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# Reinforced concrete beams strengthened in flexure with near-surface mounted (NSM) CFRP strips: Current status and research needs

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## Abstract

The near-surface mounted (NSM) FRP strengthening technique has attracted worldwide attention as an effective alternative to the externally bonded (EB) FRP strengthening technique. In the NSM FRP strengthening method, grooves are first cut in the concrete cover of a concrete member for the FRP reinforcement to be inserted and embedded using an adhesive. The NSM FRP method has many advantages over the EB FRP method, including a higher bonding efficiency and a better protection of the FRP reinforcement. Existing experimental studies have shown that FRP strips owned a better bond efficiency compared with other section shapes (e.g. round bars and square bars), due to the fact that they had a larger perimeter-to-cross-sectional area ratio. This paper presents a state-of-the-art review, particularly on the flexural strengthening of RC beams with NSM CFRP strips. The observed failure modes in laboratory experiments of such FRP-strengthened RC beams are classified and the existing strength models are examined along with the failure mechanisms behind. The main knowledge gaps to be bridged in future studies are also identified. This review partially formed the basis of the development of design provisions on the NSM strengthening technique in the relevant Hong Kong design guideline.





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# Reinforced concrete beams strengthened in flexure with near-surface mounted (NSM) CFRP strips: current status and research needs

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**Abstract:** The near-surface mounted (NSM) FRP strengthening technique has attracted worldwide attention as an effective alternative to the externally bonded (EB) FRP strengthening technique. In the NSM FRP strengthening method, grooves are first cut in the concrete cover of a concrete member for the FRP reinforcement to be inserted and embedded using an adhesive. The NSM FRP method has many advantages over the EB FRP method, including a higher bonding efficiency and a better protection of the FRP reinforcement. Existing experimental studies have shown that FRP strips owned a better bond efficiency compared with other section shapes (e.g. round bars and square bars), due to the fact that they had a larger perimeter-to-cross-sectional area ratio. This paper presents a state-of-the-art review, particularly on the flexural strengthening of RC beams with NSM CFRP strips. The observed failure modes in laboratory experiments of such FRP-strengthened RC beams are classified and the existing strength models are examined along with the failure mechanisms behind. The main knowledge gaps to be bridged in future studies are also identified. This review partially formed the basis of the development of design provisions on the NSM strengthening technique in the relevant Hong Kong design guideline.

**Keywords:** FRP, near-surface mounted (NSM), strip, RC beams

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# 1 Introduction

The externally bonded (EB) FRP method has become a prevailing technique over the last two decades [e.g. 1-2] for the strengthening of existing reinforced concrete (RC) members. In the past ten years, as a promising alternative to the EB FRP method, the near-surface mounted (NSM) FRP strengthening technique has attracted increasing worldwide attention [e.g. 3-5]. In the NSM FRP strengthening method, grooves are first cut in the concrete cover of RC members and FRP bars are then embedded into the grooves with an adhesive. FRP bars of various cross-sectional shapes can be used in the NSM FRP strengthening method, such as square, round, and rectangular bars (Fig. 1). The NSM FRP method owns many advantages over the EB FRP method, including a higher bonding efficiency and a better protection of the FRP reinforcement [e.g. 4].

De Lorenzis and Teng [4] provided a detailed and critical review of the research available to them at that time on the strengthening of concrete structures with NSM FRP reinforcement. Their review covered various aspects of the NSM FRP strengthening technique (e.g. FRP reinforcement; construction aspects; bond) for various applications (e.g. flexural strengthening; shear strengthening). De Lorenzis and Teng [4] also outlined the main research needs for more extensive applications of this strengthening technique, with the bond behaviour between NSM FRP bars and concrete being identified as an important issue to be further examined. After De Lorenzis and Teng's work [4], a significant amount of research has been conducted, including experimental [e.g. 6-20], theoretical [e.g. 14, 15, 19, 21-29] and numerical [e.g. 9, 15, 30-32] studies into the behaviour of concrete structural members strengthened with various NSM FRPs. More recently, Coelho et al. [5] conducted a review on the bond behaviour of NSM FRP technique. Their review, however, was limited to the bond behavior of NSM FRP-to-concrete bonded interfaces and did not cover the behavior of NSM

FRP-strengthened RC beams. In addition, Coelho et al.'s review [5] appears to be inadequate in the sense that (1) it did not cover some of the debonding failure modes reported in the existing literature [e.g. 7, 14, 19]; (2) it did not cover the recent advances in the development of bond-slip models [e.g. 26, 33] and bond strength models which are important for a better understanding of bond behaviour between NSM FRP and concrete.

One important finding by these more recent studies is that FRP strips (rectangular bars which have a large bar height-to-thickness ratio) are superior to NSM FRP bars of other shapes in terms of the bond performance [e.g. 5, 16, 19, 34, 35] and thus the strengthening efficiency [e.g. 3, 5]. This is due to the fact that an FRP strip usually has a larger perimeter-to-cross-sectional-area ratio and a larger embedment depth than an FRP bar of other shapes, which consequently leads to a larger bond force between NSM FRP and surrounding concrete and a higher utilization of the tensile capacity of FRP. Strips made of carbon FRP (CFRP) are more attractive than other types of FRP for NSM strengthening applications due to their high strength and stiffness which could lead to a small cross-sectional area. The recent studies on NSM CFRP strips-strengthened concrete structures have led to much improved understanding of and more rational theoretical models for such structures, especially for those where CFRP strips are used for flexural strengthening. These studies have also unpinned the first ever systematic design procedure for the NSM strengthening technique in a design guideline [36] for which the authors are among the main contributors. Against this background, this paper presents a state-of-the-art review on the flexural strengthening of RC beams with NSM CFRP strips. This review partially formed the basis of the development of design provisions on the NSM strengthening in the relevant Hong Kong design guideline [36].

# 2 Behaviour and debonding failure modes of RC beams strengthened in flexure with NSM FRP

## *2.1 General Behaviour of NSM FRP-Strengthened RC Beams*

Many laboratory tests on RC beams strengthened with NSM round FRP bars or square FRP bars have been conducted to investigate this promising technique [e.g. 3, 8, 37-41]. A significant number of experimental studies have also been conducted on RC beams strengthened in flexure with NSM CFRP strips (referred to as NSM CFRP RC beams hereafter for simplicity) in the past two decades [e.g. 3, 20, 42-56]. The existing experimental studies on NSM CFRP RC beams generally show a significant enhancement of the flexural capacity of the strengthened RC beam, with the maximum percentage increase in the flexural capacity being more than 200%. The exact amount of enhancement depends on the amount of FRP, the steel reinforcement ratio and the failure mode, among others. Compared to the results of RC beams strengthened with externally bonded FRP plates (referred to as FRP-plated RC beams hereafter for simplicity), a much higher utilization of the tensile capacity of the FRP was observed in NSM CFRP RC beams [e.g. 3, 20, 44, 48, 50]. Furthermore, similar to the observation from bonded joint tests [e.g. 34, 35], NSM CFRP strips showed much higher bond efficiency than NSM FRP round bars in NSM CFRP RC beams [e.g. 3], owing to the higher perimeter-to-cross-sectional area ratio of the former.

From the load-deflection curves at the mid-span of most NSM CFRP RC beams, it was shown that NSM CFRP strips did not contribute much to the flexural stiffness of the beam in the elastic stage (i.e. before concrete cracking). After cracking, however, the flexural stiffness of the beam can be significantly increased compared with an un-strengthened beam. The flexural strength as well as the ductility of NSM CFRP RC beams was much higher than FRP-plated RC beams [e.g. 3, 44]. Using U-shaped FRP/steel jackets for end anchorage of

NSM CFRP strips was shown to postpone the debonding failure of FRP and thus significantly improve the ductility of the beam, although the increase in the flexural capacity was not apparent [e.g. 50, 54, 57]. Information on the effect of U-shaped jacketing on the effectiveness of NSM FRP used for flexural strengthening, however, is very limited.

