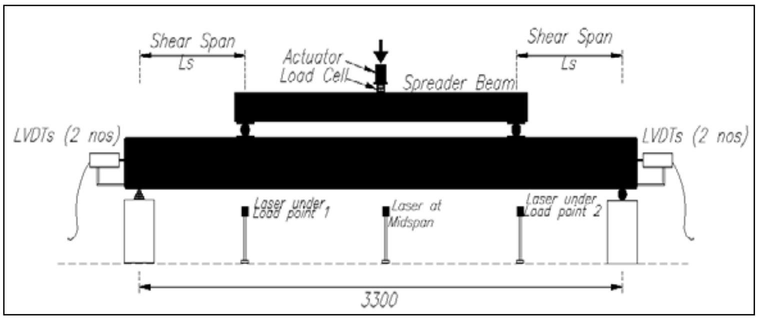

Fig. 3: Test setup and loading arrangement

Tab. 2: Nominal properties of steel fibres

Steel fibre	Diameter (mm)	Length (mm)	Mean tensile strength (MPa)	Elastic modulus (GPa)
RC80/60BN (L)	0·75	60	1225	200

The instrumentation included: (1) laser transducers, each with an accuracy of 0·01 mm, to measure the deflection of the slabs at the mid-point of each span; (2) load cells to measure the actuator loads (3) LVDTs to measure the slip between the steel deck and the concrete at each end of the slab; (4) surface mounted strain gauges and Demec gauges to measure concrete strain at the top surface of the slab at the loading points.

The test procedure for all groups was identical. Loading was applied by a servo-controlled Instron hydraulic actuator via longitudinal and transverse spreader beams based at two different positions for the shear spans. The long shear spans were at ¼ *L* and ¾ *L* of the span length *L* while the short shear spans were symmetrically at 420 mm measured from the each support. The applied load was measured by using a calibrated load cell that was placed directly under the actuator. The loading procedure was in accordance with the guidelines given in Eurocode 4 [3] with some minor modification of the determination of the range of cyclic loading. One slab from each series of every group was tested with static load only while the other two slabs of the same series were tested with 5000 cycles of loading for a duration of 3 hours, prior to loading to failure under increasing load. The first slab from each series was tested with static load only in order to determine the level of cyclic load for the other two slabs in the same series. The initial test (cyclic load test) was conducted by applying the cyclic load with the lower and upper limit of 20% and 60% of the failure load respectively obtained from the first slab tested with static load only. The failure load was taken as the load that produced substantial slips causing the load to drop significantly and this value was taken from each of the first slab from each series that was tested with only static load.

3. Test results

For each group of test specimens, accompanying material testing was also conducted. A summary of the material test results is given in Table 3.

Tab. 3: Material test results

Slab #	Compressive	Elastic	Direct	Flexural	Fracture
Group 1	50·0	30,900	4·20	4·67	125
Group 2 (30kg/m^3)	45·3	30,100	3·84	6·20	5,300
Group 3 60kg/m^3)	52·3	27,500	4·38	7·57	11,800
Group 3 (S1-A-60)	49·0	24,300	4·28	8·27	19,190

The results for the tests for the long shear span (series A) and short shear span (series B) are shown in Tables 4 and 5 respectively. All specimens behaved in a similar manner. Fig. 4 shows the load versus deflection response of the slab specimen in series A loaded with static load only. The first-crack load occurred at relatively low load: the plain concrete slab cracked at the load of 10 kN; that with a fibre dosage of 60 kg/m^3 cracked at slightly higher load of 14 kN. Slab specimens loaded with the short shear span had higher cracking loads compared to the long span series, ranging from 15 kN to 25 kN as well as higher peak loads than the series A slabs as shown in Fig. 5. The loading continued with a reduced stiffness up to the point where the slip between the deck and the concrete occurred. The load dropped as the first slipping occurred.

