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Behaviour of prestressed box beam strengthened with CFRP under effect of strand snapping

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Original scientific paper

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Behaviour of prestressed box beam strengthened with CFRP under effect of strand snapping

The study aims to evaluate behaviour of a prestressed box beam strengthened with the Carbon Fibre Reinforced Polymer (CFRP) under the snapping of prestressed strands. Three external prestressed box beams are tested until failure. The evaluation focuses on the load carrying capacity, cracks pattern and width, and torsional capacity of the beam. The results show that the use of CFRP causes significant reduction of beam deflection and an increase of its loading capacity under snapping effects. The cracks do not appear at the bottom of the beam where the CFRP is placed, and the torsion stiffness of the beam increases considerably.

Key words:
prestressed box beams, combined load, strands snapping, CFRP strengthening, load capacity

Izvorni znanstveni rad

Azrul A. Mutalib, Mohamed H. Mussa, Aizat Mohd Taib

Utjecaj pucanja užadi na ponašanje prednapete sandučaste grede ojačane CFRP-om

U radu se analizira ponašanje prednapete sandučaste grede ojačane polimerom s ugljičnim vlaknima (CFRP) za slučaj pucanja prednapetih užadi. Tri vanjske prednapete sandučaste grede ispitane su do sloma. Analiza je provedena s naglaskom na nosivost, širinu i obrazac širenja pukotina, te na torzijsku čvrstoću grede. Rezultati pokazuju da primjena CFRP-a dovodi do znatnog smanjenja progiba i do povećanja nosivosti u slučaju pucanja užadi. Pukotine se ne pojavljuju na dnu grede na koju je postavljen CFRP, a bitno je povećana i krutost grede na torziju.

Ključne riječi:
prednapete sandučaste grede, složeno opterećenje, pucanje užadi, ojačanje CFRP-om, nosivost

Wissenschaftlicher Originalbeitrag

Azrul A. Mutalib, Mohamed H. Mussa, Aizat Mohd Taib

Auswirkung von Kabelrissen auf CFK-verstärkte vorgespannte Kastenträger

Die Abhandlung analysiert das Verhalten von kohlefaserverstärkten (CFK) Kastenträgern im Falle eines Risses der vorgespannten Seile. Getestet wurden drei vorgespannte Kastenträger bis zum Bruch. Die Analyse wurde mit Schwerpunkt auf der Tragfähigkeit, der Breite und dem Muster der Rissbreite sowie auf der Torsionsfestigkeit des Trägers durchgeführt. Die Ergebnisse zeigen, dass die Anwendung von CFK zu einer signifikanten Verringerung der Durchbiegung und zur Erhöhung der Tragfähigkeit im Falle eines Kabelrisses führt. Die Risse treten nicht am Boden des Trägers auf, an welcher der CFK angebracht wurde, und auch die Steifigkeit des Trägers auf die Torsion wird erheblich erhöht.

Schlüsselwörter:
vorgespannte Kastenträger, komplexe Belastung, Kabelriss, Verstärkung durch CFK, Tragfähigkeit

1. Introduction

The first-ever bridge with external prestressing was built in Aue, Saxony, from 1935 to 1937, according to the Dischinger concept (DRP 727429) [1]. However, after the Second World War, the development of bridge structures mainly focused on internally post-tensioned systems due to their simplicity. Nevertheless, external prestressing was still applied on some bridges in Europe, particularly in Germany, but the corrosion resistance of external tendons was considered insufficient. In the mid 1980s, the corrosion protection system improved considerably and the external prestressing method was introduced as an alternative to conventional internal prestressing for special purposes such as strengthening, and as a general method with regard to the durability and maintenance. The good experience with the external prestressing method in the construction of bridges, and its maintenance benefits, were the main reasons for adopting this method as a standard method for the design of box beam bridges.

Several studies were conducted to investigate behaviour of externally prestressed box beams under various circumstances. Aimin Yuan et. al. [2] tested the bending strength of concrete box beams with several ratios of the number of internal tendons to the number of external tendons (6:2, 4:4 and 2:6). The results indicated that the ratio had a significant effect on the load-carrying capacity, ductility and failure mode of the beams. As more internal tendons were laid out, higher load-carrying capacity and better ductility were achieved. Therefore, the ratio of this hybrid tendon of no less than 1:1 was recommended. Ghallab [3] proposed a simplified method to calculate ultimate stress of external tendons in prestressed continuous concrete beams. The method was generally more accurate than other methods, but it was not suitable for beam strengthened using straight tendons. The ASSHTO [4], Naaman and Alkhairi [5], and Aravinthan et al. [6] equations were more reliable for this kind of beams. Moreover, the results indicated that the ACI 318 [7] and Eurocode 2 [8] equations were reasonably accurate for beams strengthened using external tendons within the section depth. Karayannis and Chalioris [9] reported that the design procedure of ACI 318 and Eurocode 2, based on allowable crack width, is a versatile tool for estimating the required partial prestressing force of a flexural reinforced concrete beam. Saibabu et. al. [10] studied performance under monotonic and cyclic loading of dry and epoxy jointed segmental prestressed box girders cast using segmental construction method. The results showed that the box girders with epoxy joint showed better performance than the dry jointed segmental box girder due to additional tensile strength in the joint region. Nevertheless, dry jointed bridges are preferred in some cases due to environmental constraints or site requirements.

Failure of structural members generally occurs as a result of poor workmanship during construction, inadequate design, corrosion of reinforcement, sudden or sustained temperature change, or inadequate or total lack of maintenance [11]. Structural failure usually starts with appearance of cracks, which begin to multiply and increase in width and length as the load or adverse effect increases with time. Cracks occur due to reduction in concrete alkalinity, which allows for oxidation of reinforcing steel. Therefore, engineers came up with an idea of 'strengthening' as a means to maintain and repair existing structural elements that could be salvaged on time without having to demolish the structure completely, thereby saving time and avoiding inconvenience to the user. Advantages of the maintenance by strengthening are that it can be done at a reasonable cost (usually cheaper than outright demolition) and carried out in the shortest possible time without causing undue obstruction and delay to daily activities.

Polymer fibres have been widely used to strengthen the box concrete beams of bridges when the main concern was to eliminate the corrosion problem associated with conventional steel reinforcement and, therefore, to reduce the maintenance cost considerably. Sakuraba et al. [6] investigated the strength and deformation characteristics of Carbon Fibre Reinforced Polymer (CFRP) box beams with various laminated structures under flexural load. The results showed that the strength was enhanced with a proper lamina proportion in longitudinal and diagonal directions, and three failure modes were observed: vertical cracking of web in specimens consisting of longitudinal and transverse laminas, delamination of web in specimens including diagonal laminas, and combination between vertical cracking and web delamination in the specimen exhibiting all lamina directions. Moreover, it was found that diagonal laminas were effective in increasing flexural stiffness of beams.

