Partially Connected Steel-Concrete Composite Plate Girders

(Penyambungan Separa Keluli-Konkrit Galang Berplat Komposit)

Mohd Yazmil Md Yatim, Nandivaram E. Shanmugam & Wan Hamidon Wan Badaruzzaman

ABSTRACT

A method to predict the bending behaviour and load carrying capacity of steel-concrete composite plate girders with partial shear connection under different load condition is presented. Based on curvature compatibility principle, the analytical solutions were performed by incorporating the degree of composite interaction and post-buckling strength. It is observed that the bending stiffness and ultimate load drops accordingly with decreasing degree of interaction. Accuracy of the proposed method is assessed by comparing the results with corresponding non-linear finite element analysis using LUSAS software. A satisfactorily correlation between the results has been observed.

Keywords: composite plate girders; partial interaction; ultimate load; slip

ABSTRAK

Kaedah untuk meramalkan kelakuan lenturan dan kapasiti beban galang berplat komposit keluli-konkrit tersokong mudah dengan sambungan ricih separa di bawah keadaan beban yang berbeza-beza dibentangkan. Berdasarkan prinsip keserasian lengkungan, penyelesaian secara analitikal dilakukan dengan mengambil kira darjah interaksi komposit dan kekuatan pasca-lekukan. Daripada keputusan yang diperolehi, dapat diperhatikan bahawa kekukuhan lenturan dan beban muktamad berkurangan dengan penurunan darjah interaksi. Ketepatan kaedah yang dicadangkan ini dapat dinilai melalui perbandingan keputusan dengan analisis unsur terhingga tidak linear menggunakan perisian LUSAS. Perbandingan keputusan menunjukkan korelasi yang memuaskan.

Kata kunci: galang berplat komposit; interaksi separa; beban muktamad; gelinciran

INTRODUCTION

Steel-concrete composite plate girders are widely used in civil engineering construction. The composite action in such girders is governed by the effectiveness of interaction between concrete slab and plate girder flange. In common practice, structural girders are designed as non-composite sections in which, steel girder and concrete slabs are designed independently disregarding the advantage of composite action. In some cases, composite design is normally simplified by employing full interaction theory due to the mathematical complexities to incorporate partial effect in the design. These practices lead to the uneconomical design where the oversized sections greatly affect the cost of construction. The presence of interlayer slip might induce significant additional curvature where the ignorance of this effect may result in inaccurate predictions of load carrying capacity and deflection of composite beams or plate girders (Johnson 2004). Moreover, the partial interaction design is needed in certain cases such as when the top flange of girder cannot accommodate the number of shear connectors required for full interaction.

A considerable amount of past research has been directed towards the study on the behaviour of steel plate girders. Allison et al. (1982) tested several composite plate girders subjected to combined bending and shear. A method to predict the shear strength was developed using tension field action in accordance with Evans et al. (1978). Small-scale simply supported composite girders with diagonal stiffeners at end panels were tested to failure by Porter and Cherif (1987). Methods for predicting shear strength of such girders are also presented. Shanmugam and Baskar (2003) conducted tests on several composite plate girders to investigate their shear strengths and concluded that the width of tension band in webs increased due to composite action. A number of composite girders subjected to negative bending were also tested to failure (Baskar & Shanmugam 2003). A design method for composite plate girders loaded in shear was proposed by Shanmugam and Baskar (2006) based on the failure mode observed in the experimental works. The study on the behaviour of composite plate girders has recently been directed to investigate the behaviour of horizontally curved girders by Shanmugam et al. (2009). An approximate method to determine the ultimate capacity of the horizontally curved girders containing web openings (Basher et al. 2009) and those with trapezoidal webs (Basher et al. 2011) girders was proposed. Full interaction was assumed between the slab and the girder flange throughout the study.

The behaviour of composite members depends on the type of connection between the two materials. Rigid shear connectors usually develop full composite action between the two materials, thus, conventional principle of analysis of full interaction can be applied. In contrast, flexible shear connectors permit the development of partial composite action because of strain incompatibility and therefore, the analysis procedures require consideration of the interlayer slip between the materials (Ridza 2008). Partial interaction results from the existence of imperfect composite action. Strain incompatibility occurs between the interfaces of the materials. Horizontal shear is transferred from one element to the other through the shear connectors. The second order differential equation allowing slip for composite beams was first developed by Newmark et al. (1951) by assuming equal curvature between the interacting elements. Experimental studies were made on composite beams subjected to single concentrated load at midspan. Expressions for slip and deflection were derived accounting for imperfect interaction. Adekola (1968) presented an interaction theory for composite beams allowing interface friction, slip and uplift deformation. The fourth and second order coupled differential equations were derived and solved by finite difference method. It was found that the uplift deformation is insignificant.

The current paper proposes a method for developing load-deflection relationship to predict the behaviour and ultimate strength of simply supported composite plate girders with partial interaction subjected to concentrated or uniformly distributed loads. The degree of interaction is specified by varying the longitudinal spacing of shear connectors along the steel flanges. Nevertheless, the uplift deformation is not considered herein as it is assumed that both elements equally deflect with same amount of curvature.

