EFFECTS OF STRAIN REGIMES ON THE BEHAVIOUR OF HEADED STUD SHEAR CONNECTORS FOR COMPOSITE STEEL-CONCRETE BEAMS

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ABSTRACT: In composite steel-concrete beam construction, one of the most common methods to evaluate shear connector strength and behaviour is through a push test. Push tests have been used as early as the 1960's to predict the strength and behaviour of shear studs in solid slabs. The performance of steel-concrete composite structures is greatly dependent on the load-slip characteristics of shear connectors. Significant research work has been performed on composite beams with regard to their stiffness and ductility of the shear connectors for both solid and profiled slabs. This paper describes the strength and ductility of shear connectors in composite beams with both solid and profiled steel sheeting slabs when different strain regimes are imposed on the concrete element. An accurate non-linear finite element model using ABAQUS is developed herein to study the behaviour of shear connectors for both solid and profiled steel sheeting slabs. The reason for employing different strain regimes in composite steel-concrete beams is to properly simulate the behaviour of shear connectors in composite beams where trapezoidal slabs are used. The pertinent results obtained from the finite element analysis were verified against independent experimental results and existing design standards. Based on the finite element analysis and the experimental results, it is evident that the strength and the load-slip behaviour of composite steel-concrete beams are greatly influenced by the strain regimes existent in the concrete element.

Keywords: Composite steel-concrete beams; strain regimes; finite element method; shear connectors

1. INTRODUCTION

The push test is meant to determine the basic strength of headed stud shear connectors in composite steel-concrete beams. It is always considered reliable as a lower bound method for the prediction of strength and stiffness in real composite steel-concrete beams. The stress state in and around the headed shear studs are highly complex. Metal decking used to form composite slabs has also used standard push tests to assess strength and ductility of headed stud shear connectors. Once again this has proven a reliable method for sheeting with dove tailed and re-entrant profiles. The introduction of trapezoidal profiles of late has caused some concern about the methodology in which to assess the headed stud strength and ductility. The purpose of this paper is to attempt to address some of the issues associated with the shear stud behaviour consisting of trapezoidal profiles.

The strength and ductility of headed stud shear connectors of steel and concrete composite beams is normally determined from a standard push test as outlined by Eurocode 4. Figure 1 illustrates push test for solid and profiled slabs. The objective of the analysis was to carry out numerical push testing of similar sized specimens to the conventional Eurocode 4, British Standards Institution [1] push test. Back-breaking failures have been shown to be suppressed in numerical analyses and thus it is not considered necessary to model four ribs explicitly. When a push test is carried out, the shear connectors are subjected to pure shear, however in comparison, when a composite steel-concrete beam is subjected to bending, the shear connectors will be subjected to both bending and shear.

Figure 1. Push Tests for Both Solid and Profiled Slab

Even though headed stud shear connectors can resist a complex combination of shear, bending and axial force, the magnitudes of these forces change during a push test. Therefore the behaviour of headed stud shear connectors also depends greatly on the geometry of the push test specimens and the strength of all the materials. In this study, the authors have used pure shear as a way of identifying the applied load to the specimens in order to address the real stress state of headed stud shear connectors that suppress secondary failure modes and thereby simulate the real situation in beams illustrated by Hicks [2].

For a simplified explanation, consider the push test with a horizontal axis of symmetry, shown in Figure 2. Profiled slab in Figure 2 has been modified to illustrate the realistic stresses and thus the effects of torque on the ribs are modelled and Figure 13 also highlights the complex stress states created by this modelling approach. A horizontal line of the cross-section at the interface of the concrete slab and structural steel beam will be referred to as the neutral axis of the composite steel-concrete beam. Next consider a typical element of the composite beam between two planes perpendicular to the beam's neutral axis. From an elevation, the element is denoted as $aabb$. When the element is subjected to loading in the web of the structural steel beam, shown in Figure 2a, it will be pushed as illustrated in Figure 2b. However section $aabb$ still remains undeformed. Therefore, the structure is in pure shear where critical loading conditions are applied to the shear connectors in composite steel-concrete beams. In this case, the strain will remain constant along the section, as shown in Figure 2c.

When a composite steel-concrete beam is subjected to bending moments, consider the beam with a vertical axis of symmetry given in Figure 3. The horizontal line of the cross-section is similar to the push test explained above. When the element is subjected to loading on the concrete slab shown in Figure 3a, the structure will bend as illustrated in Figure 3b, and then the section $ccdd$ deforms to become $c'c'd'd'$. Thus, the cross-section will be subjected to bending. For this case, the strain gradually changes throughout the depth of the section as illustrated in Figure 3c.

