

Ability of Replacing the Steel Strands by CFRP in Pre-stressed Concrete Girders: Art Review

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إمكانية استبدال حبال الشد الفولاذية بألياف الكربون
البوليمرية في الجسور الخرسانية المسلحة: مراجعة أدبية

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Abstract

Fiber-reinforced polymer (FRP) composites are the modern, trustworthy alternatives to steel tendons and rebars. The bulk of current research is devoted to establishing the feasibility of incorporating carbon fibre-reinforced polymer (CFRP) into reinforced concrete girders by evaluating the strength and reliability of CFRP and concrete components. In order to improve fatigue and corrosion resistance, CFRPs are crucial. Comparing reinforced prestressed concrete bridge girders strengthened with conventional steel reinforcement to those reinforced with CFRPs offers the possibility of greatly increasing service life. Also widely researched and used in bridge engineering as a corrosion-resistant substitute for steel reinforcement is CFRP composite material. The use of carbon fiber-reinforced plastic (CFRP) reinforcement for prestressing concrete girders is a practical solution to the deterioration of concrete structures caused by corrosion of steel reinforcements. Serviceability demands may act as the determining design factor for members that have been pre-stressed or strengthened using fiber-reinforced polymers. Given that CFRP reinforcement has a low elastic modulus and a modest strain at failure compared to fully prestressed buildings, partial prestressing may be the optimum method for increasing deformability and reducing the cost of concrete structures.

Keywords: Fiber-reinforced polymer (FRP), Fatigue and corrosion resistance, Serviceability demands, prestressing concrete girders.



المستخلص

تعتبر مركبات البوليمر المقوى بالألياف (FRP) هي البدائل الحديثة والجديدة بالثقة للأوتار وقضبان الفولاذ. الجزء الأكبر من الأبحاث الحالية مخصص لإثبات جدوى دمج البوليمر المقوى بألياف الكربون (CFRP) في عوارض الخرسانة المسلحة من خلال تقييم قوة وموثوقية CFRP ومكونات الخرسانة. من أجل تحسين مقاومة التعب والتآكل، تعتبر ألياف الكربون المعززة بألياف الكربون أمرًا بالغ الأهمية. إن مقارنة عوارض الجسور الخرسانية المسلحة سابقة الإجهاد والمعززة بالتسليح الفولاذي التقليدي بتلك المعززة بألياف الكربون المقوى بألياف الكربون توفر إمكانية زيادة عمر الخدمة بشكل كبير. كما تم بحثها على نطاق واسع واستخدامها في هندسة الجسور كبديل مقاوم للتآكل لتسليح الفولاذ وهي مادة مركبة من ألياف الكربون. يعد استخدام تقوية CFRP للعوارض الخرسانية سابقة الإجهاد حلًا عمليًا لتدهور الهياكل الخرسانية الناتج عن تآكل حديد التسليح. قد تكون متطلبات قابلية الخدمة بمثابة عامل التصميم المحدد للأعضاء التي تم إجهادها مسبقًا أو تقويتها باستخدام البوليمرات المقواة بالألياف. بالنظر إلى أن تقوية CFRP لها معامل مرونة منخفض وإجهاد متواضع عند الفشل مقارنة بالمباني سابقة الإجهاد بالكامل، فقد يكون الإجهاد المسبق الجزئي هو الطريقة المثلى لزيادة قابلية التشوه وتقليل تكلفة الهياكل الخرسانية.

الكلمات المفتاحية: الألياف البوليمرية المسلحة (FRP)، مقاومة التآكل والتعب، المتطلبات الخدمية، الجسور الخرسانية المسلحة والمسبقة الإجهاد.

1 - INTRODUCTION

The possibility to substitute steel rebar or strands with Fiber Reinforced Polymer (FRP) reinforcement in concrete constructions is becoming more and more prevalent in studies and experiments. This is due to the substantial properties of FRP reinforcement as compared to those of steel reinforcement, including high tensile strength in addition to non-corrosive, non-magnetic, and lightweight materials. FRP reinforcements are an appropriate alternative to steel reinforcement in concrete structures for this reason. Despite various drawbacks associated with its behavior, such as low flexibility, low transverse strength, susceptibility to stress rupture, and a high price, the material is widely used.

From a structural engineering perspective, the two most essential properties to be considered are the low plastic behavior and the weak shear strength, which lead to early failure when dowel action is provided in concrete beams (PC) as depicted in Figure (1).

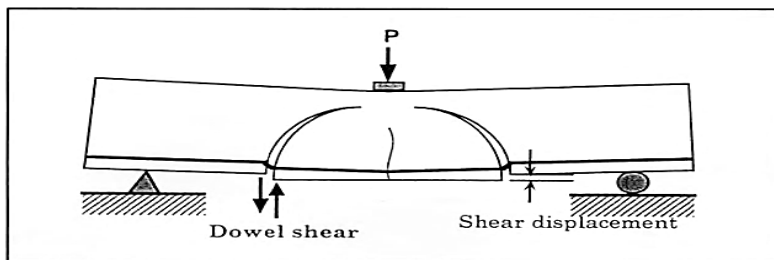


Fig. (1). Dowel action in concrete beam.

Consequently, this deformation weakens the residual tensile and shear strength in that tendon by decreasing the concrete shear resistance and ductility of PC components reinforced by FRP tendons, as seen in Figure (2).

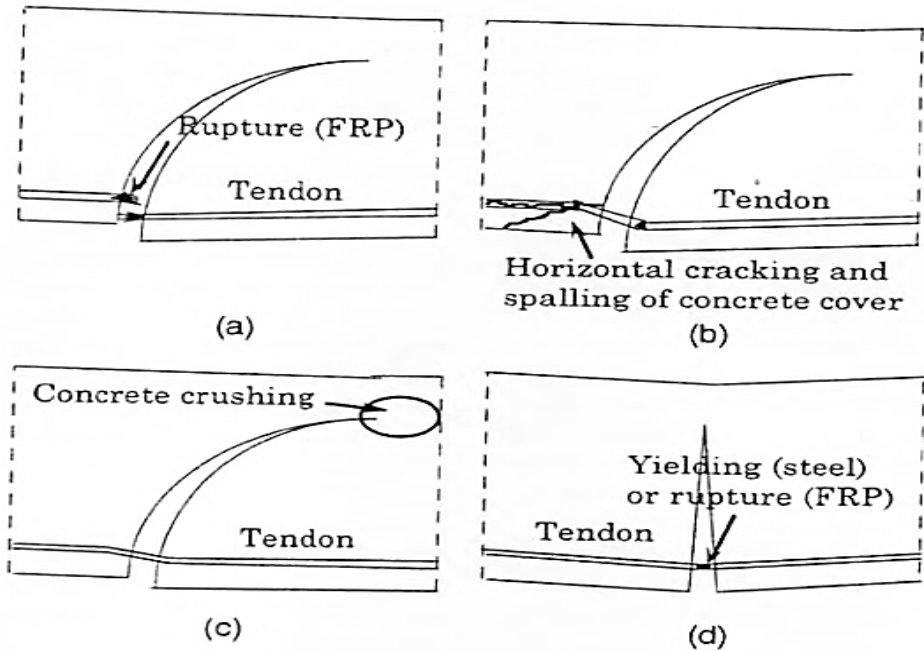


Fig. (2). Failure mechanisms detected in test beams were: (a) shear-tendon rupture; (b) shear tension; (c) shear compression; and (d) flexural tension.