INTERFACE SLIP SOLUTIONS

EQUILIBRIUM OF FORCES

A free body diagram of an elemental length, dx in a composite girder is assumed, in accordance with partial interaction theory (Johnson 2004) as shown in Figure 1. Since the uplift deformation is negligible, the normal force acting at steel-concrete interface may be disregarded.
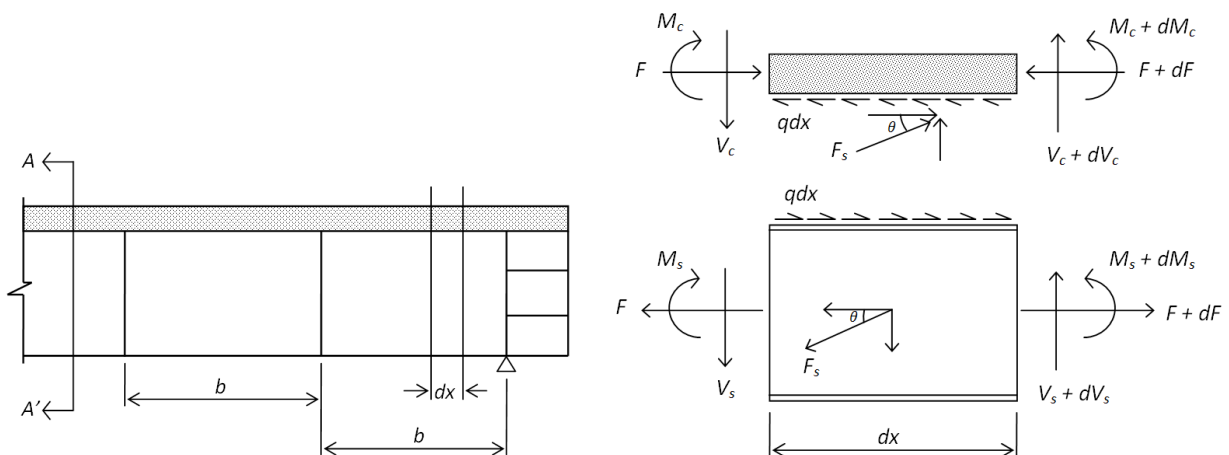

FIGURE 1. Calculation model for a typical composite plate girder

In the above figure, b is the width of web panel; M_c and M_s represent moment carried by concrete slab and steel girder respectively; V_c and V_s are corresponding shear forces; F is the compressive or tensile force exerted in concrete or steel; F_s is the tension field force in web panel and q is the horizontal shear force at steel-concrete interface.

Figures 2, 3(a) and 3(b) show the cross section of a composite girder and load definitions respectively.
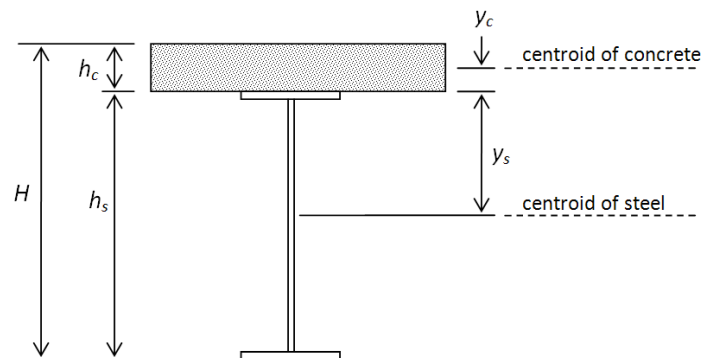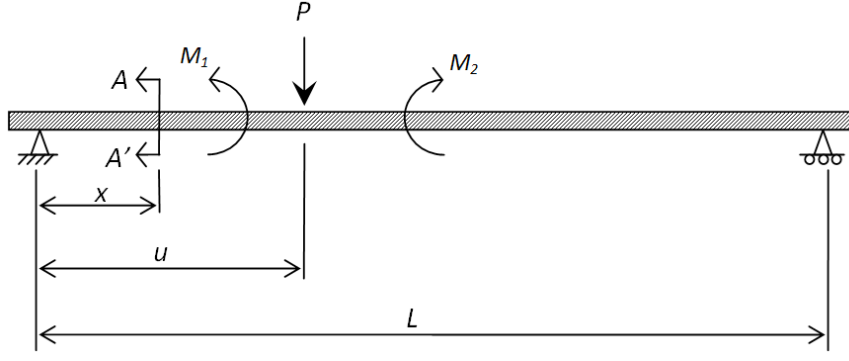

FIGURE 2. Cross section of a composite girder

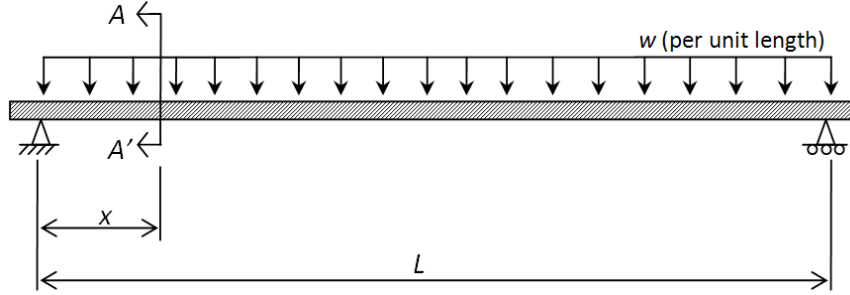FIGURE 3(a). Load condition – single concentrated load

FIGURE 3(b). Load condition – uniformly distributed load

where, x is the distance from the left support of any section A-A' along the span length. Equilibrium of forces in horizontal direction in Figure 1 gives,

$$\frac{dF}{dx} = -q + \frac{F_s \cos\theta}{b} \qquad (1)$$

SLIP SOLUTIONS

Based on the assumption that the amount of slip, s is directly proportional to horizontal shear, q at steel-concrete interface (Newmark et al. 1951; Johnson 2004), it can be written as,

$$q = \frac{ks}{p} \qquad (2)$$

in which, k is the shear stiffness of connectors and p is the longitudinal pitch between connectors.

Strains at the bottom fibre of concrete and at the top of steel may be written, respectively as,

$$\varepsilon_{cb} = \frac{M_c y_c}{E_c I_c} - \frac{F}{E_c A_c} \qquad (3)$$

$$\varepsilon_{st} = -\frac{M_s y_s}{E_s I_s} + \frac{F}{E_s A_s} \qquad (4)$$

where, E_c, E_s, I_c, I_s, A_c and A_s refer to Young's modulus, second moment of area and cross section area of the concrete and steel, respectively.

