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Design of composite slabs with profiled steel decking: a comparison between experimental and analytical studies

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Abstract

This paper presents the structural behavior of composite concrete slabs with CRIL DECKSPAN™ (Colour Roof India Limited (CRIL), Mumbai, INDIA) type profiled steel decking by experimental and analytical studies. The slab is created by composite interaction between the concrete and steel deck with embossments to improve their shear bond characteristics. However, it fails under longitudinal shear bond due to the complicated phenomenon of shear behavior. Therefore, an experimental full-size tests has been carried out to investigate the shear bond strength under bending test in accordance to Eurocode 4 - Part 1.1. Eighteen specimens are split into six sets of three specimens each in which all sets are tested for different shear span lengths under static and cyclic loadings on simply supported slabs. The longitudinal shear bond strength between the concrete and steel deck is evaluated analytically using *m-k* and partial shear connection (PSC) methods and compared the values. The experimental results is verified and compared with the results of both *m-k* and PSC methods. Comparison of experimental and analytical results of the load-carrying capacity of composite slabs revealed that agreements between these values are sufficiently good. As a result, *m-k* method proved to be more conservative than PSC method.

Keywords: Composite slab, profiled steel deck, longitudinal shear bond stress, shear span length, *m-k* method, partial shear connection method.

Introduction

A composite slab with profiled steel decking has proved over the years to be one of the simpler, faster, lighter, and economical constructions in steel-framed building systems. The system is well accepted by the construction industry due to the many advantages over other types of floor systems (Andrade 2004; Makelainen and Sum 1999). Since the last decade, the construction industry has been looking beyond the conventional methods and exploring for the better to win over today's challenges, and therefore, composite slab construction is one of the viable options. Cold-formed thin-walled profiled steel decking sheets with embossments on top flanges and webs are widely used in many composite slab constructions. Profiled steel deck performs two major functions that act as a permanent formwork during the concrete casting and also as tensile reinforcement after the concrete has hardened. The only additional nominal light mesh reinforcement bars that needs to be provided is to take care of shrinkage and temperature, usually in the form of welded wire fabric (Chen 2003; Veljkovic 1998). A detailed view of a composite slab is shown in Figure 1.

Composite slab reinforced with profiled steel decking sheet means there is a provision in the system for positive mechanical interlock between the interface of the concrete and the steel deck by means of embossments. The profiled decking sheet must provide the resistance to vertical separation and horizontal slippage between the contact surface of the concrete and the decking sheet (Poh and Attard 1993). It also permits transfer of shear stresses from the concrete slab to the steel deck. The horizontal slippage between the concrete and the steel deck will exist due to the longitudinal shear stress when the shear force of the shear connectors reaches its ultimate strength. However, it is complicated to predict exactly the longitudinal shear stress ($\tau_{u,Rd}$) under flexural

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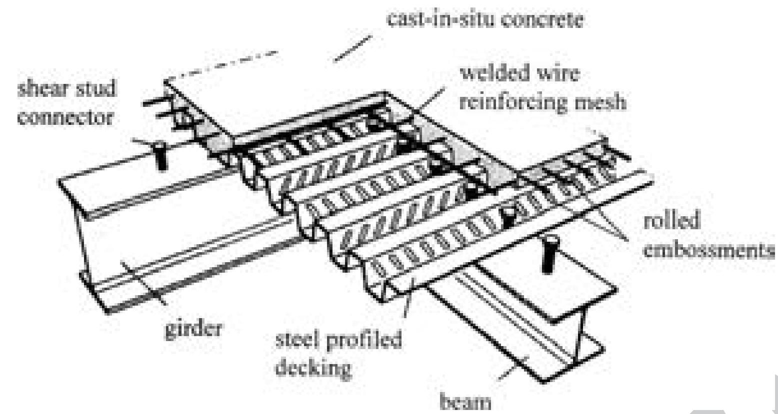

Figure 1 **Composite slab reinforced with profiled steel decking (Crisinel and Marimon 2004; Mohammed and Abdullahi 2011).**

loading; therefore, the longitudinal shear resistance of composite slabs under flexural loading is indirectly evaluated from the empirical method (Vainiunas and Valivonis 2006). Eurocode 4 - Part 1.1 offers two approaches that both necessitate serious full-size laboratory work. One is called *m-k* method (shear bond method) where *m* represents the mechanical interlocking and *k* represents the friction between concrete and steel deck (BS 5950: Part 4 1994; EN 1994-1-1 2004) and the other is partial shear connection (PSC) method (EN 1994-1-1 2004) as an alternative to *m-k* method.

Several full-size experimental tests have been proposed by past researchers to account for complex phenomenon of shear bond behavior between the steel deck-concrete interactions in composite slabs. Porter and Ekberg (1976) have carried out a large number of experimental studies on cold-formed plain trapezoidal steel deck floor slabs without intermediate stiffeners. The work primarily involved one-way full-scale slab specimens and tested up to the failure. Recommending the design procedures is based upon the computation of the shear bond and flexural strength for simply supported conditions. Porter et al. (1976) have further conducted experimental studies on the shear bond failure characteristics of one-way slab specimens with welded transverse wires are used on the top of the deck as shear-transferring devices and reported several observations on the significant parameters influencing the behavior. They have also reported a linear regression relationship between $V_u\, s/bd\sqrt{f'_c}$ and $\rho d/L'\sqrt{f'_c}$ to determine the slope (m) and intercept (k) concepts needed for design. A separate regression is recommended for each deck profile, thickness of deck, steel surface coating, and concrete strength.

Wright et al. (1987) have carried out more than 200 tests on composite slab specimens including embossment, shear stud, and intermediate stiffeners with trapezoidal deck and compared the same with BS 5950: Part 4 design methods by considering two aspects, i.e., composite slab action and composite beam action. Specimens with various concrete strength and subjected to 10,000 cyclic loading have little effect on ultimate strength compared to static loading. A reduction of about 30% in embossment height resulted in a drop of 50% in load-carrying capacity.

Calixto and Lavall (1998) carried out an experimental investigation on the structural behavior of full-scale one-way single-span composite slabs with ribbed decking. Several aspects including different steel deck thicknesses are studied, the total slab height and shear span length. In this study, the slabs fabricated with plain sheeting and shear studs attained in all cases a higher ultimate load when compared to the respective specimens built with ribbed decking only. In all cases, the failure mode was by shear bond even in the slabs fabricated with end anchorage and ribbed sheeting. The experimental results are also compared with the partial interaction design method specified in Eurocode 4 - Part 1.1. The comparison shows good correlation.

