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Design of composite plate girders under shear loading

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Abstract. Experiments have been carried out on six composite and two plain steel plate girders under shear loading to understand the elastic and inelastic behaviour of such girders. The failure mechanism assumed and used to develop design equations is normally based on the failure patterns observed in the experiments. Therefore, different types of cracks and failure patterns observed in the experiments are reviewed briefly first. Based on the observed failure patterns, a design method to predict the ultimate shear capacity of composite plate girders is proposed in this paper. The values of ultimate shear capacity obtained using the proposed design method are compared with the corresponding experimental values and it is found that the proposed method is able to predict the shear capacity accurately.

Key words: plate girders; steel-concrete composite; tension field action; web buckling; shear buckling.

1. Introduction

It is found from a detailed review (Baskar 2003) that plain steel plate girders subject to shear loading or to the combined action of shear and bending have been investigated extensively and design equations proposed. Behaviour of plain steel girders under fatigue loading is still at a research level because of problems such as long period cyclic loading, fracture failure of flange and web plates etc. In the modern construction steel-concrete composite construction has proved as an economical and efficient alternative. Though there are a number of research works reported on plate girders, only a few address the issue of steel-concrete composite plate girders.

Porter and Cherif (1987) carried out an experimental investigation on composite plate girders, with and without corner stiffeners in the web panels. Two series of experiments were carried out on simply supported quarter-scale model composite plate girders. A constant web-depth/thickness (d/t) ratio of 300 and cement mortar slab were used in this study. These girders were tested primarily under shear

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loading. Based on the experimental results, they proposed design equations in which the shear strength of the slab is added directly to the ultimate shear capacity of a simply supported bare steel plate girder. No comments were made on the variations of tension field action in web panels due to composite action with concrete slab. Composite plate girders having a constant web *d/t* ratio of 169 and containing web openings were tested to failure by Narayanan *et al.* (1989) in order to determine the effect of web openings on shear capacity of composite plate girders. The slabs in the girders were represented by micro concrete slab with wire mesh as reinforcement. Different sizes of web openings were used. A set of equations for predicting the ultimate load carrying capacity of the composite plate girders with rectangular web opening was proposed. In this case also the effect of composite action on the tension field action in web plate is not addressed.

Composite plate girders containing web openings have been studied further by Roberts and Al-Amery (1991). It was shown from the experimental observations that the tensile or pullout capacity of the connectors is primarily responsible for sustaining the composite action under predominantly shear loading. Three different web *d/t* ratios viz. 169, 240 and 300 were considered with various sizes of cut-outs. Conventional welded stud shear connectors were used initially to connect the concrete slabs with the top flange of the plate girders. Experimental investigation was also carried out on small-scale composite plate girders in which the slabs were made with micro-concrete and the reinforcements were made with wire mesh.

The reported works on composite plate girders are limited to shear loading except the one by Allison *et al.* (1982). Experimental investigations on one steel and five composite girders were carried out by Allison *et al.*, to assess the interaction between tension field action in web panels and the shear connection to the slab. All girders were tested under the action of combined shear and negative bending with the concrete slab under tension. The girders were designed with a constant web-depth to thickness (*d/t*) ratio of 130, and a single moment to shear ratio was used. It is concluded from the study that there is no variation in the tension field action in the web plate due to composite action of the deck slab. Based on the experimental observations and the solutions provided by the Cardiff model (Porter *et al.* 1975), they proposed an iterative design method.

Research findings available in the literature do not address variations in tension filed action of web plates due to composite action with deck slab. Also, it is noted that the earlier studies were carried out on small-scale models in which the slabs were made with micro-concrete slab/cement-mortar slab and wire mesh as reinforcement. Further studies on more realistic girders would enhance the understanding of the elastic and inelastic behaviour of steel-concrete composite plate girders.

Therefore, investigations have been carried out by the authors to examine the behaviour of composite plate girders under shear loading (Shanmugam and Baskar 2003) and, combined shear and bending loading (Baskar and Shanmugam 2003). A finite element model has also been proposed (Baskar *et al.* 2002) to investigate the steel-concrete composite plate girders with a main objective of examining the variations of tension field action in web plate due to composite action with deck slab. A brief discussion on the experimental investigation carried out by the authors is presented herein to highlight the parameters considered in the study. Based on the experimental observations, a design method is proposed to predict the ultimate load carrying capacity of steel-concrete composite plate girders under shear loading.