Gautam and Matsumoto [12] stated that CFRP can improve failure mechanism of CRFP box beams under bending and axial loads. Grace et al. [13] studied flexural response of a box beam bridge with several types of CFRP tendons. The dominant failure mode was concrete crushing in compression zone followed by immediate rupture of prestressing tendons/strands. A higher ultimate strength was exhibited by beams strengthened with CFRP. Xie et al. [14] investigated fatigue damage of reinforced concrete beams strengthened with the prestressed fibre reinforced polymer (FRP) under three-point bending. The results showed that the steel stress and fatigue damage can be reduced with the prestressed FRP, which could result in an extended lifespan of the beam. Furthermore, static test results revealed that the load carrying capacity of the beams with prestressed FRP is obviously greater than that of the beams with non-prestressed FRP, although the ductility of the beams was shown to be smaller. Prestressed box beams subjected to eccentric load and prestressed wire snapping are evaluated in the current study, and results of previous studies conducted in this field are given. Three beams are tested, the first is examined without snapping effects, while the second is snapped at the first crack. The third beam is strengthened with CFRP after snapping at the first crack. The results are presented in terms of the load carrying capacity, crack pattern and width, torsional capacity and modes of failure.

2. Experimental programs

2.1. Selection and design of an externally prestressed box beam

The box beam shown in Figure 1 was designed according to Eurocode 2 [8] where it is stated that the deformation of the whole member should be taken into consideration in order to calculate the increase in stress in the prestressing steel of unbonded tendons at the ultimate stage.

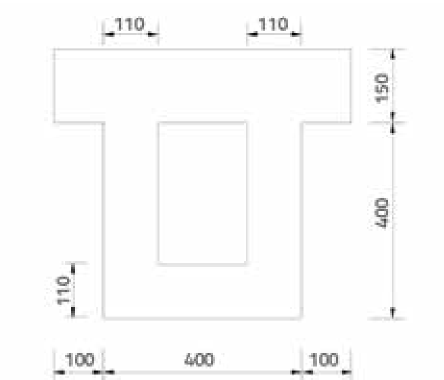

Figure 1. Cross section of box beam (all dimensions in mm)

If no detailed calculation is made, it may be assumed that the increase in stress from the effective prestress to the stress in the ultimate limit state is Δps. Eurocode 2 recommends the Δps value of 100 N/mm² and the tendon stress (fps) at the ultimate stage equals:

$$f_{ps} = f_{pe} + 100 \leq f_{py} \; [\text{MPa}] \tag{1}$$

where f_{pe} and f_{py} are effective prestressing stress and yield strength of prestressing steel, respectively.
The reinforcement and prestressing wire details are presented in Figure 2. Material properties of concrete, prestress tendons, and steel reinforcement used in the current study are shown in Table 1.

Table 1. Material properties of concrete, prestress tendons wire, and steel bars reinforcement

Material	Properties	Description
Concrete	Compressive strength (f_{cu})	50 N/mm² at 28 days
	Young modulus (E_c)	32000 N/mm²
Tendons	Number of wires	7
	Yield strength (f_{py})	1330 N/mm²
	Ultimate strength (f_{pu})	1900 N/mm²
	Poisson ratio	0.3
	Young modulus (E_s)	190000 N/mm²
	Coefficient of linear expansion (α)	12 x 10⁻⁶ per °C
Steel bars	Mild steel (R) High yield (Y)	250 MPa 460 MPa

2.2 Strengthening details of box beams

According to previous studies, CFRP has been found to be an effective fabric material for box beam strengthening. The heavy duty CFRP strengthening system (Sika Carbodur XS514)

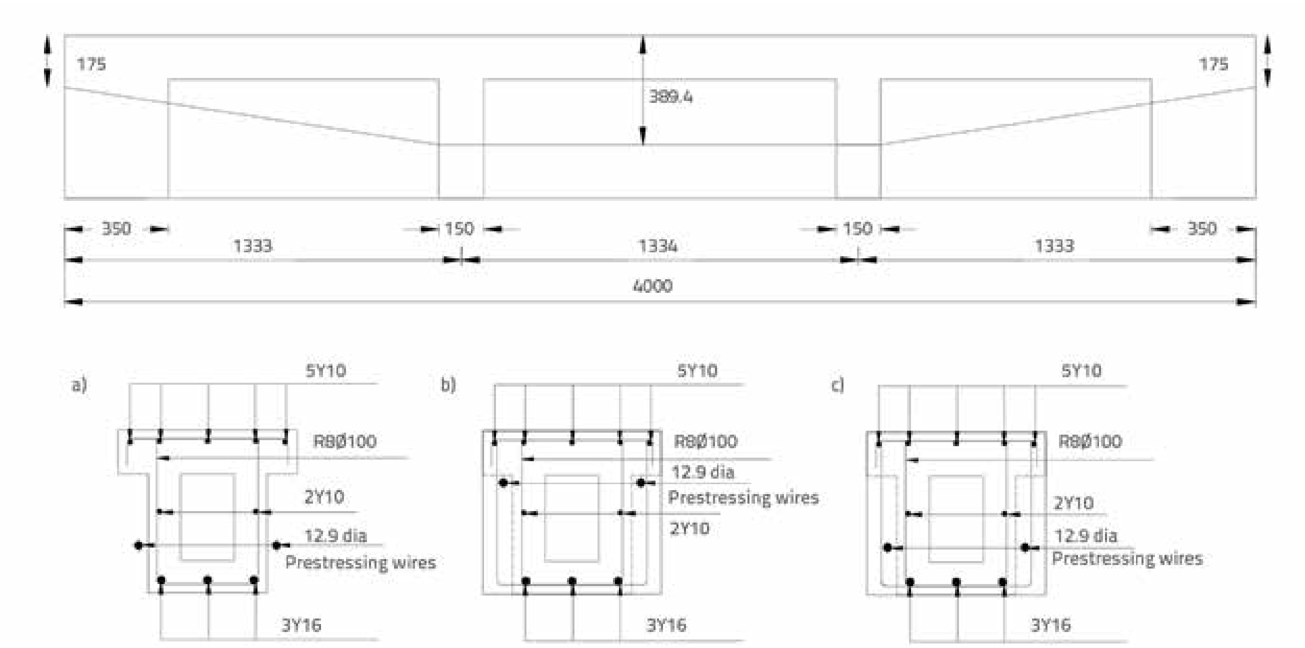

Figure 2. Reinforcement and prestressing wire details (all dimensions in mm)

provided by Sika Kimia Sdn. Bhd. Company, with the specification as shown in Table 2, was used in this study.

Table 2. Material properties of CFRP

Properties	Description
Width	50 mm
Thickness	1.4 mm
Cross-sectional area	70 mm^2
E-Modulus (mean value)	165000 N/mm^2
E-Modulus (minimum value)	>160000 N/mm^2
Tensile strength (minimum value)	>2200 N/mm^2
Tensile strength at break (mean value)	2400 N/mm^2
Strain at break (minimum value)	>1.35%

SIKADUR 30 was used as an adhesive material for the reinforcement bonding. The typical range of mechanical properties of this material is shown in Table 3.