The rate of change in slip along the steel-concrete interface (Nie & Cai 2003) is written as,

$$\frac{ds}{dx} = \varepsilon_{cb} - \varepsilon_{st} \qquad (5)$$

Using curvature compatibility principle (Newmark et al. 1951), the expression for curvature may be written as,

$$\frac{d^2 Y}{dx^2} = \frac{M_c}{E_c I_c} = \frac{M_s}{E_s I_s} = \frac{M - F d_c}{E_c I_c + E_s I_s} \qquad (6)$$

in which, Y is the deflection of the entire section; M is the moment carried by the entire section and $d_c = y_c + y_s$.

Using Equations (3), (4) and (6), the slip strain as in Equation (5) may be derived as,

$$\frac{ds}{dx} = \alpha(M - F d_c) - \beta F \qquad (7)$$

and,

$$\frac{d^2 s}{dx^2} = \alpha\left[\frac{dM}{dx} - d_c\left(\frac{dF}{dx}\right)\right] - \beta\left(\frac{dF}{dx}\right) \qquad (8)$$

where,

$$\alpha = \frac{d_c}{E_c I_c + E_s I_s} \qquad (9)$$

$$\beta = \frac{E_c A_c + E_s A_s}{E_c A_c E_s A_s} \qquad (10)$$

For single concentrated load case as shown in Figure 3(a), the moment at any section at a distance x along the span is,

$$M_1 = P\left(1 - \frac{u}{L}\right)x \qquad \text{for } 0 \leq x \leq u \tag{11}$$

$$M_2 = \frac{P(L - x)u}{L} \qquad \text{for } x > u \tag{12}$$

Substitution of Equations (1) and (2) along with the first derivative of Equation (11) in Equation (8) yields,

$$\frac{d^2s}{dx^2} = \alpha\left[P\left(1 - \frac{u}{L}\right) - d_c\left(\frac{F_s\cos\theta}{b} - \frac{ks}{p}\right)\right] - \beta\left(\frac{F_s\cos\theta}{b} - \frac{ks}{p}\right) \tag{13}$$

Integrating Equation (13) twice, and with boundary conditions $ds/dx=0$ at $x=0$ and $s=0$ at $x=u$, the slip expression may be simplified as,

$$s_1 = \frac{\alpha\left(1 - \frac{u}{L}\right)P - \psi\left(\frac{F_s\cos\theta}{b}\right)}{[2/(x^2 - u^2)] - \psi K} \qquad \text{for } 0 \leq x \leq u \tag{14}$$

Similarly, with Equation (12) for moment and boundary conditions $ds/dx=0$ at $x=L$ and $s=0$ at $x=u$, the slip solution for right hand side of the applied load is given as,

$$s_2 = \frac{\alpha Pu + \psi\left(\frac{F_s\cos\theta}{b}\right)L}{(2L/[(x - u)(2L - x - u)]) + \psi LK} \qquad \text{for } x > u \tag{15}$$

in which, $\psi = \alpha d_c + \beta$ and $K = k/p$.

For uniformly distributed load condition as in Figure 3(b), similar procedures may be carried out using the following general expression for moment,

$$M = \frac{w(L - x)x}{2} \tag{16}$$

Again, with boundary conditions $ds/dx=0$ at $x=0$, $s=0$ at $x=L/2$ (for $0 \leq x \leq L/2$) and $ds/dx=0$ at $x=L$, $s=0$ at $x=L/2$ (for $L/2 < x \leq L$), the slip expressions may be derived as,

for $0 \leq x \leq L/2$,

$$s_1 = \frac{-\alpha w(L^3 - 6Lx^2 + 4x^3) + 3\psi(L^2 - 4x^2)\left(\frac{F_s\cos\theta}{b}\right)}{24 + 3\psi K(L^2 - 4x^2)} \tag{17}$$

for $L/2 < x \leq L$,

$$s_2 = \frac{-\alpha w(L^3 - 6Lx^2 + 4x^3) - 3\psi(3L^2 - 8Lx + 4x^2)\left(\frac{F_s\cos\theta}{b}\right)}{24 - 3\psi K(3L^2 - 8Lx + 4x^2)} \tag{18}$$

ADDITIONAL DEFLECTION DUE TO SLIP

Longitudinal slip at steel-concrete interface induces additional curvature when composite structures bend. According to Nie and Cai (2003), the slip-induced curvature may be obtained from,

$$\Delta\left(\frac{d^2y}{dx^2}\right) = \frac{1}{H}\left(\frac{ds}{dx}\right) \tag{19}$$

Substituting Equation (14) into above equation and performing double integration with boundary conditions $\Delta(dy/dx)=0$ at $x=u$ and $\Delta y=0$ at $x=0$, the expression for slip-induced deflection, Δy may be obtained. Substituting all parameters; i.e. L, x, u, K, b, $\cos\theta$, α, ψ and F_s in Δy expression, the load-deflection relationships may be obtained in the following forms,

$$\Delta y_e = AP \tag{20}$$

$$\Delta y_{pb} = BP + C \tag{21}$$

in which, Δy_e and Δy_{pb} are slip-induced deflections in elastic and post-buckling range respectively; P is the concentrated load and A, B and C are constants.

STRENGTH PARAMETERS

STRENGTH OF CONCRETE SLAB

One of the approximate methods to predict the strength of composite plate girders is by simply adding the shear capacity of steel girder alone to the shear strength of concrete slab (Shanmugam & Baskar 2006; Narayanan et al. 1989). To account for partial interaction, the following relationship is introduced,

$$V_{cc} = V_a + \frac{N}{N_f}\left(V_f - V_a\right) \tag{22}$$

in which, V_{cc} being the shear resistance of concrete slab with partial interaction; V_a is the shear resistance of concrete slab alone (Eurocode 2 1992); V_f is the shear resistance of concrete slab with full interaction; N refer to the number of shear connectors and N_f is the number of shear connector required to achieve full composite interaction. V_f is taken equal to the pull-out capacity of shear connectors (Liang et al. 2004). For full shear connection, i.e. $N/N_f = 1$, Equation (22) yields,

$$V_{cc} = V_f \tag{23}$$

To justify the amount of contribution by concrete slab in the elastic and post-buckling phases, an extensive finite element analyses were carried out with different degrees of interaction. From the analyses, it may be concluded that the concrete contributes around 40% to the web buckling resistance, in addition to the resistance of steel web alone, noted as $V_{cc,e}$. Thus, the remaining concrete capacity is taken as the contribution to the post-buckling phase, noted as $V_{cc,pb}$.

