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Comparative Safety Performance Evaluation of Profiled Deck Composite Slab from the Use of Slope-Intercept and Partial Shear Methods

Izian Abd. Karim, Kachalla Mohammed, Nora Farah A. A. Aziz, Law Teik Hua

***Abstract*—**The economic use and ease of construction of profiled deck composite slab is marred with the complex and un-economic strength verification required for the serviceability and general safety considerations. Beside these, albeit factors such as shear span length, deck geometries and mechanical frictions greatly influence the longitudinal shear strength, that determines the ultimate strength of profiled deck composite slab, and number of methods available for its determination; partial shear and slope-intercept are the two methods according to Euro-code 4 provision. However, the complexity associated with shear behavior of profiled deck composite slab, the use of these methods in determining the load carrying capacities of such slab yields different and conflicting values. This couple with the time and cost constraint associated with the strength verification is a source of concern that draws more attentions nowadays, the issue is critical. Treating some of these known shear strength influencing factors as random variables, the load carrying capacity violation of profiled deck composite slab from the use of the two-methods defined according to Euro-code 4 are determined using reliability approach, and comparatively studied. The study reveals safety values from the use of *m-k* method shows good standing compared with that from the partial shear method.

***Keywords*—**Composite slab, first order reliability method, longitudinal shear, partial shear connection, slope-intercept.

I. Introduction

THE use of profiled steel deck composite slab in the construction industry is mainly for its simplicity in construction compared to other flooring system. The profiled sheeting can serve as shuttering, and shoulders' wet concrete during construction stage. Similarly, the composite action between the steel sheeting deck and the hardened concrete can effectively carries any addition loads that are consider during the design in addition to supporting their self-weight. Generally, this method of construction gain popularity by eliminating time-consuming erection and subsequent removal of temporary forms, and also the gained associated with concrete strength during service through performing the function of tensile reinforcement by the profiled steel deck [1]-[3].

The composite action between the profiled steel sheeting deck and the hardened concrete can effectively transmit with the development of longitudinal shear at the steel-concrete interface. Several studies [4]-[7] confirms the behavior of profiled deck composite slab is affected by the bond failure in the longitudinal direction. Longitudinal shear failure happens before the plastic bending capacity of the composite slab is reach, when such happens it is said that inadequate shear connection exist between the profiled sheeting deck and the hardened concrete. Intuitively, longitudinal shear capacity determines the ultimate strength of composite slab with profiled steel sheeting [8]. This is primarily due to the fact that ultimate load associated with shear bond loss between the steel sheeting and concrete is lower than the ultimate load bending failure in most studied composite slab failure modes [9].

Generally, deck fabricators provide design data for the engineers and builders for commonly available profiled sheeting deck used for composite construction. These parameters are from intensive and costlier laboratory procedures by estimating the shear bond capacity of profiled steel sheet composite deck. Presently, the shear bonds are estimated using the slope-intercept (*m-k*) method, the partial shear connection (*PSC*) method and the multi linear regression method. However, presently the two current methods adopted for the strength verification to *EC4* provision shows conflicting load capacity estimates. This can be attributed to the complexity of composite slab behaviors. It is known that horizontal shear bond governs the behavior and strength of composite slab, and the strength requirement depends on steel deck shape and profiled; frequency of embossment; load arrangement; shear span length; mechanical friction and type of end anchorage [10]. However, the combined effect of all or some of these factors treated as random variables on the safety performance of profiled deck composite slab is unknown. Hence, considering these factors as random variables, the study seeks to find which amongst the two methods provides considerable sound estimates of the failure probability from reliability perspective.

Literatures related to reliability studies on the performance of composite slab are scanty [2], very few areas is indeed covered. Degtyarev [2] presented reliability based analysis of composite slab at construction stage to US design provision of recent. The author studied the failure analysis of allowable stress design and load resistance factor design using First Order Reliability Method (*FORM*) for strength and deflection limit state violation conditions. The author finding reveals high level of conservatism in the current design method for composite steel deck construction using the US design

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provision and proposes modifications of the construction load requirement in the design code. However, in this paper the reliability indices and or the failure probability for the profile deck composite slab is computed on the basis of load capacity violation using both *PSC and m-k* method in accordance with *EC4* provision.

