QUANTITATIVE FRACTURE CHARACTERISTICS IN SHEAR LOAD

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Abstract

Today, no general design method for shear strength capacity of RC beams can predict the failure load with a high degree of accuracy. The failure load between two beams cast from the same batch may vary as much as 30 percent. This project aims at investigating factors affecting the shear strength capacity based on an understanding of micro and meso scale material properties.

Tests were performed on beams with two different types of aggregate and two different w/c. The crack propagation was monitored during the loading by means of DIC and AE. The results show that the use of natural aggregates or crushed aggregates as finer fractions strongly affect the shear strength capacity. From direct shear tests it was found that the scatter of the shear strength was much higher for the specimens with natural aggregates compared with crushed aggregates. The same tendency was found for the shear crack initiation load for the RC beams. The percentage of fractures propagating through aggregate, paste and the ITZ varied with w/c, type of aggregate and type of failure. Microscopy in combination with DIC and AE measurements makes it possible to determine at what stage different cracks have been formed and their relation to the micro structure.

Keywords: RC beams, shear strength capacity, microscopic analysis, Digital Image Correlation, Acoustic Emission
1 Introduction

Research on the shear strength capacity of reinforced concrete beams is more than 100 years but the research society has so far not been able to develop a general design method which fulfil high demands on precise determination of the failure load. Today, design and assessment with regard to shear cracking, are made using simplified methods. These assume that the shear strength capacity is directly proportional to both the beam width and the efficient height. The concrete properties are summarized in a strength value that either is based on the tensile strength or the square root or the cubic root of the compressive strength. However, in an investigation of the ACI recommendation Rebeiz (1999) concluded that there is hardly any correlation between square root of the compressive strength and the shear failure load, neither for normal strength concrete (NSC), nor for high strength concrete (HSC). Moreover, concrete types having equal strength are treated the same in the methods, independent of differences in stiffness, load-displacement curve, fracture mechanics properties, presence of micro-cracks or other defects, concrete mix, cement type, maximum aggregate size, aggregate grading, aggregate type, age, curing conditions, etc. The effect of aggregate size was recently studied by Sherwood et al. (2007). They state that it has importance since shear failure is initiated by breakdown of aggregate interlock capacity at a crack. Their experiments show that decreasing maximum aggregate size can significantly decrease the shear capacity of thick slabs.

Testing of reinforced concrete beams in shear most often gives large scatter in the test results. Even simple beams cast simultaneously of the same concrete batch may show differences in measured load-carrying capacity of 15 to 30 percent. The issue is not much discussed in the literature; however, it is important to investigate, not least today when the minimizing of natural recourses is of uppermost global interest. Traditional and conservative methods provide certainly decent security against failure but also to unnecessarily large use of natural recourses in a high percentage of the concrete structures around the world.

The present study aims at investigating factors affecting the shear strength capacity based on an understanding of micro and meso scale material properties. Direct shear tests and shear tests on reinforced concrete beams (RC beams) were performed. The concretes had a w/c of 0.38 or 0.9 and fine aggregate of natural or crushed rock material. An approach based on careful quantitative microscopical analysis of crucial microstructures in combination with 3D optical deformation measurement of cracks during loading has been used. This makes it possible to determine at what stage different groups of cracks have been formed and their relation to the micro structure. This approach offers a new possibility for understanding the scatter of shear strength capacity. Caduff and Van Mier (2010) recently used a similar methodology in their study of fracture process of concrete in compression.

2 Methods for fracture characterisation

2.1 Digital Image Correlation

In this study the crack propagation was registered in a detailed way at one side of the specimens during the testing by the use of the optical full-field deformation measurement system ARAMISTM 4M by GOM. The system uses a measurement technique based on Digital Image Correlation (DIC) with a stereoscopic camera setup, consisting of two CCD-cameras with 4.0 Mega pixel resolutions. The basic idea behind DIC is to measure the displacement of the specimen under testing by tracking the deformation of a natural occurring, or applied surface speckle pattern in a series of digital images acquired during the loading. This is done by analysing the displacement of the pattern within discretized pixel subsets or facet elements of the image. In combination with correlation based stereovision technique the measurement of 3D shapes as well as the measurement of 3D displacements fields and surface strain field is possible.
The method has proved very suitable for the study of concrete structures, where one is interested in studying cracking in a detailed manner. It is possible to follow the propagation of all different cracks in a field long before they are visible to the naked eye, and subsequently measure the crack width development locally over each crack with high accuracy.

