

Rehabilitation of RC inverted-T girders using anchored CFRP sheets

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Abstract

Existing infrastructure includes significant number of reinforced concrete (RC) bridges that are old and designed based on outdated design codes. Inverted-T beams used as bentcap girders that support the bridge's longitudinal precast beams provided a popular structural system of many existing bridges. The structural behaviour of inverted-T beam is different than that of conventional top-loaded beams, due to the fact that the loads are introduced into the bottom flange rather than the top of the beam. The objective of the present study is to experimentally examine the effectiveness of new rehabilitation techniques using anchored carbon fibre-reinforced polymer (CFRP) sheets to eliminate non-ductile failure mechanisms in hanger, web, and flange zones of RC inverted-T girders. The experimental program of this study included conducting eight tests on four simply supported RC inverted-T girders before and after rehabilitation. The girders represent a 1/3rd scale model of a prototype bentcap girder. The experimental results showed that the proposed new rehabilitation schemes proved to eliminate the non-ductile mechanisms in hanger, web, and flange zones. The rehabilitated girders were able to reach displacement ductility levels of up to 4 and showed an increase in strength up to 20% compared to the control girders. © 2007 Elsevier Ltd. All rights reserved.

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1. Introduction

Inverted-T beams used as bentcap girders that support the bridge's longitudinal precast beams (as shown in Fig. 1) provided a popular structural system of many existing bridges. The structural behaviour of inverted-T beams is different than that of conventional top-loaded beams, due to the fact that the loads are introduced into the bottom flange rather than the top of the beam. The flange of the inverted-T serves as a shallow shelf to support precast beams while the stem of the inverted-T, rising to the height of the supported beams, provides the needed depth to sustain flexure and shear forces [13].

Existing infrastructure includes significant number of reinforced concrete (RC) bridges that are old and designed based on outdated design codes. A large number

of these bridges need to be rehabilitated or upgraded due to the increase in specified loads and the deterioration due to aging and loss of reinforcement due to corrosion or construction errors. Among bridge elements that need to be rehabilitated are inverted-T bentcap girders that are susceptible to non-ductile failure once the capacity of a non-ductile failure mechanism is reached. Early investigations that studied the behaviour of inverted-T beams identified three main categories of non-ductile failure mechanisms depending on location, which are failure in hanger zone, failure in web, and failure in flange [13]. Recently, Texas Department of Transportation (TxDOT) supported a research program on investigating the signs of deterioration of existing inverted-T bentcaps that are used extensively on highway bridges in Texas [16,18] and to modify the design criteria for new bentcaps. The problem is that at service load unacceptable diagonal cracks frequently occur at the re-entrant corners between the cantilever ledges, or flanges, (that can be considered to function locally as a bracket that is supported at the face

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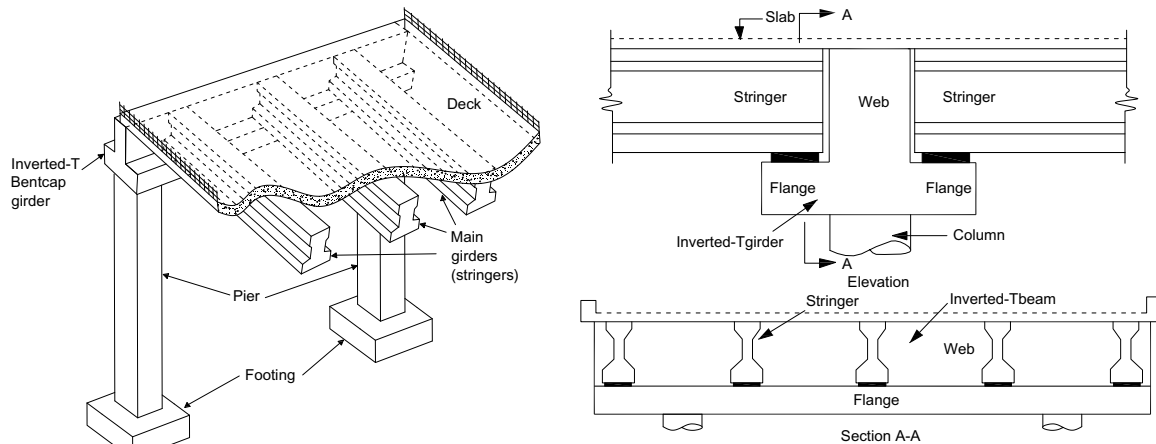


Fig. 1. Structural system of an inverted-T bentcap girder.

of the T-beam stem) and the web. In order to predict the crack widths in flanges of inverted-T bentcaps, Zhu et al. [17] developed a strut-and-tie model that was calibrated using the results of seven full-scale 2-D and ten 3-D inverted-T specimens. Furthermore, Song et al. [15] studied the efficiency of piezoceramic transducers that are embedded in concrete structures for damage detection of a 6.1 m long reinforced concrete bridge inverted-T bentcap to be used for structural health monitoring of new inverted-T bentcaps. Although the aforementioned research activities would contribute to develop a serviceability design method that considers controlling and monitoring the cracks for newly constructed inverted-T bentcaps, there is still a need to investigate possible methods to upgrade the possible non-ductile performance of existing ones.

Recent research studied the upgrade of RC bridge girders using externally bonded fibre-reinforced polymers (FRP) as an alternative for conventional strengthening techniques such as using steel plates, reinforced concrete jackets, or external post-tensioning. Externally bonded FRP reinforcement is an attractive rehabilitation technique due to its tailorable performance characteristics, ease of application, high strength-to-weight ratio, and non-corrosiveness characteristics.

Externally bonded fibre-reinforced polymer (FRP) composite sheets are used to increase the shear strength of non-ductile RC rectangular and T-beams by orienting the fibres normal to the axis of the beam or to cross potential shear cracks [9,11,4,6]. Increasing the shear strength of a beam can alter the failure mode to avoid non-ductile shear failure such that the beam fails after reaching its flexure capacity in a ductile manner.

Despite the use of inverted-T beam in several existing parking structures and bridge bentcaps that were designed according to older codes, minimal research attention had been found in the literature in evaluating their strengthening techniques using FRP composites. Most of the previous research activities focused on FRP applications in the reha-

bilitation of the RC rectangular and regular T-sections. As such, there is a need to investigate the effectiveness of FRP composite wraps in the rehabilitation of inverted-T girders that are susceptible to non-ductile failures.

2. Behaviour of RC inverted-T girders

Despite its frequent use since the 1950s, the inverted-T shape remained as one of the least investigated cross-sections till mid-1980s [14]. Until that time, no guidance for handling design problems specifically associated with the inverted-T section was available in North American Standards [1,3,8]. Therefore, engineers have tended to rely on personal judgment and discretion for design of inverted-T girders.

In rectangular RC beams, the modes of failure can be generally categorized under two distinct (or a combination of) modes of failures, namely flexure and shear. A premature shear failure without reaching the flexural capacity of the beam would result in a non-ductile failure characterized by its sudden nature. In inverted-T girders, the non-ductile modes (other than the development of flexure capacity of longitudinal flexural reinforcement) are more complicated than regular rectangular beams. Stringer bearings on the top face of the flange of an inverted-T girder produces vertical tensile forces (hanger tension) near the bottom of the web as indicated in Fig. 2. Such forces are not ordinarily encountered in conventional rectangular beams, where vertical forces are applied at the top of the web. Furthermore, the longitudinal and lateral bending of the flange of an inverted-T girder produce a very complex stress distribution in the flange. Mirza and Furlong [13] conducted experimental research on identifying the behaviour of RC inverted-T girders and categorized 3 non-ductile modes of failure according to their location, namely, hanger, web and flange. Hanger failure involves the failure of the stirrups acting as hangers and supporting the concentrated loads on the bracket. Web failures involve web-shear or shear-compression failures. Flange failures

