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FRP STRENGTHENING OF BEAM-COLUMNS JOINTS: EXPERIMENTAL VALIDATION AND ANALYTICAL MODELLING

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Abstract

Recent major earthquakes have made clear the role of seismic detailing is in the seismic performance of existing buildings in earthquake-prone areas. However, many older reinforced concrete (RC) structures —especially those built to outdated standards in the Mediterranean area— are highly vulnerable. One of the most critical parts are the beam-column joints (BC joints). Many of these joints, built according to older codes, lack sufficient transverse reinforcement or even have no stirrups, leading to premature brittle failures. Externally bonded FRPs have shown to be very effective in strengthening these joints by improving shear resistance, helping prevent joint failure, and reducing the overall risk to the building. A criticism with many strengthening layouts often used in the practice is that they have high disruption leading to the interruption of the building's usability. This study deals with the experimental validation of recently proposed FRP strengthening layouts that minimize that level of disruption to the occupants and can be applicable operation from the exterior of the building. The preliminary results of the experimental program are reported and discussed together with a close comparison with the predictions of strength capacity models available in literature.

Keywords: RC structures, BCJs, FRP, strengthening.

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1 INTRODUCTION

Recent catastrophic events have underlined the inadequacy of seismic detailing on existing reinforced concrete (RC) structures. Particularly beam-column joints, which play a critical role in the overall structural performance, are often under designed, leading to a brittle failure and triggering a partial or total collapse, especially in seismic prone areas as the Mediterranean. The vulnerability of these members lies in poor-quality materials and absence of joint stirrups, but also the use of plain bars as longitudinal reinforcement with low-bond capacity may significantly affect the joint response [1], [2]. The aforementioned inadequacies are more common in older buildings, where seismic provisions for transverse reinforcement were either nonexistent or insufficient at the time of construction. Numerous experimental tests have been conducted to assess the seismic performance of beam-column joints reinforced with FRP systems [3], [4], [5], [6]. These studies highlighted the effectiveness of FRP systems in enhancing the strength and deformation capacity of beam-column subassemblies. More recent experimental and analytical research on typical existing RC buildings has shown that using FRP materials as a local strengthening approach is a cost-effective strategy for improving overall seismic capacity. Numerous studies have investigated the seismic performance of beam-column joints strengthened with FRP (Fiber Reinforced Polymer) systems, demonstrating their effectiveness in enhancing strength and deformation capacity. Pohoryles et al. [7] conducted a comprehensive review of the capacity models outlined in existing guidelines and assessed their accuracy by comparing them with available data and findings in the literature.

However, the challenges associated with FRP debonding and the loss of concrete-fiber interaction have not received sufficient attention. Antonopoulos & Triantafillou [3] emphasized that debonding significantly influences the behavior of external reinforcement unless very low reinforcement ratios are used, or appropriate mechanical anchorages are provided. They demonstrated that wrapping longitudinal FRP sheets with transverse layers can serve as an effective anchorage method.

In typical corner joints, the presence of an orthogonal beam often prevents wrapping on both sides of the joint panel. In such scenarios, mechanical anchorage—through discrete restraints or by wrapping adjacent members—offers a viable solution to enhance the performance of the strengthening system. Ghobarah and Said [8] tested four beam-column joints strengthened with different rehabilitation schemes, some with mechanical anchorage and others without. Beams without anchorage failed due to JS rupture, while those with anchorage failed by flexural hinging. This highlighted the critical role of anchoring in FRP joint strengthening. Notably, spike anchors present a particularly effective approach when minimizing structural invasiveness is critical. Del Rey Castillo et al. [9] also examined the performance of spike anchors with externally bonded reinforcement (EBR) and analyzed the influence of different failure modes. They have also been used by [9] to strengthen deficient beam–column joints (BCJs), highlighting the advantages of improving subassemblies' seismic performance. Del Vecchio et al. [5] tested identical specimens with different strengthening and anchorage configurations, demonstrating that effective anchorage substantially improves the performance and capacity of the strengthening system. The findings highlighted that the number of anchors needed is directly related to the number of fiber layers and that a well-anchored system with fewer layers can achieve similar effectiveness to using more layers without anchorage—providing a key insight for achieving cost-effective design. It was also pointed out that limited knowledge and confidence in the design of mechanical anchors often lead to a non conservative approach, with only a minimal number of anchors (typically two per beam) being used.

