Fracture mechanics of concrete and its role in explaining structural behaviour

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ABSTRACT: Major steps forward in the history of the development of fracture mechanics, as a tool for describing or predicting the behaviour of concrete structures, have been the fictitious crack model, introduced by Hillerborg, and the compressive damage zone model, introduced by Markeset. In structural elements, however, often the conditions are less clear than in concentric tension or compression tests. The behaviour of concrete beams subjected to shear and slabs subjected to punching are illustrating examples, especially when size effects are concerned. Another phenomenon that required further study is the rotation capacity of slabs at intermediate supports. Since mostly the ultimate rotation is reached at failure of the concrete in the compression area, the question is justified whether rotation capacity is subject to size effects as well. Since concrete is a brittle material, it is necessary to design structures in such a way that the structural behaviour becomes ductile nevertheless. One way is to adequately reinforce the structure. Another way is to provide the material concrete itself with ductility. This can be done by adding steel (or other) fibers to the concrete mixture. Indeed the fracture toughness of the concrete is considerably enhanced: however further questions can be raised with regard to the influence of the production process of fiber reinforced structural members on the mechanical properties and the most appropriate method of determining the mechanical properties. An essential question is furthermore whether improving the fracture toughness of concrete by adding fibers by definition leads to an improved behaviour of the structural members.

1 INTRODUCTION

The introduction of the fictitious crack model by Hillerborg and his co-workers (1976), and Petersson (1981) has been a major step forward in understanding and modeling the behaviour of concrete structures. By providing tensile stress – crack opening relations further to stress - strain relations, it became possible to better describe the behaviour of structures, especially those which exhibit a brittle failure behaviour. The first time that this behaviour was described in a structural design code was in the CEB-FIP Model Code for Concrete Structures in 1990. Fig 1 shows the relation as given in the MC'90. The corresponding expressions for the crack widths w_1 and w_c are:

$$w_1 = 2\frac{G_F}{f_{ctm}} - 0.15 w_c \tag{1}$$

$$w_c = \alpha_F \frac{G_F}{f_{ctm}} \tag{2}$$

In the equations G_F is the fracture energy which is a function of the concrete compressive strength and the maximum aggregate size. The latter influence is justified since there is a relation between the crack band width and the maximum particle diameter. Van Mier (1996) showed, that within the band of microcracks "crack bridging" occurs: this phenomenon is the mechanism behind the residual stresses, transmitted across the crack faces during crack widening, Fig. 2.

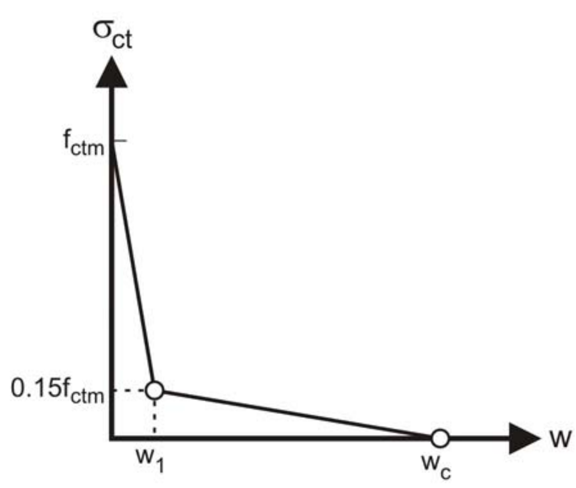

Figure 1. Stress-crack opening relation according to CEB-FIP Model Code 1990

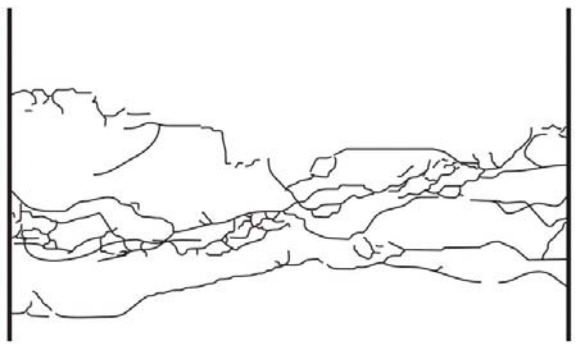

Figure 2. Crack band in a concrete with d_{max} = 16mm (van Mier, 1996).

Of similar significance is the failure behaviour of concrete in compression. Markeset (1993) assumed that compressive failure localizes in a damage zone of limited length. Her Compressive Damage Zone Model (CDZ-Model) takes into account the occurrence of longitudinal splitting cracks as well as a shear band and can be applied both to concentrically and eccentrically loaded concrete, Fig. 3.

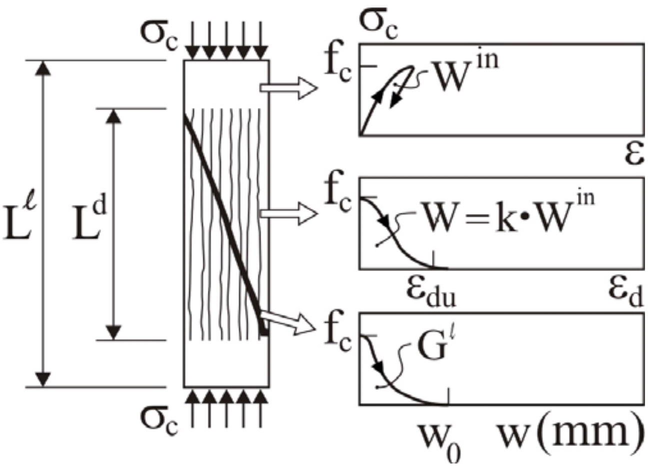

Figure 3. CDZ-model according to Markeset (1993) illustrated for a specimen loaded in concentric compression.

The damage zone length depends on the cross sectional dimensions of the specimen and the eccentricity of the load. The tensile fracture energy G_F is an important parameter in the model. The complete opening of a longitudinal crack is assumed to absorb the same amount of energy as the opening of a pure tensile crack.

Both in the case of tension and compression, the failure is mostly an integral part of a larger and more complex system.

In the following this will be illustrated for the cases of shear in slender and short members not reinforced in shear, slabs subjected to punching and the rotation capacity of statically indeterminate slabs.