Table 3. Properties of adhesive materials for reinforcement bonding

Properties	Description
Appearance	Component A = White paste Component B = Black paste Component A+B = Light grey Components A:B = 3:1 parts by weight
Mix ratio	Substrate and ambient: 10 do 35 °C
Application temperature	~ 1,7 kg per litre (Components A+B)
Density	12800 N/mm^2
Static e-modulus	Concrete failure (~15 N/mm^2)
Shear strength	Concrete failure (~ 4 N/mm^2)
Adhesive strength	9 x 10^{-5} per °C (-10 do +40 °C)
Thermal coefficient of expansion	Substrate and ambient: 10 to 35 °C

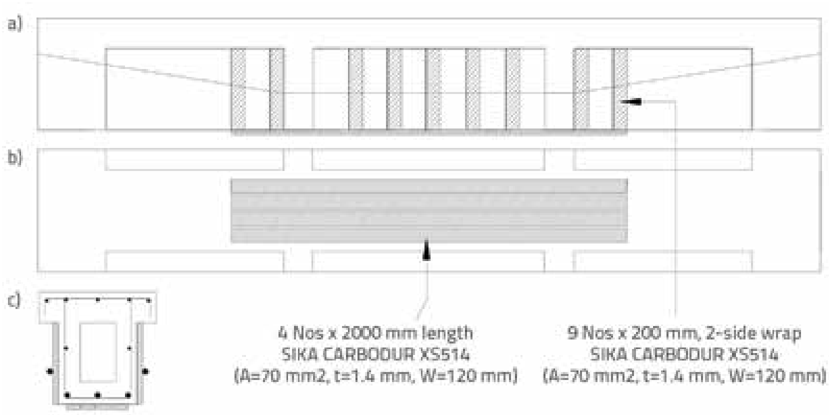

Figure 3. Wrapping configuration for CFRP strengthening

The wrapping configuration for flexural, shear, and torsional strengthening is shown in Figure 3, as based on calculation of the box beam strengthening with CFRP. Four *Sika CarboDur XS514*, 2 m in length, are located at the soffit of the beam, and nine Sika CarboDur XS514 are wrapped at 200 mm centre to centre. The configuration is dependent on initial calculation, and on estimation of the position where cracks are most likely to occur.

2.3. Preparation and prestressing of box beams

The moulds were made of plywood, and wood stiffeners were fixed at the sides of the mould to prevent outward collapse during the casting process. The reinforcement cages were prepared separately and had a polystyrene fabricated box fixed in the centre of the beam to simulate the hollow nature of the box beam. Strain gauges 5 mm in length were then placed on the reinforcement cage at expected maximum strain locations. Each reinforcement cage was then lifted into the formwork using the fork-lift. The ready-mix grade 50 concrete was ordered from the HANSON company and used to cast the beams. Concrete was poured in each beam mould and cast in layers. Concrete strength was determined by crushing cubes measuring 150 x 150 x 150 mm in four batches using the compressive "ELE" cube testing machine. Compressive strength readings were taken 7 - 28 days after beam casting. The beams were intensively cured for the first three days after casting, by means of perforated bags soaked in water. On the third day after casting, the formwork was removed from the sides and the curing process continued.
A total of three beams were externally prestressed by passing two seven-wire strands 12.9 mm in diameter through the already pre-determined holes encased with short PVC pipes to ensure smooth passage of each tendon. The tendons were anchored at each longitudinal end of the beam by wedges. The anchoring wedge used for the tendons was placed directly and was supported by a thick plate measuring 600 x 550 x 10 mm to prevent the end block of the concrete from crashing due to the prestressing force applied at the ends. All deviators measured 150 mm in width and were cast monolithically together with the rest of the beam. KYOWA strain gauges 5 mm in width in were attached to prestressed wires at specific locations of expected maximum strain. This was done to measure initial stresses during the prestressing process. All strain measurements were converted to stresses using the modulus of elasticity of 190 GPa. Two hydraulic prestressing jacks were used simultaneously to prestress the tendons to 6000 psi (138kN) which is equivalent to 70% of the total maximum prestressing force before breakage. The tendon stressing was conducted in stages: 0, 3000 and 6000 psi (0, 69, 138 kN). Strain readings were similarly taken after the end of prestressing.