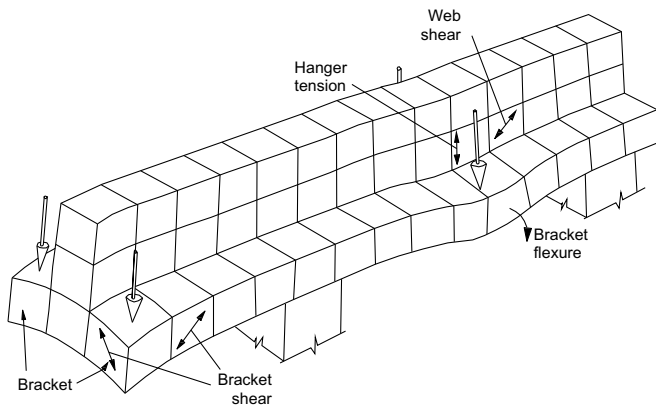
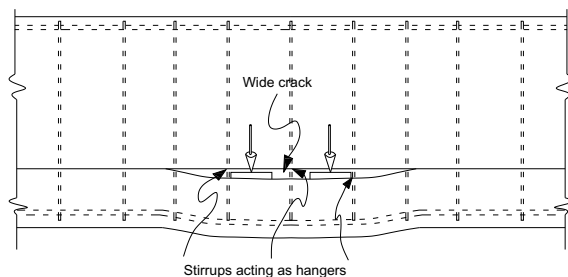
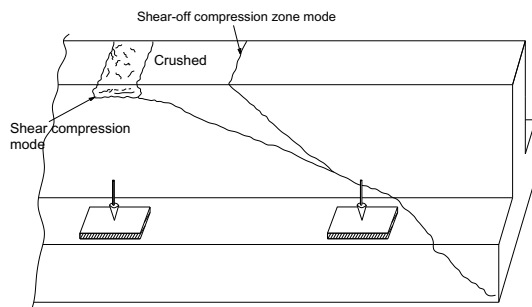


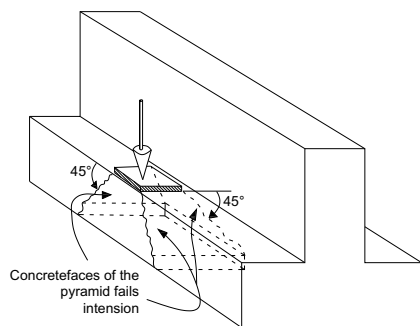
Fig. 2. Structural actions on inverted-T girder [14].



(a) Hanger failure mode



(b) Shear compression and shear-off compression zone mode in the web



(c) Punching shear failure mode in the flange

Fig. 3. Typical non-ductile failure modes of inverted-T girders [13].

were attributed either to shear friction or flexure accompanied by punching shear in the bracket. Fig. 2 shows the structural actions on inverted-T girder, while Fig. 3 shows a schematic of the three typical non-ductile failure modes.

A well-designed inverted-T girder should ensure that the load capacities of such non-ductile modes are higher than the load corresponding to the yielding of the longitudinal flexure reinforcement, i.e. ductile flexure failure occurs. On the other hand, an inverted-T girder with limited load capacity in one or more of its non-ductile mechanisms, needs to be rehabilitated.

3. Research significance

The research reported presents an experimental study on the use of externally bonded carbon fibre-reinforced polymer (CFRP) composite in strengthening existing RC inverted-T bentcap girders to eliminate non-ductile failure mechanisms on hanger, web and flange. Eight tests were conducted on four inverted-T scaled girders to assess the effectiveness of the proposed strengthening schemes for the studied failure mechanisms. The results show that the rehabilitated girders experienced an increase in their displacement ductility and load carrying capacities compared to the control girders.

4. Test setup and details of specimens

The girders were tested under 4-point loading system as shown in Fig. 4. The control girders were subjected to increasing incremental load up to the first sign of non-ductile mechanism (which is defined as a rapid decrease in the load monitored on the data acquisition system), while the rehabilitated specimens were tested up to failure (after the girder develops its full displacement ductility capacity). After each loading increment, all visible cracks were marked. The loads, deflections and strains were continuously recorded during the test. All tested girders were identical in size and proportion, with the only difference being the reinforcement content in hanger, web shear and flange loading zones. The tested girders represent a 1/3rd scale prototype of RC inverted-T bridge bentcap girders. Fig. 5 shows the dimensions and details of the reinforcement for the tested girders. The girders are simply supported over the span of 3000 mm with a height of 610 mm, flange width of 560 mm, flange height of 220 mm and web width of 180 mm. The control girders were designed to satisfy the minimum reinforcement requirements according to the ACI [2] code providing negligible capacities in hanger, web-shear and punching zones for girders G1&G2, G3 and G4, respectively. This arrangement of reinforcement was selected in order to isolate the three different non-ductile mechanisms and to avoid the complications in the interpretations of the results in case of having two, or more, combined failure mechanisms. It should be noted that an extension of the current research could study the rehabilitation of girders with combined failure mechanisms. The control girders are designated as IT-G# while the girders rehabilitated using CFRP sheets are designated as IT-G#R.

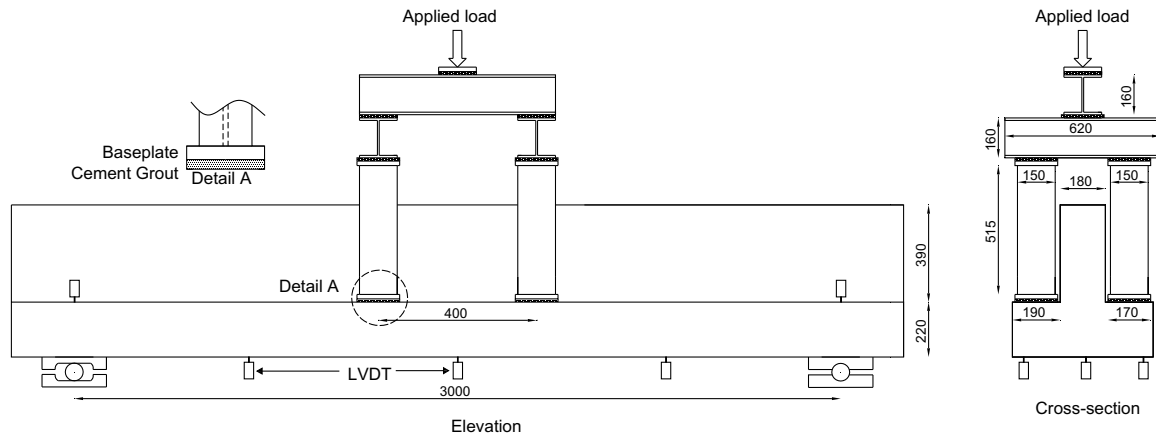


Fig. 4. Schematic of test setup.

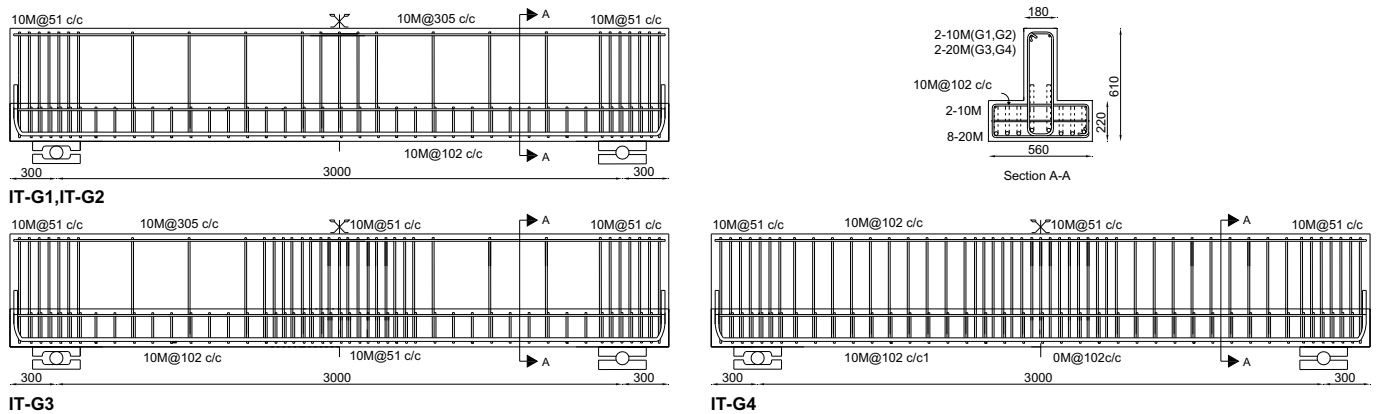


Fig. 5. Dimensions and details of reinforcement of control girders.

4.1. Specimens IT-G1 and IT-G2

The control girder IT-G1 and IT-G2 had the same dimensions and reinforcement contents and were tested up to the first sign of non-ductile failure mechanism. Stirrup spacing in the hanger zone was 102 mm whereas spacings of web-shear and flange reinforcements were 305 mm and 102 mm, respectively. The details of the longitudinal reinforcement of the girders are shown in the cross-section in Fig. 5. The objective of control specimens IT-G1 and IT-G2 was to examine the effectiveness of two anchoring techniques, used in specimens IT-G1R and IT-G2R, to strengthen the hanger zone.

