Debonding mechanisms in continuous RC beams externally strengthened with FRP

L. Vasseur, S. Matthys & L. Taerwe
Department of Structural Engineering, Ghent University, Magnel Laboratory for Concrete Research

ABSTRACT: One of the biggest challenges in structural engineering nowadays is strengthening, upgrading and retrofitting of existing structures. The use of fibre reinforced polymers (FRPs) bonded to the tension face of the structural member is an attractive technique in this field of application. The strengthening of reinforced concrete (RC) structures by means of externally bonded reinforcement (EBR) is achieved by gluing FRP reinforcement to the concrete substrate. For the efficient utilization of the FRP EBR systems, an effective stress transfer is required between the FRP and the concrete. The paper will discuss the bond behaviour between the FRP and the concrete in the case of flexural strengthening of continuous beams. With respect to this type of beams, few research has been reported. At the Magnel Laboratory currently a test program on flexural strengthening of 2-span continuous beams is ongoing.

1 INTRODUCTION

Structures may need to be strengthened for different reasons, among which a change in function, implementation of additional services or to repair damage. Different strengthening techniques exist. Often applied is externally bonded reinforcement (EBR), based on fibre reinforced polymer (FRP), the so-called FRP EBR.

FRP EBR can be applied for the strengthening of existing structures, enhancing the flexural and shear capacity or to strengthen by means of confinement. This paper discusses flexural strengthening of 2 span reinforced concrete beams. CFRP (Carbon FRP) laminates are glued on the soffit of the spans and/or on the top of the mid-support (Ashour, et al. 2004, El-Refaie, et al. 2003). The efficiency of the FRP EBR strengthening technique is often limited by the capability to transfer stresses in the bond interface. Hereby bond failure between the laminate and the concrete may occur.

For unstrengthened continuous beams a moment redistribution can be observed especially after yielding of one of the critical cross-sections. As a consequence a plastic hinge will be formed. For strengthened continuous beams, after reaching the yield moment, the FRP strengthened cross-section is still able to carry additional load and the formation of a plastic hinge will be restricted.

The aim of this study is to have a better insight in the behaviour of reinforced concrete structures strengthened in flexure in a multi-span situation.

2 CALCULATION MODEL FOR CONTINUOUS BEAMS

2.1 *Non-linear moment-curvature diagram*

Performing an analysis of a construction according to the linear elasticity theory, a linear relationship between the moment and the curvature is obtained, namely

$$\frac{1}{r} = \frac{M}{EI} \tag{1}$$

with $1/r$ the curvature, M the bending moment and $K = EI$ the bending stiffness.

This stiffness is assumed to be constant and therefore independent of the value of the bending moment. However, for the cross-section of a concrete beam the moment-curvature diagram is non-linear. This non-linear character is caused by the variable bending stiffness, as shown in Figure 1. Two cases are drawn in this graph, a cross-section with externally bonded FRP (strengthened) and a cross-section without FRP (unstrengthened). An important difference between these cases is the bending stiffness (slope of lines K_0, K_1 and K_2). With FRP higher values for K are obtained than without FRP. This different behaviour will influence the moment redistribution of a continuous beam.

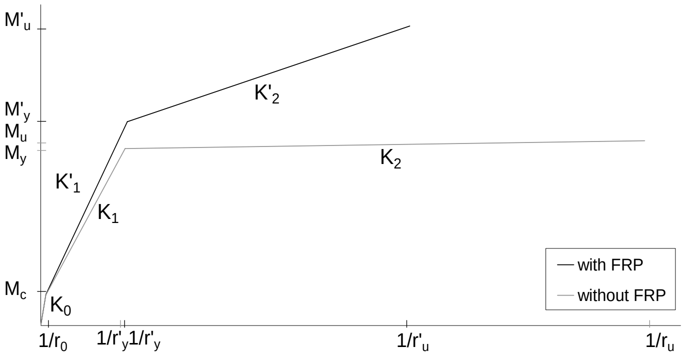

Figure 1: Moment-curvature diagram.

If Figure 1 is applied to a continuous beam, we start with the uncracked phase along the whole length of the beam, corresponding to the use of K_0 as bending stiffness. By increasing the load, the beam is characterized by cracked and uncracked zones, each with the related value of bending stiffness. This change of stiffness causes a first redistribution of moments. For the yield load F_y, one or more cross-sections reach the yield moment (M_y). In yield zones without FRP EBR, the bending stiffness K_2 is so small that plastic deformations appear in the critical cross-section and in a restricted area near to it. This is the so-called formation of a plastic hinge. The increasing load is mainly carried by the non plastic zones and during which the bending moment in the plastic hinge stays almost constant ($M_u \approx M_y$) or is slowly increasing. In zones with FRP EBR, the value of the bending stiffness is higher (K'_2). Also plastic deformations appear, but in a more limited way. The yielding zone still carries a significant part of the increasing load and the formation of the plastic hinge is restricted.

2.2 *General behaviour of continuous beams*

Consider a continuous beam with two identical spans and symmetrical loaded by two point loads (Figure 2). Focused on one span, two zones can be defined, one zone with positive moments (above the mid-support) and another with negative moments (in the spans). It is assumed that in each zone the bending stiffness is constant. So the mid-support zone and the span zone have stiffness $K_{support}$ and K_{span}, respectively.