This paper considers the behaviour of strain regimes for the concrete element. As discussed above, when the structure is subjected to pure shear, the longitudinal strain is constant throughout the section, however when it is subjected to pure bending, the strain gradually changes throughout the depth of the section. A linear variation in strain is schematically shown in Figure 4, and the value of strain is represented using a parameter defined as α . When the structure is subjected to pure shear,

the strain distribution is represented by Figure 4a where $\alpha = 1$. For the case when the structure is subjected to pure bending, the strain distribution is represented by Figure 4e where $\alpha = -1$. Figs. 4b, 4c and 4e illustrate the different variation of strain which represents the different locations of the neutral axis.

In order to modify the strain regimes in a concrete element from pure shear to pure bending, the theory of elasticity of Hooke's law according to Popov and Balan [4] was used. Authors considered the cross-section of the composite steel-concrete beam is deformed by a parallel system of small forces, F_i in the axial direction as shown in Figure 2. A combination of the two concepts above will enable the axial force acting in every cross-section to be determined.

Figure 2. Behaviour of Composite Steel-Concrete Beams in Pure Shear

Figure 3. Behaviour of Composite Steel-Concrete Beams in Pure Bending

Figure 4. The Value of α and Location of Neutral Axis for the Concrete Element

2. MECHANICAL BEHAVIOUR OF THE CONSTITUENT MATERIALS

2.1 General

In general, constitutive laws are used to define the stress-strain characteristics of the material. The accuracy of the analysis is dependent on the constitutive laws used to define the mechanical behaviour. In materials such as concrete, structural steel and reinforcing steel, profiled steel sheeting and shear connectors, the constitutive laws are represented by the stress-strain relationships of the materials.

2.2 Concrete Properties

Plain concrete was recommended by Carreira and Chu [5], where the stress in compression is assumed to be linear with respect to strain up to a stress of $0.4f'_c$. Beyond this point, the stress is represented as a function of strain according to Eq. 1.

$$\sigma_c = \frac{f'_c \gamma (\epsilon_c / \epsilon'_c)}{\gamma - 1 + (\epsilon_c / \epsilon'_c)^\gamma} \quad (1)$$

where

$$\gamma = \left| \frac{f'_c}{32.4} \right|^3 + 1.55 \quad \text{and} \quad \epsilon'_c = 0.002$$

For concrete in tension, the tensile stress is assumed to increase linearly relative to strain until the concrete cracks. After the concrete cracks, the tensile stress decreases linearly to zero. The value of strain at zero stress is usually taken to be 10 times the strain at failure, which is illustrated in Figure 5.

2.3 Structural Steel, Reinforcing Steel, Shear Connectors and Profiled Steel Sheeting Properties

The stress-strain characteristics of reinforcing steel, shear connectors and profiled steel sheeting are essentially similar to structural steel. Their behaviour is initially elastic after which yielding and strain hardening develop. A piecewise linear approach was found to be sufficiently accurate to represent the stress-strain relationships. Moreover, these curves are utilised in the model when the stress-strain data is not available.

Figure 5. Stress-Strain Relationship for Concrete, Carreira and Chu [5]

According to Loh, et al. [6], the stress-strain relationship for structural steel can be represented as a simple elastic-plastic model with strain hardening. The mechanical behaviour for both compression and tension is assumed to be similar. Figure 6 represents the stress-strain relationship for steel and Table 1 indicates the different values of stress and strain for each material.

Table 1. Stress-Strain Value for Structural Steel Beam, Shear Connectors, Profiled Steel Sheeting and Steel Reinforcing

Element	σ_{us}	ϵ_{ps}	ϵ_{us}
Steel beam	$1.28\sigma_{ys}$	$10\epsilon_{ys}$	$30\epsilon_{ys}$
Steel Reinforcing	$1.28\sigma_{ys}$	$9\epsilon_{ys}$	$40\epsilon_{ys}$
Profiled Sheeting	-	$20\epsilon_{ys}$	-
Shear Connectors	-	$25\epsilon_{ys}$	-

Figure 6. Stress-Strain Relationship for Structural Steel, Loh, et al. [6]

3. PUSH TEST FINITE ELEMENT ANALYSIS

3.1 Experimental Investigation of the Push Tests.

Initially, there are two different experimental investigations which were compared with the finite element analysis developed herein. The push tests performed by Lam and El-Lobody [7] considered shear connection of both solid and profiled slabs. The authors modified Lam's solid slab to consider a profiled slab in order to evaluate the differences between the solid and profiled slabs. With the modified profiled slab, the authors only took into account the presence of the shear stud in the middle position of the shear connectors in the profiled sheeting. Mirza and Uy [8] stated that the strength of the shear connectors can be increased by 11 % when they are placed on the strong side when compared with the weak position. Conversely when the shear connectors were positioned on the weak side, the strength of the shear connectors displayed a reduction of 13 %. The reader should bare in mind that the results shown are only applicable to the range of parameters studied herein. Qualification needs to be made against the range of parameters such as the thickness of the steel sheeting, shape of trough sheeting, arrangement and number of headed stud shear connectors in the trough as stated by Johnson and Yuan [9] and Johnson and Yuan [10] Further numerical analysis of these parameters could prove useful in the future.

