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Study on the Stress Threshold of Preventing Interfacial Fatigue Debonding in Concrete Beams Strengthened with Externally-Bonded FRP Laminates

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Abstract: Externally-bonded FRP laminate is widely used in structural strengthening due to the many advantages of FRP materials. Further enhancement of the strengthening effect can be achieved by inducing prestress into the FRP laminate. However, FRP debonding is still a main issue of this strengthening method, especially the Intermediate Crack-induced debonding (IC debonding). To better understand the impact of FRP debonding on the strengthening effect, a series of parameter analyses were conducted in this study based on the fatigue life prediction model proposed by the authors. The proposed model involves the fatigue damage accumulation of components of the beam, the mutual interaction between each component, and the impact of FRP fatigue debonding. As a result, a stress threshold for preventing FRP fatigue debonding in strengthening the concrete beam was proposed, which aimed to avoid safety hazards caused by IC debonding in practical engineering.

Keywords: fiber reinforced polymer; concrete beam; fatigue; debonding; prestress

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1. Introduction

For engineering structures, an increase in service life, environmental erosion, and increased load can all cause the degradation of their performance and cause the structures to fail to meet the requirements of use. Demolishing and rebuilding all structures that fail to meet the requirements can not only cause a lot of resource waste but also seriously delay normal social production. Therefore, the strengthening of existing structures has become an economical and efficient method of structural repair and enhancement, which can enable engineering structures to achieve higher resistance and better performance on the existing basis. Since the 1980s, Fiber Reinforced Polymer (FRP), especially Carbon Fiber Reinforced Polymer (CFRP), has become a research hotspot in the field due to its high strength, lightweightness, corrosion resistance, fatigue resistance, and other advantages. Externally-bonded FRP laminates have also become a new method of strengthening and renovation that has received much attention. Compared with conventional strengthening methods such as increasing cross-sections, bonding steel plates, and adding extra supports, the externally-bonded FRP strengthening method has advantages such as convenient construction, manpower saving, the convenient on-site cutting of FRP sheets, no need for large construction equipment during the construction process, and minimal changes in the appearance of the structure after strengthening. Therefore, it has received increasing attention from engineering and academic circles. Swiss scholar Meier first used externally-bonded FRP technology in 1982 to reinforce the Ibach bridge with CFRP sheets [1]. After the Hanshin earthquake in 1995, CFRP sheet strengthening was also widely used in the strengthening of concrete structures such as bridges and buildings in Japan. By 1997, the use of CFRP sheets alone for concrete structure strengthening in Japan had reached over 1 million square meters [2]. Subsequently, FRP sheet strengthening technology has received great attention in developed countries such as Europe, the United States, and Japan, and a large amount of related scientific research has been carried out. The United States (ACI) [3], Europe (fib) [4], the United Kingdom (Concrete Society) [5], Japan (JSCE) [6], and Canada (ISIS Canada) [7] have also successively developed relevant codes or regulations for the FRP strengthening of concrete structures.

The externally-bonded FRP strengthening technology has also been widely applied in the strengthening engineering of bending members such as bridge girders (as shown in Figures 1 and 2). The bridge girders need to withstand millions of repeated loads during their service life, and the fatigue problem caused by repeated loads has always been one of the problems that cannot be ignored in bridge structures. The phenomenon of structural failure after multiple actions with internal forces lower than the static load-bearing capacity is called structural fatigue. The fatigue failure of a structure generally starts from the initial defects inside its constituent materials, such as microcracks, pores, impurities, etc. These initial defects will cause a stress concentration inside the material. Under the repeated action of fatigue loads, cracks continue to expand, and internal damage gradually accumulates to form macroscopic cracks. Ultimately, sudden structural failure occurs due to the penetration of cracks or insufficient effective stress areas. At present, there are relatively more experimental and theoretical studies on FRP-strengthened concrete flexural members under static load, and a relatively complete system has been formed. However, to better promote the application of externally bonded FRP in bridge strengthening, further exploration is needed on the performance of externally bonded and FRP-strengthened concrete structures under fatigue loading.

Figure 1. Externally-bonded FRP strengthening of Bridges along the Shuohuang Heavy Load Railway.

Figure 2. CFRP strengthening of Changsha Tuanjie Overpass.

2. FRP Laminates in Structural Strengthening

FRP is a composite material composed of continuous fibers and resin. The basic composition and the microstructure of the FRP cross-section under electron microscopy are shown in Figure 3. According to the different types of fibers contained in FRP, FRP can usually be divided into carbon fiber reinforced polymer (CFRP), glass fiber reinforced polymer (GFRP), basalt fiber reinforced polymer (BFRP), and aramid fiber reinforced polymer (AFRP).

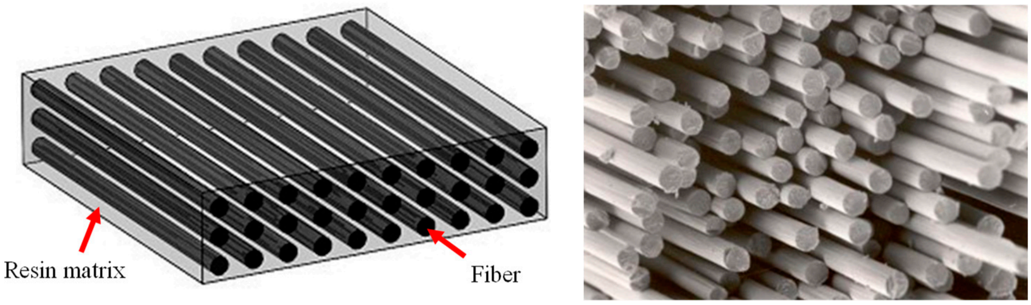

Figure 3. Composition and microstructure of FRP laminates.

In externally-bonded FRP strengthening, the applied FRP can be roughly divided into two categories based on the different forming processes: FRP sheet and FRP plate, which can also be collectively referred to as FRP laminates. FRP sheet, as shown in Figure 4a, is usually woven from bundles of fiber strands and requires resin infiltration and bonding at the construction site, known as the "wet bonding method" for construction molding. FRP plates, as shown in Figure 4b, are generally made in the factory through prefabricated molds or extrusion molding methods and are formed when leaving the factory. Both FRP laminates can be transported in rolls and are very lightweight.