As previously stated, carbon fiber reinforced polymer (CFRP), aramid fiber reinforced polymer (AFRP), or glass fiber reinforced polymer (GFRP) may be used to replace steel reinforcing rebar or strands in structural engineering applications. or by reinforcing the inadequate girders with standard FRP sheets or plates. CFRP tendons are better suited for prestressed concrete structures than other types of FRP tendons due to their higher tensile strength and stiffness. However, FRP bars typically exhibit a non-ductile material with a linear stress-strain relationship and a tendency for brittle failure under uniaxial tension Park *et al.*, (1999).



2 - EFFECT of BRITTLE BEHAVIOR of FRP

When FRP is used as a prestressed tendon in reinforced concrete beams, high durability is ensured. In reality, prestressed FRP beams are used rarely due to their reduced ductility and inadequate crack dispersion management. In addition, the key properties of FRP reinforcement are fatigue strength, a lower modulus of elasticity than steel reinforcement, and a linear connection between stress and strain. Currently, the most prevalent varieties are recommended for use in PC architectures. There are a number of alternative methods to improve the behavior of cracking and ductility by supplying non-prestressed, cured steel bars that are resistant to corrosion by using stainless, galvanized, or epoxy-coated steel. Field applications of prestressed concrete structures with FRP tendons have seen significant progress, and several studies have been conducted on PC beams with bonded FRP tendons and non-prestressed steel bars.

These studies were primarily concerned with the influence of FRP's brittleness and decreased elastic modulus on the flexural behaviour of beams. Either concrete crushing (compression failure) or FRP rupture led to the collapse of the beams in flexure (tension failure). Normal steel reinforcement with tensile yielding prior to failure: compared to other beams of tension failure, the bulk of these studies tend to believe that compression failure is the preferable failure mechanism because it is less calamitous and more progressive. (Grace *et al.*, 2013). Furthermore, in many design methods, deck slabs and conventional bridge girders suggest that stress failure is the optimal application approach (Gar, *et al.*, 2018; Forouzannia *et al.*, 2016).

By comparing the normal and balanced reinforcement ratios, one may theoretically determine the flexural failure mode. However, the actual mode



of flexural failure varies from the predicted mode, as shown by the results of these tests. This led to the establishment of a transition zone, which consisted of zones of broken concrete and shattered FRP. For structural concrete members with traditional steel reinforcements, it was suggested that the area where tension-controlled and compression-controlled sections meet should be flexible enough and give enough warning before breaking.

After understanding that FRP-reinforced concrete components show brittle failure in flexural behaviour, a novel definition of the transition zone is provided. Therefore, there is a high degree of congruence between ACI 440.4R-04 and ACI 318-14 for defining the transition area of FRP-PC sections, similar to those with an extreme tensile strain between 0.002 and 0.005 of the nominal strength in the longitudinal tensile reinforcement. For the flexural behaviour of typical grade 60 (240 mPa) steel rebar or grade 270 steel strands, these constraints were advised (1860 mPa). In the case of different stress-strain relationships, the strain limits of the transition area must be changed. As the flexural behaviour of FRP is linearly elastic to failure, the strain restrictions of steel strands in PC members may not be applicable to FRP-PC members. Numerous advantageous design techniques have recently been modified to provide a better strategy for the flexural strength of FRP-PC components.

Therefore, the state of the FRP-PC beam in the under-reinforced region is drastically different from that of the steel-PC beam. The collapse of an under reinforced section entails the yielding of the steel, followed by the crushing of the concrete region as a consequence of the yielding of the steel. Consequently, the flexural capacity may be calculated using Whitney's rectangular stress block. The failure of an under-reinforced FRP-PC beam is



the rupture of the FRP, and there is no stress distribution in that section, resulting in an abrupt and catastrophic collapse once the FRP ruptures. a technique that iteratively determines the distribution of stresses inside a compression concrete section. ACI 440.4R-04 and Grace *et al.* created an estimate for spreading concrete stresses using Whitney's rectangular stress block to avoid its complexity. This technique is seen as an overestimation of the compressive concrete stress and the lever arm, which may lead to an inadequately conservative design.

3 - LITERATURE REVIEW on DEVELOPMENT of USING CFRP PRESTRESSED CONCRETE BEAMS

McKay *et al.*, (1993) analyzed the viability of substituting noncorroding aramid fiber-reinforced polymer (AFRP) tendons for steel strand in pretensioned concrete applications. Experimentally, static and repetitive loads were applied to three 150 mm, 300 mm, and 2100 mm long AFRP-pretensioned beams. The results showed that repeated loading had no effect on the beams' load capacities. The capacity of the AFRP rods to create bigger tensile stresses than anticipated was largely responsible for the greater-than-anticipated ultimate capacities.

In 1995, The ductility index was created by **Naaman and colleagues**. The new concept is based on the relationship between ultimate energy and elastic energy when a prestressed girder fails. It applied to girders with both sensitive FRP and steel reinforcing. and it was reliable in providing a consistent comparison base. Using the proposed criteria, the ductility index of twenty-four prestressed concrete girders is experimentally evaluated by a team comprised of the authors and researchers. As shown by the data, beams



with FRP tendons have a lower ductility index than girders with steel strands. Nonetheless, more ductility may be achieved by using the appropriate design parameters or by employing a range of improvement methods, such as fiber reinforcing, confinement, or partial prestressing.

Amr et al.'s (1997) study of partial prestressing under low-jacking loads recommended designing the components of concrete under partial prestressing under CFRP reinforcement, resulting in a drop in cost and an increase in deformability. The concrete beams were thus submitted to flexural testing, and their partial prestressing by CFRP reinforcement was examined. The experimental program involves the evaluation of eight beams using CFRP bars plus two beams with ordinary steel strands. The most significant elements of this experimental program were the level of prestressing, the prestressing ratio, and the arrangement of CFRP bars inside the tension zone.

Two concrete beams with traditional steel reinforcement were tested beside eight concrete beams reinforced with carbon fiber-reinforced polymer (CFRP), commercially known as "led-line type" bars produced by Mitsubishi Kasei, Japan. The beams had a length of 6.2 meters and a width of 330 millimeters. The ratio of span to depth is the same for each of the bridge girders. The cross section of the investigated beams, as seen in Figure (3), was a T-section with two distinct flange widths of 200 and 600 mm. The two jacking stress levels used in the experiment were 50 and 70 percent of the FG of the CFRP's ultimate guaranteed strength, as provided by the manufacturer. To alter the segment's concrete stress distribution's prestressing level, two and four bars were used. Additionally, different CFRP bar distributions inside the tension zone were tested to determine their effects on the prestressed concrete beams' tracking behavior.