BUCKLING LOAD

The critical shear, V_{cr} that causes buckling in the web is taken as the sum of web panel shear resistance and the strength contribution by concrete slab in elastic phase, $V_{cc,e}$ and hence,

$$V_{cr} = \tau_{cr}\,dt + V_{cc,e} \tag{24}$$

where, d being the depth of web panel; t is the web thickness and τ_{cr} refer to the critical shear stress of web panel which is taken as,

$$\tau_{cr} = C\left[\frac{\pi^2 E_s}{12(1-v^2)}\right]\left(\frac{t}{d}\right)^2 \tag{25}$$

in which, C and v are the buckling coefficient and Poisson ratio of web material, respectively.

SHEAR IN POST-BUCKLING PHASE

From equilibrium, the shear carried by each support at any load increment in post-buckling state may be computed as,

$$V_C = V_{cr} + V_{cc,pb} + F_s \sin\theta \tag{26}$$

Substituting Equation (24) into Equation (26) and since $V_{cc,e} + V_{cc,pb} = V_{cc}$,

$$V_C = \tau_{cr}\,dt + V_{cc} + F_s \sin\theta \tag{27}$$

TENSION FIELD ACTION

Once a web plate has lost its capacity to sustain any further increase in compressive stress, a new load carrying mechanism is then developed. Further load beyond buckling are supported by an inclined tensile membrane field in the web (Evans 1983). The tensile stress has a resultant force given as,

$$F_s = \sigma_t\,tb_{tf} \tag{28}$$

where, σ_t is the tensile membrane stress and b_{tf} is the width of diagonal tension band developed in the web plate.

It should be noted that, the calculation of b_{tf} herein is basically in accordance with Cardiff method (Porter et al. 1975). Rearranging Equation (27) gives expression for tension field force as,

$$F_s = \frac{P - 2(\tau_{cr}\,dt + V_{cc})}{2\sin\theta} \tag{29}$$

Leaving P as an unknown, Equation (29) is then substituted into the expression for Δy along with other parameters to obtain the load-deflection equation at post-buckling state in the form given in Equation (21). Prior to this, setting $F_s = 0$ and substituting into the expression for Δy would yield load-deflection equation similar to Equation (20). Furthermore, substituting Equation (28) into Equation (29) gives the expression for tensile membrane stress in web plate as,

$$\sigma_t = \frac{P - 2(\tau_{cr}\,dt + V_{cc})}{2tb_{tf}\sin\theta} \tag{30}$$

LOAD-DEFLECTION RELATIONSHIP

EFFECTIVE BENDING STIFFNESS

Even in full composite design, the steel-concrete interaction is not complete where the stiffness may reduce to some extent due to the flexibility of shear connectors. In the present study, the effective bending stiffness, EI_{eff} is employed in accordance with the principle suggested by

Girhammar (2009). The bending stiffness for composite section with complete interaction may be written as,

$$EI_\infty = EI_0 + \frac{EA_p d_c^2}{EA_0} \qquad (31)$$

in which,

$$EI_0 = E_s I_s + E_c I_c \qquad (32)$$

$$EA_p = E_s A_s \cdot E_c A_c \qquad (33)$$

$$EA_0 = E_s A_s + E_c A_c \qquad (34)$$

where, EI_0 is the bending stiffness of non-composite section; EA_p is the product of axial stiffness of sub-elements and EA_0 is the sum of axial stiffness of sub-elements.

To account for partial shear connection, the effective bending stiffness is computed corresponding to the complete interaction as follows,

$$EI_{eff} = EI_0 + \xi(EI_\infty - EI_0) \qquad (35)$$

where,

$$\xi = \left[1 + \frac{\pi^2 EA_p}{KL^2 EA_0}\right]^{-1} \qquad (36)$$

It should be noted that, EI_{eff} is constant throughout the elastic phase. Once a girder has buckled, the flexural rigidity changes proportionally to the applied load. Hence, the EI_{eff} value at post-buckling state should be computed at every load increment using tangent modulus, E_t (Timoshenko & Gere 1961) to replace the constant E_s value in Equations (32), (33) and (34). The tangent modulus is written as,

$$E_t = E_s \frac{\sigma_{yw} - \sigma_\theta}{\sigma_{yw} - \omega\sigma_\theta} \qquad (37)$$

in which, for structural steel, ω=0.96 to 0.99 can be taken with good accuracy; σ_{yw} is the yield stress of web material and σ_θ being the resulting tensile stress written as,

$$\sigma_\theta = \tau_{cr} \sin 2\theta + \sigma_t \qquad (38)$$

TOTAL DEFLECTION

The deflection of whole composite section is computed at every load increment accordingly. Basically, the total deflection, Y is the sum of bending deflection, y_b, shear deflection, y_v and slip-induced deflection, Δy. For single point load at mid-span, these can be expressed as,

$$Y_e = \frac{PL^3}{48EI_{eff,e}} + \frac{PL(1+v)}{2E_s A_w} + \Delta y_e \qquad (39)$$

$$Y_{pb} = \frac{PL^3}{48EI_{eff,pb}} + \frac{PL(1+v)}{2E_t A_w} + \Delta y_{pb} \qquad (40)$$

where, Y_e, Y_{pb}, Δy_e and Δy_{pb} refer to total deflection and slip-induced deflection in elastic and post-buckling phases, respectively; $EI_{eff,e}$ and $EI_{eff,pb}$ are bending stiffness in elastic range and beyond buckling, respectively and A_w being the shear area.

CONSTRUCTION OF LOAD-DEFLECTION CURVE

In the present method, load-deflection curve is constructed by plotting the load-deflection equations obtained in Equations (20) and (21). Both relationships are then superposed to represent the whole behaviour of composite plate girders loaded to failure as illustrated in Figure 4. Point B is the upper limit for the elastic phase from which, when the shear force carried by a girder has reached the critical value as given in Equation (24).