Some models have been developed to approximate the capacity of anchored fabric as a strengthening configuration, though they are not intended for direct calculation. Antonopoulos and Triantafillou [11] developed a model which offer equations for stresses and strains at different stages of the response, up to the point of ultimate capacity, which is defined by concrete crushing or failure of the FRP due to fracture or debonding. They utilize Holzenkämpfer's (1994) [12] fracture mechanics approach with slight modifications. Bousselham [13] proposed a design model to predict the FRP's contribution to the shear strength of exterior joints without transverse reinforcement. Key aspect of the model was the determination of the effective strain of the FRP which was determined by calibrating test data from the literature on beam-column subassemblies that showed only joint-shear failure, with an upper limit of 0.004 to preserve the integrity of the concrete. This limit aligned with recommendations from [14] and guidelines ([15], [16]) but later research found this limit to be overly conservative.

Del Vecchio et al. [17] presented a simple and reliable analytical model for predicting the shear strength of corner beam-column joints strengthened with FRP under seismic loading. The model uses a principal tensile stress approach and introduces a new formulation for effective FRP strain, accounting for debonding phenomena and overcoming the conventional 0.004 strain limit in existing guidelines. Calibrated with experimental data, the model accurately predicts the performance of various FRP strengthening configurations, including different fiber types (CFRP, GFRP), layouts, fiber inclinations, mechanical anchorages, and applications to damaged and repaired joints. The model accounts for the presence of anchorage with a constant coefficient of 1.5 (+50% of the effective strain of the bonded FRP jacket) but does not allow to specifically design the number of anchors or anchors of different capacity. Thus, to date there are no models to reproduce the response of anchored BCJs. The research aims to bridge this gap by comparing experimental results with existing capacity models to help determine the geometric characteristics and quantity of anchors needed to prevent end debonding. This study discusses the novel capacity model proposed in [18] to account for the capacity of BC Joints and the experimental tests are used in its validation.

2 AVAILABLE CAPACITY MODELS FOR FRP STRENGTHENED BCJs

Numerous capacity models have been developed to predict the seismic behavior of both as-built and strengthened beam-column joints. When estimating the tensile capacity of FRP-strengthened joints, most models share a similar underlying approach. Antonopoulos & Triantafillou [11] developed a mechanical model to estimate the strength capacity of FRP-retrofitted beam-column joints, considering various failure modes such as joint core crushing, fiber rupture, debonding, and reinforcement yielding. However, the model involves a complex iterative process. Tsonos [19] proposed a simpler method by treating FRP fibers aligned with the beam axis as equivalent to steel hoops, allowing strength capacity calculation through a polynomial equation. Later, Akguzel and Pampanin [20] refined the original model with a non-iterative approach calibrated on experimental data, particularly effective for joints with plain round bars. Bousselham [21] introduced a simplified model using the principal stress approach from [1], offering an empirical formula to calculate effective FRP strain based on experimental results. To better align predictions with experimental results, the effective FRP strain ($\varepsilon_{f,e}$) was limited to 0.004, much lower than observed experimentally. This conservative limit, also adopted in [22], aims to preserve concrete integrity in confined RC members. However, it may not be suitable for shear-governed members, as evidenced by studies from [23] and [4]. Guidelines such as [24], [25], [22] and [26] used the aforementioned models and similar approaches to evaluate the capacity provided by FRP and account for FRP failure limitations. When assessing the shear capacity of FRP-strengthened beam-column joints, KANEPE considers the contributions of stirrups and FRP while neglecting the concrete's capacity. It addresses two potential failure modes: fabric rupture when the ultimate strain is reached and debonding, with the latter determined by a maximum stress that depends on the FRP's elastic modulus, material thickness, and the concrete's maximum tensile stress. EN1998 [24] adopts a similar approach considering the contribution of stirrups within the joint area. Additionally, when assessing the debonding limitation, it considers factors such as fiber inclination, anchorage length, and cross-section's characteristics. In [25] the contribution of the FRP system to a member's shear strength depends on the fiber orientation and an assumed crack pattern based on [27]. The shear strength provided by the FRP reinforcement is calculated by determining the force generated by the tensile stress in the FRP across the assumed crack. The bond reduction coefficient, which is influenced by concrete strength, wrapping scheme, and lamination stiffness, results in a maximum strain limit of 0.004. A similar approach is used in [22], where the shear capacity of a strengthened member is determined by considering the smaller value between the concrete contribution and the combined contributions of steel and FRP reinforcement. The conservative limit of 0.004 is adopted.

