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Jemaa, Yaser; Jones, Connah; Dirar, Samir

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## DEEP EMBEDMENT STRENGTHENING OF FULL-SCALE SHEAR-DEFICIENT REINFORCED CONCRETE BEAMS

Yaser Jemaa<sup>a,1</sup>, Connah Jones<sup>a,2</sup> and Samir Dirar<sup>a,3</sup>

<sup>a</sup> School of Civil Engineering, University of Birmingham  
Edgbaston, Birmingham, B15 2TT, United Kingdom

<sup>1</sup> Email: y.jemaa@bham.ac.uk

<sup>2</sup> Email: cdj186@bham.ac.uk

<sup>3</sup> Email: s.m.o.h.dirar@bham.ac.uk

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### ABSTRACT

The deep embedment (DE) technique is a recently developed shear strengthening method for existing reinforced concrete (RC) structures. The majority of RC beams strengthened using the DE method had effective depths of less than 400 mm. This is unrepresentative of several practical situations where RC girders/beams have significantly higher effective depths. This paper examines the shear behaviour of full-scale unstrengthened as well as strengthened RC T-beams. The beams had an effective depth of about 600 mm, a shear span to effective depth ratio of about 3.0, and were reinforced with internal steel shear reinforcement. The DE shear strengthening system consisted of six 12 mm diameter sand-coated glass fibre reinforced polymer (GFRP) bars in each shear span. The DE GFRP bars enhanced the shear force capacity of the strengthened beam by about 96%. The strengthened beam had a slightly higher cracked stiffness due to the presence of the DE GFRP bars which resisted inclined crack opening and consequently controlled deflection. The main shear crack in the strengthened beam had a steeper inclination. For strengthened beams with light steel shear reinforcement, the results suggest that the total shear force capacity can be calculated as the sums of the concrete, steel, and FRP contributions.

### 1 INTRODUCTION

Over the past few decades, shear strengthening techniques utilising externally bonded (EB) or near-surface mounted (NSM) fibre reinforced polymer (FRP) composites were successfully applied to enhance the shear force capacity of existing deficient reinforced concrete (RC) structures. The success of the FRP shear strengthening techniques may be attributable to their high durability, high strength-to-weight ratio, environmental resistance, and ease of application; thus leading to a more whole-life cost-effective solution compared with the more conventional shear strengthening methods such as RC Jacketing and steel plate-bonding.

The deep embedment (DE) technique [1, 2], also known as the embedded through-section (ETS) technique [3-5], is a recently developed method offering a more practical and effective shear strengthening solution for existing RC structures. In the case of a RC girder/beam, vertical holes are drilled upwards from the soffit in the shear span. High viscosity epoxy resin is then injected into the drilled holes and FRP or steel bars are embedded into place [2]. The use of FRP bars is favoured as it eliminates the possibility of corrosion of the shear strengthening system [2, 3]. Embedding the FRP bars into the concrete core provides higher strengthening effectiveness because, unlike the EB and NSM FRP techniques, the DE technique relies on the concrete core to transfer stresses between the concrete and FRP bars. The concrete core provides better confinement and consequently better bond performance to overcome the debonding failure usually associated with the EB and NSM FRP strengthening methods [4]. Other advantages of the DE FRP shear strengthening technique over the EB and NSM FRP methods include higher protection against fire and vandalism; access to the top slab and time-consuming surface preparation are not required; and less epoxy consumption.

Research investigating the shear behaviour of RC beams strengthened with DE FRP bars [1-4] has been limited. Notwithstanding this, such research has demonstrated the effectiveness of the method and provided valuable results, particularly with regard to the effects of the internal steel shear reinforcement; diameter, orientation and spacing of the FRP bars; and FRP bar surface coating on the shear behaviour [1-4]. However, the majority of specimens tested to date [1-4] had effective depths of less than 400 mm. This is unrepresentative of several practical situations, such as RC bridges, where RC girders/beams have significantly higher effective depths. Furthermore, existing research studies [1-4] focused on the use of carbon or aramid FRP bars as DE shear strengthening systems, whereas the use of DE glass FRP (GFRP) bars has not been investigated. A proper understanding of the effects of realistically-sized members and DE GFRP bars on the strengthened behaviour is vital for the best utilisation of the DE shear strengthening technique.