### *2.2 Debonding failure modes*

Despite a relatively strong bond between NSM CFRP strips and concrete, debonding failures are still likely to happen in RC beams strengthened in flexure with NSM CFRP strips. In the context of simply-supported NSM CFRP RC beams, debonding failure modes are likely to occur both at the ends of NSM CFRP strips and in the maximum moment region. Apart from interfacial debonding that occurs at or near a bi-material interface, debonding may also occur in the form of separation of concrete cover where the concrete cover containing the NSM CFRP strips are detached along the level of the steel tension bars. In this paper, the term "debonding" refers to both interfacial debonding failure and cover separation failure; that is, it refers to all failure modes where the composite action between the FRP and the concrete beam is not maintained. In the experimental studies of NSM CFRP RC beams, in addition to the two conventional failure modes of RC beams, namely, flexural failure by crushing of compressive concrete [e.g. 47, 49, 58] and flexural failure by rupture of FRP [e.g. 3, 42], the following debonding failure modes have been reported:

1) ***Intermediate crack induced debonding (referred to as IC debonding hereafter) failure*** [e.g. 49, 50]. In this failure mode, the debonding of the CFRP strip starts from the maximum moment region and propagates to one of the FRP strip ends. A typical schematic diagram of the IC debonding failure is illustrated in Fig. 2. As can been seen from Fig. 2, the IC debonding failure can be further divided into two sub-types: ***IC interfacial debonding*** [50] and ***IC cover separation*** [49]. In the IC interfacial debonding, the debonding happens between the CFRP strip and the surrounding concrete (more

accurately, in the thin concrete layer adjacent to the adhesive layer). In the IC cover separation failure, the CFRP strip together with the concrete cover is detached from the beam starting from the maximum moment region, with a major crack travelling on the plane of the steel tension bars; and

2) ***End debonding failure*** [e.g. 42-45, 51, 54, 59]. In this failure mode, the debonding of the CFRP strip starts from one end of the FRP strip and propagates to the mid-span of the beam. This failure mode is mainly due to the high interfacial shear and normal stresses caused by the abrupt termination of the CFRP strip [23, 29]. A typical schematic of the end debonding failure is illustrated in Fig. 3. As can been seen from Fig. 3, the end debonding failure can also be further divided into two sub-types: **end interfacial debonding** [e.g. 42, 54] and **end cover separation** [e.g. 20, 43-45, 51-53]. Except for the starting points of the debonding, end interfacial debonding and end cover separation are quite similar to their counterparts IC interfacial debonding and IC cover separation respectively.

While the above failure modes were reported in the existing literature, some researchers only indicated that failure of the beam was caused by concrete cover separation but did not mention where the failure initiated [e.g. 48, 52]. Among the above debonding failure modes, cover separation (i.e. IC cover separation and end cover separation) has been found to be more common than interfacial debonding (i.e. IC interfacial debonding and end interfacial debonding) in NSM CFRP RC beams. Possible reasons for this phenomenon include: (1) the strong bond between NSM CFRP strips and concrete makes the interfacial debonding failure less likely; and (2) the large radial stresses, exerted by the steel tension bars to the surrounding concrete during their tension process [31], plays an important role in accelerating the cracking in the concrete along the level of steel tension bars. Nevertheless, interfacial

debonding is also an important debonding failure mode, especially for NSM FRP-strengthened RC beams with a relatively large beam width [28]. The present paper aims to clarify the failure mechanism of the above debonding failure modes, to summarize the established strength models and to identify the gaps of knowledge for future research.

# 3 IC debonding

Although IC interfacial debonding and IC cover separation are both termed as debonding failure, the intrinsic failure mechanisms of them are quite different. The interfacial debonding failure happens on the NSM CFRP strip-to-concrete interfaces and the debonding strength is thus mainly controlled by the material and/or interfacial properties on/near such interfaces. Obviously, in order to find out the debonding mechanism and establish strength models for interfacial debonding failures in NSM CFRP RC beams, the fundamental issue is to expose the bond behaviour of the NSM CFRP strip-to-concrete interface. In contrast, the cover separation failure happens on the horizontal plane of tension steel bars with both concrete cover and FRP detached from the RC beam. Therefore, in order to establish the strength models for cover separation failures in NSM CFRP RC beams, one of the fundamental issues is to clarify the failure mechanism on the horizontal plane of tension steel bars.

## *3.1 IC Interfacial Debonding*

### *3.1.1 Failure mechanism*

The failure process and mechanism of IC interfacial debonding is usually as follows: when a dominating flexural crack occurs in/near the maximum moment zone, the tensile stress in the concrete releases and is transferred onto the tension steel bars as well as FRP reinforcement through interfacial shear stresses. Near the intersection of FRP reinforcement and the dominating flexural crack, high local interfacial shear stresses happens as a result of the geometric discontinuity of concrete due to the existence of the flexural crack. These high

local interfacial shear stresses increase as the applied load increases and finally result in the initiation of debonding between FRP and concrete while it reaches a critical level. Afterwards, the IC interfacial debonding process is mainly driven by the opening of the flexural crack which causes relatively displacements between FRP and concrete. The propagation of debonding therefore strongly depends on the bond behaviour of NSM CFRP strip-to-concrete interfaces, which can be investigated through the tests of NSM CFRP strip-to-concrete bonded joints as shown in Fig. 4. In other words, the findings from such bonded joints has the potential to be used for predicting the force (stress) that can be developed in the NSM CFRP strip at IC interfacial debonding. In the following subsections, the existing studies on NSM CFRP-concrete bonded joints/interfaces are first reviewed based on which recommendations on the bond strength of IC interfacial debonding in an NSM CFRP RC beam are provided.

*3.1.2 Behaviour of NSM CFRP strip-to-concrete interfaces*

The interfacial bond behaviour has been commonly studied using pull-out tests on NSM FRP-to-concrete bonded joints. By far, the beam pull-out test and the direct pull-out test have been adopted by existing studies to study the bond behavior between NSM FRP bars and concrete. The former one had been widely used for the study of the bond characteristics of steel bars and was introduced by Nanni et al. [60] for NSM FRP bars. The later one has three main sub-types: one-side pull-out test [e.g. 6, 21, 45, 61, two-side pull-out test [e.g. 62], and C-shaped block pull-out test [e.g. 37, 38]. As the one-side pull-out test is the simplest one to be implemented in laboratory experiments and its loading mechanism is direct and clear, it has been the most common test method adopted by researchers.

*Failure modes*

A number of failure modes have been observed in experimental studies of NSM CFRP strip-to-concrete bonded joints, including: (1) adhesion failure on the strip-to-adhesive

interface [18, 19, 58, 61, 63, 54]; (2) adhesion failure on the adhesive-to-concrete interface [14, 18, 19, 34, 35, 64]; (3) cohesion failure in a thin layer of concrete near the adhesive-concrete interface [6, 7, 14, 18, 19, 21, 63-66]; (4) cohesion failure in the adhesive [7, 16, 34, 61]; (5) splitting of the concrete block [7]; and (6) rupture of FRP strip [14, 18, 19]. Among these failure modes, the splitting of concrete block only happened in the specimens where the CFRP strips were deliberately embedded much deep in concrete blocks [7]. This failure mode is therefore unlikely to occur in RC beams as the embedment depth of FRP strips in RC beams is generally limited by the thickness of concrete cover. The rupture of FRP strip depends on the tensile strength of FRP and is thus not a property of the bonded interface. The adhesion failure at strip-to-adhesive or adhesive-to-concrete interfaces is largely a result of the poor surface condition at the corresponding interface (e.g. the surface of the CFRP strip or groove is not well cleaned) while the cohesion failure in the adhesive generally occurs when a weak adhesive is used or when the adhesive thickness is too small so that the stress concentration in the adhesive layer is significant. These three failure modes are not the desired failure modes as the failure occurs in the strengthening system and thus the strengthening efficiency would be significantly compromised. Indeed, these failure modes should be avoided in the design by a proper groove size, an appropriate treatment of the interfaces and selection of adhesives. Therefore, existing studies [4, 67] have suggested that the preferred failure mode is cohesion failure in a thin layer of the concrete near the adhesive-to-concrete interface. With such failure mode, the bond strength is governed largely by the concrete properties but not the properties on the adhesive-to-concrete interface or the CFRP-to-adhesive interface, so the development of a design theory is also easier. A few bond-slip and bond strength models have been proposed for this failure mode and are discussed later in this section.