2 FRACTURE MECHANICS CONSIDERATIONS FOR SLENDER AND SHORT CONCRETE MEMBERS FAILING IN SHEAR

2.1 *Slender shear critical beams*

The first time that a relation was found between the shear capacity of non shear reinforced concrete beams and crack propagation was in a research project at TU Delft (Walraven, 1978). Three series of three beams were tested in shear. The beams in any series differed in size. Fig. 4 shows the dimensions of the beams in series A. The heights of the beams were 150mm, 450mm and 750mm respectively. The reinforcement ratio was 0.8%. The beams contained no shear rein- forcement. The loads were applied at a distance 3d from the supports, in order to have a shear critical loading geometry. The tests were carried out in order to investigate the existence of a size effect in shear and to find an appropriate explana-

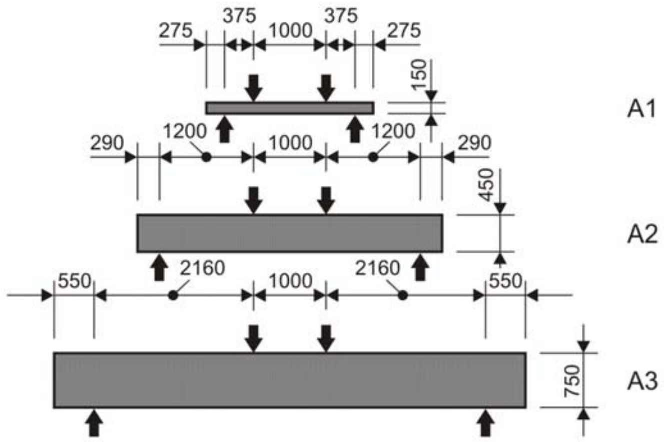

Figure 4. Size effect tests on slender beams, series A (Walraven, 1978).

tion. In those days researchers were convinced that the reason for the size effect was the size dependency of aggregate interlock in the cracks. They argued that the cracks in larger specimens would by wider for the same crack pattern. If the same concrete mixture would be used, with the same size of the aggregates, the effect of aggregate interlock in the cracks should consequently be smaller in the larger beams, which would explain the size effect. In order to verify this hypothesis, a series of beams was tested similar to those shown in Fig. 4, but now with lightweight concrete in stead of normal weight concrete. If the hypothesis of aggregate interlock would be valid, this would mean that no size effect would be observed in the lightweight concrete series, because the lightweight particles fracture at cracking.

A comparison of the nominal ultimate shear stress ($v_u = V_u/bd$) clearly showed, that also in the lightweight concrete beams a clear size effect occurred, Fig. 5.

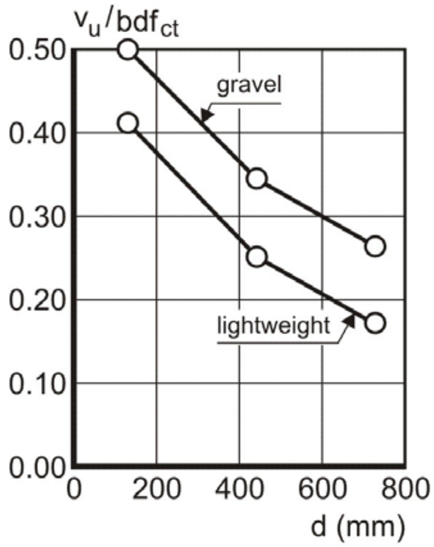

Figure 5. Nominal shear strengths as a function of the effective cross sectional depth, for gravel – and lightweight concrete (Walraven, 1978).

A comparison of the crack patterns at similar values of the nominal shear stress v = V/bd showed, that the progress of the cracks in the large specimens is significantly faster than in small specimens. This is shown in Fig. 6.

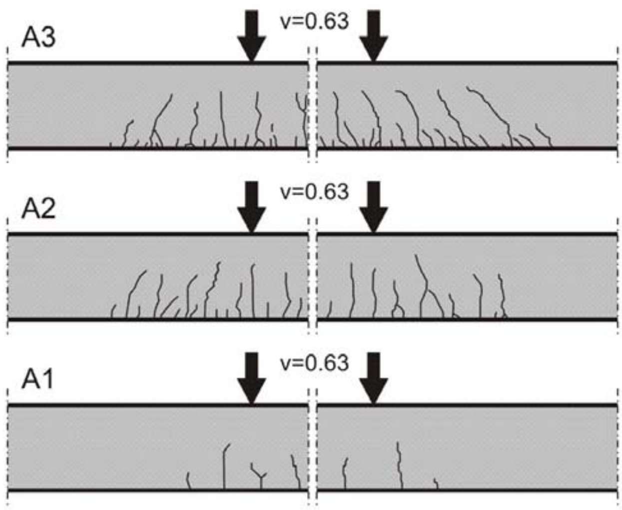

Figure 6. Crack pattern development in shear critical beams (a/d = 3), with cross sectional depths varying from 750 mm (beam A_3) to 150 mm (beam A_1), represented equally large, at the same nominal shear stress V/bd = 0.63 MPa.

In this figure the dimensions of the beams are graphically represented at the same size, in order to facilitate a good comparison of the state of development of the crack pattern. It is seen that the highest beam (A3, with h = 750mm) shows a crack pattern indicating that the beam is near to failure (which occurred at a nominal shear stress of 0.70 MPa). Failure of the beams always occurred abruptly, when one of the inclined bending cracks suddenly developed into an unstable shear crack. The failure was violent because no alternative bearing mode was available.

The bearing mechanism of the shear capacity in such type of beams is complex. In building codes with regard to the ultimate shear stress v_u normally three dominating influencing factors are distinguished, namely the concrete compressive strength f_c, the longitudinal reinforcement ratio $\rho_l = A_{sl}/bd$ and the effective member depth. E.g. in the new Eurocode for Concrete Structures the expression

$$v_d = C \cdot k\ (100\rho_l \cdot f_{ck})^{1/3} \tag{3}$$

is given, where ρ_l is the longitudinal reinforcement ratio as defined previously, f_{ck} is the characteristic cylinder compressive strength of the concrete, C is a coefficient (advisory value 0.12) and k is a size factor according to

$$k = 1 + \sqrt{200/d \le 2.0} \tag{4}$$

with the effective cross sectional depth d in mm.