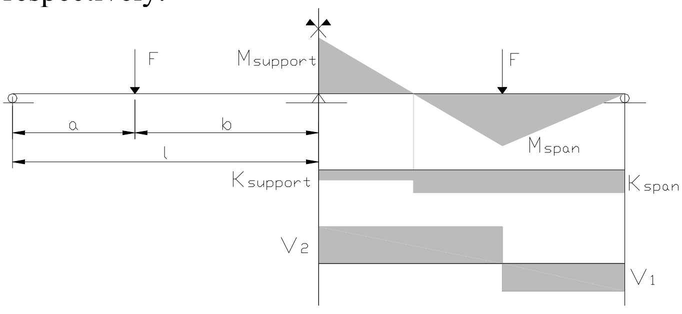

Figure 2: Continuous beam with variable bending stiffness (simplified to 2 stiffness zones).

Further, we define:

$$\lambda = \frac{a}{b} \quad m = \frac{M_{support}}{M_{span}} \quad k = \frac{K_{support}}{K_{span}} \tag{2}$$

By considering that the angle of rotation above the mid-support equals zero, the following equation can be obtained (Taerwe, et al. 1989):

$$\begin{aligned}&(2+3\lambda)m^3 + \left(3+3\lambda-2\lambda^2\right)m^2 \\ &- k\lambda(3+4\lambda)m - (1+\lambda)(1+2\lambda)k = 0\end{aligned} \tag{3}$$

With Eq. (3) the internal forces in the continuous beam can be calculated. In what follows, calculations are done for $a = 2$ m and $b = 3$ m. Hence with $\lambda = 2/3$ Eq. (3) changes into.

$$36m^3 + (45-8k)m^2 - 34km - 35k = 0 \tag{4}$$

This equation is shown in Figure 3. For loads below the cracking moment, the mid-support zone and field span zone are uncracked and the two zones nearly have the same bending stiffness. This condition correspond with $k = 1$. From Eq. (4) we obtain then $m = 0.9722 = m_{el}$. This value of m corresponds to the moment distribution following the classic theory. Hereby, the relationship between acting load and internal moment is linear, as in the case of isostatic beams. By further increasing the load, the changing bending stiffnesses in different cross-sections modifies k thus the relation between the internal moments m. As a result the moment distribution deviates from the classic theory to the so-called non-linear moment-redistribution.

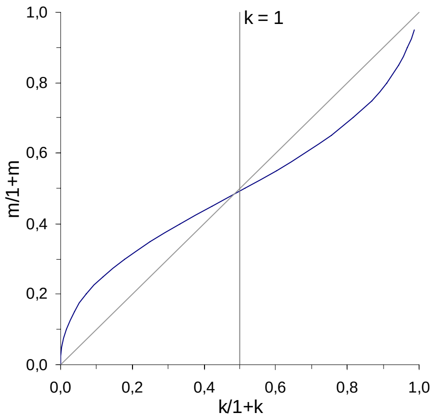

Figure 3: The relation of the moments m in function of the relation of the bending stiffnesses k.

3 DEBONDING MECHANISMS ON CONTINUOUS BEAMS

3.1 *Overview of different debonding mechanisms*

Bond failure in case of FRP EBR implies the loss of composite action between the concrete and the FRP reinforcement. This type of failure is often very sudden and brittle. According to Matthys (Matthys 2000) different bond failure aspects can be distinguished.

A first type of debonding appears when the externally bonded FRP bridges cracks. This results in elevated shear stresses at the interface and may cause some degree of debonding. In regions with significant shear forces, shear or flexural cracks have a vertical (v) and a horizontal (w) displacement. The vertical displacement of the concrete also causes tensile stress perpendicular to the FRP EBR, which enhances debonding of the laminate (Figure 4).

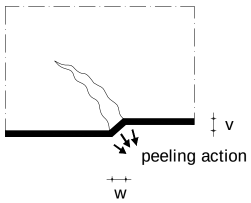

Figure 4: Peeling-off caused at shear cracks.

As second debonding mechanism there is force transfer. Herewith the variation of tensile force in the FRP (ΔN_{fd}) initiates bond shear stresses at the interface due to the composite action between the FRP EBR and the concrete beam. The bond shear stress considered between two sections at a distance Δx equals (Figure 5):

$$\tau_b = \frac{\Delta N_{fd}}{w_f \Delta x} \tag{5}$$

with w_f the width of the FRP laminate.

These shear stresses have to be smaller than the bond strength between the concrete and the FRP reinforcement.

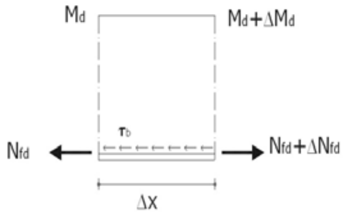

Figure 5: Peeling-off caused by force transfer.

A next debonding mechanism is anchorage failure, and relates to curtailment and anchorage length. Theoretically the FRP reinforcement can be curtailed when the axial tensile force can be carried by the internal steel only. The remaining force in the FRP at this point needs to be anchored. The anchorage capacity of the interface is however limited, and hence the FRP may be extended to zones corresponding with low FRP tensile stresses.

At last there is debonding by end shear failure, also known as concrete rip-off. If a shear crack appears at the plate-end, this crack may propagate as a debonding failure at the level of the internal steel reinforcement. In this case the laminate as well as a thick layer of concrete will be ripped off.

