Creep and Shrinkage Effects on the Bond-Slip Characteristics and Ultimate Strength of Composite Slabs

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Abstract: Composite one-way concrete slabs with profiled steel sheeting as permanent formwork are commonly used in the construction industry. The steel sheeting supports the wet concrete of a cast-in-situ reinforced or post-tensioned concrete slab and, after the concrete sets, acts as external reinforcement. In this type of slab, longitudinal shear failure between the concrete and the steel sheeting is the most common type of failure at the ultimate load stage. Design codes require the experimental evaluation of the longitudinal shear capacity of each type of steel decking using full-scale tests. This paper presents the results of the short-term testing up to failure of two types of profiled steel decking that are commonly used in the construction industry in Australia. Fourteen full-scale, simply-supported slabs were tested in four-point bending with shear spans of either span/4 or span/6. Four slabs were tested at age of 28 days and the other 10 slabs were subjected to drying shrinkage and various levels of sustained loads for a period of at least 6 months prior to testing to failure. The effects of creep and drying shrinkage on the load carrying capacity and deformation of the slabs at ultimate loads are presented and discussed. The bond-slip relationship of each slab is determined from the test data and the values of maximum longitudinal shear stress calculated using different methods are described and compared.

Key words: Composite slabs, creep and shrinkage, longitudinal shear stress, partial interaction, profiled steel sheeting.

1. Introduction

Composite slabs consisting primarily of cold-formed profiled steel sheeting and structural concrete are increasingly used in steel framed buildings worldwide. In this system, the steel sheeting is normally continuous over two-spans between the supporting steel beams and during construction the concrete is poured to form a continuous one-way composite slab.

The steel sheeting has two main roles in this type of floor system: Firstly, it serves as permanent formwork supporting the wet concrete during construction; Secondly, it acts as external reinforcement for the slab, carrying the tension induced by positive bending moment throughout the life of the structure. If the strength provided by the steel sheeting is not adequate,

additional reinforcement might be included in the concrete slab, as shown in Fig. 1.

The composite action between the steel sheeting and the hardened concrete is dependent on the transmission of horizontal shear stresses acting on the interface between the concrete slab and the steel sheeting [1-3]. Composite action and the transmission of horizontal shear stresses at the concrete-steel interface are necessary for the steel sheeting to perform its role as the tension reinforcement for the system. The composite action between the concrete and the steel sheeting is achieved, not only by chemical bonding between the sheeting and the concrete, but also by mechanical interlock between the concrete and the embossments on the profiled steel sheeting. Further composite action can be attained by attaching shear studs or similar shear devices. Resistance against vertical separation is also achieved by the embossments and by adopting a suitable shape

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for the decking profile. Since the longitudinal shear capacity of a particular type of steel decking is geometry dependent, it is generally evaluated experimentally using full-scale load tests.

The effects of creep and non-uniform drying shrinkage on the serviceability of composite slabs have recently been quantified and reported [4-9], but little research has been carried out to evaluate these effects on the load bearing capacity, ultimate deflection and bond-slip characteristics of composite slabs.

In this paper, a series of full-scale, simply-supported slab tests is described using two types of profile decking that are widely used in Australia and the results are presented and evaluated. Four slabs were tested at age 28 days and the other 10 slabs were subjected to drying shrinkage and various levels of sustained loads for a period of at least 6 months prior to testing to failure. Slab specimens were tested in four-point bending with shear spans of either span/4 or span/6. The bond-slip relationship of each slab was determined during the testing and the values of maximum longitudinal shear stress calculated using two different methods (subsequently introduced as the

“partial shear connection” and “force equilibrium” methods) are presented and compared.

The measured mid-span deflection, the end slip and the mid-span steel and concrete strains are also presented.

2. Bond-Slip Relationship

A composite slab must be designed to carry the factored design loads at the ultimate limit state. Fig. 2a shows an elevation and a cross-section of a typical composite slab specimen in a four-point bending test. There are three likely failure modes: (1) failure in flexure at the peak moment region (i.e., at Section b-b); (2) longitudinal shear failure at Section c-c; (3) vertical shear failure at Section a-a.

Fig. 1 Elements of a composite slab and notation.

Fig. 2 Shear stress versus inverted slenderness in a composite slab in a four-point bending test: (a) elevation and section of a slab specimen in four-point bending test; (b) nominal shear stress versus inverted slenderness of slab.