König et al (1993) showed on the basis of fracture mechanics considerations, that it would be scientifically more correct to directly involve the fracture energy G_F and the concrete tensile strength f_{ct}. This can be done by introducing the characteristic length l_{ch}, which is defined as

$$l_{ch} = E \cdot G_F / f_{ct}^{\,2} \tag{5}$$

The mean ultimate nominal shear strength can then be formulated as:

$$v_u = C \cdot f_{ct} \cdot \sqrt[3]{(l_{ch}\rho_l / d)} \tag{6}$$

Such a formulation will prove its value in future, since many innovative concretes will be introduced, for which the compressive strength, the tensile strength and the fracture energy cannot be related anymore by straightforward expressions.

E.g, if a concrete gets artificial ductility by steel fibres, a formula of the type given by Eq. 5/6 may be a better basis than that of Eq. 4.

A very interesting new area for which well based expertise in the field of shear is asked, is the determination of the shear capacity of existing concrete bridges. Recently in The Netherlands investigations on existing bridges were carried out, with the aim to determine the actual shear bearing capacity. This was necessary because the Dutch government had decided to extend a large number of existing highroads with an additional lane. This includes, that also the bridges have to be extended with an additional lane, which raises the question of the actual bearing capacity. Tests on drilled cylinders showed, that the centric tensile strength of the concrete was unexpectedly low. For a compressive cylinder strength of 60-65 MPa a centric tensile strength in the order of only 1-1.5 MPa was found. It is clear that here new considerations with regard to the shear capacity have to be developed. Fig. 7 shows a shear test on a strip, sawn out of an existing slab bridge, investigated in the laboratory of TU Delft.

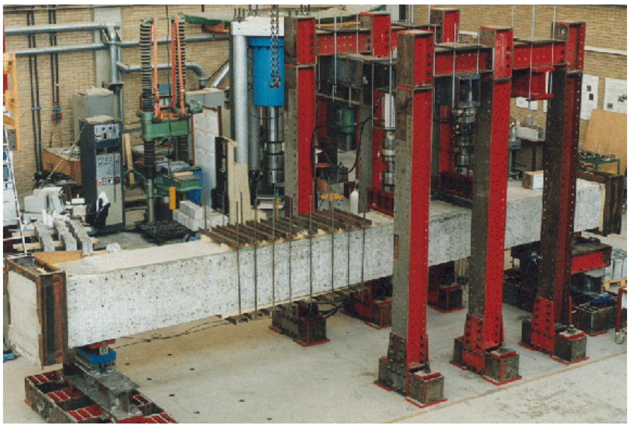

Figure 7. Shear test on strip, sawn out of an existing slab viaduct, with an unusual relation of concrete tensile to concrete compressive strength.

2.2 *Short deep beams loaded in shear*

Detailing of concrete structures has always been a point of concern, because damage to concrete structures could often be traced back to insufficient knowledge of the designing engineer with regard to appropriate detailing. Especially the work of Schlaich and Schäfer (1987), developing strut and tie models for detailing regions, contributed to the development of rational detailing models. The principle of such models is the following:

- orient the compression struts to the compressive stress trajectories in the uncracked loading stage
- apply tensile ties to obtain an equilibrium system, considering that the shortest tensile ties offer the best solution, since they guarantee the smallest deformation and the lowest degree of redistribution of forces
- connect the compression struts and the tensile ties by appropriate nodes, striving at confining the concrete at the nodes.

Fig. 8 gives and example of the application of the strut and tie model to a short deep beam with a load at midspan. The model shown is, in terms of the theory of plasticity, a lower bound solution if the allowable stress in the components is nowhere exceeded. The maximum allowable stress in the concrete strut is mostly expressed by the relation

$$\sigma_{cu} = \nu \cdot f'_c \qquad (7)$$

where ν is the so-called effectivity factor, taking into account the effects of for instance an unequal stress distribution in the cross section of the compressive strut ($\nu < 1$). However, although this representation is very transparent at first sight, it is a simplification of a mechanism which is considerably more complex in reality. A shortcoming which is detected quickly is that the shear bearing capacity, if the longitudinal reinforcement does not yield, is equal to

$$V_u = \nu \cdot f'_c bk \qquad (8)$$

where k is the length of the bearing plate and b is the width of the specimen. So, the bearing capacity is independent of the angle θ. This is contrary to all experimental findings, which confirm a strong dependence on the angle θ (or the ratio a/d). An important question is further whether in such a case a size effect occurs or not. The simplified model, shown in Fig. 8, suggests that this is not to be expected. In order to investigate this question a series

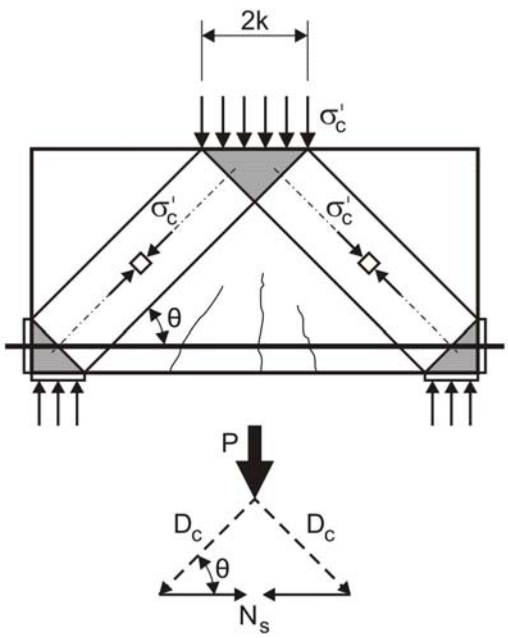

Figure 8. Strut and tie model for short deep beam loaded by a concentrated load at mid-span.

of 5 tests was carried out on short deep beams with different dimensions (Lehwalter, 1988). The specimens were geometrically similar, but differed in size, where the cross sectional depth varied from a lowest value of 200 mm to a highest value of 1000mm, Fig. 9.

