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Composite slab strength determination approach through reliability analysis

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ABSTRACT

The economic use and ease of construction of profiled deck composite slabs faces the challenge of complex and costly strength determination procedures. This is through the longitudinal shear strength determination that shows the level of composite action between the decking sheet and concrete, and a number of methods are available for its determination; the partial shear method is one such method. The Eurocode design provision requires experimental procedures in establishing the shear strength parameter. However, the cost and time constraint associated with the strength verification is a critical issue of major concern that is currently receiving attention. This study proposes to address these challenges by implementing a rational based approach in developing a numerical function for profiled composite slab strength devoid of experimental procedure. The developed methodology is from reliability-based analyses of longitudinal shear load carrying capacity of profiled deck composite slab from partial shear connection method to Eurocode provision. The proposed methodology results indicate good agreement with the performance of full-scale experimental tests.

1. Introduction

The use of profiled deck composite slab in the construction industry has many advantages including the simplicity in construction compared to other flooring system. The profiled sheeting serve's as shuttering by shouldering wet concrete during construction stage, for example. This composite construction method gained popularity for eliminating time-consuming erection and subsequent removal of temporary forms [1–3]. The composite action between the profiled sheeting deck and the hardened concrete will come into play with effective development of longitudinal shear at the steel-concrete interface. Several studies [4–7] shows the behaviour of profiled deck composite slab is affected by the bond failure in the longitudinal direction. Intuitively, longitudinal shear capacity determines the ultimate strength of profiled deck composite slab. [8].

The use of bonding adhesion or mechanical interlock greatly enhances the shear resistance between steel sheeting deck and concrete [9,10]. The metal deck embossing provides equivalent shear resistance characteristics for effective composite action between sheeting deck and hardened concrete similar to those mentioned previously [9,11]. A number of factors are known to affect the longitudinal shear capacity; for example the type and level of embossment, the steel strain, shear span length, etc..[10], and these hinders the deterministic based strength capacity model development for profiled composite slab (PCS). Besides those factors, the shear strength parameters are determined only after the capital-intensive laboratory procedures, and this could be through the slope-intercept, the partial shear connection (*PSC*) methods amongst others. These drawbacks constitute a serious challenge in developing a numerical strength determination model considering the associated random variabilities [12]. Despite several research attempts [13–15], the complex interface between profiled sheeting deck and the concrete hinders the much needed breakthrough in the development for simplified procedure for strength determination of profiled deck composite slab using PSC method. The objective of this probabilistic study is to define the safety bounds, and develop a simplified numerical model for determining the shear strength parameters for PCS devoid of expensive laboratory works.

2. Reviews on alternate means to laboratory test of composite slab strength determination

Several numerical approaches were developed in order to replace the uneconomical and complex strength verification of composite slab. The strength behaviour of PCS depends on the horizontal shear bond, and it is influenced by the steel deck shape, embossing frequency, load arrangement, shear span length, mechanical friction and type of end

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anchorage [16]. Literature finding shows the difficulties in providing strength determination function applicable to all composite slabs because of those strength-influencing factors. Hence, the need to depend on full-scale experimental test is unavoidable. Abdullah and Samuel Easterling [16], proposed new method for modelling composite slab horizontal shear capacity that takes in to account the slab slenderness as a major shear influencing parameter. The author's uses force equilibrium method in determining the shear bond-end slip behaviour of the composite slab in bending. The result shows that the shear bond varies with the slenderness in bending. Furthermore, the authors use finite element analysis with the aim of replacing uneconomical and time-consuming full-scale test on composite slab, but those factors affecting the shear bond capacity hinders in getting effective results due to lack of quantitative information on them.

Furthermore, in a related numerical finite element modelling study, the simulation results for long slab resembled true slab performance. However, comparative behavioural analysis using short span shows significant variations between modelled and the real slab behaviour [14,17]. Generally, in the critics of FE analysis application for shear bond capacity determination for PCS (geometry dependent) requires full scale experimental test data before utilizing for the modelling, as such FE modelling is still considered uneconomical [16]. To augment this drawback there is a need to use a different numerical approach in finding solution to the problem, and reliability method is one good option other than finite element approach. Therefore, in this paper, this study focuses on application of reliability method on the load carrying capacity of PCS, and this will steer the direction for the development of numerical strength determination function for PCS.

Literature related to reliability studies on the performance of composite slab is scant [2], very few areas is indeed covered. Much recently, Degtyarev [2] presented reliability based analysis of composite slab at construction stage to US design provision. The author investigated the failure analysis using allowable stress design and load resistance factor design using First Order Reliability Method (FORM). The results showed high level of conservatism in the US design provision for the composite steel deck design, and this led to proposals for modifications of the construction load requirement for that code. This paper consideration for the PCS performance function is on the longitudinal shear capacity design in accordance with *EC-4* provision employing the use of FORM in determining the safety performance. Similarly, from number of statistical judgement (*p-value*) while analysing this study results, a sequential scheme will led to the development of numerical strength load function. This sequence includes the performance function value, the metal deck strength and dimensions characteristics, for example. Afterwards, this study designed experimental test for the validation of the numerical strength determination function.

3. Longitudinal shear capacity of profiled deck composite slab

Design strength verification for composite slab found in code of practice is complicated and largely uneconomical because of the mandatory laboratory procedures that are required for the determination of its strength parameter [1,3]. The EC4 [18] provides a general guide for the bending resistance calculation for composite slab, the *m-k* or the partial interaction method are widely use. The study explores the use of PSC method in the determinations of longitudinal shear resistance parameter for the PCS.

3.1. Partial connection method

Partial connection method can be used to obtained longitudinal shear strength of PCS, where complete re-distribution of longitudinal shear is assumed between the sheeting deck and the concrete interface [19]. The degree of shear connection, ξ (N_c/N_{cf})defines the level of re-distribution; $\xi = 0$ signifying no composite action and $\xi = 1$ for full shear connection while slip and strain are assumed to be zero, ξ value between 0 and 1, signifies partial shear connection between the sheeting deck and the concrete.