4.2. Specimen IT-G3

The objective of specimens IT-G3 and IT-G3R was to determine the feasibility of strengthening the web-shear zone of inverted-T girders using CFRP sheets. The control girder had minimum shear reinforcement content, with 10 mm stirrups spaced at 305 mm. In order to avoid the possibility of failure in hanger or in flange, the reinforcement spacing in the hanger and punching zones was

decreased to 51 mm up to 370 mm on both sides from center of the girder as shown in Fig. 5.

4.3. Specimen IT-G4

The objective of specimens IT-G4 and IT-G4R was to determine the feasibility of strengthening the punching zone in the flanges of inverted-T girders using CFRP sheets. Therefore, the control girder had high web-shear and hanger reinforcement contents (10 M @ 102 mm and 10 M @ 51 mm, respectively) as shown in Fig. 5. The reinforcement content in the punching zone of the flanges was reduced by increasing the spacing of the flange reinforcement from 51 mm to 102 mm.

4.4. Anchorage of externally bonded FRP

Although the FRP composites have been used in many strengthening projects, there have always been concerns of debonding of FRP at high stress levels, or due to fatigue limits resulting from stress reversals as in the case of bridge girders. This is particularly important in non-rectangular shapes, i.e. L-, or T-shapes, where the possibility of peeling-off of

FRP is high at the intersection of web and flange due to minimal bonded length. Karantzakis et al. [12] conducted an experimental investigation of non-conventional confinement for L-shaped concrete columns using FRP. They concluded that the use of FRP jackets without anchors results in limited increase in strength and deformability regardless of the FRP thickness due to the premature debonding at re-entrant corners. This premature failure can be prevented, thus insuring the full utilization of the FRP wrap strength, by using end anchorage such as embedding of FRP fan anchors or steel anchor bolts. Details of the FRP-anchors or fan type anchors used in the current experimental study are explained in the investigated rehabilitation schemes for the rehabilitated girders.

4.5. Specimen IT-G1R

A new rehabilitation scheme using CFRP sheets was used for girder IT-G1R in order to eliminate the non-ductile failure in the hanger zone observed in girder IT-G1. The girder was rehabilitated using 3 layers of Tyfo[®] SCH-11UP carbon fibre sheets [10]. The difficulties in proposing an FRP-rehabilitation scheme for inverted-T girders compared to regular rectangular girders is the inaccessibility at the locations of the loading points. Therefore, in order to ensure that the FRP sheets will not peel-off the concrete at high stress levels, FRP sheets were anchored to concrete as shown in Fig. 6. At the cross-section of the

rehabilitation scheme at locations away from the loading plates (section A–A) holes with depth of 3" were pre-drilled in the girder's web and flange to allow for anchoring using 3/8" threaded rods after applying FRP sheets. At the connection between the web and the flange, concave grout with a radius of 30 mm was used in order to allow for the smooth transition of stresses in the FRP sheets. A 6 mm thick curved angle plate that is anchored to concrete was used to clamp the FRP in position as shown in the figure. At the cross-section of the rehabilitation scheme at locations of loading plates (section B–B), it was not possible to extend the FRP sheets to the top of the flange. Another anchoring technique to anchor the FRP on the web of the girder to concrete was used. Two 3" deep pre-drilled holes and CFRP fan-type anchors or fibre anchors were used to transfer the stresses from the FRP to the girder as shown in Fig. 6.

4.6. Specimen IT-G2R

In specimen IT-G2R, another rehabilitation scheme as shown in Fig. 7 was used in order to examine the effect of absence of fibre anchors that are used in the loading zones of girder IT-G1R. The girder was rehabilitated using 3 layers of CFRP sheets that have an inverted U shape over the web portion of the girder along the web shear zones and at the locations of the loading plates. Flange portion of the girder were also wrapped completely beneath and

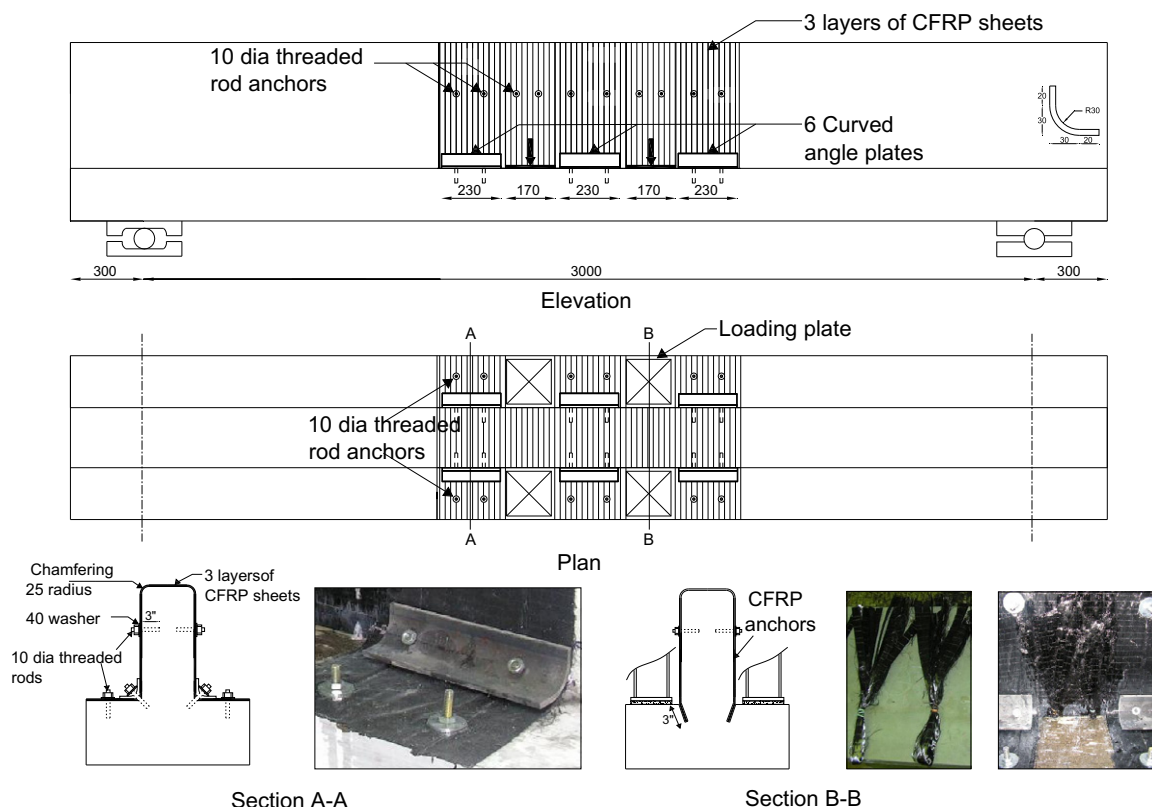


Fig. 6. Rehabilitation scheme for girder IT-G1R.