3.2 *Avoiding some debonding mechanisms in continuous beams*

To predict the debonding load, the available calculation models (fib 2001) are based on formulas which basically relate to experiments on isostatic reinforced beams and pure shear bond tests.

The difference between isostatic beams and continuous beams, which may influence the debonding mechanisms in continuous reinforced concrete beams, is the moment line with opposite signs. Whereas the moment in the span is positive, the moment at the mid-support is negative. As a result, the compression zones in the spans are situated at the top of the beam, at the mid-support the compression zone is situated at the soffit of the beam (shaded zones in Figure 6). This allows in contrast to reinforced isostatic beams, to anchor the CFRP laminates in the compression zones (except for the outer supports) (Figure 6). By extending a laminate into these compression zones, two out of the four different debonding mechanisms will be avoided: debonding by a limited anchorage length and debonding by end shear failure (concrete rip-off).

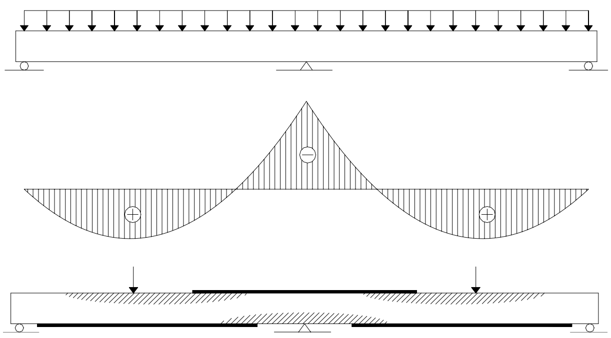
Figure 6: Moments with opposite signs in continuous beams and anchoring laminates into compression zones.

Debonding by limited anchorage length is prevented by extending the laminate into the compression zone because in this situation the tensile stress in the laminate is gradually reduced to zero, and anchored in a zone with small compressive stresses (no significant risk for buckling).

Debonding by end shear failure occurs when a shear crack appears at the end of the laminate. By extending the laminate into the compression zone, the plate-end reaches a zone where no shear cracks can be formed and neither concrete rip-off will appear.

To anchor the laminate into the compression zone, it is extended beyond the point of contraflexure, which is the location where the internal moment equals zero. For calculating the exact location of this point, it has to be noticed that the point of contraflexure moves with increasing load, due to the non-linear moment redistribution.

3.3 *Specific debonding aspects related to continuous beams*

In the case of strengthened continuous beams, some particular aspects can be noted, which may also influence the moment of debonding. This is illustrated in the following by means of an analytical study for the beam and strengthening configuration in Figure 7. The applied internal reinforcement is kept constant during the analytical study and is based on the linear theory. In this case almost the same amount of internal reinforcement is used in the spans and in the mid-support (reinforcement ratio's $\rho_{s,span}$ = 0.68 % and $\rho_{s,support}$ = 0.61 %). The properties assumed in the analysis are given in Table 1, whereas the amount of FRP strengthening in the spans and mid-support zone is varied (FRP widths of 60 mm, 100 mm, 150 mm and 200 mm are used in this study).

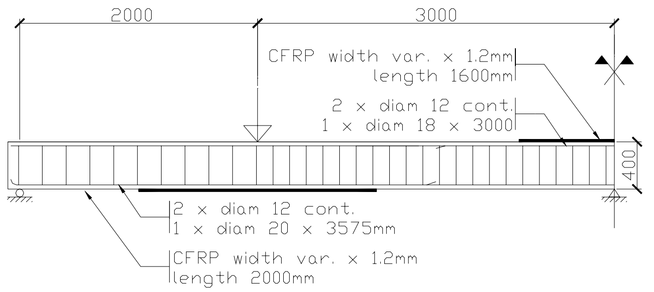

Figure 7: Internal and external reinforcement configuration.

Table 1: Properties of concrete and reinforcement materials.

	Concrete	Steel	CFRP
Compres. strength [N/mm^2]	36.0	-	-
Yielding strength [N/mm^2]	-	570	-
Yielding strain [%]	-	0.28	-
Tensile strength [N/mm^2]	3.3	670	2768
Failure strain [%]	0.35*	12.40	1.46
E-modulus [N/mm^2]	32000	210000	189900

* in compression

The length of the FRP is chosen in such a way that all four debonding mechanisms can occur. Herewith the laminates are not anchored into the compression zones as described in paragraph 3.2. The length of the laminate at the soffit of the span equals 2000 mm and is applied in such a way that the center of the laminate is just beneath the point load. The laminate at the top of the beam above the mid-support equals 1600 mm (see Figure 7).