The authors also considered the experiments conducted by Hicks [2] for both the push and beam tests as a comparison with the finite element analysis for the profiled slabs. For these analyses, the authors used the shear studs that were placed in the favourable position as it was adopted in the experimental program.

Figure 7. Finite Element Mesh and Boundary Condition of Solid Slab Model, Lam and El-Lobody [7]

Figure 8. Finite Element Mesh and Boundary Condition of Profiled Steel Sheeting Slab Model, Lam and El-Lobody [7]

3.2 Parametric Studies for the Push Tests.

The authors also undertook parametric studies by considering different types of profiled steel sheeting and their effects on the shear connector behaviour when different strain regimes were considered. Three different types of profiled steel sheeting were considered, and they include G1, G2 and G3 profiled steel decks. The dimensions and profiled steel sheeting details are illustrated in Table 2.

3.3 Finite Element Type and Mesh

The finite element program ABAQUS, Karlsson and Sonrensen [11], Karlsson and Sonrensen [12] and Karlsson and Sonrensen [13] were used to quantify the behaviour of the shear connection in composite beams. Three dimensional solid elements were used to model the push off test specimens in order to achieve an accurate result from the finite element analysis. For both the concrete slab and the structural steel beam, a three-dimensional eight node element (C3D8R) was used to improve the rate of convergence. A three dimensional thirty-node quadratic brick element (C3D20R) for the shear connectors was chosen due to their ability to capture stress concentrations more effectively and better for modelling geometric features. Furthermore a four-node doubly curved thin shell element (S4R) for the profiled steel sheeting because it is the most appropriate type of element to model thin walled steel structures and lastly a two-node linear three dimensional truss element (T3D2) for the steel reinforcing was used because the axial direction is released using two node linear displacement equation provided in ABAQUS. The main reason was that the authors were not too concerned with the slip between the reinforcing bar and concrete.

For the experiments of Lam and El-Lobody [7], the finite element model was used to represent half of a stud of the push test, due to symmetrical conditions for both the solid and modified profiled slabs which are illustrated in Figs 7 and 8 respectively. Whilst Hicks [2] experiments, the finite element model used to represent half of the push test model is shown in Figure 9. The reason the authors used different models for Lam and El-Lobody [7] and Hicks [2] was to reduce the simulation cost.

Table 2. Dimensions and Profiled Steel Sheeting Details of Parametric Study

Group	Specimen	Dimensions							Strain Profiles α
		Profiled Sheeting			Stud		Slab		
		b_0 (mm)	h_p (mm)	t (mm)	d (mm)	h (mm)	B (mm)	D (mm)	
G1	G1-1	136	60.9	0.9	19	100	806	405	1
	G1-2	136	60.9	0.9	19	100	806	405	0.5
	G1-3	136	60.9	0.9	19	100	806	405	0
	G1-4	136	60.9	0.9	19	100	806	405	-0.5
	G1-5	136	60.9	0.9	19	100	806	405	-1
G2	G2-1	136	55	0.9	19	100	806	405	1
	G2-2	136	55	0.9	19	100	806	405	0.5
	G2-3	136	55	0.9	19	100	806	405	0
	G2-4	136	55	0.9	19	100	806	405	-0.5
	G2-5	136	55	0.9	19	100	806	405	-1
G3	G3-1	144	60.9	0.9	19	100	806	405	1
	G3-2	144	60.9	0.9	19	100	806	405	0.5
	G3-3	144	60.9	0.9	19	100	806	405	0
	G3-4	144	60.9	0.9	19	100	806	405	-0.5
	G3-5	144	60.9	0.9	19	100	806	405	-1

Figure 9. Finite Element Mesh and Boundary Condition of Profiled Steel Sheeting Slab Model, Hicks [2]

3.4 Boundary Conditions and Load Application

In Figs. 7 and 8, the nodes that lie on the other symmetrical surface (Surface 1) for concrete, shear connectors, structural steel beam, steel reinforcing and profiled steel sheeting are restricted from moving in the x -direction. All the nodes in the middle of the structural steel beam web, which are designated as Surface 2, are restricted to move in the y -direction. All the nodes of the concrete and the profiled steel sheeting, which are designated Surface 3, are restricted to move in the z -direction. Whilst in Figure 9, Surface 3 which consists of a concrete slab, structural steel beam, steel reinforcing and profiled steel sheeting are restricted to move in the z -direction.