The end slip was also recorded electronically. Tables 7 and 8 show the key values of load at 0·1 mm slip and the recorded slip at failure for every slab specimen in each series respectively. It can be seen that from the tables that significant slip occurred at failure. The ratios of $P_u/P_{0.1}$ are all greater than 1·1 which classify the slab behavior as ductile in accordance with Eurocode 4. The end slip versus applied load for the slab specimens tested with static load from both series A and B are shown in Figs. 6 and 7 respectively. The slab with a long shear span and with a dosage of 60kg/m^3 was not instrumented with electronic transducers to measure the end slips, but rather with mechanical gauges. As such, the reading obtained was deemed considerably inaccurate and omitted from the results.

Slip took place at both ends of the test specimen. It was observed that the slip rates were different between the two ends. As the load increased, the cracked constant moment region became noticeably larger and eventually the slabs failed due to the loss of longitudinal shear strength resulting in significant interface slip at the shear span region as shown in Fig. 8.

4. Evaluation of bond shear strength

The results based on the *m-k* procedure are given in Figs. 9 to 11, while those based on the partial shear connection procedure are given in Fig. 12. In order to determine the longitudinal shear strength τ_u, and in order 3 and simplify the spreadsheet construction of the curve, values of η = 0, 0·2, 0·4, 0·6, 0·8 and 1 were assumed, and the graph is given in Fig. 12.

Tab. 4: Test results for Group A (shear span = 825 mm)

Slab #	Peak load P_u (kN)	Cracking load P_{cr} (kN)	Cracking moment M_{cr} (kNm)	Ultimate moment M_u (kNm)	Moment ratio M_u/M_{cr}	Deflection at failure d_{max} (mm)
S1-A-0	37	10	4.13	15.26	3.700	64
S2-A-0	33	10.75	4.43	13.61	3.070	61
S1-A-30	44.8	12	4.95	18.48	3.733	60
S2-A-30	50	11.75	4.85	20.63	4.255	58
S3-A-30	47	12.75	5.26	19.39	3.686	72
S1-A-60	52.6	14	5.78	21.08	3.650	32
S2-A-60	51	16.75	6.91	21.04	3.045	45
S3-A-60	55.75	15	6.19	23.00	3.717	67

Tab.5: Test results for Group B (shear span = 420 mm)

Slab #	Peak load P_u (kN)	Cracking load P_{cr} (kN)	Cracking moment M_{cr} (kNm)	Ultimate moment M_u (kNm)	Moment ratio M_u/M_{cr}	Deflection at failure d_{max} (mm)
S1-B-0	59	20	8.25	24.34	2.950	26
S2-B-0	59.5	21	8.66	24.54	2.833	48
S3-B-0	57	21.5	8.87	23.51	2.651	42
S1-B-30	91.5	20	8.25	37.74	4.575	57
S2-B-30	90.5	25	10.31	37.33	3.620	44.5
S3-B-30	94	15	6.19	38.78	6.267	43
S1-B-60	100.3	22	9.08	41.25	4.545	36
S2-B-60	108.5	18	7.43	44.76	6.028	52
S3-B-60	111.3	19	7.84	45.79	5.842	55

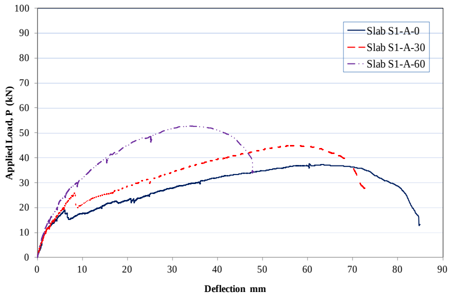

Fig. 4: Load-deflection for Series A

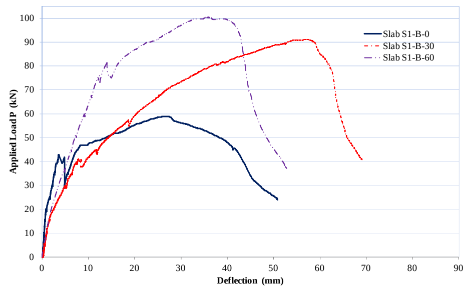

Fig. 5: Load-deflection for Series B

In determining the plastic moment of resistance of the composite beam, the plastic moment for the profiled sheeting is needed, and this needs an effective area to be calculated to account for local buckling of the thin steel sheeting [1]. The effective area can be computed from the effective width b_{eff}, taken as

$$b_{eff} = \frac{857 t_s}{\sqrt{f_{yp}}}\left(1 - \frac{857 t_s}{b\sqrt{f_{yp}}}\right) \leq \frac{b-15}{2} \text{ mm} \qquad (1)$$

where f_{yp} = 650 MPa is the yield strength of the profiled sheeting of thickness t_s = 1 mm.