II. Longitudinal Shear Capacity

Design and verification of composite slab found in standard and codes are complicated and largely uneconomical because of the mandatory laboratory procedures that are required for the determination of its strength parameter [1], [3]. The shear bond parameters are from the full-scale experimental procedure. EC4 [11] provides a general guide for the bending resistance calculation for composite slab using either the *m-k* or the partial interaction method. The use of these methods in the determination of load carrying capacities yields different results [3], [12], [13].

In an experiment conducted by Cifuentes and Medina [12], comparisons between the load capacity estimates from the use of *m-k* and *PSC* methods shows a decrease of about 12% and 38% respectively [3]. In a similar experiment, the longitudinal shear strength from the use of m-k proves better than the other alternate method in *EC4* [13].

A. Slope-Intercept Method

The standard full-scale laboratory test procedure for longitudinal shear value parameter of profiled deck composite slab requires a minimum of six tests of three long, *X* and three short, *Y* specimens (Fig. 1). In this method, an initial load of 5000 cycles applied on the composite slab test specimen as shown in Fig. 1, with intention of separating the interface bond between sheeting deck and concrete. This action will eventually result in only mechanical interlock effect between them. The test load, w increased progressively afterwards until failure. Similarly, the parameters shown in Fig. 1, where the vertical shear stress, $V_t \ / \ bd_p$ is plotted against shear bond, $A_p \ / \ bl_s$ to get the *m* and *k* parameters in (1). These parameters are popularly termed as the slope and the intercept respectively. Where A_p is the sheeting deck effective cross-sectional area with yield strength, f_{yp} and d_p is the clear distance from the centroid-al distance of the profile sheeting to the topmost face of the concrete. The shear span length l_s is approximately $l \ / \ 4$, and clear span between supports, l is 3.0 *m* [14].

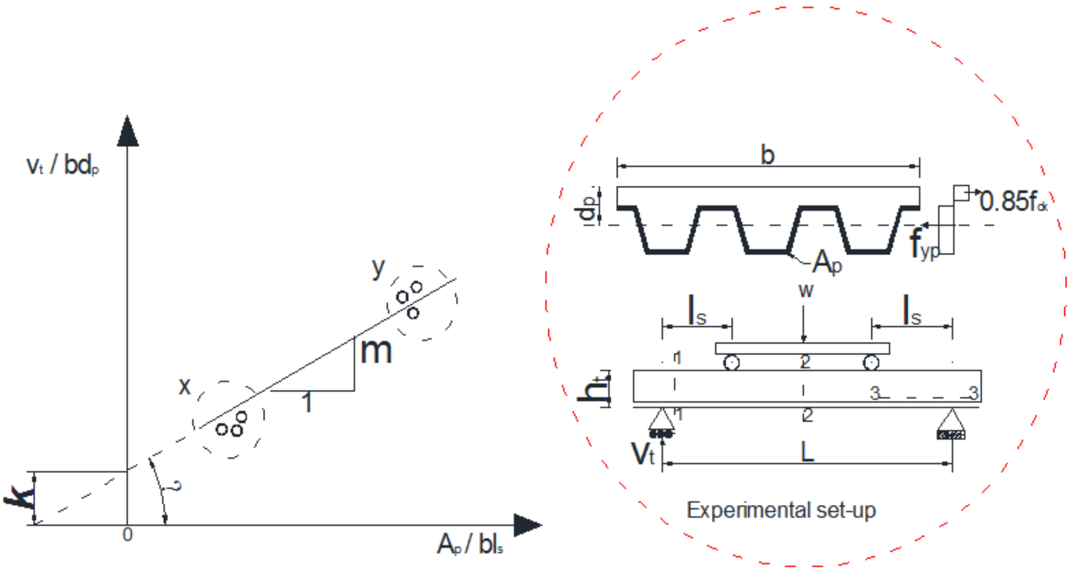

Fig. 1 Schematic *m-k* parameters determination from the plot of vertical shear against shear bond

$$V_{i,Rd} = \frac{bd_p}{j_q}\left(m\frac{A_p}{bl_s} + k\right) \qquad (1)$$

In (1), the design shears resistance, $V_{i,Rd}$ should at lesser than the vertical stress for a given width, *b* of the profiled deck composite slab. The parameter j_q in (1), is a factor for shear connection with a recommended value of 1.25 [1].