2.2 Acoustic Emission

Acoustic Emission (AE) are elastic stress waves produced by sudden movements in stressed in materials caused by crack growth and plastic deformation, etc. A sudden movement at the source produces a stress wave, which radiates out into the structure and excites the AE sensors. This type of AE often have very small amplitudes and are in principle always high frequency. Typical frequencies tend to be optimal for AE measurements can be within 60 kHz - 300 kHz. Therefore, AE are measured with highly sensitive piezoelectric resonant sensors in the ultrasonic range. The key element in an AE resonant sensor is the piezoelectric crystal that converts the movements into electrical voltage signals that are sent to a measuring computer for further signal processing.

AE offers real-time measurement of crack formation and is not limited to a single measuring point, but is volumetric. AE could detect crack initiation and crack growth, and provide information on when damage is accelerating. If the running time is measured from the AE source to each sensor and the speed of sound is known, the origin of the signal can be located. Localization requires a clear signal separation and that the signal reaches a number of sensors. Depending on the number of sensors, it is possible to locate the source in line, in plane and in space.

2.3 Micro- and meso-scale analysis

The analyses of the tested samples have been performed using fluorescence microscopy on thin sections of samples impregnated with epoxy with fluorescent dye. The analysed thin sections were cut parallel to the surface analysed using DIC. The size of the thin sections were approximately 70×50 mm\(^2\) with their length axis oriented parallel to the main shear cracks. The measurements have been performed using the KS400 image analysis program on images covering an area of 2.8×2.1 mm\(^2\).

The parameter used to describe aggregate particle shape was the minimum length divided by the maximum length ($F_{\text{min}}/F_{\text{max}}$). In order to reduce stereological artefacts the half of the objects with lowest $F_{\text{max}}$ has been excluded from the measurement. Generally this is the objects with an $F_{\text{max}}$ smaller than about 100 micron. The remaining measured particles were to 80 vol-% is in the size range 0.25 to 2 mm assessed according to NT BUILD 486. Orientation of $F_{\text{max}}$ on elongated aggregate particles was measured. In this measurement were also objects with $F_{\text{min}}/F_{\text{max}}$ larger than 0.6 have been excluded. The edge objects has been excluded both in the measurement of shape and orientation (Lindqvist & Johansson 2007).

The macroscopic analysis was performed on lapped concrete slabs of the approximate size 450×150 mm\(^2\). The slabs were impregnated with epoxy glue containing fluorescence dye. The cracks were manually colour coded and subsequently measured using the image analysis programme KS400. The result is presented as percentages of the crack length in the various constituents in relation to the total crack length.

3 Test description

In this study direct shear tests and shear tests on RC beams were performed. The test series includes four different concrete recipes. The concretes had a w/c of about 0.38 or 0.9 and fine aggregate (< 8 mm) of natural or crushed rock material. The detail of the composition of the mixture used in this test series is shown in Tab. 1. For each recipe four direct shear test specimens and four RC beams were manufactured (in total 32 specimens). The cube compressive strengths were determined at 28 days and at the time of testing (208 days). The cubes (150x150x150 mm\(^3\)) were stored along with the specimens.
3.1 Specimen preparation

The specimens were sawn cut from slabs of dimensions 1200x700x180 mm³, designed according to Fig. 1. The slabs were cast in moulds made of shuttering plywood, see Fig. 2. The reinforcement was placed in pre-drilled holes in the mould long sides, which ensure that the reinforcement bars were placed in the correct position with high accuracy. An internal vibration was used for the compaction of the concrete. After casting the slabs were covered with plastics and cured at approximately 20 °C until the specimens were cut.

The smooth bottom surface of the slab against the shuttering plywood was used as the bottom of the specimens. The other three sides of the specimens were sawn out. The saw cuts are schematically shown in Fig. 1. After preparation the specimens were stored at approximately 20 °C until testing. The design of the direct shear test specimens is shown in Fig. 3. The specimens were 350 mm in length and had a square cross-section of 150×150 mm². Four 10 mm deep and 2.5 mm wide notches were sawn in the upper and lower edges of the specimens to define shear failure planes with a ligament area $A_{lig}$ of 150×130 mm². The RC beam specimens were designed to receive a shear failure. The specimens had a length of 700 mm, a square cross-section of 150×150 mm² and were provided with 3 ribbed bars Ø12 mm as bottom reinforcement, see Fig. 4.

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**Tab. 1** Material description.