Crisinel and Marimon (2004) have proposed a simplified design method for the calculation of load-carrying capacity of composite slabs. This method combines the results from standard material tests and small-scale tests with a simple calculation model to obtain the moment-curvature relationship at the critical cross-section. Results obtained using this new design approach have been verified by comparison with large-scale tests using simple span slabs loaded by two-line load at the quarter spans. It shows good agreement between the calculated moments and moments from the slab bending tests, both at the first slip and ultimate load levels.

Mohan et al. (2005) have presented a simplified approach for the design of composite slabs. This approach utilizes the results of the slip block test with a simple calculation model to obtain the moment of resistance based on the partial interaction method of composite slab governed by horizontal shear resistance. It is observed that the moment of resistance predicted by the slip block and *m-k* tests shows good agreement in quantitative terms.

Marimuthu and Seetharaman (2007) carried out 18 tests to investigate primarily the shear bond behavior of the embossed composite deck slab using trapezoidal profiled steel decking under simulated imposed loads and to evaluate the *m-k* values. The longitudinal shear strength of the composite slab calculated using *m-k* method is verified with the results obtained by partial shear connection method in Eurocode 4 - Part 1.1 and is differed by about 26% in the average.

Mohammed (2010) carried out an experimental work to study the fresh and hardened properties of concrete containing crumb rubber as replacement to fine aggregate. The strength of composite slab lies within the bond between the concrete and the profiled steel sheeting; therefore, the use of lighter in weight and more ductile concrete such as CRC to toping the steel sheeting could produce a new composite slab system. Two sets of slabs, each set comprising three CRC composite slabs and one conventional concrete slab, have been tested with two shear spans. It is found that the shear bond capacity obtained by *m-k* method was slightly higher compared to the value obtained by partial shear connection method of the Eurocode 4 - Part 1.1.

Mohammed and Abdullahi (2011) carried out an experimental investigation by palm oil clinker (POC) aggregate which is used to fully replace normal aggregate to produce structural lightweight concrete in the construction of composite slab with profiled steel sheet. A total of eight full-scale composite slabs, six palm oil clinker concrete (POCC) slabs, and two conventional concrete slabs have been tested in accordance to Eurocode 4 - Part 1.1 with two shear span. The structural behavior and the horizontal shear bond strength of the POCC slabs are nearly similar to the conventional concrete slabs. The design horizontal shear bond strength using *m-k* and PSC methods is 0.248 and 0.215 N/mm^2, respectively.

The review of literature shows that the strength of longitudinal shear bond achieved depends on many factors, among which include the shape of steel deck profile, type and frequency of embossments, thickness of steel decking, arrangement of load, length of shear span, slenderness of the slab, and type of end anchorage. The *m-k* and partial shear connection design methods using data from numerous full-size tests suffer drawbacks such as being expensive and time consuming. However, an accurate determination of strength for a new steel deck profile type is possible only by full-size testing.

This paper deals with the evaluation of longitudinal shear stress using the experimental evaluation of *m-k* values for ultimate strength design of composite slabs reinforced with new trapezoidal profiled steel decking sheet with rectangular dishing type embossments. The longitudinal shear stress resulting from *m-k* method is compared with PSC method, and the comments to evaluate the longitudinal shear stress of composite slabs are discussed. Also, to study the load-deflection curves, load-end slip curves and failure modes subject to imposed loads. The steel decks (CRIL DECKSPAN^{TM}) are manufactured and supplied by Colour Roof India Limited (CRIL), Mumbai, INDIA. A total of 18 full-scale , one-way, single-span, simply supported composite slab specimens are tested using M20 grade concrete subjected to two equal line loads placed symmetrically at six different shear span lengths. The ultimate load-carrying capacity of the composite slabs is calculated using *m-k* method and is verified with the results obtained by the PSC method as per Eurocode 4 - Part 1.1.

Experimental program

A total of 18 full-scale composite slab specimens are built and tested in accordance with the Eurocode 4 - Part 1.1 to determine (1) the structural behavior and (2) the load carrying capacity and provide the necessary information to validate the analytical procedures. According to that, the tests are designed to provide fundamental information on the behavior of composite slabs with realistic geometric and material characteristics. Experimental program include static and cyclic tests on six sets of slab specimens subjected to six varying shear span 300, 375, 450, 525, 600, and 675 mm. For each set of three specimens, one specimen is tested to know about the failure under monotonic loading, and the other two specimens are tested for cyclic loading (BS 5950: Part 4 1994; EN 1994-1-1 2004). Subsequent sets of test are conducted in similar manner with remaining shear spans. A description of the specimen details and testing arrangement is included hereafter. Subsequent sections of the paper discuss the experimental and analytical observations and results.

Profiled steel decking properties

Thin-walled cold-formed profiled steel decks used to build the slab specimens are made of structural quality steel sheets conforming to ASTM A653 (2008) and IS 1079 (1994). A galvanized surface coating with an average thickness of 0.0254 mm is finished on each face of the steel deck. The total specimens are carried out with 0.8-mm thickness (20 gauge) which have a cross sectional area (A_p) of 839 mm^2, a yield strength (f_{yp}) of 250 N/mm^2, and second moment of inertia (I_p) of 0.364×10^6 mm^4. Figure 2 illustrates the geometric shape of the profiled steel deck with embossments opposite on

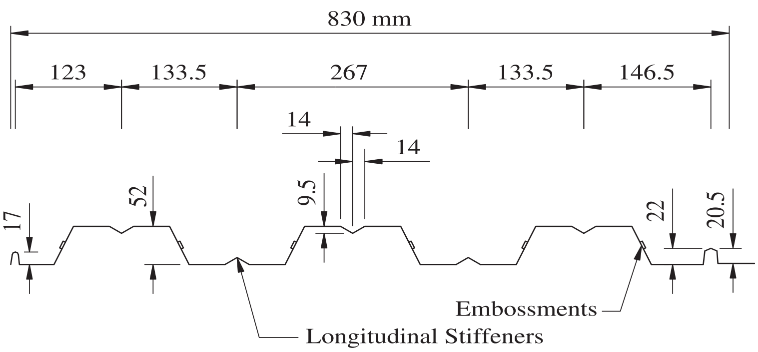

Figure 2 Cross-section of trapezoidal profiled deck and dimensions.

adjacent webs. Shape, size, and frequency of the embossment are shown in Figure 3.