Fig. 2b graphs the relationship between the nominal shear stress (V_t/bd_p) and the inverted slenderness of the slab (A_p/bL_s), where, V_t is the support reaction; A_p is the cross-sectional area of the steel sheeting; b is the width of the slab cross-section; d_p is the effective depth of the slab and L_s is the shear span. Each of the above failure modes occurs in different parts on the graph, with flexural failure controlling when L_s is large, vertical shear failure when L_s is small and longitudinal shear failure for intermediate values of L_s . As shown later, the terms of (A_p/bL_s) and (V_t/bd_p) are also used in Eurocode 4 [10] for evaluating of longitudinal shear stress capacity.

Longitudinal shear failure in the shear span L_s due to shear stress at the steel-concrete interface is the most common type of failure for medium-span slabs at the ultimate limit state. Load tests carried out on full-scale composite slabs have shown that significant slip between the steel sheeting and the concrete occurs at relatively high levels of loading and both stiffness and strength are significantly reduced [2, 3, 11-14].

The longitudinal shear capacity of a particular type of steel decking is dependent on many aspects including the decking thickness and geometry, loading arrangement, shear span length and slenderness of the slab. Because of this diverse dependency, it is not possible to present an explicit value for shear stress capacity that can be employed in design for all slab conditions and therefore, it is generally evaluated experimentally using full-scale load tests.

The longitudinal shear stress-interface slip behavior of a certain type of profiled steel decking is frequently examined by performing push-out tests (mainly due to its simplicity). However, the main defect of push-out tests is related to the fact that the flexural effects of the concrete slab and steel decking are not considered, and hence the attained results are not suitable to be used in the design in accordance with current specifications.

Eurocode 4 gives two methods to evaluate the longitudinal shear capacity of composite slabs, known

as the “ m - k ” and “partial shear connection” methods. In both methods, the longitudinal shear capacity is assessed by using full-scale laboratory tests to measure slab performance. Full-scale slab specimens are required because the longitudinal shear capacity is dependent on the geometry and flexibility of the particular type of steel sheeting, including the size and spacing of the embossment on the sheeting, as well as on the slenderness of the slab (i.e., the span-to-depth ratio).

The m - k method to evaluate the design resistance against longitudinal shear is mainly based on the work of Porter and Ekberg [13, 14]. After much experimentation, they proposed the following shear design equation for one way slabs with cold formed steel decking as:

$$\frac{V_u s}{b d_p} = m \frac{A_p}{b L_s} + k \sqrt{f'_c} \quad (1)$$

where, V_u is the ultimate shear force (N); s is the spacing of shear transferring devices (mm); b is the width of slab (mm); f'_c is the compressive strength of concrete (MPa); m and k are empirical values to be determined from the tests.

In the m - k method, it must be shown that the maximum design vertical shear V_{Ed} in the slab does not exceed the design shear resistance $V_{1,Rd}$ calculated as:

$$V_{1,Rd} = \frac{b d_p}{\gamma_{VS}} \left(m \frac{A_p}{b L_s} + k \right) \quad (2)$$

where, γ_{VS} is the partial safety factor for the ultimate limit state with a recommended value of $\gamma_{VS} = 1.25$ [10].

In the “partial shear connection” method, the flexural capacity of a composite slab is calculated by using simple plastic analysis of the section and by employing rectangular stress blocks for the concrete and profiled steel sheeting as shown in Fig. 3 [3, 10]. According to Eurocode 4 [10], this method may only be used when the longitudinal shear behavior has been shown in tests to be ductile. By definition, the longitudinal shear behavior may be considered to be

ductile if the failure load exceeds the load causing a recorded end slip of 0.1 mm by more than 10%. As a result of a longitudinal shear test, the bending moment at mid-span of a slab is M_{test} and the degree of shear connection η_{test} is defined as: $\eta_{\text{test}} = N_c/N_{cf}$ where, N_c is the compression force in the concrete and N_{cf} is the compression force in the concrete at full shear connection. At the full shear connection stage, the flexural resistance of the section is $M_{p,Rm}$. As shown in Fig. 3, in this method, the stress in profiled steel sheeting is taken as its yield strength f_{yp} and the stress in the concrete is taken to be $0.85f_{cm}$ where, f_{cm} is the mean cylinder compressive strength of concrete.