*Bond-slip models*

An accurate bond-slip model for the NSM CFRP strip-to-concrete interface is crucial to an in-depth understanding of debonding failures in NSM FRP-strengthened RC members, and is necessary for developing accurate bond strength models for NSM CFRP strip-to-concrete interfaces. In addition, an accurate bond-slip model is critical to establishing accurate finite element models of NSM FRP-strengthened RC members for predicting debonding processes. Compared to bond-slip models developed for EB FRP laminate/plate-to-concrete interfaces [e.g. 68, 69], existing work on bond-slip models for NSM FRP strip-to-concrete interfaces is still relatively limited. Similar to EB FRP systems, the bond-slip model for NSM FRP-to-concrete interfaces can be developed using experimental approaches (i.e., direct regression of experimental results) [e.g. 21, 38, 70] or numerical parametric studies [e.g. 69]. When the experimental approach is adopted, the bond-slip model may be determined from axial strain distributions of the CFRP bar obtained by strain instruments [37] or from the average bond stress versus average slip (obtained from loaded-end slip and free-end slip) curve [e.g. 38]. The large scatter of test results as a result of the heterogeneity of concrete [4] may influence the accuracy of the proposed bond-slip curves. In addition, the bond behaviour on the FRP-to-adhesive interface might be interfered by the installation of strain gauges for the strain measurement. By far, four bond-slip models have been proposed by De Lorenzis and her co-workers [38, 71-73] for NSM FRP round bars-to-concrete interface based on the test results. However, these bond-slip models cannot be directly applied to NSM CFRP strip-to-concrete interfaces. The stress state in the adhesive and the surrounding concrete is relatively complicated for NSM FRP round bars than for NSM FRP strips especially when ribbed bars/spirally wounded bars are used. For NSM CFRP strip-to-concrete interface, a bond-slip model was proposed by Sena-Cruz and Barros [74], based on a model adopted for steel bar-to-concrete interfaces [75]; their model was calibrated with their tests in which the

concrete was deliberately strengthened with steel fibres to avoid failure in the concrete. The bond-slip model adopted by CEB-FIP [76] for steel bar-to-concrete interfaces was recommended by Borchert and Zilch and the linear-softening bond-slip model which has the same form as that for EB FRP-to-concrete interface was used by Seracino et al. [21] for NSM CFRP strip-to-concrete interface. The validity of the above models is still uncertain in the sense that they were based on previous work on steel bar-to-concrete interfaces or on EB FRP reinforcements but not directly deduced from experimental tests or verified numerical simulations. However, the proposals of these bond-slip models provide useful attempts and help to some extent understand the bond behaviour of NSM CFRP strip-to-concrete interfaces. More recently, using the verified 3-D meso-scale FE model proposed by Teng et al. [67, 77], Zhang et al. [26] conducted a parametric study to examine the bond-slip relationship on NSM CFRP strip-to-concrete interfaces. It was found from the study that: (1) the bond-slip curve has a nonlinear ascending branch with the slope continuously decreasing; (2) the descending branch is also nonlinear with the magnitude of the slope increasing first and then decreaseing with the increase of slip; (3) the ascending and descending branches are smoothly connected. Finally they proposed the bond-slip relationship (Eq. 1) for such bonded interfaces with the currently preferred failure mode (i.e. cohesion failure in the concrete near the adhesive-concrete interface), in which the concrete cylinder strength ( $f_c$ ) and the groove height ( $h_g$ )-to-width ( $w_g$ ) ratio were finally selected as the two key parameters that influence the bond behaviour.

$$\tau = A\left(\frac{2B-s}{B}\right)^2 \sin\left(\frac{\pi}{2}\cdot\frac{2B-s}{B}\right) \quad \text{with} \quad s \le 2B \tag{1}$$

where $\tau$ is the bond stress, $s$ is the slip, $A = 0.72\gamma^{0.138} f_c^{0.613}$ and $B = 0.37\gamma^{0.284} f_c^{0.006}$,

$\gamma = \frac{h_g}{w_g}$ is the groove height to-width ratio and $f_c$ is the cylinder compressive strength of concrete.

*Bond strength models*

The bond strength is the ultimate tensile force that can be developed in the FRP reinforcement in a pull-out test before the debonding between FRP and concrete occurs. It has been found in existing studies [e.g. 6, 63] that the bond strength of NSM CFRP strip-to-concrete interface increases with when the bond length is relatively small, but when the bond length exceeds a threshold value, a further increase in the bond strength cannot be obtained from a further increase in the bond length. The threshold value of bond length has been referred to as the effective bond length ( $L_e$ ) [e.g. 78, 79]. The fracture mechanics-based approaches [78, 81] can well interpret the existence of an effective bond length and can be used to establish the bond strength based on a given bond-slip curve [80, 81]. By now, four bond strength models of NSM CFRP-to-concrete interfaces have been proposed for the currently preferred failure mode. They are introduced below.

***Seracino and co-workers' model [7, 21, 65]***

Based on their test results, Seracino and co-workers [7, 21, 65] proposed a bond strength model for cohesion failure in a thin layer of concrete near the adhesive-to-concrete interface (referred to as debonding in Refs. [7], [21] and [65]). In this model, the bond strength $P_{u,s}$ of NSM CFRP strip-to-concrete interface is expressed as

$$P_{u,s} = \alpha_p 0.85 \beta_e \beta_{L,s} \gamma_s^{0.25} f_c^{0.33} \sqrt{E_f A_f L_{failure} \left( \frac{h_f + c_a}{c_a} \right)^{1.2}} \quad (2)$$

304 $$\alpha_p = \begin{cases} 1 & \text{for the mean value} \\ 0.85 & \text{for the 95\% lower bound} \end{cases} \quad (3)$$

305 $$\beta_e = 0.283\frac{a_e}{h_f} + 0.196 \le 1 \quad (4)$$

306 $$\beta_{L,s} = \frac{L_b}{L_{e,s}} \le 1 \quad (5)$$

307 $$L_{e,s} = \frac{\pi}{2(0.802 + 0.078)}\sqrt{\frac{0.976\gamma_s^{0.526} E_f A_f}{f_c^{0.6} L_{failure}}} \quad (6)$$

308

309 where the reduction factor $\beta_e$ accounts for the effect of edge distance $a_e$ of concrete block
310 on the bond strength [65]; $\gamma_s$ is the height-to-width ratio of the failure contour [21] where
311 the height of contour is equal to CFRP strip height $h_f$ +1 mm and the width is equal to the
312 CFRP strip thickness $t_f$ + 2 mm; $f_c$ is the cylinder compressive strength of concrete; $E_f$
313 is the elastic modulus of CFRP strip, $A_f$ is the cross-sectional area of the CFRP strip;
314 $L_{failure} = 2h_f + t_f + 4\,mm$ is the length of the failure contour; $c_a$ is the smallest distance
315 between the CFRP strip and the surface of the concrete [7]; reduction factor $\beta_{L,s}$ accounts
316 for the bond length of CFRP strip $L_b$ on the bond strength; and $L_{e,s}$ is the effective bond
317 length.

318

319 ***Oehlers et al.'s model [23]***

320 Based on the model by Seracino and co-workers (Eq. 2), Oehlers et al. [23] proposed a bond
321 strength model for NSM FRP-to-concrete bonded joints containing several NSM CFRP
322 strips:

323 $$P_{u,o} = \alpha_p 0.85\beta_e\beta_{L,s}\gamma_s^{0.25} f_c^{0.33} L_{failure}\sqrt{n_f E_f A_f + k_{EAC} E_c A_c} \quad (7)$$

$$k_{EAC} = 0.45 - 0.03(M_u / M_{cr}) \quad (8)$$

where $n$ is the number of NSM FRP strips, $E_c$ is the elastic modulus of concrete, $A_c$ is the cross-sectional area encompassed by the failure plane, $k_{EAC}$ is a reduction factor for axial rigidity of concrete, $M_u$ is the applied moment at IC debonding (equal to zero for NSM FRP-to-concrete bonded joint ), and $M_{cr}$ is the moment at the initial cracking of the beam.