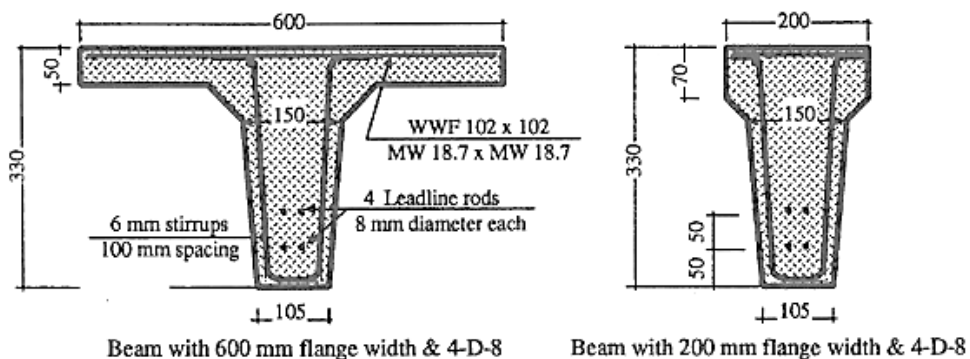


Fig. (3). The beam's cross section during testing.

After comparing prestressed reinforced concrete beams using CFRP bars to those of two equivalent beams prestressed with conventional steel strands, it was determined that the CFRP beams were better. The bottom line is:

At the zone of prestress transfer, the bonding qualities of CFRP bars are equal to those of steel strands. Given that the failure was induced by crushing the compression zone, the deflection of concrete RC beams prestressed with CFRP was about equivalent to that of beams prestressed with steel. As the failure of the concrete beam was determined by the rupture of the CFRP bars, the deflection is far smaller than that of equivalent reinforced concrete beams prestressed with conventional steel strands.

The frequency of fractures in beams partly prestressed with CFRP was lower than that of beams partially prestressed with steel strands because the flexural bond strength of the CFRP bars was inadequate. As a consequence, at a certain stress level, the crack width and spacing are often greater. The fracture width of a beam partly prestressed by CFRP or steel strands is the same for a certain reinforcing strain level.



Amir *et al.*, (1997) evaluated the behavior of five I-girders, each measuring 9.3 m, that were experimentally reinforced for shear and prestressing using CFRP. The other girder was experimentally prestressed using standard steel strands for flexure and steel stirrups for shear. The test beams use two different kinds of CFRP reinforcements for shearing and flexural prestressing and are 1:1.6 scale replicas of bridge girders that will be constructed in Manitoba, Canada. Seven days after the girder was prestressed, a slab was cast, and stirrups were projected from the girder into it. This composite procedure was repeated between the girder and bridge deck. For all types of CFRP reinforcement, different web reinforcement percentages were used. comparing the test findings to the improved compression field theory and the ACI building code.

1. The stress level generated by stirrups and the diagonal crack width have an indirect effect on the web reinforcement ratio proportional to the ratio.
2. The increased magnitude of CFRP's elastic modulus compared to other FRP reinforcements had no direct effect on the strain generated in the stirrups or the diagonal crack width.
3. Similar stiffness in the flexural behaviour of beams pre-stressed with CFRP tendons and beams prestressed with steel strands after flexural cracking and up to yielding of the steel.
4. Following cracking, the contribution of concrete V_{ci} was gradually reduced, and the theory of a modified compression field was used to properly predict their overall response.
5. The predominant practical approach of draping CFRP tendons has little impact on flexural capacity.



Therefore, there was no induced movement between the top slab and the girder.

6. This demonstrated that the strength of the concrete at the girder-to-slab contact was adequate to convey the horizontal shear forces.
7. The modification in the web reinforcement ratio has no effect on the flexural behaviour of the beams.

The behaviour of FRP-reinforced, simply supported beams was examined by Nabil *et al.*, (1998). Seven fundamental beams with rectangular sections and seven continuous beams with tee-shaped sections were tested experimentally by the researchers. The longitudinal and stirrup reinforcing bars were made of carbon, glass fiber-reinforced polymer, or steel (GFRP). The use of GFRP stirrups improved deflection and shear deformation, as shown by the test results. Depending on the kind of reinforcing bars and the inclusion of GFRP stirrups, the failure mode varied from flexural to shear or flexural-shear (FRP or steel). Moreover, the application of FRP reinforcement in continuous beams increased deformation. At the level of service load, the extra deformation remained negligible and manageable but grew much closer to failure. It was revealed that various configurations of FRP reinforcement in conventional beams had the same load capacity as steel reinforcements but displayed unique ductility and failure processes. The failure mechanism was determined by the kind of reinforcing bars and stirrups. As a measure of ductility, the energy ratio, defined as the ratio of absorbed energy at failure to total energy, was used. On the basis of this categorization, the degrees of ductility were recommended to be defined as ductile, semi-ductile, and



brittle behaviour. Experiments validated the theoretical findings reached using the suggested method. Continuous beams have higher "energy ratios" than simple beams.

According to a study by Saafi *et al.*, (1998), the use of FRP composites in construction and infrastructure applications has skyrocketed owing to their durability in harsh climatic conditions. There is the possibility of using rectangular aramid fiber composite tendons to reinforce concrete structures. However, the brittle failure mechanism is unfavorable for the maintenance and risk prediction of structures. Most research on prestressed concrete beams revealed brittle flexural failure caused by the FRPs' elastic rupture. Flexural tests were conducted on beams with bonded and/or unbonded rectangular bars or with additional non-tensioned (regularly reinforced) rebars to promote ductility. In comparison to the beam with bonded rectangular rebars, the beam with unbonded rectangular rebars deformed 250 percent more. The addition of rectangular reinforced rebars increased ultimate deformation by 450%. Combining bonded and unbonded rectangular reinforcing bars significantly enhanced ductility, according to the results.

Abdelrahman *et al.*, (1999) provided a more straightforward approach for calculating the deflection of prestressed CFRP-reinforced concrete beams subjected to static and repeated loads. In addition, the applicability of current deflection evaluation approaches was evaluated. Eight reinforced concrete beams of length 6.2 m and depth 330 mm with prestressed CFRP reinforcement, also known industrially as lead line bars, and two beams prestressed by traditional steel strands were tested; all tested beams had an I-cross portion with two different flange widths at the bottom and top (200



and 600 mm, respectively). The length of the beams with simple support was 5.8 meters, and the length expanded from each end was 200 millimeters. Based on an experimental observation, researchers at the University of Manitoba presented connection criteria to account for the tension stiffening of concrete-reinforced beams prestressed by CFRP. The results of several experimental programs are used to calibrate a proposed method for locating the centroid of cracked prestressed sections. To calculate the deflection of beams partially reinforced with CFRP, design guidelines are proposed. As indicated by ACI Committee 435, the deflection of beams prestressed using CFRP bars may be estimated using the effective moment of inertia as follows:

$$I_e = \Psi^2 I_g + (1 - \Psi^3) I_{cr} \leq I_g$$

Where:

I_g : gross moment of inertia.