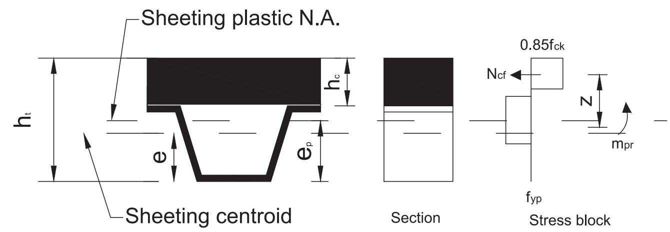

Fig. 1. Typical stress-strain diagram under PSC method.

Johnson [20], developed the formulae for longitudinal shear, τ_u for a given value of bending resistance as shown using Eq. (1).

$$\tau_u = \frac{\xi_{test} N_{cf}}{b\,(l_s + l_o)} \tag{1}$$

Where l_o and l_s are the overhang and shear span lengths for a given width, b of profiled sheeting deck having a yield force, N_{cf} computed from Eq. (2). The parameter ξ_{test} is the degree of shear connection. The design shear strength $\tau_{u,Rd}$ is from the experimental test results by dividing characteristic strength, $\tau_{u,Rk}$with a partial safety factor of 1.25. The minimum value reduced by 10% gives $\tau_{u,Rk}$ [21].

$$N_{cf} = 0.85 A_p f_{yp} \tag{2}$$

Fig. 1 shows the stress-strain diagram under PSC method where the compressive force, N_c is less than or equals N_{cf}, hence, the bending resistance determination is highly dependent on the neutral axis, $N.\ A.$ position within the system determined from the stress block depth,x given by the expression in Eq. (3).

$$x = \frac{N_{cf}}{0.85 f_{ck}\, b} \le h_c \tag{3}$$

In Eq. (3), the concrete thickness, $h_c \ge 40$ mm, aimed at controlling the minimum fire protection requirement. Hence, the design bending resistance, $m_{p,Rd}$ is

$$m_{p,Rd} = N_{cf}\, z + m_{pr} \tag{4}$$

The plastic moment of resistance, m_{pr} and the lever arm, z in Eq. (4) are as follows

$$m_{pr} = 1.25 m_{pa} (1 - \xi) \le m_{pa}$$
$$z = h_t - e_p - 0.5x + (e_p - e)\xi \tag{5}$$

Where m_{pa} plastic moment of resistance of the profile deck, $e$and $e_p$are the centroid distance and the plastic neutral axis above the base, respectively (see Fig. 2). The EC4 specification for PCS thickness h_t should not be less than 80 mm, and this study experimental specimen thickness value is 120 mm.

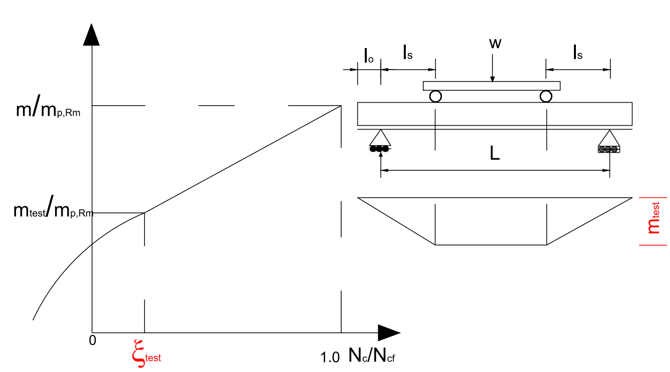

Fig. 2. Typical PSC interaction curve.

Table 1
Slab parameters with experimental failure test load.

source	Label	l_s (mm)	FTL (kN)	τ_u (N/mm^2)
Marimuthu, et al. [3]	1	320	55.625	0.318
	2	350	52.191	0.303
	3	380	47.340	0.284
	4	850	22.612	0.156
	5	950	26.920	0.167
	6	1150	16.391	0.118
Cifuentes and Medina [9]	AWS-1	575	45.79	0.138
	AWS-2	575	46.44	0.140
	AWS-3	575	45.39	0.136
	AWL-1	1000	47.69	0.157
	AWL-2	1000	46.34	0.153
	AWL-3	1000	49.44	0.163
Hedaoo, et al. [11]	1–3	300	54.301	0.322
	4–6	375	50.595	0.266
	7–9	450	42.650	0.230
	10–12	525	37.195	0.204
	13–15	600	31.523	0.184
	16–18	675	21.109	0.169
Cifuentes and Medina [9]	BTS-1	750	58.70	0.284
	BTS-2	750	60.58	0.293
	BTS-3	750	59.77	0.290
	BTL-1	1000	67.33	0.216
	BTL-2	1000	65.56	0.210
	BTL-3	1000	64.38	0.206

In Fig. 2, the shear span length, l_s is normally taken as $l/4$ where l is the clear span between supports [20]. For ductile failure condition, the support reaction, V_t is computed using Eq. (6).

$$V_t = w/2 \tag{6}$$

For brittle failure case, the failure test load, w is reduced by 20% in magnitude [19]. Generally, irrespective of the three failure modes (flexure, longitudinal shear or in vertical shear failures) associated with PCS [20], the ratio l_s/d_p"herein referred to as inverted slenderness in this paper" plays a critical role in achieving this study objective. Therefore, this study requires the use of experimental test results from several studies in developing the performance function. The following section provides the details for the consulted experimental test results found in literature.

4. Background on study experimental data

This study uses full-scale experimental laboratory test data from several studies [3,9,11] that worked with profiled sheeting deck. The test results served as input variables for the failure test load (FTL) in establishing safety performance for the PCS. Table 1 shows the specimens test parameters and the FTL values from several studies. The detailed experimental procedure and other parameters can be found in the literature under [3,9,11].