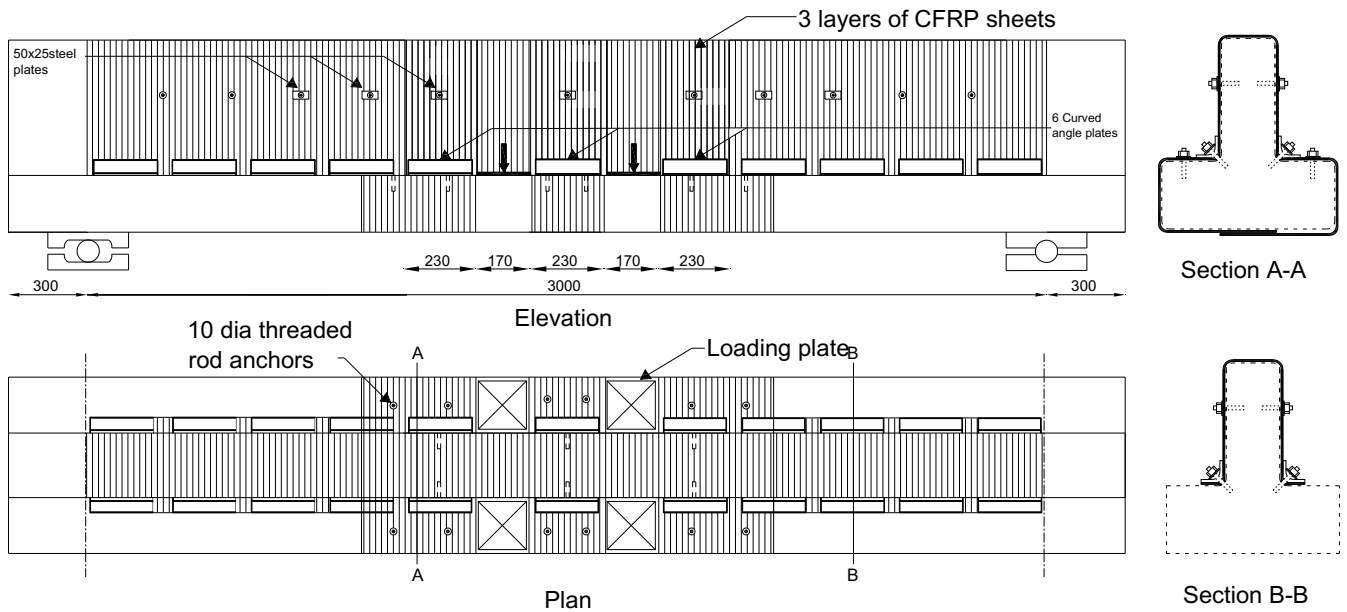


Fig. 7. Rehabilitation scheme for girder IT-G2R.

anchored on the top of the flange. The CFRP sheets were anchored using similar techniques used in IT-G1R except that fibre anchors were not used in the loading zones.

4.7. Specimen IT-G3R

The rehabilitation scheme shown in Fig. 8 was used for girder IT-G3R in order to eliminate the web-shear failure mechanism of girder IT-G3. The girder was rehabilitated using 3

layers of CFRP sheets with an inverted U shape in the web-shear zones. At the connection between the web and the flange, 6" curved angle plate were used similar to IT-G1R.

4.8. Specimen IT-G4R

Based on the failure mechanism of IT-G4, the rehabilitation scheme for girder IT-G4R shown in Fig. 9 was proposed to eliminate punching shear failure in the flanges

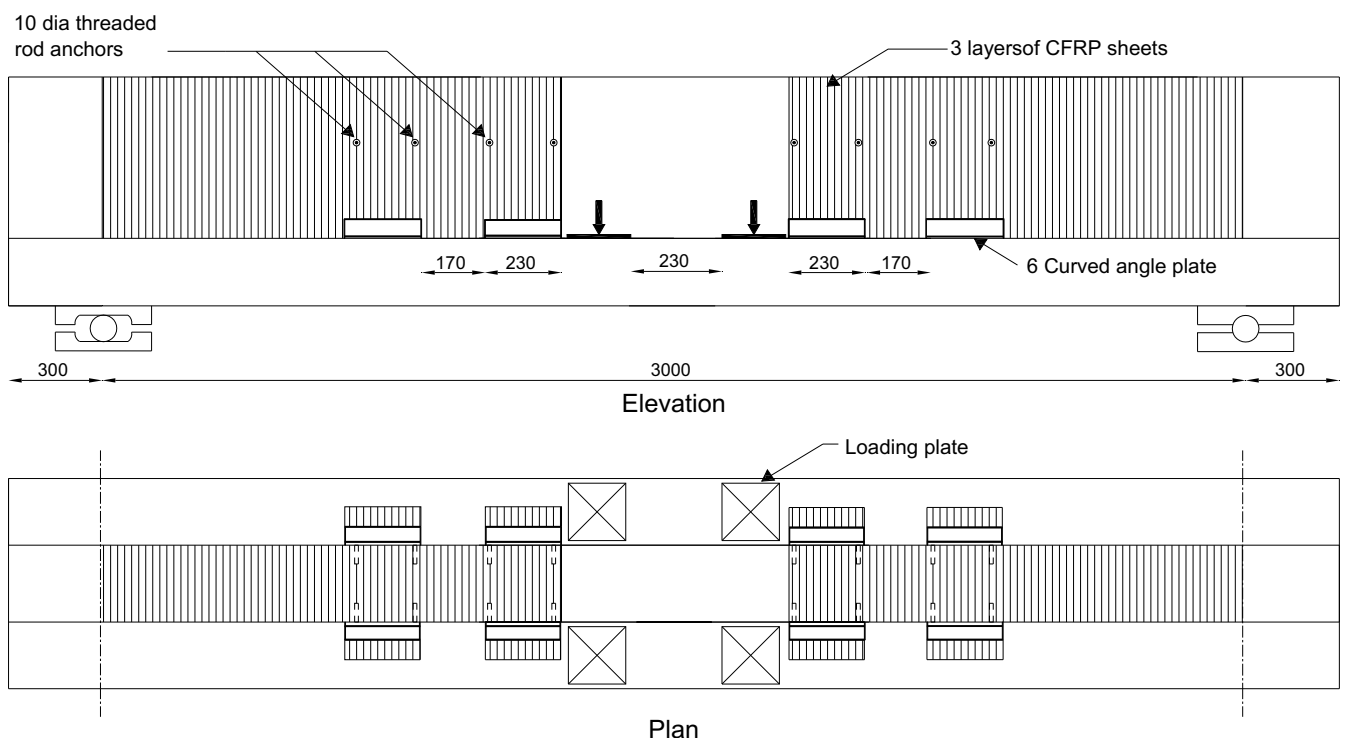


Fig. 8. Rehabilitation scheme for girder IT-G3R.

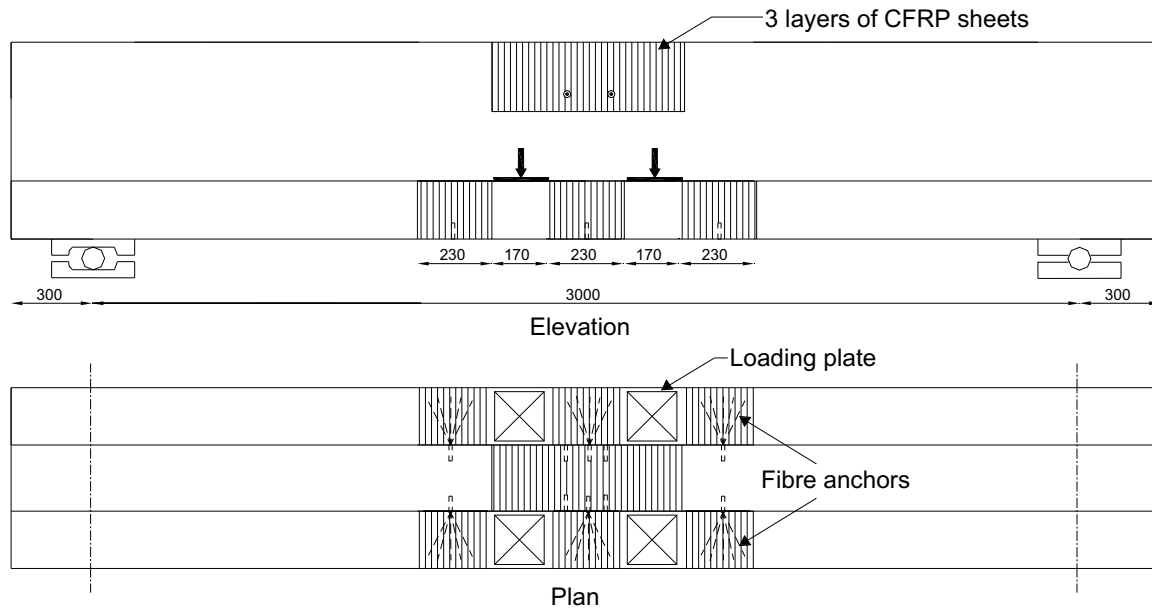


Fig. 9. Rehabilitation scheme for girders IT-G4R.

and to increase the compression capacity of the web by confining the compression zone in the pure flexure region between the loading points (i.e. web-compression zone). To eliminate punching failure mechanism in the flange, 3 layers of CFRP sheets that have a $\square$ shape were wrapped around the two flanges of the girder. In order to transfer the stresses from the CFRP wrap to the web of the girder, fibre anchors that were sandwiched between the layers of the CFRP wrap on both sides of the flange were embedded a distance of 3" into the web as shown figure. To increase the compression capacity of the web of the girder, 3 layers of CFRP sheets that have an inverted $\sqcap$ shape were wrapped over the web up to half of the web height above the flange and were anchored with threaded rods.