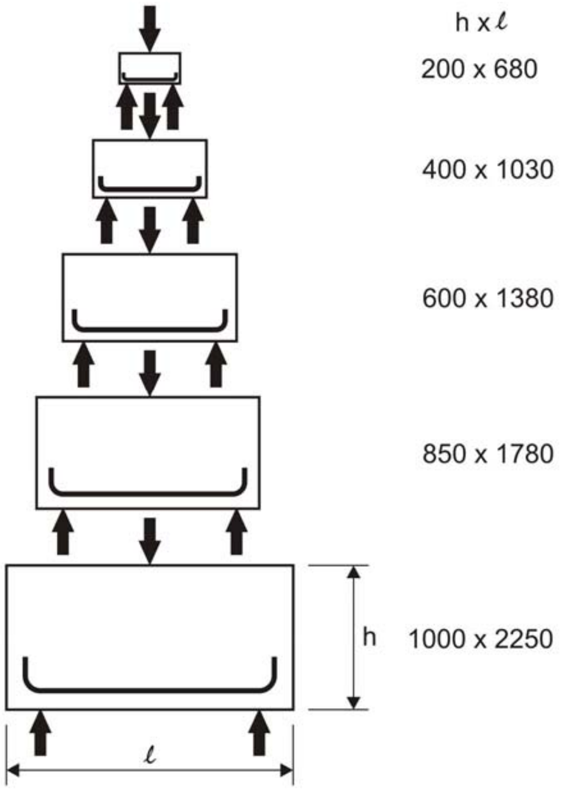

Figure 9. Shear tests on small beams with the same geometry but different size, Lehwalter, 1988.

The concrete strength was about 20 MPa and the longitudinal reinforcement ratio ρ_l was in all cases about 1.1%, which means that they were all overreinforced in bending. The behaviour was remarkable in a number of respects. Like in the case of slender beams, the progress of the cracks in the first stage of loading strongly depended on the size of the beam. In the largest specimens the development of the crack pattern was much faster than in the smaller specimens, in analogy with the slender beams (Fig. 6). However, in this case the occurrence of an inclined shear crack did not lead to failure, contrary to the slender beams. The explanation to this is that a redistribution of forces is possible to another bearing mode, in a qualitative sense resembling the model shown in Fig. 8. Fig. 10 shows that there is still a significant reserve capacity after inclined cracking. The figure shows that, with regard to the bearing capacity, a significant size effect exists, in spite of the different bearing mode. With regards to crack propagation and failure two further observations are done, see fig 11:

- The inclined crack intersects the compression strut near to the loaded area
- Failure occurs at the upper part of the compression strut, adjacent to the node area.

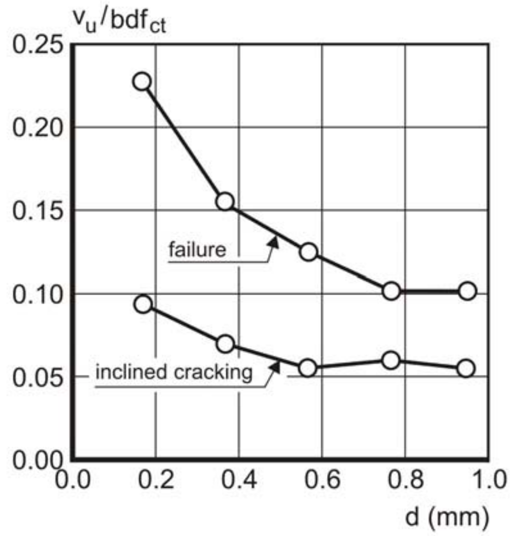

Figure 10. Relative shear stress at inclined cracking and at failure for short deep beams with a/d = 1.

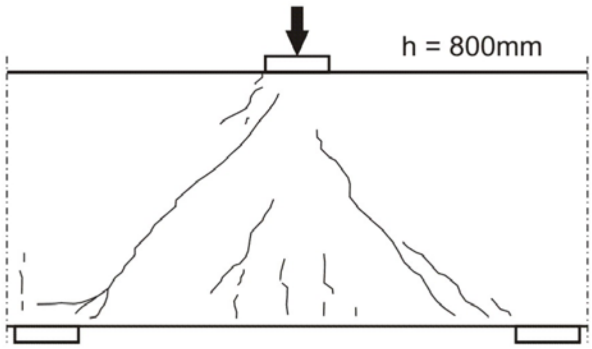

Figure 11. Failure of compression strut at the junction with the loading area.

This suggests that a static system with hinges, like shown at the bottom of fig. 8 is actually not a good description of the real behaviour.

In order to refine the description of the behaviour Asin (2000), introduced a damage localization zone. Recalculating Lehwalter's tests he found that a reasonable agreement could be obtained by introducing a localization zone with a length s = 200 mm and a failure strain of $4{\cdot}10^{-3}$ at the compressed side of the localization area. The localization area is subjected to a combination of normal force and bending, due to the fixed node between the struts just below the load introduction area. With this relatively simple model, which was not worked out further, it was shown that a reasonable description of the behaviour is obtained, including the size effect,. It would be worthwhile to see if an improved formulation, for instance based on Markeset's CDZ-model, would further improve the agreement.

An interesting observation is that, during redistribution from the one bearing mode (beam action, before inclined shear cracking) to the second (final) bearing mode (strut and tie model), the compression struts are first damaged by the inclined crack, progressing through the strut, and finally loaded in an unfavourable way (non-uniform loading, due to bending at the top). Considering these observations, it seems that the redistribution from the one bearing mode to the other goes along with damage of the final system. Beeby (2000) carried out a very interesting series of experiments, in order to verify this. He compared two types of beams. The reference type of beam is a traditional beam, in which a process of redistribution, as sketched before, can occur. The second type of beam was made in the same mould and had the same reinforcement but much of the tension zone was blocked out with polystyrene. The objective was to induce the beam to behave as a truss rather than as a beam. Fig. 12 shows the beams, which had a compressive strength of around 50 MPa. The longitudinal reinforcement ratio was 0.87% and the a/d ratio was 2.5. The regular beams A and B failed in shear at loads of 135 and 123 kN respectively. Beam C, where a triangular part of the concrete was removed, failed at a load of 202 kN. So, the removal of a part of the concrete generated an increase of the bearing capacity of about 60%. The failure of beam C was not shear but, finally, a bond failure over one support. Nevertheless, the beam was carrying a load at failure in excess of the