The mean value for the ultimate longitudinal shear stress τ_u can be calculated as:

$$\tau_u = \frac{\eta_{\text{test}} N_{cf}}{b(L_s + L_o)} \quad (3)$$

If the extra longitudinal shear stress produced by the support reaction is considered, the value of τ_u may be calculated as:

Fig. 3 Degree of shear connection determined from M_{test} .

Fig. 4 Free body diagram, strain variation and internal forces.

$$\tau_u = \frac{\eta_{\text{test}} N_{cf} - \mu R}{b(L_s + L_o)} \quad (4)$$

where, μ is the friction coefficient (taken as $\mu = 0.5$) and R is the support reaction at the ultimate test load. This is done for a range of shear spans, and the lowest value found is adopted for design [3].

In order to carry out numerical modeling by the FEM (finite element method), the bond-slip relationship between the steel decking and the concrete is necessary. The average bond-slip relationship is calculated using the “force equilibrium” method. This method was initially used by An [15] and Abdullah and Easterling [16] to calculate the relationship between longitudinal shear force and end-slip in a bending test. Fig. 4 shows a free body diagram of the shear span of the slab at failure and the strain variation and internal force distribution at the critical section. After slip occurs, the steel decking takes part of the load by bending about its own axis M_r . In Fig. 5, the resisting moment of steel decking M_r can be calculated from:

Fig. 5 Deflection and radius of curvature of the decking.

$$L' - 2L_s = \frac{(\delta_1 + \delta_2)R_f}{L_s}, \frac{1}{R_f} = \frac{M_r}{E_s I_s}, \quad (5)$$

$$M_r = \frac{(\delta_1 + \delta_2)E_s I_s}{L_s (L' - 2L_s)}. \quad (6)$$

If the self-weight of concrete is neglected, the longitudinal shear force F can be written as:

$$F = T_s = \frac{\left(\frac{PL_s}{2} - M_r\right)}{z} \quad (7)$$

where, z is the moment lever arm; δ_1 and δ_2 are the deflections measured under each applied loads $P/2$ and E_s and I_s are the modulus of elasticity and moment of inertia of steel decking, respectively.

The value of moment lever arm z depends on the location of the neutral axis of composite section. With the measured strain in steel ϵ_s , strain in concrete ϵ_c and slip strain ϵ_{sl} all obtained from the tests, the neutral axis depth x can be written as:

$$x = \frac{\epsilon_c}{\epsilon_c + \epsilon_s + \epsilon_{sl}} \cdot t \quad (8)$$

$$z = t - y_{sd} - \frac{x}{3} = d_p - \frac{x}{3} \quad (9)$$

where, t is the overall thickness of the slab and y_{sd} is the height to the centroid of the steel profile measured from the soffit of the slab. The longitudinal shear stress distribution is assumed to be uniform in the shear span, and hence the longitudinal shear stress is calculated using the horizontal contact area of the

steel sheeting and concrete in the shear span as indicated in Eq. (10). If the extra longitudinal shear stress produced by the support reaction is considered, the value of τ_{ave} can be written as in Eq. (11):

$$\tau_{ave} = \frac{F}{b(L_s + L_o)}, \quad (10)$$

$$\tau_{ave} = \frac{F - \frac{\mu P}{2}}{b(L_s + L_o)}. \quad (11)$$

The maximum shear stress value obtained in this method can be used in other analytical methods, such as “partial shear connection” method for slab design [10].

3. Experimental Study

3.1 Test Specimens

The behavior of two different commonly used trapezoidal (or wave-form) profiles (KF40, KF70) manufactured by Fielders Australia [17] has been studied. The shape, dimensions and properties of these profiles are shown in Fig. 6. The thickness of the steel sheeting for both profiles used in the experiments was $t_s = 0.75$ mm.

For each KF70 and KF40 decking profile, seven slabs were cast and then cured for seven days under wet hessian. The steel decking was fully supported on the laboratory floor during casting of the concrete to minimize initial stress or deformation in the steel sheeting. No reinforcing steel was included in the slabs. The mean compressive strength of the concrete at the age of testing (with and without prior sustained loading) for each slab type was determined from six standard 100 mm diameter cylinder tests. The elastic modulus E_s and the yield stress f_{yp} of the steel sheeting were also measured on three coupons cut from each sheet of decking. The material properties are shown in Table 1.