Concrete properties

Concrete used for the specimens is of normal weight, designed for compressive strength of 25.984 N/mm^2. Concrete compressive strength is determined from concrete cubes 150 mm × 150 mm × 150-mm size according to IS 456 (2000) procedures. Three cubes are tested on the same day as the slab test to determine the concrete compressive strength. Course aggregate size used in the concrete is 20-mm down. Concrete proportion used in the mixture is 1:1.42:3.09 (cement/fine aggregate/course aggregate).

Preparation of slab specimens

A total of 18 full-scale (CRIL DECKSPAN^{TM}) composite slab specimens are constructed with 102-mm nominal depth (h_t), 830-mm width (b) and 3,000-mm span ($L+L_0$). The thickness of the concrete above the flange (h_c) is 50 mm while depth of the profiled steel deck (h_p) is 52 mm. All composite slab specimens are cast with full support on the plain surface concrete flooring in the Composite Testing Laboratory. Steel-decking surface is well cleaned before casting of the concrete.

All slabs are constructed utilizing M20 grade of concrete obtained from a hand mixing method. The 70-mm depth of slabs is cast first over which mild steel mesh reinforcement (0.1% of the cross-sectional area of the concrete) of four steel bars, 6 mm in diameter, is placed at a center to center distance of 250 mm in the longitudinal direction and 12 at a spacing of 250 mm in transverse direction to complete cross-sectional dimension of the slab and tied with binding wires (Oehlers and Bradford 1995). Mild steel mesh reinforcement is used as shrinkage and temperature control reinforcements as specified in the ASCE (1985) specification. The remaining 32-mm depth of the slab is cast and finished the top surface by proper compaction of concrete (BS 5950: Part 4 1994) as shown in Figure 4.

The curing period of all 18 slabs is 28 days. Concrete test cylinders and concrete cubes are made at intervals while concrete is being placed according to IS 456 (2000) and cured in the same manner as the slab specimens. Despite all required preventive measures during transport phase, specimen 12CT525 presented premature slippage, probably due to riding procedure, invalidating the test.

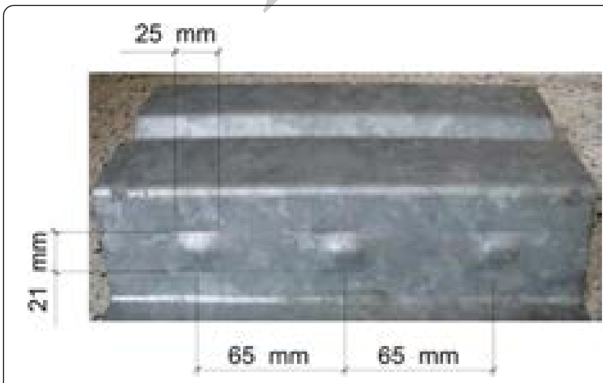

Figure 3 Shape, size, and frequency of embossment.

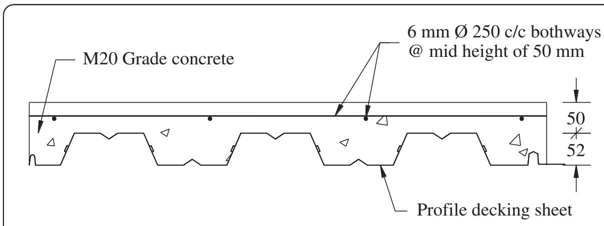

Figure 4 Cross-section of test specimen.

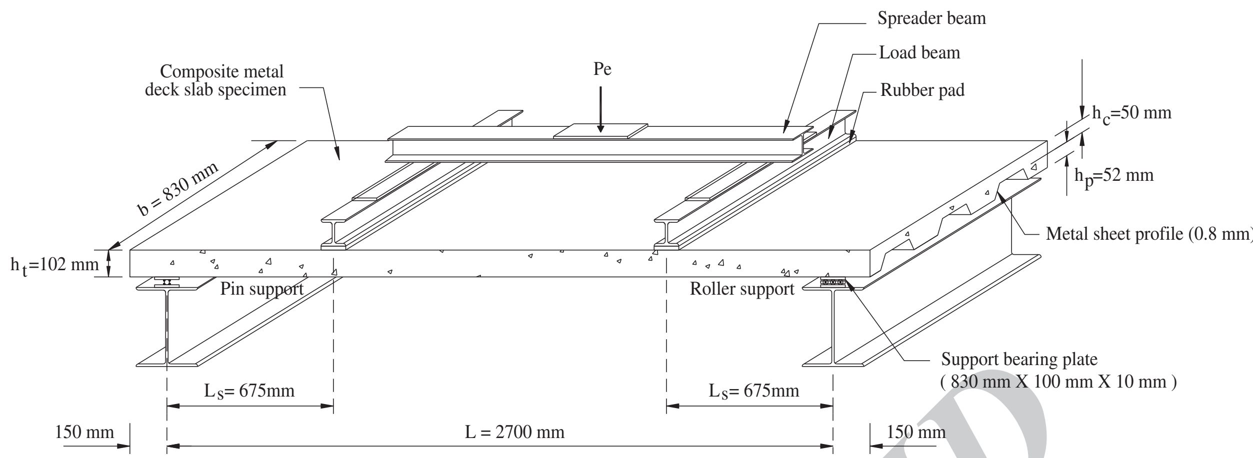

Figure 5 Schematic view of the experimental test setup.

Description of test setup

The schematic view of arrangement for the simply supported composite slab configuration with an effective span (*L*) of 2.7 m subjected to two symmetrically located uniformly distributed line loads is shown in Figure 5. Roller and hinge supports are specially fabricated for study. The schematic view of the roller and hinge supports is shown in Figures 6 and 7, respectively. Figure 8 shows the complete experimental setup.

Loading is applied by a single hydraulic jack system mounted on structural spreader beam section (ISMB 150), beneath the structural load beams (2 ISMC 100, placed back to back), and load is measured with the help of cell at the point of application. Uniform loading is applied by inflating a 15-mm thick by 100-mm wide hard rubber pad, which is confined by the top surface of the test slab. A steel plate with 10-mm thick by 100-mm wide is placed on the top of the pad.

