Fracture process in CFRP Plate Debonding Fracture

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Abstract

In RC beams retrofitted by carbon fibre reinforced polymer (CFRP) plates, small flaws in the concrete close to the CFRP-concrete interface can trigger plate debonding and then structure collapse. Experimental observations of fracture propagation, obtained using digital image correlation techniques particularly designed for debonding fracture inspection, allow the strain fields for the debonding fracture to be studied at a detailed scale. The results show that there is no large fracture process zone associated with debonding and that the debonding fracture resistance does not vary with the debonding crack length. Most significantly, the results show that although aggregate interlocking, occurs, which is essentially a Mode II process, the much lower fracture energy associated with Mode I fracture dominates the behaviour and the Mode I energy should be used in debonding calculations.

Keywords

Debonding fracture propagation; Wedge-split test; Image correlation technique; Strain field; Fracture process zone.

Note for Reviewers

This paper is one of three that have been submitted to different journals and which cross-refer. For the benefit of reviewers only, copies of all three papers, as submitted, can be downloaded.
**Introduction**

Reinforced concrete (RC) beams retrofitted with carbon fibre reinforced polymer (CFRP) plates commonly fail in a sudden manner by premature debonding before the target flexural capacity. A thin layer of concrete is usually attached to the debonded plate, indicating that debonding is a fracture event initiated from the inevitable flaws in the concrete cover layer between the external FRP plate to the internal steel. In FRP retrofitted RC beams, these small flaws can trigger structure collapse, so need to be investigated in detail. However, primarily because debonding is a fast process, there is a lack of direct fracture observation and the correct mode of debonding fracture has been a matter of considerable debate.

To investigate this phenomenon, the authors designed a Wedge-Split Peel-Off test to determine the fracture energy of the fracture process (Guan & Burgoyne 2014a) using Digital Image Correlation (DIC) techniques to determine the extent of fracture and the distribution of strains within the concrete (Guan & Burgoyne 2014b). That work showed that the Fracture Energy associated with debonding was in the range 0.03 - 0.15 N/mm, a value consistent with the Mode I fracture energy of concrete, and a value that, when used in a Global Energy Balance study of the debonding process, correctly identifies the load at which debonding should occur (Guan & Burgoyne 2014c). Besides determining the fracture energy, the test geometry allows other phenomena to be investigated, including the issue of Mode Mixity and
the question of aggregate interlock; The DIC technique allows the detailed strain patterns to be observed in greater detail than has hitherto been possible, and therefore the size of the fracture process zone ahead of the crack tip can be determined, which is often a matter for discussion.

The fracture energy associated with Mode II fracture, usually measured in a bond-slip test, is in the range of 0.5 – 2.0 N/mm and an order of magnitude greater than that measured in Mode I tests. When used in a GEBA analysis of debonding it does not predict the correct failure load. Debonding typically takes place in the cover zone between the CFRP at the surface and the steel bars, which may be between 30 and 50 mm thick. So the size of the FPZ is relevant, since if it is similar in extent to the cover, or larger, the presence of the steel may constrain the fracture and needs to be study. There is also the issue of length effects. If the fracture energy associated with crack extension changes significantly with the length of the crack (often referred to as an R-curve phenomenon: Rossi et al. (1991), Wecharatana & Shah (1982)), then the analyst needs to know exactly how the initial crack forms and how it propagates, information which cannot be known because it would require knowledge of the exact form of the initial crack and details of the aggregate distribution. If however it turns out that the fracture energy is not length-dependent, then it only has to be supposed that a small initiating crack exists, not its details. The exact shape of the adhesive layer also ceases to be relevant. This paper aims to address these issues that are important when considering plate debonding.

The actual stress in the vicinity of plate end (PE) debonding is complex. The FRP plate end is under zero stress, while the concrete in the beam next to it undergoes bending. It is bridged by the adhesive layer where the variations of stress and strain have to be gradual. Fig. 1 shows a magnified picture of this mismatch at the plate end. The distance between the plate end and the fully-bonded location is the transfer zone where the FRP plate is partially-bonded and the FRP-RC bond develops.
Figure 1  Modelling details in debonding vicinity: $M$, $N$ and $V$ refer to the bending moment, normal force and shear force respectively; $u$ and $v$ refer to the deformations in the horizontal and vertical directions; $A_{cut}$ and $\bar{y}$ are the area of the cut section and the distance from the cut section to the beam section neutral axis; $K_u$ and $K_v$ are the compliances in the horizontal and vertical directions, and subscripts $t$ and $b$ refer to top and bottom.