5. The Reliability function

The inherent uncertainties associated with the design variables necessitated the application of certain measures in curtailing failures; for example the use of safety factors under the deterministic design condition, by amplifying the design load and reducing the strength parameters [22]. Intuitively, higher the strength, R in comparison to the applied load, Q there will be some degree of structural safety than otherwise (unwanted situation). Hence, treating the R and Q as random variables [23], the failure probability, p_f estimation for the unwanted domain is by

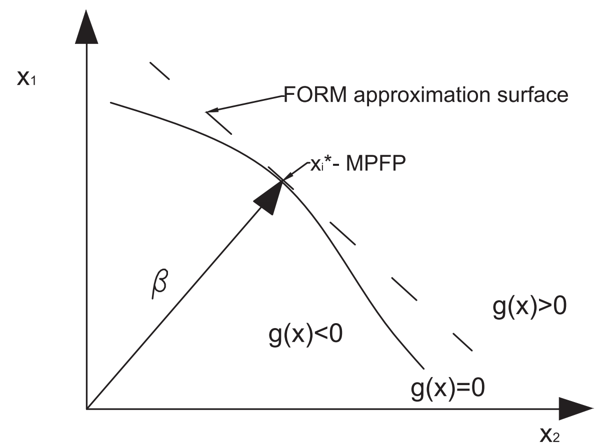

Fig. 3. Typical limit state surface and reliability indices.

$$p_f = prob(R - Q < 0) = p(X < 0)$$
$$p_f = p(G \le 0) = \int_G^{\infty} F_R(x) f_Q(x)\,dx \tag{7}$$

Where G is the limit state function or performance function that defines the bounds; desired boundary $G \ge 0$ from the un-desired boundary condition, $G < 0$. The parameters F_R and f_Q are the cumulative probability density function and the probability density function of the resistance and load effect respectively.

The p_f value is a real non-negative number between 0 and 1, but it is usually expressed using reliability index or safety index, β [2,22], and this can be determined using the First Order Reliability Method (FORM). A good example for safety index value is shown using Fig. 3; the safety index is the shortest possible distance from the origin to the performance function curve termed as the most probable failure point, *MPFP* or the design point, x^*. Hence, Eq. (7) in simplified form is given by the expression in Eq. (8).

$$p_f = \Phi(-\beta) \tag{8}$$

where Φ is the inverse of the standardised normal distribution function.

5.1. Reliability analysis

This paper presents the reliability analysis of profiled deck composite slab where the focal point is on the material load carrying capacity and the design load estimated from the shear resistance of composite slab under *PSC* approach. Fig. 4 shows the explicit procedure involved in this study for the determination of the performance index value. The failure domain is when the design load exceeds the PCS load capacity. In this study, the maximum *FTL* (in Table 1), is taken as the ultimate strength resistance of the material. Hence, the computation of the reliability index for the capacity limit state violation follows using First Order Reliability Method (FORM) for several shear span lengths and reduced (%) strength test load value.

Therefore, the mean resistance, Q_m value for the PCS is according to [23,24]

$$Q_m = Q_n(M_n F_n P_n) \tag{9}$$

where Q_n is the nominal resistance taken as the ratio of FTL over the span length with an assumed bias factor of 1.0. The variables M_n, F_n, P_n are material fabrication factor, factor for geometry and dimension of the component, and professional factor for approximation in structural analysis, respectively. These factors equivalent mean resistance coefficient of variation, V_Q is by the expression from Eq. (10).

$$V_Q = \sqrt{(v_m^2 v_f^2 v_p^2)} \tag{10}$$

The parameters, v_m, v_f and v_p are corresponding coefficient of

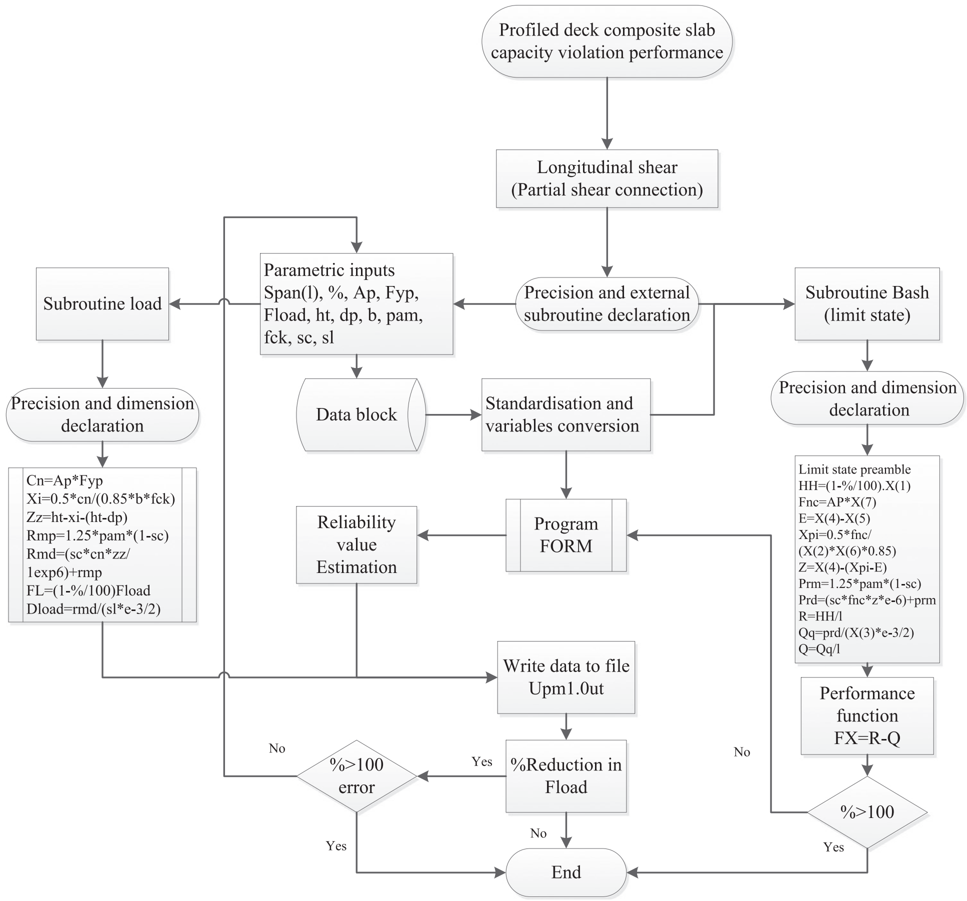

Fig. 4. Safety index determination flow.