The dimensions of each slab were 3,300 mm $\times$ 1,200 mm $\times$ 150 mm for KF70 slabs or 3,400 mm $\times$ 1,200 mm $\times$ 150 mm for KF40 slabs, respectively. The center to center distance between the two end supports (one hinge and one roller) was 3,100 mm. For each profile type, two shear spans of $L_s = L/4$ and $L_s = L/6$ were tested. A photograph of the five KF70 slabs showing the different loading arrangements for the sustained loading period and the slab designations are shown in Fig. 7.

The dimensions, self-weight, applied superimposed sustained load and reinforcement ratio of each slab are given in Table 2. The first two letters in the designation of each slab indicate the nature of the test, with LT for long-term and ST for short-term. The next two numbers indicate the type of decking (with 70 and 40 for KF70 and KF40, respectively). The next digit indicates the shear span length, with 4 and 6 representing shear spans of $L_s = L/4$ and $L_s = L/6$, respectively. The final digit indicates the approximate value of the maximum superimposed sustained

loading (in kPa) that each slab carried during period of sustained load.

3.2 Short-Term Testing Setup

For each decking profile two slabs were tested with shear span of $L_s = L/4$ and $L_s = L/6$. Each slab had a roller support at one end and a hinge support at the other end. The deflection at mid-span and under the applied concentrated loads and the end slip at both the roller and hinge supports were measured using LVDTs (linear variable displacement transducers). The strains in concrete slab and steel sheeting were measured at selected sections on the top and bottom surfaces of slabs using 60 mm long strain gauges. A schematic view of the experimental setup and the measured parameters are shown in Fig. 8. The load was applied in a displacement control manner at a rate of 0.3 mm/min.

3.3 Long-Term Loading Procedure and Testing Setup

Prior to loading to failure, the remaining five slabs for each decking profile were put onto supports to experience shrinkage and different levels of sustained load. Each of the KF70 slabs was placed onto its supports at age seven days and remained unloaded (except for its self-weight) until age 64 days. At age 64 days, with the exception of LT70-6-0, each slab was subjected to superimposed sustained loads in the form of concrete blocks. Each concrete block was placed on 60 mm high timber blocks to ensure a largely uninterrupted air flow over the top surface of the slabs and allow the concrete to shrink freely on the top surface.

Slab LT70-6-0 carried only self-weight for the full test duration of 240 days (3.0 kPa). Slabs LT70-4-3

Table 1 Material properties of each slab.

Slab decking	Age	f_{cm} (MPa)	$f_{ct,f}$ (MPa)	E_c (MPa)	f_{yp} (MPa)	E_s (GPa)
KF70	64 days	28.0	3.50	30,725	544	212
	247 days	29.8	4.54	31,650		
KF40	28 days	35.5	3.80	28,200	475	193
	251 days	42.7	5.05	31,600		

Fig. 7 View of KF70 slabs during the sustained load period.

and LT70-6-3 were identical, carrying a constant superimposed sustained load of 3.4 kPa from age 64 days to 247 days, i.e., a total sustained load of 6.4 kPa. Slab LT70-6-6 carried a constant superimposed sustained load of 6.0 kPa from age 64 days to 247 days, i.e., a total sustained load of 9.0 kPa. Slab LT70-6-8 carried a constant superimposed sustained load of 6.1 kPa from age 64 days to 197 days, i.e., a total sustained load of 9.1 kPa and from age 197 days to 247 days the superimposed sustained load increased to 7.9 kPa, i.e., a total sustained load of 10.9 kPa.

Table 2 Dimensions and reinforcement ratio of each slab.