B. Partial Connection Method

Partial interaction is also another means other than the *m-k* method that can be used to obtained longitudinal shear strength. In this method, complete re-distribution of longitudinal shear is assumed between the sheeting deck and the concrete interface [15]. The degree of shear connection, $x(N_c \ / \ N_{cf})$ defines the level of re-distribution; $x = 0$ signifying no composite action, and $x = 1$ for full shear connection while slip and strain are assumed to be non-existence, x is between 0 and 1, partial shear connection is said to exist between the sheeting deck and the concrete. However, standard laboratory procedure is similar to that of *m-k* method, but the longitudinal shear, t_u developed by Johnson [14], for a given value of bending resistance, is by the use of (2).

$$t_u = \frac{x_{test}N_{cf}}{b(l_s + l_o)} \tag{2}$$

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, $N_{cf} = 0.85A_p f_{yp}$ .The design shear strength, $t_{u,Rd}$ is from the test result by dividing characteristic strength, $t_{u,Rk}$ with a partial safety factor of 1.25. The minimum value reduced by 10% for resisting slipping because of mechanical interlock, t_u gives $t_{u,Rk}$ [16].

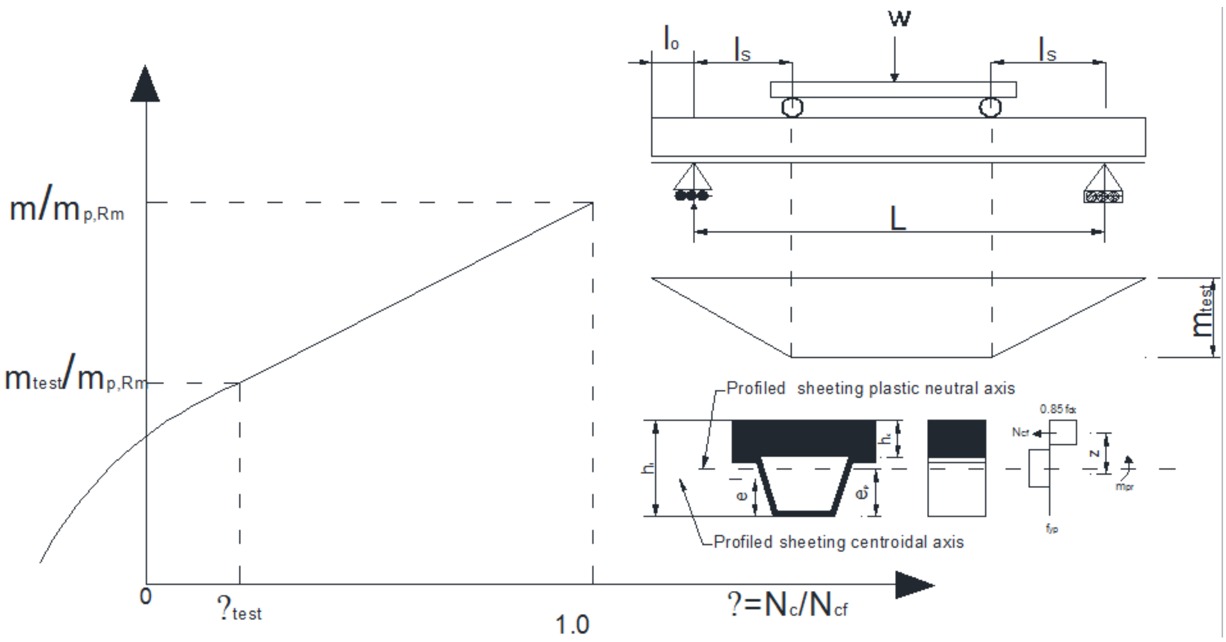

Fig. 2 Typical PSC interaction diagram

In this method, the bending resistance $m_{p,Rd}$ is highly dependent on the neutral axis, $N.A.$ positions within the system define using the stress block depth, x and obtained in use of (3).