This paper critically examines the structural behaviour of full-scale unstrengthened as well as DE GFRP shear strengthened RC T-beams. The experimental results in terms of shear force capacity, shear force-deflection curves, crack patterns, and strain in the steel and FRP shear reinforcement are presented and discussed. The results demonstrate the potential of the DE technique in enhancing the shear behaviour of shear-deficient RC beams.

## 2 TEST PROGRAMME

### 2.1 Test Specimens

Two (unstrengthened and strengthened) shear-deficient full-scale RC T-beams were tested in a four-point bending configuration as shown in Figure 1. The beams were 4.7 m long and had an effective depth of about 600 mm and a shear span to effective depth ratio of about 3.0. The flange was 400 mm wide and 125 mm deep; the web was 150 mm wide; and the overall depth of the cross-section was 650 mm. The tension steel reinforcement consisted of two layers of two 25 mm bars; whereas the flange was reinforced with one layer of four 12 mm steel bars. Steel plates, 125 mm × 125 mm × 25 mm and 380 mm × 25 mm × 25 mm, were welded at the ends of the tension and compression steel bars, respectively, to provide proper anchorage.

The shear spans in both beams were reinforced with 8 mm internal steel shear links spaced at 600 mm centre-to-centre (c/c). This shear link spacing is representative of earlier design practice which allowed shear link spacing of up to the effective member depth [6]. The DE shear strengthening system, which consisted of six 12 mm diameter sand-coated GFRP bars in each shear span (see Figure 1), was designed according to the TR55 [7] shear strengthening guidelines.

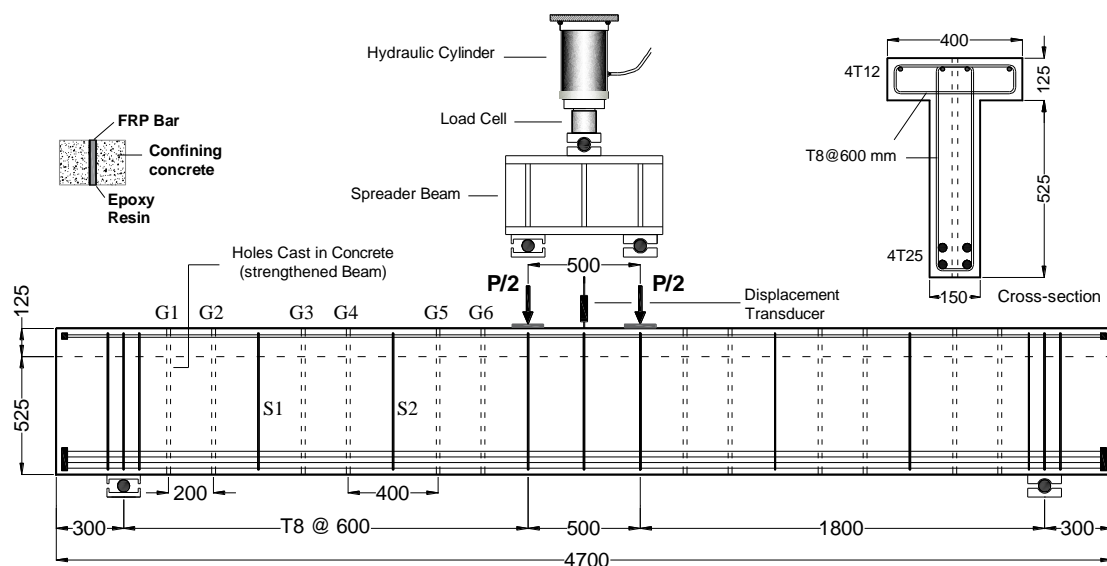


Figure 1 Dimensions and reinforcement details of the tested beams – all dimensions in mm

## 2.2 Materials

The two RC T-beams, together with six cube and six cylinder ancillary specimens, were cast with one batch of ready-mix concrete from a local supplier. Compressive tests carried out on the day of testing showed that the average concrete cube and cylinder compressive strengths were 48.1 MPa and 40.4 MPa, respectively.

The yield ( $f_y$ ) and ultimate ( $f_u$ ) strengths of the longitudinal steel reinforcement were 580MPa and 680MPa, respectively, whereas the corresponding values for the steel shear links were 540MPa and 680MPa, respectively.

The 12 mm sand-coated GFRP bars had a tensile strength, elastic modulus, and ultimate strain of 973 MPa, 40 GPa, and 2.43%, respectively. A commercially available high viscosity epoxy resin was used for bonding the GFRP rods to the concrete. As specified by the manufacturer, it had a bond strength, compressive strength, compressive modulus, tensile strength, and elongation at failure of 12.4 MPa, 82.7 MPa, 1,493 MPa, 43.5 MPa, and 2%, respectively.