variation, *cov* for the factors M_n, F_n, and P_n. The study mean and *cov* values for those factors are 1.10, 0.1; 1.0, 0.05 and 1.11, 0.09 with all having normal distribution function [2]. Furthermore, on the basis of Ellingwood and Galambos [25], the *cov* and distribution type for the b and l_s parameters are 0.17 and log-normal distribution, having a bias factor of 1.0 both. Hence, this study limit state violation function for the PCS is given by the expression shown in Eq. (11).

$$Q_m - \frac{m_{p,Rd}}{0.5 l_s} = R - Q \tag{11}$$

In Eq. (11), the $m_{p,Rd}$ and Q_m parameters are from the use of Eqs. (4) and (9), respectively. Eq. (12) shows the transformation of the performance function in Eq. (11) in basic variables form. The function contains seven discrete variables; $X(1-7)$; FTL, b, l_s, h_t, d_p, f_{ck} and f_{yp} (see Fig. 4).

$$\begin{aligned}
&R = [(1 - \%/100)X(1)]/l\\
&Q = [(sc^*A_p{}^*X(7)^*X(4)) - [(0.5^*A_p{}^*X(7)/\\
&(X(2)^*X(6)^*0.85) - (X(4) - X(5))^*\exp^{-6}] +\\
&(1.25^*pam^*(1 - sc))]/\\
&(X(3)^*\exp^{-3}{}^*0.5)
\end{aligned} \tag{12}$$

The variables *sc* and *pam* in Eq. (12) represents the degree of shear connection and the sagging moment capacity of the profiled sheeting deck. The statistical parameters for the first three variables are previously detailed, and the values for the remaining four variables had log-normal distribution excerpt f_{ck} which has normal distribution characteristics [26]. Similarly, h_t and d_p have same bias factor and *cov* value (1.0, 0.05), while f_{ck} and f_{yp} had (1.4, 0.10) and (1.17, 0.17), respectively.

6. Experimental test set-up

The experimental test scope considered in this study comprises the testing of PCS at the Universiti Putra Malaysia. A total of eight PCS specimens consisting two each for both short and long shear span lengths of 228 mm, 243 mm, and 305 mm, 320 mm, respectively. For simplicity, these specimens are identified using notations SS and LS; for example, SS-228 and LS-305 represents short and long specimen with shear span length of 228 mm and 305 mm, respectively.

6.1. Materials properties and slab specimen casting

The metal deck thickness is about 0.47 mm, and it is 1829 mm long (L), having width (b) value of 820 mm as shown in Fig. 5. The desired concrete strength is normal grade concrete, and the mix is prepared using 20 mm aggregate for 120 mm thick concrete. For hydration control, 5.1 mm mild bars are mesh through at 220 mm both ways, and placed 20 mm above the metal deck. Similarly, the fabrication of the concrete mould is with plywood, and the concreting works is under supportive laboratory conditions.

The required necessary laboratory checks on the concrete mix design prior to concreting are fully adhered to according to the ACI-318 standard, and the mix design found to be workable. Moreover, cubes for the determination of the compressive strength from the batch mixes for testing after 28 days by covering concrete surface with Gunny bags, and shows an average compressive strength of 28.5 MPa. Afterwards, the prepared test specimens was carried to the testing site from the curing yard with the aid of proper support; bearing in mind the consequences of transferring any flexural load to the slab while in transit will be disastrous.

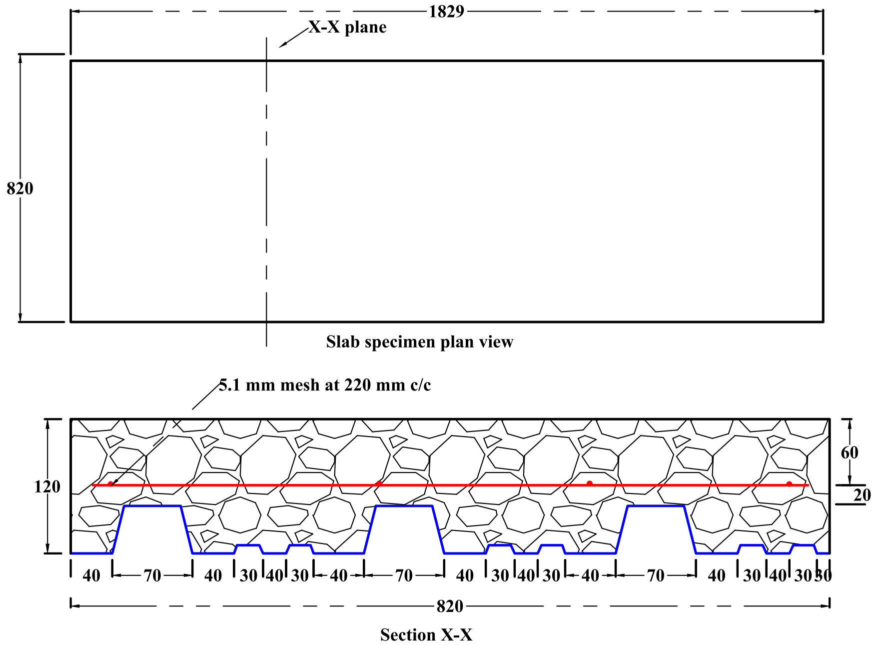

Fig. 5. Test specimen profile.