$$m_{p,Rd} = N_{cf}z + m_{pr} \tag{3}$$

The plastic moment of resistance, m_{pr} and the lever arm, z in (3) is from the use of expression in (4).

$$\begin{aligned} m_{pr} &= 1.25m_{pa}(1 - x) \le m_{pa} \\ z &= h_t - e_p - 0.5x + (e_p - e)x \end{aligned} \tag{4}$$

In (4), m_{pa} represent plastic moment of resistance of the profile sheeting deck, e and e_p are the centroid distance and the plastic neutral axis above the base (Fig. 2). The *EC4* specifications for profiled deck composite slab thickness h_t should be greater than 80 *mm*.

III. Background on Study Experimental Data for Failure Test Load

In this study *FTL* values which are essential for the strength parameters determination are from full-scale experimental test work from different authors [3], [13]. Marimuthu, Seetharaman [3] carried out the experimental evaluation of profiled deck composite slab having rectangular embossing (width of 21 *mm* and length of 25 *mm*) and characteristics A_p , f_{yp} , d_p and h_t values of 420 mm^2 , 250 N/mm^2 , 77.5 *mm* and 105 *mm* respectively. The author cast the test according to *EC4* provision with *M20* grade concrete with 6 *mm* bars spaced @ 250 *mm c/c* and shear span lengths, l_s of 320, 350, 380 *mm* and 850, 950, 1150 *mm* for short and long specimens. Similarly, Hedaoo, Gupta [13] also carried out the experiments with Colour Roof India (CRIL) deck span; A_p is 839 mm^2, galvanized with 0.00254 *mm* coating on either face of the deck, and with embossment placed on adjacent webs. The composite slab has a nominal depth of 102 *mm*, width, b of 830 *mm*, concrete thickness above flange, h_c of 50 *mm*, d_p of 76.77 *mm* and a span length, l of 3 *m* respectively. To control temperature and shrinkage, 6 *mm* diameter meshes placed at 250 *mm* c/c both ways at mid concrete depth of 25 *mm* from top surface. The author conducted the test in accordance with standard provision [11] under varying l_s values of 300, 375, 450, 525, 600 and 675 *mm*.

IV. Structural Reliability

The inherent uncertainties present structural members caused by variability in both material strengths and loadings applied to the elements, eg variable load, wind loads or even energy dissipated from earthquakes. Conventionally, in the deterministic design, those un-certainties are accounted for in the structural design by using safety factors that amplify the design load and reduces the strength parameters [17]. Intuitively, higher the strength parameter, *R* compared with the applied load, *Q* safer the structural unit and vice versa. The *R* and *Q* parameter involves a considerable degree of uncertainty as such be treated as random variables [18].

TABLE I
Typical Experimental Failure Test Load Results Adopted for the Study

Source	Label	l_s (*mm*)	FTL (*kN*)	τ_u (*N/mm*^{2})
Marimuthu, Seetharaman [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
Hedaoo, Gupta [13]	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

Failure probability, p_f estimate of structural unit is the chance that a particular point the difference between the resistances offered by the material in relation to the applied load will result in value, i.e the resistance effect is less that the load applied. Mathematically, this is express as

$$p_f = prob(R - Q < 0) = p(k < 0) \tag{5}$$

where *k* is the limit state function or performance function; it defines the desired boundary, $k \geq 0$ from the un-desired boundary condition, $k < 0$. In general the performance function is

$$G = k(X_1, X_2, \ldots\ldots, X_n) \tag{6}$$

The basic discrete variables, X_i in (6) have influence on the performance function, hence, expression in (5) will be

$$p_f = p(G < 0) = \int F_R(x) f_Q(x) dx \tag{7}$$

where F_R and f_Q are the cumulative probability density function and the probability density function of the resistance and load effect respectively.