***Zhang et al.'s model [27]***

Zhang et al. [27] proposed another bond strength model, as expressed in Eq. 9. In this model, the effective bond length and the reduction factor accounting for the detrimental effect of an insufficient bond length were developed based on the bond-slip model proposed by Zhang et al. [26].

$$P_{u,z} = \beta_L \sqrt{2 G_f E_f A_f C_{failure}} \quad (9)$$

$$G_f = 0.40 \gamma^{0.422} f_c^{0.619} \quad (10)$$

$$L_{e,z} = \frac{1.66}{\eta} \quad \text{where} \quad \eta^2 = \frac{\tau_{\max}^2 C_{failure}}{2 G_f E_f A_f} \quad (11)$$

$$\beta_{L,z} = \frac{L_b}{L_{e,z}}\left(2.08 - 1.08 \frac{L_b}{L_{e,z}}\right) \text{ when } L_b < L_{e,z} \quad \text{and} \quad \beta_{L,z} = 1 \quad \text{when } L_b \geq L_{e,z} \quad (12)$$

where $L_b$ and $L_{e,z}$ are the bond length and effective bond length respectively; $G_f$ is the interfacial fracture energy between NSM CFRP strip and concrete; the cross-sectional contour of the failure surface $C_{failure}$ is equal to the sum of the three side lengths of the groove; and the reduction factor $\beta_{L,z}$ accounts for the detrimental effect of insufficient bond lengths on the bond strength.

***Bilotta et al.'s model [20]***

More recently, Bilotta et al. [20] proposed a bond strength model based on the regression of test results collected by them. This bond strength model is originally expressed in terms of maximum strain $\varepsilon_{\max}$ in the FRP (Eq. 13).

$$\varepsilon_{\max} = 157 \frac{\left(C_{failure}\right)^{0.66}}{\left(E_f A_f\right)^{0.823}} \tag{13}$$

The bond strength in terms of ultimate load can thus be obtained by multiplying Eq. 13 with $E_f A_f$:

$$P_{u,b} = E_f A_f \varepsilon_{\max} = 157 \left(C_{failure}\right)^{0.66} \left(E_f A_f\right)^{0.177} \tag{14}$$

***Discussion on bond strength models and future research needs***

Among the above existing bond strength models, Bilotta et al.'s model [20] appears most inferior as it ignores the influence of both concrete strength and bond length on the bond strength. In Oehlers et al.'s model [23], the applied moment at IC debonding in a beam needs to be given first to calculate the reduction factor $k_{EAC}$, thus it is not a truly predictive model. Furthermore, the influence of bond length on the bond strength is also not considered in this model. Comparison between Seracino and co-workers' model [7, 21, 65] and Zhang et al.'s model [27], which was reported in Ref. [27] making use of results of 51 test specimens collected from existing studies, revealed that both models provide close predictions for the test results when the bond length of the CFRP strip is sufficiently long (not smaller than the effective bond length), but the model proposed by Zhang et al. [27] performs significantly better than Seracino and co-workers' model [7, 21, 65] when the bond length is insufficient (smaller than the effective bond length). This is mainly because that the effective bond length equation and the corresponding bond length reduction factor in the model proposed by Zhang et al. [27] are both based on an accurate bond-slip relationship obtained using a verified FE

model, while the effective bond length equation in the model by Seracino and co-workers [7, 21, 65] is based on an assumed linear-softening bond-slip relationship and the bond length reduction factor is described using an assumed linear function.

It should be noted that the edge distance and the groove spacing have a significant effect on the bond strength, and their influences have not been fully studied. In the model by Seracino and co-workers [7, 21, 65], although a reduction factor accounting for the effect of edge distance was included, it was based on regression of only limited test results by them [65]. In Oehlers et al.'s model [23], although the involvement degree of the concrete encompassed by the failure plane could be reflected by the groove spacing, the effect of the groove spacing on the bond behavior of each FRP strip was still not considered. Some studies conducted by Barros and co-workers [e.g. 82-86] on RC beams strengthened in shear with NSM FRP strips can be referred to for considering the effect of groove spacing on the bond strength. Considering that the failure modes of FRP strips NSM to concrete resemble those of adhesive anchors, Barros and co-workers develop a model which relates the bond strength of NSM FRP-concrete interface in shear-strengthened beams to the so-called "semiconical fracture surface of concrete" surrounding the NSM strip. The NSM FRP strip is thought to be pulled out when the principal tensile stress of concrete on this semiconical surface exceeds the tensile strength of concrete. The method is able to consider the effect of groove spacing (i.e., the "interaction among adjacent strips" in their papers) on the bond behaviour between NSM FRP strip and concrete: when the groove spacing is small, the semiconical fracture surfaces of adjacent NSM FRP strips overlap with each other and thus the total efficient/envelop fracture area becomes smaller than the direct summation of the semiconical fracture area of each NSM FRP strip. While this method has the potential to be extended to study the effect of groove spacing on the bond strength of bonded joints with multiple NSM FRP strips, future

research is needed to develop a large experimental database on such bonded joints for verification/refinement of the method. It should be also noted that most existing studies on the bond behaviour between NSM FRP and concrete were based on ambient temperature. The effect of elevated temperature on the bond behaviour as well as the strengthening efficiency of NSM FRP strips have not been clarified, while preliminary studies have been carried out by researchers [e.g., 87-90]. Further studies are therefore needed to clarify the effects of edge distance, groove spacing and elevated temperature for more accurate bond-strength models.

*3.1.3 IC interfacial debonding strength model*

Vasquez and Seracino [24] directly used the bond strength mode proposed by Seracino and co-workers [7, 21, 65] (as expressed in Eq. 2) for NSM CFRP strip-to-concrete bonded joint to predict the force in the NSM FRP strip at IC debonding in NSM CFRP RC beams. Vasquez and Seracino [24] assessed this model (Eq. 2) using results of NSM CFRP RC beams collected from existing studies and found that the prediction-to-test ratio is 0.88. This conservative prediction is mainly because that in RC beams there usually exist more than one major flexural cracks and the debonding force in the FRP between two adjacent cracks is larger than that in one-side pull out test of the corresponding bonded joints, as has been proved by Teng et al. [91] by using an analytical solution in an EB FRP-to-concrete bonded joint. Although the above method cannot be much criticized considering that this bond strength on NSM CFRP strip-to-concrete interfaces generally offers a lower bound to the IC interfacial debonding strength of RC beams (thus provide a conservative prediction for design purpose), a more accurate design model which can take into count the influence of multi-cracks still needs to be pursued.

It can be expected that the bond strength model proposed by Zhang et al. [27] provides similar prediction of IC debonding strength to that by Seracino and co-workers [7, 21, 65], in

the sense that in RC beams the bond length of FRP is normally larger than the effective bond length. The difference between the bond strength model by Zhang et al. [27] and that by Seracino and co-workers [7, 21, 65] may only be reflected in situations where the bond length of NSM FRP is limited, such as in the shear strengthening of RC beams.

### *3.2 IC Cover separation*

The failure process and mechanism of IC cover separation is as follows: the transfer of the tensile stress from the cracked concrete onto the tension steel bars after the formation of a dominating flexural crack incurs high local interfacial shear stresses near the intersection of tension steel bars and the dominating flexural crack. Besides, as the steel bars are usually not smooth but have some ribs on it, the relative displacement between steel bars and concrete also incurs radial stresses onto the surrounding concrete as shown in Fig. 5. These high local interfacial shear stresses and radial stresses increase with the applied load and finally induce separation failure on the plane of the tension steel bars. Due to their different failure mechanisms, the bond strength of NSM CFRP strip-to-concrete interfaces cannot be used in predicting the strength of IC cover separation in NSM CFRP RC beams.

By far, although IC cover separation failure has been observed in experimental tests, it has not yet attracted enough research attention. There has been no established strength model for IC cover separation failure, probably because its failure mechanism is relatively complicated. As mentioned earlier, the failure happens on the horizontal plane of tension steel bars, and at this failure plane, the clear concrete width is smaller than the beam width because of the existence of the steel bars. In addition, radial stresses exerted by the steel tension bars to the surrounding concrete when the slip between the concrete and the steel tension bar develop have found to further weaken the critical plane [30, 31]. An FE model taking into account the above effects has been developed for establishing strength models for end cover separation in

444 EB/NSM FRP-strengthened RC beams [31] but has not yet been extended to study IC cover
445 separation. Further studies are needed to develop strength models for IC cover separation.
446

447 # 4 End debonding

448 End debonidng failure mode also contains two sub-types: end interfacial debonding and end
449 cover separation. Similar to IC debonding, these two sub-types of failure were controlled by
450 different failure mechanisms and should be treated separately. Although more and more
451 attentions have been drawn to the development of end debonding strength models in NSM
452 CFRP RC beams, the existing models have been still relatively limited.