Testing procedure

Details of test specimen

A reference system is adopted to label each specimen as shown in Table 1. The specimens are labeled in the form of 'i-j-k' where i, j, and k are variables indicating serial

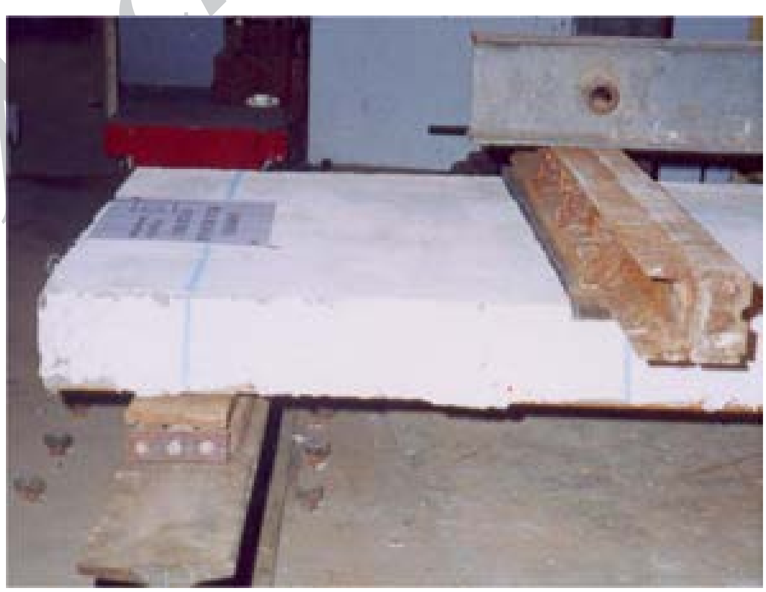

Figure 6 Actual view of roller support.

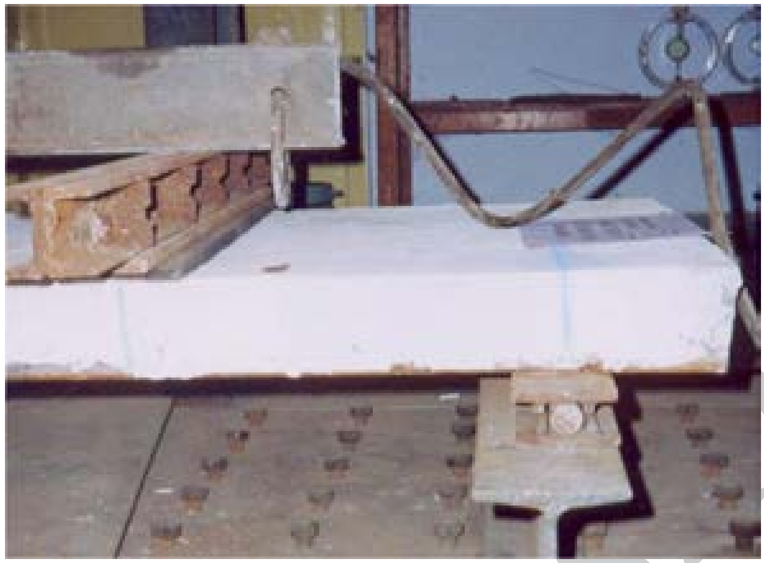

Figure 7 Actual view of pin support.

number of test specimen, static or cyclic test, and shear span (mm), respectively. Hence, '01ST300' refers to the specimen using first test specimen static loading and 300-mm shear span.

Static test

Specimen is placed over roller-hinge supports, and loading points are marked on shear span. Load is applied incrementally by single hydraulic jack system. Rate of loading is adjusted in such a way that failure does not occur in less than 1 h. Rate of loading adopted for static test is 0.1 mm/s. Tests are determined as per the maximum design value or discontinued when the deflections reach $L/50$ where L is the effective span.

Cyclic test

Cyclic loading is required to be implemented in the tests prior to the static loading. Hence, two specimens under

Figure 8 Experimental test setup.

Table 1 Details of shear span loading and its behavior

Test number	Test specimen ID number	Average failure load (kN)	Structural behavior
1	01ST300	54.301	Shear cracks are formed near the loading point. *Slip:* Slip is observed by 2.9 mm, region A to B in Figure 14.
2	02CT300		
3	03CT300		
4	04ST375	50.595	Shear cracks are formed near the loading points and then flexural cracks are formed near the center of the span. *Slip:* Slip is observed by 3.55 mm, region A to B in Figure 15, and the rate of slip is increased after this region.
5	05CT375		
6	06CT375		
7	07ST450	42.650	Shear cracks are formed near the loading points. Flexural cracks are formed near the center of the span and then formed in between the loading points. *Slip:* Slip is observed by 3.6 mm, region A to B in Figure 16, and rate of slip was increase after this region.
8	08CT450		
9	09CT450		
10	10ST525	37.195	Flexural cracks are formed near the center of the span and then shear cracks were formed near the loading points. *Slip:* Slip is observed by 2.0 mm, region A to B in Figure 17.
11	11CT525		
12	12CT525		
13	13ST600	31.523	Flexural cracks are formed near the center of the span. Shear cracks are formed near the loading points and then formed in between the loading points. *Slip:* Slip (3.2 mm) is observed from early stage of loading, region A to B in Figure 18.
14	14CT600		
15	15CT600		
16	16ST675	27.109	Flexural cracks are formed in between the loading points accompanied by a sudden drop in the capacity. *Slip:* Slip is observed by 3.27 mm, region A to B in Figure 19.
17	17CT675		
18	18CT675		

each shear span are subjected to preliminary cyclic loading. This preliminary cycling loading ensures that any kind of chemical bond formed between concrete and steel is removed, and the static load applied later would provide the true indication of the mechanical bond formed by the embossment. Slab is subjected to 3 cycles of loading applied in a time span of 3 h according to BS 5950: Part 4 (1994).

The vertical mid-span deflection is measured using microlevel equipment as shown in Figure 9. For end-slip measurements, two dial gauges are attached to one end of the composite slab in order to measure the relative slip between the concrete and the steel deck as shown in Figure 10. After completing all the static and cyclic tests, the total load at failure is calculated by adding the values of self-weight of the slab and weight of the distribution beams to the applied load at failure for each specimen. Average value of the total load at failure (average of one statically loaded and two cyclically loaded) is calculated for each set of specimen (Table 1).