I_{cr} : cracked moment of inertia.

$$\Psi = \frac{M_{cr} - M_{dc}}{M_s - M_{dc}}$$

Two full-scale, high-strength concrete bridge girders that were reinforced using FRP materials for prestressing and shear reinforcement were designed experimentally and tested until they failed, according to Stoll *et al.*, (2000). High-strength concrete was used to create two full-scale AASHTO Type 2 girders, and FRP methods for prestressing and reinforcing were developed. The beams were put to the test in a four-point bending test and failed. The properties of the materials, the design of the beam, and the test results are all fully documented. The following sentences list important observations and lessons that may be drawn from them.



1. ACI-440 effectively addressed significant issues relating to the use of FRPs as concrete reinforcement, including prestressing. A design code for FRP-prestressed concrete was being developed by American Concrete Institute Subcommittee 440-I on FRP Prestressing.
2. When determining the usual strength values for CFRP tendons, there were discrepancies across commercial manufacturers. Different permitted-to-ultimate stress ratios were adopted by two market leaders. The lead line CFRP tendons' predicted ultimate strength was roughly 3450 MPa, which is 33% more powerful than the manufacturer's declared value of 2600 MPa.
3. Connecting CFRP tendons to steel cables during pre-tensioning (done to finish the length of prestressing beds and/or to allow the use of conventional pre-tensioning equipment) might cause the steel cable to unravel during tensioning, which can cause considerable twisting distortion in the CFRP tendon. Where the CFRP tendon departs from the grip or anchor, this might result in stress concentrations that weaken the area of tissue.
4. It is important to look at the fabrication handling and safety precautions when CFRP prestressing tendons are employed. Carbon fiber-reinforced plastic (CFRP) tendon injury or breakage may be caused by simple methods used with steel cables.
5. In two final load tests of bending-strength-critical beam designs, the CFRP cable performed dependably. Prior to tendon failure, both 12.19-m beams showed substantial cracking and large deflections at load values that are in good agreement with



analytical predictions based on an effective tendon strength of 3450 MPa. The largest centre deflection measured in a single four-point bending test was nearly 230 mm (length/53).

6. Tension cracking of the beams occurred during loading at a lower load than expected. The following are some of the potential causes:
 - (i) The real pretension losses exceeded the projected levels.
 - (ii) The rupture modulus anticipated by AASHTO was higher than the value of high-strength concrete.
7. GFRP rebars for shear stirrups were employed in the test beams. Neither during stress testing nor when the beams failed, these stirrups failed. The design relationships used for steel shear reinforcement may need to be altered for GFRP products due to the lower axial modulus of GFRP rebar compared to steel (it is only one-fifth as stiff), lower shear strength, and lower stiffness.

Additionally, Rahman *et al.*, (2000) investigated a full-scale model of an isotropically reinforced bridge deck slab (FRP). The length of the slab was 6 metres, its thickness was 185 millimetres, and it featured two spans of 2 metres with cantilevers on both ends of 1 meter. In the beginning, the weighted slab repeatedly fractured the concrete. Then, it was cyclically loaded in three stages comprising 4 million cycles each with a frequency of 5 Hz, with the load varying from 0 to 100 kN in the initial two stages and 0 to 125 kN in the final stage. Ultimately, it was repeatedly loaded and failed. The suitability of FRP reinforcement for slab constructability was determined. The cyclic load test found minor strains and deflections, and a 25% increase in stresses and deflections following 4 million cycles of loading suggested little degradation. When the slab's ultimate load capacity crossed five times



the highest wheel load of 100 kN, punching shear was the failure mechanism. To establish the cost-effectiveness and durability of FRP reinforcement in bridge decks, the prolonged behaviour of FRP reinforcement there under the combined impacts of load exposure and a chemical environment must be examined.

Experimental research Zhang *et al.*, (2001) investigated the tensile properties of FRP ground anchors. On anchor models, several variables, such as anchor type, fixed length, and tendon composition, were evaluated. Different loads were applied to 16 monorod and 4 multirod grouted aramid fiber-reinforced polymer (FRP) (Arapree and Technora) and carbon fiber-reinforced polymer (FRP) (CFCC and Leadline) anchors. The test findings indicated that AFRP Arapree and Technora monorod anchors had more displacement and sliding than CFRP CFCC and Lead line anchors.

Technora anchors failed soon as a consequence of fibres detaching from the centre of the rod. AFRP anchors have a lower tensile strength and creep displacement than CFRP anchors. All of the tested CFRP monorod and FRP multirod anchors with a fixed length of 1,000 millimetres demonstrated satisfactory tensile performance in compliance with current requirements. The tensile strength of prestressed FRP ground anchors seems to be determined by their long-term creep behaviour. It is recommended that working loads for prestressed FRP ground anchors be 0.40 fpu for AFRP rods and 0.50 fpu for CFRP rods, where fpu is the ultimate load or strength of anchor tendons. Examined FRP rods are shown in Figure (4).

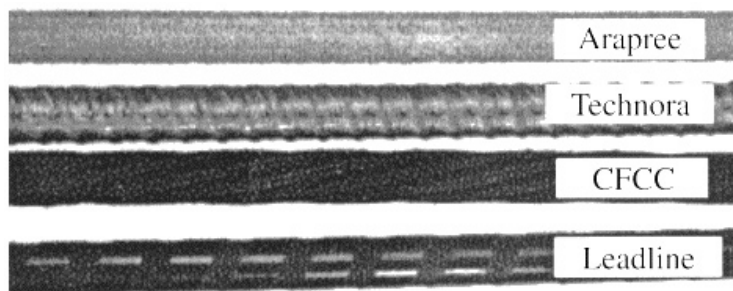
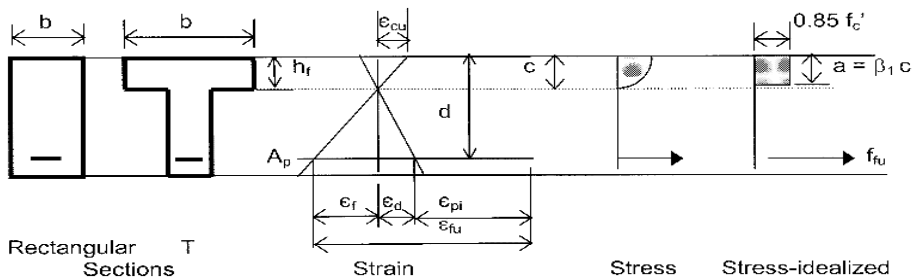


Fig. (4). FRP Rods Used in Study.