453 ## *4.1 End Interfacial Debonding*

454 ### *4.1.1 Failure mechanism*

455 When the FRP-strengthened beam is under loading, high interfacial shear and normal stresses
456 develop near the end of NSM CFRP strip as a result of the abrupt termination of the strip [e.g.
457 25, 29]. Due to the high interfacial stresses, an inclined crack usually occurs near the end of
458 the NSM FRP, and another flexural-shear crack usually appears in the bonded region of FRP
459 at a certain distance (i.e. the crack spacing) as shown in Fig. 3a. These interfacial stresses
460 increase with the applied load and finally induce the debonding between FRP and concrete.

461 ### *4.1.2 Strength models*

462 Two strength models have been established for end interfacial debonding failure. The first
463 one is proposed by Hassan and Rizkalla [42] based on interfacial stresses between NSM
464 CFRP strip and concrete, and the other one is originally proposed by Oehlers et al. [92, 93]
465 for FRP-plated RC beams and modified by Vasquez and Seracino [24] for NSM CFRP RC
466 beams. It should be noted that existing strength models of end debonding for FRP-plated RC
467 beams are generally not applicable to NSM CFRP RC beams, because some parameters in

these models were calibrated using test results of FRP-plated RC beams [94]. The two existing strength models are introduced in details here followed by a discussion of these two models.

***Hassan and Rizkalla's model [42]***

For end interfacial debonding failure, Hassan and Rizkalla [42] proposed an approach for predicting the strength of NSM CFRP RC beams. Based on the interfacial stress analysis of Malek et al. [95] for EB FRP systems, Hassan and Rizkalla [42] gave closed-form expressions to predict the interfacial shear stress $\tau$ between NSM CFRP strip and concrete, as expressed in Eqs. 15 and 16 respectively for a simply-supported beam subjected to a point load and a simply-supported beam subjected to two symmetric point loads:

$$\tau = \frac{t_f}{2}\left[\frac{n_f P a y_{eff}}{2I_{eff}}\omega e^{-\omega x} + \frac{n_f P y_{eff}}{2I_{eff}}\right] \quad (15)$$

$$\tau = \frac{t_f}{2}\left[\frac{n_f P y_{eff}}{I_{eff}} + \frac{n_f P y_{eff} a}{I_{eff}}\omega e^{-\omega x}\right] \quad (16)$$

$$\omega^2 = \frac{2G_a}{t_a t_f E_f} \quad (17)$$

where $x$ is the horizontal distance from the strip end, $t_f$ is the thickness of the CFRP strip, $n_f = E_f / E_c$ is the modulus ratio between FRP and concrete, $P$ is the point load, $y_{eff}$ is the distance from the strip centroid to the neutral axis of the section, $a$ is the distance from the strip end to the nearest support; $I_{eff}$ is the effective moment of inertia and is expressed in Eq. 18, $e$ is the base of the natural algorithm, and $G_a$ and $t_a$ are the shear modulus and thickness of the adhesive respectively,

$$I_{eff} = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} \quad (18)$$

where $M_{cr}$ and $M_a$ are the cracking and applied moments on a beam section respectively, $I_g$ is the transformed gross moment of inertia in terms of concrete of the strengthened section, and $I_{cr}$ is the transformed moment of inertia in terms of concrete of the cracked section.

Obviously, the interfacial stresses obtained from Eqs. 15 and 16 peak when $x = 0$, indicating that the cut-off point is the critical location for the initiation of debonding failure. By introducing the Mohr-Coulomb failure criterion, the interfacial stress $\tau_{\max}$ at failure can be expressed as

$$\tau_{\max} = \frac{f_c f_t}{f_c + f_t} \quad (19)$$

where $f_c$ and $f_t$ are the cylinder compressive strength and tensile strength of concrete respectively.

Substituting Eq. 19 into Eq. 15 or Eq. 16 with $x = 0$ yields the applied load at end debonding failure of the strengthened beam.

***Oehlers et al.'s model [92, 93]***

Oehlers et al. [92, 93] referred to end debonding failures with the end of the EB FRP plate located in the shear span as the Critical Diagonal Crack (CDC) debonding. Based on Zhang's method [96] for determining the shear strength of RC beams, Oehlers et al. [92, 93] proposed a "passive prestress model" of CDC debonding for simply supported beams under

concentrated loads. In this approach, two shear forces are considered: the shear force $V_{crack}$ causing the diagonal crack (Eq. 20) and the shear force $V_{slide}$ causing the sliding of the diagonal crack (Eq. 21). CDC debonding failure is assumed to occur when these two shear forces become equal to each other.

$$V_{crack} a_v = (x_d^2 + h^2)\left(\frac{b f_{tef}}{2} + \frac{f_t n_f A_f}{h}\right) + F_{ps} h_{ps} \tag{20}$$

$$V_{slide} = 0.4 f_c b h \left(1 + 2\frac{F_{ps} + P_{axial}}{f_c b h}\right)\left(\sqrt{1 + \left(\frac{x_d}{h}\right)^2} - \frac{x_d}{h}\right) f_1 f_2 f_3 \tag{21}$$

$$f_1 = \frac{3.5}{\sqrt{f_c}} \tag{22}$$

$$f_2 = 0.27\left(1 + \frac{31.6}{\sqrt{h}}\right) \tag{23}$$

$$f_3 = 15\frac{A_s}{b h_0} + 0.58 \tag{24}$$

where $f_{tef} = 0.156(f_c)^{2/3}\left(\frac{h}{100}\right)^{-0.3}$ and $f_t$ are the the effective tensile strength and tensile strength of concrete respectively, $a_v$ is the shear span of the beam, $x_d$ is the horizontal distance between the bottom position of the diagonal crack and the applied concentrated load, $h$ is the beam height, $A_f$ is the cross-sectional area of FRP, $P_{axial}$ is the maximum axial force in the FRP, $F_{ps}$ is the tendon prestressing force, $h_{ps}$ is the depth of the tendon prestressing force position, and $f_1$, $f_2$ and $f_3$ are functions of concrete strength, beam height and tension reinforcement ratio respectively.

For FRP-plated RC beams, the value of $P_{axial}$ can be calculated using the bond strength model proposed by Chen and Teng [78], as recommended by Oehlers et al. [92, 93]. To make

529 this model applicable to NSM CFRP RC beams, Vasquez and Seracino [24] recommended
530 that the value of $P_{axial}$ can be predicted by the bond strength model proposed by Seracino et
531 al. [21] for NSM CFRP strip-to-concrete interface.

532 *4.1.3 Discussions and future research needs*

533 Hassan and Rizkalla' model [42] offered a valuable pioneering study on end debonding
534 strength models in RC beams strengthened with NSM CFRP strips. In this model, however,
535 only the longitudinal shear stress is taken into account to determine whether debonding
536 failure occurs, which to some extent lacks rigor. Vasquez and Seracino [24] assessed this
537 model and found that the model was significantly conservative. This implies that the local
538 failure in the concrete layer at the end of the FRP strip does not mean the debonding failure
539 of the beam. As indicated by Vasquez and Seracino [24], Oehlers et al.'s model [92, 93] may
540 be overly conservative, because the contribution of stirrups was not taken into account. The
541 omission of the contribution of stirrups may be acceptable for FRP-plated RC beams but not
542 reasonable for NSM CFRP RC beams. This is because that the debonding strain in the NSM
543 FRP strip is usually larger than that in EB FRP plate, thus a higher strain in stirrup can be
544 possibly developed. A comparison made by Vasquez and Seracino [24] showed that Oehlers
545 et al.'s model [92, 93] gave an average prediction-to-test ratio of 0.74 for the collected
546 specimens.