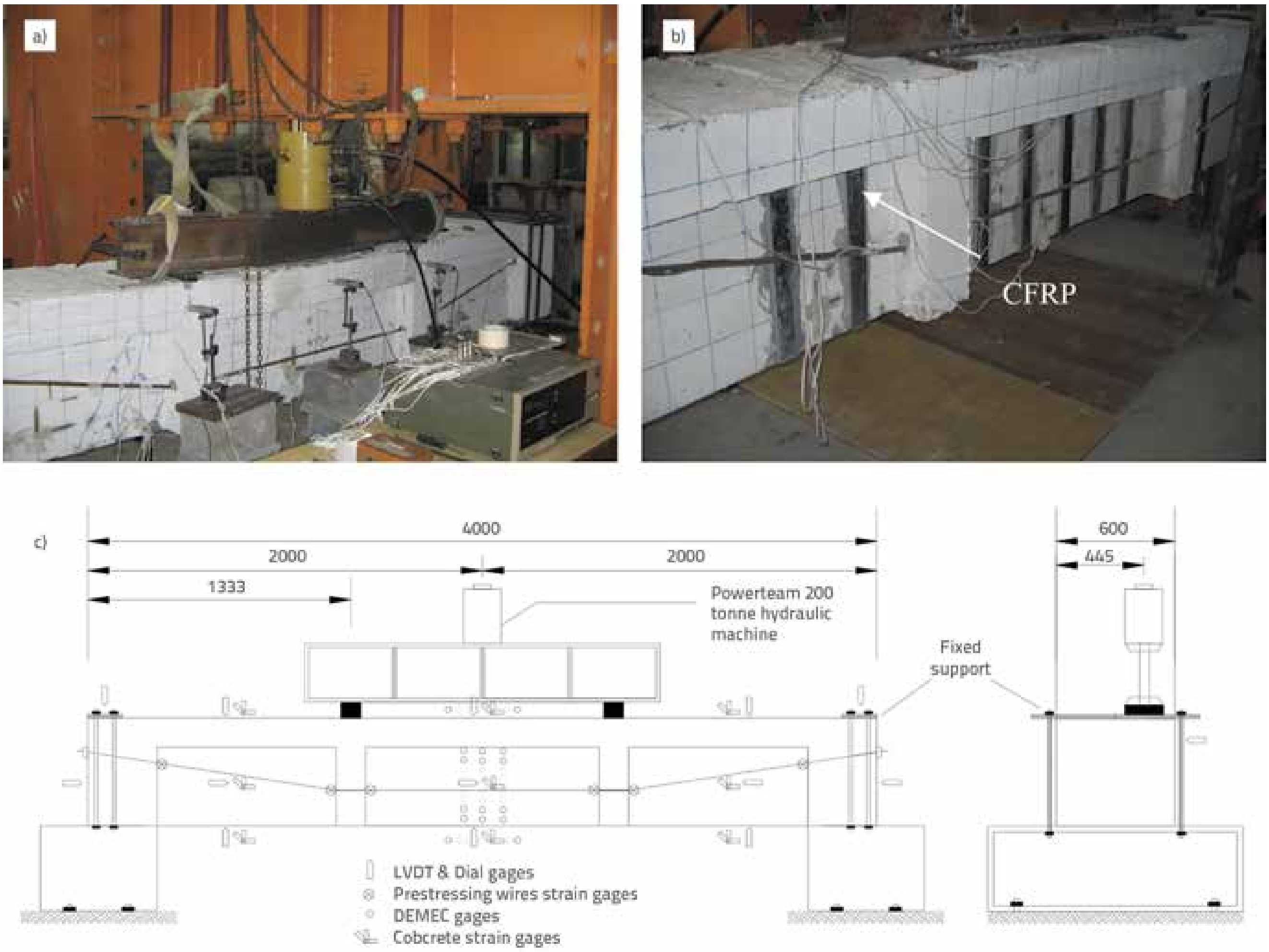

Figure 4. Typical test setup: a) PB1; b) PB3; c) sschematic view

2.4. Testing procedure

Three externally prestressed box beams were tested and symbolled as PB1 (control beam without snapping), PB2 (control beam with snapping), and PB3 (strengthened beam with CFRP under snapping). Test specimens are 4 m in length and 550 mm in depth. The typical setup of the testing procedure is shown in Figure 4. The beams were tested under a combined load applied at a distance of 145 mm from the centre of the cross-section of the beam by using ENERPAC hydraulic cylinder with a capacity of 200 tons.

For PB1 beam, the specimen was loaded slowly and measurements of ultimate load and strain of concrete, reinforcement, and prestressed wires, crack width and deflection, were recorded at each load increment until failure. For PB2 beam, the specimen was loaded and then stopped at the first crack to snap the prestressed wires at a loaded side in order to create great torsion as shown in Figure 5. The snapping at first crack was chosen because the first crack gives an indication of beam deterioration, whereas worse stress in the beam might occur at this point as stated by Dickson et. al [15]. The beam was also loaded until failure and similar measurements were recorded. In the last beam PB3, the specimen was loaded and then snapped at first crack and the CFRP strips 2 to 3 mm in thickness were immediately pressed down onto the concrete surface using compressed air.

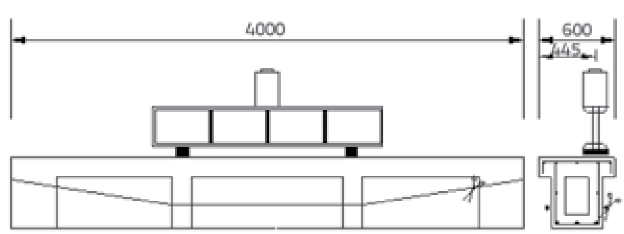

Figure 5. Setup for snapping of prestressing wires

3. Results and Discussions

3.1. Load-deflection response of box beams

The behaviour of studied beams under varied load eccentricity and snapping conditions was described in term of load-deflection curves, based on measurements by dial gauges and LVDT placed under the beam soffit as shown in Figure 6.

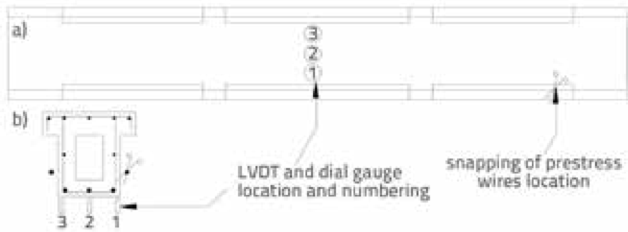

Figure 6. Location of LVDT and dial gauge

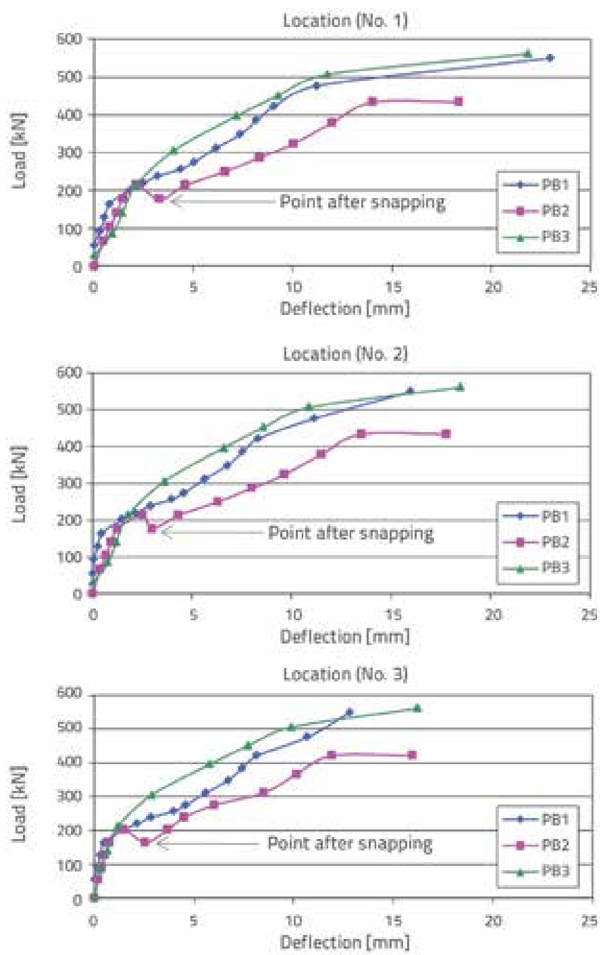

Figure 7. Load-deflection curves of beams at different locations

It is shown in Figure 7 that the PB3 beam has the highest load carrying capacity at mid-span compared to other beams, despite the snapping effect that was clearly noted in case of the PB2 beam. This behaviour is attributed to the existence of CFRP which greatly enhanced the strength of the box beam.

Beam deflection patterns show a similar initial stiffness until a load of 210 kN, when the initial cracking starts. However, after one-third of the ultimate load, these specimens exhibit a different level of ductility pattern. The maximum deflection observed in case of PB2 (control beam with snapping) especially after the point of snapping, clearly reflected the snapping effect which considerably lowered beam stiffness. The strengthening of PB3 beam with CFRP showed that the deflection curve could tremendously improve when the load is increased compared to the PB2. However, the maximum deflection was recorded at location number 1 situated near the snapping of prestressed wires due to torsional load.

3.2. Load versus change in tendon stress (Δfps)

The curves of load vs stress change in tendons (Δfp) was recorded at 12 positions in case of PB1 beam, as shown in Figure 8. The results show that the maximum change in tendon stress was recorded between positions 3 and 4, where it amounted to 733 N/mm^2 and 817 N/mm^2, respectively. The effect of torsional loading at the loaded side induced higher stresses in the prestressed wires especially at positions 1 to 6. The average change in stress of PB1 beam was 561.91 N/mm^2. The loaded side revealed an average change in stress of tendon wires of 601.5 N/mm^2, which was by 16 % higher than the value of 520.33 N/mm^2 recorded at the unloaded side, as shown in Figure 9.