The FRP-RC bond needs to bend the unbonded part to the RC beam, by stretching the FRP plate in the axial direction and pulling it to deflect upwards. This bond effect needs to be modelled by manipulating the loading responses of the adhesive layer to incorporate the mismatch. Beam-foundation models with different degrees of complication for layered
materials are commonly used for this modelling, and a review of these models can be found in Carpinteri et al. (2009), Wang (2007) and Rabinovitch (2004).

Although unrealistic, the equivalent beam model forms the basis for more sophisticated models, shown in Fig. 1(c), assuming the whole FRP-RC section is planar, and the adhesive layer infinitely stiff. The shear lag model is a first-step development (Carpinteri et al. (2009), Triantafillou & Deskovic (1991), Wu et al. (2002)), which uses a finite stiffness in the longitudinal direction and assumes that the shear stress remains constant across the adhesive thickness. It neglects the normal stresses and usually uses a linear-elastic constitutive relationship for shear response and a nonlinear shear stress distribution along the transfer zone. Bond-slip tests are used to calibrate the model, and various shear-slip relationships have been proposed for this model use (Yuan et al. (2004), Zhou et al. (2010), Carrara et al. (2011)). However, this model assumes, without proof, that debonding is a Mode II fracture. As a result, the fracture energy \( (G_f) \) obtained from the bond-slip test (usually in the range of 0.5 – 2 N/mm) is found to be unrealistic when used in fracture debonding analysis.

Various fracture debonding analyses have been developed to estimate the structural load at which debonding occurs, and they have been demonstrated to be accurate if a peel-off concrete fracture energy \( (G_f) \) (around 0.05 – 0.15 N/mm) is used as the debonding criterion (Guan & Burgoyne (2014c), Achintha & Burgoyne (2008), Gunes et al. (2009), Carpinteri et al. (2009), and Rabinovitch (2004)). This value is the same as that from conventional Mode I three point bend (TPB) tests, but over an order of magnitude smaller than the \( G_f \) values from bond slip tests where debonding is forced to occur by slipping (Mode II). This indicates that \( G_f \) should be Mode I dominant. Furthermore, debonding is inherently peel-off rather than slip-off, although the shear mode is widely claimed to be the dominant failure mechanism (Rabinovitch (2004), Carpinteri et al. (2009)). Since there exist limited tests for peel-off debonding, particularly for detailed fracture process inspections, the debate about the
debonding modes has lasted for a long time (Taljsten (1997), Rabinovitch (2004), Pan et al. (2007), Burgoyne et al. (2012)). Thus, it is necessary to provide detailed fracture process observations experimentally.

Wedge-split Peel-off Test

The wedge-split test setup used for debonding fracture inspection are shown in Fig. 2. The details of the test can be found in Guan & Burgoyne (2014a). A wedge was used so that the vertical load caused the specimen to split, and a roller bearing system was used to transfer the wedge compression into splitting loads. The far end was supported on rollers and the region from the tip of the pre-notch will be inspected using Digital Image Correlation (DIC) technique to investigate the crack propagation and construct a strain field for the debonding fracture. The DIC technique uses photos taken before loading, and at different loading stages. The loading photos are compared with the photo before loading to identify the changes (cracks and strains). This DIC technique was specifically designed to inspect debonding fracture, and the details can be found in Guan & Burgoyne (2014b).

Three types of specimens were used. Double-cantilever beam (DCB) tests were symmetrical and plated on both sides. A wedge was driven between the two cantilevers, causing the initial crack, formed between the two cantilevers to open, as shown in Fig. 2(c). A cross-crack then formed to one side which then propagated along the interface between the CFRP and the concrete leading to debonding. It was observed that the highest load occurred when the cross-crack formed, while a lower and fairly constant load was required to make the crack extend. That load was used to determine the fracture energy together with a simple beam-foundation model, and the details can be found in Guan & Burgoyne (2014a).
With the observation of cross-cracks during the DCB tests, a second set of tests was carried out with timber-headed (TH) specimens to eliminate the influence of cross-crack on the determination of fracture energy during debond. The TH specimens were made using unplated DCB specimen that had been tested to failure. A piece of timber, with the same shape as the head of the DCB specimen, was screwed to the CFRP plate that was then bonded to one side of the specimen (Fig. 2(d)). The purpose of the timber head was to match the wedge for loading, and there was a gap between the piece of timber and the concrete. The only fracture that occurred was when the debonding fracture propagated.