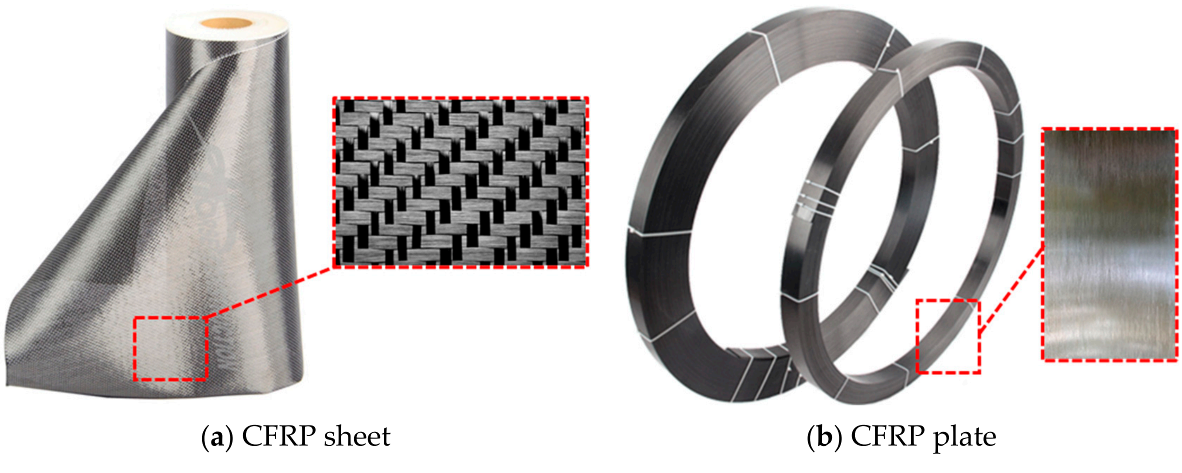

Figure 4. Two types of FRP laminates.

Compared to steel, FRP materials have some obvious advantages [8–12]: (1) Low density: the density of FRP materials is only about 1/8–1/4 of steel. As a result, FRP materials also significantly reduce the difficulty of construction and increase the length of continuously strengthened members; (2) High tensile strength: taking CFRP as an example, its tensile strength can reach 3–4 times that of steel, which further reflects the lightweight and high-strength characteristics of FRP materials; (3) Corrosion resistance: due to the fact that FRP materials are not corroded by acid, alkali, or chloride salts, they will not rust like steel. A large amount of FRP materials are used in the reinforcement engineering of coastal bridges; (4) Fatigue resistance: except for the slightly inferior fatigue performance of fiberglass materials, other common FRP materials exhibit good fatigue resistance, among which CFRP has the best fatigue resistance and is an ideal material for strengthening structures that need to withstand fatigue loads and is considered an ideal material to replace steel plates for externally-bonded strengthening.

Along with the many advantages mentioned above, FRP materials also have some undeniable drawbacks [13,14]: (1) Low material utilization: due to the premature debonding of externally-bonded FRP from the strengthened members [15], it loses its role in improving the load-bearing performance of the strengthened members; (2) High sensitivity to the performance of the bonded surface: due to the high strength of FRP material itself, the debonding of FRP normally initiates from the bonding surface. Especially when FRP material is bonded to concrete structures, debonding failure is an extremely common form of failure. Therefore, the strength of FRP materials depends heavily on the bonding quality or the stress performance of the substrate material; (3) Low elastic modulus: the elastic modulus of most FRP materials is lower than that of steel, so members strengthened with FRP materials often require greater deformation or deflection to enable the FRP material to exert its expected strength.

In structural strengthening, FRP laminates are generally bonded on the tensile side of the flexural member, and the tensile force is jointly borne by FRP and steel reinforcement to improve the flexural performance of the strengthened member. Furthermore, inducing prestress into FRP laminates can further improve the strengthening effect [16–21]. The prestress tensioning and anchoring system (PTA system) [22] development by the authors' research group is shown in Figure 5. The PTA system mainly consists of the following: (a) a CFRP plate tensioning end support, (b) a CFRP plate fixed end support, (c) a CFRP plate tensioning end anchorage, (d) a CFRP plate, (e) a CFRP plate fixed end anchorage, (f) a tensioning threaded rod, (g) a reaction steel plate, (h) a hydraulic jack, and (i) is composed of chemical anchor bolts and (j) anchor nuts. During tensioning, the hydraulic jack pulls the CFRP plate via the reaction steel plate and tensioning threaded rod. After reaching the controlling prestress, the fixed nuts on the CFRP plate tensioning end support are used to maintain the prestress. The schematic diagram of the tensioning process is shown in Figure 6.

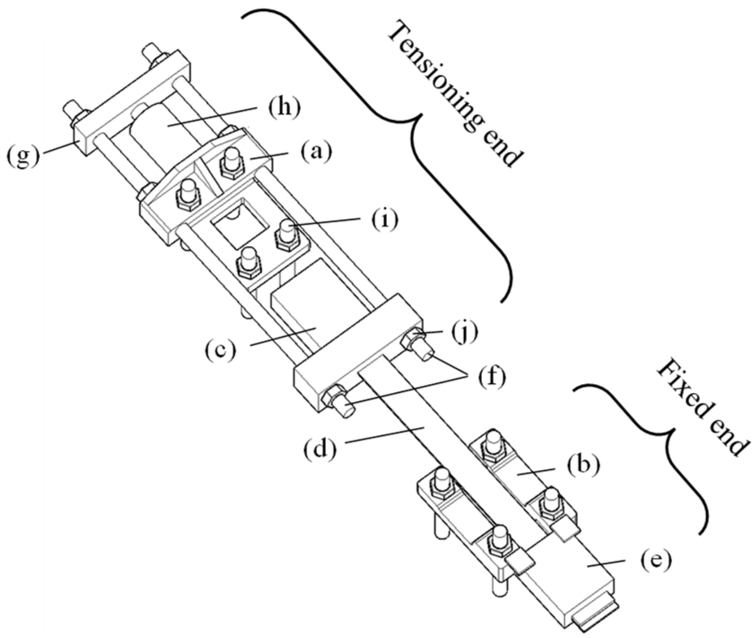

Figure 5. The Prestress Tensioning and Anchoring system (PTA system).