The influence of the amount of FRP strengthening on the acting shear forces is illustrated in Figure 8, for a point load F of 100 kN. Herewith, V_1 (solid lines) is the shear force acting between the outer support and the point load. $V_2 = F - V_1$ (dashed lines) is the shear force acting between the point load and the mid-support (Figure 2). As can be noted, the value of V_1 (and hence V_2) is influenced by the FRP reinforcement ratio's of both the span ($\rho_{f,span}$) and the mid-support ($\rho_{f,support}$). By increasing the width of the laminate above the mid-support (increasing $w_{f,support}$ or $\rho_{f,support}$), for $w_{f,span}$ = cte, V_1 decreases and V_2 increases. This is due to the moment redistribution which is dependent on both the exter-

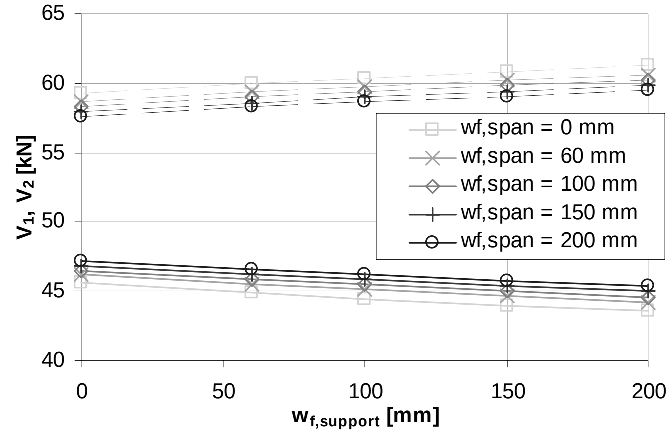

Figure 8: Shear force V_1 in function of width of laminates.

nal and internal reinforcement ratio used over the length of the beam. Owing to this, also the distribution of the reactive forces in the supports is dependent on the reinforcement ratio's. As a result, the part of the applied load which is carried by the mid-support increases with an increasing amount of FRP at the mid-support. If $w_{f,span}$ (or $\rho_{f,span}$) is increased as well, the decrease of V_1 will be less pronounced and V_1 may even increase (compared to the unstrengthened beam).

As debonding phenomena often relate to the acting shear force, this means that possible debonding of a FRP laminate not only depends on the FRP configuration at that location, but also on the amount of FRP in the zone with opposite moment sign.

Another significant aspect with respect to the values of $\rho_{f,span}$ and $\rho_{f,support}$ relative to each other, is their influence on the point of contraflexure. Indeed, by increasing $\rho_{f,support}$ (increasing the width of the laminate above the mid-support), $M_{support}$ will increase and M_{span} will decrease. Herewith, the point of contraflexure moves towards the mid-support. On the opposite, by increasing $\rho_{f,span}$ at the soffit of the span, $M_{support}$ will decrease and M_{span} will increase. As a result, the point of contraflexure moves away from the mid-support. Because of this, a change of the distance between the laminate end and the place where the internal moment equals zero (L) can be observed. Indirectly also the anchorage length (l_t) will change. Due to this the debonding mechanisms anchorage failure and concrete rip-off once more will be dependent on the amount of external reinforcement along the beam.

In the following paragraphs, the load at which debonding occurs, for the different debonding mechanisms will be investigated in function of both $\rho_{f,span}$ and $\rho_{f,support}$. Hereby, a differentiation is made between debonding of the top laminate (case A), debonding of the laminate at the soffit of the span between the point load and the mid-support (case B) and debonding of the laminate at the soffit of the span between the point load and the outer support (case C) (Figure 9).

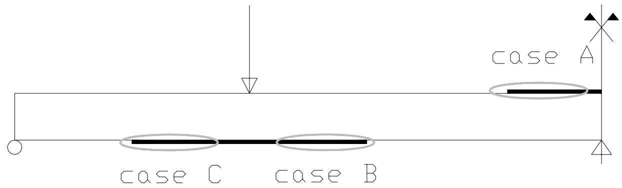

Figure 9: Differentiation between places of debonding.

The calculations are performed according to section 2 and fib-bulletin 14 (fib 2001). Results of the debonding load calculations are given as far as they do not exceed the ultimate load of the strengthened beam assuming full composite action. Herewith it is assumed that debonding of the FRP does not occur, and that the construction only can fail by concrete crushing or by exceeding the tensile strength of steel or FRP reinforcement.

In Table 2 a summary is given of the effect of the external reinforcement on the debonding load. In paragraph 3.3.1 till 3.3.4 a more detailed explanation is given about these findings.

Table 2: Effect of external reinforcement on debonding load.

		Amount of top laminate above the mid-support ↑	Amount of laminate at the soffit of the span ↑
Crack bridging	A	debonding load ↑	debonding load ↑
	B	debonding load ↓	debonding load ↑
	C	debonding load ↑	debonding load ↑
Force transfer	A	debonding load ↑	debonding load ↑
	B	debonding load ↓	debonding load ↑
	C	debonding load ↑	debonding load ↑
Anchorage failure	A	debonding load ↓	debonding load ↑
	B	debonding load ↑	debonding load ↓
	C	debonding load ↑	debonding load ↓
Concrete rip-off	A	debonding load ↓	debonding load ↑
	B	debonding load ↑	debonding load ↓
	C	debonding load ↑	debonding load ↓

3.3.1 *Crack bridging*

Debonding by crack bridging in case of risk for vertical crack displacement can be modeled according to (fib 2001), based on the following simplified equation:

$$V_{Rpd} = \tau_{Rp} bd \tag{6}$$

$$\tau_{Rp} = 0{,}38 + 1{,}51\rho_{eq} \tag{7}$$

with b the width of the beam, d the effective depth of the beam, τ_{Rp} the nominal shear stress corresponding to V_{Rpd}, $\rho_{eq} = (A_s + A_f.E_f/E_s)/bd$ the equivalent reinforcement ratio and A and E the cross-section and modulus of elasticity of the reinforcement (s: steel and f: FRP).

