EXPERIENCES FROM 10-YEAR FIELD EXPOSURE OF CRACKED STEEL FIBRE REINFORCED SHOTCRETE

ERFAHRUNGEN MIT GERISSENEM STAHLFASER - SPRITZBETON NACH 10 JAHREN BEWETTERUNG

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Ongoing research at Vattenfall has attempted to define the active mechanisms for initiation and propagation of steel fibre corrosion in cracked shotcrete and to determine the effect on load-bearing capacity. After 10 years of exposure corrosion is ongoing at exposure sites where thaw salts is used. An average loss of approximately 20 % of the fibre diameter is detected in the outer 25 mm of a crack with a width of 1 mm. At small deformations in bending test, the residual strength showed an initial increase explained by continued hydration, but at larger deformations (2 mm) the effect of corroded fibres instead gives a reduction of the ductility. This is most obvious for longer fibres that corrode more.

Bei Vattenfall laufen Untersuchungen um den Mechanismus der Entstehung von Korrosion und des Korrosionsfortschrittes in gerissenem Stahlfaser-Spritzbeton zu erklären und die Auswirkung auf das Tragvermögen zu bestimmen. Anforderungen an die Dauerhaftigkeit von Spritzbeton werden in schwedischen Regelwerken für einen Zeitraum von über 100 Jahren gestellt. Ein System aus Ankern und Faserverstärktem Spritzbeton für die Sicherung von Tunnels in standfestem Gebirge hängt stark von der dauerhaften Erhaltung der Restzugfestigkeit des gerissenen Betons ab. Daher ist es wichtig Kriterien für die Beurteilung der Korrosion in den Rissen aufzustellen. Dabei interessiert auch die Entwicklung der Tragfähigkeit, wenn die Korrosion fortschreitet. Mit diesen Kenntnissen wird eine Abschätzung der Lebensdauer des Stahlfaserspritzbetons möglich. Gerissene Stahlfaserspritzbetonbalken wurden dazu über 10 Jahre in drei verschiedenen Plätzen ausgelagert. Die Rissweiten, die Faserlänge, die Mischungsrezeptur, der Beschleuniger und das Auftragsverfahren (Trocken-/ Nassspritzverfahren) waren die Versuchsparameter. Die Ergebnisse nach diesen 10 Jahren sind Gegenstand dieses Beitrags.

Nach 10 Jahren Auslagerung an Orten, wo Salze eingesetzt werden, sind folgende Korrosionsraten feststellbar: Bei Rissen mit 1 mm Rissweite ist in den obersten 25 mm der Risse ein durchschnittlicher Querschnittsverlust der Fasern von 20 % des Faserdurchmessers feststellbar. Im Biegeversuch steigt bei kleiner Durchbiegung die Resttragfähigkeit infolge fortschreitender Zementhydratation an. Bei größeren Verformungen (2 mm) verursacht aber die Faserkorrosion eine Reduktion der Duktilität. Dieser Effekt ist besonders bei langen Fasern deutlich ausgeprägt, da diese mehr korrodieren.

1. Introduction

Durability against corrosion of steel fibre reinforcement has been proved to be good, especially in concrete without cracks [1]. However, results from research on cracked concrete are limited, especially in non-marine environments. It has been stated that steel fibres are more resistant under conditions in which conventional reinforcement exhibits extensive corrosion [2]. No full explanation of the active mechanisms exists. Other studies in the area are performed by e.g. *Schiessl & Weydert* [3].

Research at Vattenfall tries to define the active mechanisms for initiation and propagation of steel fibre corrosion in cracked concrete. Another objective is to determine the effect on load-bearing capacity, assuming corrosion is ongoing. This knowledge will make it possible to estimate the service life of Steel Fibre Reinforced Shotcrete SFRS.

In the following, results from ongoing field exposure tests after 10 years of exposure are presented. A more detailed description of the project and the results after 5 years can be found in the author's doctoral thesis [4].

2. Test program

The field exposure tests with cracked SFRS samples were started in September 1997 and evaluations of results have been made after 1, 2.5, 5 and 10 years. All evaluations were performed during autumn except the one at 2.5 years, which were performed during wintertime.