The final set of experiments was with wider specimens (BIG) that contained reinforcements (Fig. 2(e)). The pre-crack was formed in the cover layer, that is, between the two layers of rebars, and the process of crack formation could be studied to see how the crack progressed to the surface and how it was affected by the presence of the steel.

The height and the width of the DCB and TH specimens are 600 mm and 50 mm respectively, while the height and width of the BIG specimen are 600 mm and 155 mm. All specimens were 100 mm thick. The specimens were plated with CFRP plates (Sika CarboDur S1012, 1.2 mm thick) using Sikadur 30 structural adhesive. Specimens with 10 mm and 20 mm maximum aggregate size were tested, since the concrete fracture process is commonly recognised to be influenced by the aggregate size; concrete with larger aggregates is claimed to have higher fracture energy (Bazant & Becq-Giraudon (2002)).
Figure 2  Wedge-split test setup with DIC techniques: (a) & (b) test setup; (c) DCB specimen; (d) TH specimen; (e) BIG specimen.

Typical Specimen Failure

A typical failure of the DCB specimens is shown in Fig. 3: During the test a cross-crack in the transverse direction occurs first starting from the pre-crack tip; after the cross-crack reaches the edge, a debonding crack propagates in the longitudinal direction along the concrete-plate interface; a thin layer of concrete is commonly found attached to the debonded plate, which indicates the failure is in concrete, not the adhesive. This is similar to the behaviour in a real RC beam, where debonding initiates from shear-flexural cracks (i.e. cross-cracks here); a layer of concrete normally remains attached to the debonded plate. For the TH specimens there would be no cross-crack, and for the BIG specimens the cross-cracking stage is influenced by the presence of the steel bars, which will be discussed in detail later.
Fracture for DCB specimens

The specimen responded almost elastically up to the peak load but then dropped quickly as the cross-crack develops. The load then remained fairly constant as the debonding crack propagated. A detailed study of the principal tensile strains for the test was given in (Guan & Burgoyne 2014a), but other important results can be deduced from looking at the horizontal (Fig. 4) and vertical (Fig. 5) strains separately. The strains mainly occurred in the vertical direction in the cross-cracking stage, and in the horizontal direction in the debonding stage.
Figure 4 Horizontal strains for Specimen DCB-CS-20-1
Figure 5 Vertical strains for Specimen DCB-CS-20-1
No obvious strain in the horizontal direction is noted where the cross-crack forms, except in Fig. 4(c) as it initiates from the pre-notch. From strain fields (e) and (f) in both Figs. 4 and 5 it is clear that the debonding regions show horizontal strains over 0.005, but little vertical strain.

There are some inconsistent high strain “dots” on the vertical strain figures which are probably due to uncertainties in the DIC searching and the strain contour interpolation, since they are very close to the area that is heavily influenced by the debonding crack. Only a small region just ahead of the high strain (0.005) tip shows both horizontal and vertical strains, with the horizontal strain much larger, as seen in strain fields (g) and (h). When tracing along a crack line it is concluded that the concrete crack forms by opening rather than slipping, and the region where both occur together is limited to a zone of a few mm in extent at most near the crack tip.

This mixing up of opening and slipping is likely to be due to interlocking of elements in concrete. The size of the mixing region in the debonding stage is larger than that in the cross-cracking stage, as can be seen, for example, by comparing the region of “horizontal strain in horizontal cross-crack” in Fig. 4(c) with the region of “vertical strain in vertical debonding crack” in Fig. 5(g) and (h). This indicates that the CFRP plate has the effect of connecting the concrete elements and improving their ability to interlock. After the test, the plate was peeled away manually, during which process the region with strain over 0.01 was noted to be able to take some loads by aggregate interlock. Although the strains can be determined, the stresses cannot be measured along the crack line, and therefore it is impossible to identify the traction-free crack tip. If a crack needs to propagate, it has to overcome both the resistance of the mixing region at the crack tip, and interlocking in a long crack line behind it. Thus concrete cracking shows mixed mode behaviour but only at a detailed scale.
Crack opening width