This means that the moment of debonding will be both dependent on the acting shear force (influenced by both $\rho_{f,span}$ and $\rho_{f,support}$) as well as the amount of FRP (A_f) (influenced by $\rho_{f,support}$ in case A and $\rho_{f,span}$ in cases B and C)

The debonding load of the top laminate (case A) is given in Figure 10. By increasing the width of the top laminate, the resistance V_{Rp} increases (increase of A_f in equation 7) and to a lesser extent the acting shear load V_2 also increases (Figure 8). If at the same time the amount of FRP in the spans increases, this further enhances the debonding load due to a reduction of the acting shear force V_2.

The change in debonding load in case B is illustrated in Figure 11. Increasing the widths of the laminate in the span (for $w_{f,support}$ = cte.), increases the debonding resistance (increase of A_f) at one hand and decreases the acting shear load (V_2) at the other hand. This causes a higher debonding load. Increasing also the width of the top laminate will result in a somewhat higher acting shear force V_2 (Figure 8) and has a negative influence on the debonding load.

For the beam considered in this study (Figure 7, Table 1), debonding in case C appeared not governing compared to the ultimate load of the strengthened beam assuming full composite action. Nevertheless, similar to the above argumentation an increased debonding resistance can be expected for a higher width of the laminate in the span. Also increasing the amount of FRP at the mid-support, this will further enhance the debonding load due to a reduction of the acting shear load V_1 (Figure 8).

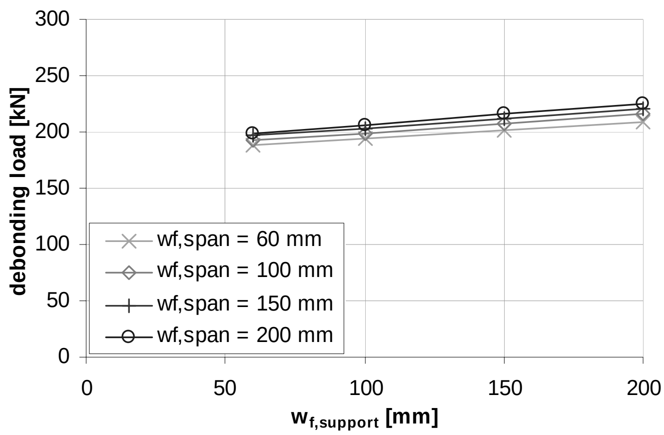

Figure 10: Influence of external reinforcement on the debonding mechanism: Debonding at cracks (case A).

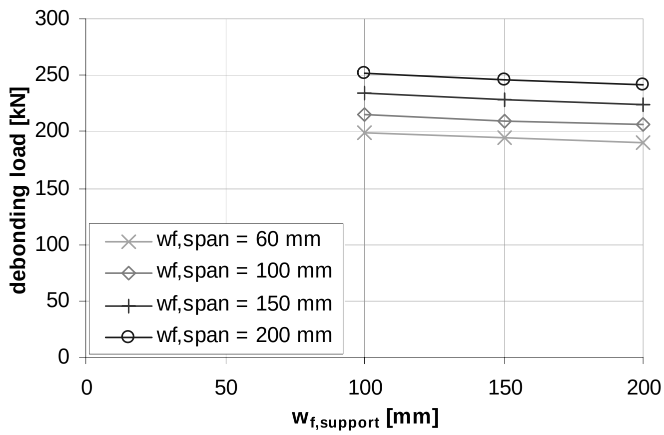

Figure 11: Influence of external reinforcement on the debonding mechanism: Debonding at cracks (case B).

3.3.2 *Debonding due to force transfer*

The influence of $\rho_{f,span}$ and $\rho_{f,support}$ on the debonding load of this mechanism is similar as described in the previous section (3.3.1)

Firstly, there is the change of shear force ratio caused by the moment redistribution. By increasing the width of the top laminate above the mid-support, V_2 increase and V_1 decrease (Figure 8). Herewith a lower debonding load is expected in cases A and B and a higher debonding load is expected in case C. By increasing the width of the laminate at the soffit of the span, the opposite effect occurs.

Secondly, the width of the laminate (w_f) is also an important factor in the calculation of the resisting shear force, V_{Rbd} (equation 8) (fib 2001).

$$V_{Rbd} = f_{cbd}\, 0{,}95 d w_f \tag{8}$$

with $f_{cbd} = 1{,}8 f_{ctk}/\gamma_c$, f_{ctk} the characteristic tensile strength of the concrete and γ_c the safety factor (γ_c = 1,50).

Increasing the width of the laminate has a positive influence on its debonding load. This effect is more pronounced than the acting shear load effect. The combined effect is illustrated in Figures 12 till 14, for cases A, B and C respectively.

Increasing the width of the top laminate results in a large increase of the debonding load for case A (Figure 12). Increasing the width of the laminate in the span also has a favourable influence, yet to a lesser extent.

In Figure 13 the debonding load of case B is illustrated. Here it can be noticed that the increase of the width of the laminate at the soffit of the span has an important positive influence on the debonding load and will delay it, while the increase of the width of the top laminate has a negative influence and results in a lower debonding load.

In Figure 14 the debonding load of case C is illustrated. Here it can be noticed that the increase of the width of both the laminates at the span and at the mid-support increase the debonding load, whereas the laminate at the soffit has the most pronounced effect.