<table>
<thead>
<tr>
<th></th>
<th>C0.90</th>
<th>N0.90</th>
<th>C0.38</th>
<th>N0.38</th>
</tr>
</thead>
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<tr>
<td><strong>w/c</strong></td>
<td>0.94</td>
<td>0.90</td>
<td>0.38</td>
<td>0.38</td>
</tr>
<tr>
<td><strong>Fine aggregate 0-5 mm [kg/m³]</strong></td>
<td>1126 Crushed</td>
<td>1082</td>
<td>1064 Crushed</td>
<td>900</td>
</tr>
<tr>
<td><strong>Coarse aggregate 8-16 mm [kg/m³]</strong></td>
<td>720</td>
<td>815</td>
<td>708</td>
<td>900</td>
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<tr>
<td><strong>Water [kg/m³]</strong></td>
<td>220</td>
<td>198</td>
<td>164</td>
<td>162</td>
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<tr>
<td><strong>Cement [kg/m³]</strong></td>
<td>234</td>
<td>220</td>
<td>429</td>
<td>426</td>
</tr>
<tr>
<td><strong>Water reducer [kg/m³]</strong></td>
<td>0</td>
<td>0</td>
<td>1.1</td>
<td>0.7</td>
</tr>
<tr>
<td>$f_{c,cube}$ at 28 days [MPa]</td>
<td>19.2</td>
<td>21.2</td>
<td>80.9</td>
<td>91.5</td>
</tr>
<tr>
<td>$f_{c,cube}$ at 208 days [MPa]</td>
<td>20.9</td>
<td>21.6</td>
<td>84.5</td>
<td>96.9</td>
</tr>
</tbody>
</table>
3.2 Test set-up and performance

The tests were carried out in an Instron 2008 with a maximum load capacity of 1.3 MN. The tests were carried out under displacement control with a rate of 0.1 mm/min in the direct shear tests and 0.5 mm/min in the shear tests of the RC beams. The force was recorded by load cell and the specimen deformations were recorded by potentiometers, which were stuck on the upper loading plate and connected to a high-speed data logger. The crack propagation was monitored at one side of the specimen during the loading by means of DIC with an image capture frequency of 1 Hz.

The overall direct shear test set-up is shown in Fig. 3 and Fig. 4. This kind of direct shear test is often referred to as "punching shear tests". In this test the shear load is applied by a loading block with two quadrilateral prism edges 150 mm apart. The specimen is supported on another rigid block over a pair of quadrilateral prism edges that are 155 mm apart. Thus, a narrow, 2.5 mm wide region of the specimen in between the loading and the supporting quadrilateral prism edges is subjected to a concentrated shear stress. In these narrow shear spans, the notched sections were placed to initiate the crack and to dictate the shear failure planes.

![Diagram of test set-up and design of direct shear specimens and RC beam specimens](image)

**Fig. 3** Illustration of the test set-up and design of the direct shear specimens (left) and the RC beam specimens (right). A DIC measuring area of 200x200 mm$^2$ was used in the direct shear tests and in one of the RC beam tests. For the remaining RC beam tests a measuring area 500x500 mm$^2$ was used.

The RC beam specimens were loaded in four point bending as shown in Fig. 3 and Fig. 4. The beams were loaded through and supported by half-moon shaped steel rulers with a radius of 20 mm. The free distance between the supports was 450 mm and the shear spans between the loading and the supporting ruler were 150 mm. During two of the RC beam tests the cracking process was studied also by acoustic emission. The AE activity was recorded by a total of four AE-sensors; one attached to each side of each beam end, see Fig. 4. The AE-sensors had a peak sensitivity of 60
kHz. In these two tests the loading was somewhat different. First, the beams were loaded to 150 kN where after the load was held constant for 5 minutes. Then the load was decreased to 100 kN and held constant for 1 minute before the beams finely were loaded to failure.