On a general term, the reliability of structural component is typically expressed by reliability index or safety index, β [2], [17]. Fig. 3 show the safety index as the shortest possible distance from the origin to the performance function curve, and is generally, known as the Most Probable Failure Point, *MPFP* or the design point, k^*. Hence, (7) can be expressed using (8), and Φ represents the inverse of the standard normal distribution function.

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

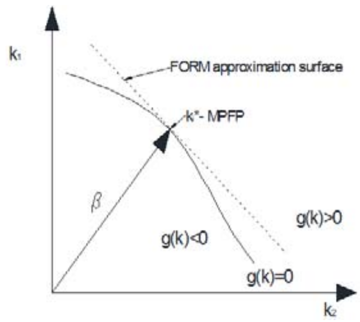

Fig. 3 Typical limit state surface approximations with FORM

V. Reliability Analysis

The paper presents, a comparative analysis of the reliability indices of profiled deck composite slab from the use of both *m-k* and *PSC* methods. The focus is on the material load carrying capacity and the design load estimated from the shear resistance of composite slab. The failure domain is when the design load exceeds the load carrying capacity of the composite profiled deck section. The study considers the *FTL* value as the material ultimate strength resistance. Afterward, the safety value determination is through *FORM* analysis of load carrying capacities limits states violations.

A. Resistance Model

Accounting for the random variability, the mean resistance, Q_m of the profiled deck composite slab is define accordingly [18], [19] as shown in (9).

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

where Q_m is the nominal resistance taken as the ratio of *FTL* over the span length with an assumed bias factor of 1.0. Similarly, M_n, F_n, and P_n are factors for material fabrication in particular for strength and modulus of elasticity, mean ratio of actual section to nominal value or fabrication factor for geometry and dimension of the component, and professional factor for approximation in structural analysis respectively. Similarly, the mean resistance coefficient of variation, V_Q is calculated from the expression in (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 variation, *COV* for the factors M_n , F_n , and P_n respectively. The study mean and *COV* values for these factors are 1.10, 0.1; 1.0, 0.05 and 1.11, 0.09 with all normally distributed [2] accordingly. Consequently, V_Q value for the study is 0.14 from the use of (10). Based on [20], the *COV* and distribution type for the parameters *b* and l_s are 0.17 and log-normal distribution, and a bias factor of 1.0 for both variables.

B. Limit State Formulation

The limit state violations define for this study, shown in expressions in (11) and (12) are from the use of *m-k* and *PSC* methods respectively.

$$Q_m - \frac{2V_{i,Rd}}{L} = R - Q \tag{11}$$

$$Q_m - \frac{m_{p,Rd}}{0.5 l_s} = R - Q \tag{12}$$

where *L* and l_s are the span and shear span lengths, and the parameters $V_{i,Rd}$, $m_{p,Rd}$ and Q_m are from the use of (1), (3) and (9) respectively. These expressions have to transform to basic variables formations, (13)-(14) shows the transformed equivalent function of expression in (11)-(12). There are three (X(1-3); *FTL*, *b*, and l_s) and seven (X(1-7); *FTL*, *b*, l_s , h_t , d_p , f_{ck} and f_{yp}) discrete variables associated with the respective functions.

$$\begin{aligned} R &= [(1 - \%/100)X(1)]/l \\ Q &= slope * ((A_p / (X(2) * X(3) + \sin tercept * 2 * X(2) * d_p / \\ &\quad (span * 1.25 * 10^3) \end{aligned} \tag{13}$$

$$\begin{aligned} R &= [(1 - \%/100)X(1)]/l \\ Q &= [(sc * A_p * X(7) * X(4)) - [(0.5 * A_p * X(7) / \\ &\quad (X(2) * X(6) * 0.85) - (X(4) - X(5)) * \exp^{-6}] + \\ &\quad (1.25 * pam * (1 - sc))]/ \\ &\quad (X(3) * \exp^{-3} * 0.5) \end{aligned} \tag{14}$$

In (13), *slope* and *intercept* stands for the *m* and *k* parameters. The variables; *sc* and *pam* in (14), similarly stands for the degree of shear connection and the sagging moment capacity of the profiled sheeting deck. While the basic variables statistical parameters are mentioned earlier in this section, the other remaining four variables all have log-normal distribution excerpt f_{ck} which has normal distribution characteristics [21]. Similarly, h_t and d_p have the similar statistical parameters in terms of bias factor and *cov* values (1.0, 0.05), while the duos of f_{ck} and f_{yp} has (1.4, 0.10) and (1.17, 0.17) respectively. All other parameters apart from the previously defined seven parameters are deterministic in this paper.