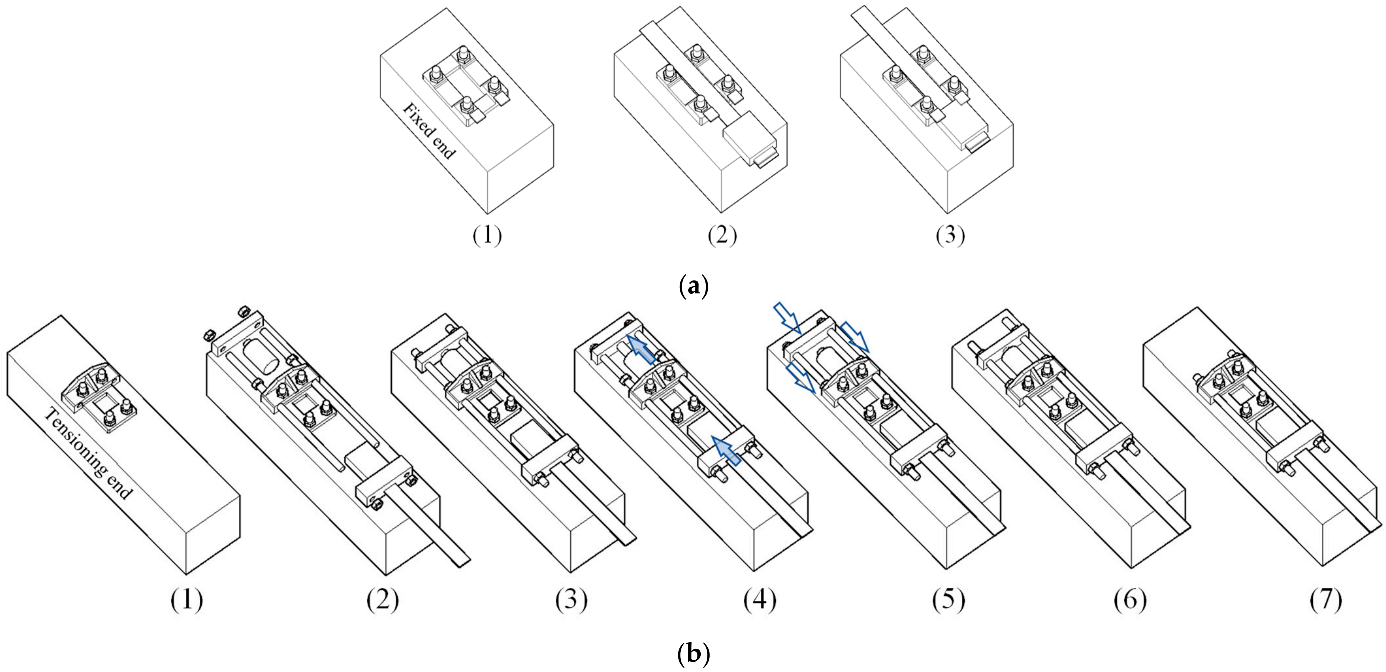

Figure 6. The process of applying prestress to the CFRP plate. (**a**) Anchoring process of the CFRP fixed end. ((1) Install the CFRP plate fixed-end support; (2) Install the CFRP plate fixed-end anchorage; (3) Fix the anchorage in position.); (**b**) Pre-tensioning and anchoring process of the CFRP tensioning end. ((1) Install the CFRP plate tensioning end support; (2)–(3) Install the accessories of the tensioning end; (4) Tension the CFRP plate; (5) Tighten the nuts and unload the jack for another tensioning process if needed; (6) CFRP plate reaches the controlling prestress; (7) remove the jack and the redundant length of tensioning threaded rods).

3. FRP Fatigue Debonding in Concrete Beam Strengthening

Externally-bonded FRP can bear tension on the tensile side of the strengthened member, thereby reducing the stress on the steel reinforcement inside the member, increasing fatigue life, reducing the deflection of the strengthened beam, and improving the crack resistance performance of the member. However, when subjected to external loads, the bonded FRP may endure debonding problems. After FRP debonding occurs, the tension originally borne by FRP in the debonding area will distribute to the steel rebars, increasing the stress on the steel rebars and seriously affecting the mechanical performance of the strengthened members [23–25]. In order to study the effects of FRP fatigue debonding in strengthened beams, the authors' group conducted a series of experimental studies [22,26]. The CFRP plate used in the tests was 1.4 mm in thickness and 100 mm in width. The tensile strength and longitudinal elastic modulus were 2758 MPa and 175 GPa, respectively. The details of the fatigue beam tests are shown in Figure 7. The test program and results are listed in Table 1. All experimental beams experienced FRP debonding during both the static and fatigue tests. The final fatigue failure modes of the strengthened beams were the same, which were the fatigue fracture of the tensile steel rebars at the main crack of the cross-section under one loading point. Research has shown that the debonding of FRP would affect the fatigue life of the strengthened beams by affecting the stress of the steel rebars [22].

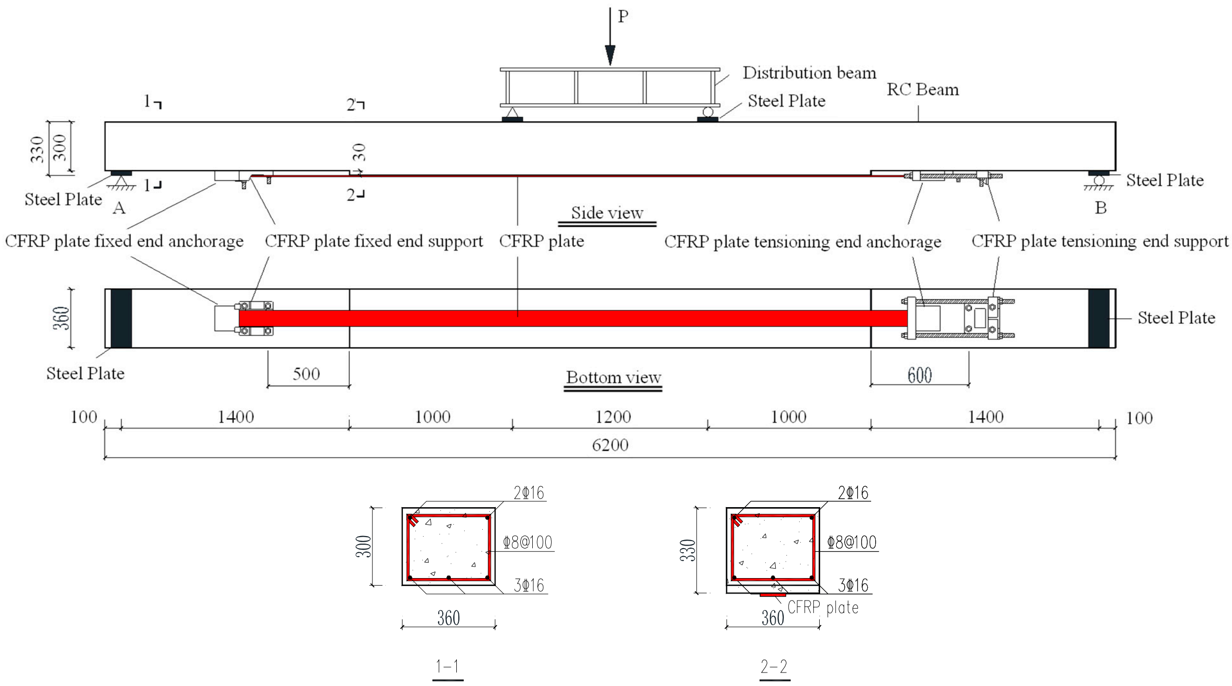

Figure 7. Details of the fatigue test specimens [22] (unit: mm).

Table 1. Summary of the test program and results of each fatigue specimen in [22].

Specimen	Designed Effective Prestress (MPa)	Maximum Load during Fatigue Test P_{max} (kN)	Minimum Load during Fatigue Test P_{min} (kN)	Fatigue Load Level $S_{max} = P_{max}/P_u$ *	Fatigue Life (N_f, Cycles)	Failure Mode
BF1-PS	1000	64	20	0.55	280,861	I *
BF2-PS	1000	54	20	0.47	883,645	I
BF3-PS	1000	48	20	0.41	>2,000,000	II *
BF4-S	0	48	20	0.46	351,761	I

* P_u—The ultimate load capacity of the specimen determined by monotonic testing. Naming of specimens: "F" = Fatigue test, "S" = Strengthened, "P" = Prestressed. Failure mode: I—fatigue fracture of steel reinforcement; II—did not fail during fatigue testing and was therefore monotonically loaded to failure after 2,000,000 loading cycles.