547

548 Obviously, the existing strength models for end interfacial debonding are quite limited. The
549 interfacial stress based model [42] usually underestimates the debonding strength of the beam.
550 The local failure in the concrete at the end of NSM CFRP strip cannot be treated as the
551 debonding failure of the strengthened RC beam, as cracks on the tension surface of the beam
552 makes interfacial stress redistribute before debonding failure. The prestress beam model [92,
553 93] ignores the contribution of the stirrups, which is not reasonable in RC beams

554 strengthened with NSM CFRP strips. In fact, the “concrete tooth model”, which has been
555 used in the establishment of end debonidng strength models in steel/FRP-plated RC beams
556 [e.g. 97, 98], is worth studying as the description of the failure mechanism is clear and is
557 similar to the observation in tests. Unfortunately, however, no such attempt has been carried
558 out in establishing strength models of end interfacial debonding in RC beams strengthened
559 with NSM CFRP strips.

560 ***4.2 End cover separation***

561 *4.2.1 Failure mechanism*

562 Similar to end interfacial debonding, due to high interfacial stresses developed near the end
563 of NSM FRP strips [e.g. 25, 29], an inclined crack first occurs near the end of the NSM FRP,
564 and then another flexural-shear crack appears in the bonded region of FRP at a certain
565 distance (i.e. the crack spacing) as shown in Fig. 3b. When these cracks further develop and
566 intersect with the steel tension bars, the concrete cover between the two cracks forms a “tooth”
567 whose top is dragged by the NSM CFRP strip in the shear direction. The concrete near the
568 intersection of the tension steel bar and the inclined crack near the end of the NSM FRP
569 (Point A in Fig. 3b) is subjected to a combined effect of the following aspects: (1) the tensile
570 stress induced by the bending moment as a result of the drag force on the top of the “tooth”;
571 (2) the shear stress induced by the drag force on the top of the “tooth”; (3) the clear concrete
572 width is smaller than the beam due to the existence of the steel bars; and (4) more importantly,
573 radial stresses (as shown in Fig. 5) may be generated by the steel tension bars onto the
574 surrounding concrete when slips between the concrete and the steel occur. Therefore, the
575 plane of the tension steel bars becomes the critical plane and a major crack forms on this
576 plane when the tensile and shear stress on this place reaches a critical level. This major crack
577 travels along the steel bar from the end of the NSM CFRP strip to mid-span of the beam as
578 the applied load goes up, as shown in Fig. 3b.

*4.2.2 Strength models*

By far, two strength models of end cover separation have been respectively proposed by De Lorenzis and Nanni [99] and Al-Mahmoud et al. [9] for NSM round FRP bar-strengthened RC beams, based on the "concrete tooth" concept, in which, the concrete cover between two adjacent cracks was treated as a concrete tooth (cantilever) under the horizontal shear force exerted by the attached FRP. These two models can be also applied to NSM FRP strip-strengthened RC beams with proper modifications. More recently, Teng et al. [28] developed a strength model for end cover separation in NSM FRP strip-strengthened RC beams, also based on the "concrete tooth" concept. These three strength models are described here followed by a discussion of their performance.

***De Lorenzis and Nanni's model [99]***

De Lorenzis and Nanni [99] extended the strength model proposed by Zhang et al. [97] and Raoof and Zhang [100] to calculate the strength of RC beams strengthened with NSM round FRP bars at end cover separation. This model was the first attempt to expose end cover separation strength model for NSM FRP-strengthened RC beams and is based on the concept of the concept of "concrete tooth". By assuming a linear elastic behaviour, the normal stress $\sigma_A$ at the tension corner near the root of the concrete tooth (i.e. Point A in Fig. 3b which is near the intersection of the tension steel bar and the inclined crack near the end of the NSM FRP) could be calculated as

$$\sigma_A = \frac{M_A}{I_A}\left(\frac{l}{2}\right) \tag{25}$$

where $M_A = \pi n d_b l h'$ is the bending moment at the root of the concrete tooth, $I_A = bl^3/12$ is the sectional moment of inertia of the concrete tooth, $l$ is the minimum crack spacing $l_{\min}$ or maximum crack spacing $l_{\max} = 2l_{\min}$, $h'$ is the vertical distance from the root of the

concrete tooth to the centroid of NSM FRP, $b$ is the beam width, $d_b$ and $n$ are the diameter and number of the FRP round bars respectively, and $\tau$ is the shear bond stress between NSM FRP bar and concrete. By assuming that failure of the beam occurs when the stress $\sigma_A$ is equal to the tensile strength of concrete $f_t$, the shear bond stress $\tau_{failure}$ at failure can be expressed as

$$\tau_{failure} = \frac{f_t l}{6h'} \frac{b}{n\pi d_b} \tag{26}$$

The shear stress should be equilibrated by the FRP axial stress. At the critical section (such as the section corresponding to the loading points), the critical FRP axial stress can be calculated as

$$\sigma_{failure} = \frac{4\tau_{\text{failure}} L_p}{d_b} \tag{27}$$

where $L_p$ is the effective length of the NSM FRP bar in the shear span within which the interfacial stress is assumed to be uniformly distributed. The value of $L_p$ was determined by these authors to be the smaller one of $L_{p1}$ and $L_{p2}$, where $L_{p1}$ is the length of the NSM FRP bar in the shear span and $L_{p2}$ is the equivalent length given by

$$L_{p2} = \begin{cases} 1.86 l_{\min}^2 - 127 l_{\min} + 2436 & if \quad l_{\min} \le 50mm \\ 736 & if \quad l_{\min} > 50mm \end{cases} \tag{28}$$

where the minimum crack spacing $l_{\min}$ can be calculated as

$$l_{\min} = \frac{A_e f_t}{u_s \sum O_s + u_f \sum O_f} \tag{29}$$

where $A_e$ is the area of concrete in tension and is assumed to be product of the beam width and twice of the distance from the centroid of steel tension bars to the soffit of the beam, $f_t = 0.36\sqrt{f_{cu}}$ is the tensile strength of concrete, $f_{cu}$ is the cube compressive strength of

concrete, $\sum O_s$ is the total perimeter of the steel tension bars, $\sum O_f$ is the total perimeter of the NSM FRP round bars, $u_s = 0.28\sqrt{f_{cu}}$ is the average local bond strength between steel bars and concrete, and $u_f$ is the average bond strength between NSM FRP bars and concrete and was recommended by De Lorenzis and Nanni [99] to be the local bond strength.

In order to apply the above model to NSM CFRP RC beams, the following modifications need to be made: (1) the bending moment $M_A$ at the root of the concrete tooth should be calculated based on the geometry of NSM CFRP strips; (2) the effective length $L_p$ should be recalibrated using test results of NSM CFRP strip-strengthened RC beams ; and (3) the average bond strength between NSM FRP bars and concrete $u_f$ should be calibrated using bond strength model of NSM CFRP strip-to-concrete interface.

***Al-Mahmoud et al.'s model [9]***

The model proposed by Al-Mahmoud et al. [9] is quite similar to that proposed by De Lorenzis and Nanni [99]. In this model, the bending moment $M_A$ at the root of the concrete tooth is related to the FRP axial stress at the left cracked section (if the FRP is terminated on the right) as

$$M_A = \sigma_f A_f h' \tag{30}$$

The axial stresses in the FRP at the left cracked section can therefore be calculated as

$$\sigma_f = \frac{M_A}{A_f h'} = \frac{bl^2}{6A_f d_b} \tag{31}$$

$\sigma_f$ can also be expressed in terms of the bending moment $M_l$ of the strengthened beam at the left cracked section as

$$\sigma_f = n_f\left(\frac{d_f - y_0}{I_{cr}}\right)M_l \tag{32}$$

With the assumption that the end cover separation happens as the tensile stress in the concrete at the tension corner near the root of the concrete tooth (i.e. Point A in Fig. 3b), combining Eqs. 31 and 32 gives the following equation for calculating the bending moment of the strengthened beam at the left cracked section at debonding failure:

$$M_l = \frac{f_t I_{cr} b l^2}{6 n_f A_f d_b (d_f - y_0)} \tag{33}$$

Where $I_{cr}$ is the transformed moment of inertia in terms of concrete of the cracked section, $d_f$ is the vertical distance from the centroid of the NSM FRP to the top surface (in compression) of the beam, and $y_0$ is the vertical distance from the neutral axis of the cracked section to the top surface of the beam.