For the application of load, a static concentrated load was applied to the centre of the web for Lam's model [5] and a uniformly distributed load was applied to the web surface of the structural steel for the Hicks' model [7]. The modified RIKS method was employed to the load in order for the load to be obtained through a series of iterations for each increment for a non-linear structure. This method was used to predict unstable and nonlinear collapse of the model. In order to achieve accurate results in ABAQUS, the RIKS method has the ability to use the arc length along the static equilibrium in load-displacement space. The initial increments will be adjusted if the finite element model fails to converge. Finally the value of deformation after each increment is computed automatically. The final result will either be the maximum value of the load or the maximum displacement value.

4. RESULTS AND DISCUSSION

4.1 Lam and El-Lobody's Experiments

For the purpose of this paper, finite element push tests were modelled to determine the load-slip behaviour of the shear connectors. The first series of analyses considered was compared with the experimental investigations undertaken by Lam and El-Lobody [7] for solid slabs. The authors then modified the solid slab to simulate a profiled slab to consider the differences in the results.

It can be observed in Figure 10 that the stiffness is similar for both the experiments and the finite element analyses within the elastic region. The experimental study showed that the maximum shear connector capacity was 118 kN whilst the finite element result obtained was 119 kN. This shows that the finite element model accurately analysed the experiments with a discrepancy of 0.7 %. When the model is subjected to bending where $\alpha = -1$, after the elastic region, the finite element analysis had a higher shear connector capacity when compared with the model subjected to pure shear where $\alpha = 1$.

From the finite element model, the authors have also shown that when the composite steel-concrete beam is subjected to pure shear, the structure is subjected to the most detrimental loading case. This can be verified from Figure 10 where the model is subjected to bending, the shear connector capacity increased by 2.2 % to 121 kN. Even though the loading behaviour of the push test changed, 2.2 % of increment in the shear connector capacity is not significant. In order to look at the different behaviour of the shear connectors when exposed to different loading conditions with varying neutral axes, Figure 10 established that the different strain regimes in solid slabs is not critical where increments in 0 %, 0.7 % and 1.3 % for $\alpha = 0.5$, $\alpha = 0$ and $\alpha = -0.5$ respectively. Therefore, it is illustrated that the push test to determine the shear connector capacity for composite steel-concrete solid slabs is reliable and accurate.

From Figure 10, it is observed that the failure mode changes when the value of α changes. When $\alpha = 1$, the failure mode is governed by shear stud failure. When $\alpha = 0.5, 0, -0.5$ and -1 , the failure mode is governed by concrete failure where the concrete starts to crack surrounding the shear connector due to the addition of load when the value of α changes. The failure modes can also be observed graphically in Figure 11.

Figure 12 illustrates that the stiffness is similar for the finite element analyses within the elastic region. The finite element analysis showed that the maximum shear connector capacity was 68 kN when $\alpha = 1$ for the push test under pure shear, however when the model is subjected to the bending condition where $\alpha = -1$, the maximum shear connector capacity was 92 kN. This shows a significant increment of 26.6 %. The result is expected when the push test changes from pure shear to pure bending due to the loading condition for the worst case scenario when compared with pure bending.

Figure 12 illustrates that the different strain regimes in the profiled slab are crucial where an increment in 6.8 %, 12.5 % and 18.5 % for $\alpha = 0.5, 0$ and -0.5 respectively. Therefore, it is illustrated that the push test to determine the shear connector capacity for composite steel-concrete profiled slabs is unreliable as it does not allow for the inclusion of the beneficial effects of bending.

From Figure 12, it can be seen that the failure mode changes when the value of α changes. When $\alpha = 1$ and 0.5 , the failure mode is governed by concrete failure where the concrete cracks through the middle of the trough. This failure mode observed is similar to experimental studies undertaken by El-Lobody and Young [14]. When $\alpha = 0, -0.5$ and -1 , the failure mode is dominated by stud failure where the shear connectors started to shear off the concrete, causing the concrete to crack surrounding the shear connector. The failure mode also can be observed in Figure 13.

Figure 10. Lam and El-Lobody [5] Experiment Compared with Different Strain Regimes

Figure 11. Stress Contours and Failure Modes for Solid Slabs

Figure 12. Lam and El-Lobody's Modified Push Test with Different Strain Regimes

4.2 Hicks' Experiments

The second experimental series considered was that associated with the experiments conducted by Hicks [2] for both push tests and beam tests. Figure 14 shows the comparison between the experimental tests undertaken by Hicks' and the finite element model performed by the authors. The results are as stated by Hicks [2], the initial similarity is the stiffness for both the push test and beam test. However, the maximum shear connector capacity and slip measured in the push test is well below the level attained in the beam test. Figure 14 verifies the accuracy of the finite element analysis. For the push tests, the experimental study showed a maximum shear capacity of 84 kN and the finite element analysis showed a value of 89 kN with a discrepancy of 5.7 %. As for the beam tests, the experimental study and finite element analysis illustrated that the maximum shear capacities were 124 kN and 128 kN respectively, with a discrepancy of 3.3 %.