Slab designation	Self-weight (kPa)	Superimposed sustained load (kPa)	L' (mm)	d_p (mm)	A_p (mm ²)	$\rho (A_p/bd_p)$	Shear span, L_s (mm)
ST70-4	3.0	-	3,100	122.3	1,320	0.0090	$L'/4 = 775$
LT70-4-3	3.0	3.4	3,100	122.3	1,320	0.0090	$L'/4 = 775$
ST70-6	3.0	-	3,100	122.3	1,320	0.0090	$L'/6 = 517$
LT70-6-0	3.0	0.0	3,100	122.3	1,320	0.0090	$L'/6 = 517$
LT70-6-3	3.0	3.4	3,100	122.3	1,320	0.0090	$L'/6 = 517$
LT70-6-6	3.0	6.0	3,100	122.3	1,320	0.0090	$L'/6 = 517$
LT70-6-8	3.0	7.9	3,100	122.3	1,320	0.0090	$L'/6 = 517$
ST40-4	3.2	-	3,100	136.0	1,248	0.0076	$L'/4 = 775$
LT40-4-3	3.2	3.4	3,100	136.0	1,248	0.0076	$L'/4 = 775$
LT40-4-6	3.2	6.4	3,100	136.0	1,248	0.0076	$L'/4 = 775$
ST40-6	3.2	-	3,100	136.0	1,248	0.0076	$L'/6 = 517$
LT40-6-0	3.2	0.0	3,100	136.0	1,248	0.0076	$L'/6 = 517$
LT40-6-3	3.2	3.4	3,100	136.0	1,248	0.0076	$L'/6 = 517$
LT40-6-6	3.2	6.4	3,100	136.0	1,248	0.0076	$L'/6 = 517$

Fig. 8 Test setup and measured parameters.

Table 3 Measured mid-span deflections.

Slab	Time-dependent deflection (mm)					
	57 days drying		190 days drying		240 days drying	
	Before	After	Before	After	Before	After
LT70-6-0	2.92	2.92	4.24	4.24	4.04	4.04
LT70-4-3	3.54	4.29	6.74	6.74	6.72	6.01
LT70-6-3	2.97	3.63	5.80	5.80	5.84	5.16
LT70-6-6	2.18	3.38	6.37	6.37	6.40	5.31
LT70-6-8	2.94	4.23	6.56	6.96	7.23	5.78

Slab	Time-dependent deflection (mm)					
	21 days drying		28 days drying	56 days drying	244 days drying	
	Before	After			Before	After
LT40-6-0	2.83	2.83	3.15	3.87	4.99	4.99
LT40-6-3	3.33	4.14	4.72	5.68	7.30	6.62
LT40-4-3	2.72	4.12	4.70	5.38	6.57	5.53
LT40-6-6	2.95	4.35	4.60	5.90	6.94	5.68
LT40-4-6	3.30	5.10	5.52	6.72	8.26	7.81

Each of the KF40 slabs was placed onto the supports at age 7 days and remained unloaded except for its self-weight, i.e., 3.2 kPa until age 28 days. At age 28 days (after 21 days drying), with the exception of LT40-6-0, each slab was subjected to superimposed sustained loads with the block layouts similar to that used for the KF70 slabs and shown in Fig. 7. Slab LT40-6-0 carried only self-weight for the full test duration of 244 days. Slabs LT40-4-3 and LT40-6-3 were identical, carrying a constant superimposed sustained load of 3.4 kPa from age 28 days to 251 days, i.e., a total sustained load of 6.6 kPa. Slabs LT40-4-6 and LT40-6-6 were also identical and carried a constant superimposed sustained load of 6.4 kPa from age 28 days to 251 days, i.e., a total sustained load of 9.6 kPa.

Temperature (T) and relative humidity (RH) were also measured in the laboratory throughout the period of testing. The average measured relative humidity (RH_{ave}) in the KF70 and KF40 tests were 67% and 72%, respectively. The average measured temperature (T_{ave}) for the KF70 and KF40 test specimens were 17.4 °C and 20.4 °C, respectively.

The creep coefficient and drying shrinkage strain for the concrete were measured on companion specimens cast with the slabs and cured similarly. The

measured creep coefficient at the end of test for concrete cylinders cast with the KF70 slabs and first loaded at age 64 days was $\varphi_{cc} = 1.62$. For the KF40 slabs, the creep coefficient at the end of the test (age 251 days) for the concrete first loaded at age 28 days was $\varphi_{cc} = 1.50$.

The average of the measured shrinkage strain on two standard shrinkage prisms, 75 mm $\times$ 75 mm $\times$ 275 mm, from the day after removing the wet hessian until the end of the test in KF70 slabs was $\varepsilon_{sh} = 512 \mu\epsilon$. Similarly, for the KF40 slabs, the average measured shrinkage strain at the end of tests was $\varepsilon_{sh} = 630 \mu\epsilon$.