Results and discussion

Static test

Load deflection behavior

Two stages of load deflection behavior are observed in all specimens. Figure 11a,b,c,d,e and f shows the load-deflection curves for all shear span specimens. For the shear spans, namely, 300, 375 and 450 mm, at first, initial shear cracks formed near the loading point and then flexural cracks formed near the center of span at the bottom of the concrete. As the load is further increased, a number of cracks at the bottom of the concrete progressively spread towards the top of the concrete at the loading point. A slip between steel deck and concrete is observed (region A to B) in Figures 11a,b and c. Secondly, there is a slight load pick-up and subsequent flexural failure of specimen (region B to C).

For the shear spans, namely, 525, 600, and 675 mm, first initial flexural cracks formed at the bottom of the concrete near the center of span and then shear cracks formed near the loading points. Also, flexural cracks are formed in between the loading points. Figure 11d,e,f, point A denotes when visible flexural cracks start forming. Portion A-B shows slip load between steel deck and concrete, and region B to C shows regaining of load to ultimate failure. Table 1 shows failure load capacity and behavior characteristics of slab specimens. Figures 12 and 13 show typical visible crack formation for 300- to 450-mm and 525- to 675-mm shear span specimens, respectively. Total vertical mid-span deflections are measured at point C. All slabs reach a service deflection criterion by span/250 and also earlier to ultimate failure criterion by span/50.

Slip behavior of composite slabs

The end slip is observed from early stage of loading and it is zero at initial loading. At the range of 75% to 80% of

Figure 9 Microlevel deflection measurement equipment.

total loading capacity of composite slabs, the first crack appears. In the first group of shear span, the end slip up to the first crack appearance is gradually decreasing up to certain loading, and in the second group of shear span, the end slip up to the first crack appearance is suddenly dropping down up to certain loading. After that, the rate of end slips increases gradually up to the ultimate failure as shown in Figure 14. As provided in Table 1, the end slip at the ultimate load failure is observed between 2 to 3.6 mm. Curves depict gradual de-bonding of slab. Figures 15 and 16 show the differential movement of the concrete slab and steel deck for

Figure 10 Dial gauges to measure the end slip.

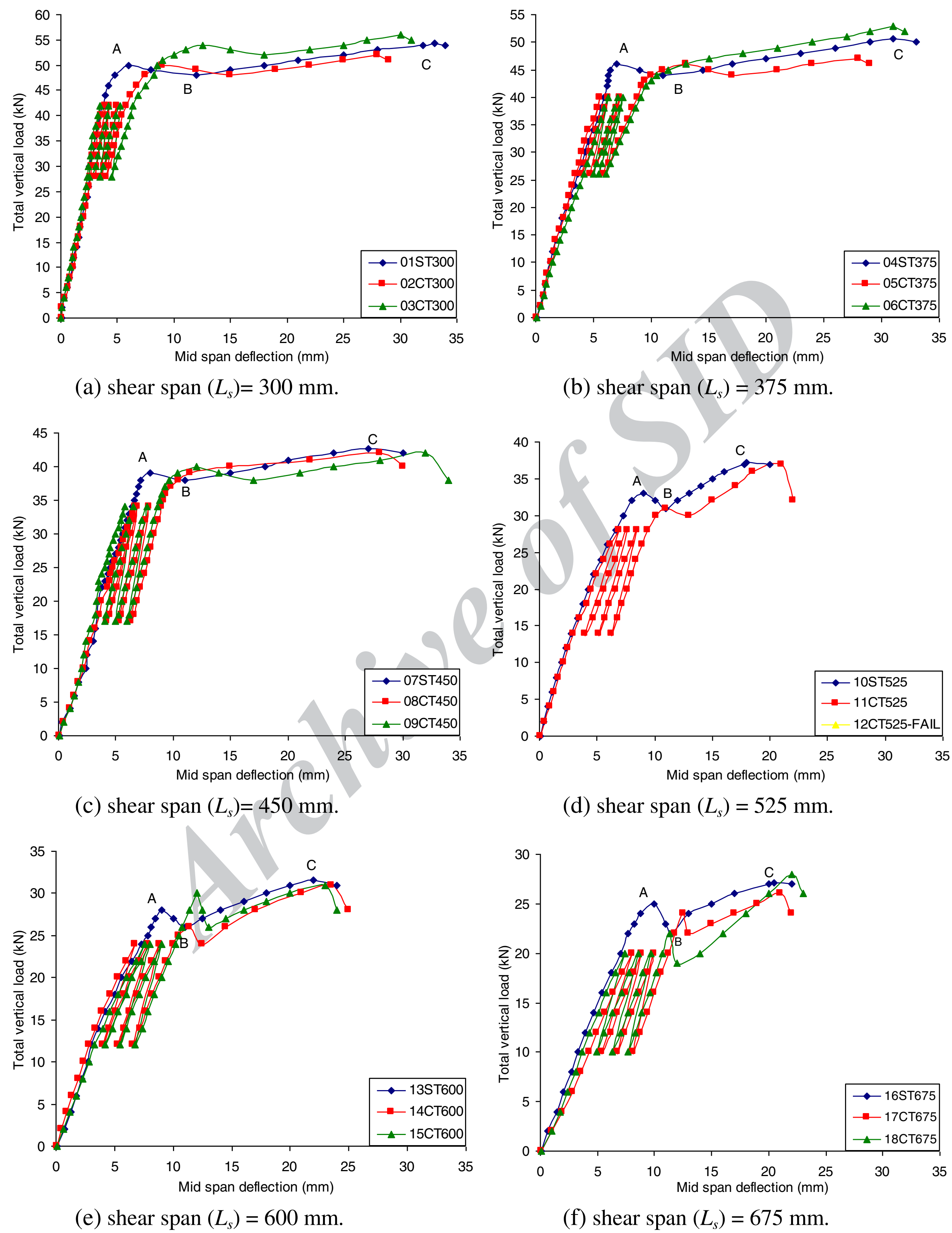

Figure 11 Experimental and analytical load-deflection curves. Shear span (L_s) = 300 (**a**), 375 (**b**), 450 (**c**), 525 (**d**), 600 (**e**), and 675 mm (**f**).

Figure 12 Crack formation for 300- to 450-mm shear span at the ultimate stage.

300- and 600-mm shear span. At initial formation of cracks and at same loading point, rate of end slip is almost similar in all shear spans. Load-carrying capacity of composite slab decreased due to the load position moving towards the mid-span. Slip is observed from both sides of profile towards the center of slab.

Cyclic test

The behavior and capacity are slightly less than obtained in case of the static loading.