Furthermore, Figure 14 shows that the maximum shear connector capacity for push tests was 89 kN, whilst for the beam tests, the maximum shear connector capacity was 128 kN. This illustrates an increment of 30.6 %. From the experimental studies and finite element analyses, failure in the push tests was governed by concrete failure at the trough rather than stud failure. When the simulated beam test is considered, the failure is dominated by stud failure. The behaviour of the shear connectors in the push test provides a conservative result and they do not replicate the strength and ductility that can be achieved in a beam test. Therefore, the standard push test to determine the shear connector capacity for a profiled slab is questionable.

Figure 13. Stress Contours and Failure Modes for Profiled Slabs

Figure 14. Comparison of Experimental Data and Finite Element Result for Hicks [2]

Figure 15. Dimensions of Profiled Steel Sheeting

4.3 Parametric Studies

4.3.1 General

The finite element analysis has allowed parametric studies to be carried out and the case that the authors solved has been modelled particularly in calibration with independent experimental test results undertaken by Hicks [2]. This may not be the case for real buildings. However, the trends in this paper may be of use for further studies. In this case, it has been shown that the finite element model correctly predicts the behaviour of shear connectors in composite steel-concrete structures for both solid and profiled slabs. For solid slabs, when the strain regimes were varied from pure shear to pure bending, the maximum capacity did not show any critical change, whilst for profiled slabs, any minor change was crucial. Hence, a parametric study was conducted to study the effects of the capacity and behaviour of shear connectors by changing the profiled steel sheeting geometries. A total of fifteen push tests were investigated in the parametric study. The push tests were divided into three major groups. These were G1, G2 and G3. Each group included five different push tests having different α values with the same type of profiled steel sheeting, as described in Figure 4. The dimensions of the profiled steel sheeting and details of the parametric study are explained in Table 2 and shown in Figure 15.

Figure 16 illustrates the force-slip relationship for profile G1, G2 and G3 profiled steel sheeting types. It can be seen that between the three profiled steel sheeting types, G1 profiled steel sheeting can withstand larger forces when compared with the other two sheeting types. Therefore, the G1 type has a higher strength capacity and is considerably more ductile. Figure 16 reveals a maximum shear connector capacity of 135 kN, 125 kN and 128 kN for G1, G2 and G3 profiled steel sheeting respectively. G1 illustrates that it has a shear connector capacity 4.8 % higher than that of the G3 trapezoidal profiled sheeting and 7.3 % higher than that of the G2 profiled steel sheeting. Figure 16 also shows that the shear strength capacity of composite steel-concrete profiled slabs is greatly dependent on the width and depth of the ribs of the profiled steel sheeting.

Figure 16. Comparison of Force Slip Relationship for Different Type of Profiled Steel Sheeting

4.3.2 G1 profiled

The first group of parametric studies, the authors varied the strain regimes from $\alpha = 1, 0.5, 0, -0.5$ and -1 for the G1 profiled steel sheeting. Figure 17 depicts the force-slip relationship of the G1 profiled steel sheeting with the variation of α value. It can be observed that the composite steel concrete beam increases the shear strength and ductility when the structure is varied from pure shear to pure bending. When the structure is subjected to pure shear, the maximum shear connector capacity of 98 kN is achieved. However, when it is subjected to pure bending, the maximum shear capacity increases to 135 kN, which is a 27.2 % shear capacity increment.

In order to add the α value into the force-slip relationship, the Aribert and Labib [15] force-slip relationship was used which is described in Eq. 2.

$$Q = Q_u (1 - e^{-c_1 d})^{c_2} \quad (2)$$

where

Q is the force of the shear connector

Q_u is the ultimate force

c_1 and c_2 are the parameters of the model

d is the slip capacity of the shear connector

Due to Q_u, c_1 and c_2 are the changing parameters in the equation, the relationship of these values with respect to α and is shown in Eq. 3. Each of the parameter values with respect to α can be obtained in Table 3 and plotted in Figures 18 to 20.

$$Q = f_1(\alpha) (1 - e^{-f_2(\alpha, d)})^{f_3(\alpha)} \quad (3)$$

Table 3. Parameter of Force Slip Relationship with Respect to α .