2. Experimental investigation

Two bare steel plate girders (SPG1, SPG2) and six composite plate girders (CPG1 to CPG6) were

Table 1 Details of the test specimens

Specimen	d/t ratio	Test span(m)	Flanges (mm)		M/M_p	V/V_{ult}	M/V	Expected mode of failure	Slab position
			Width	Thick					
SPG1	250	2.4	200	20	0.33	1.0	1.2	Shear	No slab
SPG2	150	2.4	260	20	0.44	1.0	1.2		
CPG1	250	2.4	200	20	0.33	1.0	1.2	Shear	Under compression
CPG2	150	2.4	260	20	0.44	1.0	1.2		
CPG3	250	2.4	200	20	0.33	1.0	1.2	Shear with	Under compression
CPG4	150	2.4	260	20	0.44	1.0	1.2	bending	Slab with shear link
CPG5	250	2.4	200	20	0.33	1.0	1.2	Shear	Under tension
CPG6	150	2.4	260	20	0.44	1.0	1.2		

tested to failure. The basic dimensions of all girders were kept the same so that the girders have a constant panel aspect ratio and span. Specimens SPG1 and SPG2 were designed as reference girders for web-depth to thickness (d/t) ratios of 250 and 150, respectively. The composite girders CPG1, CPG3 and CPG5 were made by adding necessary shear studs, concrete deck slab and shear links to the steel girders having the same dimensions as SPG1. Similarly, the composite girders CPG2, CPG4 and CPG6 were made by simply adding the necessary shear studs, deck slab and shear links to the girders of same dimensions as those of SPG2. The flanges were chosen as a plastic section in order to prevent local buckling, and also the cross section of flange plates was designed such that the girder would not fail in lateral torsional buckling (LTB) mode. The ratio between buckling moment capacity (M_b) and actual moment (M) due to applied load was kept as 2.38 and 1.80 for girders with d/t ratio of 250 and 150, respectively. These values were chosen based on results from a number of finite element analyses on steel girder trial sections in order to prevent lateral torsional buckling and overturning of the girders. End and load bearing stiffeners and end posts were designed according to BS5950: Part1.

In all the composite girders the bond between steel girder and deck slab was achieved by means of shear studs, designed with full shear connection theory as per BS5950: Part 3: Section 3.1: 1990. 19 mm dia, 100 mm long headed shear studs were welded in two rows at an interval of 155 mm centre to centre along the span length. The spacing of shear studs was kept constant for all the composite girders. Typical details of the test specimens are given in Table 1. All girders, simply supported at the ends and subjected to central concentrated load, were tested to failure. Extensive measurements of strain and displacement were made in order to capture the details of elastic and inelastic behavior of the girders. Ultimate failure loads and failure modes were noted for each of the girders.

3. Design method

Details of the experiments, observed failure patterns and the results obtained have been reported earlier by Shanmugam and Baskar (2003). Three different failure mechanisms in the concrete slab at different stages of loading were observed during the tests. At the initial stages of loading and up to the elastic buckling of web, the composite girders acted like composite beams with hot rolled sections. In the slab flexural hairline cracks were found near the load point and over the two end supports as shown in Figs. 1 and 2. After the initial hairline cracks in the slab, the girders continued to resist loading