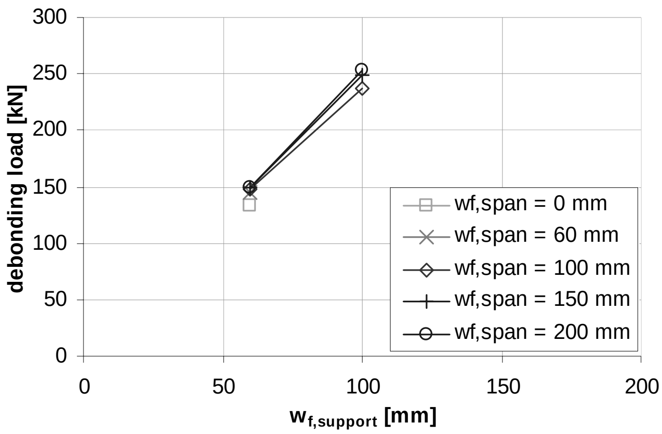

Figure 12: Influence of external reinforcement on the debonding mechanism: Debonding by force transfer (case A).

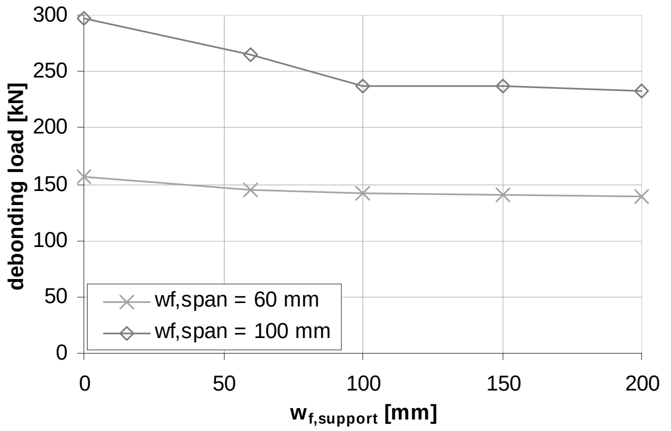

Figure 13: Influence of external reinforcement on the debonding mechanism: Debonding by force transfer (case B).

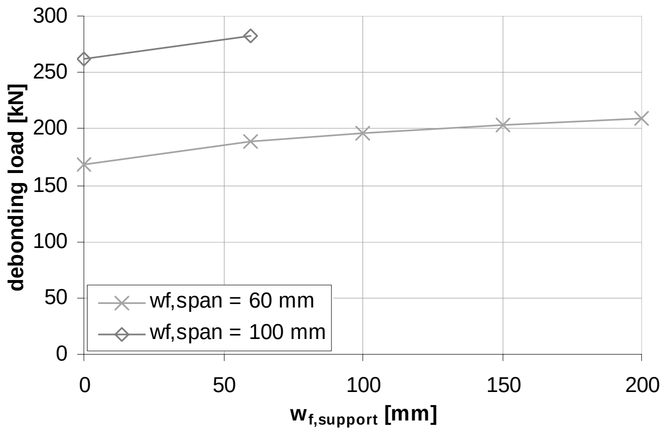

Figure 14: Influence of external reinforcement on the debonding mechanism: Debonding by force transfer (case C).

3.3.3 *Anchorage failure*

Considering debonding by anchorage failure, the variability of the shear forces V_1 and V_2 according to the external reinforcement ratio's is of no importance. What, however, is important, is the redistribution of the internal moments $M_{support}$ and M_{span} and its impact on the location where the internal moment equals M_{curt}. Herewith, M_{curt} is the internal moment for which the axial tensile force can be carried by the internal steel only and consequently the external reinforcement theoretically can be curtailed. M_{curt} can also be seen as the position along the length of the beam where the anchorage length l_t starts. Because of the change in position of M_{curt}, the available anchorage length is dependent on the moment redistribution and consequently on the reinforcement ratio's along the continuous beam.

By increasing the amount of FRP reinforcement above the mid-support, $M_{support}$ increases and M_{span} decreases. This results in a movement of the position where the internal moment equals M_{curt}: (1) towards the laminate end (shorter anchorage length) for the laminate at the top of the beam (case A), and (2) away from the laminate end (larger anchorage length) for the laminates at the soffit of the beam (cases B and C).

By increasing the amount of FRP reinforcement at the soffit of the beam an opposite effect is obtained.

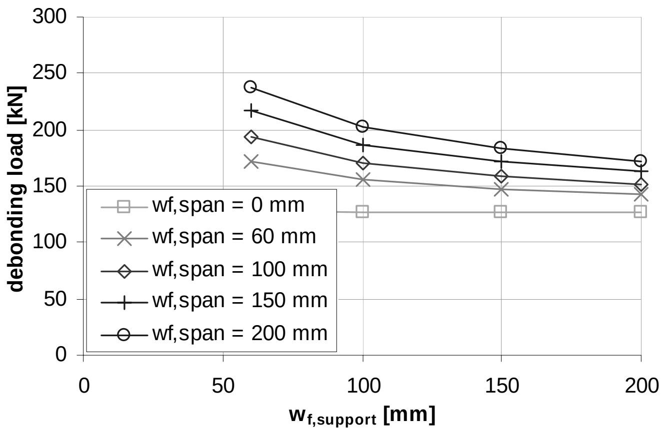

Figure 15: Influence of external reinforcement on the debonding mechanism: Anchorage failure (case A).