Profiled Type	$f_1(\alpha) = a\alpha^3 + b\alpha^2 + c\alpha + d$				$f_2(\alpha) = e\alpha^4 + f\alpha^3 + g\alpha^2 + h\alpha + j$					$f_3(\alpha) = k\alpha^4 + l\alpha^3 + m\alpha^2 + n\alpha + p$				
	a	b	c	d	e	f	g	h	j	k	l	m	n	p
G1			-17.94	117.2		-0.06	0.13	0.12	1.2	-0.07	-0.03	0.17	0.03	0.4
G2	13.30	-1.51	-28.60	111.5		0.15	0.15	0.35	0.96	0.02	-0.07	-0.03	0.07	0.4
G3			-18.36	109.74	0.67	0.2	0.27	0.3	1.05	-0.07	-0.03	0.17	0.03	0.4

Figure 17. Force Slip Relationship for G1 Profiled Steel Sheeting with Variation of α Value

Figure 18. Ultimate Load with Respect to α for G1 Profiled Steel Sheeting Figure 19. c_1 with Respect to α for G1 Profiled Steel Sheeting Figure 20. c_2 with Respect to α for G1 Profiled Steel Sheeting

4.3.3 G2 profiled

The second group of parametric studies, the authors varied the strain regimes which included $\alpha = 1, 0.5, 0, -0.5$ and -1 for G2 profiled steel sheeting. Figure 21 shows the force-slip relationship of G2 profiled steel sheeting with the variation of α value. The observation of the shear strength and ductility is similar to that mentioned in Section 4.3.2 above. When the structure is subjected to pure shear and pure bending, the maximum shear connector capacity is 94 kN and 125 kN respectively, which is a 25 % shear capacity increment. The definition of f_1 , f_2 and f_3 with respect to α is determined in Table 3 and plotted in Figures 22 to 24.

Figure 21. Force Slip Relationship for G2 Profiled Steel Sheeting with Variation of α Values

Figure 22. Ultimate Load with Respect to α for G2 Profiled Steel Sheeting

Figure 23. c_1 with Respect to α for G2 Profiled Steel Sheeting

Figure 24. c_2 with Respect to α for G2 Profiled Steel Sheeting

4.3.4 G3 profiled

The third group of parametric studies, G3 is used to verify Hicks' trapezoidal profiled steel sheeting. Figure 25 illustrates the force-slip relationship of G3 trapezoidal profiled steel sheeting with the variation of α value. The results illustrate that the shear strength and ductility is similar as mentioned in Section 4.3.2 above. When the structure is subjected to pure shear and pure bending, the maximum shear connector capacity is 91 kN and 128 kN respectively which is a 29 % increase in shear capacity. The definition of f_1 , f_2 and f_3 with respect to α is determined in Table 3 and plotted in Figures 26 to 28.

An example of Hicks [2] experimental study was used to verify the accuracy of these equations. Figure 29 shows the cross-section through the beam, and from Figure 29, the calculation to determine the α value is as below:

$$f'_c = 28 \text{ N/mm}^2$$

$$f'_y = 378 \text{ N/mm}^2$$

$$N_{concrete} = 0.85 f'_c B D_n$$

$$N_{concrete} = 0.85(28)(2500)(80)$$

$$N_{concrete} = 4760 \text{ kN}$$

$$N_{steel} = 2N_{flange} + N_{web}$$

$$N_{steel} = 2b_f d_f f'_y + b_w d_w f'_y$$

$$N_{steel} = 2(10.4)(167)(378) + 7.3(289.2)(378)$$

$$N_{steel} = 2111 \text{ kN}$$

From the above equations,

$$N_{concrete} > N_{steel}$$

Therefore the neutral axis, N.A. falls in the concrete element.

Compression, C = Tension, T

$$0.85(28)(2500)d_n = 2111$$

$$d_n = 35.5 \text{ mm}$$

In order to find the value of α

$$\alpha = \frac{d_n}{D}$$

$$\alpha = -\frac{35.5}{140 - 35.5}$$

$$\alpha = -0.34$$

From Figures 26 to 28, the Q_u , c_1 and c_2 can be determined and the force-slip relationship is derived as shown in Eq. 4.

$$Q = 115.98(1 - e^{-0.92.d})^{0.41} \quad (4)$$

Eq. 4 is plotted in Figure 30, and from Figure 30, it is verified that when the correct α value of -0.34 is determined, the discrepancy of the result is only 0.3 % when compared with the initial assumption of $\alpha = -1$ where the discrepancy is 4 %.