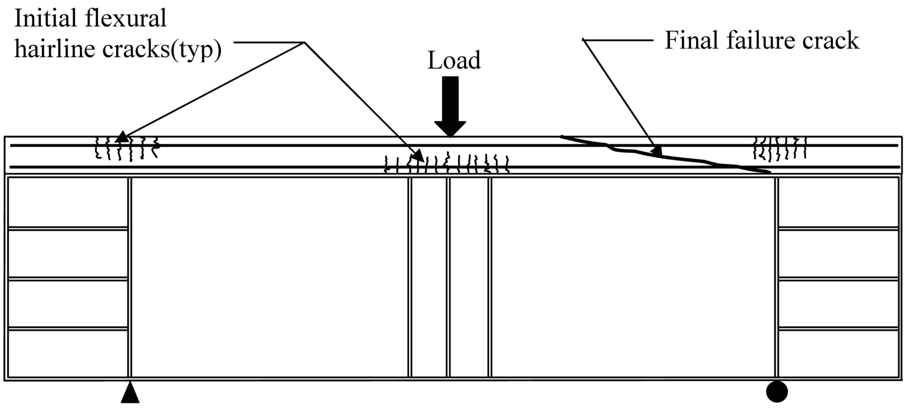

Fig. 1 Typical crack pattern observed in girder CPG1

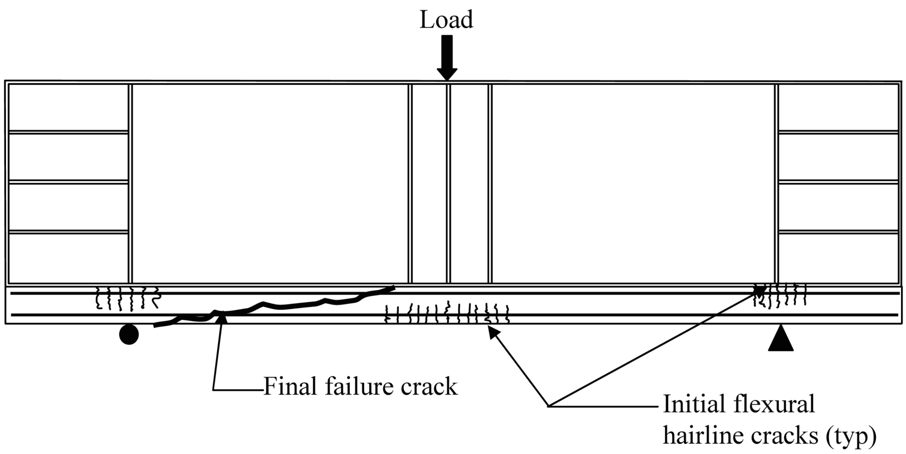

Fig. 2 Typical crack pattern observed in girder CPG5

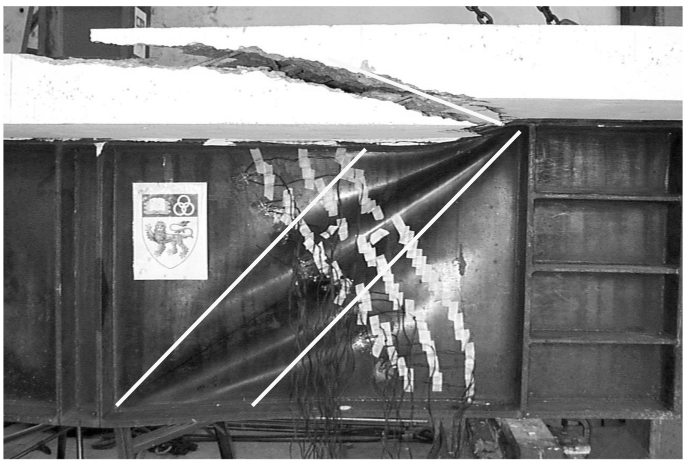

Fig. 3 Failure pattern observed during ultimate load condition [CPG1]

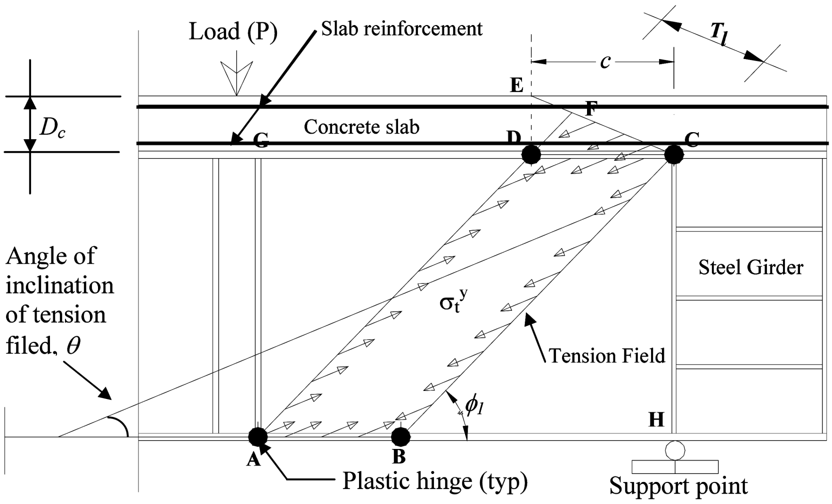

Fig. 4 Assumed failure mechanism for composite plate girders under shear loading

without any noticeable change in the stiffness of the girder. The second mode of cracks that were observed in the girders under shear loading was a cone shaped pull out failure at the girder ends. These cracks affected the stiffness of the composite girder to a certain extent but allowed the girder to continue to resist further load. At the ultimate condition the concrete slab was subjected to sudden failure in the form of split tensile failure. Based on the observations as shown in Figs. 1-3, a failure mechanism is assumed as shown in Fig. 4 to calculate the ultimate shear carrying capacity.

3.1. Assumptions

The following assumptions are made along with the assumed failure mechanism.