5. Materials

5.1. Steel reinforcement

The flexural reinforcement used as tension (bottom) reinforcement consisted of 20 M (19.5 mm nominal diameter) with cross-sectional area of 300 mm² deformed steel rebars. 10 M rebars (11.3 mm nominal diameter) with cross-sectional area of 100 mm² were used for other reinforcement in the web and flanges. Samples of the rebars were tested to obtain their tensile stress–strain relationship. The tested rebars had an average yield strain of 2200 microstrain, yield stress of 440 MPa and modulus of elasticity of 200 GPa.

5.2. Concrete

Ready mix concrete was ordered from the plant and was delivered to the laboratory. Concrete with specified compressive strength $f'_c = 30$ MPa, slump of 125 mm, which

provided adequate concrete workability during the concrete casting in the forms and maximum aggregate size of 20 mm was ordered from the ready mix plant. During casting, a total of 15 concrete cylinders (100 × 200 mm) were cast and cured under the same conditions as the test girders. The curing consisted of moist curing by covering the whole girder with burlap. Three cylinders were tested after 7, 14, and 28 days and another three were tested on the day of testing of the girders commenced. Table 1 shows the results for the concrete compressive and tensile strengths at 28 days of the tested girders.

5.3. CFRP sheets

The girder was rehabilitated using Tyfo[®] SCH-11UP unidirectional carbon fibre sheets [10] which was delivered in rolls and a two-component epoxy resin that were mixed and applied to the fibres to form the composite material. The carbon fibres are characterized by a very high tensile strength, a linearly elastic stress–strain relationship up to failure, and a modulus of elasticity slightly higher than that of steel. Table 2a shows the typical dry fibre properties

Table 1

Concrete compressive and tensile strengths at 28 days of the tested girders

Specimen	f'_c (MPa)	f_t (MPa)
IT-G1	32	3.32
IT-G2	31	3.27
IT-G3	33	3.34
IT-G4	33.5	3.33
IT-G1R	32.7	3.29
IT-G2R	31.6	3.20
IT-G3R	33.5	3.37
IT-G4R	33.9	3.32

Table 2a
Typical dry fibre properties (as provided by the supplier Fyfe [10])

Property	Typical test value
Tensile strength	3.79 GPa
Tensile modulus	230 GPa
Ultimate elongation	1.7%
Density	1.74 g/cm ³
Weight	298 g/m ²
Fibre thickness	0.127 mm

Table 2b
Composite gross laminate properties (as provided by the supplier Fyfe [10])

Property	ASTM method [5]	Typical test value
Ultimate tensile strength in primary fibre direction	D-3039	1062 MPa
Elongation at break	D-3039	1.05%
Tensile modulus	D-3039	102 GPa
Ultimate tensile strength 90° to primary fibre	D-3039	0
Laminate thickness	–	0.25 mm

while Table 2b shows the composite gross laminate properties as provided by the supplier [10].

6. Behaviour of the rehabilitated girders

6.1. Rehabilitation of hanger zone

Four tests were conducted on IT-G1 and IT-G2 in order to study the hanger failure mechanism. The failure mode of

the control specimens IT-G1 and IT-G2 tested in hanger zone was similar and was characterized by a non-ductile failure at load of 460 kN and a displacement of approximately 12 mm as shown in Fig. 10.

On the other hand, the rehabilitated specimen IT-G1R using the proposed rehabilitation scheme had a higher load carrying capacity of 585 kN and behaved in a more ductile manner with a deflection at maximum load of 44 mm, which corresponds to displacement ductility higher than 4. At failure, the rehabilitated girder had a mid-span deflection of 52 mm. The rehabilitation scheme was successful in eliminating hanger failure mechanism. Similarly it can be seen that the girder IT-G2R has a low stiffness but resulted in increase in the displacement ductility of approximately 1.8 when compared to that of un-rehabilitated specimen. Although it was expected that IT-G2R would have a ductile behaviour that is similar (or even better) than that of IT-G1R (due to its higher FRP content), yet the absence of the fibre anchors at the loading zones resulted in a premature failure without achieving high ductility limits. This verifies the need for anchoring the FRP at the flange-web intersection zone.

6.2. Rehabilitation of web zone

Girder IT-G3 was tested as a control specimen, while girder IT-G3R was tested after being rehabilitated with CFRP sheets. Since the control girder was weak in web-shear, it failed at 508 kN without increase in load carrying capacity. Fig. 11 shows the rehabilitated girder IT-G3R at failure and load versus mid-span deflection of girders

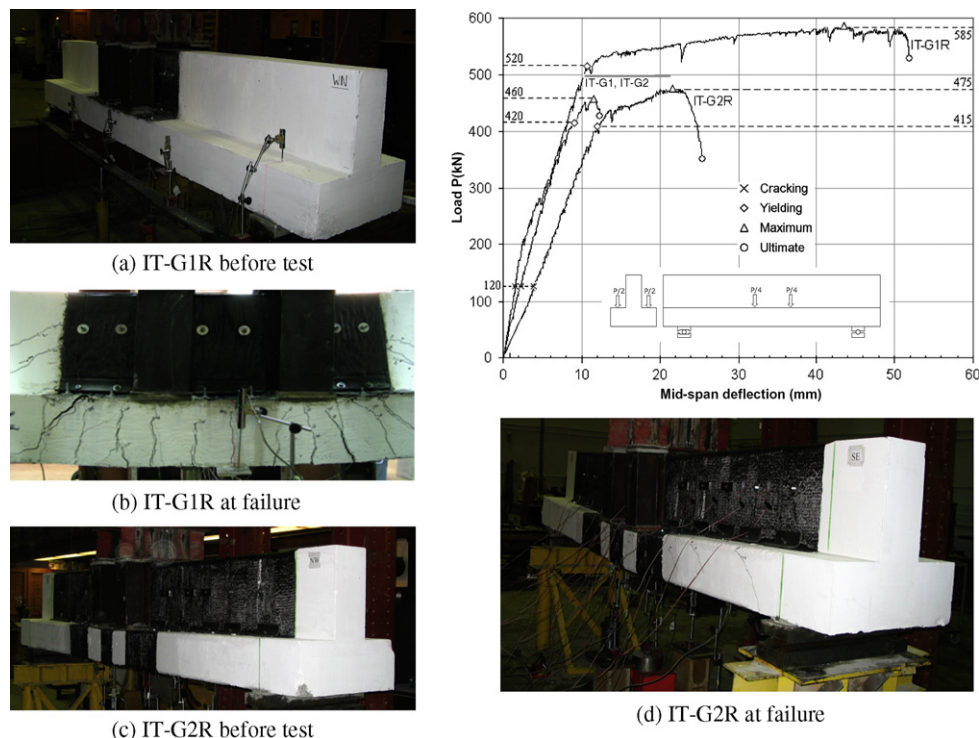


Fig. 10. Failure and load-deflection relationship for girders rehabilitated in hanger zone.

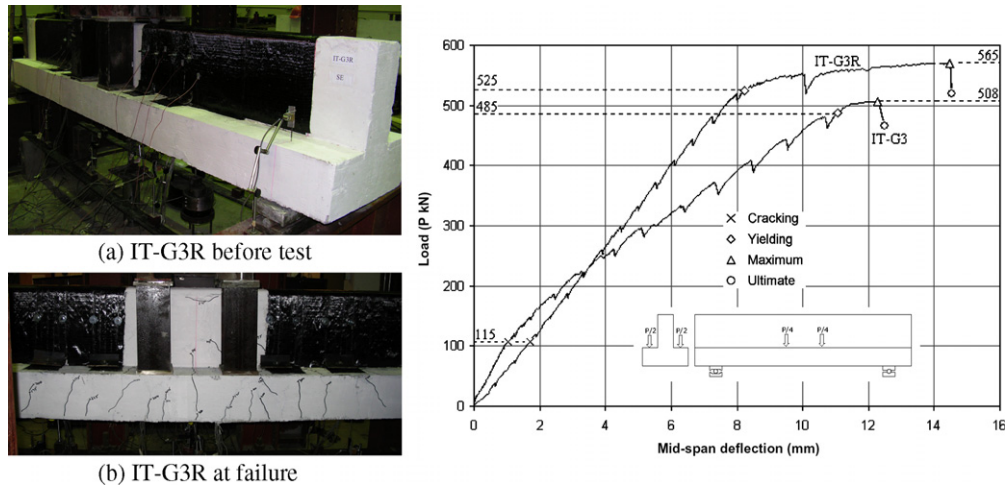


Fig. 11. Failure and load-deflection relationship for girder rehabilitated in web zone.