The purpose with the evaluations is to examine the status of samples with steel fibres crossing cracks after different time of exposure. Another parameter to study is the change of load bearing capacity and the chloride ingress (when applicable) in the exposed SFRS samples.

2.1 Mix design

Four different concrete mix-types are used. The wet-mix shotcrete with 30 mm fibres and sodium silicate accelerator (WA30) is the main mix used in all combinations of exposure type and crack widths. All mixes and the abbreviations are presented in Table 1.

Tab. 1: Mix types used in the field exposure tests

	Wet-mix	Dry-mix	Accelerator	Dramix 30/0.5[1]	Dramix 40/0.5[1]
WA30[2]	x	-	x	x	-
W30	x	-	-	x	-
WA40	x	-	x	-	x
D30	-	x	-	x	-

[1] The nomenclature at the time was *fibre length/diameter* in mm:s

[2] Notations: W= Wet mix, D= Dry mix, A= accelerator, 30 & 40= fibre length (mm)

The cement type used in all mixes is a Swedish Portland low heat and low alkali cement called Degerhamn Std P (CEM I 42.5 N BV/SR/LA). The cement type is commonly used for infrastructural purposes. Data of the concrete mixes are given in Table 2.

A sodium silicate based accelerator was used in production of mixes WA30 and WA40. For the time of producing the samples and previously this was the most common type of accelerator in Sweden.

Tab. 2: Concrete mix design used in the field exposure tests

		WA30	WA40	W30	D30
w/c		0.42	0.42	0.42	0.3[1]
Cement	(kg/m^3)	510	510	510	500
Aggregate 0-8 mm	(kg/m^3)	1202	1202	1202	815
Aggregate 4-8 mm	(kg/m^3)	-	-	-	286
Aggregate 2-5 mm	(kg/m^3)	-	-	-	260
Aggregate 0-1 mm	(kg/m^3)	298	298	298	138
Plasticiser (melamine)	(%/kg C)	1.4	1.4	1.4	-
Accelerator	(%/kg C)	3.5	3.5	-	-
Fibre	(kg/m^3)	70	70	70	65

1) Approximated by measuring the amount of water required during spraying

2.2 Manufacturing of samples

Firstly the concrete was shot as large slabs (2*1.2*0.15 m). The purpose with shooting the large slabs was to reduce the amount of rebound and receive a more homogenous composition. Totally 11 large slabs were shot. After storing the slabs 28-50 days with constant watering, more than 300 beams were sawn with the dimension 75*125*500 mm.

After approximately 56 days the beams were loaded in bending to obtain the desired crack width. Flexural cracking was thus obtained by performing a four-point load set up. Mainly the flexural test was in accordance with the ASTM C1018 test, however with larger beam dimensions and higher rate of deflection (0.25 mm/min).

2.3 Exposure environment

The two major climatic factors ruling the rate of corrosion are relative humidity and presence of chlorides. The choice of exposure environment had to be made carefully to make the field exposure tests relevant to actual situations, where steel fibre reinforced shotcrete is commonly used. In the field exposure tests three different sites were thus chosen together with the reference site in the laboratory (20°C and 65% RH):

- Rv40: National Road 40, Borås - outdoors along motor highway.
- DAL: The Dalälven river, Älvkarleby – outdoors, specimen partly immersed.
- EUG: Eugenia tunnel, Stockholm - road tunnel (79.500 vehicles/day).

Details and examples of the situation are shown in Table 3. A complete sketch of the test program and all tested parameters can be seen in figure 1.