The plastic moment of resistance of the plain concrete composite slabs ($M_{pl,Rd}$) for full interaction is 48·94 kNm while the plastic moment resistance of the profiled sheet alone (M_{pa}) is 5·80 kNm. The degree of interaction for each group was determined based on the ultimate bending moment obtained from the test results. The values of degree of shear connection and the longitudinal shear resistance for each test group, ignoring the effect of friction at the support reaction, are tabulated in Table 9.

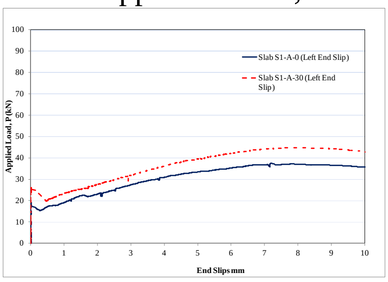

Fig. 6: Load-slip for Series A.

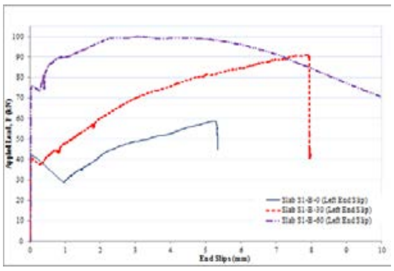

Fig. 7: Load-slip for Series B

Fig. 8: Typical slab failure

Tab. 6: Slip results for Group A (shear span = 825 mm)

Slab #	Load at 0·1 mm slip $P_{0.1}$ (kN)	Peak load P_u (kN)	Ratio $P_u/P_{0.1}$	Slip at failure S_f (mm)
S1-A-0	17.5	37	2.1	7.6
S2-A-0	21.0	33	1.6	4.7
S1-A-30	26.2	44.8	1.7	8.1
S2-A-30	32.7	50	1.5	8.1
S3-A-30	17.0	47	2.7	9.8
S1-A-60	-	51.1	-	-
S2-A-60	31.0	51	1.6	6.0
S3-A-60	30.7	55.75	1.8	6.5

Tab. 7: Slip results for Group B (shear span = 420 mm)

Slab #	Load at 0·1 mm slip $P_{0.1}$ (kN)	Peak load P_u (kN)	Ratio $P_u/P_{0.1}$	Slip at failure S_f (mm)
S1-B-0	42.5	59	1.4	5.5
S2-B-0	40	59.5	1.5	7
S3-B-0	40.75	57	1.4	5.8
S1-B-30	41	91.5	2.3	8
S2-B-30	46.5	90.5	1.9	7.5
S3-B-30	34	94	2.8	8.5
S1-B-60	76	100	1.3	4.5
S2-B-60	47.5	108.5	2.3	7.2
S3-B-60	57.7	111	1.9	6.5