IT-G3 and IT-G3R. It can be seen that the rehabilitated girder failed at a higher load compared to the control specimen with a displacement ductility capacity of 1.81. The girder failed after reaching its flexural web-compression capacity. There were no signs of delamination of CFRP sheets from the concrete and fully bonded to the concrete at failure. The rehabilitation scheme was successful in eliminating the web-shear failure mechanism. The reason that IT-G3R did not reach high displacement ductility level compared to IT-G1R, can be attributed to the lack of confinement of concrete in the web-compression zone.

6.3. Rehabilitation of flange

Girder IT-G4 was tested as control specimen, failed due to non-ductile punching shear failure in the flange with a maximum deflection of 12 mm at a peak load of 515 kN as shown in Fig. 12, while the rehabilitated girder IT-G4R behaved in a more ductile manner with a carrying

load capacity of 653 kN. The rehabilitated girder had a displacement ductility higher than 4 after reaching its full flexural capacity. This shows that the rehabilitation scheme was successful in eliminating the punching shear failure mechanism.

The decrease in the stiffness, as well as the absence of initial stiffness up to the cracking point of concrete, for the rehabilitated girders (i.e. IT-G1R to IT-G4R) seen in Figs. 10–12 was expected due to the cracks formed upon loading the corresponding control specimens which resulted in a reduced stiffness of the rehabilitated ones upon reloading. Hence, it can be said that wrapping the girders with FRP did not contribute to their flexure stiffness, yet it increased the girders' capacities in the non-ductile modes, i.e. hanger, web, and flange, which consequently increased the girders' displacement ductility and load carrying capacities.

Table 3 shows a summary of the experimental results and the behaviour and failure modes of the eight tests.

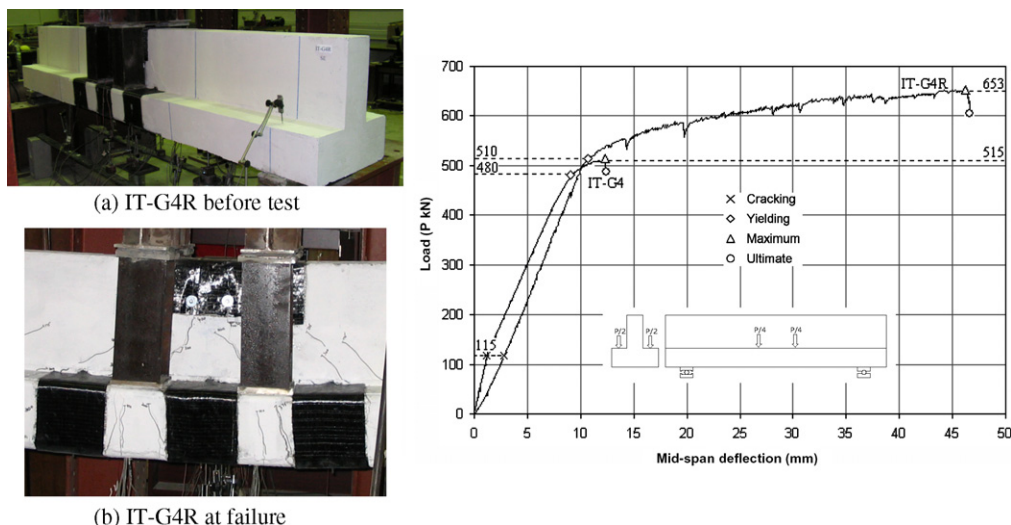


Fig. 12. Failure and load-deflection relationship for girder rehabilitated in flange zone.

Table 3
Comparison of the load and deflection capacities of the tested girders

	Maximum load, P (kN)	Deflection at max. load (mm)	Displacement ductility capacity $\mu_d = \Delta_m/\Delta_y$	Increase in load capacity $P_{\text{rehab}}/P_{\text{control}}$	Behaviour/failure mode
IT-G1, IT-G2	460	12	1.33	–	Non-ductile/hanger failure
IT-G3	508	12	1.10	–	Non-ductile/web shear failure
IT-G4	515	12	1.33	–	Non-ductile/punching shear failure
IT-G1R	585	44	4.19	1.27	Ductile/flexural
IT-G2R	475	22	1.83	1.03	Moderately-ductile/FRP debonding
IT-G3R	565	14.5	1.81	1.11	Moderately-ductile/flexural web-compression
IT-G4R	653	46.5	4.43	1.27	Ductile/flexural

7. Comparison between the predicted and experimental shear capacities

In general, the nominal shear capacity, V_n , of a RC beam rehabilitated with FRP composites results from the contributions of three mechanisms, namely, concrete V_c , transverse steel V_s , and FRP V_f such that

$$V_n = V_c + V_s + V_f. \quad (1)$$

In this study, the contributions of the concrete V_c and steel V_s mechanisms for a RC inverted-T girder are adopted from the equations proposed by Mirza and Furlong [14] and summarized in Appendix A.

On the other hand, the contribution of the FRP wrap of a total thickness t_{FRP} to the shear mechanism V_f is taken as that provided by ACI [4] as follows:

$$V_f = \frac{A_{fv} f_{fe} (\sin \alpha + \cos \alpha) d_f}{s_f}, \quad (2)$$

where $A_{fv} = 2nt_f w_f$; $f_{fe} = \varepsilon_{fe} E_f$; ε_{fe} = effective strain in CFRP taken as 0.004; E_f = tensile modulus of CFRP sheets; n = number of CFRP layers on one side of the beam; t_f = thickness of one layer of CFRP; and w_f/s_f = the ratio of the width of the CFRP strip to its spacing (=1 in case of continuous strip). An example on the substitution in the formulas for calculating the shear capacities $V_c + V_s$ and V_f for IT-G3R is given in Appendix B.

A well designed inverted-T girder (where the capacity of the non-ductile mechanisms exceeds those of the flexural ones) should experience a ductile behaviour through the yielding of the flexural reinforcement, followed by a strain hardening until the full flexural capacity is reached. On the other hand, if the capacity of any of the three non-ductile mechanisms, i.e. hanger, web-shear, or flange mechanisms, is less than the yielding or ultimate flexure capacities, the girder would have a non-ductile or moderately-ductile behaviour.

Table 4 shows the analytically predicted capacities of the three non-ductile mechanisms and the ductile flexural ones. The values are shown in terms of the shear capacity of the section. According to the four-point loading system of the tested girders, the shear capacity will be equal to half of the total load applied on the girder. An unrehabilitated inverted-T girder that is loaded with a monotonically increasing load is expected to trigger the mechanism with the least capacity. If the shear capacity of any of the hanger, web-shear, or flange mechanisms, $V_c + V_s$, of an unrehabilitated girder is less than the shear force corresponding to the yielding of the longitudinal reinforcement, a non-ductile failure is expected. Thus, the goal of the proposed FRP-rehabilitation schemes is to increase the shear capacity of the governing non-ductile mechanism of the girder by adding the component, V_f , such that the rehabilitated girder would experience a ductile failure. It should be noted that the

Table 4
Comparison of the predicted and experimental shear capacities (in kN) of the four rehabilitated girders

Failure mechanisms	IT-G1R				IT-G2R				IT-G3R				IT-G4R			
	Predicted		Exp.		Predicted		Exp.		Predicted		Exp.		Predicted		Exp.	
	$V_c + V_s$	V_f	V_n	$V = P_{\text{exp}}/2$	$V_c + V_s$	V_f	V_n	$V = P_{\text{exp}}/2$	$V_c + V_s$	V_f	V_n	$V = P_{\text{exp}}/2$	$V_c + V_s$	V_f	V_n	$V = P_{\text{exp}}/2$
Hanger	230	225	455		230	225	455	237.5 ^a	690		690		690		690	
Web shear	285		285	292.5 ^a	285	225	510		285	225	510		615		615	
Flange	320		320		320	125	445		320		320		260	125	385	
Flexure (ductile)	$V_{\text{flex,y}}$	270	270	260	270		270	207.5	270		270	262.5	270		270	255
	$V_{\text{flex,u}}$	335	335		335		335		335		335	282.5 ^a	335		335	326.5 ^a

$V_{\text{flex,y}}$ = shear force corresponding to yielding of the longitudinal reinforcement.