Beyond web buckling, the load-deflection relationship exhibits different behaviour compared to the unbuckled state. The change of slope at every load increment resulting from the stiffness reduction yields the non-linear pattern. Similar procedure is carried out to construct the curve BCD using Equation (21). To determine the total deflection, Equations (37), (39) and (40) may be used. The change of slope shown by curve BC is due to the reduced flexural rigidity for which, induces larger deflection even for small amount of load increment. The plot is ended when σ_θ reaches σ_{yw} where at this point, indicated as point C, the web has yielded and a girder has reached the maximum load it could sustain. Beyond this point, further increase in Δy_{pb} would not induce significant changes in P. Thus, the curve levelled off as portrayed by line CD.
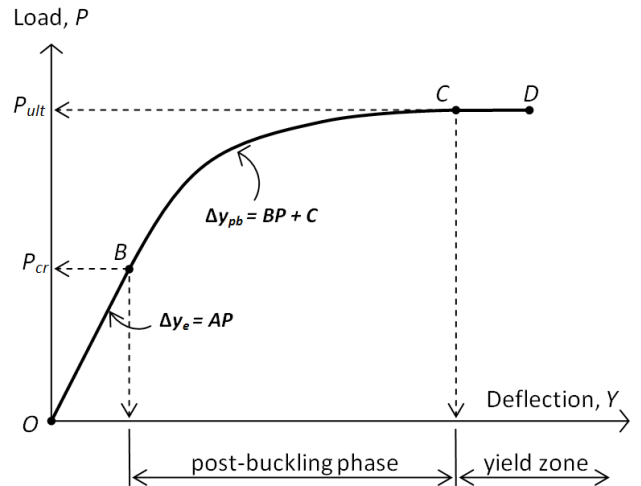

FIGURE 4. Construction of load-deflection curve

NUMERICAL ANALYSIS

Non-linear finite element analyses were carried out to assess the accuracy of the proposed method. Composite plate girders tested earlier by other researchers were analysed using the finite element package, LUSAS. Three-

dimensional models were developed by adopting shell and brick elements in the LUSAS element library to idealise the steel plate girders and concrete slabs. The steel plate girders were modelled using ungraded mild steel material with Poisson ratio of 0.30. The Young's modulus and yield stress, however, are in accordance with experimental tests in past works. Finite element mesh with element size of 50×50 mm, shown to provide solutions with acceptable accuracy and computational time, was adopted in the analysis. A typical mesh is shown in Figure 5.
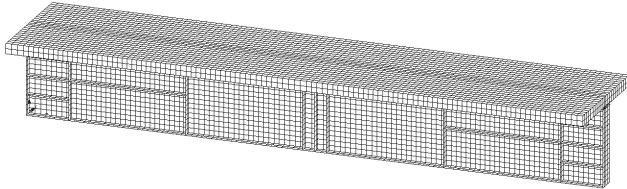
FIGURE 5. Typical finite element mesh

EVALUATION OF RESULTS

Four girders namely CPG 1, CPG 2, CPG 7 and CPG 8, tested in past works (Shanmugam & Baskar 2003; Baskar & Shanmugam 2003) were analysed by the proposed method and finite element modelling. Five different degrees of interaction defined by stiffness parameter, K were considered so as to examine the effects of partial interaction on the strength and bending behaviour of the girders. The ultimate loads and load-deflection behaviour obtained are presented herein.

ULTIMATE LOAD CAPACITY

The results for ultimate loads obtained by the proposed method and finite element analysis are presented in Table 1 for the case of concentrated load applied at the mid-span and in Table 2 for the girder under uniformly distributed load through the entire span. It can be observed from Table 1 that, the reduction in K results in lower load carrying capacity. For example, the ultimate load decreases around 17% for CPG 1, 10% for CPG 2, 19% for CPG 7 and 8% for CPG 8 when K is reduced from 0.65 to 0.17/0.19. Similar variations of strength can be observed for girders under uniformly distributed load as shown in Table 2. Close agreement between the ultimate loads predicted by the two methods show that the proposed method is able to predict the capacity of composite plate girders with sufficient accuracy.

LOAD-DEFLECTION BEHAVIOUR

Load-deflection behaviour obtained by proposed method for typical girders are shown in Figure 6 for the case of concentrated load applied at the mid-span and in Figure 7 for the girders subjected to uniformly distributed load on the

TABLE 1. Comparison of ultimate loads (single concentrated load condition)

Specimens	K	P_{ult} (kN/m)	$P_{ult\ LUSAS}$ (kN/m)	P_{ult} / $P_{ult\ LUSAS}$
CPG 1	0.65	827	911	0.91
	0.51	787	875	0.90
	0.37	746	793	0.94
	0.31	726	715	1.02
	0.17	686	655	1.05
CPG 2	0.65	1355	1371	0.98
	0.51	1315	1324	0.99
	0.37	1276	1282	0.99
	0.31	1255	1226	1.02
	0.17	1216	1171	1.04
CPG 7	0.65	754	775	0.97
	0.51	714	762	0.94
	0.38	675	725	0.93
	0.32	653	723	0.90
	0.19	612	689	0.89
CPG 8	0.65	1266	1172	1.08
	0.51	1238	1121	1.10
	0.38	1210	1105	1.09
	0.32	1195	1092	1.09
	0.19	1166	1058	1.10

TABLE 2. Comparison of ultimate loads (uniformly distributed load condition)