Key deflection values for each slab during long-term tests are summarized in Table 3. The measured deflection includes the deflection caused by shrinkage, the creep-induced deflection due to the sustained load (including self-weight), the short-term deflection caused by the superimposed loads (concrete blocks) and the deflection caused by the loss of stiffness resulting from time-dependent cracking (if any). It does not include the initial deflection of the uncracked slab at age seven days due to self-weight (which has been calculated to be about 0.5 mm for both the KF70 and KF40 slabs). The deflection after unloading each slab at the end of the test shows the irrecoverable (permanent) deflection due to shrinkage

and creep of concrete. The complete results of long-term experimental study are reported and discussed elsewhere [5-7].

After unloading at the end of long-term testing, each slab was lifted onto the supports to carry out the short-term testing similar to what is described in Section 3.2 with a shear span of either $L_s = L/4$ or $L_s = L/6$.

4. Experimental Results

4.1 General Observations

The mid-span deflection versus applied load for each slab is shown in Fig. 9. At first loading, the slabs were all relatively stiff until flexural cracks formed in the constant moment region between the applied loads and propagated into the cross-section. Cracking was associated with significant local increases in the tensile stress in the steel decking and the compressive stress in the concrete in the top fibers, as well as significant loss of stiffness. A well-developed pattern of primary cracks at reasonably regular spacing developed prior to slip between the decking and the

concrete. First slip occurred after a significant percentage of the ultimate load was applied and resulted in a sudden drop in the applied load. Eventually all slabs failed due to a loss of bond at the interface of steel sheeting and the concrete. Even in the post-peak region after large deformation, there was little vertical separation between the steel sheeting and concrete. Abrupt increases in mid-span deflection and end slip were observed after the loss of composite action.

After the peak load had been reached, wide flexural cracks below the applied line loads eventually lengthened to the surface of the slabs and triggered failure and a gradual unloading. This was associated with excessive slip between the steel decking and the concrete in the shear span. The load versus end slip curves are shown in Fig. 10. For the deep trapezoidal profile (KF70), first slip was measured at about 50% of the ultimate load. However for the KF40 profile significant slip did not occur until the load level approached (within 10% of) the ultimate load.

The measured peak load capacity P_{max} , the load causing an end slip of 0.1 mm $P_{(0.1 \text{ mm})}$, and the peak

Fig. 9 Mid-span deflection versus applied load.

Fig. 10 End slip versus applied load.

moment $M_{\max}$ at mid-span are summarized in Table 4. The mid-span deflection $\Delta_{\max}$, the end slip $s_{\max}$, the bottom steel strain at mid-span $(\varepsilon_s)_{\max}$ and the top fibre concrete strain at mid-span $(\varepsilon_c)_{\max}$ all corresponding to $M_{\max}$ are also included in Table 4. The mid-span deflection at peak load varied from $(L/45)$ for Slab ST70-4 to $(L/225)$ for Slab LT40-4-3. According to the Eurocode 4 definition of ductility, all slabs failed in a ductile manner.

The tensile strain in steel sheeting at the soffit of the slab at mid-span in all specimens remained less than the yield strain throughout the test (in most cases much less, especially for the KF40 slabs) and the concrete compressive strains on the top surface at mid-span were generally less than 0.001 throughout the tests. This indicates that the ultimate load capacity was not controlled by either compression in concrete or tension in steel decking, but by the bond-slip relationship between the two materials.

4.2 Effect of Creep and Shrinkage

After being subjected to a period of sustained load

and drying shrinkage, the ultimate capacity of slab LT70-4-3 was about 13% lower than the ultimate capacity of slab ST70-4 and the maximum deflection at peak load was 8% lower. However, for the KF70 slabs with a shear span of $L_s = L/6$, the period of shrinkage and creep had an insignificant effect on the ultimate capacity, but the deflection at peak load was significantly reduced for the slabs that were subjected to the heavier sustained loads (slabs LT70-6-6 and LT70-8-6).

The period of sustained loading and shrinkage actually increased the ultimate capacity of the KF40 slabs tested with shear span of $L_s = L/4$, but there was no consistent trend regarding the comparison of maximum deflection with that of the similar slab tested at 28 days. In KF40 slabs tested with shear span of $L_s = L/6$, the slab LT40-6-0 showed a 22% increase of ultimate capacity compared to the slab tested at 28 days (ST40-6), and 21% decrease in the maximum deflection. However, the ultimate capacity and maximum deflection of slabs LT40-6-3 and LT40-6-6 were similar to that of slab ST40-6.