$V_{\text{flex,u}}$ = shear force corresponding to ultimate flexural capacity of the girder.

Exp. = experimental result.

P_{exp} = total applied load on the four pads.

^a Failure.

values of the shear forces corresponding to yielding of the longitudinal reinforcement and the ultimate flexure capacities, $V_{\text{flex},y}$ and $V_{\text{flex},u}$, respectively, were obtained from a sectional analysis using the Response-2000 [7] program.

Girder IT-G1 that had inadequate hanger and web-shear reinforcement experienced a non-ductile failure mechanism in the hanger zone at a shear capacity of 230 kN (i.e. load of 460 kN), which is similar to the predicted analytical capacity, $V_c + V_s$, of the hanger mechanism shown in Table 4. Using 3 layers of anchored CFRP sheets in the hanger zone of girder IT-G1R resulted in an increase in its hanger shear capacity from 230 kN to 455 kN. Consequently, upon loading IT-G1R, the flexure mechanism was triggered through yielding of the longitudinal reinforcement at a shear force of 260 kN (load of 520 kN). IT-G1R had a ductile behaviour with a displacement ductility higher than 4. The rehabilitated girder failed at a shear force of 292.5 kN (load of 585 kN), which is slightly higher than the predicted capacity of the web-shear mechanism of 285 kN. It should be noted that it was expected that the girder could have reached a higher load capacity (i.e. to reach the predicted shear force corresponding to ultimate flexural capacity of the girder of 335 kN) if the web-shear mechanism was rehabilitated.

Although girder IT-G2R was rehabilitated such that the FRP would contribute to the shear capacity of the three non-ductile mechanisms, as shown in Table 4, in an attempt to develop the ultimate flexure capacity of the girder (that corresponds to a shear force of 335 kN), yet the anchoring technique of the CFRP sheets using curved steel angles at the web-flange intersection in the hanger zone did not fully utilize the shear capacity contribution of CFRP sheets. The specimen failed at a load of 475 kN (shear capacity of 237.5 kN). After reaching the load level of 475 kN, the sudden debonding of the CFRP sheets resulted in sudden reduction in the total hanger resistance of the girder which lead to the premature failure of the girder without fully utilizing the contribution of FRP sheets.

Girder IT-G3 that had inadequate web-shear reinforcement experienced a non-ductile shear failure mechanism in the web-shear zone at a shear capacity of 254 kN, which is about 10% less than the predicted shear capacity of the web shear mechanism of 285 kN. Using 3 layers of anchored CFRP sheets in the web-shear zone of girder IT-G3R resulted in an increase in its predicted web-shear capacity and consequently the girder was able to develop a ductile behaviour in flexure. The longitudinal reinforcement of the girder yielded at a load of 525 kN (shear force of 262.5 kN). The girder failed after reaching its flexural web-compression capacity of 565 kN ($V = 282.5$ kN) which is less than the analytically predicted and by about 16%. It should be noted that it was expected that the full flexural capacity, with a higher displacement ductility capacity, could have been reached if the web-compression zone was confined (as was observed in girder IT-G4R).

Girder IT-G4 that had inadequate flange reinforcement experienced a non-ductile punching shear failure mechanism

in the flange zone at a punching shear capacity of 275.5 kN (load of 515 kN), which is close to the predicted shear capacity of the flange mechanism of 260 kN. Using 3 layers of anchored CFRP sheets in the punching zone, along with 3 layers confining the web-compression zone, resulted in an increase in its predicted shear capacity to be 385 kN, which is higher than the flexure capacity, thus enforcing a flexure ductile behaviour through yielding of the tension reinforcement at a corresponding shear force of 255 kN (which is close to the predicted value of 270 kN). The rehabilitated girder was capable of eliminating the non-ductile flange mechanism, having a displacement ductility capacity higher than 4, and developing the ultimate flexural capacity of the girder at a shear force of 326.5 kN which is close to the predicted value of 335 kN.

8. Conclusions

The current research investigates the possibility of using carbon fibre-reinforced polymer (CFRP) wraps in the rehabilitation of existing RC inverted-T bentcaps to eliminate non-ductile failure mechanisms in the hanger, web, and flange zones such that the girders could develop their flexural capacity and behave in a more ductile manner. Eight tests were conducted on four 1/3rd-scale RC inverted-T girders. The following conclusions are drawn from the experimental study:

1. Rehabilitating a RC inverted-T girder that have inadequate hanger and web-shear reinforcement using 3 layers of anchored CFRP sheets in the hanger zone eliminates the non-ductile shear-compression failure mechanism, increases the displacement ductility capacity up to 4, and increases the load carrying capacity of the girder by about 20%.
2. Rehabilitating a RC inverted-T girder that has inadequate web-shear reinforcement using 3 layers of anchored CFRP sheets in the web-shear zone eliminates the non-ductile web-shear failure mechanism, increases the displacement ductility capacity up to 1.8, and increases the load carrying capacity of the girder by about 10%.
3. Rehabilitating a RC inverted-T girder that has inadequate punching shear capacity of the flanges with 3 layers of anchored CFRP sheets in the punching and web-compression zones eliminates the non-ductile failure mechanism, increases the displacement ductility capacity up to 4, and increases the load carrying capacity by about 20%.
4. Confining the web-compression zone of RC inverted-T girders ensures the formation of the full flexural capacity of the girder.
5. Anchoring the CFRP sheets that are epoxied on the web or the flange into the girder using sandwiched (fan type) CFRP fibre anchors that are aligned with the CFRP sheets' fibres showed better performance compared to not using it in the loading zone.

It should be noted that, while the test results of the rehabilitated girders are encouraging, a large number of parameters need to be studied and verified experimentally and analytically in order to develop design guidelines for the rehabilitation of RC inverted-T girders using FRP composites.

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Appendix A

This section explains the empirical formulas proposed by Mirza and Furlong [14] for the design of RC inverted-T girders and different failure mechanisms. The formulas are used to calculate the values of the sum of the contributions of the concrete and steel, i.e. $V_c + V_s$, for the hanger, web shear, and flange mechanisms for the four rehabilitated specimens shown in Table 4. According to the four-point loading system in the current experimental work, the sum of the contribution of the concrete and steel mechanisms that would resist the failure mechanisms described in Sections A.1 and A.3 will be twice the ultimate concentrated load acting on one bearing pad $P_{u,p}$, i.e. $V_c + V_s = 2P_{u,p}$.

A.1. Hanger failure of stirrups (Hanger mechanism)

To achieve safe delivery of flange loads into the web, the necessary hanger reinforcement should satisfy the following two equations proposed by Mirza and Furlong [14]:

$$\frac{A_v}{s} = \frac{2P_{u,p} - 0.332\sqrt{f'_c}b_f d_f}{f_y(B + 2d_f)}. \quad (\text{A.1a})$$

In the above equation: b_f = overall flange width; d_f = flange depth from top of flange to center of bottom longitudinal reinforcement; $P_{u,p}$ = ultimate concentrated load acting on one bearing pad; and $\frac{A_v}{s}$ = cross-section area of both legs of a web stirrup at the hanger zone divided by the spacing of stirrups.

And

$$\frac{A_v}{s} = \frac{2P_{u,p}}{f_y S}, \quad (\text{A.1b})$$

where S is the spacing between the stringers along the girder length.

A.2. Shear resistance in the web (web shear mechanism)

The shear force was assumed to be resisted by forces in the transverse steel, referred to as the steel contribution, V_s , such that

$$V_s = \frac{A_{vw}f_y d}{s_{vw}}. \quad (\text{A.2})$$

As in the tradition approach, the concrete contribution to the shear strength is related to square root of the concrete compressive strength ($\sqrt{f'_c}$) such that

$$V_c = 0.2\sqrt{f'_c}b_w d, \quad (\text{A.3})$$

where $\frac{A_{vw}}{s_{vw}}$ = cross-section area of both legs of a web stirrup divided by the spacing of stirrups.

A.3. Flange mechanism

Mirza and Furlong [14] identified four possible sources of non-ductile failure in the flange of an inverted-T girder. The following subsections summarize the types and governing formulas of the four types of flange mechanisms, which are abbreviated as F1–F4. The least capacity of the four mechanisms will govern the Flange failure mechanism capacity of the inverted-T girder.