Effective interfacial stress transfer is the foundation of the collaboration between the bonded FRP and concrete soffit in externally bonded and FRP-strengthened concrete flexural members. However, under static or fatigue loads, there may be debonding between FRP and concrete, which greatly weakens the strengthening effect and accelerates the failure process of the strengthened members. It is generally believed that the interfacial stress between FRP and concrete is composed of interfacial shear stress and interfacial normal stress, and interfacial shear stress is the major part [27,28]. The interfacial normal stress only affects specific areas in certain special cases [29–31]. For example, at the plate end of externally-bonded FRP without mechanical anchoring, due to sudden changes in flexural stiffness near the FRP cut-off position, interfacial normal stress is generated between the FRP and beam, resulting in the problem of FRP plate-end debonding [32,33]. Moreover, when the beam on both sides of the inclined crack undergoes vertical relative displacement, normal stress will also be generated in the bonding interface between the FRP and the beam, thereby exacerbating the debonding of FRP. However, these bonding problems caused by interface normal stress can be solved by adding mechanical anchoring or pressure steel plates, and it is generally believed that normal stress is not the key to the debonding problem in externally bonded and FRP-strengthened members [27–31].

The Intermediate Crack-induced debonding (IC debonding) [27,30,34–36], which is dominated by interfacial shear stress, generally occurs in FRP-strengthened concrete flexural members with robust anchorages at the plate ends. The IC debonding commonly initiates from the major bending crack under the loading point and propagates toward the end of the CFRP plate. The IC debonding during the fatigue loading process will further lead to the formation of partially bonded prestresses in the strengthened beam, which is between bonded and unbonded prestressed members. That is, in the strengthened beam, the FRP and concrete beam in the bonded beam section maintain good adhesion and are bonded prestressed members in this section. However, in the beam section where IC debonding occurred, the FRP and the concrete bottom debonded, resulting in an approximate unbonded prestressed state. Therefore, FRP has completely different stress states in the bonded and debonded sections and has the stress transfer phenomenon between FRP and concrete in the transition zone of the two sections. The determination of the stress state of the strengthened beam plays a decisive role in the analysis, thereby affecting the fatigue life of the strengthened member. The FRP fatigue debonding is also coupled with the accumulation of fatigue damage in the concrete beam itself, as shown in Figure 8. For example, the stress amplitude of the steel rebars in specimen BF2-PS in [22] is lower than the fatigue requirements in the relevant National Design Code of China, "Code for the design of concrete structures (GB50010)", that is, the theoretical fatigue life should exceed 2 million cycles. However, with fatigue loading, debonding occurred in the beam, leading to an increase in stress amplitude in the steel rebars, and the beam ultimately failed at only 0.88 million cycles during the fatigue test. Therefore, the fatigue-checking calculation of externally bonded and FRP-strengthened concrete members cannot be simply completed by limiting the stress amplitude of the steel rebars.

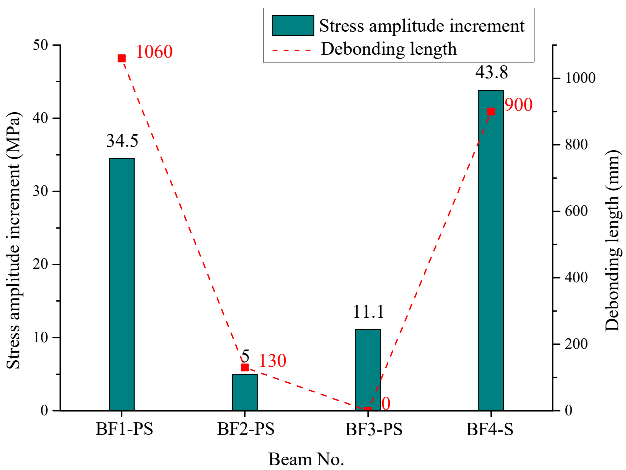

Figure 8. Steel reinforcement stress amplitudes and CFRP plate debonding lengths in the fatigue loading specimens after the 10,000th loading cycle [22].

In order to prevent the fatigue failure of flexural members with externally-bonded FRP laminates, some countries/regions have proposed requirements for the stress or strain limits of the FRP (as shown in Table 2). The ACI [3] in the United States believes that the creep and fatigue characteristics of FRP materials should be comprehensively considered, and it is stipulated that the stress in the FRP material on the strengthened beam should not exceed 55% of its ultimate strength regardless of static or fatigue loads. However, this regulation is not based on the impact of fatigue loading, as it does not take into account the average stress level of fatigue load. The Concrete Society in the UK [5] believes that, when bonded with FRP laminates, the stress limit of the FRP can reach 80% of the tensile strength. The European fib [4] believes that there is no essential difference in fatigue issues between FRP-strengthened beams and ordinary reinforced concrete beams. Therefore, it is recommended that the allowable fatigue stress range of longitudinal bars in FRP-strengthened beams and ordinary beams be consistent, which also omits the FRP debonding issue. The Canadian CSA S806 [37] stipulates that the strain of FRP should be less than 0.007 to prevent bonding failure. The Italian CNR [38] stipulates that the safety factor for all FRP materials is set at 0.5 to prevent potential FRP material failure. These existing provisions imply that the fatigue failure of FRP-strengthened beams may occur within the FRP materials, which means that the fatigue fracture of bonded FRP laminates may occur under fatigue loading. However, the fatigue failure of FRP-strengthened beams generally manifests as the fatigue fracture of steel rebars or the debonding of FRP. Brena [39,40] pointed out that FRP sheets can only use 15% to 25% of their ultimate strength in practical use, which is far from reaching the stress limit specified by the ACI. Many other literature [41–44] also hold the same view and believe that existing regulations or studies ignore the impact of FRP debonding on the stress of steel rebars in concrete beams. Therefore, it is necessary to propose an index that can comprehensively consider the influence of prestress, FRP stress and steel reinforcement stress. By limiting this index, it can be ensured that the externally bonded and FRP-strengthened concrete members can prevent FRP debonding issues and fatigue failure.

Table 2. Stress limits for externally-bonded FRP in various countries.

Country/Region	Code/Guideline	CFRP	GFRP	AFRP
United States	ACI 440.2R-02	$0.55 f_{fu}$	$0.20 f_{fu}$	$0.30 f_{fu}$
UK	Technical Report 55	$0.80 f_{fu}$	$0.30 f_{fu}$	$0.70 f_{fu}$
Europe	fib Biulletin 14	-	-	-
Italy	CNR-D 200/2004	$0.50 f_{fu}$	$0.50 f_{fu}$	$0.50 f_{fu}$

Note: f_{fu} is the tensile strength of FRP laminate.