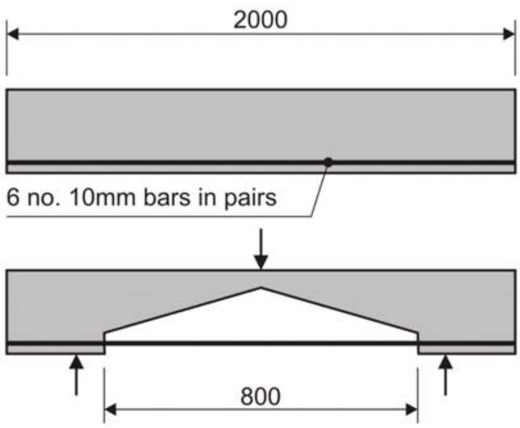

Figure 12. Test beams according to Beeby (2000), showing that removal of a part of the concrete can cause a considerable increase of the bearing capacity.

calculated flexural strength and so the reinforcement was likely to have started to yield. The reason of the much better behaviour of beam C is that, by adapting its shape, the member is much better conditioned to act as a strut and tie system.

2.3 *Punching shear and fracture mechanics*

Punching shear is a complicated phenomenon. This explains that still PhD-theses are written, further exploring the mechanism of sudden and violent failure. The only models that describes the behaviour from the formation of radial and tangential cracks to the final failure of the concrete around the column was developed by Kinnunen et al. (1960). With this model Kinnunen succeeded to model the role of nearly all parameters, such as the radial and tangential reinforcing ratio, the punching shear reinforcing ratio, the concrete strength and all geometrical effects. Unfortunately the model did not include any size effect. The model was based on the development of a kinematic mechanism, shown in Fig. 13. The only weakness was the failure criterion: the structure was supposed to fail when the tangential strain ε_{cT} in the bottom of the slab around the column, reached a limit value, determined experimentally.

For $B/d \leq 2$

$$\varepsilon_{cT,} = 0.0035(1-0.22\ B/d) \tag{9}$$

and for $B/d > 2$ as

$$\varepsilon_{cT,} = 0.0019 \tag{10}$$

where B is the diameter of the cross section of a circular column and d is the effective slab depth.

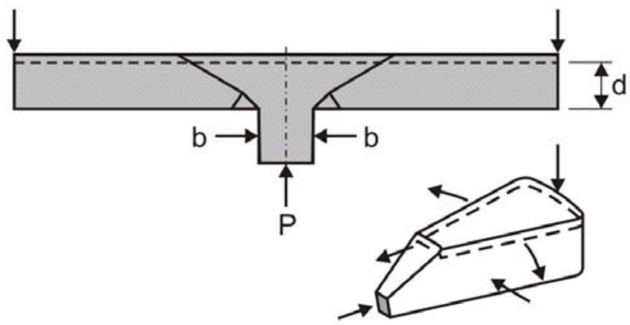

Figure 13. Mechanical model by Kinnunen and Nylander (1960) decribing punching shear.

36 Years later Hallgren (1996) found the key to the complete description of the phenomenon, completing Kinnunen's model with a fracture mechanics component, for which he based himself on the Multifractal Scaling Law by Carpinteri and Chiaia, (1995). The structural size, which is the main variable of the law, was set equal to x, the depth of the compression zone. The Multifractal Scaling Law gives:

$$G_F = G_F^{\infty}(1+\frac{\alpha_F \cdot d_a}{x})^{-1/2}$$

where G_F^∞ is the fracture energy for an infinitely large structural size, d_a is the maximum aggregate size and α_F is an empirical factor, which was determined to be 13. A FEM analysis showed that the radial length of the zone with tensile strains at about r = B/2 + y is approximately equal to the compression zone depth x (y is the length of the truncated wedge in the model of Kinnunen). After an extensive FEM analysis a modified criterion for the ultimate tangential strain was formulated, according to:

$$\varepsilon_{cTu} = \frac{3.6 \cdot G_F^\infty}{x \cdot f_{ct}} (1 + \frac{13 \cdot d_a}{x})^{-1/2}$$

where x is the depth of the compression zone and f_{ct} = concrete tensile strength. With this formulation the size effect could be well described, which made the model complete.

2.4 *Rotation capacity and fracture mechanics*

The rotation capacity of plastic hinges in reinforced concrete slabs is a tool that allows the designer to make optimum use of the potential of redistribution of bending moments by virtue of the yielding capacity of reinforcing steel. In slabs with low reinforcement ratios, fracture of the steel limits the rotation capacity. For higher reinforcement ratio's the rotation capacity is reached by crushing of the concrete in the compression

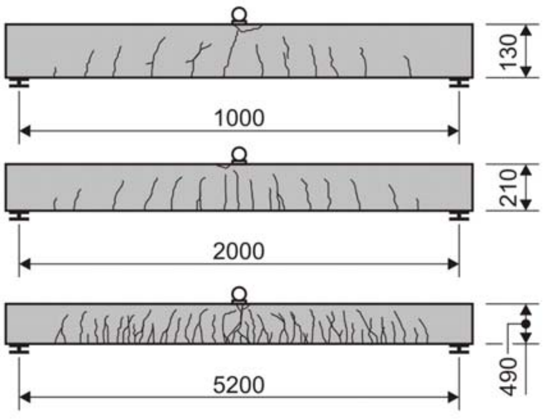

Figure 14. Crack patterns for rotational capacity tests on beams with different depths and a reinforcement ratio of 1.12%, after Bigaj, 1999.

zone of the element. Considering the effect of localization in compression, see also Fig. 2, Hillerborg, 1989, supposed that the rotational capacity of plastic hinges would be size dependent in the latter case. In order to investigate this phenomenon, a series of tests was carried out in order to investigate this hypothesis, Bigaj (1999). Tests were carried out for two longitudinal reinforcement ratio's: 0.28% and 1.12%. For every reinforcement ratio a series of three beams was tested. The beams were geometrically similar but of different size. The cross sectional depths were 90mm, 180mm and 450mm respectively. Fig. 14 shows the final crack patterns for the beams with a reinforcement ratio of 1.12%. The plastic rotation as observed in the tests is shown in Fig. 15.