6.2. *Experimental test set-up*

The hydraulic jack load is applied upon the test specimen with the two spreader roller weighing about 10 kg each, that are placed on top of the slab specimen with the intention of applying the two point load from cross beam that also weigh about 70 kg. The overhang length l_o is 100 mm from both ends. In determining the slab failure mode during the test procedure, linear variable displacement transducers (LVDT) were at the edges of the decking sheet and the concrete as depicted in Fig. 6. Similar LVDT placements are also at the mid-span to measure the slab deflection (Fig. 6). The data logger-TDS-530 records the values for the end-slip, the mid-span deflection including the test load. The testing were halted if the maximum applied load drops by about 20%, or the mid-span deflection value is approaching l/300 [27].

The experimental test results will be compared to the numerical solution estimation for the strength capacity determination of PCS. The closeness between the compared results will validate the suitability of the developed model for strength capacity estimation of PCS (Fig. 7).

7. Reliability analysis results

The load ratio l_r function is the ratio of FTL over estimated design load from the use of the longitudinal shear value, and it is labelled using *A-D* in this study. For example, the ratio of Marimuthu, *et al.* [3] experimental FTL value to the deterministically computed design load is depicted using letter *A* in this paper, and similar letters *B*, *C*, and *D* stands for the respective ratios from Hedaoo, *et al.* [11] and Cifuentes and Medina [9] *AW* and *BT* slabs respectively. Hence, Fig. 8 shows the

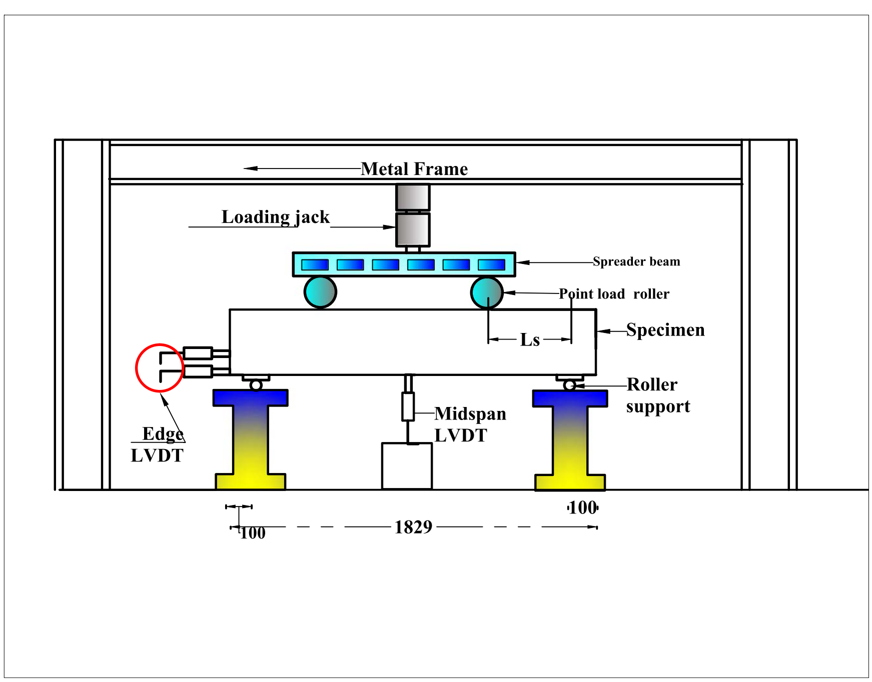

Fig. 6. Experimental test set-up.

Fig. 7. Test in progress.

behaviour of the computed safety indices of the PCS in relation with the l_r function. In that figure, the symbol ∝ stands for shear span length; for example, ∝320 indicate shear span length of 320 mm. Similarly, the indent marks on each plot lines shows the influence of FTL reduction (0–20%). The reason for this action is to evaluate the influence of the present capacity reduction factor of 0.8 that are applied to the failure while computing the shear bond capacity of profiled deck composite slab [3].

The results from Fig. 8 demonstrated linear relationship between l_r and β. The safety indices value is higher with increasing l_r value. Adopting to use the upper point with higher $l_r$value as the upper tail and vice versa for all ∝ values, the safety values indicates high likely hood of undesired situation. This behaviour is because of the higher magnitude of the estimated design load from the longitudinal shear using *PSC* method over the actual test load.

7.1. Section slenderness

In this study, the PCS performance is reported using the inverted slenderness function, d_p/l_s, that also takes into account the differences in cross section and yield strengths of the respective sheeting deck. The resulting property termed as ϖ ($A_p f_{yp} d_p/l_s$) is evaluated against $p_f$as shown in Fig. 9. In that figure the influence of FTL reduction (0–20%) on the performance estimation behaviour is also presented. The result indicates a very distinct plots variation for the four different PCS because of sheeting deck properties difference. The results indicate that decreasing FTL increases the $p_f$value with homogenous ϖfunction estimation. The homogeneity is because of un-influencing effect of f_{yp} on the longitudinal strength of composite slab [20].

Although the data size is limited, that still notwithstanding, this study seeks to establish relationship between p_f and ϖparameter. In other to achieve this objective, statistical fitting is super-imposed on the 20% penalised *FTL* performance function as shown in Fig. 10. The reason for choosing the 20% penalised performance function is based on the consideration for the present capacity reduction factor of 0.8 that are applied to the failure while computing the shear bond capacity of profiled deck composite slab [3].

Fig. 10 shows the linear, quadratic and cubic fittings including their corresponding equations for the p_f function as dependent variable. Statistically, assuming equal variance in the determination of the safety values against the fitted function, all shows no significant variations with the data points; linear ($t = -0.163$, $dof = 34$, $p>0.05$), quadratic ($t = -0.081$, $dof = 34$, $p>0.05$) and cubic ($t = -0.33$, $dof = 34$, $p>0.05$). The comparison of these basic fittings shows that the quadratic fitting alpha value is about 7% higher than the closet alpha value of 0.436 from linear fitting. Appling other fitness of fitting measures, linear fitting ($r^2 = 0.89$, $SE = 3\%$) can suitably mimic the data behaviour than either quadratic ($r^2 = 0.82$, $SE = 3\%$) or cubic ($r^2 = 0.84$, $SE = 3\%$) fittings. Hence, the result indicates that the relation between p_f and ϖparameter can be modelled effectively using the linear function as shown in Fig. 10.