4. Fatigue Prediction Model for the FRP Strengthened Beams

Developing a model to analyze FRP-strengthened concrete beams under fatigue loading requires considering the influence of many parameters and variables, such as fatigue load level, loading rate, concrete strength, steel reinforcement ratio, FRP material properties, etc. Therefore, it is very difficult to make the model accurately reflect the actual situation of FRP debonding, concrete degradation, and damage accumulation of the steel reinforcement. To accurately predict the fatigue life of concrete beams with externally-bonded FRP while considering computational efficiency, the existing models have adopted different degrees of simplification, such as the assumption of plane section and the assumption of perfect bonding of FRP throughout the fatigue loading process. These assumptions greatly reduce the complexity of fatigue life prediction but also, to some extent, sacrifice accuracy. Existing experimental studies have shown that the interface between FRP and concrete will experience a certain degree of debonding under fatigue loading, and this issue could cause stress redistribution in the strengthened beam, that is, a decrease in FRP stress and an increase in rebar stress, resulting in a significantly lower fatigue life of the strengthened beam than expected. Meanwhile, in prestressed FRP-strengthened concrete beams, the debonding of FRP is related to the level of prestressing, considering the application of prestressing undoubtedly increases the complexity of the fatigue prediction model.

A full-range fatigue life prediction model for RC beams strengthened with prestressed CFRP plates was proposed by the authors [45], which can comprehensively consider the fatigue damage accumulation process of the beam, the FRP debonding growth, and the stress redistribution of the beam caused by FRP debonding. This model was based on the finite strip method and was analyzed by using Matlab programming. The model can obtain the tensile stress of steel rebars, the length of FRP debonding length, and the strain of CFRP plates under any number of fatigue loading cycles.

The key to this prediction model is to analyze the beam by dividing it into multiple segments based on whether FRP debonding occurs and whether there is shear force in the beam segment. The critical section is under one of the loading points. It was found that the debonding caused by fatigue loading started at the cross-section below the loading point and gradually propagated towards its adjacent support with the increase of loading cycles. Therefore, the beam was analyzed in three different segments along the beam length, namely (a) the pure bending bonded segment (PBBS), (b) the debonded segment in the shear span (DBS), and (c) the bonded segment in the shear span (BS), as shown in Figure 9. Due to the FRP debonding in segment DBS, there is no shear transfer between the FRP and concrete in this segment, and the deformations are not coordinated between the FRP and concrete bottom. The flowchart of the analysis process is shown in Figure 10, and the detailed process can be found in another paper written by the authors [45].

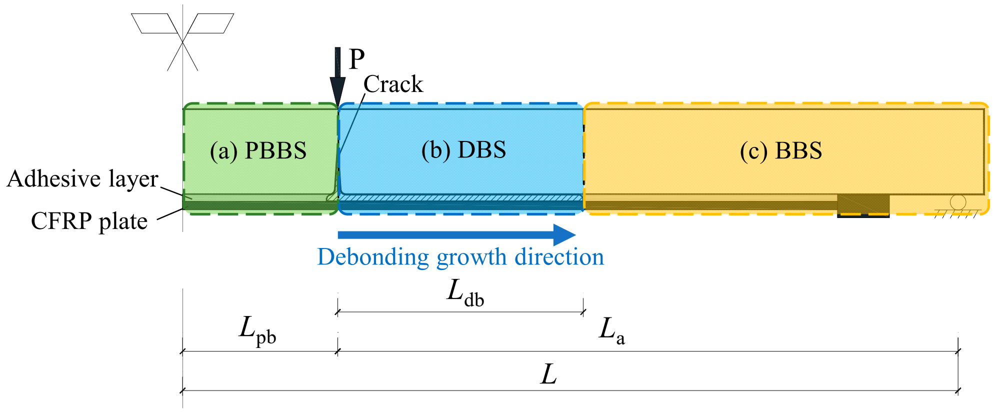

Figure 9. Beam segmentation in the prediction model (where L_{db} is the FRP debonding length, L_{pb} is the length of the pure bending region, L_a is the length of the shear span, and L is half of the beam span).

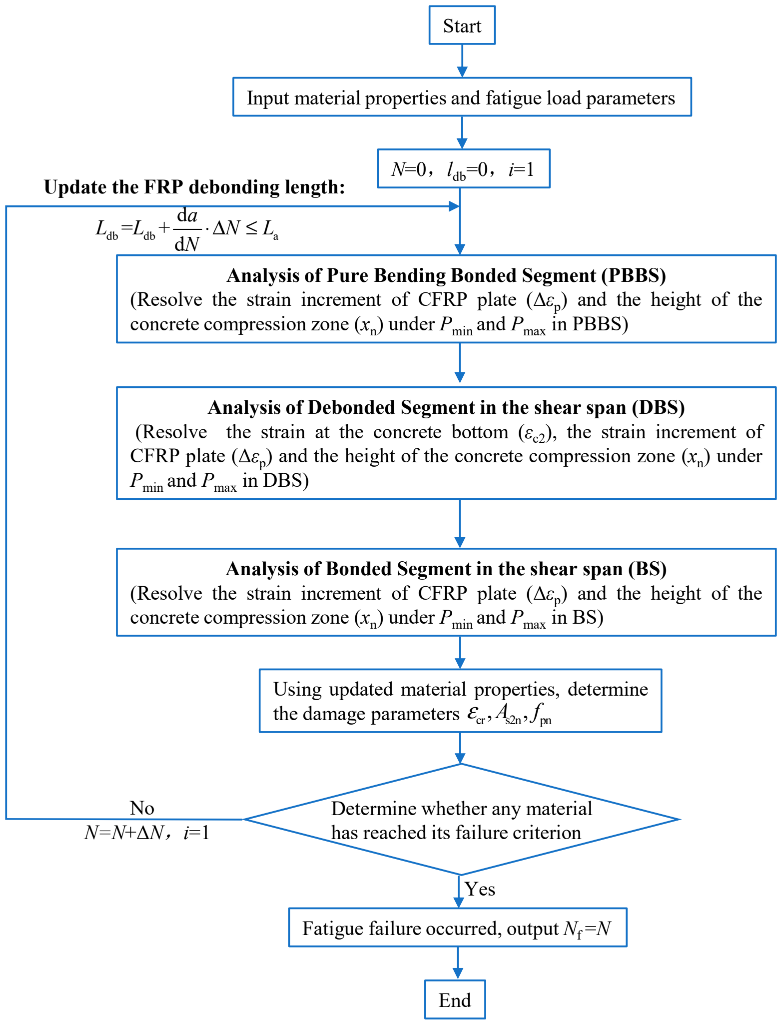

Figure 10. Flow chart for the full-range fatigue prediction model proposed by the authors [26].

5. The Proposed Stress Threshold for Preventing FRP Fatigue Debonding in the Strengthened Concrete Beam

There are many factors that can affect the fatigue life of the beams. Therefore, based on the proposed fatigue prediction model and according to the dimensions and reinforcement of the test beams in [22] as the benchmark (see Table 1), the main parameters affecting the fatigue life of CFRP plate reinforced concrete beams are quantitatively analyzed, and recommendations for the key parameters and the threshold of FRP debonding due to fatigue loading in the externally bonded and FRP-strengthened concrete beams are proposed.