Del Vecchio et al. [17] introduced a simplified model to predict the total principal tensile stress ($p_{t,tot}$) by accounting for both concrete and FRP contributions. The concrete contribution is linked to the compressive strength of the concrete and a coefficient k that reflects the rebar type (plain or deformed). The FRP contribution is calculated based on the total fiber area and the effective tensile strain of the FRP fibers. The total FRP area depends on the fiber orientation, with fibers typically applied horizontally, vertically, or in multiple directions (e.g., 0°, 90°, ±45°) when multi-axial fabrics are used. To simplify the model further, specific equations for $A_{f,eq}$ were developed for common applications by assuming fixed inclinations of the FRP strengthening system. To determine the effective tensile strain of FRP a simplified, more refined expression was calibrated based on experimental data and is expressed as follows:

$$\varepsilon_{f,e} = 31.6 \cdot C_{I.D.} \cdot C_{M.A.} \cdot \left(\frac{f_c^{2/3}}{A_{f,eq} \cdot E_f} \right)^{0.6} \tag{1}$$

Incorporating two coefficients: $C_{I.D.}$ is 1 for undamaged joints and 0.8 for joints with initial damage, while $C_{M.A.}$ is 1 for non-anchored systems and 1.5 for mechanically anchored FRP strengthening, emphasizing the critical role of mechanical anchorages in enhancing FRP performance. Bulletin of *fib* [28] adopted the aforementioned approach in calculation of the capacity of a FRP strengthened BCJ, with only change in the calculation of design FRP strain, which is expressed as

$$\varepsilon_{fd} = 34\left(\frac{f_{cm}^{2/3}}{A_f \cdot E_f} \right)^{0.6} \tag{2}$$

multiplied with 0.8 in case the strengthenening will be applied on a repaired substrate.

CNR [18] adopts the same approach for calculating the FRP contribution in RC beam-column joints, assessing the FRP reinforcement area while considering various fiber orientations. An adjustment is made in evaluating the effective strain of the FRP. The effective strain of the FRP can be calculated as:

$$\varepsilon_{fd} = \min\left\{\eta_a \frac{\varepsilon_{fk}}{\gamma_{f1}}; 34\left(\frac{f_{cm}^{2/3}}{A_f E_f}\right)^{0.6}\right\} \tag{3}$$

It is the minimum of the effective strain as defined in the [28], and the design ultimate tensile strain of the FRP. The latter can be calculated as the characteristic strain of the fabric multiplied by the environmental conversion factor and then divided by the partial safety coefficient of the composite material.