Specimens	K	w_{ult} (kN/m)	$w_{ult\ LUSAS}$ (kN/m)	w_{ult} / $w_{ult\ LUSAS}$
CPG 1	0.65	344	332	1.04
	0.51	328	319	1.03
	0.37	311	301	1.03
	0.31	302	280	1.08
	0.17	286	262	1.09
CPG 2	0.65	565	540	1.05
	0.51	549	531	1.03
	0.37	532	519	1.03
	0.31	524	498	1.05
	0.17	507	474	1.07
CPG 7	0.65	157	166	0.95
	0.51	148	161	0.92
	0.38	140	152	0.92
	0.32	135	143	0.94
	0.19	127	116	1.09
CPG 8	0.65	264	252	1.05
	0.51	258	247	1.04
	0.38	252	240	1.05
	0.32	249	235	1.06
	0.19	243	223	1.09

entire span. It can be seen that all girders exhibit identical behaviour up to the respective yield point. Apparently, the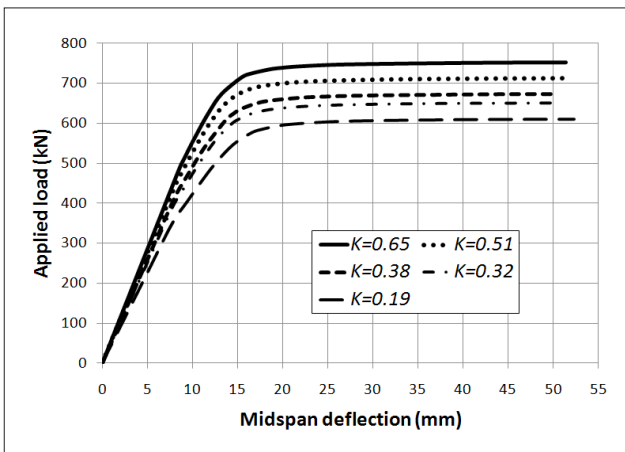 degree of shear connection governs the flexural stiffness of composite plate girders as indicated by the reduction in the slope of the load-deflection curves. This reduction, however, is not significant at the initial stages of loading.
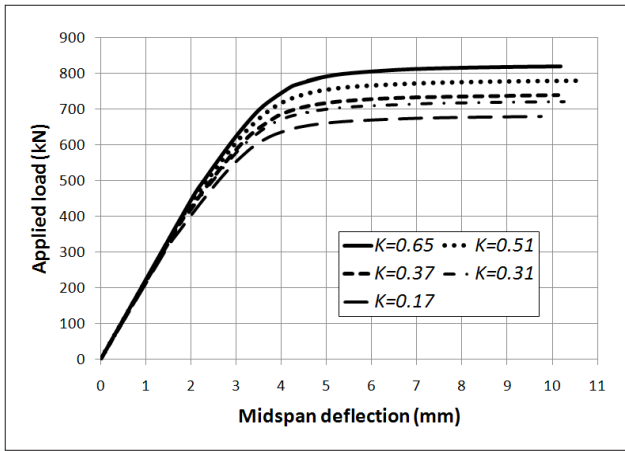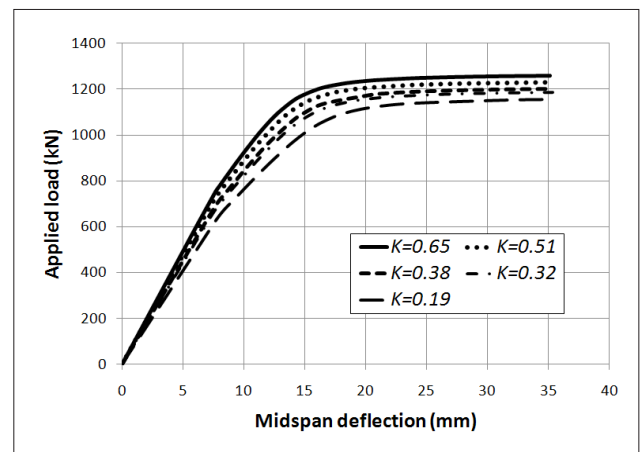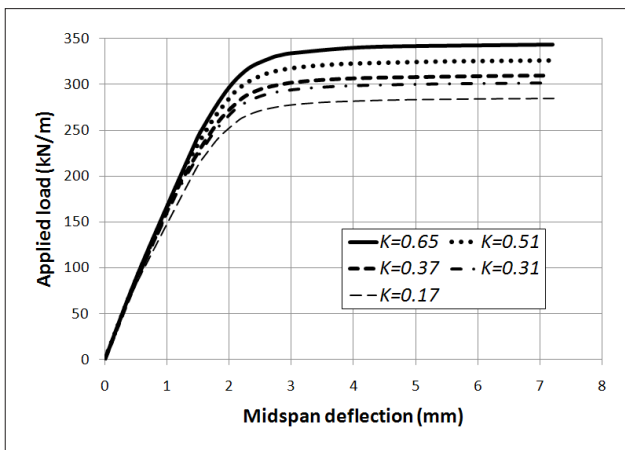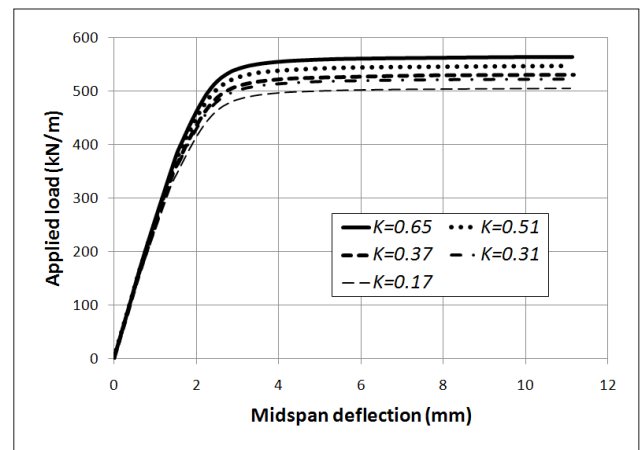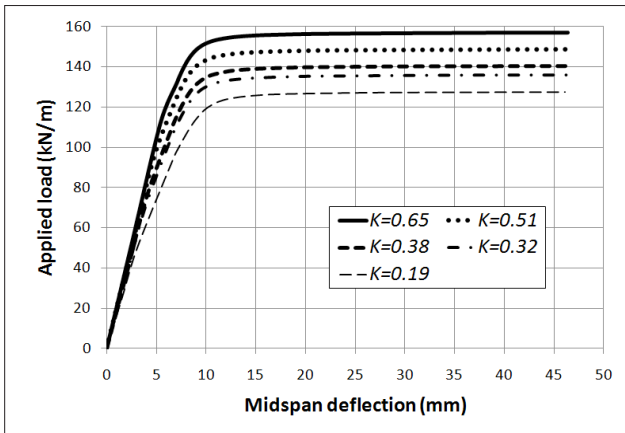
(a) CPG 1
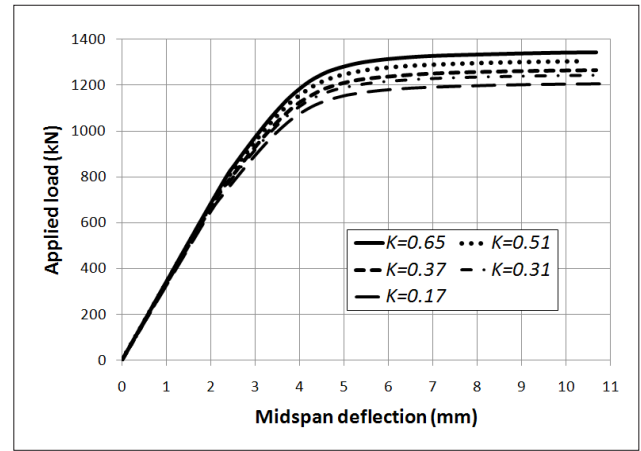
(b) CPG 2