In this study, the design load, ξ is from the shear value estimation, and it is highly dependent on the shear span length as demonstrated in Eq. (11). Taking into consideration that dependency factor together with the sheeting deck function, ϖ(defined previously), this study seeks to understand how this relates with the ξ estimation. Fig. 11 presents such relationship for the combined data points principally from the experiments conducted in accordance with standard EC4 provision. The results from that figure shows a significant linear relation between the ξ value and the ϖ parameter, and it is represented mathematically using the expression in Eq. (13). Statistically, assuming equal variance, there is no significant difference between the data points and the fitted linear function ($t = -0.025$, $dof = 22$, $p>0.05$). Comparatively, similar quadratic fitting (Fig. 11) shows similar performance($t = -0.030$, $dof = 34$, $p>0.05$), but with inferior *p- value* in relation to the linear function.

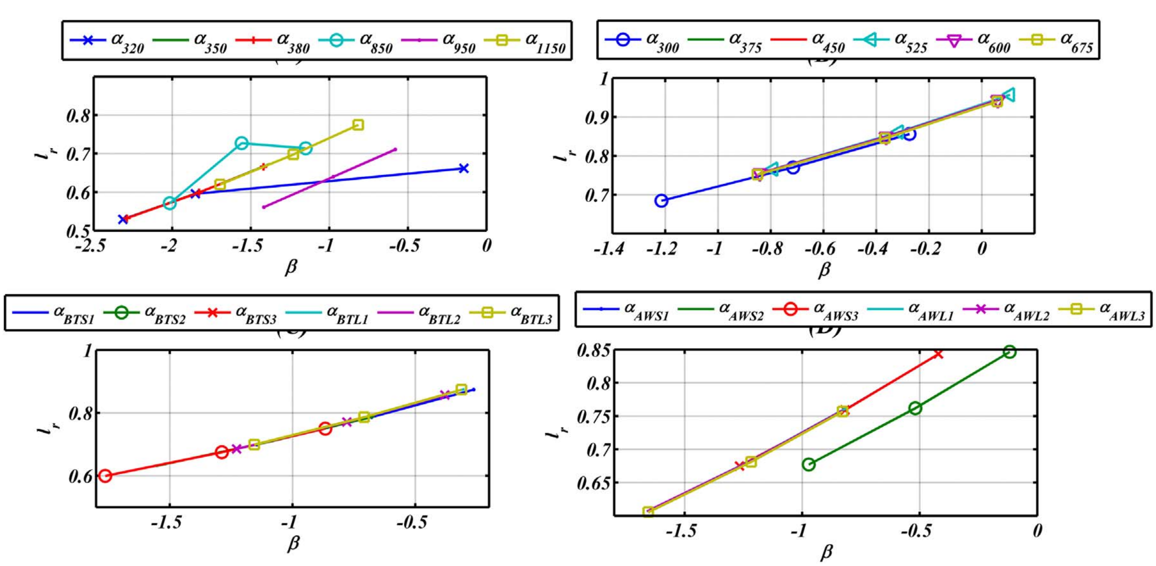

Fig. 8. Performance indices.

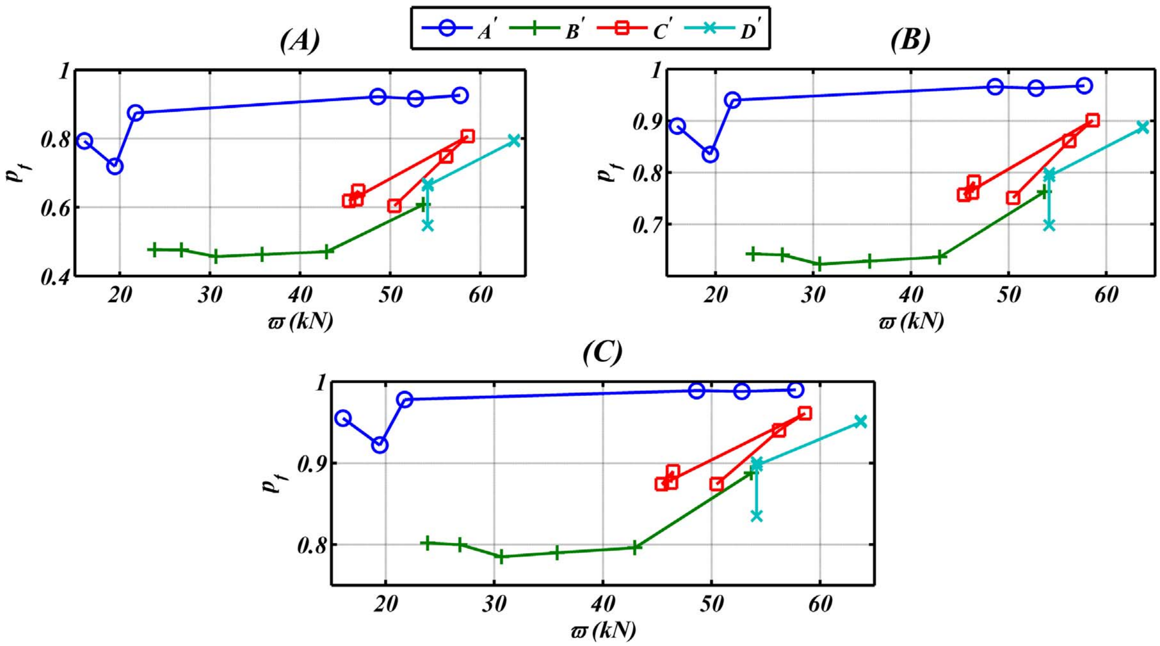

Fig. 9. Safety performance characterised by FTL value: (A) 100% FTL (B) 90% FTL (C) 80% FTL.