Table 4 Summary of test results.

Slab	$P_{(0.1\text{ mm})}$ (kN)	$P_{\max}$ (kN)	$M_{\max}$ (kN·m)	$\Delta_{\max}$ (mm)	$s_{\max}$ (mm)	$\varepsilon_{s(\max)}$ ($\mu\varepsilon$)	$\varepsilon_{c(\max)}$ ($\mu\varepsilon$)
ST70-4	37.7	84.0	32.6	68.6	7.5	1,440	225
LT70-4-3	33.2	73.2	28.4	63.0	8.2	1,815	285
ST70-6	63.5	116.5	30.1	64.7	9.7	1,720	360
LT70-6-0	54.2	116.3	30.0	54.5	7.7	1,275	240
LT70-6-3	48.2	121.0	31.3	66.0	8.4	2,450	190
LT70-6-6	56.5	120.5	31.1	37.8	6.8	1,570	920
LT70-6-8	59.8	107.8	27.8	47.0	10.1	1,780	150
ST40-4	61.8	74.4	28.8	17.6	0.7	1,160	375
LT40-4-3	82.2	94.6	36.7	13.8	0.4	1,070	230
LT40-4-6	72.0	82.8	32.1	28.7	5.1	1,020	1,040
ST40-6	98.4	122.0	31.5	21.2	0.9	1,150	330
LT40-6-0	103.0	149.2	38.5	16.7	0.1	1,240	1,270
LT40-6-3	97.6	122.2	31.6	20.6	3.8	1,210	540
LT40-6-6	108.0	127.8	33.0	19.1	1.8	1,080	320

Table 5 Shear stress values calculated from test results.

Slab	$M_{p,Rm}$ (kN·m)	M_{test} (kN·m)	η	V_u (kN)	ηN_{cf} (kN)	τ_{u1} (MPa)	τ_{u2} (MPa)
ST70-4	78.8	36.9	0.27	47.6	190.8	0.18	0.16
LT70-4-3	79.3	32.7	0.20	42.2	140.2	0.13	0.14
ST70-6	78.8	34.4	0.23	63.8	162.1	0.22	0.23
LT70-6-0	79.3	34.4	0.22	63.7	159.7	0.22	0.25
LT70-6-3	79.3	35.6	0.24	66.1	173.8	0.23	0.25
LT70-6-6	79.3	35.5	0.24	65.8	172.3	0.23	0.31
LT70-6-8	79.3	32.2	0.19	59.5	134.2	0.18	0.23
ST40-4	75.8	33.5	0.32	43.2	190.5	0.17	0.19
LT40-4-3	76.6	41.3	0.43	53.3	256.1	0.23	0.24
LT40-4-6	76.6	36.8	0.36	47.4	216.2	0.19	0.22
ST40-6	75.8	36.2	0.36	67.0	214.2	0.27	0.29
LT40-6-0	76.6	43.2	0.46	80.6	272.5	0.34	0.36
LT40-6-3	76.6	36.3	0.36	67.1	211.7	0.26	0.28
LT40-6-6	76.6	37.7	0.38	69.9	224.3	0.28	0.31

The period of sustained loading and shrinkage had a variable effect on the ultimate strength and deformation of these slabs. Any increase in ultimate load capacity was more likely due to the strength gain of the concrete during the sustained load period rather than the effects of creep and shrinkage.

5. Bond-Slip Evaluation

Numerical calculations are carried out to determine the value of ultimate shear stress τ_{u1} by using Eq. (3)

and the method discussed previously in Section 2. The results are shown in Table 5. The mid-span bending moment due to the self-weight of each slab is included in the calculation. The maximum value of the shear stress at any point τ_{u2} for each slab is also included in Table 5.

A comparison between the values of τ_{u1} , τ_{u2} shows a very good agreement between the values obtained from the two different methods.

The relatively small values obtained for degree of

shear connection η and ultimate shear stress τ_u are indicative of the observation that the full flexural capacity of each slab could not be achieved due to slip between the steel sheeting and the concrete slab.

The degree of shear connection for KF40 slabs varied between 0.32 and 0.46 and was significantly higher than the range of 0.19 to 0.27 for KF70 slabs.