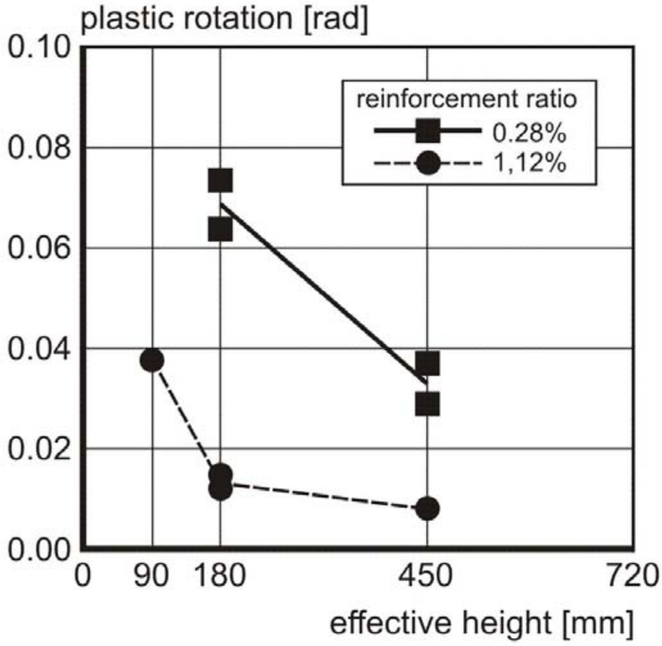

Figure 15. Observed plastic rotations at peak loadversus effective depth of the beams (Bigaj, 1999).

The analysis of the results showed, that the composition of the longitudinal reinforcement is even more important than the effect of localization of damage in the compression area. The tensile reinforcement is hard to correctly scale, because it is not possible to proportionally increase both the bar diameter and the bar distances at the same time. The bond properties of the bars determine the crack distance and as such the number of cracks that is involved in plastic deformation. Furthermore the bar distances and the concrete cover play a role, because of the occurrence of eventual splitting cracks around the reinforcing bars and their influence on the yielding length of the bar. Moreover the type of steel (f_u/f_y and ε_u) plays a role. In order to fundamentally investigate the behaviour of plastic hinges, at first an extensive study was carried out with regard to the bond of reinforcing bars, including the yielding stage of the steel. This resulted in complete bond-slip relations, including the effect of steel strain and magnitude of the concrete cover, for low, medium and high strength concrete. The relations observed were implemented in a plastic hinge model, where for the compression zone Markeset's CDZ-model was used. A substantial number of simulations was carried out. Fig. 16 shows one of them. The steel properties were kept constant (steel A: f_y = 550

MPa, f_{max} = 594 MPa, ε_u = 5%), whereas the layout of the tensile tie was varied:

- *Layout I*: constant bar diameter d_s = 16mm, bar spacing adjusted to the actual member size and to the required reinforcement ratio
- *Layout II*: constant bar spacing s = 50mm, bar diameter adjusted to the actual member size and to the required reinforcement ratio

Fig 16, simulating the behaviour for a reinforcement ratio of 0.25% and the two reinforcement layouts, shows that the layout determines whether there is a size effect or not. The figure shows for any combination a band, rather than a line. This is to take account of the probabilistic aspect of crack formation: the number of cracks contributing to the rotation capacity depends on the accidental position of the cracks, adjacent to the load. Whether one or two cracks contribute means a significant difference.

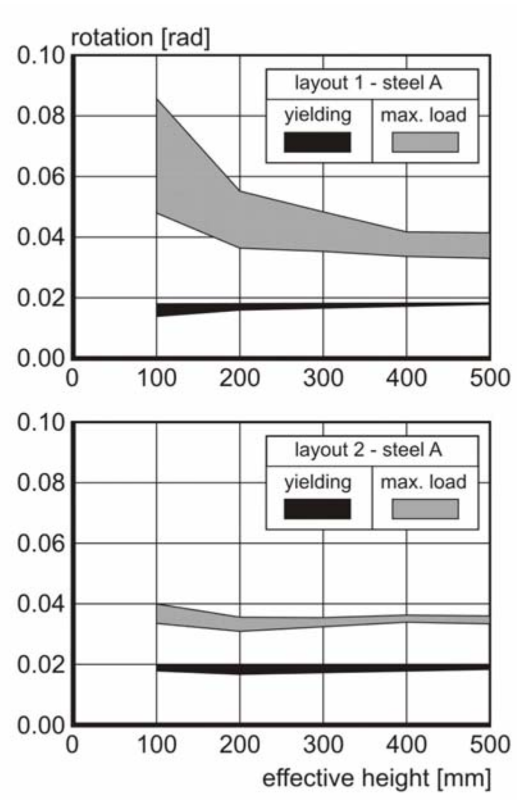

Figure 16. Calculated rotation at the onset of yielding of the reinforcement and at maximum load as a function of the effective member depth and the reinforcement layout, for a longitudinal reinforcement ratio of 0,25%, after Bigaj (1999).

3 INCREASING THE DUCTILITY OF CONCRETE BY ADDING STEEL FIBRES: ACHIEVEMENTS AND ANOMALIES

3.1 *Optimizing the properties of fiber concrete by defined performance mix design*

The brittleness of concrete and its relatively low tensile strength have always required special design skill in order to create non-brittle structures. Providing fibers to concrete makes the material much more ductile. Looking back to three decades of development in fiber concrete, it can be concluded that the major steps in the development have been made during the last few years. At first it has finally been understood that a fibre concrete should be "designed" in order to get appropriate properties. Whereas in the past fibers were simply added to conventional concrete mixtures, without further considerations, now it has been understood that fibers and the aggregate particle skeleton interact and that an optimum combination of those components has to be found in order to get the most appropriate mixtures. Fig. 17 shows an experiment carried out by Grünewald (2004). He mixed fibres with combinations of aggregate particles, consisting of different volumes of sand and gravel (no other components like cement or water were added). He then vibrated this mixture until the highest packing density was obtained. Fig. 17 shows the maximum packing density values, obtained for volumes of 1.5 Vol.% of different steel fibres, as a function of the ratio sand to gravel. It can be seen that by adding fibers, keeping the sand to total aggregate ratio constant, the packing density decreases. This means basically, that the fibers disturb the aggregate skeleton. The maximum packing density moves to higher values of the ratio sand/total aggregate. This shows that, to compensate for the effect of the fibers, the mixture composition has to be adjusted by increasing the content of grains that are relatively small compared with the fiber length (cement, fillers or small aggregate grains).