Dolan *et al.*, (2001) developed a standardized method for facilitating the flexural construction of reinforced concrete girders with FRP reinforcements. The focus of theoretical analyses of flexural capacity has been on FRP tendons that rupture linearly elastically. Current industry standard defines tendon performance by the tendon's design strength. The method of designing for strength depends on proportional balance. ρ is the ratio of reinforcement when the compression of concrete compression occurs simultaneous with tendon rupture. Any reinforcement ratio increase above this value causes primary compression failure in concrete, while any decrease below the balanced ratio causes tendon rupture failure. Figure (5) depicts a balanced relationship between tension and strain requirements.

Dolan *et al.*, (2001) established a similarity between the balanced ratios for FRP prestressed beams and reinforced concrete design in order to predict ductile behavior. Due to the difference in brittleness between steel and FRP tendons, the FRP-balanced ratio is presented as a failure scenario as opposed to a guarantee of ductility. Not included in the standard definition of ductility are FRP-reinforced or pre-stressed beams. In addition, the strain necessary for a steel tendon to fail and produce a yield state is less than that of a FRP tendon.

Dolan *et al.*, (2001) analysed the reinforced concrete cross section by computing the balanced ratio, b , which depends on the compatibility of strain in the cross section and is derived based on four fundamental assumptions: The failure of a tendon can be defined as the maximum tensile strain of that tendon, μ_u , and prestressing tendons are laid out in a single layer. Figure (5) illustrates or depicts these conditions.



Given is the expression for the proportional equilibrium:

$$\rho_b = 0.85\beta_1 \frac{f'_c}{f_{fu}} \frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_{fu} - \epsilon_{pi}}$$



where

β_1 = a factor derived as the ratio between the equivalent rectangular stress block depth and the distance between the extreme compression fiber and neutral axis depth.

f'_c = the required compressive strength of concrete.

f_{fu} = tendon's ultimate tensile stress.

ϵ_{cu} = ultimate strain of compression concrete, 0.003.

ϵ_{pi} = strain of initial prestressing.

ϵ_{fu} = tendon's ultimate strain capacity.

$$T_n = A_p f_{fu} = \rho b d f_{fu}$$

where

A_p = area of FRP tendon

f_{fu} = ultimate tensile strength of tendon

ρ = reinforcement ratio = A_p/bd

b = width of compression face

d = distance from centroid of outermost reinforcement to extreme compression fiber Summing the moment of the forces around the compression centroid gives the nominal moment capacity:

$$M_n = \rho b d^2 f_{fu} (1 - k/3)$$

where:

$$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n$$

and n is the modular ratio.

in the situation of under reinforced beams with reinforcement ratios ranging from 0.5 to b . Significant nonlinear behavior was observed in beams prior to tendon collapse, particularly in the concrete. Using a rectangular



stress block, the tensile capacity of the tendon and the summation of moments around the centroid of the rectangular equivalent compression block led to the following results:

$$M_n = \rho b d f_{fu} \left(d - \frac{a}{2} \right)$$

M_n corresponds to the notional moment capacity of the segment.

a = The following equation shows that the depth of an equivalent rectangular compression block is determined by the balance of forces on the block's cross section.

$$a = \frac{\rho b d f_{fu}}{0.85 f'_c}$$

When many reinforcing layers are used, this equation must be changed. When subjected to tremendous stress, the outermost layer of a tendon will fail first. By equilibrium and compatibility, the moment capacity of over-reinforced beams is equivalent to that of under-reinforced beams. In this case, the tendon strain is unknown, and a rectangular stress block represents the concrete's stress field. To locate the neutral axis, a series of strain compatibility equations must be solved. The following is the definition of the distance to the neutral axis:

$$c = k u d$$

where c is the distance between the fiber under high compression and the neutral axis. The following equation defines the coefficient k_u :

$$k_u = \sqrt{\rho \lambda + \left[\frac{\rho \lambda}{2} \left(1 - \frac{\varepsilon_{pi}}{\varepsilon_{cu}} \right) \right]^2} - \frac{\rho \lambda}{2} \left(1 - \frac{\varepsilon_{pi}}{\varepsilon_{fu}} \right)$$

where λ is a material constant and using the tendon elastic modulus, E_f defined in equation below:



$$\lambda = \frac{E_f \varepsilon_{cu}}{0.85 f'_c \beta_1}$$

$a = \beta_1 c = \beta_1 k_u d$. The equation below for the nominal moment capacity of an over-reinforced beam is derived by adding the moment forces around the tendon:

$$M_n = 0.85 f'_c b \beta_1 k_u d^2 \left(1 - \frac{\beta_1 k_u}{2} \right)$$

Twelve double-T (DT) beams were instrumented, produced, and subjected to flexural testing as full-scale prestressed beams employing unbonded post-tensioning carbon fibre composite cable (CFCC) strands and bonded a post-tension CFRP Lead Line TM tendon, according to a research by Nabil *et al.*, (2003) The three-span Bridge Street Bridge is the first vehicle concrete bridge in the USA to employ CFRP material as the primary structural reinforcement. The goal of the beam design was to analyze the reliability of the DT beams used in its construction. The testing settings were designed to measure the strain patterns over the beam's whole depth and length as well as its deflection and camber., pressures in post-tensioning strands, fracture load, ultimate load-carrying capacity, and failure causes. In addition, a theoretical approach for evaluating the tested beam's reaction was provided. Minor separation between the top and the beam flange, which resulted in the crushing of the concrete top and consequent fracture of the bottom tendons, is considered to have caused the most extensive damage to the beam. It was revealed that the tested beams had considerable reserve strength outside of the service load. Correlating theoretical values to real-world data, particularly in service demand situations.



Morais *et al.*, (2003) carried out experimental research using prestressed concrete beams of rectangular section as the basic support. Others were pretensioned using AFRP tendons. The elastic linear behaviour of FRP materials for prestressing beams is a recognized concern; this results in brittle structural failure. Utilize and reinforce the material's plasticity to improve the ductility of concrete structures. For the compression zone of the AFRP beams, three types of concrete were used: regular concrete, steel fiber-reinforced concrete (SFRC), and concrete with an AFRP spiral embedded in it. It is feasible to acquire the necessary ductile behaviour with FRP-prestressed concrete beams by determining the energy dissipation of beams by applying loads to loaded and unloaded beams at regular intervals, using limited concrete and over-reinforced beams.

Mertol *et al.*, (2006) compared the outcomes of carbon fiber-reinforced plastic (CFRP) bars-prestressed concrete beams to another set of steel wire-prestressed concrete beams. 15 beams have been produced and examined under various mechanical and climatic circumstances. The endured stress in the strands (55 to 70% of their tensile stress), the chemical influences (air and continuous exposure to 15% by weight saline water spray at 54 °C), the time under residual load (9 to 18 months), and the testing procedures (cyclical loading applied or not prior to static testing to failure) were all programmed parameters. The findings showed that prestressed beams made of steel wires could not endure an environmental exposure of one year, whereas those made of CFRP bars were able to withstand an environmental exposure of 18 months.