6. Conclusions

The results of short-term testing up to failure of 14 composite slabs constructed using two types of profiled steel sheeting that are widely used in construction industry in Australia are presented and discussed. Four slabs were tested at age 28 days and the other 10 slabs were subjected to shrinkage and different levels of sustained load for a period of at least 6 months prior to testing to failure.

For all slabs, the maximum flexural capacity was controlled by slip at the concrete-steel interface in the shear span and the plastic flexural capacity of the slabs was never approached. The effects of creep and shrinkage during the period of sustained loading on the ultimate load capacity of slabs and maximum deflection was variable, and generally must be regarded as insignificant.

The measured ultimate longitudinal shear stress for all KF70 slabs was in the range 0.13 MPa to 0.23 MPa and was significantly lower than the range of 0.17 MPa to 0.34 MPa for KF40 slabs.

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References

- [1] J. Stark, Design of composite floors with profiled steel sheet, in: Proceedings of the 4th International Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, 1978, pp. 893-922.
- [2] D.J. Oehlers, M.A. Bradford, Composite Steel and Concrete Structural Members—Fundamental Behaviour, Pergamon Press, Oxford, UK, 1995.
- [3] R.P. Johnson, Composite Structures of Steel and Concrete, 3rd ed., Blackwell Scientific Publications, Oxford, 2004.
- [4] R.I. Gilbert, M.A. Bradford, A. Gholamhoseini, Z.T. Chang, Effects of shrinkage on the long-term stresses and deformations of composite concrete slabs, Engineering Structures Journal 40 (2012) 9-19.
- [5] A. Gholamhoseini, R.I. Gilbert, M.A. Bradford, Z.T. Chang, Long-term deformation of composite concrete slabs, Concrete in Australia Journal 38 (2012) 25-32.
- [6] A. Gholamhoseini, R.I. Gilbert, M.A. Bradford, Z.T. Chang, Long-term deformation of composite concrete slabs under sustained loading, in: Proceedings of the 22nd Australasian Conference on the Mechanics of Structures and Materials, Sydney, Australia, 2012, pp. 67-72.
- [7] A. Gholamhoseini, R.I. Gilbert, M.A. Bradford, Z.T. Chang, Time-dependent deflection of composite concrete slabs, American Concrete Institute Structural Journal 111 (4) (2014) 765-776.
- [8] M.A. Bradford, Generic modelling of composite steel-concrete slabs subjected to shrinkage, creep and thermal strains including partial interaction, Engineering Structures Journal 32 (2010) 1459-1465.
- [9] G. Ranzi, L. Ambrogi, S. Al-Deen, B. Uy, Long-term experiments of post-tensioned composite slabs, in: Proceedings of the 10th International Conference on Advances in Steel Concrete Composite and Hybrid Structures, Singapore, 2012, pp. 477-484.
- [10] ENV 1994-1-2, Eurocode 4, Design of Composite Steel and Concrete Structures, Part 1.1 General Rules, ECS (European Committee for Standardization), Brussels, 1994.
- [11] A. Gholamhoseini, R.I. Gilbert, M.A. Bradford, Z.T. Chang, Evaluation of longitudinal shear stress and bond-slip relationships in composite concrete slabs using partial shear connection, in: Proceedings of the 22nd Australasian Conference on the Mechanics of Structures and Materials, Sydney, Australia, 2012, pp. 269-274.
- [12] M. Attard, W. Poh, Calculating the load-deflection behavior of simply-supported composite slabs with interface slip, Engineering Structures Journal 15 (1993) 359-367.
- [13] M. Porter, C. Ekberg, Design recommendations for steel deck floor slabs, in: Proceedings of the 3rd International Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, 1975, pp. 761-791.
- [14] M. Porter, C. Ekberg, Design recommendations for steel deck floor slabs, American Society of Civil Engineers Journal of the Structural Division 102 (1976) 2121-2136.

- [15] L. An, Load bearing capacity and behavior of composite slabs with profiled steel sheet, Ph.D. Thesis, Chalmers University of Technology, Sweden, 1993.
- [16] R. Abdullah, W.S. Easterling, New evaluation and modeling procedure for horizontal shear bond in composite slabs, Journal of Constructional Steel Research 65 (2009) 891-899.
- [17] Specifying Fielders KingFlor Composite Steel Formwork System, Design Manual, Fielders Australia Pty. Ltd., 2008.