1. The ultimate shear carrying capacity of steel-concrete composite plate girders (V_{ult}) may be obtained as the sum of the shear carrying capacity of bare steel plate girder (V_g), contribution of concrete slab to the shear carrying capacity (V_s), and the contribution of link reinforcement, if any, to the shear carrying capacity (V_{rbar}).

$$V_{ult} = V_g + V_s + V_{rbar} \tag{1}$$

2. Even though an increase in the width of tension field is reported (Shanmugam and Baskar 2003) in the webs of composite plate girders, it is noted that final sway mechanism in the steel part of the girders is similar to the mechanism proposed for steel plate girders subjected to shear loading. Therefore, it is assumed that the tension field theory, to determine the ultimate load/shear carrying capacity of steel plate girders (Porter *et al.* 1975), is applicable to the shear capacity of the steel part of the composite plate girders.
3. The diagonal tension field in the web of steel-concrete composite plate girder is partly anchored to the concrete slab through composite action and therefore, load/shear carrying capacity is enhanced.

4. The extent of tension field anchored/shared by the slab depends on the plastic hinge location, angle of inclination of the tension field, tensile strength of concrete, and the shear strength of concrete slab.

Values of V_g, V_s and V_{rbar} in Eq. (1) are calculated as follows:

3.2. Shear capacity of the steel part of the composite girder - V_g

Extensive research works have been carried out and design equations established for steel plate girders. It is found that the Cardiff model (Porter *et al.* 1975) is capable of predicting the ultimate load capacity of steel plate girders accurately. It has been established, based on several parametric studies, that the model could cover a wide range of geometries and loading conditions. Therefore, the Cardiff model is assumed in the present study to predict the shear capacity (V_g) of the steel part of the composite girders. V_g can be determined as

$$V_g = \tau_{cr}dt + \sigma_t^y t\ \sin^2\theta(d\cot\theta - b) + 4dt\sin\theta\sqrt{(\sigma_{yw}M_p^*\sigma_t^y)} \tag{2}$$

In which

τ_{cr} : Elastic buckling strength of web plate
d : Depth of web plate
t : Thickness of web plate
σ_t^y : Membrane stress acting over the tension band
θ : Angle of inclination of the tension field
b : Panel width of web plate
σ_{yw} : Yield strength of web plate
M_p^* : Flange parameters

Details of the method may be found elsewhere (Porter *et al.* 1975).

3.3. Contribution of concrete slab to the shear capacity - V_s

The contribution by concrete slab to shear capacity of the girder is determined based on the assumed failure mechanism shown in Fig. 4. Composite action increases the load carrying capacity of the girder due to additional anchorage to the tension field. The changes in the girder parameters such as location of plastic hinges and angle of inclination of tension field are accounted for in the proposed model. The contribution by the slab to shear carrying capacity of steel-concrete composite plate girder is given by

$$V_s = b_c \times T_l \times f_{ta} \tag{3}$$

in which,

b_c : effective width of the slab.
T_l : anchor length (Fig. 4)
f_{ta} : allowable split tensile stress of concrete

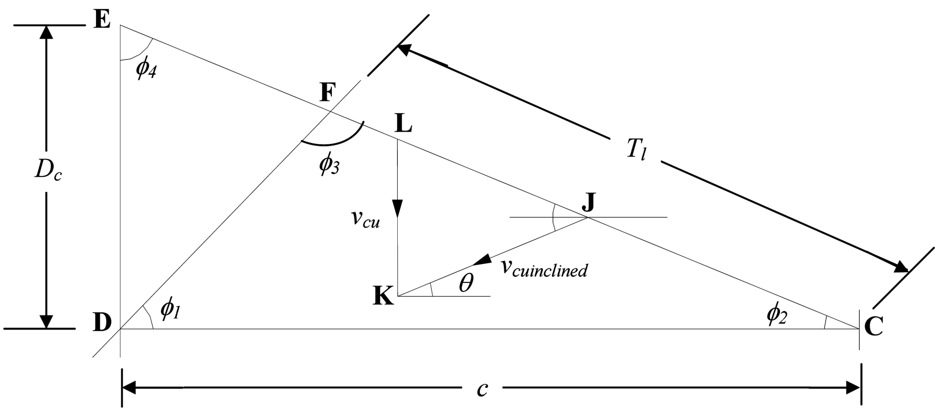

Fig. 5 Magnified view of the portion CDEF shown in Fig. 4

In the above equation, value of 'b_c' may be taken as the effective width of concrete slab determined based on BS5950: part3: 1990. For an isolated composite girder such as the test specimen, the whole width of the slab may be assumed to be effective especially under shear loading. The value of anchor length, T_l, and the allowable split tensile stress, f_{ta}, are calculated as follows:

3.3.1. Anchor length (T_l)

The anchor length (T_l) is determined from the assumed failure mechanism shown in Fig. 4. In the figure, c is the distance between the plastic hinges, θ angle of inclination of the membrane tension field and ϕ_1 angle of inclination of the assumed sway mechanism.