4 Test results

4.1 Direct shear test

A summary of the direct shear test results can be found in Tab 2. The shear strength was evaluated as half the total applied load at failure divided by the area of the shear ligament where failure occurred.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Shear strength [MPa]</th>
<th>Mean [MPa]</th>
<th>CV [%]</th>
<th>Specimen</th>
<th>Shear strength [MPa]</th>
<th>Mean [MPa]</th>
<th>CV [%]</th>
</tr>
</thead>
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<tr>
<td>N0.90DS_1</td>
<td>2.55</td>
<td>2.44</td>
<td>9.2</td>
<td>C0.90DS_1</td>
<td>4.36</td>
<td>4.05</td>
<td>8.20</td>
</tr>
<tr>
<td>N0.90DS_2</td>
<td>3.44</td>
<td>3.82</td>
<td>3.94</td>
<td>C0.90DS_2</td>
<td>3.51</td>
<td>2.99</td>
<td>2.75</td>
</tr>
<tr>
<td>N0.90DS_3</td>
<td>2.49</td>
<td>2.88</td>
<td>2.14</td>
<td>C0.90DS_3</td>
<td>3.83</td>
<td>3.45</td>
<td>2.25</td>
</tr>
<tr>
<td>N0.90DS_4</td>
<td>3.82</td>
<td>3.65</td>
<td>1.4</td>
<td>C0.90DS_4</td>
<td>4.05</td>
<td>3.78</td>
<td>1.25</td>
</tr>
</tbody>
</table>

From the DIC measurements, it could be observed that the failure of the C0.38 and N0.38 specimens were much more sudden compared with the C0.90 and N0.90 specimens which exhibit a slightly more gradual crack propagation. Furthermore, in the C0.90 and N0.90 cracks usually developed in both ligaments while it in the C0.38 and N0.38 most often occurred as a sudden break in one ligament, see example in Fig. 5. The cracks were not always completely created within in the ligament dictated by the upper and lower notch. It could also be seen that the crack propagation in many cases clearly was controlled by the larger aggregate grains by following the weaker adhesion zones.

![Visualization of cracking for C0.38DS_4 and N0.90DS_3 at failure.](image)

4.2 Shear test on RC beams

The load-displacement relations from the shear tests on the RC beams are presented in Fig 6 and a summary of the results can be found in Tab 3. The shear capacity was evaluated as half the total applied load at failure.
Fig. 6 Load-displacement relations from the shear tests on RC beams.

Tab. 3 Summary of results for the RC beam tests.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Final failure</th>
<th>Shear capacity [kN]</th>
<th>Mean [kN]</th>
<th>CV</th>
<th>Specimen</th>
<th>Final failure</th>
<th>Shear capacity [kN]</th>
<th>Mean [kN]</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>N0.90RC_1</td>
<td>shear</td>
<td>73.1</td>
<td>74.8</td>
<td>2.4</td>
<td>N0.38RC_1</td>
<td>crushing</td>
<td>128.3</td>
<td>128.2</td>
<td>1.5</td>
</tr>
<tr>
<td>N0.90RC_2</td>
<td>shear</td>
<td>74.6</td>
<td></td>
<td></td>
<td>N0.38RC_2</td>
<td>crushing</td>
<td>130.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>N0.90RC_3</td>
<td>shear</td>
<td>76.7</td>
<td></td>
<td></td>
<td>N0.38RC_3</td>
<td>crushing</td>
<td>125.5</td>
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<td>N0.90RC_4</td>
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<td>-</td>
<td></td>
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<td>N0.38RC_4</td>
<td>crushing</td>
<td>128.8</td>
<td></td>
<td></td>
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<tr>
<td>C0.90RC_1</td>
<td>shear</td>
<td>76.0</td>
<td>72.7</td>
<td>13.2</td>
<td>C0.38RC_1</td>
<td>crushing</td>
<td>130.9</td>
<td>130.1</td>
<td>2.4</td>
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<tr>
<td>C0.90RC_2</td>
<td>anchorage</td>
<td>61.9</td>
<td></td>
<td></td>
<td>C0.38RC_2</td>
<td>crushing</td>
<td>132.7</td>
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<tr>
<td>C0.90RC_3</td>
<td>shear</td>
<td>80.2</td>
<td></td>
<td></td>
<td>C0.38RC_3</td>
<td>crushing</td>
<td>126.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C0.90RC_4</td>
<td>1)</td>
<td>-</td>
<td></td>
<td></td>
<td>C0.38RC_4</td>
<td>1)</td>
<td>-</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1) Not tested.

With results from the DIC measuring system, it was possible to study how individual cracks developed during the test. In Fig. 7, the crack formation for C0.38RC_3 and N0.38RC_4 are visualized at 150 kN and at maximum load. These were the two specimens that also were registered by AE. These were the two specimens that also were registered by AE. The final crack pattern obtained by vacuum impregnation with fluorescent epoxy after testing is presented in Fig. 8.