Figure 11 shows the evolutions of critical FRP strain for prestressed strengthened beams under various fatigue loadings. For each beam, the effective prestress (σ_{pe}) is 1000 MPa, and the lower limit of fatigue load (P_{min}) is 20 kN. The upper load limit (P_{max}) varied from 45 kN to 64 kN. From Figure 11, it can be seen that the evolutions of FRP strain under the loading point show two trends as follows: Trend 1 (the downward-stable trend): the FRP strain decreases with the progress of fatigue loading and eventually tends to stabilize; and Trend 2 (the undulate trend): the FRP strain increases initially and then decreases with the fatigue loading cycles, and then reciprocates accordingly.

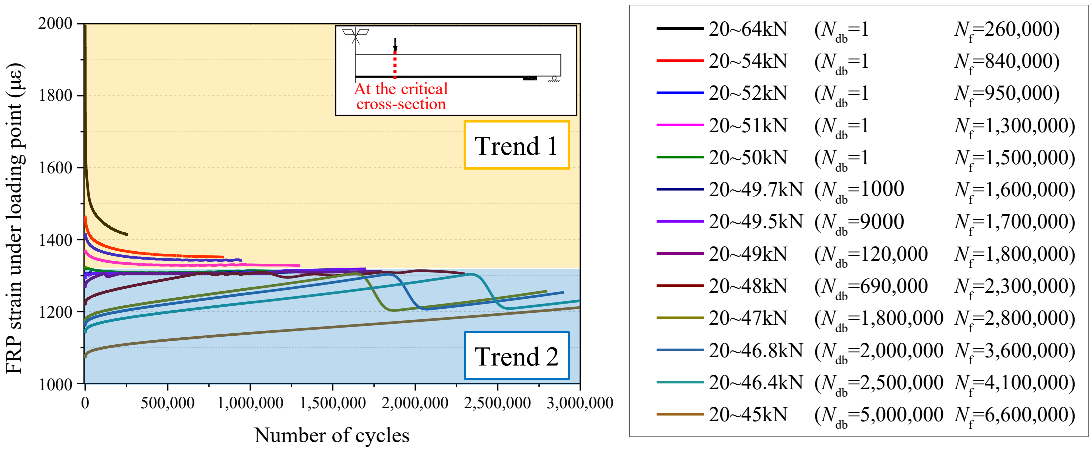

Figure 11. Evolutions of FRP strain at the critical cross-section for prestressed strengthened beams (σ_{pe} = 1000 MPa) under various fatigue loadings (Note: N_{db} is the loading cycle when the FRP debonding initiates, and N_f is the fatigue life).

For Trend 1, in the early stage of fatigue loading, the strain of FRP plates decreases continuously with the progress of fatigue loading because the fatigue load levels acting on these reinforced beams are relatively high, resulting in the FRP strain in the key section exceeding the FRP fatigue debonding strain threshold. That is, the fatigue load could lead to the debonding of FRP plates under the loading point from the beginning of tests. At the same time, even as debonding grows, the FRP strain will still remain at a high level during subsequent fatigue loading; that is, it will always exceed the debonding threshold at the interface. Debonding continues with the increase of fatigue loading cycles. FRP debonding causes the local area of the CFRP plate to be in an unbonded prestressed state. The strain in this debonded section is roughly homogenized, resulting in a continuous decrease in the strain of the FRP plate under the loading point. After the FRP debonded along the entire shear span, the strain of the FRP plate tends to stabilize, only slightly decreasing due to the deterioration of the concrete beam.

For Trend 2, in the early stage of fatigue loading, the strain of FRP plates increases continuously with the increasing number of loading cycles. The strain of FRP plates in key sections is lower than the debonding threshold, so the early fatigue loading could not lead to the initiation of FRP debonding. However, as fatigue loading continues, both concrete and steel rebars exhibit varying degrees of degradation. The degradation of rebars leads to a continuous decrease in the tensile force they bear, resulting in an increase in strain on the FRP plate. Eventually, after a certain number of fatigue loading cycles, the strain on the FRP plate exceeds the debonding threshold and initiates debonding. After the partial length of the FRP plate is debonded, the strain of the FRP plate in the debonded region decreases, which again returns to lower than the fatigue debonding threshold of the FRP plate, and the debonding growth pauses temporarily. Until the damage area of the steel reinforcement further increases due to the continuous fatigue loading, the strain of the FRP plate can increase to exceed the FRP fatigue debonding threshold at the front end of the debonding region, leading to the debonding growth of the FRP plate. Henceforth, the FRP fatigue debonding alternates between propagating and pausing.

5.1. The Impact of Prestress

Table 3 and Figure 12 show the impact of prestress in the FRP plate on the fatigue life of FRP-strengthened beams. It can be seen that when the fatigue load and FRP strengthening amount remain unchanged, the fatigue lives of the strengthened beams increase rapidly with the increase of the prestress level. When the FRP plate is not prestressed (σ_{pe} = 0), the fatigue life (N_f) of the strengthened beam under a fatigue loading of 20–54 kN is only about 170,000 cycles, which is also the shortest fatigue life among all beams. When prestressing the FRP plate and the effective prestressing reaches 500 MPa (approximately 21% f_p, where f_p is the FRP tensile strength), the fatigue life of the strengthened beam becomes 400,000 times, which is 2.35 times that of non-prestressed strengthened beams. By further increasing the effective prestress to 750 MPa, 1000 MPa, and 1250 MPa, the fatigue lives of the strengthened beams were increased to 3.29 times, 4.94 times, and 10 times higher than that of non-prestressed specimens, respectively. With effective prestress, σ_{pe} = 1250 MPa (about 52% f_p), the fatigue life of the strengthened beam achieves about 1.7 million cycles. Furthermore, when the FRP plate is prestressed to 1500 MPa (about 63% f_p), its fatigue life can reach 5.6 million cycles. It can be seen that if the amount of FRP plate remains unchanged, a much longer fatigue life can be achieved by inducing prestress to the FRP plate. In other words, the design threshold for preventing FRP fatigue debonding should account for the impact of prestress in the bonded FRP.

Table 3. Fatigue lives of prestressed FRP strengthened beams under different fatigue loadings.