(Bischoff, *et al.*, 2007) examined the theoretical formulations of the effective moment of inertia provided by Branson in 1963 and implemented into the ACI Code; these expressions are reevaluated. It was decided

that Branson's statement was true for members with more than 1% of reinforcement steel.

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^3 I_g + \left(1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right) I_{cr} \leq I_g$$

The instant of effective inertia I_e As the ratio of service load moment M_a to cracking moment M_{cr} grows, the technique developed by Branson permits a progressive transition of the altered section (from uncracked to cracked). This transition is represented by the formula, and a plot of I_e / I_g versus for both techniques is shown in Figure (6). This formulation resulted in an overestimation of member stiffness at low reinforcement ratios and an underestimation of member deflection, as proven by a comparison to experimental data. It was shown that a modified equation is acceptable for all steel and FRP reinforcement ratios.

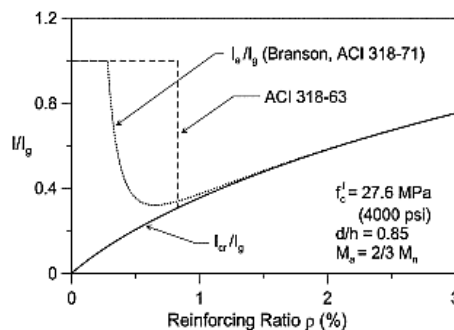


Fig. (6). Effective moment of inertia at service loads ($M_a = 2/3 M_n$).

In addition, several studies (ACI Committee 440, 2006) have shown that the original formula of Branson results in an excessively stiff reaction for beams reinforced using FRP bars, as well as a much lower modulus of elasticity than that for steel reinforcement. The modified Branson equation has been suggested for FRP reinforcing members (ACI Committee 440, 2006):

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} \leq I_g$$

where the adjustment factor $\beta_d = 0.2p/p_b \leq 1.0$ was experimentally determined by statistically fitting the available data. p_b denotes the ratio of balanced reinforcing bars.

Noel and Soudki (2011) compared the serviceability and ultimate limit states for FRP-RC slab bridges specified by AASHTO-LRFD by analysing the flexural behaviour of five slabs made from self-consolidating concrete (SCC) with two kinds of reinforcement: active and passive, and one reinforced concrete beam with steel reinforcement. Multiple relevant results may be gained from the experimental investigation.

For the given service load, slabs of reinforced concrete with CFRP tendons deformed considerably less than those without. Also, when compared to the steel-reinforced control concrete slab, the prestressed CFRP slabs had the same or better serviceability.

1. At the prescribed service load, the reinforced concrete slabs using non-stressed FRP reinforcement almost met the AASHTO-LRFD crack width limit of 0.5 mm. The application of post-tensioned reinforcement in the slabs reduced the crack widths of the members under service loads by more than 75%, easily meeting the AASHTO-LRFD criteria for crack width.
2. Lowering the depth of a prestressed cross-section slab had the same effect on load deflection as reducing the depth of a non-prestressed slab of the same cross-section; however, crack widths were reduced for all loads.



3. The approach of prestressing was effective in reducing the tensile stresses at service loads in the passive FRP reinforcement, hence satisfying the service stress criteria.
4. All of the FRP-reinforced slabs got curved enough to give users a visual warning before they broke, which is important when using linearly elastic materials like FRPs.
5. The posttensioned slabs performed appropriately in terms of deflection, crack size, limited service stress, as well as ultimate capacity for the prescribed service and ultimate loads. In the case of shear deficiency, shear reinforcement or an increase in pre-tensioning force may resolve the issue.

Behnam and Eamon (2013) evaluated possible strength-reduction factors; a reliable study was conducted on tension-controlled concrete flexural components reinforced with ductile hybrid FRP (DHFRP). Considered flexural members comprise a range of typical bridge decks and building beams intended to fulfil strength requirements and achieve target reliability levels determined by applicable engineering standards. The nominal moment capacity is derived from established analytical models and equated to the initial failure of the DHFRP material. In the reliability model, the statistical parameters for the random variables load and resistance are consistent with past code calibration attempts. The obtained resistance factors for tension-controlled sections varied from 0.61 to 0.64, indicating a possible increase in permissible strength compared to flexural members using nonductile bars.

Nabil *et al.*, (2013) reported the results of an experiment conducted on a 1/3-scale bridge model of the AASHTO-type, comprised of five AASHTO I-beams spaced 502 mm apart and connected by five uniformly spaced 64

mm CFCC strands. To fully study the flexural behavior of the bridge model, it was designed, built, instrumented, and tested. ultimate load failure, and strain response. a unique one-third scale I-beam of AASHTO type with a span of 12,141 mm and a CFCC-reinforced composite deck slab of 64 mm thickness. AASHTO I-beams with a 502 mm deep cross section, 203 mm wide top and bottom flanges, and a 95 mm thick web were produced and evaluated as control beams. Both the control beam model and the bridge model, in general, demonstrated compression-controlled failure, as expected. Prior to the collapse, substantial cracking and deflection were seen. As shown in Figures 7 and 8, the final strengths of the control beam and the bridge model nearly matched the values estimated using the unified design technique.

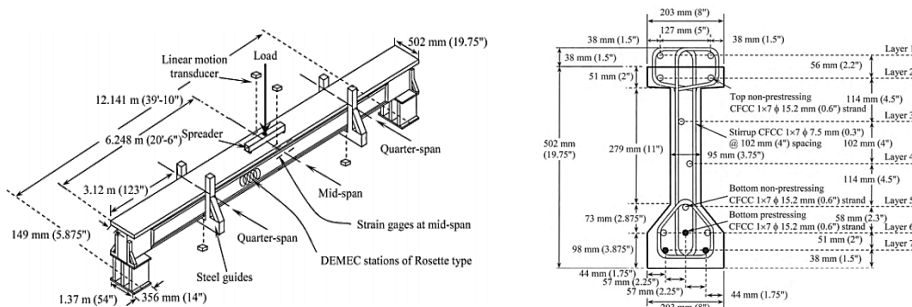


Fig. (7). Dimensions and Cross-sectional details of I-beam.

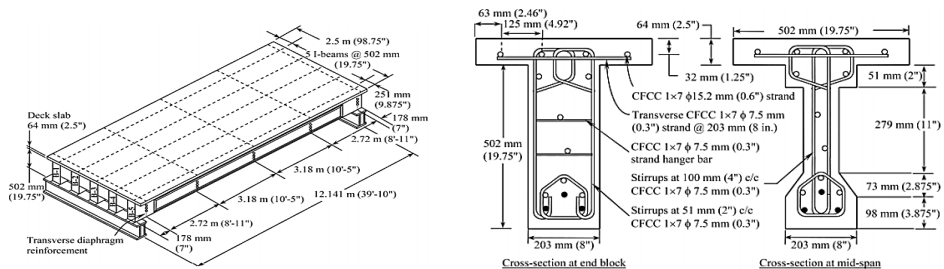


Fig. (8). Dimensions of bridge model.