C. Reliability Analysis Result

In this study, load ratio, l_r is the ratio of *FTL* value over estimated design load derived using the longitudinal shear, and are identified using a reference letter's A & B in this study, and this shows the relation to the study source for the experimental *FTL* data's. For example, the ratio of [3] experimental *FTL* value to the deterministically computed design load is defined as A, and similar letter B stand for the respective ratio from [13]. Similar notations A' and B' designated in the paper represents a characteristics computations related to *FTL* source in this paper. Hence, Fig. 4 and 5 shows the behavior of safety index, b value in relation with the function. In these figures, the symbol ∝ stands for shear span length; ∝320 indicates shear span length of 320 mm for example. The *FTL* values are reduced from full tests load to 70% in magnitude, the markers on these figures shows the influence of this action on the safety performances. The reason for this action is to evaluate the influence of the present capacity reduction factor of 0.8 applied to the failure test load while computing the shear bond capacity of profiled deck composite slab [3].

Fig. 4 and 5 depicts Comparisons of the safety value estimates using the two methods. It is evident from this results that safety values obtained using *m-k* method shows good standing in comparison with *PSC* method. This could be attributed to the higher shear bond capacity results from the use of the former than the latter method [13], [22]. This is true because of reported that decreased in load carrying capacity is about 38% with *PSC* method when compared with only 12% using m-k approach [12]. Thus, this shows a decrease of about 26% in load carrying capacity, comparatively between *PSC* and *m-k* method [3].

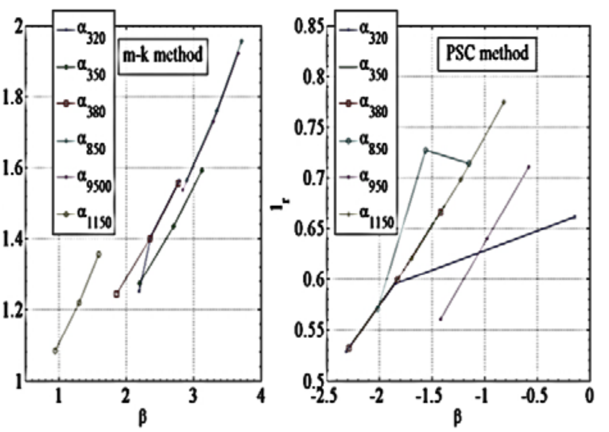

Fig. 4 Safety indices characteristics using A

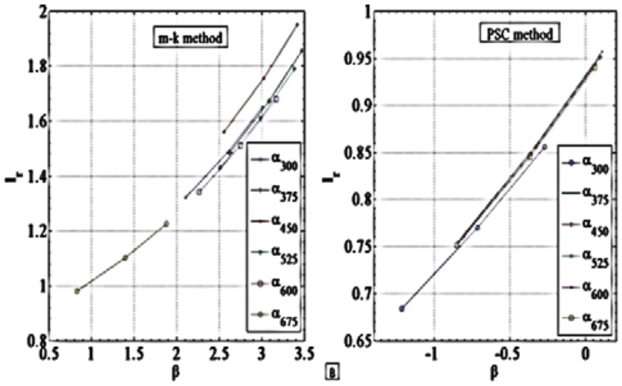

Fig. 5 Safety indices characteristics using B

Furthermore, the influence of greater span length are evidently shown to have lower safety values in comparisons with other shear span length using m-k approach; ∝1150 and ∝675 in Fig. 4 and 5, respectively, for example. However, this is due to the reported failed cyclic loading test during the experimental procedure, and their effect on lengthiest span length is significant [12].