No.	Effective Prestress, σ_{pe} (MPa)	Fatigue Loading (kN)	Fatigue Life, N_f (Cycles)
1	σ_{pe} = 0	P_{min} = 20, P_{max} = 54	170,000
2	σ_{pe} = 500	P_{min} = 20, P_{max} = 54	400,000
3	σ_{pe} = 750	P_{min} = 20, P_{max} = 54	560,000
4	σ_{pe} = 1000	P_{min} = 20, P_{max} = 54	840,000
5	σ_{pe} = 1250	P_{min} = 20, P_{max} = 54	1,700,000
6	σ_{pe} = 1500	P_{min} = 20, P_{max} = 54	5,600,000

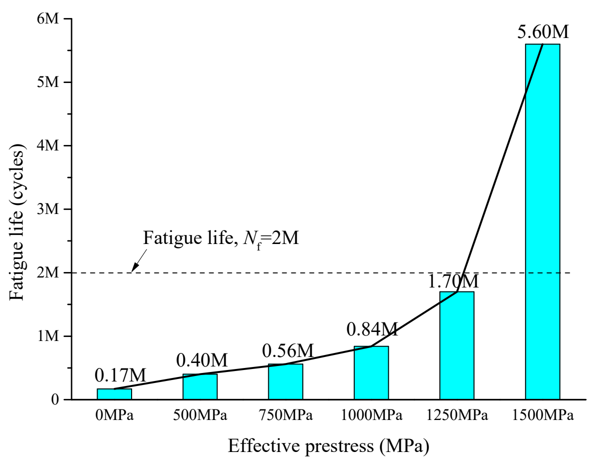

Figure 12. Relationships between effective prestresses and fatigue lives of FRP strengthened beams.

5.2. The Impact of FRP Fatigue Debonding

The fatigue debonding of the FRP plate has a significant impact on the mechanical performance of strengthened beams. In order to investigate the influence of this factor, a series of simulation analyses were carried out based on the full-range fatigue prediction model proposed by the authors [45], but without considering the FRP debonding. Specifically, throughout the entire analysis and calculation process, it is always believed that FRP was perfectly bonded to the bottom of the concrete beam, and the deformations of the FRP plate and the beam bottom were coordinated. The representative cross-sections of all elements can be analyzed by using the assumption of a flat section. Four test beams (BF1-PS, BF2-PS, BF3-PS, and BF4-S) in [22] were analyzed by this modified prediction model. The results of the analysis are shown in Table 4 and Figure 13. It can be seen that due to the neglect of FRP fatigue debonding, the stress on the steel rebars did not increase rapidly in the debonded region. When FRP debonding is not considered, the fatigue lives of the four test beams increased by 1.2 to 4.2 times compared to the results obtained by the fatigue prediction model that considers FRP fatigue debonding. Therefore, if the debonding is neglected in the analysis, it will greatly overestimate the fatigue life of the strengthened beam, resulting in significant safety hazards. Therefore, the impact of debonding also needs to be taken into consideration in strengthening design.

Table 4. Fatigue lives of FRP strengthened beams neglecting or accounting for the FRP debonding.

No.	Specimen	Fatigue Loading (kN)	Effective Prestress, σ_{pe}	Fatigue Life, N_f
1	BF1-PS, but neglect debonding	P_{min} = 20, P_{max} = 64	1000 MPa	573,000
2	BF1-PS	P_{min} = 20, P_{max} = 64	1000 MPa	260,000
3	BF2-PS, but neglect debonding	P_{min} = 20, P_{max} = 54	1000 MPa	1,925,000
4	BF2-PS	P_{min} = 20, P_{max} = 54	1000 MPa	840,000
5	BF3-PS, but neglect debonding	P_{min} = 20, P_{max} = 48	1000 MPa	5,300,000
6	BF3-PS	P_{min} = 20, P_{max} = 48	1000 MPa	2,300,000
7	BF4-S, but neglect debonding	P_{min} = 20, P_{max} = 48	0 MPa	1,267,000
8	BF4-S	P_{min} = 20, P_{max} = 48	0 MPa	300,000

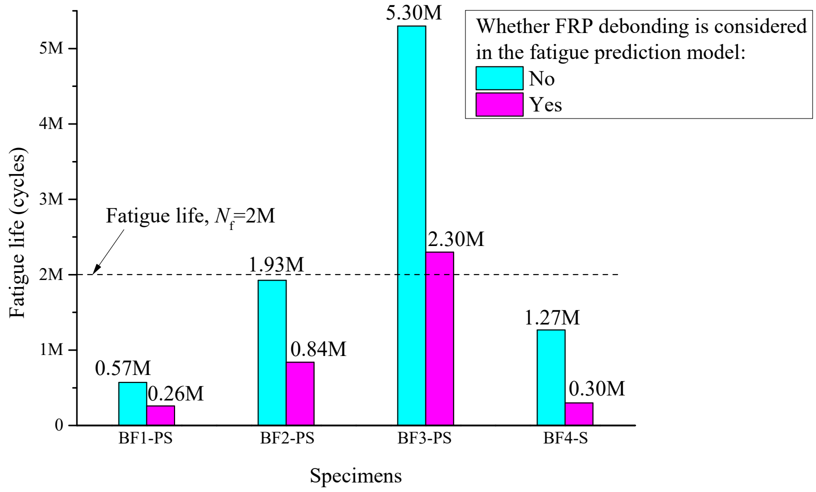

Figure 13. The impacts of FRP debonding on the predicted fatigue lives of FRP strengthened beams.

5.3. The Proposed Stress Threshold

For Trend 2 in Figure 11, which states that the debonding of the FRP plate does not occur during the initial stage of fatigue loading but only initiates after a certain number of fatigue cycles, the number of loading cycles, N, at this time is named "FRP debonding initiating a number of cycles, N_{db}," and the upper load limit corresponding to N_{db} is named $P_{db,N}$. The symbol "db" in the subscript indicates debonding, and "N" indicates the debonding initiating cycle number.

In order to be applicable to a wider range of cross-sectional dimensions and reinforcement of FRP-strengthened concrete beams, and to comprehensively consider the impacts of prestressing and FRP debonding, this paper proposes, "The tensile stress at the tensioning edge of the transformed section of the externally bonded and FRP-strengthened RC beams, $\sigma_{\mathrm{c0db},N}$". This design threshold refers to the calculation method of the cracking moment, M_{cr}, of prestressed concrete beams [46]. The rationale lies in the fact that IC debonding generally initiates from a major concrete crack, which is highly related to the calculation of M_{cr}. The "c0" in the subscript represents the transformed cross-section of concrete. The specific calculation method of $\sigma_{\mathrm{c0db},N}$ is as follows:

$$\sigma_{\mathrm{c0db},N} = \frac{M_{\mathrm{db},N}}{I_0} y_0 - \sigma_{\mathrm{pc}} \quad (1)$$

where $M_{\mathrm{db},N}$ is the upper moment limit corresponding to N_{db}, σ_{pc} is the normal stress at the tensioning edge of the beam caused by prestress, I_0 is the second moment of area of the transformed section, and y_0 is the distance from the center of the transformed section to the calculated position. The definition and calculation methods of σ_{pc}, I_0, and y0 are all the same as the pertinent provisions 10.1.6 in the national standard of China "Code for the Design of Concrete Structures (GB 50010-2010) [46]".