These guidelines introduce an innovative approach to assessing FRP reinforcement capacity by offering formulas for directly calculating anchorage demand and determining the required number of anchors. Mechanical anchors, such as unraveled connectors, help minimize disruption to occupants, allowing reinforcement to be applied externally to perimeter joints. The guidelines also examine the different failure modes of FRP spike anchors. A brief overview of the proposed anchorage-related formulations is provided below. For shear-tension reinforcement of beam-column joints, the required number of anchors to ensure the design strain capacity can be determined by satisfying the following relationship:

$$F_{anc,d}(z) \geq A_f E_f \varepsilon_{fd} \tag{4}$$

where the area A_f is assumed to be equal to the are of the FRP strengthening on the joint panel minus the contribution of the fibers oriented along the axis of the column since they will not load the anchors located on the beam end.

The resistance of the anchored fabric can be calculated as the sum of the bond strength and the contribution of the anchors as follows:

$$F_{anc,d}(z) = \min\left(k_k \cdot \frac{f_{fdm} \cdot b_f \cdot t_f + \left[n_z \cdot \min\{N_{PO,m}, N_{S,m}, N_{DB,k}, N_{FR,k}\}\right]}{\gamma_{f2}}; \frac{\eta_a \cdot f_{fk}}{\gamma_{f1}} b_f \cdot t_f\right) \tag{5}$$

where:

k_k an experimentally calibrated coefficient that returns the characteristic value of the stress equal to 0.7;

f_{fdm} the mean stress at the end-debonding of the FRP fabric;

b_f and t_f the width and the thickness of the reinforcement, that in case of the FRP strengthened of joint panel should be subtituted with A_f;

n_z the number of spike anchors placed in distance z;

γ_{f2} the partial factor which is assumed to be 1.30 for the Ultimate Limit State in the case of end debonding.

$N_{PO,m}$, $N_{S,m}$, $N_{DB,m}$, $N_{FR,k}$ refers to the failure of anchors that can occur through different modes.These include:

a) Pull-out failure (PO), where the anchors detaches from the substrate with a concrete wedge, potentially along the entire anchoring depth or partially, sometimes combined with mixed failure involving partial sliding along the hole;

b) Sliding failure (S), caused by cohesive failure in the resin or concrete around the hole;

c) Debonding (DB), where the fan detaches at the interface with the FRP fabric;

d) Fiber Rupture (FR), resulting from tensile failure in the fibers of the anchors under shear/flexural loads. It accounts for the load direction and the bent anchors, thus reducing the strength by reducing the bend anchor from 180° (straigth anchor) to 90° (bent anchor).

Furthermore, if a fixed number of anchors is used, the resistance of the strengthened joint can be determined by equating capacity and demand, while specifying the design strain as follows:

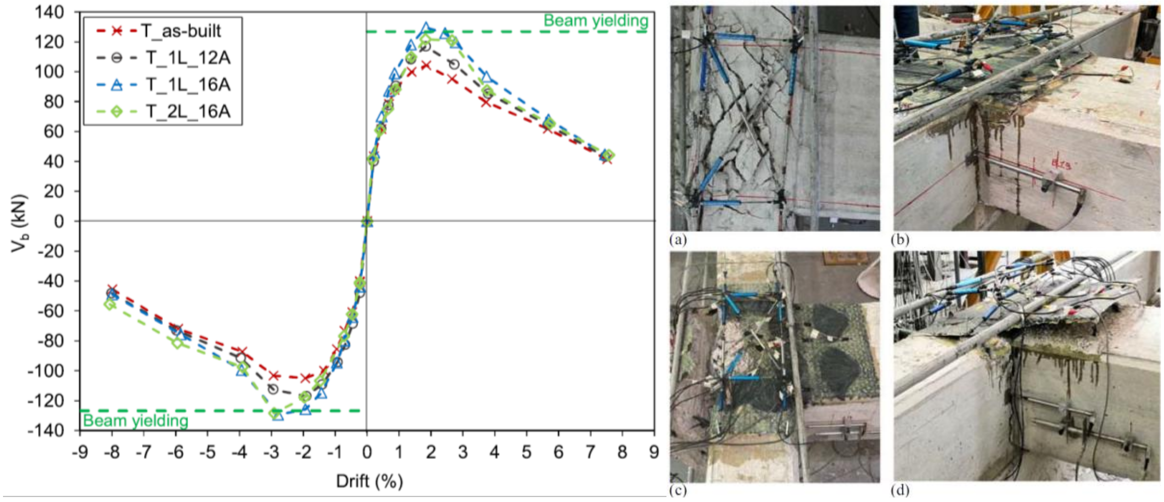
$$\varepsilon_{\mathrm{fdd,anc}} = \frac{F_{\mathrm{anc,d}}(z)}{A_{\mathrm{f}} E_{\mathrm{f}}} \tag{6}$$