Park *et al.*, (2016) investigated the influence of high-strength strands on the flexural behaviour of five major post-tensioned girders depending on the concrete's compressive strength and the strands' tensile strength. Regardless of the tensile strength of the strands, the test findings demonstrated that the actual flexural behaviours were in excellent accord with the predictions of the existing code. The specimens displayed ductile behaviour and all exhibited identical fracture patterns. Certain specimens subjected to service loads displayed fracture widths and stresses in the tensile reinforcements that exceeded the existing code limits by a small margin. With the right placement of deformed rebars, adequate crack control may be accomplished since the excess was not excessive.

A thorough analysis of the literature on reinforced concrete pre-stressed bridge beams utilizing CFRP strands was performed by Poudel *et al.*, (2018). Analysing existing design suggestions from various parts of the globe, the requirements from these recommendations are contrasted with testing information obtained from published literature. Future study requirements are compared to prior experimental studies to identify the gaps. Hugh Sand and Pultrall V-rod Brothers Aslan 200 CFRP bars are examples of commercially accessible CFRP bars, whilst carbon fibre composite cables (CFCCs) made by Tokyo Rope are instances of CFRP cables.

For prestressed CFRP, the manufacturer determines the anchoring method, unlike for steel. All manufacturers attach the CFRP cable or bar to the prestressing steel, which is then connected to the abutments, during prestressing using transfer boxes (also known as couplers). The coupler system for Tokyo Rope CFCC is shown in Figure 9.a and comprises of a steel wedge and sleeve that are attached to a steel coupler block that in turn is

coupled to a steel strand anchoring system. The pre-stressed CFRP is enclosed with a buffer material made of a stainless-steel mesh sheet and braid grip to avoid damage. The rectangular transfer box (9.b) in Figure 9a shows how the coupler system for the Pultrall V-rod also consists of a steel wedge and sleeve that is coupled to the anchored steel strand system. In this instance, copper is braided around the pre-stressed CFRP to guard against damage. An expanding material-filled threaded barrel serves as the anchoring mechanism for Aslan 200. Diagram (9.c). This threaded barrel is connected to the steel strand anchoring system by a rectangular transfer box. This arrangement works well with wedge grips that are directly installed.

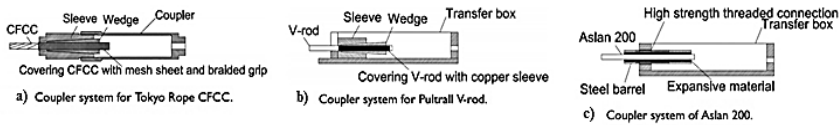


Fig. (9). a) Coupler system for the Tokyo Rope CFCC. b) System coupler for Pultrall V-rod and c) The Aslan 200 coupler system.

Using the moment-curvature behaviour of FRP/PSC sections, Pirayeh *et al.*, (2018) developed a mechanics-based formula for the effective moment of inertia (I_e). This allows for the development of a beam deflection formula for typical loading conditions. Various cracked-to-gross moment of inertia ratios (I_c/I_g) are determined experimentally to verify the proposed equation. The results of the comparison indicate that the ACI formula underestimates the deflection for small I_c/I_g ratios (<0.04), as is the case with weakly reinforced FRP/PSC beams and one-way slabs. At higher I_c/I_g ratios (>0.08), when the semi-empirical ACI formula was calibrated initially, however, accurate predictions are seen. The suggested equation, which gives an implied flexibility expression as opposed to the stiffness formulation utilized

by ACI, provides more accurate predictions of deflection, irrespective of the I_{cr}/I_g ratio. For each specimen, where b is the section width and d is the height of the effective section, measured from the center of reinforcement to the extreme fiber section, the location of the neutral axis of the cracked section was calculated as kd , where k is equal to:

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho \quad (19)$$

where $(n \frac{1}{4} E_p) = E$ is the modulus ratio, E and E_p being the elastic moduli of concrete and FRP, respectively and $p \frac{1}{4} A_p = bd$ is the reinforcement ratio, A_p being the FRP reinforcement area. The I_{cr}/I_g being each experimental specimen ratio may then be determined using the formulas shown below:

$$\frac{I_{cr}}{I_g} = 12 \left(\frac{d}{h} \right)^3 \left[\frac{k^3}{3} + n\rho(1-k)^2 \right] \quad (20)$$

The experimental behaviour of seven thin reinforced concrete beams with CFRP bars under progressive static loading is explored by Chris *et al.*, (2018). Capacity limits, deflections, pre-cracking and post-cracking stiffness, rapid local strength decreases, failure mechanisms, and cracking have been shown and discussed. The bonding circumstances of the anchoring lengths of the carbon-FRP tensile bars have been given special consideration. In some samples, local confinement characteristics along the anchoring distances of the CFRP bars seem to impact their cracking behaviour. However, further study is necessary in this area. In addition, experimental data for carbon-FRP beams and that for reinforced beams using GFRP bars collected from current literature are compared and discussed.

An easy-to-use method for determining the ductility of reinforced concrete tee-beams using prestressed and regular FRP was provided by Fei



Peng and Weichen Xue (2018). In terms of the ratio of provided-to-balanced reinforcement, a novel transition zone between tension- and compression-controlled sections was discovered using statistical evaluation of an experimental database of 83 beams ($e/b < e \leq 1.5 e/b$). Later, a computational approach for tension-controlled areas were developed employing a precise stress block to mimic the nonlinear compressive distribution of stresses in concrete. Multiple regression studies are performed to offer streamlined design estimates for tension-controlled flexural capacity in sections, which are based on a thorough parametric examination of more than 160,000 sections. Then, design formulae for the compression-controlled parts' flexural strength were provided.