Evaluation of longitudinal shear bond strength of composite slabs

Analysis using m-k method according to Eurocode 4

The *m-k* values define shear transferring capacity of the profiled steel deck, where *m* represents the empirical value of mechanical interlocking between concrete and profiled steel decking, and *k* stands for the empirical value for friction between them. The recommended design Equation 1 for shear bond capacity of composite slabs is given by ASCE (1985), EN 1994-1-1 (2004), Porter et al. (1976), Marimuthu and Seetharaman (2007), Mohammed (2010), and Mohammed and Abdullahi (2011) which in the form of an equation for a straight line $y = mx + c$:

$$\frac{V_u}{bd_p} = m\frac{A_p}{bL_s} + k \tag{1}$$

where V_u is the maximum ultimate shear force in Newton; b, the width of the slab in mm; d_p, the distance between the centroidal axis of the steel decking and the extreme fiber of the composite slab in compression; L_s, the length of shear span in millimeter; A_p, the area of cross-section of the profile in square millimeter; and m, k, the design value for the empirical factor in Newton per square millimeter obtained from the slab testing.

Table 2 shows the necessary parameters for plotting *m-k* curve from the test data in accordance with varying shear spans of composite slabs. The capacity reduction factor, Φ, accounts for differences between failure and design strength of a member occurring through variations in material strength, workmanship, tolerances, and supervision and inspection. The capacity reduction factor is selected based both on the mode of failure and associated behavior characteristics occurring prior to failure. Most shear bond failures occur suddenly without ample warning of impending failure. Since, for calculating V_u, a capacity reduction factor $\Phi = 0.8$ is applied to average failure load (ASCE 1985; Marimuthu and Seetharaman 2007). Eurocode 4 omits the concrete strength from Equation 1 because it may give unsatisfactory values for *m* and *k* if the concrete strength varies widely within a series of tests. Many researchers have reported that the concrete strength does not have a significant effect on the capacity (ASCE 1985; Johnson 2004; Luttrell 1987; Mohammed 2010; Mohammed and Abdullahi 2011).

The ASCE (1985) specifies that the reduction of 10% is applied to obtain reduced regression line based on which values of regression *m* and *k* is computed. The

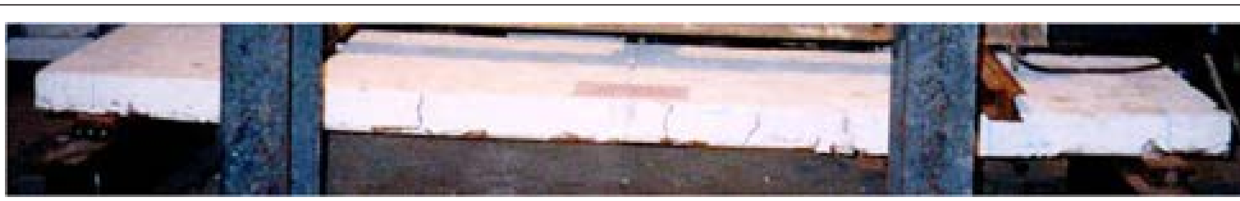

Figure 13 Crack formation for 525- to 675-mm shear span at the ultimate stage.

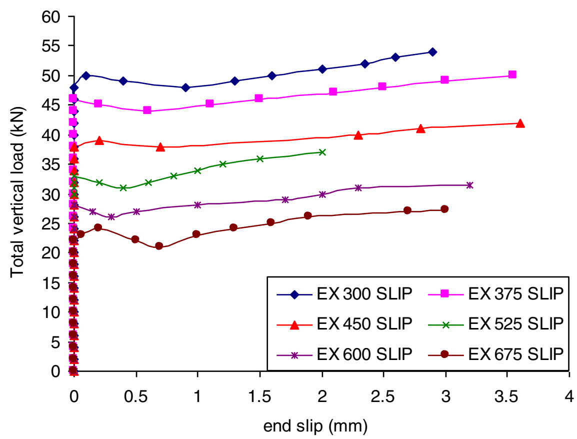

Figure 14 Load-end slip curves for slab specimens.

reduction is to account for test variations and also to assure that line approaches a lower bound for experimental values, therefore, somewhat conservative. The curve is plotted by empirical *m-k* method as shown in Figure 17. From the experimental data, values of m and k for steel deck are 81.95 and 0.046 N/mm^2, respectively. The values are compared with other profiled decks (Chen 2003, Marimuthu and Seetharaman 2007; Mohammed 2010; Wright et al. 1987).

Design shear-bond strength ($\tau_{u,Rd}$) using m-k method according to Eurocode 4

For shear span L_s = 675 mm, the design shear bond strength is as follows:

$$\frac{V_u}{bd_p} = \tau_{u,Rd} = \left(m\frac{A_p}{bL_s} + k\right) \tag{2}$$

$$\tau_{u,Rd} = \left(m\frac{A_p}{bL_s} + k\right) \tag{3}$$

$$\tau_{u,Rd} = \left(\tfrac{81.95\times 839}{830\times 675} + 0.046\right) = 0.169 \text{ N/mm}^2.$$

Determination of design loads using m-k method

For shear span L_s = 675 mm, the maximum design shear is as follows:

$$V_{1,Rd} = \frac{bd_p}{\gamma_{Vs}}\left[\frac{mA_p}{bL_s} + k\right] \tag{4}$$

where γ_{vs} is the partial safety factor for shear connection (1.25)

$$V_{1,Rd} = \tfrac{830\times 76.77}{1.25}\left[\tfrac{81.95\times 839}{830\times 675} + 0.046\right] = 8.60 \text{ kN}$$

Total applied load (w) = 8.60 × 2 = 17.20 kN. The design load (w_{design}) = 17.20/2.7 × 1 = 6.37 kN/m.

Design shear bond strength ($\tau_{u,Rd}$) using PSC method according to Eurocode 4

The PSC method to calculate the longitudinal shear resistance ($\tau_{u,Rd}$) of the composite slab has been detailed in Annex E of the Eurocode 4. According to this method, the degree of shear connection (η_{test}) = 0.310, 0.415, 0.420, 0.430, 0.415, and 0.390 for 300-, 375-, 450-, 524-, 600-, and 675-mm shear span, respectively. For example, the degree of shear connection (η_{test}) = 0.390 for 675 mm shear span is shown in Figure 18.