Figure 25. Force Slip Relationship for G3 Profiled Steel Sheeting with Variation of α

Figure 26. Ultimate Load with Respect to α for G3 Profiled Steel Sheeting

Figure 27. c_1 with Respect to α for G3 Profiled Steel Sheeting

Figure 28. c_2 with Respect to α for G3 Profiled Steel Sheeting

Figure 29. Beam Cross Section for G3 Profiled Steel Sheeting

Figure 30. Comparison between Experimental Data and Finite Element Model for Hick's Trapezoidal Profiled Steel Sheeting

4.4 Profiled Slabs Compared with Available International Standards

A comparison was made to evaluate the finite element solutions from G1, G2 and G3 with three existing international standards. They included the Australian Standard AS 2327 [16], British Standards Institute [17] and American Standard AISC [18].

Two failure modes can be observed to investigate the ultimate shear capacity of the shear connector in the push tests. They include shear failure and concrete failure. When the concrete strength is relatively high, the fracture of studs will often govern the design, and it is given by Eq. 5a, b and c.

$$P_{AS} = 0.63d^2 f_{us} \quad (5a)$$

$$P_{EC} = 0.628d^2 f_{us} \quad (5b)$$

$$P_{AISC} = 0.785d^2 f_{us} \quad (5c)$$

where

E_{sc} is the modulus of elasticity of the shear connector,

f_c is the compressive strength of the concrete,

f_{us} is the ultimate strength of the shear connector.

However, when the shear connector is stronger than concrete strength, Eq. 6a, b and c will tend to govern.

$$P_{AS} = 0.31d^2 \sqrt{f_{ck} E_c} \quad (6a)$$

$$P_{EC} = 0.29d^2 \sqrt{f_{ck} E_c} \quad (6b)$$

$$P_{AISC} = 0.39d^2 \sqrt{f_{ck} E_c} \quad (6c)$$

where

f_{ck} is the characteristic compressive cylinder strength of the concrete

E_c is defined as the mean modulus of elasticity of the concrete.

When the steel profiled slab is taken into consideration, equations from (5 and 6) above are multiplied by a reduction factor k . The value of k is defined in the equations below:

$$k_{AS} = 1.148 - (0.18 / \sqrt{n}) \quad (7a)$$

$$k_{EC} = \left(\frac{0.7}{\sqrt{n}} \right) \left(\frac{b_o}{h_p} \right) \left(\left(\frac{h}{h_p} \right) - 1 \right) \leq k_{t, \lim} \quad (7b)$$

$$k_{AISC} = \left(\frac{0.85}{\sqrt{n}} \right) \left(\frac{b_o}{h_p} \right) \left(\left(\frac{h}{h_p} \right) - 1 \right) \leq 1 \quad (7c)$$

where

n is the number of studs

b_o is the width of the steel profiled rib

h_p is the height of the decking profile

h is the height of the stud

Table 4. Comparison of Ultimate Load for Shear Connector between Finite Element Models and International Standards

Concrete Property, f _c = 20 MPa										
Profiled Type	FEM Result	Australian Standard		Eurocode		American Standard		FEM/Standard		
		SF	CF	SF	CF	SF	CF	AS	EC	AISC
	(kN)									
G1	73.1 (CF)	116.1	74.6	80.5	48.5	122.1	79.3	0.98	1.51	0.92
G2	70.3 (CF)	116.1	74.6	113.5	68.5	141.7	111.8	0.94	1.03	0.63
G3	68.8 (CF)	116.1	74.6	85.2	51.4	129.3	83.9	0.92	1.34	0.82
Concrete Properties, f _c = 25 MPa										
Profiled Type	FEM Result	Australian Standard		Eurocode		American Standard		FEM/Standard		
		SF	CF	SF	CF	SF	CF	AS	EC	AISC
	(kN)									
G1	81.5 (CF)	116.1	89.6	80.5	58.3	122.1	95.2	0.70	1.01	0.67
G2	78.2 (CF)	116.1	89.6	113.5	82.2	141.7	110.4	0.67	0.69	0.55
G3	76.2 (CF)	116.1	89.6	85.2	61.7	129.3	100.8	0.66	0.89	0.59
Concrete Properties, f _c = 32 MPa										
Profiled Type	FEM Result	Australian Standard		Eurocode		American Standard		FEM/Standard		
		SF	CF	SF	CF	SF	CF	AS	EC	AISC
	(kN)									
G1	93.2 (CF)	116.1	113.3	80.5	73.7	122.1	120.4	0.80	1.16	0.76
G2	89.3 (CF)	116.1	113.3	113.5	104.1	141.7	139.7	0.77	0.79	0.63
G3	86.6 (CF)	116.1	113.3	85.2	78.1	129.3	127.5	0.75	1.02	0.67
Concrete Properties, f _c = 35 MPa										
Profiled Type	FEM Result	Australian Standard		Eurocode		American Standard		FEM/Standard		
		SF	CF	SF	CF	SF	CF	AS	EC	AISC
	(kN)									
G1	98.2 (CF)	116.1	110.7	80.5	72	122.1	117.6	0.85	1.22	0.80
G2	94.1 (CF)	116.1	110.7	113.5	101.6	141.7	136.4	0.81	0.83	0.66
G3	91.1 (CF)	116.1	110.7	85.2	76.2	129.3	124.5	0.78	1.07	0.70
Concrete Properties, f _c = 40 MPa										
Profiled Type	FEM Result	Australian Standard		Eurocode		American Standard		FEM/Standard		
		SF	CF	SF	CF	SF	CF	AS	EC	AISC
	(kN)									
G1	103.7 (SF)	116.1	126.7	80.4	82.4	122.1	134.6	0.89	1.29	0.85
G2	100.4 (SF)	116.1	126.7	113.5	116.3	141.7	156.2	0.86	0.88	0.71
G3	97.4 (SF)	116.1	126.7	85.1	87.3	129.3	142.5	0.84	1.14	0.75