Grünewald showed that, based on such type of considerations, high performance self compacting fiber reinforced concrete mixtures with fiber volumes over 125 kg/m^3 are possible.

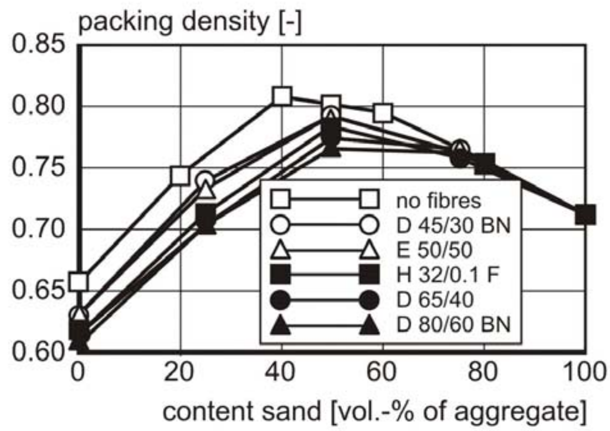

Figure 17. Effect of the sand content and the type of steel fibres (at 1.5 Vol.%) on the packing density (fibre types: first index: L_f/d_f: second index: L_f), according to Grünewald, 2004.

Another interesting technology is that of combining short and long fibers in the same concrete mixture. Markovic (2006) showed the potential of this method. The short fibers are activated as soon as microcracks occur, so that the behaviour remains quasi elastic until the formation of macrocracks. Then the long fibres take over. Fig. 18 shows, that flexural tensile strengths of 40 MPa and more can be achieved by appropriate combinations of fibers. The figure shows moreover, that by a suitable choice of the fibers optimization is well possible: with 1 Vol.% of fibers 13mm and 1 Vol.% of fibers 40mm, about the same flexural tensile strength is obtained as with 4 Vol. % of fibers 6mm in combination with 1 Vol.% of fibers 40mm.

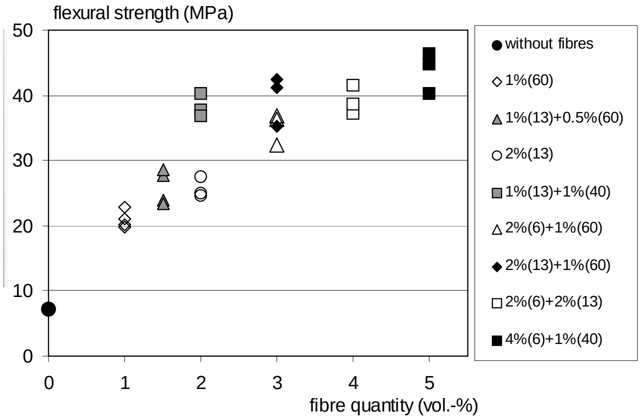

Figure 18. Flexural tensile strengths of various types of hybrid fiber concrete, Markovic, 2006.

3.2 *Measuring the mechanical properties of high performance fiber concrete*

A distinct advantage of those new, high performance fiber concretes is that they show a pronounced hardening behaviour, which makes them to an appropriate material for structural design.

Any material that is used for structural design purposes should be qualified in terms of mechanical behaviour. The question which is the most appropriate test, has been a subject of intensive discussions. RILEM defined a standard tests, by which the load deflection relation is established on a specimen like shown in Fig. 19.

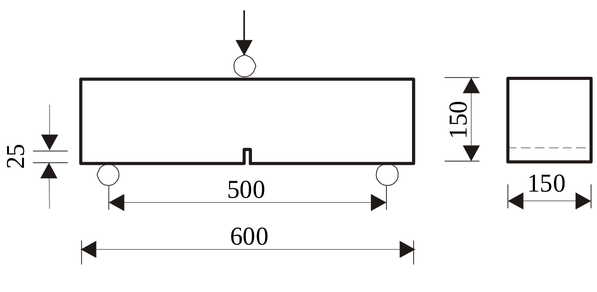

Figure 19. Rilem standard test for fiber concrete.

This load deflection relation is the basis for the stress – crack opening relation. Since it is known that the result of the RILEM bending test is very sensitive to the production of the specimen and the execution of the test, recommendations are given on how to fill the mould and to compact the specimen. Filling the mould is done in a certain, well described sequence: at first a batch of concrete is placed in the middle of the mould, and subsequently at the two ends. After filling the mould in this way, the concrete is compacted with a prescribed intensity and time. However, modern high performance fiber concretes have, as an advantage, mostly a very good workability. Many of them are even self compacting. This means that filling the mould in the RILEM way is not an option.

Grünewald, 2004, showed that considerable differences are obtained between two mixtures C55/67, which both contained 60 kg/m^3 hooked end steel fibers Dramix 80/60 BP. One was a traditional mixture, which allowed filling of the mould in the RILEM way, and one was self compacting. In the last case the concrete was poured into the mould from one side. Fig. 20 shows the load deflection relations obtained: the difference is remarkable. The bending capacity of the self compacting concrete is about twice as large as that of the conventional fiber concrete with the same compressive strength. Moreover the scatter in the results is smaller. The analysis showed that there are two reasons for the better behaviour. The pull-out resistance of the fibers in the self compacting concrete is better, because of the better embedment of the fibers in the matrix. Furthermore the orientation of the fibers in the self compacting concrete was more advantageous, as a result of the flow of the concrete from the one end of the mould to the other. This shows that care should be taken to arrive at test methods which give representative results for design, and design rules that take account of the effects of fiber orientation at the production plant or at the site.