## 2.3 Installation of the GFRP Bars

In order to install the GFRP bars, 18 mm diameter vertical holes were created in the shear spans, through the centerline of the cross-section, at the locations shown in Figure 1. The vertical holes were created by installing 18 mm diameter PVC rods at the required positions within the steel reinforcement cage before casting the concrete. The PVC rods were removed from the concrete 7 days after casting. Prior to installing the GFRP bars, the holes were cleaned by a wire brush and compressed air to remove any cement or aggregate residues. The lower ends of the holes were sealed and a high viscosity epoxy resin was used to fill two-thirds of the holes. The GFRP bars were covered with a thin layer of the adhesive and inserted into the holes. Any excess epoxy was removed. Valerio et al. [2] demonstrated that it was possible to install FRP bars by drilling vertical holes upwards from the soffit. The procedure explained earlier for installing the GFRP bars was used for simplicity as it did not require drilling holes. However, it should be noted that drilling would probably provide a rougher hole surface and consequently improve the bond between the GFRP bars and the concrete.

## 2.4 Instrumentation

A 1000 kN load cell measured the total load which was applied using a 1000 kN hydraulic cylinder (see Figure 1). The vertical deflection at mid-span was measured using both a linear resistance displacement transducer (LRDT) and a dial gauge. Strain gauges (6 mm, 120  $\Omega$ ) measured the strain in the shear links and embedded GFRP bars. The readings of the 1000 kN load cell, LRDT, and strain gauges were obtained using a data logger. The readings of the dial gauge were manually recorded.

# 3 RESULTS AND DISCUSSION

## 3.1 Shear Force Capacity

The unstrengthened shear force capacity of each beam, together with the shear force at failure, gain in shear strength attributable to the DE GFRP bars, and failure mode are given in Table 1.

Table 1 Test results

Beam	Unstrengthened Shear Force Capacity (kN)	Shear Force at Failure (kN)	Gain Attributable to GFRP (kN)	Gain Attributable to GFRP (%)	Failure Mode
Control	149.5	149.5	-	0	Shear
Strengthened	149.5	293	143.5	96	Shear

Both beams failed in shear as expected. The shear force carried by the control beam at failure was 149.5 kN. The strengthened beam failed at a shear force of 293 kN, attaining a 96% increase in shear

force capacity. Similar to the internal shear links, the DE GFRP bars contribute to the shear force capacity by resisting crack opening.

### 3.2 Shear Force-Deflection Response

Figure 2 compares the shear force-mid-span deflection curves of the two tested beams. Both beams had similar linear shear force-deflection response up to the formation of flexural cracks at a shear force of about 28 kN. Above this shear force level, the shear force-deflection curves turned nonlinear due to the propagation of flexural cracks. The stiffness of both beams remained comparable up to a shear force of about 70 kN, which corresponds to the formation of inclined cracks. Upon further loading, the strengthened beam had a slightly stiffer response due to the presence of the DE GFRP bars which resisted inclined crack opening and consequently controlled deflection. For both beams, there was a sudden drop in load at peak shear force, which is a characteristic of shear failure.

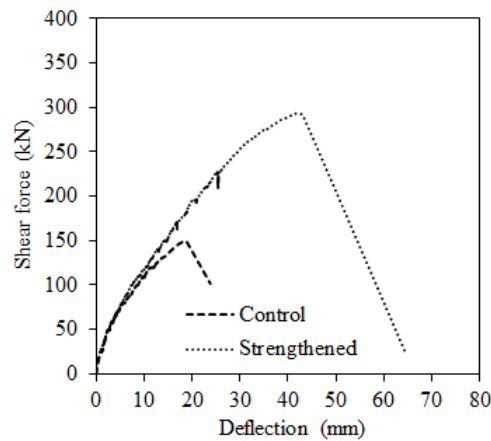


Figure 2 Shear force-mid-span deflection curves

The inclusion of the DE GFRP bars enhanced the deflection capacity of the strengthened beam, which had a mid-span deflection at failure of about 41.5 mm. This represents an increase of 121% over the mid-span deflection at failure of the control beam (18.8 mm).

