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Tests of partially connected composite plate girders

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ABSTRACT

This paper is concerned with the strength and behaviour of partially connected composite plate girders. Eight simply supported composite plate girders were tested to failure under a concentrated load applied at the mid-span. The main variables considered in this study are the longitudinal spacing of stud connector, diameter of stud shank, number of studs along the upper flange and concrete strength. Test results have shown considerable variations in ultimate strength, behaviour and failure characteristic in all specimens due to effects of partial interaction. An analytical method to determine the ultimate strength of partially connected composite plate girders is outlined. Three-dimensional non-linear finite element analyses are also carried out on the girders using the general purpose computer code, LUSAS. Results obtained through the analytical method are compared with the corresponding finite element and experimental ones in respect of ultimate load, load–deflection response and deformation behaviour. Reasonably close agreement between the results establishes the accuracy of the analytical method and finite element simulation.

1. Introduction

Plate girders are expected to support concrete deck slabs when used in buildings or bridges. The need for cost-effective construction with satisfactory performance has led to the use of composite action between the steel girder and concrete slab. These two structural elements, in a typical composite construction, are firmly connected together through shear connectors so that they can act compositely as a single unit, thus fully exploiting the advantages of the two different materials. The performance of a composite plate girder is dictated by the individual strength of concrete slab and steel girder as well as shear connection stiffness between the interacting elements. Full composite interaction is achieved when use of rigid connectors can perfectly prevent the relative horizontal movement and vertical separation at the steel–concrete interface. Headed studs, the most widely used type of shear connector, are flexible in the sense that they may deform to a certain extent depending on the connection stiffness and magnitude of shear force transmitted through the connectors. Based on this, the shear connection is categorised as partial even though full interaction is employed in the design [1].

Partial interaction permits longitudinal slip arising from strain discontinuity at the steel–concrete interface. The presence of slip gives rise to deflection due to additional curvature which in turn, lowers the flexural stiffness and strength of the composite members [2]. In many instances, it may be found that strength of composite members is excessive when full interaction is assumed. It is often advantageous to provide fewer stud connectors than the number required for full interaction. Fewer number of studs may be insufficient to develop full composite strength but yet adequate to just provide the strength required. Partial interaction design becomes the solution particularly when the surface area of top flange is insufficient to accommodate the number of studs required for full interaction. Wider spacing of stud connectors helps reduce obstruction or detailing problems that often arise between transverse slab reinforcement and stud connectors. Partial interaction design is useful also when the concrete slab is cast on metal decking with corrugated profiles that run across the flange of the beam. Voids under the corrugations limit the amount of shear that can be transferred from the slab to the beam which in turn, make full interaction design impossible. This portrays the economical aspects in partial interaction design. The advantages of composite action can still be realised but in a somewhat reduced manner [3].

Research works have been directed in the past towards the study on different aspects of composite beams and composite plate girders. Tests on steel–concrete composite beams having a continuous imperfect shear connection were carried out by Newmark et al. [4]. Based on small deformation elastic analysis and considering Euler–Bernoulli's beam theory, a second order differential equation for slip-allowed composite beam was derived by assuming equal curvature between the interacting elements. Daniels and Fisher [5] reported results from tests on two simple span composite beams to investigate the load–deflection, load–slip and load–rotation relationships. Adekola [6] presented an interaction theory for composite beams allowing for interface friction, slip and negative uplift deformation. Fourth and

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second order coupled differential equations connecting the uplift tension and axial force within the elements were derived and solved by a finite difference method. Theoretical formulations for short-term and time dependent response of composite beams under sustained loads accounting for the effect of interfacial slip were proposed by Bradford and Gilbert [7], leading to an iterative solution for curvatures, strains and deflections.

Betti and Gjelsvik [8] applied an analogue-beam model to derive a sixth order differential equation for vertical deflection of composite beams. Two-dimensional finite element formulation for simply supported and continuous composite beams with flexible shear connection developed by Oven et al. [9] has shown that maximum interface slip occurs at both ends. It was found that stud connectors located near end supports are stressed beyond elastic limit. In continuous beams, however, the slip behaviour at the interior support regions is so uncertain that no clear pattern of slip progression can be identified. Partial interaction theory was applied to develop equations for deflection of composite beams with varying shear connector density along the span [10]. A general design chart for deflection at the mid-span was proposed. Seracino et al. [11] introduced the concept of focal point to derive partial interaction flexural stresses from full interaction analysis. The concept was also applied to continuous composite beams [12,13]. Queiroz et al. [14] presented a modelling technique to analyse composite beams accounting for full and partial shear connection using a finite element package. Use of discrete non-linear spring elements for modelling the studs was found to be efficient compared to solid elements. A novel numerical formulation based on higher order beam theory was presented by Chakrabarti et al. [15] for the analysis of composite beams considering the effect of partial interaction and transverse shear deformation.