For the PB2 beam, the change in stress of tendons was determined at 10 positions, as shown in Figure 10. It was found that an average change of stress in PB2 is 314.25 N/mm^2 which is by 78% lower compared to PB1. Moreover, the maximum change in stress of 424.32 N/mm^2 occurred at position 4 and the loaded side exhibited the higher change in tendons stress of 29 % when compared to the unloaded side, as shown in Fig. 11. A sudden reduction (zig-zag behaviour) in the load capacity of PB2 was observed among the most investigated positions, especially at the point directly after the load of 210kN due to snapping effect.

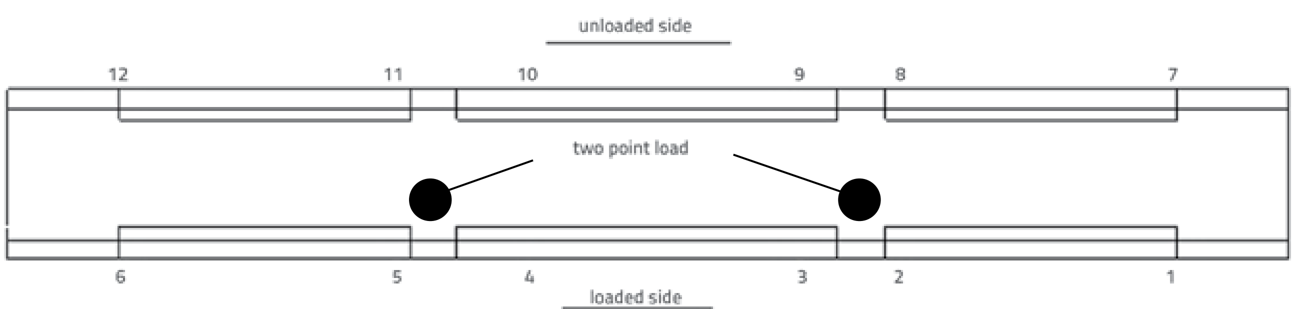

Figure 8. Positions of load versus stress change in tendons of PB1 beam

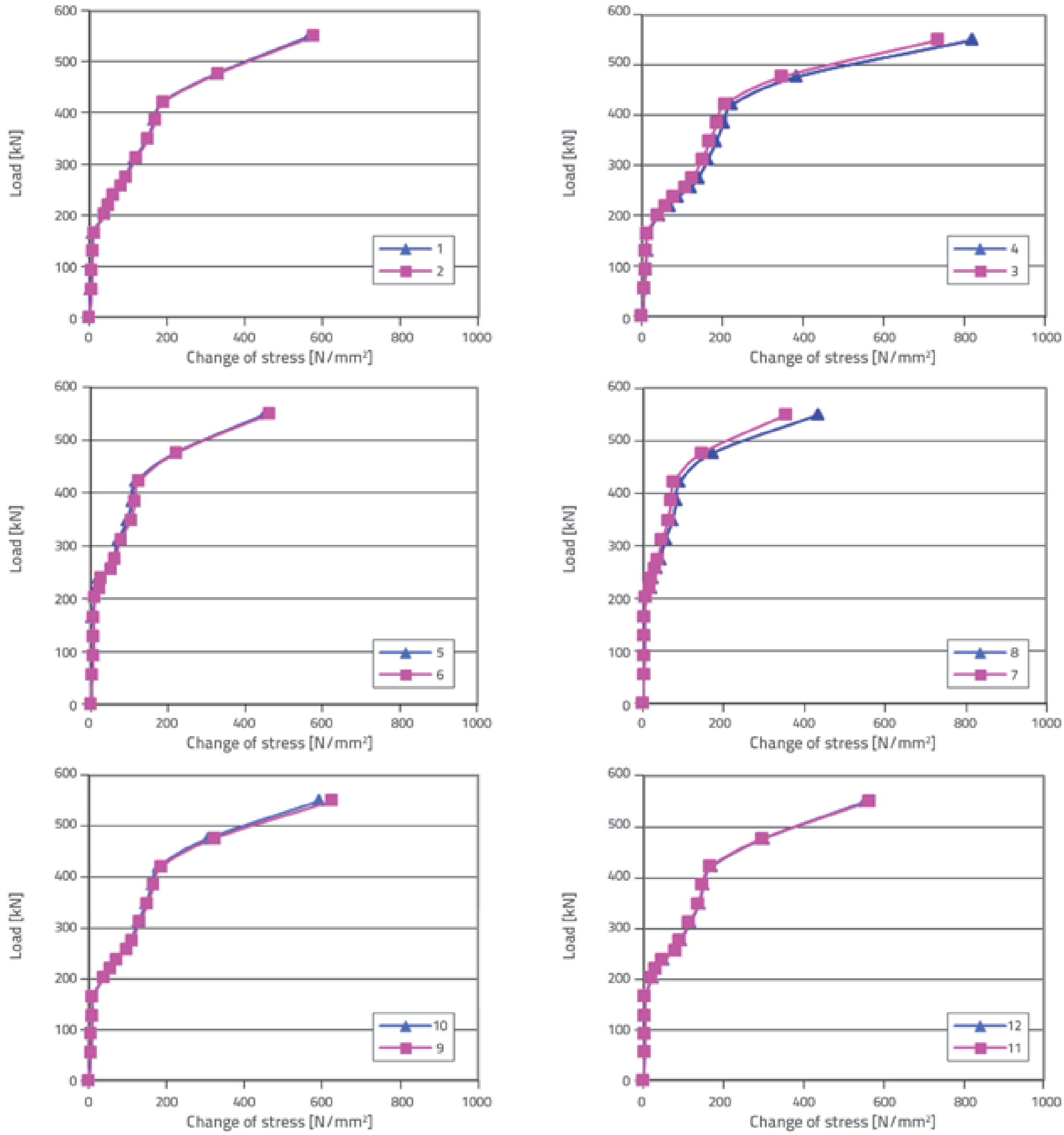

Figure 9. Curves of load versus change of stress for PB1 at various positions

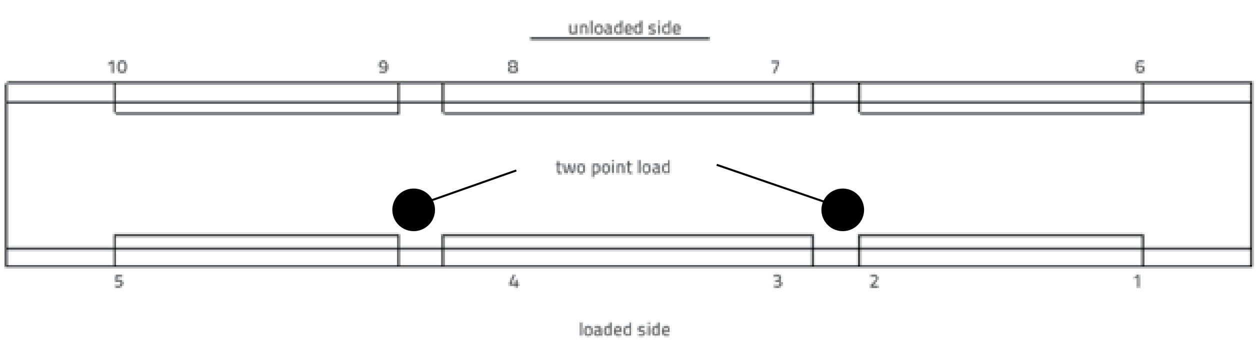

Figure 10. Positions of load versus stress change in tendons of PB2 beam

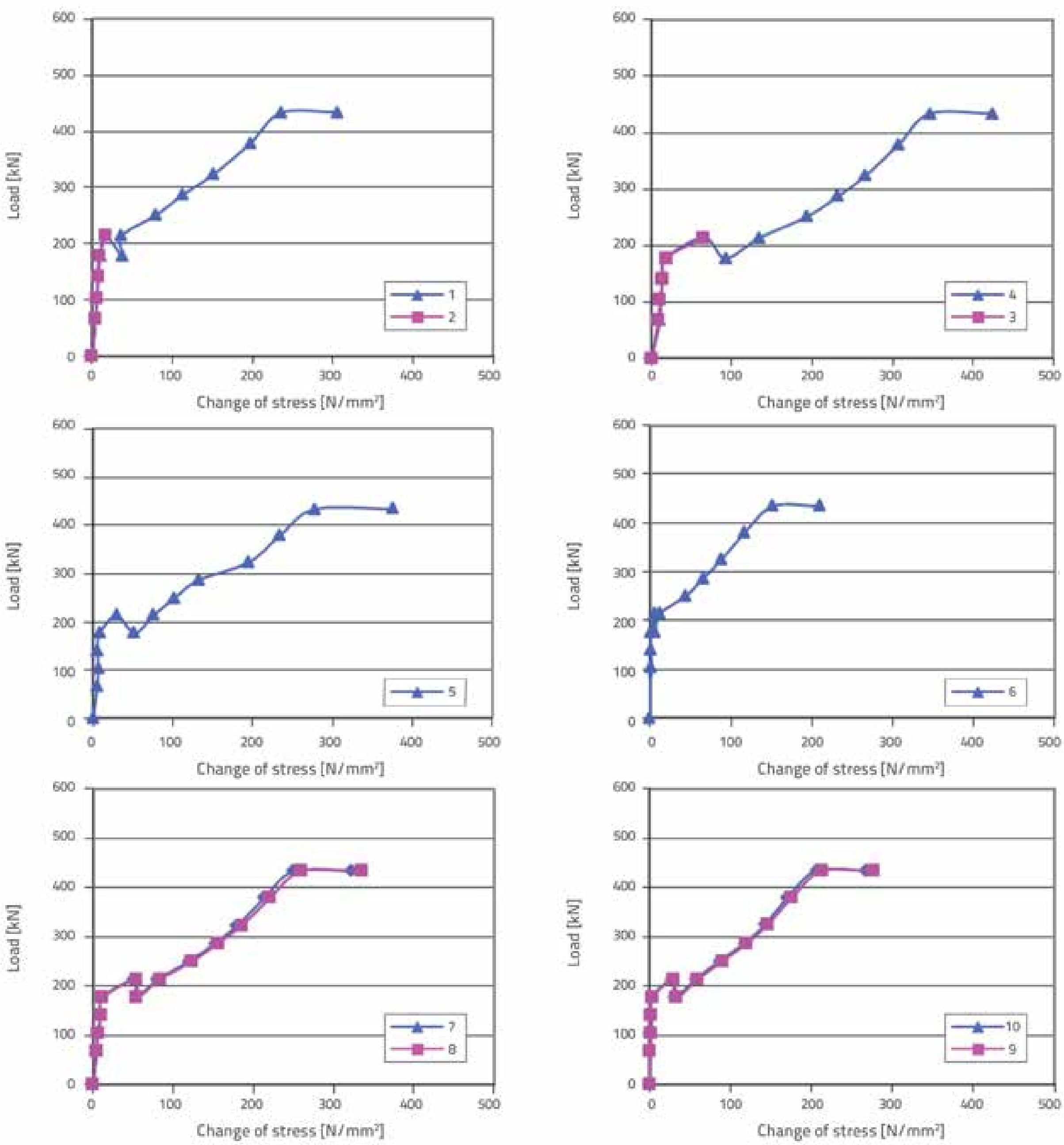

Figure 11. Curves of load versus change of stress for PB2 at different positions

Finally, the change in stress of PB3 beam strengthened with CRFP was studied at positions similar to those of PB2 described in Figure 10 above. It was noted that the position 8 exhibited the highest change in stress of tendons amounting to 570 N/mm^2. The effect of snapping and torsional loading at the loaded side induced higher stress in the prestressed wires, especially at positions 5 to 10. An average change of stress in PB3 beam was 456.14 N/mm^2, which is lower by 19% compared to PB1 and higher by 45% compared to PB2. That observation may significantly highlight the great role of CFRP in the box beam. Moreover, an average change in tendon stress at the loaded side was 275.14 N/mm^2, which is by 40 % lower than at the unloaded side where this change amounted to 462.33 N/mm^2, as shown in Figure 12. Local relaxation of some curves can be noted. This is due to the vibration of strain gauge signal during the test. In addition, some reading could not be completely captured in other positions owing to the damage of these gauges.