### 3.3 Cracking and Failure Mode

In both beams, flexural cracks appeared in the constant moment region at a shear force of about 28 kN. Upon further loading, the flexural cracks extended into the shear spans. The outermost flexural cracks in the shear spans turned into inclined cracks at a shear force of about 70 kN. With increased loading, more inclined cracks appeared in the shear spans and, eventually, the outermost inclined crack extended from support to loading plates causing failure. Figure 3 shows the crack patterns at failure.

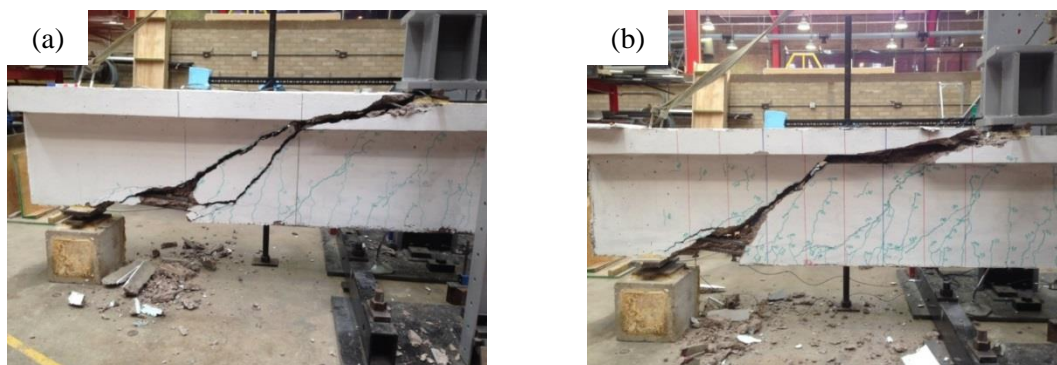


Figure 3 Crack patterns at failure: (a) control beam and (b) strengthened beam

The main inclined crack that caused failure of the control beam had two branches in the beam web (see Figure 3a). The outer branch had an inclination of about  $26^\circ$  to the beam longitudinal axis whereas the inner branch had an inclination of about  $45^\circ$ . In the flange, the main crack had an inclination of about  $14^\circ$ .

The strengthened beam failed due to an inclined crack that followed a path at an angle of about  $33^\circ$  in the web and  $13^\circ$  in the flange. Failure was preceded by popping noise suggesting debonding of some of the DE GFRP bars.

### 3.4 Strain Response

Figure 4 shows the shear force-strain curves for the shear links (see Figure 1 for shear link designation). The curves depicted in Figure 4 can be divided into three distinct stages. In the first stage, the strain in the shear links was insignificant and they did not contribute to the shear force capacity. In the second stage, the shear links were crossed by inclined cracks and they started to develop strain with increased loading. This is represented by the ascending parts of the curves. The shear forces at which the shear links became active varied between 60 and 100 kN for the control and strengthened beams respectively. It can be seen that the shear links in the strengthened beam developed strain at a much slower rate compared to those in the control beam. This is attributable to the presence of the DE GFRP bars which limited crack opening and thus helped reducing the strain in the shear links. The third stage is marked by yielding of the shear links. It should be noted that all shear links attained or exceeded the yield strain of 0.003 at failure. Due to yielding of the steel shear links at failure, the steel contribution to the shear force capacity ( $V_s$ ) may be calculated as 108 kN, i.e. the yield strength (540 MPa) multiplied by the cross-sectional area of the shear links ( $100 \text{ mm}^2$  per shear link). Hence, in the case of the control beam, the concrete contribution to the shear force capacity ( $V_c$ ) may be calculated as  $149.5 \text{ kN} - 108 \text{ kN} = 41.5 \text{ kN}$ .