Concrete Properties, f _c = 50 MPa										
Profiled Type	FEM Result	Australian Standard		Eurocode		American Standard		FEM/Standard		
		SF	CF	SF	CF	SF	CF	AS	EC	AISC
	(kN)									
G1	114.7 (SF)	116.1	141.7	80.4	92.2	122.1	150.5	0.99	1.43	0.94
G2	112.9 (SF)	116.1	141.7	113.5	130.1	141.7	174.6	0.97	0.99	0.80
G3	110.1 (SF)	116.1	141.7	85.1	97.6	129.3	159.4	0.95	1.29	0.85

* S.F denotes stud failure and C.F concrete failure and

* Shaded area denotes failure mode govern

A summary of the ultimate loads for the shear connector, P which are calculated using the Australian Standard, Eurocode and American Standard is given in Table 4. Moreover, authors analysed the finite element model using different concrete strength which includes $f'_c = 20 \text{ MPa}$, 25 MPa , 32 MPa , 35 MPa , 40 MPa and 50 MPa . Table 4 verified that when the concrete strength is lower the concrete failure will govern whilst when the concrete strength is higher, the stud fracture failure tends to govern.

For concrete strength with $f'_c = 20 \text{ MPa}$ to 35 MPa , the finite element model demonstrated that the concrete stress reached maximum value before the stress in shear studs. Therefore, this verified that the push tests failed due to concrete failure. The failure modes predicted by existing standards also similar to the finite element models. From Table 4, the finite element model showed a more conservative value when compared to Australian and American Standard. On the other hand, Eurocode seems to be overly conservative.

When the concrete strength increased, the finite element model shows that the shear stud reaches its maximum stress before concrete slab. This proved that the failure mode is governed by stud fractured. When compared with the existing standards, the calculated theoretical values for both the Australian and American standards seem to have higher values when compared with finite element models. This confirmed that the finite element model is more conservative than the two existing standards. On the contrary, the Eurocode seems to be very conservative, where the calculated theoretical value is lower than the finite element predicted value.

5. CONCLUSIONS

Shear connection nonlinearity always results in significant changes in the strength and ductility of composite steel-concrete beams. In order to highlight such phenomena, a non-linear procedure with different strain regimes implemented in the concrete element has been presented to account for both the shear connection strength and ductility.

A comparison with experimental results allows the validation of such a procedure with reference to both solid slabs and profiled slabs for both push tests and beam tests. The finite element analysis that has been undertaken in this paper demonstrated that the push test to determine the shear connection capacity for solid slabs is reliable and accurate. However this is not the case for slabs with profiled steel sheeting.

An extensive parametric study of fifteen push test specimens with different profiled steel sheeting geometries was performed using the established finite element models. The comparison of shear connector capacities obtained from the finite element models proved that they depend significantly on the width and rib types of profiled steel sheeting.

The introduction of the parameters such as α , Q_u , c_1 and c_2 are the key characteristics of the proposed method because they account for the behaviour of the shear connection nonlinearity effects. When accurate parameter values were applied to the force-slip relationship equations, it seems to suggest that the proposed method has less discrepancy when compared with the experimental study, and it is also useful for design engineers using these profiles in industry.

When the Australian, Eurocode and American Standards are compared with finite element results, the Australian Standards and American Standards appeared to be overestimating the shear connector capacity whilst Eurocode grossly underestimated the strength of shear connectors for profiled slabs. From the three existing standards, the American Standard seems to be less conservative where higher ultimate load was observed.

This study also provides a very initial study of this issue. Further experimental studies and more substantial parametric studies would be necessary before being able to employ these results in international code recommendations.

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