In both cases one can observe that the first flexural cracks occurs around 37-40 kN near the center of the beam. This fits well with the time when the acoustic emission measurements started to register activity. Localization of AE activity was possible only in the beginning of the cracking since the amount of simultaneous signals then became too large to separate. With increased loading flexural-shear cracks then arises closer to the supports. The inclined shear cracks occurs at around 120-145 kN for N0.38RC and around 150-170 kN for C0.38RC. They start approximately 40 mm from the bottom beam edge and grow downwards to respective support and upwards connecting to existing flexural-shear cracks and further towards the loading ruler. After 40 to 50 kN the AE energy increases for all sensors with increasing load and exhibit peaks associated with sudden and intense crack growth.

As can be seen in Fig. 7 N0.38RC exhibits considerably more cracking than C0.38RC at the load level of 150 kN. This is consistent with the amount of AE which is recorded for each beam at that time. At loads of about 160 kN the amount of AE decreases for N0.38RC while the activity lasts longer for C0.38RC. By the DIC it can be seen that several new cracks were created for C0.38RC while for N0.38RC the existing cracks instead grow in width, which probably explains...
the difference in the amount of AE in the later part of the loading. As indicated by the load-displacement curve in Fig. 6 in the reinforcement most likely starts to yield at a load of approximately 250 kN. In both cases the inclined shear cracks starts to go horizontally as they grows closer to the neutral axis and final failure is dictated by crushing/spalling (denoted “crushing” in Tab. 3) of the compressive zone. This can also clearly be seen in Fig. 8.

The crack development for C0.90RC and N0.90RC were similar to C0.38RC och N0.38RC. The flexural cracks appear between 20-30 kN. The flexural-shear cracks starts at around 35-45 kN for N0.90RC and at around 50-60 kN for C0.90RC. The inclined shear cracks starts in most cases approximately 50 mm from the bottom beam edge at around 70-80 kN and then grow downwards to respective support and upwards connecting to existing flexural-shear cracks. In some cases the inclined shear cracks are instead branching from the existing flexural-shear cracks downwards to the support. Except in one case, the final failure occurs when the inclined shear crack passes through the compressive zone just outside the loading ruler (denoted “shear” in Tab. 3) as the combination of shear stresses and compressive stresses becomes too high. For C0.90RC_2 a premature failure occurs at 62 kN due to inadequate anchorage capacity (denoted “anchorage” in Tab. 3) at the support when the inclined shear crack starts to propagate along the reinforcements.

5 Micro- and mesoscale analysis

The analysis shows that there is difference between the samples (Tab 4 and Fig. 8). Low w/c gives a higher proportion of failure in aggregate and a lower proportion of adhesion cracks. The proportion of adhesion cracks is higher in the RC-beams while the direct shear gives a higher proportion of cracks in the aggregate. Point counting on the polished samples gives a total aggregate content of 35 vol-% which is higher than the percentage of failure in aggregate for all samples. The percentage of failure in dark amphibole bearing rocks is 6% and in granitic rocks 94% of the total crack length aggregate. The amphibole bearing rocks in the aggregate, determined through point counting on the polished samples, is about 20% while the amount granitic rocks is about 80%. It is only in the concrete with w/c 0.38 and natural aggregate that failure occurs in dark amphibole bearing rocks. Thus cracks have a relative preference for cement paste and adhesion zones rather than aggregate and in aggregate rather granite than amphibolite.
Fig. 8 The figure shows the different crack geometry in the shear test of the RC-beams C0.38RC_3 and N0.38RC_4 seen in Fig. 7 with crushed (0.38C) respectively natural aggregate (0.38N). The lower part of the beam and the reinforcement was sawed away in the preparation.

Fig. 9 Shows results from the microscopic analyses of the tested RC-beams. The left hand diagram shows the shape distribution for the fine fraction of the natural and the crushed aggregate particles. A particle with a $F_{\min}/F_{\max}$ in the range 0.9-1.0 is spherical or almost spherical while particles with low $F_{\min}/F_{\max}$ have a more flaky shape. The right hand figure shows the lack of orientation of the aggregate seen as a total measured on several images distributed in the whole thin section. The particles that plot in the class 0-22.5 degrees are parallel or sub-parallel to the main shear cracks and the particles that that plot in the class 77.5 to 90 degrees are perpendicular to the cracks.