By using this effective strain to calculate the FRP contribution to tensile capacity, the total tensile capacity of a strengthened BCJ with anchored fabric can be determined. Of course, the demand depends on the shear capacity of the joint up to the full hardening stress (or strain) of the tensile steel rebar of the beam.

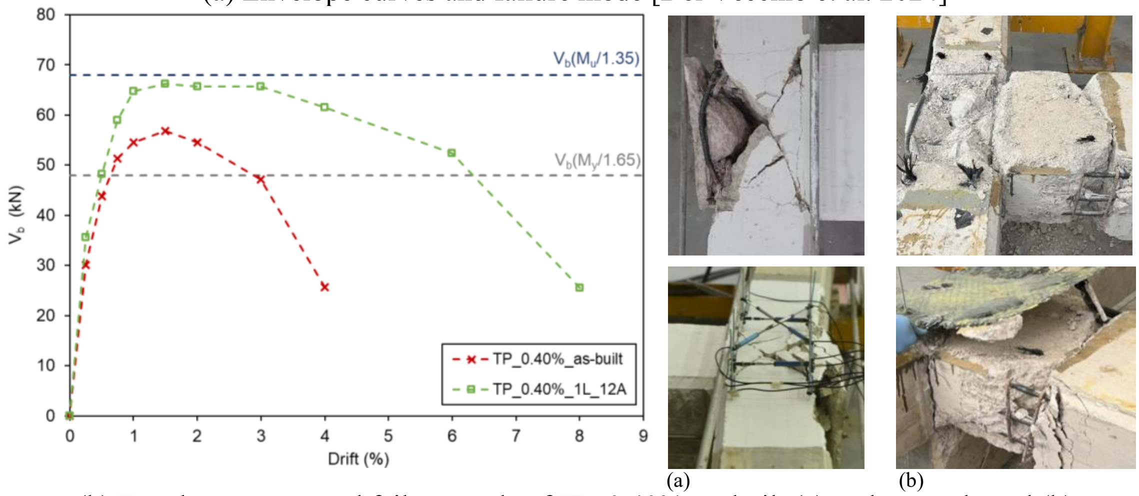
3 EXPERIMENTAL RESULTS AND DISCUSSION

The effectiveness of FRP systems as seismic strengthening of poorly detailed beam-column joints is extensively proven by experimental tests. Although the mechanical behavior of FRP strengthening in joint panels is similar to that of internal transverse reinforcements [4], debonding and the presence of transverse beams limits the confinement effect of the FRP, which distinguishes it from internal reinforcements [3]. Furthermore, debonding can significantly compromise the performance of FRP systems, especially when the concrete has poor mechanical properties, as highlighted by [29], [4] and [30]. To mitigate this, surface treatments and bonding devices can be used. A major challenge when strengthening beam-column joints with externally bonded FRP systems is the limited size of the joint panel, which leads to a short anchorage length for the fibers. To overcome this, various anchorage solutions, both with and without mechanical anchors, have been proposed. However, despite the examination of several anchoring configurations, the importance of proper anchorage and its precise contribution to the effectiveness of strengthening remains open to interpretation based on experimental results. An overview of experiments on strengthened beam-column subassemblies with anchored FRP fabric subjected to cyclic loads is reported herein, in order to point out the critical aspects in the definition of a simplified theoretical expression to predict and design anchored FRP retrofit systems. Ghobarah & Said [8] as well as [9] and [5] experimentally highlighted the crucial role of anchoring in FRP joint reinforcement, demonstrating that anchoring not only enhances strength but can also alter the failure mode of the specimens.