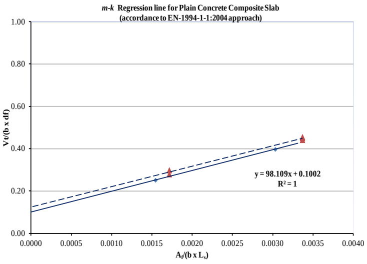

Fig. 9: *m-k* for plain concrete

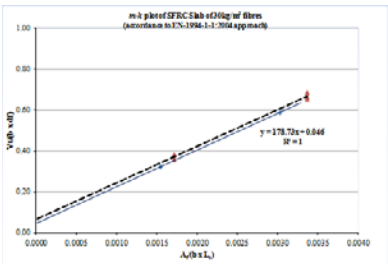

Fig. 10: *m-k* for 30 kg/m^3

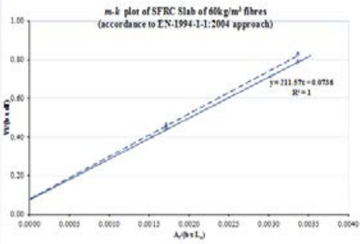

Fig. 11: *m-k* for 60 kg/m^3

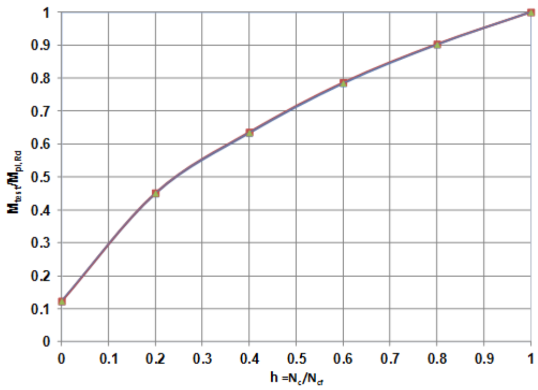

Fig. 12: Interaction diagram for Series A

Tab. 8: *m-k* parameters from tests

Slab type	m (MPa)	k (MPa)	τ_u (MPa)
Plain concrete composite slab	98	0·100	0·27
Composite slab with 30kg/m^3 steel fibres	179	0·046	0·35
Composite slab with 60kg/m^3 steel fibres	209	0·038	0·40

Tab. 9: Shear bond strengths

Test Group	Slab #	$M_{ult}/ M_{pl,Rd}$	η	τ_u	Average τ_u
Plain concrete composite slab	S1-A-0	0·42	0·15	0·100	0·095
	S2-A-0	0·38	0·13	0·089	
SFRC Composite slab with 30kg/m^3 steel fibres	S1-A-30	0·50	0·20	0·138	0·147
	S2-A-30	0·55	0·23	0·158	
	S3-A-30	0·52	0·21	0·145	
SFRC Composite slab with 60kg/m^3 steel fibres	S1-A-60	0·58	0·24	0·165	0·167
	S2-A-60	0·56	0·23	0·158	
	S3-A-60	0·61	0·26	0·179	

Tab. 10: Pertinent values of longitudinal shear stress

	Low-slip shear span			High-slip shear span		
Slab	Shear stress (MPa)	Max. shear (MPa)	Slip at peak (mm)	Shear stress at (MPa)	Max. shear (MPa)	Slip at peak (mm)
S1-A-0	0·136	0·147	2·84	0·118	0·137	7·60
S2-A-0	0·133	0·142	4·50	0·152	0·157	4·70
S1-A-30	0·134	0·146	1·30	0·165	0·245	8·14
S2-A-30	0·176	0·227	2·82	0·205	0·205	8·02
S3-A-30	0·205	0·218	3·90	0·126	0·215	9·80

For the force equilibrium procedure, the moment lever arm is calculated based on strain gauge readings obtained at the top surface of the concrete, the bottom fibre of the profiled sheeting and from slip strains determined from the slip displacements at the end of the shear span. Figs. 13 and 14 show the shear stress versus end slip at both ends for plain concrete and SFRC with a dosage of 30 kg/m^3. From these graphs, it can be seen that the slabs with

SFRC have higher shear bond stress at the initiation of slip. It can also be seen that the initiation of slip is different at each end of the specimen (this is typical behaviour) and, as a result, different values of the longitudinal shear strength eventuate. The more ductile behaviour at the "high slip" end (Fig. 14) is associated with lower strengths than at the "low slip" end (Fig. 13). Table 10 presents the pertinent values of the longitudinal shear stress, viz. at first slip and at peak load.