For example, the FRP debonding did not initiate in the strengthened beam until the 1000th loading cycle (N_{db} = 1000); the corresponding fatigue load upper limit is named $P_{\mathrm{db},1000}$, and the relevant moment is $M_{\mathrm{db},1000}$. Consequently, the maximum tensile stress of the transformed cross-section, $\sigma_{\mathrm{c0db},1000}$, can be obtained via Equation (1). Table 5 shows the $\sigma_{\mathrm{c0db},1000}$ for FRP-strengthened beams with different effective prestress. It can be seen that even for beams without the occurrence of FRP debonding during the initial loading, it is still possible for the debonding to occur after a short period of loading cycles. Taking the strengthened beam with effective prestress of 1000 MPa as an example, when $\sigma_{\mathrm{c0db},1000}$ = 4.1 MPa, the FRP plate will begin to debond after 1000 loading cycles, resulting in an increase in tensile steel stress in the concrete beam, and the fatigue life of the beam will correspondingly decrease. Therefore, for FRP-strengthened beams that did not experience FRP debonding during the initial loading stage, the debonding verification should also be carried out. Otherwise, it may result in the failure to reach the design fatigue life. The analysis results also indicate that the fatigue debonding issue needs to be taken seriously in the design threshold.

Table 5. The maximum tensile stress of the transformed cross-section of the FRP-strengthened beam when FRP debonding initiates at the 1000th loading cycle: $\sigma_{\mathrm{c0db},1000}$.

Effective Prestress, σ_{pe} (MPa)	**FRP Debonding Initiating Number of Cycles, N_{db} (Cycles)**	**Upper Load Limit Corresponding to N_{db}, $P_{\mathrm{db},N}$ (kN)**	**Upper Moment Limit Corresponding to N_{db}, $M_{\mathrm{db},n}$ (kN·m)**	**Maximum Tensile Stress of the Transformed Cross-Section Corresponding to N_{db}, $\sigma_{\mathrm{c0db},1000}$ (MPa)**
0	1000	26.7	32.0	4.6
250	1000	32.4	38.9	4.5
500	1000	38.2	45.8	4.4
750	1000	43.8	52.6	4.2
1000	1000	49.6	59.5	4.1
1250	1000	55.4	66.5	4.0
1500	1000	61.2	73.4	3.9

Therefore, Table 6 shows the design threshold, $\sigma_{\mathrm{c0db,2E6}}$, which presents the FRP-strengthened beams with different prestress levels that do not experience FRP debonding until reaching 2 million fatigue loading cycles. Taking the strengthened beam with an effective prestress of 1000 MPa as an example, when $\sigma_{\mathrm{c0db,2E6}}$ = 3.6 MP, the FRP debonding will initiate after 2 million loading cycles. Therefore, to prevent FRP fatigue from debonding within 2 million loading cycles, the tensile stress at the tensioning edge of the transformed section of the strengthened RC beam should not exceed this design threshold.

Table 6. The maximum tensile stress of the transformed cross-section of the FRP-strengthened beam when FRP debonding initiates at the 2,000,000th loading cycle: $\sigma_{c0db,2E6}$.

Effective Prestress, σ_{pe} (MPa)	FRP Debonding Initiation Cycle Numbers, N_{db} (Cycles)	Upper Load Limit Corresponding to N_{db}, $P_{db,N}$ (kN)	Upper Moment Limit Corresponding to N_{db}, $M_{db,n}$ (kN·m)	Maximum Tensile Stress of the Transformed Cross-Section Corresponding to N_{db}, $\sigma_{c0db,2E6}$ (MPa)
0	2,000,000	25.4	30.5	4.4
250	2,000,000	31.0	37.2	4.2
500	2,000,000	36.6	43.9	4.1
750	2,000,000	41.7	49.8	3.8
1000	2,000,000	46.8	56.1	3.6
1250	2,000,000	51.6	61.9	3.3
1500	2,000,000	56.5	67.8	3.1

For the fatigue verification of reinforced concrete flexural members strengthened with externally-bonded FRP laminates, if it can be ensured that no debonding occurs during the 2 million fatigue loading process, then the conventional fatigue verification method of concrete flexural members can still be used [47]. Therefore, this paper suggests that the tensile stress at the tensioning edge of the transformed section of the externally bonded and FRP-strengthened RC beams needs to be lower than the design threshold $\sigma_{c0db,2E6}$ in Table 6, which is:

$$\sigma_{c0} = \frac{M}{I_0} y_0 - \sigma_{pc} < \sigma_{c0db,2E6} \tag{2}$$

where M is the applied moment on the member, σ_{c0} is the tensile stress at the tensioning edge of the transformed section of the strengthened beam, and $\sigma_{c0db,2E6}$ is the design stress threshold shown in Table 6. The stress threshold that is not listed in the table can be calculated by interpolation corresponding to the effective prestress level.

6. Conclusions

The fatigue problem of reinforced concrete beams strengthened with externally-bonded FRP laminates involves the fatigue damage accumulation inside the beam components and the interactions between different components. For externally bonded and FRP-strengthened concrete beams, even if there is no FRP debonding initiated during the initial loading stage, it may still be generated after a certain number of fatigue loading cycles. This study was based on the fatigue life prediction model proposed by the authors, and a series of parameter analyses were conducted. Furthermore, the stress threshold for preventing FRP debonding in the strengthened beams was also proposed. The following conclusions can be drawn:

(1) Through parameter analysis, it can be found that increasing the prestress level of FRP laminates can significantly increase the fatigue life of strengthened beams when the usage amount of FRP is constant. The inducing of prestress can not only improve the strengthening efficiency but also achieve a higher utilization rate of FRP material.

(2) The evolutions of FRP strain under the loading point for the strengthened beams under fatigue loadings can be categorized into two trends: the downward-stable trend and the undulate trend. The second trend can be used to guide the proposal of the stress threshold of FRP fatigue debonding in the strengthened beams.

(3) For the fatigue prediction of externally bonded and FRP-strengthened beams, the fatigue life of the beam could be greatly overestimated if the impact of FRP fatigue debonding is not taken into account, which can further lead to serious safety hazards.

(4) It can be found through parameter analysis and experimental observation that even if the FRP plate does not debond during the initial fatigue loading stage, the debonding can still initiate during the subsequent loading process. Therefore, based on the fatigue life prediction model proposed by the authors, a design stress threshold, $\sigma_{c0db,2E6}$, was proposed to prevent FRP fatigue debonding in the externally bonded and FRP-strengthened concrete beams. The tensile stress at the tensioning edge of the transformed beam section (σ_{c0}) needs to be lower than the stress threshold ($\sigma_{c0db,2E6}$) to ensure that there is no FRP debonding initiating during 2 million fatigue loading cycles. Further research is still desired to validate this suggestion.

(5) In general, to obtain a higher performance of the FRP-strengthened concrete beam, the FRP laminate has to be prestressed to maximize the utilization of its high tensile strength. In addition, robust mechanical anchorages need to be used at the ends of FRP laminate in order to prevent the debonding from its cut-off ends. Furthermore, the strengthened beams should be served within a reasonable load range and lower than the stress threshold to prevent IC debonding from the intermediate concrete crack opening. Thus, the advantages of FRP materials could be utilized to a greater extent.

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