This study presents experimental results of strengthened beam-column subassemblies with anchored FRP fabric, alongside analytical modeling of predicted strengthening capacity, to assess and validate the accuracy of the models. With this objective, six BCJs are examined, including two as-built specimens. Specifically, four identical specimens are tested: one as-built and three strengthened with varying configurations of FRP layers and anchors. The remaining two specimens consist of another as-built specimen and its corresponding strengthened version. It is worth mentioning that although all six specimens have identical geometric characteristics, the four and the remaining two specimens differ in materials and longitudinal reinforcement. The backbone envelopes and the failure of the tested specimens are depicted in the figures below.



(a) Envelope curves and failure mode [Del Vecchio et al. 2024]



(b) Envelope curves and failure mode of TP_0.40%, as-built (a) and strengthened (b)

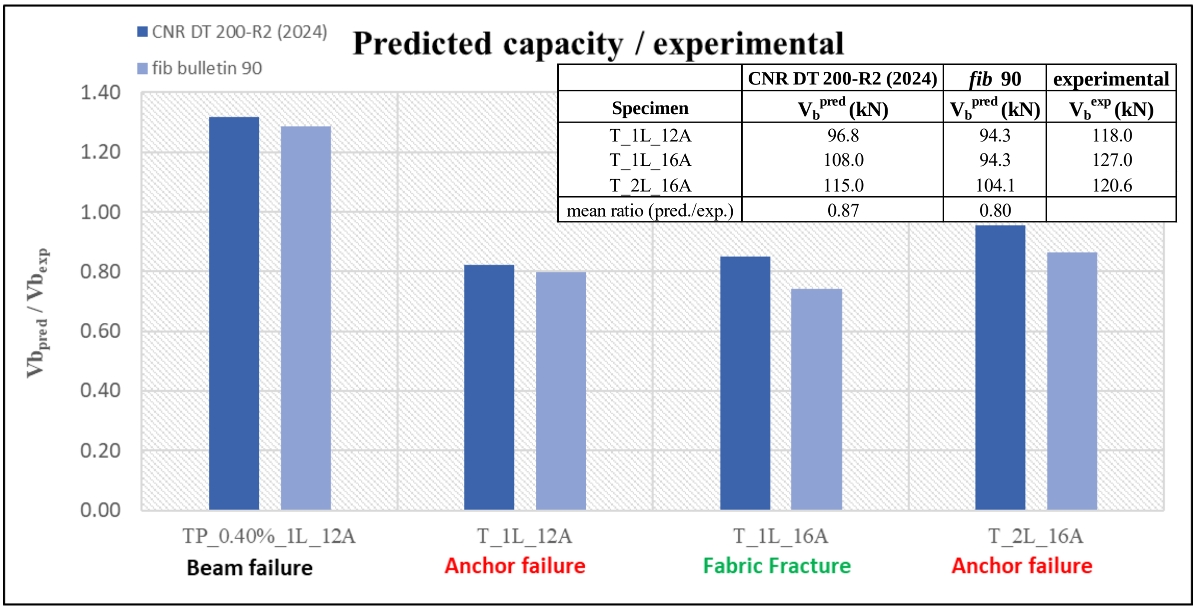
Figure 1. Envelope curves and failure modes