It is evident that the debonding crack is localised, so it is useful to determine the crack opening displacement. In order to study this, a region is identified so that the variations in displacement can be studied as the crack passes a particular point. The region identified by the dashed line in Fig. 4(d) is studied in detail. The nodal horizontal deformations are determined over the region consisting of ten columns of nodes at 1 mm intervals closest to the interface, and in 20 rows of 2 mm intervals, shown in Fig. 6: the horizontal axis value is the distance of a node from the interface before loading, with Node 0 = 1 mm, Node 1 = 2 mm, etc. The change in distances between a node and the right edge (interface) for all the nodes at various crack propagating stages (loading stage) is given by the vertical axis value. Positive distance change means elongation and negative means contraction, and it should be noted that “elongation” is due to crack opening at this level of details. Different curves represent the rows of nodes at different vertical levels. The distance change larger than 100 μm is set as 100 μm for better figure construction. Note that the scale of the plots is different before and after the crack passes through the region.
Figure 6 Change of distance between grid nodes closest to the debonding interface. The vertical axis is the change in distance (μm) with positive and negative values represent elongation and contraction, respectively. The changes of distance for the row of nodes just in front of the debonding crack tip are plotted in red, while the plot for the top row in the inspected region (Fig. 4 (d)) is made black. The plots for the rows between them, which are the rows influenced by the debonding crack, are in blue; otherwise they are shown in grey.

The first five figures show the situation before the debonding crack enters the inspected region; the displacements are evenly distributed and show small (< 2 μm) elongations (crack opening). No concrete crack was noted at this stage so it can be concluded that concrete with
relative displacements less than 2 µm is reasonably taken as uncracked. The debonding crack starts to enter the inspected region in Fig. 6(f), and the region for the transition from uncracked to cracked is circled. A consistently decreasing elongation from about 45 µm to close to zero was recorded from the top row in the inspected region (the black dashed line) to the 4th row (8 mm away from top); but as the elongation reduces the width of the elongating region increases up to about 4.5 mm away from the interface. The initial elongation trend is noted at the node about 4 mm from the interface and 8 mm from the 0.01 high strain zone (i.e. the 4th node on the red curve) where the elongation is about 4 µm.

This phenomenon is found consistently in the later figures, given by the red lines for the initiation of crack, which indicates that if concrete opens by more than 4 µm, it should be counted as cracked. The overall width influenced by the debonding crack is found to be less than 5 mm. Thus, a single concrete crack can be represented by a continuous concrete opening of over 4 µm, corresponding to a strain of 0.004 in the strain field plot in Fig. 4 & 5 (because the gauge length is 1 mm), with a width usually less than 5 mm. The corresponding FPZ must be inside this region so is small.

Introduction to the TH and BIG specimens

The results from the DCB specimens were fairly clear, but two further sets of tests were carried out to check particular influences.

Timber head (TH) specimens enabled the crack to reach the concrete/CFRP interface without having to form the cross-crack, and thus minimise the influence to the debonding crack study. A total of six specimens were tested, each three with 10 mm and 20 mm aggregates, and the results are summarised in Table 1.
Table 1  Test results for TH specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Adhesive</th>
<th>Aggr. Size (mm)</th>
<th>Peak Load (N)</th>
<th>Average Gf (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TH-CS-10-1</td>
<td>Sikadur</td>
<td>10</td>
<td>83.2</td>
<td>0.127</td>
</tr>
<tr>
<td>TH-CS-10-2</td>
<td>Sikadur</td>
<td>10</td>
<td>131.6</td>
<td>0.175</td>
</tr>
<tr>
<td>TH-CS-10-3</td>
<td>Sikadur</td>
<td>10</td>
<td>183.7</td>
<td>0.232</td>
</tr>
<tr>
<td>TH-CS-20-1</td>
<td>Sikadur</td>
<td>20</td>
<td>81.8</td>
<td>0.092</td>
</tr>
<tr>
<td>TH-CS-20-2</td>
<td>Sikadur</td>
<td>20</td>
<td>139.2</td>
<td>0.135</td>
</tr>
<tr>
<td>TH-CS-20-3</td>
<td>Sikadur</td>
<td>20</td>
<td>78.5</td>
<td>0.123</td>
</tr>
</tbody>
</table>

A typical load-deflection curve and the evolution of the principal tensile strain fields for Specimen TH-CS-20-1 are shown in Fig. 7, the loading states for the strain fields are denoted on the load-deflection curve by dots. Note here that there is no initial peak load associated with the formation of the cross-crack, as would be expected. The load continues to increase as the crack extends mainly due to the elastic bending of the CFRP plate, which will be explained later.
Figure 7 Principal tensile strain for Specimen TH-CS-20-1