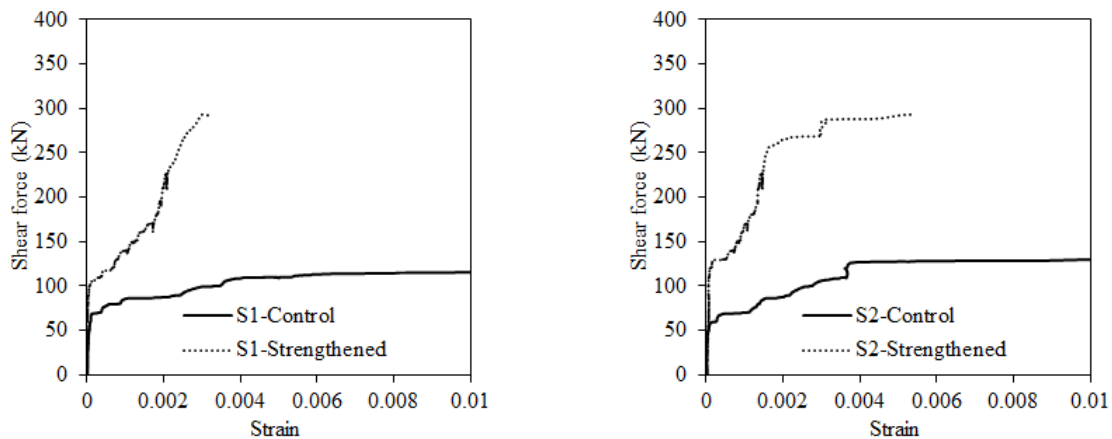


Figure 4 Shear force versus strain in the shear links

Figure 5 shows the shear force-strain curves for the DE GFRP bars (see Figure 1 for GFRP bar designation). Similar to that of the shear links, the strain response of the DE GFRP bars can also be divided into three stages. In the first stage, the DE GFRP bars remained inactive and did not contribute to the shear force capacity. In the second stage, the DE GFRP bars developed strain with increased loading up to about 80% of the shear force capacity. In the third stage, some of the GFRP bars started to debond. This is represented by a decrease in strain with increased loading. The strain in G1, G2, G3, G4, G5 and G6 at failure was 0.0031, 0.0018, 0.0091, 0.0035, 0.0069 and 0.0036 respectively. As the six DE GFRP bars were crossed by the main shear crack (see Figure 3b), the FRP contribution to the shear force capacity ( $V_f$ ) may be calculated as 127.4 kN. Hence, in the case of the strengthened beam, the concrete contribution to the shear force capacity was 57.6 kN. This represents a 38.8% increase in  $V_c$  for the strengthened beam compared to that of the control beam. This result is important because it shows that (i) for strengthened beams with light steel shear reinforcement, the total shear force capacity can be calculated as  $V_c + V_s + V_f$ , and (ii) the DE technique can significantly enhance  $V_c$ .

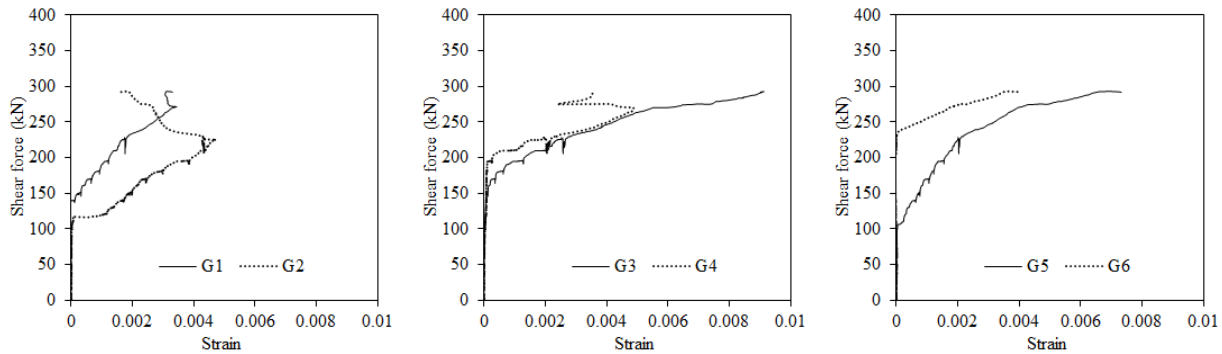


Figure 5 Shear force versus strain in the DE GFRP bars

#### 4 CONCLUSIONS

This paper critically examined the shear behaviour of full-scale unstrengthened as well as DE GFRP shear strengthened RC T-beams. The main findings of this research are summarised below.

- The DE technique enhanced the shear force capacity of the strengthened beam by about 96%.
- The DE GFRP bars did not affect beam stiffness up to the formation of inclined cracks. Subsequently, the strengthened beam had a slightly stiffer response as the DE GFRP bars resisted inclined crack opening and consequently controlled deflection.
- The main shear crack in the strengthened beam had a steeper inclination ( $33^\circ$ ) compared to that of the corresponding crack in the unstrengthened beam ( $26^\circ$ ).
- For strengthened beams with light steel reinforcement, the total shear force capacity can be calculated as the sums of the concrete, steel, and FRP contributions.

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