An experimental investigation on small-scale models of composite plate girders was reported by Porter and Cherif [16], who outlined a theoretical solution to predict the ultimate strength based on the observed failure mechanism. Shanmugam and Baskar [17] studied experimentally the ultimate load behaviour of composite plate girders under shear and negative bending moment. The test specimens were designed with full composite interaction. Principal strain in webs was measured to examine the extent of tension field due to composite action. Shanmugam et al. [18] utilised a finite element method to analyse horizontally curved composite plate girders by assuming perfect composite action between concrete slabs and steel girders. Based on the design approach proposed by Shanmugam and Baskar [19] for straight composite plate girders, an approximate method to predict the shear strength of curved girders was proposed. On the basis of earlier works [20–22], Darehshouri et al. [23] proposed an analytical method for ultimate shear strength of composite plate girders predominantly under shear. Accuracy of the method was established through comparisons with the corresponding experimental results and finite element predictions. Sherafati et al. [24] reported recently an experimental study on composite plate girders under shear and positive bending to examine the mechanics of failure, flexural behaviour, out-of-plane deformation of web surface and principal strains at the centre of the web.

Studies on composite plate girders with partial interaction are rare, except for the work by Allison et al. [25], who carried out tests on large-scale composite plate girders under the action of combined shear and negative bending to examine the interaction between web tension field and shear connection. The related studies solely involved numerical formulations or computer modelling which do not provide a broad perspective in respect of the behaviour of such girders. Therefore, there is a necessity to get a clear picture of the behaviour of these girders through physical observations in order to enhance the understanding of the elastic and inelastic behaviour of composite plate girders with partial interaction. An experimental investigation on composite plate girders with partial interaction has, therefore, been undertaken. Details of the experiments are presented in this paper along with the results obtained. Girders of practical size were tested to failure under monotonic loading applied at the mid-span. Different levels of shear connection were considered by varying concrete strength and shear stud spacing, size and number along the flange. Attention was focused on variation of load carrying capacity, slip at the interface, strain in the girders, mode of failure and overall behaviour due to partial interaction. The tested girders were analysed using an analytical method [26] proposed earlier by the authors for partially connected composite plate girders and the results are compared to assess the accuracy of the analytical method. The accuracy of the analytical method is also established by comparing the results with corresponding values obtained by using a non-linear finite element analysis. The analytical method and the finite element modelling are described briefly in the paper.

2. Experimental investigations

2.1. Details of the test specimens

Steel plate girders were designed based on those tested by Shanmugam and Baskar [17] by modifying the girder configurations to suit the test facilities available in the laboratory and the requirements of the testing intentions. The test specimens were designed in accordance with code BS 5950-1: “Structural use of steelwork in building”: 2000 which allows for a maximum value of slenderness ratio, d/t, up to 250. The flanges were so chosen that they remained compact and that the girder will not fail in lateral torsional buckling mode.

Eight identical steel plate girders were fabricated using mild steel plates of Grade 43A (equivalent to Grade S275) complying with code BS 4360: “Specification for weldable structural steels”: 1990. Flat plates of different thicknesses were measured, marked and machined accurately to size. All of the components were welded together with continuous fillet welds using a low temperature system to minimise the welding distortion. Sufficient care was taken when welding the thin web plate by providing lateral supports at certain intervals to avoid large initial imperfections of the web. Stiffeners were welded accordingly on both sides of the web plate. The basic dimensions were kept the same in all the girders in order to have a constant span length, $L=3655$ mm, thickness of web, $t=3$ mm, flange width, $b_f=200$ mm, flange thickness, $t_f=20$ mm, d/t ratio of 250 and b/d ratio of 1.16. Shear connectors were welded at different longitudinal spacings on the top flange of the girder to obtain a specified degree of shear connexion when they act compositely with the concrete slab. Three different headed studs of 16 mm, 19 mm and 25 mm shank diameters with ultimate tensile strength f_u not greater than 500 N/mm^2 were used as shear connectors. Calculations for the number of shear studs required to achieve full interaction are made in accordance with Eurocode 4 [27]. The degree of interaction, defined by percentage, was specified by changing the longitudinal spacing or diameter of shear studs along the length of the girders.

The girders are identified in the text as G1C20, G1C30, G2C30, G3C30, G4C20, G4C30, G5C30 and G6C30. Notations C20 and C30 refer to grade of concrete whilst notations G1–G6 indicate different diameters or spacings of stud connectors along the span. Details of the girders along with shear connexion properties are summarised in Table 1. In the table, B_c and H_c denote width and thickness of the concrete slab, respectively. Fig. 1(a) and (b, c) shows the elevation and cross-sections of a typical test girder, respectively. Details of headed studs are shown in Fig. 2.

The concrete mixture comprises crushed natural aggregates with maximum size of 20 mm and 60% of fine aggregate passing through the 600 μm test sieve. Slump of 50 mm was specified to give medium workability mixes. Water-to-cement ratio was determined