It is observed that Specimen T_1L_12A, which was strengthened with one layer of CFRP fabric and 12 anchors (2 on each face of the columns and beams), resulted in a 12% strength increase compared to the as-built specimen. However, premature debonding at the beam's end limited the full capacity of the CFRP fabric, leading to shear failure in the joint panel without yielding of the beam's longitudinal reinforcement. Specimen T_1L_16A, in which 16 anchors were used (4 at the end of each beam and 2 on each face of the columns), significantly improved bonding, leading to CFRP fiber fracture in the joint panel. This resulted in a 17% increase in energy dissipation and a 24% strength increase compared to the reference specimen. However, the same number of anchors could not support the demand of two CFRP layers, causing debonding at the beam's end in Specimen T_2L_16A. This occurred after the beam's longitudinal reinforcement yielded, preventing full utilization of its ductile capacity. A strength increase of 17%-25% and a 41% improvement in energy dissipation were observed. On the other hand, Specimen TP_0.40%_1L_12A was strengthened with a single layer of CFRP fabric and 12 spike anchors, placed at the ends of the fabric to prevent debonding. This configuration led to a 20.6% increase in strength, measured across both loading directions. Although minor FRP fiber debonding occurred at peak strength, the anchors helped to maintain the specimen's load capacity until the longitudinal plain reinforcement reached full hardening, forming a plastic hinge. Consequently, the failure mode shifted from a brittle shear failure in the joint panel to a more ductile mechanism, significantly enhancing energy dissipation by 176% with respect to as-built joint.

4 MODEL VALIDATION

4.1 Design procedure

To assess the accuracy of the proposed approach, the experimental results in terms of beam shear are compared with the predicted shear at the design capacity of the anchored fabric. The design procedure proposed by the CNR DT200/2024 is used to estimate the capacity in terms of beam shear following these steps: first, the effective strain in the FRP fabric at the maximum capacity of the anchored fabric is computed (eq. 14.52 of CNR). Next, the contribution of the FRP to the overall capacity is calculated and added to the concrete contribution, following Equations 14.53 and 14.54 of the CNR. The joint shear stress is then obtained by inverting formulation 8-55 from *fib* Bulletin 90. By multiplying this stress by the joint's area, the joint shear force is computed. Finally, the inverted formulation 8-50 from *fib* Bulletin 90 is used to determine the bending moment at the column's interface. The bending moment (M_b) is divided by the shear span to define the shear force (V_b), which is then compared with the test results. This systematic approach ensures a comprehensive assessment of the model's predictive capability.



	CNR DT 200-R2 (2024)	*fib* 90	experimental
Specimen	V_b^{pred} (kN)	V_b^{pred} (kN)	V_b^{exp} (kN)
T_1L_12A	96.8	94.3	118.0
T_1L_16A	108.0	94.3	127.0
T_2L_16A	115.0	104.1	120.6
mean ratio (pred./exp.)	0.87	0.80	

Figure 2. Comparison between predicted and experimental capacity values

The comparison between the predicted shear strength and the experimental results, as illustrated in Figure 2, demonstrates the accuracy of the analytical CNR DT200/2024 model in comparison with the model proposed by *fib* bulletin 90 using effective strain suggested by [17] for anchored fabric. Specifically, for the last three specimens, the average ratio of predicted over experimental values is approximately 0.87 for CNR model, while for the *fib* 90 model, it is 0.80, emphasizing the higher accuracy associated with CNR's model. Moreover, according to CNR DT200/2024 the predicted effective strain in Specimen T_1L_16A is calculated as

0.012, falling within a small margin of the ultimate strain value (ε_f^{ult}=0.015). This prediction is validated by the experimental observations, as fabric fracture was recorded during testing. For all specimens, the predicted strain deviates from the mean of maximum strains by a range of 0.4×10^{-3} to 3.4×10^{-3}, confirming the CNR model's accuracy in predicting strain behavior.