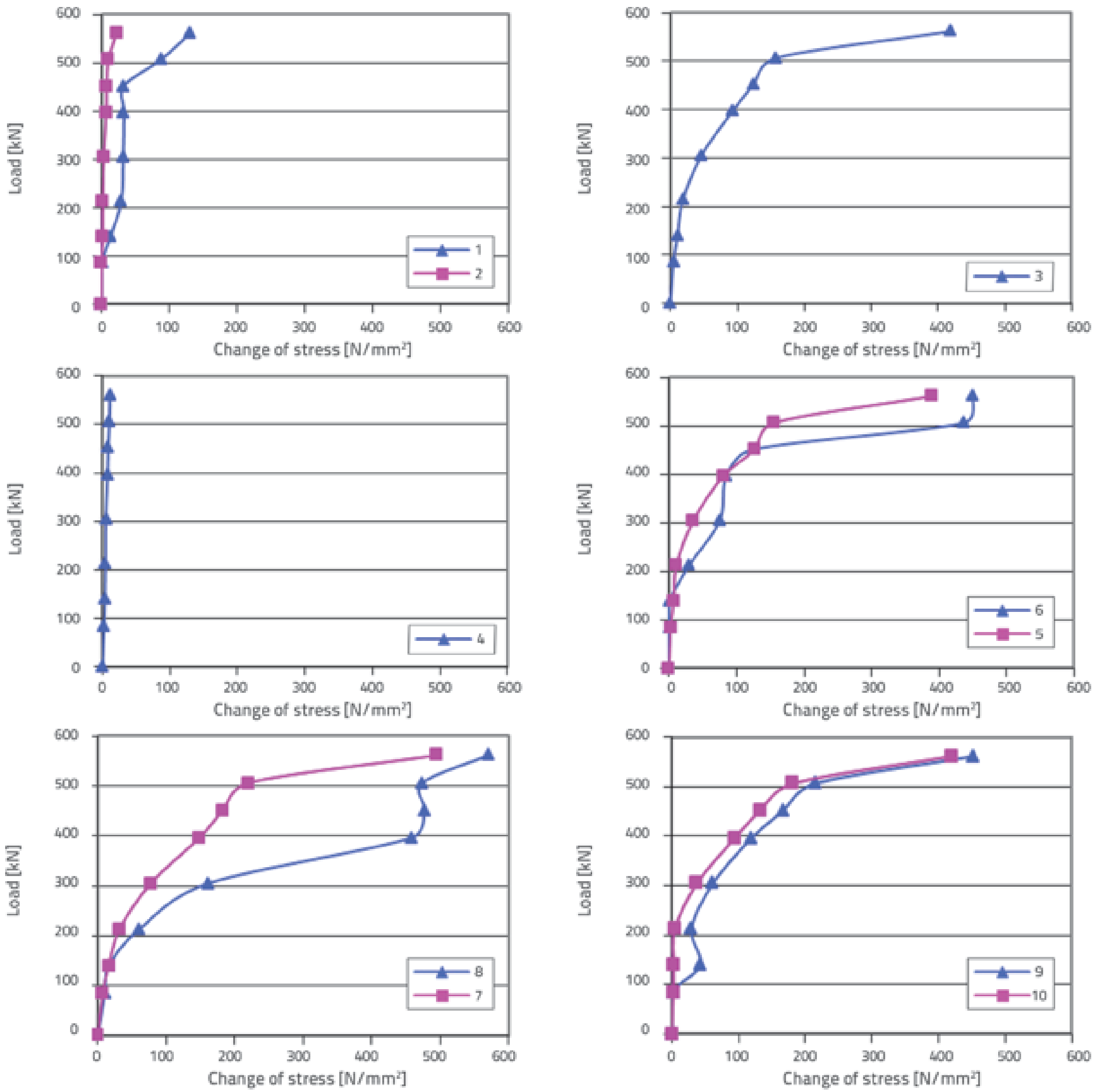

Figure 12. Curves of load versus change of stress for PB3 at different positions

3.3. Cracking loads and failure modes

The initial cracking load test results for PB1, PB2 and PB3 beams were compared with the predicted results based on the empirical formula according to Eurocode 2, as shown in Table 4 and Figure 13 [8]. The results proved a good consistency with the empirical results of Eurocode 2. Moreover, the results proved that CFRP highly enhanced the strength of box beam under snapping effects, whereas the cracking load of PB3 was higher than that of PB1 and PB2 beams by 50.03 % and 66.67 %, respectively. For PB1 beam, the first cracking at 183.3 kN occurred in the form of hairline cracks on both sides, especially in the middle third of the region. The width of cracks started to show and an obvious increase was noted as from 476.7 kN where the cracks ranged from 1 to 2 mm, with a pattern of steady increase. At failure load of 550 kN the cracking width ranged from 5 to 10 mm, the latter being the maximum width observed. The widest crack of 10 mm occurred at the unloaded side (far away from the loading point) and it propagated right up to the top and through the flange. The widest cracks were usually observed in the middle third of the beam, while the cracking width of 0.5 to 1.5 mm was maintained on the other side. Moreover, several shear and torsional diagonal cracks were noted, particularly at the loaded side (close to the loading point). The majority of shear cracks at the unloaded side exhibited an almost straight upward pattern.

Table 4. Comparison of cracking load results between experiment and Eurocode 2

Beam designation	Eurocode 2 [kN]	Experimental load [kN]	Differences [%]
PB1	153.59	183.30	19
PB2	146.02	165.00	13
PB3	179.70	275.00	53

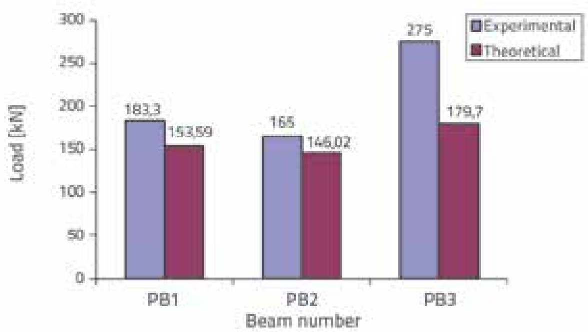

Figure 13. Experimental and theoretical cracking load results

For PB2 beam, the first cracks, less than 1 mm in width, occurred at 165 kN. Several cracks appeared at 201.7 kN in the middle third of the beam and close to the deviators. The cracks on the unloaded side of the beam - both in the middle and after the deviator - were mainly flexural cracks that started from the bottom and gradually propagated upwards and branched out towards the flange. At the loaded side, the cracks in the middle third were a combination of shear and torsional cracks that occurred at an inclination. Several small cracks, mainly flexural in nature, and some inclined cracks at the flange, appeared immediately after snapping at 201.7 kN. After the snapping of wires, the already existing cracks increased in length and propagated upwards. The majority of the cracks appeared on the loaded side. Several cracks appeared in the flange at a load of 434 kN. All these cracks are about 1 mm in width, unlike the control beam without snapping.

Finally, first cracks appeared at 275 kN on the loaded side of the PB3 beam after application of the CFRP indicating the effect of torsion. More cracks appeared after the deviator with an increase of load to 366.7 kN. This time the cracks appeared on both sides and were mainly flexural, both in the middle and after the deviator, while on the loaded side after the deviator the cracks were inclined and propagated toward the flange. The maximum crack width at mid-span of the loaded side was 1mm while it was 3 mm to 4 mm at the left and right spans of the loaded side. At the unloaded side, the maximum crack width was 10 mm to 13 mm at mid-span, and 4 mm at the right and left spans. At the bottom of the beam where the CFRP was placed, no cracks appeared within the distance of the CFRP. Instead, the cracks occurred at 421.7 kN outside of the CFRP length on both sides close to the supports. This points to the strength and effectiveness of the CFRP in improving flexural capacity even with snapping of prestressed wires. The crack width at the bottom of the CFRP outside the CFRP locations was 20 mm with the failure occurring due to the concrete breaking away together with CFRP. No failure or peeling off of the CFRP strip was observed.