It is assumed that the anchor plane lies along the line CE shown in Fig. 4 and inclined at an angle of ϕ_2 with respect to the connecting steel flange (assumed horizontal). The magnified view of the portion CDEF is shown in Fig. 5.

The node E in the concrete slab lies on a vertical line through node D. The anchor plane is assumed to lie in the plane connecting the nodes C and E. Even though the anchor plane is between the nodes C and E, the effective anchor length, T_l, is taken as the length between the nodes C and F as shown in Figs. 4 and 5 in order to account for the effectiveness of tension field. Angle ϕ_2, dependant on the position of plastic hinges and depth of concrete slab, is calculated as

$$\phi_2 = \tan^{-1}\frac{D_c}{c}$$

Anchor length, T_l is given as $T_l = \dfrac{c}{\sin\phi_3} \times \sin\phi_1$

Where,

$$\phi_3 = \angle CFD = 180 - (\angle FDC + \angle DCF) = 180 - (\phi_1 + \phi_2)$$

3.3.2. Allowable split tensile stress (f_{ta}) along the anchor plane

The allowable split tensile stress, f_{ta}, along the anchor plane is dependant on the angle of inclination of the anchor plane, angle of inclination of tension field and the maximum allowable shear and tensile strength of concrete. Let us consider ΔLJK shown in Fig. 5. The node L is taken as an arbitrary point along the anchor plane and assumed that the vertical shear stress, v_c, is acting through the line LK. The components of the shear stress acting in the direction of anchor plane and in the directions of tension field are represented by the lines LJ and JK. The vertical shear stress, v_c, and the component of vertical shear stress acting in the direction of tension field, $v_{cinclined}$, are acting from the initial to the final stages of loading along the anchor plane as shown in Fig. 5. At the ultimate load condition, it is assumed that the shear strength of concrete will reach to its maximum value (v_{cu}) and therefore, its inclined component is $v_{cuinclined}$.

The component of shear stress, $v_{cuinclined}$, along the anchor plane is assumed as a pre-existing tensile stress over the plane and acting in the direction of tension field. This value of the pre-existing tensile stress is deducted from the measured ultimate split tensile stress of concrete, f_{tu}, in order to determine the allowable split tensile stress, f_{ta}, along the anchor plane:

$$\therefore \text{ The allowable split tensile stress of concrete, } f_{ta} = f_{tu} - v_{cuinlined} \tag{5}$$

Considering the ΔLJK in Fig. 5, the value of $v_{cuinclined}$ is given as

$$v_{cuinlined} = \frac{v_{cu}}{\sin(\theta + \phi_2)} \times \sin(\phi_4)$$

In the above equations the value v_{cu} is the maximum vertical shear stress that can be resisted by concrete. Codes and researchers recommend different values of 'v_{cu}' for an unreinforced and minimum reinforced concrete sections. Narayanan *et al.* (1989), related this value 'v_{cu}' approximately to the concrete cube strength, f_{cu}, as

$$v_{cu} = 0.3\sqrt{f_{cu}} \tag{6}$$

Value of v_{cu} given by Eq. (6) is considered in the present study to determine the contribution of concrete slab on shear carrying capacity of composite plate girders.

3.4. Contribution of link reinforcements to the shear capacity - V_{rbar}

When the concrete slab in a composite plate girder is provided with additional link reinforcements, the contribution by such reinforcements (V_{rbar}) to the shear capacity of the composite girder has to be accounted for in predicting the ultimate shear capacity as shown in Eq. (1).

Link reinforcements could be made using 6 mm diameter mild steel bars bent to a shape such that they are perpendicular to the crack directions observed in girders CPG1 and CPG2. In the experiments (Shanmugam and Baskar 2003, Baskar and Shanmugam 2003) the link reinforcements were provided as per BS8110: Part 1:1997 at a spacing of 0.75 times the effective depth of the concrete slab. Therefore, it can be assumed that the link reinforcements are subjected to direct tensile stress due to the anchor force of tension field. The force taken by the link reinforcements (V_{rbar}) is a function of the number of link reinforcements provided over the anchor plane, area of cross-section of link reinforcements and the stress developed in the link reinforcement during the failure of concrete slab. The value, V_{rbar}, may thus be calculated as

$$V_{rbar} = N.A_{rbar}.\sigma_{rbar} \tag{7}$$

In which,

N : number of link reinforcements crossing the anchor plane
A_{rbar} : cross-sectional area of a link reinforcement
σ_{rbar} : stress developed in the link reinforcement during failure of concrete slab

3.5. Numerical example for a composite girder

A typical design calculation to determine the ultimate shear capacity is presented herein for the girder CPG1. Distance between the plastic hinges, '*c*', the angle of inclination of tension field and the shear capacity of the steel part of the composite girder are calculated using the tension field theory (Porter *et al.* 1975).