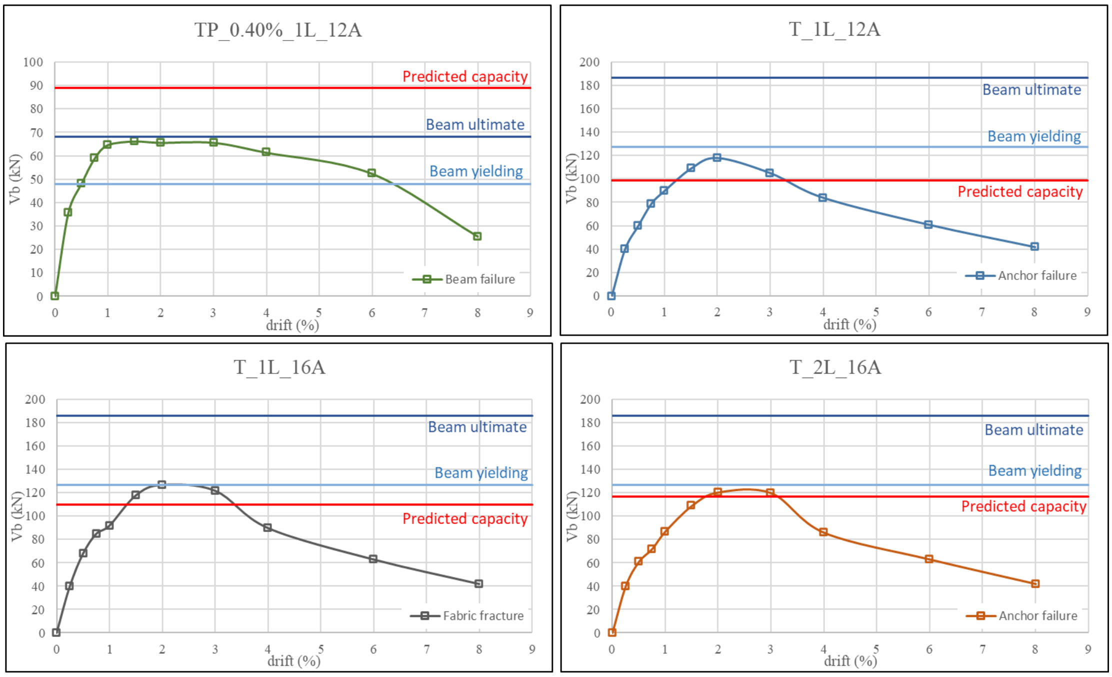


Figure 3. Backbone envelopes aligned with predicted values

This reinforces the reliability of the analytical approach in assessing FRP material performance under load. Most importantly, the analytical model allows designers to utilize the FRP strain at debonding well beyond 0.004, surpassing the conservative limits of existing recommendations.

Figure 3 demonstrates that for the last three specimens, the approach is both reasonably accurate and conservative, with predicted-to-experimental value ratios of 0.82, 0.85, and 0.95. For TP_0.40%_1L_12A, the beam's longitudinal reinforcement reached full hardening, allowing the cross-section to achieve its maximum capacity, resulting in the formation of a plastic hinge. Therefore, in that case, the predicted capacity exceeds by far the experimental values, which clearly indicates that the provided retrofit was heavier than needed.

5 CONCLUSIONS

This paper discusses the available analytical model for existing beam-column joints strengthened with FRP strengthening layouts designed to minimize disruption to building. The accuracy of existing models is evaluated against available experimental results. The main findings are outlined below:

- Most capacity models fail to include or account for the contribution of the fabric's anchorage in the FRP system.
- Until recently, the few approaches that considered the effect of anchored fabric primarily focused on the fabric's effective strain. The new CNR DT200/2024 introduces a method

for calculating the precise contribution of anchorage in FRP strengthening, along with predicting the failure mode of the mechanical anchor.

- The CNR DT200/2024 model predicts with high accuracy the capacity of the anchored FRP strengthening system, as well as the effective strain acting on the fabric which is far above 0.004 which is the conservative limit of most existing recommendations.

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