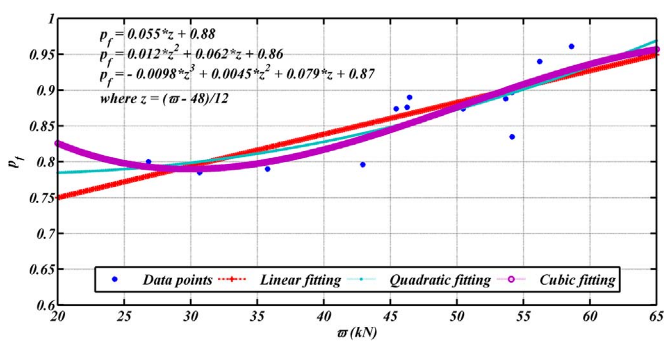

Fig. 10. Statistical curve fitting in establishing relation between p_f and ϖparameter.

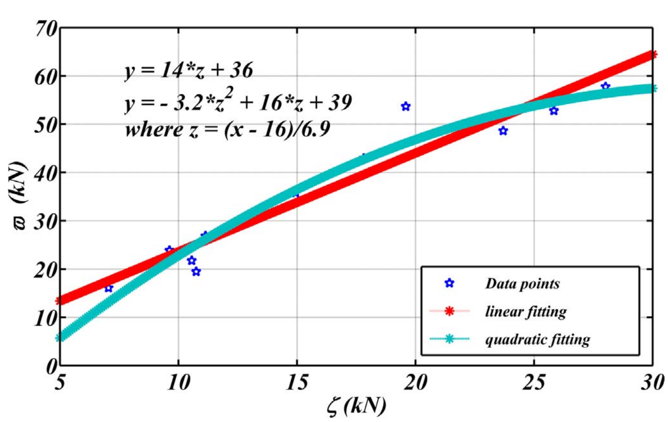

Fig. 11. Statistical curve fitting showing the relationship between ξ and ϖ.

$$2\xi = \varpi - 3.1 \tag{13}$$

Fig. 12 shows the established relationship between the estimated ξvalue and the performance function as previously mentioned. The ξ value in this study is categorised into lower and upper threshold values of 8.34 and 23.81 kN/m, respectively. These values are from 7.05, 28.02 and 9.62, 19.59 kN that represents the minimum and maximum ξ values from the longitudinal shear estimation that considers the FTL values from Marimuthu, *et al.* [3] and Hedaoo, *et al.* [11] respectively. In the same vain, 7.25 and 17.66 kN/m are the corresponding

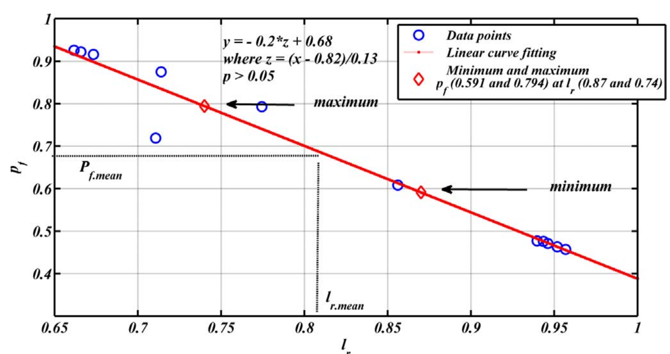

Fig. 12. Established relation between Pf and lr.

minimum and maximum *FTL*/*l* values. Hence, from those values, 0.87 and 0.74 are the l_r bounds from the established behaviour and are used in obtaining the mean p_{fmean} value of 0.69 ($\beta = -0.513$) at l_{rmean} of 0.81 (Fig. 12).

Generally, reducing the *FTL* value makes the system more susceptible to failure, because the design load estimation in this paper from the longitudinal shear value is fairly affected; the higher l_r value, the lower the $p_f$value. Therefore, collating the study results and assuming p_{fmean} is an equivalent target safety value, the predicted FTL value estimation function in Eq. (14) is from the use of the defined relationship in Fig. 12 in conjunction with the expression in Eq. (13).

$$P.\ FTL = 0.41(\varpi - 3.1) \tag{14}$$

The use of lowly p_{fmean} value obtained from the study approach in predicting the PCS performance is indeed arguably, since the value is well below the expectant. The reason for the low value could be attributed to the influence of slab compactness or slenderness that are two important issues that affects the behaviour of PCS which are not properly taken in to account under PSC approach [28].

7.2. Predicted failure test load estimation performance

The performance of the FTL estimation function using the expression given in Eq. (14) that takes in to accounts the factor of random variabilities associated with both strength and load variables tested through comparative analysis with several full-scale laboratories ex-

Table 2
Comparative analysis between Experimental and Predicted FTL values.

Label	A_p (mm^2)	f_{yp} (Mpa)	d_p (mm)	l_s (mm)	l (m)	Exp. FTL (kN)	Source	P. FTL (kN)
ST57-4	1434	536	135.9	850	3.4	92.8	Gholamhoseini, et al. [1]	94.03
ST57-6	1434	536	135.9	567	3.4	154		142.18
ST55-4	1485	534	134.6	850	3.4	67		96.14
ST55-6	1485	534	134.6	567	3.4	102.5		145.34
ST70-4	1320	544	122.3	775	3.1	84		86.52
ST70-6	1320	544	122.3	517	3.1	116.50		130.91
ST40-4	1248	475	136.0	775	3.1	74.40		79.22
ST40-6	1248	475	136.0	517	3.1	122		119.97
Slab 1	980	550	93.0	900	2.7	46.8	Mohammed [29]	41.28
Slab 2	980	550	93.0	900	2.7	38.1		41.28
Slab 3	980	550	93.0	900	2.7	49.1		41.28
Slab 4	980	550	93.0	450	2.7	61.9		85.01
Slab 5	980	550	93.0	450	2.7	63.7		85.01
Slab 6	980	550	93.0	450	2.7	65.6		85.01
SS-228	496.71	340	100.0	228	1.8	45.97	This study experiment	41.28
SS-243	496.71	340	100.0	243	1.8	41.73		52.17
LS-305	496.71	340	100.0	305	1.8	33.50		41.03
LS-320	496.71	340	100.0	320	1.8	27.97		38.94