3.4. Ultimate load

Furthermore, experimental ultimate loads were compared with the results of Eurocode 2 [8] as shown in Table 5 and Figure 14. A good consistency was observed between experimental and empirical results with maximum differences of 1.68 % in the case of PB1. It can clearly be observed that the box beam strengthened with CFRP was capable of restoring ultimate capacity of the defected beam. The ultimate capacity of PB3 was higher than that of PB1 and PB2 by 2.13 % and 29.42 %, respectively. Moreover, the results indicated that snapping could reduce the ultimate load capacity of PB2 by 21.1 % as compared to PB1.

Table 5. Comparison of ultimate load obtained experimentally and according to Eurocode 2

Beam designation	Eurocode 2 [kN]	Experimental load [kN]	Differences [%]
PB1	326.9	550.0	68
PB2	295.7	434.0	47
PB3	433.7	561.7	30

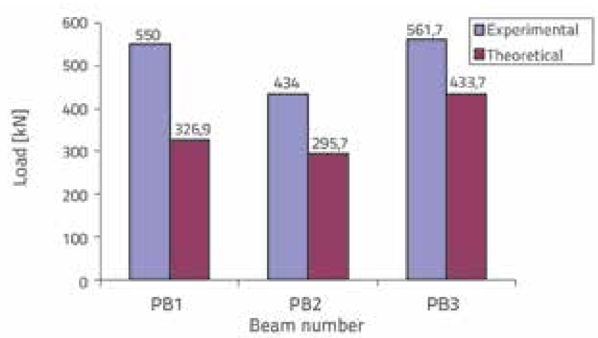

Figure 14. Experimental and theoretical ultimate load values

3.5. Torque versus twist relationship for box beams

Before the appearance of cracks in beams, the torque-twist curves were almost identical for all beams, as shown in Figure 15. After snapping, the torsional stiffness was significantly reduced in PB2. However, an increase in twist occurred for PB2 beam after snapping because of release of higher forces in prestressed wires with more brittle crushing as concrete strength increased. The torsion stiffness under the snapping effect could be improved by using CFRP as shown by the torque-twist relationship of the PB3 beam. The twist in PB3 was higher than that of PB1, which may be attributed to the snapping effect.

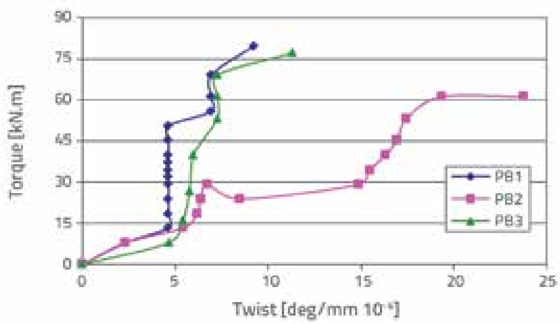

Figure 15. Torque-twist curve of box beams

4. Conclusion

The results can be summarized as follows:

- Load-deflection curves proved that the deflection of box beam under snapping effects was significantly reduced when CFRP was used.
- An average change in stress of the beam strengthened with CFRP was 456.14 N/mm^2, which is lower by 19 % compared to PB1 and higher by 45 % compared to PB2. In addition, an average change in tendon stress at the loaded side of PB3 was 275.14 N/mm^2, which is by 40 % lower compared to the unloaded side where the value of 462.33 N/mm^2 was registered.
- CFRP highly enhanced the strength of box beam under snapping effects, and so the cracking load of box beam under snapping effects was higher by 50.03 % and 66.67 % as compared to PB1 and PB2, respectively.
- At CFRP beam, the cracks first appeared at a load of 275 kN, while the cracks of PB1 and PB2 were initiated at 183.3 kN and 165 kN, respectively. The cracks were not registered at the bottom of the beam where the CFRP was placed. Instead, the cracks occurred outside the CFRP length on both sides close to the supports; thus, CFRP is a very effective method for improving flexural capacity even with the snapping of prestressed wires.
- The snapping can reduce the ultimate load capacity of the box beam; nevertheless, the beam strengthened with CFRP (PB3) exhibited a higher ultimate load by 2.13 % as compared to PB1.
- The use of CFRP could considerably enhance torsion stiffness of the box beam and minimize the twist under snapping effects.

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REFERENCES

[1] Espion, B., Kurrer, K.E., Lorenz, W., Wetzk, V.: Early Applications of Prestressing to Bridges and Footbridges in Brussels Area. 3rd International Conference on Construction History, Brussel, pp. 535-541, 2009.

[2] Yuan, A., Dai, H., Sun, D., Cai, J.: Behaviors of segmental concrete box beams with internal tendons and external tendons under bending. Engineering Structures, 48 (2013), pp. 623-634.

[3] Ghallab, A.: Calculating ultimate tendon stress in externally prestressed continuous concrete beams using simplified formulas. Engineering Structures, 46 (2013), pp. 417-430.

[4] AASHTO, Bridge design specifications. 1998, American Association of State Highway and Transportation Officials, Washington, DC.

[5] Naaman, A.E., Alkhairi, F.M.: Stress at ultimate in unbonded post-tensioning tendons: Part 2 - proposed methodology. Structural Journal, 88 (1992) 6, pp. 683-692.

[6] Aravinthan, T., Mutsuyoshi, H., Nitsu, T., Chen, A.: Flexural behavior of externally prestressed beams with large eccentricities. Transactions of the Japan Concrete Institute, 20 (1999), pp. 165-170.

[7] Building Code Requirements for Structural Concrete and Commentary (ACI 318M-05). American Concrete Institute, Farmington Hill, Michigan, 2005.

[8] CEN., Eurocode 2. Design of concrete structures – Part 1-1: General rules and rules for buildings. BS EN 1992-1-1, 2004.

[9] Karayannis, C.G., Chalioris, C.E.: Design of partially prestressed concrete beams based on the cracking control provisions. Engineering structures, 48 (2013), pp. 402-416.

[10] Saibabu, S., Srinivas, V., Sasmal, S., Lakshmanan, N., Iyer, N.R.: Performance evaluation of dry and epoxy jointed segmental prestressed box girders under monotonic and cyclic loading. Construction and Building Materials, 38 (2013), pp. 931-940.

[11] Wouters, J., Kesner, K., Poston, R.: Tendon corrosion in precast segmental bridges. Transportation Research Record, Journal of the Transportation Research Board, 1654 (1999), pp. 128-132.

[12] Bishnu, P., Takashi, M.: Failure mechanism of empty and concrete-filled CFRP box beams, Composite Structure, 89 (2009) 1.

[13] Grace, N.F., Enomoto, T., Sachidanandan, S., Puravankara, S.: Use of CFRP/CFCC reinforcement in prestressed concrete box-beam bridges, ACI structural journal, 103 (2006) 1, pp. 123-132.

[14] Xie, J., P. Huang, Guo, Y.: Fatigue behavior of reinforced concrete beams strengthened with prestressed fiber reinforced polymer, Construction and Building Materials, 27 (2012) 1, pp. 149-157.

[15] Dickson, T., Tabatabai, H., Whiting, D.: Corrosion assessment of a 34-year-old precast post-tensioned concrete girder, Precast/ Prestressed Concrete Institute Journal, 38 (1993) 6.