The debonding crack does not initiate directly from the specimen top edge, but from a flaw about 15 mm lower down (Fig. 7(d)). Once the high strain from the flaw reaches the interface, the debonding crack propagates both upwards to join the initial interface crack and downwards (Fig. 7(e) & (f)). Continuous strain development with relatively low value is also noted in the middle region from strain fields (f) to (h) about 10 – 30 mm away from the interface, which is comparable to the aggregate size. Debonding is not the only phenomenon during the process and it is accompanied by interlocking and tendency of cracking away from the interface. The last two strain fields were obtained at the same wedge displacement at an interval of about 5 minutes. A small drop of load and a small growth in the debonding cracking region are noted, which indicates that debonding cracking is affected by a slow creep.
The almost vertical unloading curve indicates again the load resistance from interlocking. It is evident that interlocking has an unpreventable influence on fracture behaviour even in such a test that the debonding was made to appear as peeling off, and the influence should depend on the concrete heterogeneity.

The variation in the fracture energy with wedge displacement, calculated using the Fixed-End Cantilever model described in Guan & Burgoyne (2014a) is shown in Figure 8.

The average debonding fracture energy is found to be 0.147 N/mm with a standard deviation of 0.072 N/mm, which are very similar to the values for the DCB specimens reported in Guan & Burgoyne (2014a) (0.142 and 0.077 N/mm respectively). The comparison with the DCB results is notable. In those specimens, there was little interlocking and friction associated.

Figure 8  Debonding fracture energy obtained for TH specimens
with debonding cracking. In the TH specimens there is much more. Yet both give very
similar values for the Fracture Energy. So it can be concluded that the influence of possible
interlocking and friction from the cross-crack region in DCB specimens on the debonding
crack is negligible. The TH specimens with 20 mm aggregates give a slightly lower $G_f$ value,
with an average of 0.117 N/mm, which is opposite to the normal observations for the fracture
energy in bulk concrete that concrete with larger aggregates should give higher fracture
energy (Bazant & Becq-Giraudon (2002)). This is probably due to the poorer packing of
concrete with larger aggregates.

Interlocking is essentially a Mode II behaviour, but the fracture energy ends up being very
similar to the conventionally accepted value for Mode I and is effectively indistinguishable
from the value for the DCB tests. The fracture process here, with both Mode I and Mode II
elements, is a mixed-mode process, but with a low fracture energy in Mode I, and a much
higher value in Mode II, a specimen preferentially takes any possible Mode I fracture route.
Even if there is interlocking, the fracture energy is dominated by the Mode I value.

Fractures in BIG specimen

It was noted above that the debonding observed in the DCB and TH tests occurred at the
cement/adhesive interface, whereas in most observations on real beams debonding takes
place at the steel level. The BIG specimens were thus designed to include steel trying to give,
but it was noted in the tests that the steel has little effect on the onset of the debonding. It is
likely that in real beam debonding there is pre-existing damage at the steel lever from
flexural/shear cracks before debonding. Hence the weakest location in the cover layer is not
at the interface but in the concrete further away. Although the debonding crack was still
along the interface, the BIG specimen tests have shown interesting observations of the
influence of steel bars in debonding and concrete fracture; typical results are presented below.

Note here that the debonding fracture energy was not calculated for these specimens since:

(i) there were complex multiple cracks, as well as the debonding crack, so it is not clear which part of the energy should be allocated to the debonding crack, and

(ii) the deformation of steel bar (plastic and elastic) consumed energy which could not easily be determined.

Figs. 9 and 10 show the load-wedge displacement curve and the principal tensile strains for Specimen BIG-CS-10-2. The positions of the steel bars (4 mm diameter) are marked in Fig. 10(b).

Figure 9 W-D curve for Specimen BIG-CS-10-2
Figure 10 Principal tensile strain for Specimen BIG-CS-10-2
The pre-peak response in this case is similar to the DCB specimens, with an almost linearly elastic response while the cross-crack forms (Strain Field (a)); the steel bar has little effect on increasing this elastic peak strength. However, there is no sudden drop of strength after the elastic stage, and the cross-crack does not propagate directly to the debonding surface: instead it bends into the vertical direction, as shown in strain fields (b) and (c). It is also noted that the cross-crack does not propagate rapidly, but opens wider. This is similar to the ordinary flexural behaviour RC beam where steel plays an increasing important role as concrete cracks.