***Teng et al.'s model [28]***

The model developed by Teng et al. [28] to predict the end cover separation strength of NSM CFRP RC beams is based on the following idea: if the FRP strain on the left crack section (Point B in Fig. 6) at end cover separation failure is known, the bending moment on the corresponding section can be obtained through a section analysis, and the ultimate load can then be easily calculated by dividing the bending moment by the horizontal distance from the left crack section to the nearest support. To obtain the strain in the FRP at the left cracked section at failure (Point B in Fig. 6), the simplified FE model proposed by Zhang and Teng [32] was adopted in Teng et al. [28]. In this FE model, the part of the RC beam between the two cracks near the FRP end was isolated from the beam (Fig. 6), the bending moments acting on the two cracked sections was realized through the external loads as shown in Fig. 6, and the plane section assumption was achieved using a rigid plate attached to each cracked

668 section. Furthermore, the radial stresses exerted by the tension steel bars onto the surrounding
669 concrete were modelled using a proposed cohesive-element-pair (CEP). The plane section
670 assumption may not be exactly valid here, but it can simplify the FE model and may not
671 introduce substantial errors. An equation for the FRP strain at the left cracked section at end
672 cover separation failure was then developed through the regression of results of a numerical
673 parametric study using the abovementioned simplified FE model:

674 $$\varepsilon_{db} = 10^4 \times \beta_{cs}\beta_{AE}\beta_{bod}b_{clear}\sqrt{f_c} \tag{34}$$

675 $$\beta_{cs} = (\frac{4.5}{s_c^{0.3}} - \frac{c}{s_c})(\frac{s_c}{100} - 0.1) \tag{35}$$

676 $$\beta_{AE} = \frac{1}{(A_f E_f)^{0.9}} \tag{36}$$

677 $$\beta_{bod} = \left(\frac{b_{clear}}{D_t}\right)^{0.1} \tag{37}$$

678 where $\beta_{cs}$ is a factor accounting for the combined effect of $s_c$ (crack spacing, in *mm*) and
679 $c_d$ (distance from the centroid of steel bars to that of FRP reinforcements, in *mm*) on the
680 failure strain; $\beta_{AE}$ is a factor accounting for the effect of axial stiffness of FRP strip $A_f E_f$
681 ($A_f$ in $mm^2$ and $E_f$ in *GPa*); and $\beta_{bod}$ is a factor accounting for the effect of ratio
682 between the clear concrete width $b_{clear}$ (in *mm*) and the sum of steel tension bar diameters
683 $D_t$ (in *mm*). The cylinder compressive strength of concrete $f_c$ is given in *MPa*.

684

685 The value of the cracking spacing influences the FRP strain at the left cracked section at
686 failure and the distance from the left cracked section to the nearest support. In Teng et al.'s
687 model [28], the adopted model for minimum crack spacing $s_c^{\min}$ is also the one proposed by
688 Zhang et al. [97], as expressed in Eq. 29 with $u_f = 0.28\sqrt{f_{cu}}$. According to Zhang et al. [97],

689 the possible crack spacing value should be in the range from $s_c^{\min}$ to $2s_c^{\min}$.

690 *4.2.3 Discussions and future research needs*

691 As De Lorenzis and Nanni's model [99] was originally proposed for NSM FRP round
692 bar-strengthened RC beams, modifications should be made first to satisfy the geometric and
693 mechanical properties of NSM CFRP strips. By now, however, the effective length $L_p$ and
694 the average bond strength $u_f$ have not been calibrated by the authors or other researchers
695 using test results of NSM FRP strip-strengthened RC beams, thus it cannot yet be used for
696 predicting the cover separation strength in such FRP-strengthened RC beams. Both De
697 Lorenzis and Nanni's model [99] and Al-Mahmoud et al. 's model [9] only took account for
698 the tensile stress induced by the bending moment as a result of the drag force on the top of
699 the "tooth" but not the shear stress induced at the same time. Furthermore, the weakness of
700 the beam by the tension steel bars and the radial stresses were not considered in these two
701 strength models. Teng et al.'s model [28] was based on results of the parametric study using
702 an FE model which reflected all the above mentioned influencing factors. The performance of
703 Teng et al.'s model [28], however, is significantly influenced by the accuracy of the model of
704 crack spacing which is usually in a range from $s_c^{\min}$ to $2s_c^{\min}$. Teng et al. [28] compared the
705 predictions of their model with collected test specimens, with the crack spacing being the
706 minimum stabilized value $s_c^{\min}$, the maximum stabilized value $2s_c^{\min}$, and an intermediate
707 value $1.5s_c^{\min}$ respectively to examine the effect of crack spacing. It was found that the
708 predictions of Teng et al.'s model [28] with crack spacings of $1.5s_c^{\min}$ and $2s_c^{\min}$ led to
709 average prediction-to-test ratios of 1.10 and 1.17 respectively; their standard deviations
710 (STDs) were 0.119 and 0.172 and their coefficient of variations (CoVs) were 0.108 and 0.147
711 respectively. These statistics were much better than predictions of Teng et al.'s model [28]
712 obtained with a crack spacing of $s_c^{\min}$ whose average prediction-to-test ratio, STD and CoV

713 are 0.863, 0.155 and 0.180 respectively. Nevertheless, the model by Teng et al. [28] with any
714 of the above three values of crack spacing offered much closer predictions to the test results
715 than the model Al-Mahmoud et al. [9], the predictions of which were un-conservative, with
716 the average prediction-to-test ratio, STD and CoV being 1.90, 1.34 and 0.702 respectively.
717

718 ***4.3 Anchorage Measures for Preventing End Debonding***

719 In design, if debonding cannot be eliminated, IC debonding is preferable to end debonding
720 because the latter usually happens in a brittle manner without any noticeable early warning.
721 By now, metallic and non-metallic anchorage measures have been investigated in
722 experimental tests for preventing/mitigating end debonding in RC beams strengthened in
723 flexure with an EB FRP/steel plate. The metallic anchorage measures, in the form of steel
724 bolts, steel clamps or steel U-jackets, were initially proposed for preventing end debonding in
725 steel-plated RC beams [e.g. 101, 102]. The metallic anchorage measures, however, suffer
726 from the following two disadvantages: the difficulty of installation and the poor resistance to
727 corrosion. Therefore, non-metallic anchorage measures (such as FRP-based anchorage
728 measures) are more attractive than metallic anchorage measures in FRP-strengthened RC
729 beams for preventing end debonding. A number of studies have been conducted to explore
730 the effectiveness of FRP U-jackets in preventing/mitigating end debonding failure in
731 FRP-plated RC beams [e.g. 103-108], while the studies on the use of FRP U-jackets as
732 anchorage measures for NSM FRP bars in NSM FRP-strengthened RC beams have been
733 rather limited. These limited existing studies, however, have revealed that FRP U-jackets are
734 quite effective in both postponing the end debonding of the beam and enhancing the ductility
735 of the beam [50, 57, 109]. Before a reliable and economical design procedure for FRP
736 U-jackets can be established for confident use in practice, future research should be
737 conducted to address the following issues: (1) more experimental studies on the use of FRP

U-jackets as the end anchorage measure of NSM FRP reinforcement should be conducted to provide a larger database; (2) the effect of the angle of FRP U-jackets inclined with respect to the beam axis on the effectiveness of preventing end debonding in NSM FRP-strengthened RC beams needs to be clarified; (3) the effect of termination position of NSM FRP reinforcement (resulting in different section moment-shear force combination at the FRP end) on the performance of FRP U-jackets in preventing/mitigating end debonding failure in NSM FRP-strengthened RC beams needs to be studied; and (4) reliable FE approaches need to be established for a reliable design procedure for FRP U-jackets as end anchorage measures.