3.5.1. Shear carrying capacity of steel part of the composite girder CPG1 - V_g

Ultimate shear carrying capacity of a plate girder web using the tension field theory is calculated using Eq. (2).

In Eq. (2) $\tau_{cr}dt$ is the component of shear carrying capacity due to the elastic shear buckling of web and is identified as $vs1$ in the text. The second term in the equation, $\sigma_t^y t\sin^2\theta(d\cot\theta - b)$ is the component of shear carrying capacity due to tension field action in the web panel after the elastic buckling of web plate and is identified in the text as $vs2$. The third term, $4dt\sin\theta\sqrt{(\sigma_{yw} M_p^* \sigma_t^y)}$ is the component of shear carrying capacity due to yielding of flanges and is identified in the text as $vs3$. V_g is obtained as the sum of $vs1$, $vs2$ and $vs3$ the calculations of which are presented below:

Data corresponding to the girder CPG1

Thickness of web plate	t_w	= 3 mm
Clear depth of web panel between flanges	d	= 750 mm
Panel aspect ratio	b/d	= 1.5
Width of flanges (Both top and bottom)	b_f	= 200 mm
Thickness of flanges (Both top and bottom)	t_f	= 20 mm
Yield strength of flange	σ_{yf}	= 272 N/mm^2
Yield strength of web plate	σ_{yw}	= 286 N/mm^2

<u>*Calculation of vs1:*</u>

The elastic buckling stress of web panel, $\tau_{cr,}$ for a simply supported rectangular plate is calculated as

$$\tau_{cr} = K\left[\frac{\pi^2 E}{12(1-\mu^2)}\right]\left(\frac{t_w}{d}\right)^2$$

where

$$K = 5.35 + 4\left(\frac{d}{b}\right)^2 \qquad \frac{b}{d} \geq 1.0$$

$$K = 5.35\left(\frac{d}{b}\right)^2 + 4 \qquad \frac{b}{d} \leq 1.0$$

panel aspect ratio of girder CPG1, $b/d = 1.5 > 1$

Therefore $$K = 5.35 + 4\left(\frac{750}{1125}\right)^2 = 7.127$$

$$\tau_{cr} = 7.127\left[\frac{\pi^2 \times 205000}{12(1-0.3^2)}\right]\left(\frac{3}{750}\right)^2 = 21.13\,\text{N/mm}^2$$

$$vs1 = 21.130 \times 750 \times 3 \times 10^{-3} = 47.5\ \text{kN}$$

<u>*Calculation of vs2:*</u>

$$vs2 = \sigma_t^y t \sin^2\theta (d\cot\theta - b)$$

$$\sigma_t^y = \left(\sqrt{\left(1-\left(\frac{\tau_{cr}}{\tau_{yw}}\right)^2\left(1-\frac{3}{4}\sin^2 2\theta\right)\right)} - \frac{\sqrt{3}}{2}\frac{\tau_{cr}}{\tau_{yw}}\sin 2\theta\right)\sigma_{yw}$$

where $$\tau_{yw} = \sigma_{yw}/\sqrt{3} = 286/\sqrt{3} = 165.12\,\text{N/mm}^2$$

$$\theta = \frac{2}{3}\tan^{-1}\left(\frac{d_w}{b}\right) = \frac{2}{3}\tan^{-1}\left(\frac{750}{1125}\right) = 22.46^{\circ}$$

therefore,

$$\sigma_t^y = \left(\sqrt{\left(1-\left(\frac{21.13}{165.12}\right)^2\left(1-\frac{3}{4}\sin^2 2(22.46)\right)\right)} - \frac{\sqrt{3}}{2}\frac{21.13}{165.12}\sin 2(22.46)\right)286$$

$$= 325.4\ \text{N/mm}^2$$

$$vs2 = 325.4 \times 3 \times \sin^2(22.46) \times ((750 \times \cot(22.46)) - 1125) \times 10^{-3} = 98.2\ \text{kN}$$