A.3.1. Punching shear in the flange (flange F1)

The flange should be deep enough to prevent punching shear failure. This can be achieved by satisfying the following equation proposed by Mirza and Furlong [14] for effective flange depth (d_f) from the top face of the flange to the top of the bottom layer of transverse reinforcement in the flange.

$$0.332\sqrt{f'_c}(B_p + 2d_f)d_f = P_{u,p}. \quad (\text{A.4})$$

In the above equation: d_f = effective depth from top of flange to center of bottom flange reinforcement; $B_p = B + 2B_w$, where B = length of bearing pad along edge of flange and B_w = width of bearing pad perpendicular to beam axis; and $P_{u,p}$ = ultimate concentrated load acting on one bearing pad.

Mirza and Furlong [14] indicated the Eq. (A.4) is valid if the spacing between the stringers in the longitudinal direction, S , is bigger than $(B_p + 2d_f)$, which if not satisfied implies the use of a modification factor α_L to the formula, where $\alpha_L = \frac{\frac{S}{2} + \frac{B}{2} + B_w + d_f}{B_p + 2d_f} \leq 1$. Similarly, a modification factor α_T should be applied to the left-hand-side of Eq. (A.4) if the distance between the center of the loading pads in the transverse direction is less than $2d_f + B_w$, such that $\alpha_T = \frac{b_w + 2a}{2d_f + B_w} \leq 1$, where a = distance from face of web to center of bearing pad.

A.3.2. Bracket-type shear friction in the flange (flange F2)

The effective depth of flange, d_f , from centroid of top layer of flange transverse reinforcement to the bottom of the flange reinforcement that is required to fulfil shear friction requirements was proposed by Mirza and Furlong [14] to satisfy the following equation:

$$0.2f'_c d_f (B + 4a) = P_{u,p}. \quad (\text{A.5})$$

In the above equation $0.2f'_c$ = shear strength of concrete resisting shear friction.

A.3.3. Transverse reinforcement required to satisfy shear friction in the flange (flange F3)

The transverse reinforcement required to satisfy shear friction should be placed within a distance $2a$ each side from the edge of a bearing pad. The area of cross-section of such reinforcement, A_{vf} , should satisfy the following expression:

$$\mu A_{vf} f_y = P_{u,p}. \quad (\text{A.6})$$

In the above equation: μ = coefficient of sliding friction taken as 1.4 for normal weight concrete cast monolithically; and f_y = specified yield strength of reinforcement.

A.3.4. Flange transverse reinforcement required to resist flexural tension (Flange F4)

The flange transverse reinforcement placed in the top layer also resists flexural tension, which is caused by the cantilever action of the flange, and should satisfy the following equation:

$$0.8d_f A_{sf} f_y = P_{u,p} a. \quad (\text{A.7})$$

In the above equation: A_{sf} = cross-section area of transverse reinforcement required to resist flexural tension; and $0.8d_f$ = effective distance between centroid of compression and centroid of tension ($j d_f$) for calculating flexural reinforcement in flange.

Appendix B

The following is a numerical example on the calculation of the predicted shear capacities of specimen IT-G3R, $V_c + V_s$ and V_f , that are listed in Table 4.

B.1. Hanger failure mechanism

Solving for $2P_{u,p}$ in Eq. (A.1a), we get

$$2P_{u,p} = \frac{A_v}{s} \cdot f_y \cdot (B + 2d_f) + 0.332\sqrt{f'_c} b_f d_f$$

therefore

$$2P_{u,p} = \frac{200}{51} \times 440 \times (170 + 2 \times 180) + 0.332\sqrt{33} \times 560 \times 180 = 1105 \text{ kN}.$$

Solving for $2P_{u,p}$ in Eq. (A.1b), we get

$$2P_{u,p} = \frac{A_v}{s} \times f_y S$$

therefore

$$2P_{u,p} = \frac{200}{51} \times 440 \times 400 = 690 \text{ kN}.$$

The least of the two above values, i.e. $2P_{u,p} = 690 \text{ kN}$, is more critical. Consequently, the shear capacity of the hanger mechanism is $V_c + V_s = 2P_{u,p} = 690 \text{ kN}$.

B.2. Web shear mechanism

The contribution of concrete and steel can be obtained by adding Eqs. (A.3) and (A.2), respectively, i.e. $V_c + V_s = 0.2\lambda\sqrt{f'_c}b_w d + \frac{A_v f_y d}{s}$.

Therefore

$$V_c + V_s = 0.2 \times 1 \times \sqrt{33} \times 180 \times 570 + \frac{200 \times 440 \times 570}{305} = 285 \text{ kN}.$$

B.3. Flange mechanism

There are four possible flange mechanisms, F1–F4. The flange mechanism capacity will be governed by the least value of the four mechanisms.

B.3.1. Punching shear (flange F1)

Solving for $P_{u,p}$ in Eq. (A.2) and applying the modification factors for the spacing in the longitudinal and transverse directions of the girder α_L and α_T , respectively, we obtain

$$P_{u,p} = 0.332\sqrt{f'_c}(B_p + 2d_f)d_f\alpha_L\alpha_T,$$

where

$$\alpha_L = \frac{\frac{s}{2} + \frac{B}{2} + B_w + d_f}{B_p + 2d_f} = \frac{\frac{400}{2} + \frac{170}{2} + 170 + 190}{(170 + 2 \times 170) + 2 \times 190} = 0.725$$

and

$$\alpha_T = \frac{b_w + 2e}{2d_f + B_w} = \frac{180 + 2 \times 95}{2 \times 190 + 170} = 0.673.$$

Therefore

$$P_{u,p} = 0.332\sqrt{33}(510 + 2 \times 190) \times 190 \times 0.725 \times 0.673 = 160 \text{ kN}.$$

Substituting the above values, we get the shear capacity of the punching shear of the flange $V_c + V_s = 2P_{u,p} = 320 \text{ kN}$.

B.3.2. Bracket-type shear friction (flange F2)

The ultimate concentrated load acting on one bearing pad $P_{u,p}$ to cause a bracket-type shear friction in the flange can be estimated using Eq. (A.5), such that

$$P_{u,p} = 0.2f'_c d_f S = P_{u,p} = 0.2f'_c d_f \times \text{least of } \{(B + 4a), S\} = 0.2 \times 33 \times 195 \times 400 = 515 \text{ kN}.$$

Therefore, the shear capacity of the bracket-type shear friction failure of the flange $V_c + V_s = 2P_{u,p} = 1030 \text{ kN}$.

B.3.3. Transverse reinforcement required to satisfy shear friction (flange F3)

Eq. (A.6) states that $P_{u,p} = \mu A_{sf} f_y$.

In our case, there are eight bars that contribute in a distance of $(\frac{B+4a}{2} + \frac{s}{2}) = (\frac{170+4 \times 95}{2} + \frac{400}{2}) = 475 \text{ mm}$. Therefore $P_{u,p} = 1.4 \times 800 \times 440 = 490 \text{ kN}$.

Substituting the above values, we get shear resistance $V_c + V_s = 2P_{u,p} = 980$ kN.

B.3.4. Flange transverse reinforcement required to resist flexural tension (flange F4)

According to Eq. (A.7), $P_{u,p} = \frac{0.8d_f A_{sf} f_y}{a}$.

There are nine bars that contribute in a distance of

$$\left(\frac{B + 5a}{2} + \frac{400}{2}\right) = \left(\frac{170 + 5 \times 95}{2} + \frac{400}{2}\right) = 523 \text{ mm.}$$

Therefore

$$P_{u,p} = \frac{0.8 \times 195 \times 900 \times 440}{95} = 650 \text{ kN.}$$

Substituting the above values, we get the shear capacity of the flange transverse reinforcement that resist flexural tension $V_c + V_s = 2P_{u,p} = 1300$ kN.

From the above, it can be seen that the shear capacity $V_c + V_s$ of the four flange mechanisms F1–F4 are 320, 1030, 980, and 1300 kN, respectively. Therefore, the critical mechanism that would govern a failure in the flange will be the punching shear with a shear capacity of 320 kN.

B.4. Contribution of the FRP wrap to the shear capacity

Eq. (2) can be simplified to be

$$V_f = \frac{(2nt_f w_f) \times (\epsilon_{fe} E_f)(\sin \alpha + \cos \alpha) d_f}{s_f}.$$

Thus

$$V_f = (2nt_f) \times (\epsilon_{fe} E_f)(\sin \alpha + \cos \alpha) d_f,$$

i.e.

$$V_f = (2 \times 3 \times 0.25 \text{ mm}) \times (0.004 \times 102 \times 10^3) \times (1 + 0) \times 370 = 225 \text{ kN.}$$

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