D. Section Slenderness

Shear resistance of composite slab is heavily inclined to l_s / d_p ratio. Typically, in Fig. 1, the associated failure modes for composite slab deck are indicated using three different sections; flexural failure (1-1), vertical shear failure (2-2) and longitudinal shear failure (3-3) respectively. Johnson [14] shows vertical shear failures occurs with low l_s / d_p ratio, and higher ratio will result in flexural failure. Nevertheless, in between these ratios longitudinal shear failure occurs. In this study, the *PDC* slab performance are also considered using the inverted slenderness function, d_p / l_s taking into account the differences in cross section and yield strengths of the respective sheeting deck. The resulting property, v $(A_p f_{yp} d_p / l_s)$ defined for this study, plotted against p_f (Figs. 6 and 7) to know its influences using the two earlier methods mentioned. In these figures, the p_f values from the limit state function are from the penalized *FTL* (0-20%) value.

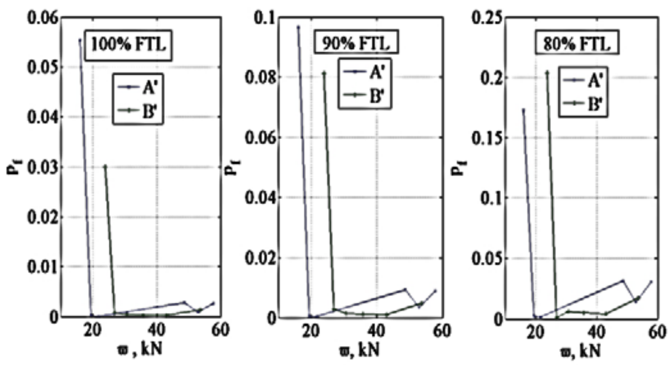

Fig. 6 Sheeting deck relationship with *FTL* using *m-k* method

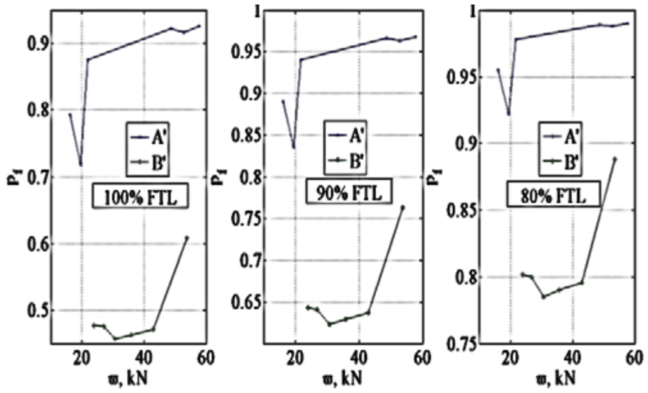

Fig. 7 Sheeting deck relationship with *FTL* using *PSC* method

In Fig. 7, the p_f function showed very distinct plot behaviors in comparison with that of Fig. 6. Generally, the performance behavior as expected decreases with decreasing *FTL* values from full to 80% values as indicted in both Fig. 6 and 7. Cautiously omitting the failed points during cyclic loading test as mentioned earlier, exponential trends $y = 0.0004e^{0.0741x}$ suitably defines the relation between v and p_f function, and this shows a good correlation, $r^2 = 80\%$ between them. This result suggest the possibility of using deck properties to predict the performance of *PDC* prior to constructions without necessarily conducting the costlier and time consuming laboratory procedures for its strength verification, but with the use of *m-k* approach.

VI. Summary

This paper presents reliability-based study of load capacity violation of profiled deck composite slab design provision using partial shear connection and slope-intercept method methods. The results from these methods according to *EC4* design provision for the load carrying capacity yields different and conflicting values. Shear span lengths and mechanical frictions greatly influence the behavior of profiled deck composite slab. Treating these factors as random variables, the load carrying capacity violation of *PDC* slab using the two-methods defined in *EC4* are determined using reliability approach, and comparatively studied. The study reveals safety values from the use of m-k method shows good standing compared with that from the *PSC* method.

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