The cross-crack turns into the debonding crack from Strain Field (d) on, and the strength of the specimen reduces gradually as the debonding crack propagates. The bending and dowel action of the steel bar lead to complex secondary cracks, inclined from the cross-crack level to the debonding interface. These are identified in Fig 10 as interlocking regions and are outlined with a dashed line. All the secondary crack development starts from interlocks along the existing cracks. The size of the interlocking region, with strains less than 0.01, is about 5 – 10 mm long before becoming a crack, which is comparable to the aggregate size. It is evident that the effect of steel bars is to distribute the crack, making the debonding process gradual.

Although the crack pattern is complicated, it is clear under the DIC inspection that they are individual cracks having their individual fracture process zones, each of which is small. The secondary cracks tend to develop individually, rather than joining together. For example, the vertically developing cracks at the steel bar level do not join the debonding cracks or join together from strain fields (e) to (g). When the CFRP plate was peeled off manually after the test, a layer of concrete fell as well, which confirms that the thick layer of concrete attached to the plates in real beam debonding is damaged in processes prior to debonding. In unloading, the strength drops suddenly as in the previous cases, indicating that interlocking is significant.
Fractures in unplated BIG specimen

The debonding crack is not the only dominant crack in the BIG specimens, so the unplated BIG specimens were used as a comparison to investigate the influence induced by CFRP plate. Figs. 11 and 12 show the W-D curve and the principal tensile strain fields for Specimen BIG-UP-10-2: load and reload cycles were carried out in the test.

Figure 11  W-D curve for Specimen BIG-UP-10-2
Initially the load is mainly taken by the concrete, and once the concrete cracks the strength drops, which corresponds to strain fields (a) to (c). It can be seen that the cross-cracks do not
follow the horizontal direction and a secondary crack along the steel bar level starts to
develop just after the elastic peak load. This is similar to the transition from cross-crack to
debonding crack in the DCB specimen, but now with the interface level at the steel bar. From
Strain Field (d) on, the steel bars gradually yield, and the interlocking of concrete also
dissipates energy, so the load-reload cycles are inelastic. Comparing the strain fields (d) to (f),
and (g) to (h), the regions with strains of about 0.001 – 0.002 away from the high strain region
(> 0.005) can recover during unloading, while the high strain region cannot. It is consistent
with the crack determination previously in Fig. 6.

**Strain fields for timber splitting test**

Concrete is recognised as a quasi-brittle material and argued to have a large FPZ, for which,
however, there is a lack of experimental evidence. The FPZ observed above is small,
contradicting the large-concrete-FPZ argument, so it is important to prove that this small FPZ
is real rather than induced by the DIC technique. To do this, the ability of DIC to capture the
FPZ is studied for timber which is an easily accessible quasi-brittle material with a well-
recognised large FPZ. An unplated timber DCB specimen was tested, with the grain of the
wood vertical so the wedge is cleaving the wood apart along a line of weakness. The results
are shown in Fig. 13: The viewing window is about 40 mm square across almost the whole
width of the specimen. The principal tensile strain field is overlapped on the cracked timber
image. The FPZ for timber appears as a zone rather than a crack line, and the zone is large,
extending across the whole width of the specimen. So the DIC technique can detect a large
FPZ, and thus its absence in the concrete tests is real.
Conclusions

The strain fields for debonding fracture provide a clear observation of the fracture process, and clarify the size of the fracture process zone associated with debonding and the sources of fracture resistance. At the detailed scale, all the concrete fracture is resisted by a mixing of cohesion, interlock and friction. It has been shown that there exists no real large fracture process zone for concrete fractures in debonding, in contrast with timber. It has been found that interlocking is inevitable along a crack line, and cannot be prevented. Hence crack opening is always in mixed mode when considered at a detailed scale. The resistance from interlocks varies from case to case, but generally has an insignificant effect on the final fracture energy value associated with debonding. The Mode II fracture energy is much higher than Mode I and a crack would preferentially follow any existing Mode I routes to progress. Generally, concrete with cracks less than 2 μm wide can be taken as intact, while with cracks over 4 μm wide should be taken as damaged, even though the material along both sides of the crack may still be able to take some load. Despite the presence of interlocking in some of the
tests, the average fracture energy measured in the tests described here is about 0.14 N/mm, which is very close to the value commonly accepted for the Mode I fracture of concrete.

References