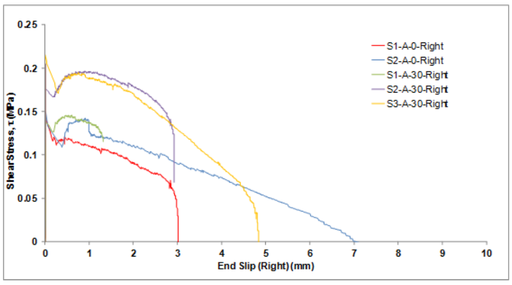

Fig. 13: Bond stress-slip at "low slip" end

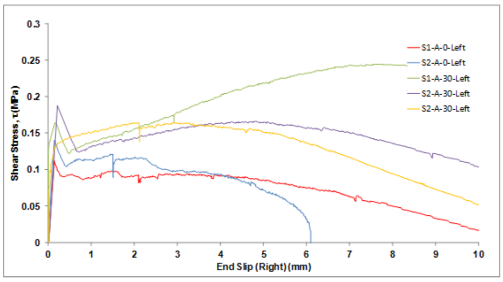

Fig. 14: Bond stress-slip at "high slip" end

Tab. 10: Pertinent values of longitudinal shear stress

Slab specimen	Low-slip shear span: Shear stress at first slip (MPa)	Low-slip shear span: Max. shear stress (MPa)	Low-slip shear span: Slip at peak load (mm)	High-slip shear span: Shear stress at first slip (MPa)	High-slip shear span: Max. shear stress (MPa)	High-slip shear span: Slip at peak load (mm)
S1-A-0	0·136	0·147	2·84	0·118	0·137	7·60
S2-A-0	0·133	0·142	4·50	0·152	0·157	4·70
S1-A-30	0·134	0·146	1·30	0·165	0·245	8·14
S2-A-30	0·176	0·227	2·82	0·205	0·205	8·02
S3-A-30	0·205	0·218	3·90	0·126	0·215	9·80

5. Discussion

All composite slabs tested showed significant post-cracking moment capacity after first slip, and showed significant ductility prior to failure by longitudinal shear well below their flexural capacity. Table 11 compares the longitudinal shear strength between the plain concrete or SFRC slab and the steel sheeting. The *m-k* method produces a higher value of the shear bond strength, whilst the results based on the partial shear connection and force equilibrium procedures are in close agreement.

Tab. 11: Comparison of longitudinal shear strengths for longer shear spans

Slab Specimen	Average longitudinal shear bond strength (MPa)		
	m-k	Partial interaction	Force-equilibrium
Plain concrete	0·27	0·095	0·112
SFRC with 30kg/m^3	0·35	0·147	0·165
SFRC with 60kg/m^3	0·40	0·167	-

6. Conclusions

An experimental program involving the full-scale testing of simply supported plain concrete and SFRC composite slabs with profiled steel sheeting were presented and discussed. All slab specimens were tested under four-point bending with shear spans of either 820 mm or 420 mm. The shear bond strength in the test specimens were assessed by *m-k* procedure, the partial shear connection procedure and the force equilibrium procedure.

All slabs failed by longitudinal shear with substantial end slip displacements being recorded at both ends of the specimens with the sheeting remaining in the elastic range of structural response. Slabs containing steel fibres had higher peak loading and shear bond strengths compared to their plain-concrete counterparts. Additionally, the shear strengths derived from the *m-k* procedure were higher than those determined from the partial shear connection and force equilibrium procedures, which showed closer agreement.

Acknowledgements

The work reported in this paper was undertaken with the financial support of the Australian Research Council through Linkage Project LP0991495, BlueScope Lysaght Pty Limited and BOSFA Pty Limited. The second author was also supported by the Australian Research Council through Australian Laureate Fellowship FL100100063. This support is gratefully acknowledged.

References

[1] OEHLERS, D. J., BRADFORD, M. A. *Composite steel and concrete structural members: Fundamental behaviour*. Oxford: Pergamon, 1995.

[2] PORTER, M., EKBERG, C. Design reccomndations for steel deck floor slabs. *3rd International Specialty Conference on Cold-Formed Structures*, University of Missouri-Rolla, MO, USA. 1975.

[3] BRITISH STANDARDS INSTITUTION. Eurocode 4. London, BSI, 2004.