The shear bond strength ($\tau_{u,Rd}$) for L_s = 675 mm:

$$\tau_{u,Rd} = \left[\frac{\eta_{test}\times N_{cf}}{b(L_s + L_0)}\right] \times \frac{0.9}{\gamma_{vs}} \tag{5}$$

$$\tau_{u,Rd} = \left[\tfrac{0.39\times 209750}{830(675+100)}\right] \times \tfrac{0.9}{1.25} = 0.091 \text{ N/mm}^2,$$

where L_0 is length of the overhang, and N_{cf} is the compressive normal force in the concrete flange with full shear connection.

Determination of design loads using PSC method

Total load for L_s = 675 mm:

$$w = \frac{M_{Rd}}{(L_s/2)} = \frac{8.30}{0.3375} = 24.59\text{kN}$$

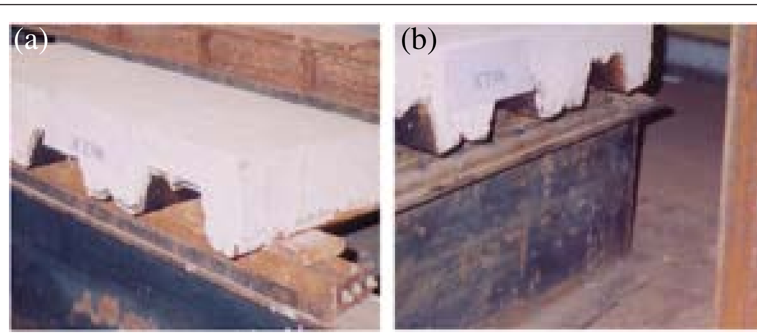

Figure 15 Photograph of end slips for L_s = 300 mm. From the (**a**) left and (**b**) right sides of the specimen.

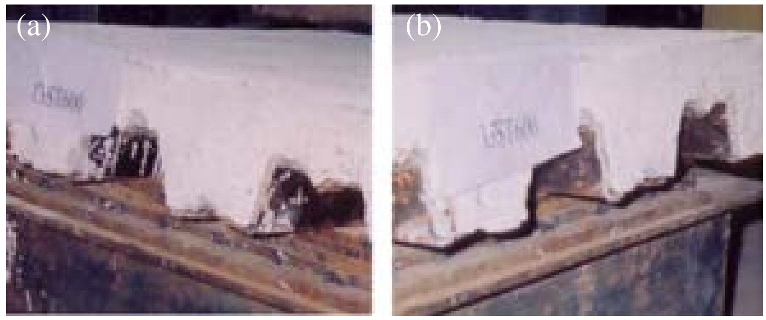

Figure 16 Photograph of end slips for L_s = 600 mm. From the (a) left and (b) right sides of the specimen.

Design load (w_{design}) = 24.59/2.7 = 9.10 kN/m.

Longitudinal shear bond resistance and design load of composite slabs are evaluated by *m-k* and PSC methods and presented in Table 2. The longitudinal shear bond resistances evaluated by *m-k* method are 0.322, 0.266, 0.230, 0.204, 0.184, and 0.169 N/mm^2 and by PSC method are 0.147, 0.158, 0.138, 0.125, 0.107, and 0.091 N/mm^2 for the shear span 300, 375, 450, 525, 600, and 675 mm, respectively. It was found that the longitudinal shear strength values obtained by *m-k* method are slightly higher compared to the values obtained by the PSC method. However, the design load values are slightly lesser.

Figure 19 shows the design longitudinal shear stress using m-k and PSC methods with the shear span length and is presented in Table 2. As the shear span length increased, the longitudinal shear stress of slab decreased. The design longitudinal shear stress values of slabs resulting from line loads obtained by *m-k* method is slightly higher compared with PSC method. The values are compared with other type of profiled decks (Mohammed 2010; Mohammed and Abdullahi 2011). It can be concluded that the *m-k* method has better longitudinal shear strength than the PSC method.

Figure 20 shows the variation of failure/design load using experimental and analytical (*m-k* and PSC) methods with the shear span. As the shear span length increased, the failure/design load of slab decreased. A comparison of experimental and PSC method results of the load-carrying capacity of the composite slabs revealed that agreements between these values are sufficiently good. The results are within 12.5% difference in the average. However, the *m-k* method results are lesser than the experimental method by 43%. This difference occurred since the design load values for *m-k* method are based on regression values reduced by 10% and the use of γ_{vs} of 1.25. Hence, there is significant difference between actual failure load and design load.

Table 2 shows the comparison of experimental failure load with design load capacity which is expressed by two ratios, 1.72 for *m-k* method and 1.11 for PSC method. These ratios represent the safety factors for the design model. Safety factors for both procedures are satisfactory with *m-k* values slightly more safety than PSC values.

Conclusions

In this study, experimental and analytical studies for the design strength determination of composite slab with new profiled steel decking have been presented. The study is based on ASCE standard, Eurocode 4 - Part 1.1 and BS 5950: Part 4 (1994). Results from 18 experimental full-size slab tests, which are used to validate the analytical results using *m-k* and PSC methods have been presented. The two longitudinal shear stresses are evaluated and compared with each other. Based on the study outlined in this paper, the following conclusions are made:

1. A comparison of experimental and partial shear connection method results of the load-carrying capacity of the composite slabs revealed that agreement between these values are sufficiently good. The results are within 12.5% difference in the average (Table 2).
2. For PSC method, analysis is based on actual measured strengths, and hence, it indicates a very less difference between actual failure load and design load.
3. However, the *m-k* method results are weaker than the experimental method by 43%. This difference occurred since the design load values for *m-k* method are based on regression values reduced by 10% and the use of γ_{vs} of 1.25. Hence, there is significant difference between actual failure load and design load. As a result *m-k* method proved to be more conservative than PSC method.
4. Therefore, from the design perspective of the composite slabs, PSC method will give optimum design as compared to *m-k* method.