(c) CPG 7

(d) CPG 8

FIGURE 6. Load-deflection behaviour for girders under concentrated load at mid-span

(a) CPG 1

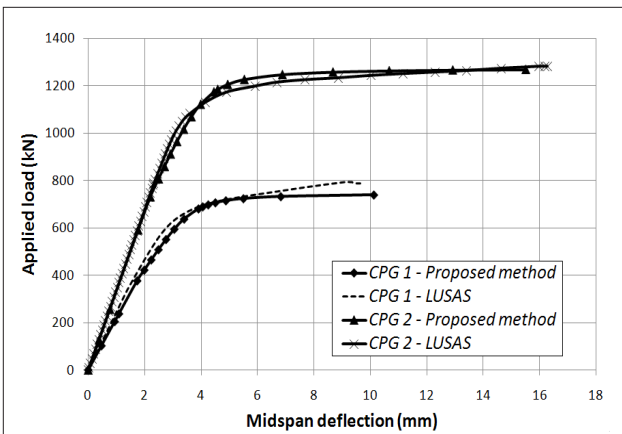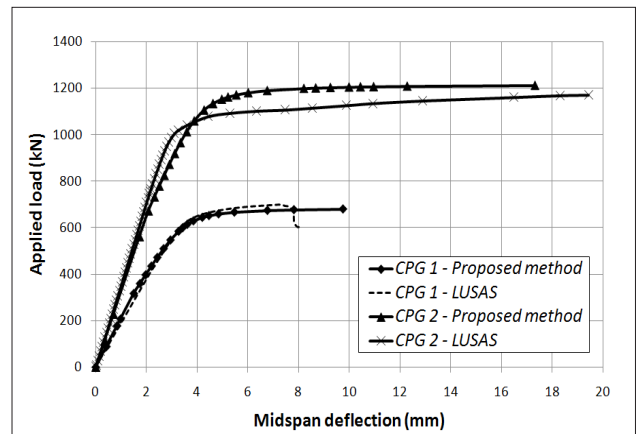(b) CPG 2
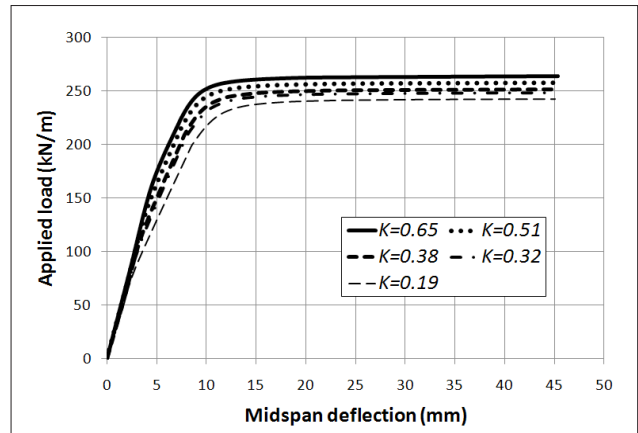
(d) CPG 8   (c) CPG 7

FIGURE 7. Load-deflection behaviour for girders under uniformly distributed load

In addition, the accuracy of the proposed method has been also assessed by comparing the predicted load-deflection behaviour with the corresponding results obtained from the finite element method. Typical results presented in Figure 8 show the variation of mid-span deflection with the applied load for selected girders with different K values. It can be seen from the figures that, the two results are close each other from the initial stage of loading to the ultimate load condition. The observation is true for different loading conditions and for different K values. The results show that the proposed method is also capable of predicting the behaviour with sufficient accuracy.
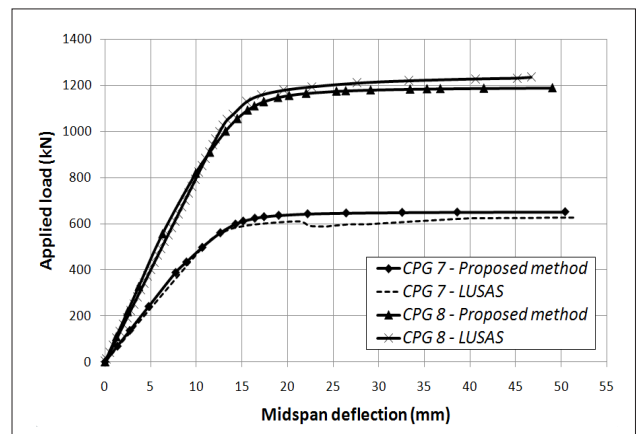