<u>*Calculation of vs3:*</u>

$$vs3 = 4dt\sin\theta\sqrt{(\sigma_{yw} M_p^* \sigma_t^y)}$$

The flange strength parameter M_P^* in the above equation is calculated as

$$M_p^* = \frac{M_{pf}}{d^2 t \sigma_{yw}}$$

in which

M_{pf}, plastic moment capacity of the flange $= 0.25\ b_f\ t_f^2 \sigma_{yf}$

$$= 0.25 \times 200 \times 20^2 \times 272 = 5440000$$

$$\therefore M_p^* = \frac{5440000}{750^2 \times 3 \times 286} = 0.01127$$

The value of M_p^* must be compared with $M_{p\,lim\,it}^*$ for larger flange if any. The value of $M_{p\,lim\,it}^*$ is given as

$$M^*_{p,\,lim\,it} = \frac{1}{8}\left(\frac{b}{d}\right)^2\left[\sqrt{\left(1-\frac{1}{4}\left(\frac{\tau_{cr}}{\tau_{yw}}\right)^2\right)} - \frac{\sqrt{3}}{2}\left(\frac{\tau_{cr}}{\tau_{yw}}\right)\right]$$

i.e.,
$$M^*_{p,\,lim\,it} = \frac{1}{8}\left(\frac{1125}{750}\right)^2\left[\sqrt{\left(1-\frac{1}{4}\left(\frac{21.13}{165.12}\right)^2\right)} - \frac{\sqrt{3}}{2}\left(\frac{21.13}{165.12}\right)\right] = 0.2495$$

the value M_p^* is less than $M_{p,\,lim\,it}^*$. Therefore ultimate shear carrying capacity is given as.

$$vs3 = 4 \times 750 \times 3 \times \sin 22.46 \times (\sqrt{286 \times 0.01127 \times 325.39}) \times 10^{-3} = 111.4 \text{ kN}$$

the total shear carrying capacity of the steel girder V_g is calculated as

$$\begin{aligned} V_g &= vs1 + vs2 + vs3 \\ &= 47.5 + 98.2 + 111.4 \\ &= 257.1 \text{ kN} \end{aligned}$$

The distance between the plastic hinges, c, is calculated as

$$c = \frac{2}{\sin\theta}\sqrt{\left(\frac{M_{pf}}{\sigma_t^y t}\right)} = \frac{2}{\sin 22.46}\sqrt{\left(\frac{5440000}{325.39 \times 3}\right)} = 390.80 \text{ mm}$$

For the composite girder CPG1:

Cube strength of concrete, $f_{cu} = 40.2$ N/mm^2

Split tensile strength of concrete, $f_{tu} = 3.1$ N/mm^2

Width of concrete slab, $b_c = 1000$ mm

Depth of Concrete slab, $D_c = 150$ mm

Maximum shear stress $v_{cu} = 0.3\sqrt{f_{cu}} = 0.3\sqrt{40.2} = 1.90 \text{N/mm}^2$

Allowable split tensile stress $f_{ta} = f_{tu} - v_{cuinclined} = 3.1 - \dfrac{v_{cu}}{\sin(\theta + \phi_2)}\sin(\phi_4)$

For the girder,
$\theta = 22.46$ degrees
$\phi_2 = 21.00$ degrees
$\phi_4 = 69.0$ degrees

Therefore, the allowable split tensile stress, $f_{ta} = 0.518$ N/mm^2

Anchor length $T_l = \dfrac{390.8}{\sin(113.4)} \times \sin(45.60) = 304.24$ mm

The contribution of concrete slab to the shear capacity of composite girder is given as

$$V_s = f_{ta}\, b_c\, T_l = (0.518)(1000)(304.24) \times 10^{-3} = 157.7 \text{ kN}$$

Since there was no link reinforcement provided in the CPG1, the term V_{rbar} in Eq. (1) is neglected. The total shear carrying capacity of the composite plate girder CPG1 is, therefore,

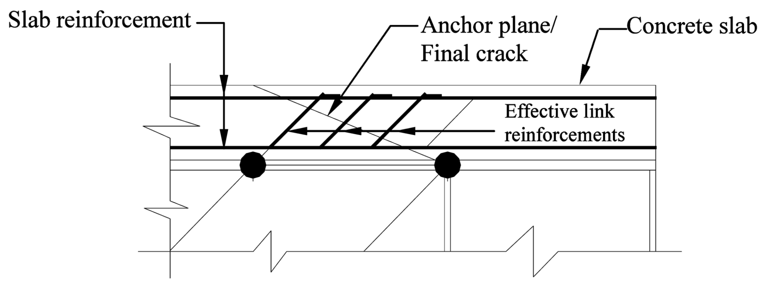

Fig. 6 Effective link reinforcements along the anchor plane

$$V_{ult} = V_g + V_s = 257.1 + 157.7 = 414.8 \text{ kN}$$

Similar calculations have been made to determine the ultimate shear capacity of composite girder CPG2. In the case of girders CPG3 and CPG4, the additional term, V_{rbar}, has been included to incorporate the force taken by the link reinforcements. A typical calculation made for CPG3 is as follows.