## 5 Concluding remarks

This paper has presented a critical review of the existing knowledge on NSM CFRP strips for flexural strengthening of RC beams. This review has been focused on the debonding failure modes in such FRP-strengthened RC beams, the mechanisms behind, and the corresponding strength models. The following conclusions can be made from the review:

1) The NSM FRP strengthening method is much more efficient than the EB FRP method in the flexural strengthening of RC beams, and NSM CFRP strips are superior to NSM FRP bars of other sectional forms (such as round bars and square bars) due to a larger perimeter-to-sectional-area-ratio of the former;
2) The desired debonding failure mode at the NSM FRP-to-concrete interface is the cohesion failure in a thin layer of concrete near the adhesive-to-concrete interface. This failure model can be achieved if the surfaces of concrete and CFRP are appropriately treated and a proper adhesive is used;

3) Several local bond-slip models and bond strength models have been proposed for NSM CFRP strips-to-concrete interfaces. Some of them can now provide accurate predictions for single NSM FRP strip-to-concrete joints with sufficient concrete edge distances;
4) A number of experimental studies have been conducted on NSM CFRP RC beams, which have led to the identification of four debonding failure modes. Concrete cover separation has been found to be more often than interfacial debonding in NSM CFRP RC beams.

The review presented in this paper also suggests that the existing research is still very limited and the major gaps which need to be addressed by future research include:

1) There is a lack of experimental tests with sophisticated instrumentation which is necessary to thoroughly demonstrate the validity the existing bond-slip models for NSM CFRP strip-to-concrete bonded joints;
2) There is a lack of understanding on the effect of the concrete edge distance, the groove spacing and elevated temperature on the bond behaviour of NSM FRP-to-concrete joints;
3) Most experimental studies were focused on simply supported RC beams where NSM FRP reinforcement was applied in a sagging moment region, while little research has been carried out on the use of NSM FRP reinforcement in hogging moment regions (e.g. in RC frames). In the latter case, future research is needed to clarify the possible difference in the strengthening mechanism, especially in terms of the anchorage failure of the NSM reinforcement;
4) Only a limited number of strength models were proposed for IC interfacial debonding, end interfacial debonding and end cover separation in NSM CFRP RC beams, while no strength model has been established for IC cover separation. Most existing strength

models for NSM CFRP RC beams are preliminary in nature and have been based on limited understanding of failure mechanisms. Although Teng et al.'s model [28] for end cover separation, proposed based on a comprehensive numerical parametric study, captures the failure mechanism of such failure mode, the accuracy of this model needs to be further verified with more test data;

5) Using U-shaped FRP jackets for end anchorage of NSM CFRP strips was shown to enhance the strengthening efficiency. However, its effect has not been quantitatively investigated and no design method is available now.

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## Figures

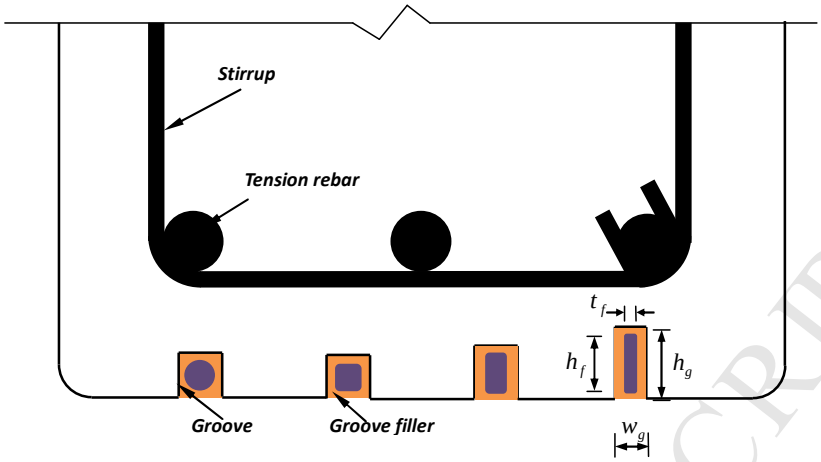
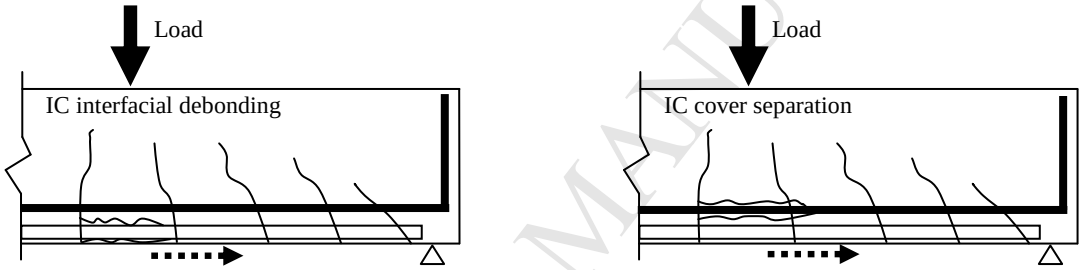
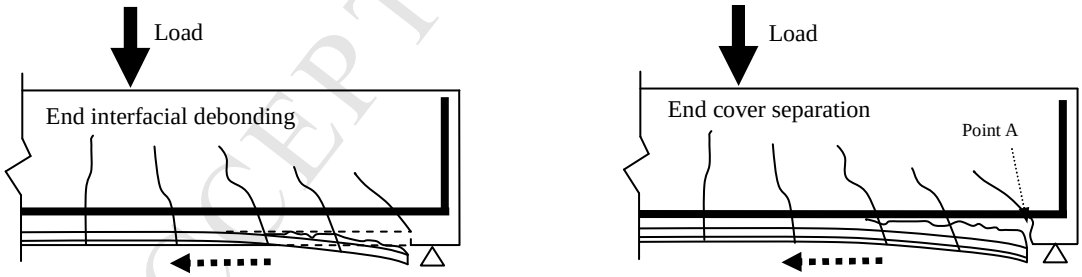



Fig. 1. Schematic of NSM FRP strengthening systems




(a) IC interfacial debonding (b) IC cover separation

Fig. 2. Schematic of the IC debonding




(a) End interfacial debonding (b) End cover separation

Fig. 3. Schematic of the end debonding

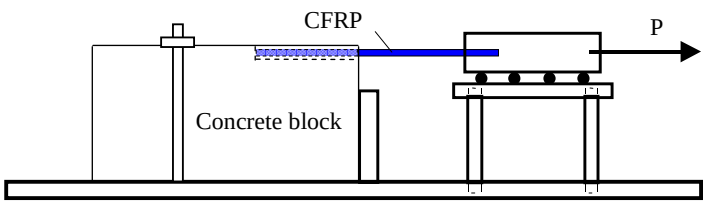


Fig. 4. Test setup of NSM FRP bonded joints

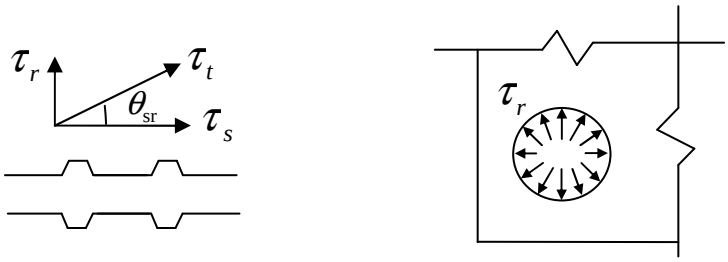



Fig. 5. Bond stresses between steel and concrete

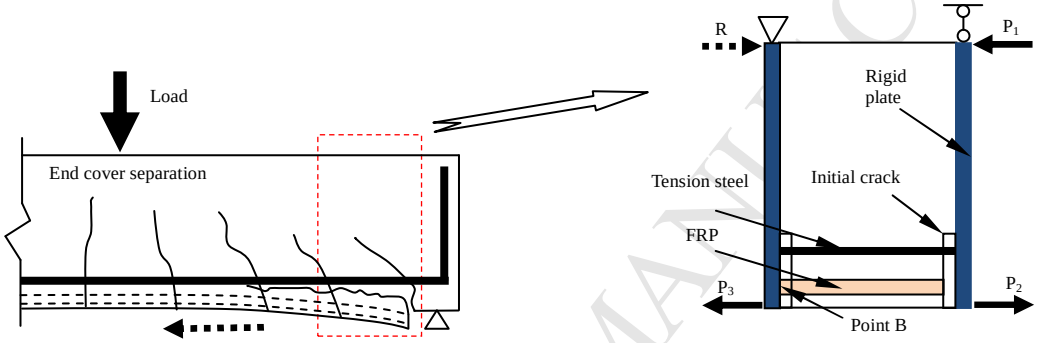



Fig. 6. Simplified FE model