(a) CPG 1 & CPG 2 (K=0.37)   (b) CPG 1 & CPG 2 (K=0.17)
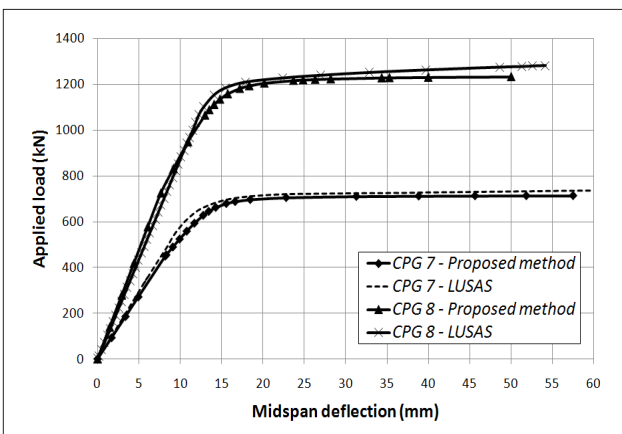
(c) CPG 7 & CPG 8 (K=0.51)   (d) CPG 7 & CPG 8 (K=0.32)

FIGURE 8. Comparisons of load-deflection plots-typical behaviour

CONCLUSIONS

The study presents a method to determine the behaviour and flexural capacity of thin webbed composite plate girders with partial interaction subjected to single concentrated and uniformly distributed loads. It is observed that the bending stiffness and load carrying capacities of composite plate girders significantly reduced with decreasing degree of shear connection. The flexural behaviour of all girders exhibit similar pattern up to the respective yield point as portrayed in the load-deflection curves. Comparison for ultimate loads with finite element analysis show acceptable correlation within ±10%. Hence, it may be generally concluded that the proposed method is capable of predicting the strength and flexural behaviour of composite plate girders with partial interaction to an acceptable level of accuracy.

REFERENCES

Adekola, A. O. 1968. Partial Interaction Between Elastically Connected Elements of a Composite Beam. International Journal of Solids Structures 4: 1125-1135.

Allison, R. W., Johnson, R. P. & May, I. M. 1982. Tension Field Action in Composite Plate Girders. Proc. of the Instn. of Civil Engrs, London, 73, pp. 255-276.

Basher, M. A., Shanmugam, N. E. & Khalim, A. R. 2009. Web Openings in Horizontally Curved Composite Plate Girders. Journal of Constructional Steel Research 65: 1694-1704.

Basher, M., Shanmugam, N. E. & Khalim, A. R. 2011. Horizontally Curved Composite Plate Girders with Trapezoidally Corrugated Webs. Journal of Constructional Steel Research 67: 947-956.

Baskar, K. & Shanmugam, N. E. 2003. Steel–Concrete Composite Plate Girders Subject to Combined Shear and Bending. Journal of Constructional Steel Research 59: 531-557.

Eurocode 2. 1992. Design of Concrete Structures. British Standard.

Evans, H. R. 1983. Plated Structures – Stability and Strength, edited by Narayanan, R. Applied Science Publishers Ltd.

Evans, H. R., Porter, D. M. & Rockey, K.C. 1978. The Collapse Behaviour of Plate Girders Subjected to Shear and Bending. Proc. of IABSE, 4, pp. 1-20.

Girhammar, U. A. 2009. A Simplified Analysis Method for Composite Beams with Interlayer Slip. International Journal of Mechanical Sciences 51: 515-530.

Johnson, R. P. 2004. Composite Structures of Steel and Concrete. Blackwell Scientific Publications Ltd.

Liang, Q. Q., Uy, B., Bradford, M. A. & Ronagh, H. R. 2004. Ultimate Strength of Continuous Composite Beams in Combined Bending and Shear. Journal of Constructional Steel Research 60: 1109-1128.

Narayanan, R., Al-Amery, R. I. M. & Roberts, T. M. 1989. Shear Strength of Composite Plate Girders with Rectangular Web Cut-Outs. Journal of Constructional Steel Research 12: 151-166.

Newmark, N. M., Siess, C. P. & Viest, I. M. 1951. Test and Analysis of Composite Beams with Incomplete Interaction. Proc. of the Soc. of Exp. Stress Analysis, pp. 75-92.

Nie, J. & Cai, C. S. 2003. Steel-Concrete Composite Beams Considering Shear Slip Effects. Journal of Structural Engineering 129: 495-506.

Porter, D. M., Rockey, K. C. & Evans, H. R. 1975. The Collapse Behaviour of Plate Girders Loaded in Shear. The Structural Engineer 53: 313-325.

Porter, D. M. & Cherif, Z. E. A. 1987. Ultimate Shear Strength of Thin Webbed Steel and Concrete Composite Girders. Proc. of International Conference of Steel and Aluminium Structures, pp. 55-64.

Ridza, M. H. 2008. Finite Element Formulation for Free Vibration of Composite Beams. Masters Diss., Universiti Teknologi Malaysia.

Shanmugam, N. E. & Baskar, K. 2003. Steel–Concrete Composite Plate Girders Subject to Shear Loading. Journal of Structural Engineering 129: 1230-1242.

Shanmugam, N. E. & Baskar, K. 2006. Design of Composite Plate Girders Under Shear Loading. Steel and Composite Structures 6(1): 1-14.

Shanmugam, N. E., Basher, M. A. & Khalim, A. R. 2009. Ultimate Load Behaviour of Horizontally Curved Composite Plate Girders. Steel and Composite Structures 9(4): 325-348.

Timoshenko, S. P. & Gere, J. M. 1961. Theory of Elastic Stability. McGraw-Hill Book Company.

Mohd Yazmil Md Yatim*
Nandivaram E. Shanmugam
Wan Hamidon Wan Badaruzzaman

Structures, Materials & Construction Research Group
Department of Civil & Structural Engineering
Faculty of Engineering & Built Environment
Universiti Kebangsaan Malaysia
43600 UKM Bangi, Selangor, Malaysia

*Corresponding e-mail: mymy@eng.ukm.my