3.5.2. Calculation of force taken by the link reinforcements

The girder CPG3 is similar to CPG1 except that link reinforcements were provided in the concrete slab of CPG3. The contribution from the link reinforcement is calculated as follows and added to the shear strength of CPG3.

Force taken by the link reinforcements, $V_{rbar} = N.A_{rbar}.\sigma_{rbar}$

There are 11 link reinforcements along the width of the girder and they are placed at a spacing of 75 mm along the span of the girder. It was observed in the experiment that the split tensile crack crossed three rows of link reinforcements along the span as shown in Fig. 6. Therefore, total number of link reinforcement, N, sharing the anchor force of tension field is found to be 33 Nos.

The ultimate tensile strain, ε_t, in concrete due to split tensile stress is calculated by using the Young's modulus of concrete, E_c, equal to 26900 MPa, determined from the control tests.

The strain corresponding to the maximum split tensile stress $= \dfrac{stress}{E_c} = \dfrac{3.7}{26900} = 137.546 \times 10^{-6}$

Table 2 Ultimate shear capacity of composite plate girders

Girder	f_{cu} N/mm^2	f_{tu} N/mm^2	v_{cu} N/mm^2	f_{ta} N/mm^2	c mm	T_l mm	V_s kN	V_g kN	V_{rbar} kN	$V_{ultpred}$ kN	$V_{ultexpt}$ kN	$\dfrac{V_{ultpred}}{V_{ult\,expt}}$
SPG1	-	-	-	-	390.8	-	-	252.5	-	252.5	244.0	1.03
SPG2	-	-	-	-	406.1	-	-	395.0	-	395.0	402.5	0.98
CPG1	40.2	3.1	1.90	0.518	390.8	304.3	157.7	257.1	-	414.8	430.5	0.96
CPG2	41.9	3.2	1.94	0.515	406.1	319.7	164.8	413.0	-	577.8	562.0	1.03
CPG3	45.9	3.7	2.03	0.94	390.8	304.3	286.4	248.4	26.3	561.1	542.5	1.03
CPG4	45.0	3.6	2.01	0.818	406.1	319.7	261.5	401.3	26.3	689.1	675.0	1.02
CPG5	49.5	3.4	2.11	0.535	409.1	322.8	172.7	244.0	-	416.7	410.5	1.01
CPG6	41.7	3.1	1.94	0.422	392.5	306.0	129.1	410.0	-	539.1	555.5	0.97

Since the strain in the link reinforcements is the same as that in concrete, the stress developed in the link reinforcements at failure is calculated as

$$\sigma_{rbar} = 137.546 \times 10^{-6} \times 205000 = 28.2 \text{ N/mm}^2$$

Total tensile force taken by the link reinforcements, V_{rbar} is given as

$$V_{rbar} = 33 \times \left(\frac{\pi \times 6^2}{4}\right) 28.2 \times 10^{-3} = 26.3 \text{ kN}$$

The values of V_g, V_s and V_{rbar} and hence that of V_{ult} are thus calculated for the composite plate girders tested in shear. The results obtained are summarised in Table 2 along with the comparison between the predicted values and the experimental results. It is noted from the comparison that the predicted and experimental values for ultimate shear capacity are close to each other, the maximum deviation ranging from +3% to -4%. The mean value of the ratio between the two values is 0.99 and the standard deviation is 0.03. It is, therefore, concluded that the proposed method is capable of predicting the ultimate shear capacity with sufficient accuracy.

4. Conclusions

Experiments have been carried out on steel-concrete composite plate girders under shear loading to understand the elastic and inelastic behaviour of such girders. Different types of cracks and failure patterns observed in the experiments are discussed briefly. Based on the experimentally observed failure patterns, a design method is proposed to predict the ultimate load carrying capacity of steel-concrete composite plate girders under shear loading. From the comparison of the predicted values with the corresponding experimental values it is concluded that the proposed method is able to predict the shear capacity with sufficient accuracy.

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