periments including this study design experiment is shown in Table 2. Comparatively, the estimates from the approximates solution shows similar variance with the experimental FTL values from both Gholamhoseini, *et al.* [1] ($t = -0.74$, $dof = 14$, $p>0.05$), Mohammed [29] ($t = -0.83$, $dof = 10$, $p>0.05$) and this study experiment ($t = -1.67$, $dof = 6$, $p>0.05$).

Further testing the statistical significance of this new approximate method for the FTL determination, Fig. 13 shows the shear values comparisons between experimental, theoretical and new theoretical values. In that figure, the experimental shear values with notations in

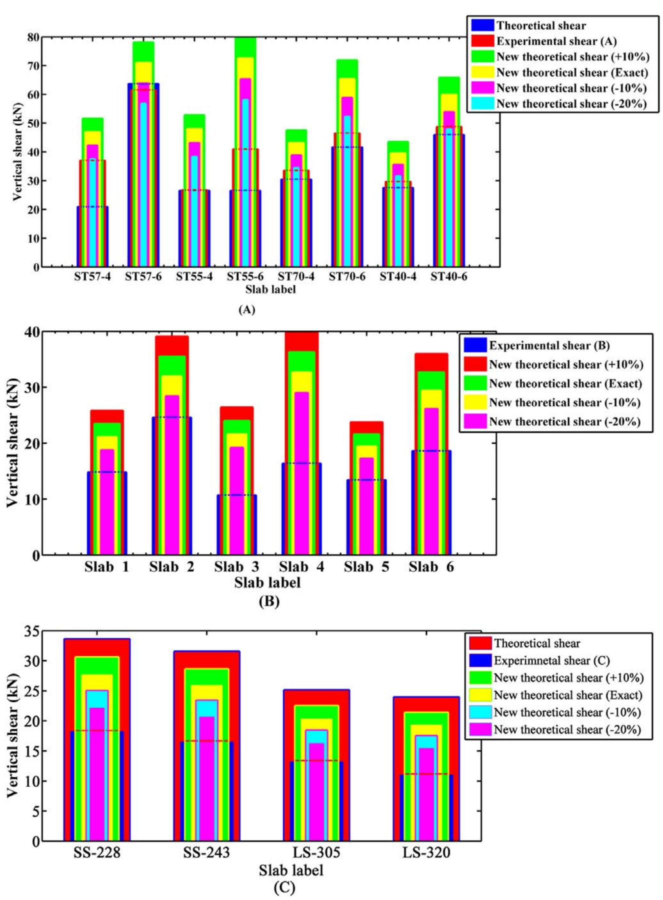

Fig. 13. Experimental and theoretical shear results comparisons: The experimental shear results are from: (A) [1] (B) [29] (C) This study experiment.

parenthesis *A* and *B* are from the literature as explained previously, while this study experiment is with notation *C*. The experimental shear value is computed using the experimental *FTL* values and the theoretical is from the design load expression given in Eq. (4). However, conflicting information on the profiled deck type used for the experiment by the author [29], hampers the computation of M_{rd}, because the sagging moment capacity, M_{pa}essential for the determination of theoretical shear is conflicting. For example, 1.0 mm LYSAGHT BONDEK cross sectional area is 1678 mm^2 [30] against 980 mm^2 given in [29]. Furthermore, the corresponding values of the new theoretical shear from the approximate estimation are varied (10%, exact, ± 20%). Hence, critically examining the shear behaviour shown in Fig. 13(A), the six shear groups differs significantly, $F(5, 42) = 4.75$, $p<0.05$. That behaviour is equally exhibited by this study designed experiment in comparisons; $F(5, 18) = 6.171$, $p<0.05$.

The results in Fig. 13(B) with five shear groups shows good agreement with the previous two shear groups, $F(4, 25) = 5.7$, $p<0.05$. The results are not surprising because of the penalty applied upon the FTL values. Evidently, the shear behaviour reported is in close agreement with literature findings; for example the works by [5,16]. It is the conclusion of this study that despite the lowly safety performance yield, the study derived FTL function under PSC compares well with the experimental results. The goodness of the closeness can be attributed to the use of sheeting deck characteristics as a major determinant in the proposed FTL function taking into considerations the compactness of the section. Abdullah, et al. [28], demonstrated this in similar, but deterministic study.

8. Conclusions

This paper presents reliability-based study of profiled deck composite slab employing partial shear connection method for the longitudinal shear estimation. The use of full-scale experimental test results led to the reliability estimation expressed in terms of safety indices using first order reliability method. In an attempt to presents a simplified load carrying capacity determination for PCS, this study developed a numerical scheme for PCS strength determination without necessarily conducting the present costlier strength verification of profiled deck composite slab using PSC method to EC4 design provision. The concise relation presented herein is simple with less computational cost. The accuracy of the predicted performance is well within the acceptable limits compared to the full-scale experimental test results as demonstrated by the statistical evidence. This study reaches the following conclusion:

- Experimental test load penalty has negligible influence on the shear strength value under PSC method in the estimation of design load value from the longitudinal shear.
- The relationship between sheeting deck characteristic including its slenderness function against the PCS performance function can be modelled effectively using the linear function.
- The longitudinally based safety performance is comparatively below the target safety value, but the derived numerical function for strength determination of PCS compares well with the experimental results. Hence, this function can aid in determining the PCS strength without the need for the costlier experimental procedures.

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