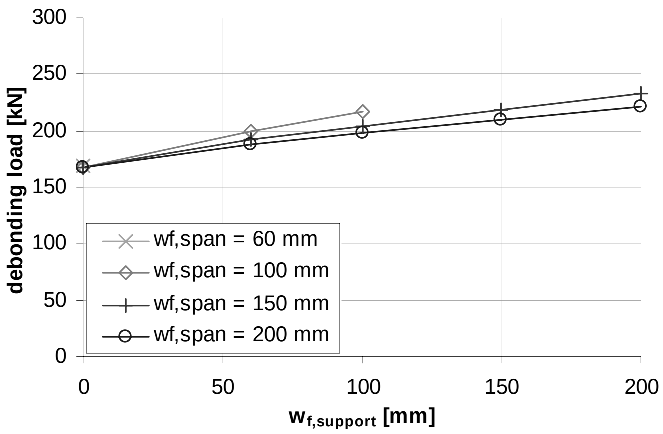

Figure 16: Influence of external reinforcement on the debonding mechanism: Anchorage failure (case B).

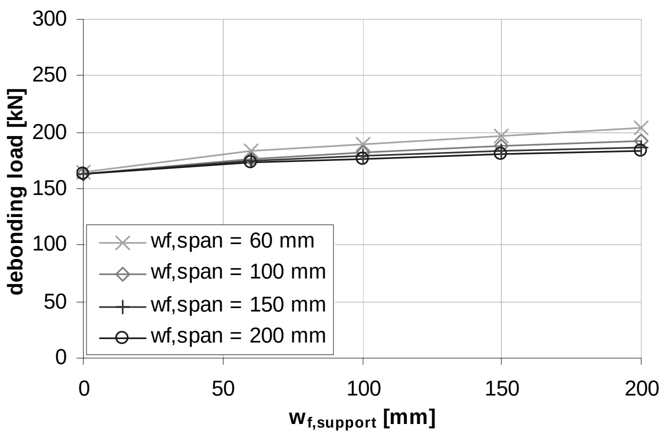

Figure 17: Influence of external reinforcement on the debonding mechanism: Anchorage failure (case C).

It can be concluded (keeping the total FRP length constant) that a reduced anchorage length is obtained in case A when the amount of FRP above the mid-support is increased and in cases B and C when the amount of FRP at the soffit of the span is increased. Due to this reduction of anchorage length, a lower debonding resistance may be obtained. This influence of the laminate widths (or ρ_f) on debonding at the anchorage zone is illustrated in Figures 15 till 17 for the considered beam and strengthening configuration.

3.3.4 *Concrete rip-off*

To check concrete rip-off, a resistant shear stress, τ_{Rd} (= V_{Rd}/bd), has to be calculated (equations 9 and 10) (fib 2001). Herewith the changes of the shear forces according to the reinforcement ratios are playing an important role. The influence will be similar as earlier discussed in sections 3.3.1 and 3.3.2.

$$\tau_{Rd} = 0{,}15 \sqrt[3]{3\frac{d}{a_L}\left(1+\sqrt{\frac{200}{d}}\right)} \sqrt[3]{100\rho_s f_{ck}} \qquad (9)$$

$$a_L = \sqrt[4]{\frac{\left(1-\sqrt{\rho_s}\right)^2}{\rho_s} dL^3} \qquad (10)$$

where $\rho_s = A_s/bd$, f_{ctk} is the characteristic compression strength of the concrete and L is the distance between the laminate end and the point where the internal moment equals zero (figure 18).

By increasing the amount of FRP above the mid-support, a larger part of the applied load is carried by the mid-support. As a result the internal shear force V_1 decreases and V_2 increases. This decrease of V_1 has a positive influence on the debonding load for case C, while the increase of V_2 has a negative influence for cases A and B. By increasing the amount of FRP at the soffit of the beam, the opposite effect is obtained.

In addition to the redistribution of the acting shear force, the debonding load is also governed by the influence of L on the debonding resistance. This distance relates in cases A and B to the point of contraflexure. In case C this distance relates to the outer support (Figure 18). A higher debonding resistance is obtained for decreasing values of L. For the cases A and B, L depends on the location of the point of contraflexure. As a result, the debonding due to concrete rip-off will also depend on the moment redistribution.

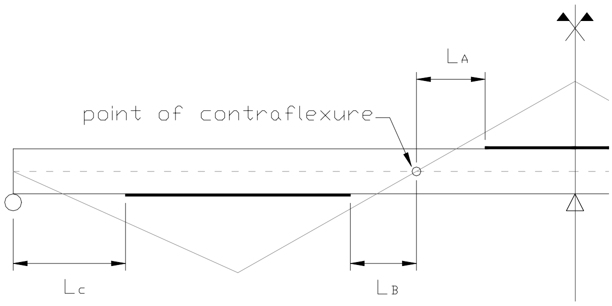

Figure 18: The distance L between the laminate end and the point where the internal moment equals zero.

Consequently, increasing the amount of FRP above the mid-support moves the point of contraflexure to the left in Figure 18. This causes an increased value L_A, and an equally decreased value L_B (for $L_A + L_B$ = cte.). Hence, a decrease of the resisting debonding force of the FRP laminate at the top of the beam (case A) and an increased value of the debonding resistance of the FRP laminate at the soffit of the beam (case B) are obtained. An opposite effect is obtained when increasing the amount of FRP at the soffit of the beam.