Table 2 Longitudinal shear strength and design loads using *m-k* and PSC methods

Test number	Average failure load, P (kN)	Failure load from full-size test, w_{failure} (kN/m)	$P \times 0.8$ (kN)	Vertical shear force V_u (kN)	V_u/bd_p (N/mm^2)	A_p/bL_s	Longitudinal shear strength, $\tau_{u,Rd}$(N/mm^2)		Design load based on shear bond capacity (kN/m)		Model factor	
							m-k method	PSC method	*m-k* method w_{design}	PSC method w_{design}	*m-k* method $\frac{w_{\text{failure}}}{w_{\text{design}}}$	PSC method $\frac{w_{\text{failure}}}{w_{\text{design}}}$
1 to 3	54.301	20.111	43.44	21.72	0.3408	0.0034	0.322	0.147	12.16	20.49	1.65	0.98
4 to 6	50.595	18.738	40.47	20.23	0.3176	0.0027	0.266	0.158	10.07	16.39	1.86	1.14
7 to 9	42.650	15.796	34.12	17.06	0.2677	0.0023	0.230	0.138	8.68	13.66	1.81	1.15
10 to 12	37.195	13.775	29.75	14.87	0.2334	0.0019	0.204	0.125	7.69	11.71	1.79	1.17
13 to 15	31.523	11.675	25.21	12.60	0.1978	0.0017	0.184	0.107	6.95	10.24	1.67	1.14
16 to 18	27.109	10.040	21.68	10.84	0.1701	0.0015	0.169	0.091	6.37	9.10	1.57	1.10
Average value							**0.229**	**0.128**			**1.72**	**1.11**

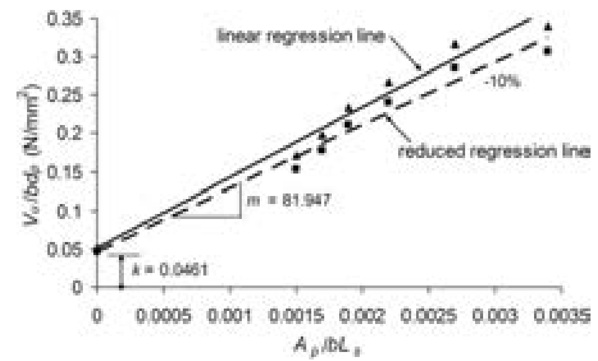

Figure 17 ***m-k*** **Curve from experimental test results.**

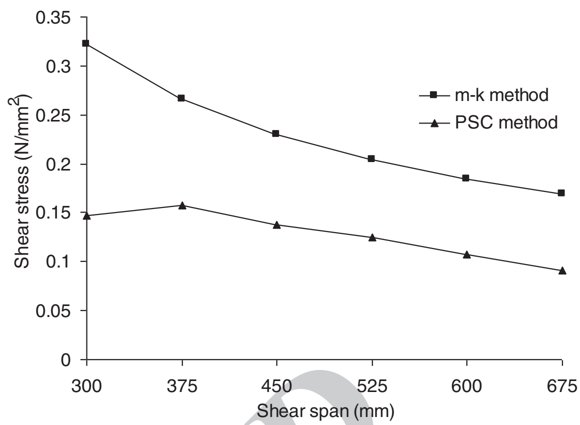

Figure 19 Longitudinal shear stress to shear span under flexural loading.

5. Application of preliminary cyclic loading is carried out as per provisions in EC4. However, there is negligible effect of the cyclic loading on the load-carrying capacity of the composite slabs as compared to static loading (Figures 11a,b,c,d,e and f).
6. The ultimate failure load of the composite slab decreases from shorter to longer shear span and moves towards the midspan (Table 1).
7. For shorter shear spans, strength of slab is governed by only shear bond failure. For shorter to longer shear span, the behavior of slab is governed by shear bond to flexural failure, respectively.
8. Failure modes of all experimental specimens are determined in accordance with the EC4 definition and exhibited a ductile failure.
9. The partial composite action between the concrete and the steel started after the loss of the chemical bonding and could be identified by the formation of the first crack and the beginning of end slip. In all the specimens, the end slip is observed from an early stage of loading, i.e., 75% to 80% of failure load (Figures 11a,b,c,d,e and f).
10. The *m* and *k* values are 81.95 and 0.046 N/mm^2, respectively (Figure 17).
11. As the shear span length increased, the longitudinal shear stress of slab decreased. The design longitudinal shear stress values of slabs resulting from line loads obtained by *m-k* method is slightly higher as compared to PSC method. It can be

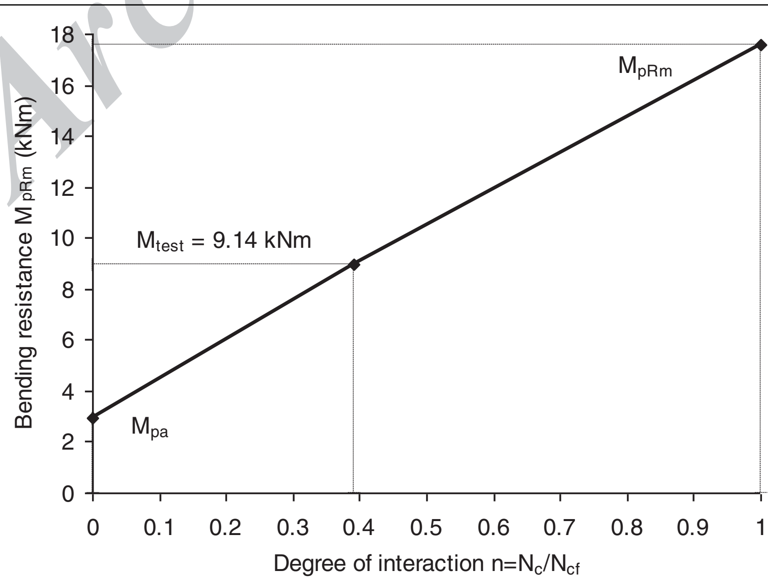

Figure 18 Determination of the degree of shear connection (η_{test}) for L_s = 675 mm.

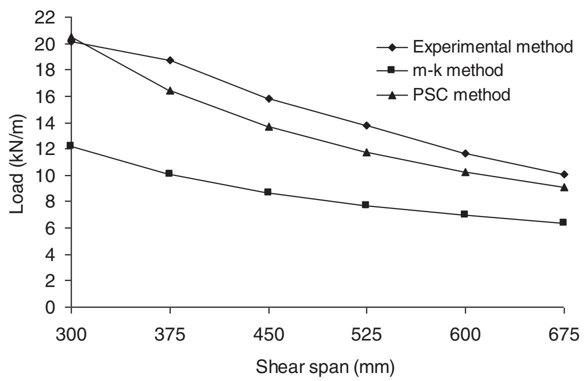

Figure 20 Failure/design load to shear span under flexural loading.

concluded that the *m-k* method has better longitudinal shear strength than the PSC method (Table 2).

Competing interests
The authors declare that they have no competing interests.

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