Tab. 4 The results from the macroscopic quantification given as percentage crack length in the various constituents. Rocks are divided into amphibolite and granite. Samples denoted N contains natural aggregate and C crushed aggregate; DS is direct shear and RC is the reinforced beams; the number gives the w/c 0.38 respectively 0.90.
The frequency distribution of the aggregate shapes shows that the crushed aggregate in the 0.25 to 4 mm fraction contains a higher amount of elongated and flaky particles, with low $F_{\text{min}}/F_{\text{max}}$, compared to the natural aggregate. The natural aggregate on the other hand has, as expected, a higher number of rounded particles. The quantitative microscopic analysis of the aggregate particles shows a significant orientation in most of the analyzed images (Fig. 9). The orientation of the aggregate differs however between the different images (2.8×2.1 mm²) and there is no general orientation that is consistent in the area of a thin section (70×50 mm² Fig. 9). This orientation pattern occurs in both the samples with natural and crushed aggregate. Microscopic analysis of samples with crushed aggregate in the fine fraction shows that the walls of the cracks are to a larger extent intersected by fine elongated aggregate particles compared to the crack walls in natural aggregate samples. This gives rougher and less straight cracks in the crushed aggregate samples compared to natural aggregate samples.

Fig. 10 The images, with their length axis parallel to the main shear cracks, show the variation in particle orientation in the samples with crushed aggregate and w/c 0.90. The histograms show the percentage of particles with length axes oriented in 0 to 90° to the direction of the main cracks. The orientation of three particles is given in the left hand image. The legend to the histograms is given in Fig. 9. Each image covers approximately 2.8×2.1 mm².

6 Concluding remarks

From the direct shear tests it was found that the scatter of the shear strength was much higher for the specimens with natural aggregates (up to 21%) compared with the specimens with crushed aggregates (up to 9%). The scatter for the compressive strength was lower than 5% for all four materials. Moreover, it was found that the shear strength was higher for the specimens with crushed aggregates compared to the specimens with natural aggregates, despite the fact that the relation in compressive strength between the comparable materials was the opposite. This indicates that the composition of the concrete has a great influence on the shear strength and that a design model for shear capacity not solely can be based on the compressive strength.

Inclined shear cracks were obtained in all tested RC beam specimens. However, the final failure differed between the low strength and the high strength specimens. In the low strength specimens the final failure occurred when the combination of shear stresses and compressive stresses becomes too high and inclined shear crack passes through the compressive zone, whereas for the high strength specimens the compressive zone withstand the shear stresses and the final failure was instead dictated by crushing/spalling of the compressive zone. The scatters of the failure load are relatively low for the RC beam tests. This might be explained by the fact that the failure mechanism is more or less dictated to the compressive strength of the concrete. If one instead compare the load at which the shear cracks are initiated the picture becomes slightly different; the shear crack initiation load is in general higher for the specimens with crushed aggregates compared to the specimens with natural aggregates and the scatter is higher within each test group.

The microscopic and the meso-scale analysis show how the shape of the fine aggregate particles and the aggregate rock type influence the fracture geometry. It also provides detailed qualitative and quantitative information on which constituents the fractures propagate through. A major difference is the higher percentage of adhesion cracks in the low strength concretes. The
elongated particles oriented nearly perpendicular to the direction of crack propagation acts as energy barriers for the crack tip where it changes direction or propagates through also very fine particles. As a result the crack, both in micro and meso-scale, is closer to a straight line in the samples with natural aggregate compared to the more complex crack pattern in the samples with crushed aggregate. In the RC-beam tests and in the determination of cube-strength this is not the case. A suggestion is that in micro domain with tensile load the elongated fine particles rather act as disturbances and in domains with an unsuitable particle orientation they may interact as a critical disturbance. The results furthermore indicate that the cracks are less likely to propagate through amphibolite than through granite. This is probably due to higher fracture toughness in the amphibolite compared to granite.

The crack propagation was monitored during the loading by means of 3D Digital Image Correlation (DIC) and Acoustic Emission (AE). AE offers real-time measurement of crack activity within the specimen. By comparison with DIC results it was confirmed that AE could detect crack initiation and crack growth, and provide information on when cracking is accelerating.

The results from the DIC were confirmed by the results from the fluorescence microscopy performed after testing. By the DIC it is possible to follow the cracks from relatively early stages of loading and by the fluorescence microscopy it possible to study the final crack formation in a detailed manner. Hence, fluorescence microscopy in combination with DIC measurements makes it possible to determine at what stage different cracks have been formed and their relation to the micro structure. The approach used in the present investigation makes it possible to identify the limiting parameters in the concrete depending on the type of load. This information can in a subsequent step be used for optimizing critical concrete properties.

References