Kim *et al.*, (2018) examined the flexural characteristics of girders in highway bridges using FRP tendon connections. Intriguing technical issues identified by ACI Subcommittee 440-I (FRP-Prestressed Concrete) include long-term multipliers, deformability, and minimum reinforcing. Reinforcing concrete girders using FRP composites of aramid and carbon (AFRP and CFRP, respectively) On the basis of analytical models, new design expressions for the previously described components are generated, followed by an assessment using laboratory test data and full-scale reference bridges. The long-term multipliers calibrated according to dependability theory differ considerably from the empirical coefficients used in ACI 440.4R-04. The prestressed AFRP/CFRP girders bend well in flexure, despite the reality that their moment-curvature responses are still not equivalent to those of steel-prestressed girders. The new deformability index describes the parameters for designing AFRP/CFRP-prestressed components with tension- or compression-controlled sections. Analyzing the importance of a potential

modulus change in FRP during the life span of RC prestressed girders for prestressed AFRP or CFRP girders, a computed ratio of ultimate-to-cracking moment $\phi M_n / M_{cr} = 1.2$ was suggested, which is compatible with the stipulations of existing design guidelines and regulations for prestressed concrete. Standard CR prestressed beams may flex sufficiently under flexural stresses if the following conditions are met:

$$\frac{c}{h} \leq 120\epsilon_{cu}$$

And concluded that prestressed members should demonstrate acceptable flexural deformation independent of prestressing material and formulation of the following equation:

$$DI \geq \frac{d_f / (120h) - \epsilon_{cu} + \epsilon_{fe}}{\epsilon_{fe}}$$

Karayannis *et al.*, (2018) studied the mainly elastic response, the lower bond capacity under cyclic load, and the poor ductility of RC beams with FRP bars, which have so far restricted their broad implementation in construction. Also appreciated were the advantages of this polymer reinforcement. Therefore, the experimental behavior of seven thin RC beams using CFRP bars was examined under increasing static loads. Capabilities, deflections, pre- and post-cracking stiffness, rapid local decreases in strength, failure mechanisms, and crack propagation have been examined. Special attention has been given to the bonding characteristics of the anchoring lengths of the carbon-FRP tensile bars. Local confinement properties for all the anchoring lengths of the CFRP bars in some specimens appear to influence their cracking behavior. Nonetheless, further research is required in this field. Comparisons of experimental data from the current literature for CFRP beams and RC



beams with glass-fiber reinforced plastic bars were also investigated and addressed.

Peng and Sue (2019) proposed reliability-based design criteria with a focus on strength reduction factors and the transition zone between tension and compression-controlled sections for the flexural strength of RC bridge girders prestressed by bonded FRP tendons. First, a collection of 48 bridge girders that represent various design options are subjected to stochastic simulation. In order to evaluate the statistical properties of resistance, Monte-Carlo simulation is performed. Then, in order to achieve the AASHTO LRFD's consistent goal dependability level of $T = 3.5$, the strength reduction factors were adjusted using the first-order and second-moment parameters technique. Last but not least, a probabilistic analysis of flexural collapse modes is conducted to identify a transition zone in comparison to the typical net tensile strain constraints in ACI 440.4R-04 in terms of the percentage of provided-to-balanced reinforcement ($\epsilon_b < \epsilon \leq 1.5 \epsilon_b$). According to this study, compression-controlled portions should have reduction factors of 0.85 and 0.80, respectively, with a linear fluctuation in the transition region.

In 2019, Yasir and Franz published the results of an experimental investigation on CFRP-reinforced concrete beams. Four rectangular-section prestressed RC beams were constructed and put to the test until they failed under cyclic and then monotonic loads. Additionally, each beam was pretensioned using a single 13 mm long, 0.5 inch diameter CFRP strand. Results showed that the primary cause of fragile and early failures was certainly a failed bond combining CFRP strands and bonding concrete. The ability to resist slippage was strengthened by adding additional steel stirrups, but it wasn't enough to prevent slippage at heavier weights. A



revolutionary approach was developed and put into use by fastening the FRP composite strand's endpoints with such a steel-tube anchorage system. The new technique reduced slippage and enhanced the flexural moment's capacity by 39%. In order to predict the load versus deflection reactions of the beams, analytical computational models were applied. Using a single computer programme to compare the performance of CFRP- and steel-stranded beams It was discovered that CFRP beams have stronger flexural strength but poorer ductility when the manufactured beam was intended to support identical service loads.

The flexural behaviour of damaged partially prestressed concrete beams reinforced with exterior post-tensioned tendons was the main focus of Weichen Xue, Yuan Tan, and Fei Peng's (2020) research. Six beams with reinforcements and two additional beams without reinforcements—considered control specimens—were subjected to failure testing. The test settings included two different types of prestressed tendons, including steel CFRP, and varying degrees of damage at the beginning. The test results indicated that external tendons might be used to reinforce the beams, increasing their functionality and ultimate flexural strength while maintaining their deformability. Additionally, the initial damage had a little impact on ductility and ultimate capacity, but it had an impact on the cracking and deflection behaviour. Comparable ductility measurements were made between RC girders with exposed CFRP tendons and steel strands. After comparing experimental results to theoretical hypotheses derived from various design approaches, it was found that while the flexural capacity of reinforced beams with initial damage can be studied similarly to those without initial damage, the effect of early damage must be taken into account



when calculating the deflection of beams with overloaded initial damage.

Through the design, construction, and testing of five 12.8-m-long girders with sections that were Type II AASHTO and prestressed with 15.2-mm-diameter duplex high-strength stainless steel (HSSS) strands in flexure, Anwer and Michelle (2021) clarified the flexural behaviour of stainless-steel strand prestressed concrete girders. The reinforcement ratio of prestressed girder was altered in the experimental technique. The length of the transfer and pretension losses of 15.2 mm HSSS strands were measured by the researchers. By calculating the load that produced cracking, the carrying capacity-determining ultimate load, the deflection response under that load, and the failure mechanism, the behaviour of flexural in the girders was evaluated. The rupture of the HSSS strands caused all of the girders to fail as planned. Experimental results show that even if the HSSS-strand has low ductility and may limit the capacity of the girders, there may be ample warning in HSSS-strand prestressed concrete girders (noticeable deformation and numerous fractures before collapse). The moment strengths predicted by the numerical analysis and observed values for the five girders were quite near to each other. Although the numerical method is simpler to employ for design reasons, the analytical model produced more precise predictions. Flexural design is recommended for HSSS strand RC prestressed I-girders where strand rupture is a permitted failure mechanism. The results of this testing programme will be used to create new design guidelines for RC girders with stainless steel strands.



CONCLUSION

Taking into account all previous studies on the flexural behavior of RC prestressed beams with CFRP instead of steel strands, there is evidence that since the ultimate strengths were generally greater than predicted, primarily due to the FRP rods' ability to develop greater tensile stresses than predicted, greater ductility can be achieved by employing adequate design requirements or by considering a number of improvement methods. Furthermore, recent research shows that certain assumptions used in the current bond properties of CFRP bars at the prestress transfer zone are similar to those of steel strands, and that the deflection of CFRP-prestressed beams is comparable to that of steel-prestressed beams as long as the failure is controlled by rupturing the concrete in the compression zone.

Due to the lower flexural bond strength of CFRP bars, there are fewer fractures in beams which are prestressed partially by CFRP than in beams which are partially prestressed by steel strands. As a result, for a given degree of stress, the spacing and width of the crack are often larger. Nevertheless, at a given strain level in the reinforcement, the fracture width of a beam partially prestressed by CFRP or steel strands is the same. Few studies had been done, according to all previously listed sources, to assess whether or not the suggested method's theoretical findings were validated by empirical evidence. To learn more about these issues, several studies were done. Continuous beams have larger "energy ratios" compared to simple beams. When using over-reinforced beams with limited concrete, FRP-prestressed concrete sections may display ductile behavior.



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