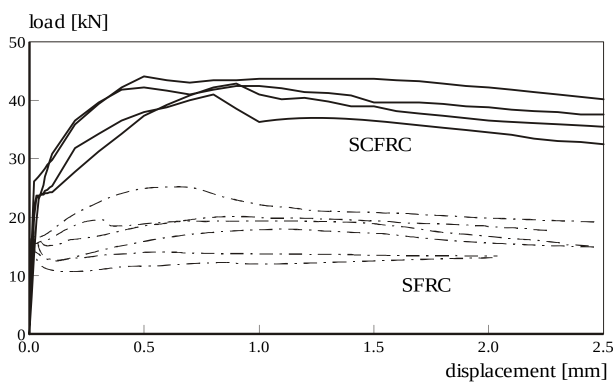

Figure 20. Test results of three point bending tests on self compacting concrete and conventional concrete, with the same concrete strength and the same amount of fibers, according to Grünewald, 2004.

On the other hand, it should be noted that the same considerations that apply for test specimens are valid for structural elements as well. An interesting experiment was done with self compacting fiber concrete. A self compacting fiber mixture was used to cast a tunnel lining element. This curved element was cast through a window at the centre of the curved side. The concrete was delivered from a truck mixer: it flew through a half pipe into the mould for the tunnel element, were it spread until the whole mould was filled. After hardening cylinders were drilled at various locations and in various directions. From those cylinders discs were sawn, which were subjected to X-ray photography.

Fig. 21 shows clearly that there is a strong orientation, due to the flow of the concrete. The fiber efficiency varied between the extreme values 0.24 and 0.91. So, the fiber efficiency may vary substantially throughout the specimen. Up to now this has not been considered in building codes. Therefore general provisions should be formulated to take account of this phenomenon. On the other hand profit should be taken from the possibility to orient fibers intentionally.

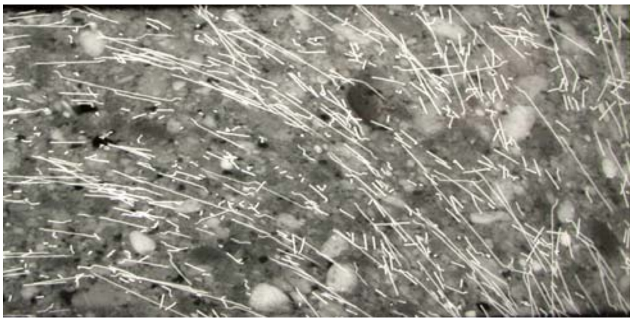

Figure 21. X-Ray photo of fiber orientation in tunnel element, cast with self compacting fiber concrete

3.3 *Combining steel fibers with traditional reinforcement*

An interesting possibility for structural design is to combine fibers with traditional reinforcement. In this way very thin walled structures can be obtained. The rebars take care of the main bearing function, whereas the fibers give the concrete a large ductility, which is favourable in any region where stress concentrations can be expected (anchorage regions, regions with lapped bars, impact loading, vibrations). Moreover the fibers contribute to an advantageous cracking behaviour, with fine cracks at small distances. An important question is still which is the best ratio of fibers to classic steel. In order to answer this question at TU Delft a research program has been started in which concentric tensile tests on prismatic concrete bars, reinforced with different ratio's of classic steel and fibers, are carried out. The concrete types investigated had compressive strengths of 130 and 180 MPa respectively. The fiber content was 0 Vol.%, 0.8 Vol.% and 1.6 Vol.%, which corresponds to 0, 60 and 120 kg/m^3 steel fibers. Fig. 22 shows the cracking patterns that were obtained. The role of the fibers on the cracking pattern is clearly recognizable.

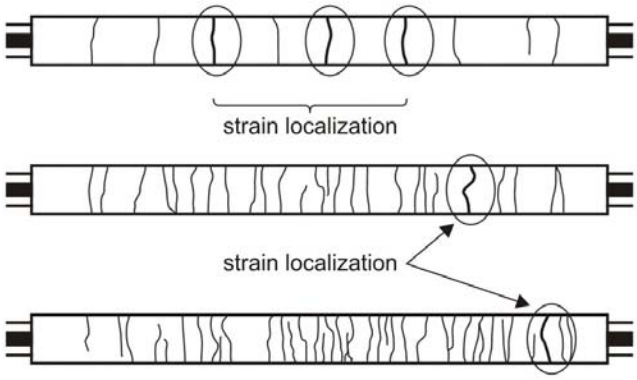

Figure 22. Crack patterns in centrically loaded reinforced tensile bars. From top to bottom 0, 0.8 and 1.6 Vol.% short steel fibers: (Shionaga, 2006)

However, also another interesting phenomenon is observed: if fibers are added, the plastic deformation localizes finally in one single crack, whereas the specimen without fibers shows localization in several cracks. This may have consequences for cases where design for ductility is concerned, as shown in the following case.

3.4 *Means adding fibers to a concrete always an improvement of the structural behaviour?*

It was argued already before, that just adding fibers to a concrete does not necessarily mean an improvement of the material properties. A good design of the material should make sure that fibers indeed generate improved properties in strength and ductility. However the question can be raised if a well designed fiber concrete by definition improves the structural behaviour. In order to answer this question the phenomenon of rotation capacity is treated again, now in combination with fibers.

In Fig. 22 it was shown that the combination fibers and conventional reinforcement leads to strain localization in a single crack. The reason for this is that there is a certain scatter, both in the homogeneity of fiber distribution and in fiber orientation. So, if cracks have been formed, the capacity of transmission of forces across the crack varies from crack to crack, because of the scatter of the fiber capacity in the subsequent sections. This explains the localization in one crack after yielding of the reinforcing bars. A similar observation was done in recent research on the rotation capacity of reinforced concrete hinges, with and without fibres (Schumacher, 2006), Fig. 23 shows the difference between concrete with and without fibers.

Figure 23. Plastic hinge in plain concrete (top) and fiber reinforced concrete (bottom) (Schumacher, 2006).

The figure shows for the plain concrete a more brittle behaviour of the compression zone, but a localization of steel yielding in more cracks. The fiber reinforced concrete shows a more ductile compression zone but localization of bar yielding in one crack. The rotation capacity of fiber reinforced concrete was finally smaller than that of plain concrete.

4 CONCLUSIONS

1. Fracture mechanics still learns us a lot with regard to material and structural behaviour.
2. Initiatives should be taken to implement fracture mechanics parameters in structural design recommendations. This will be useful for the design of structures with defined performance materials in future.

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