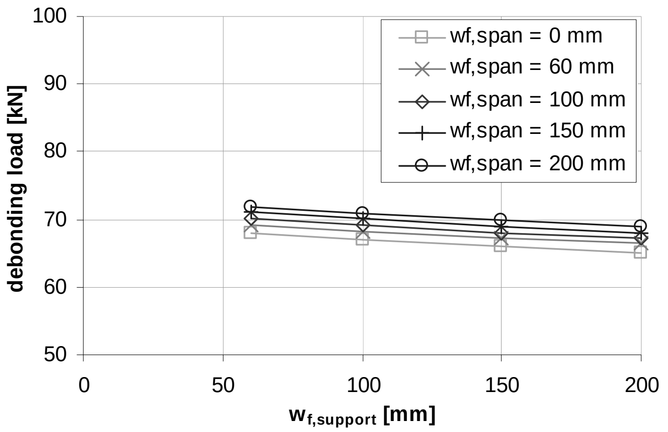

Figure 19: Influence of external reinforcement on the debonding mechanism: Concrete rip-off (case A).

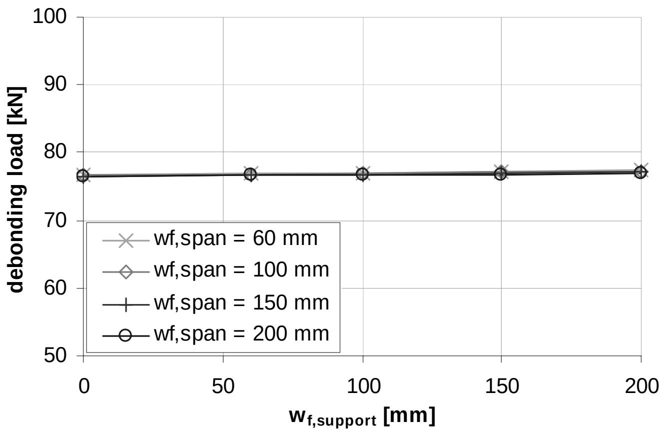

Figure 20: Influence of external reinforcement on the debonding mechanism: Concrete rip-off (case B).

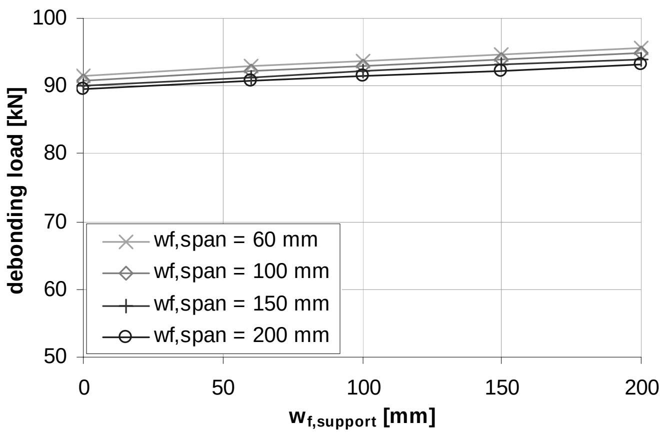

Figure 21: Influence of external reinforcement on the debonding mechanism: Concrete rip-off (case C).

The combined effect of shear and moment redistribution on the concrete rip-off debonding load is illustrated in Figures 19 till 21. For the considered case, and whereby the total length of the FRP is kept constant, the influence of the shear and moment redistribution appears less pronounced than for the other debonding mechanisms. Especially in case B (Figure 20), for which the shear and moment redistribution effect counter act each other, the combined influence is insignificant.

4 CONCLUSION

In continuous beams compression zones are available at which FRP laminates can be anchored. Here two debonding mechanisms (concrete rip-off and anchorage failure) can be avoided.

By means of an analytical study, it has been demonstrated that the debonding loads are also governed by the shear force and moment redistribution. This redistribution is occurring in FRP strengthened continuous beams and depends on the amount of FRP in the spans and at the mid-support, relative to each other. Because of the specific influence on the debonding load, redistribution of internal forces should be considered when verifying the debonding load of strengthened continuous beams. Depending on the situation (amount of FRP, type and location of the debonding phenomenon) both an increased or decreased value of the debonding load may be obtained.

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REFERENCES

Ashour, A. F., El-Refaie, S. A. and Garrity, S. W. 2004 Flexural strengthening of RC continuous beams using CFRP laminates. Cement and Concrete Composites **26**(7): p. 765-775.

El-Refaie, S. A., Ashour, A. F. and Garrity, S. W. 2003 Sagging and Hogging Strengthening of Continuous Reinforced Concrete Beams Using Carbon Fiber-Reinforced polymer Sheets. ACI Structural Journal. **Vol. 100**: p.446-453.

fib 2001 fib bulletin 14, Externally bonded FRP reinforcement for RC structures. International federation for structural concrete, Lausanne.

Matthys, S. 2000 Structural behaviour and design of concrete members strengthened with externally bonded FRP reinforcement. Ghent University, Department of structural engineering.

Taerwe, L. and Espion, B. 1989 Serviceability and the Nonlinear Design of Concrete Structures. IABSE PERIODICA 2/1989.