Tab. 3: Concrete mix design used in the field exposure tests

Location	Type of exposure	Representing structure type
Eugenia tunnel, Stockholm	Humid Chlorides Sheltered from rain Acidifying gases	Rock strengthening in tunnels
Main road Rv40, Borås	Humid Chlorides (direct splashing) Rain	Rock strengthening of open cuts Concrete repairs
Dalälven river, Älvkarleby	Humid Rain	Intake channel Intake tunnel

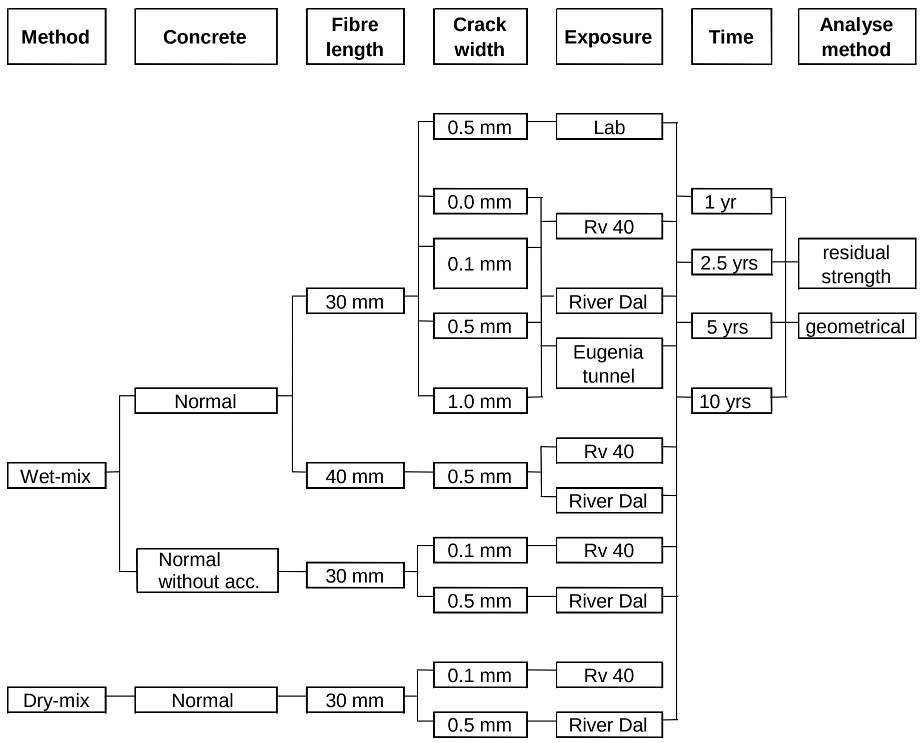

Figure 1: Field exposure test program

3. Evaluation after exposure

In the following, the techniques for evaluation are described. In the first evaluations only two beams from each site were collected. One of the beams was used to measure the residual strength after exposure and the chloride contents. The other beam was used for evaluation of the corrosion on single fibres. For sites, where ongoing corrosion could be stated, the number of beams was doubled at the next evaluation.

3.1 Residual strength

Steel fibres are used in the design process e.g. for permanent linings in underground construction to achieve sufficient post-crack behaviour of the shotcrete. The function of a system with steel fibre reinforced shotcrete combined with bolts is fully dependent on sufficient residual strength. Therefore, the influence of corrosion on the long-term residual strength is of great interest and thus studied here.

The pullout resistance created via interaction between fibre and concrete matrix give ductility in concrete containing steel fibres. If corrosion has been initiated, the fibre cross-section decreases locally in the crack region. This could give a change from a ductile pullout behavior of the fibre from the concrete matrix to a brittle fibre failure, when the fibre diameter is too small.

By comparing the residual strength levels at initial flexural cracking with the ones after different time of exposure, this effect can be investigated. By re-loading the beams with flexural load up to 5 mm deflection after exposure, it has been possible to analyze the change in residual strength. The residual strength levels achieved in the re-load are compared with the levels of the initial flexural load test that was interrupted before exposure.

3.2 Removal of fibres

To check if corrosion has been initiated (or possibly the degree of corrosion on fibres crossing cracks), the fibres must be removed from the concrete matrix. Crushing of the concrete plates implies a high risk for steel fibre destruction and ruined possibilities for evaluation. This is valid especially for fibres showing corrosion to a great extent. Small plates were sawn at different distances from the crack mouth in the beams not subjected to continued flexural load (see Figure 2).

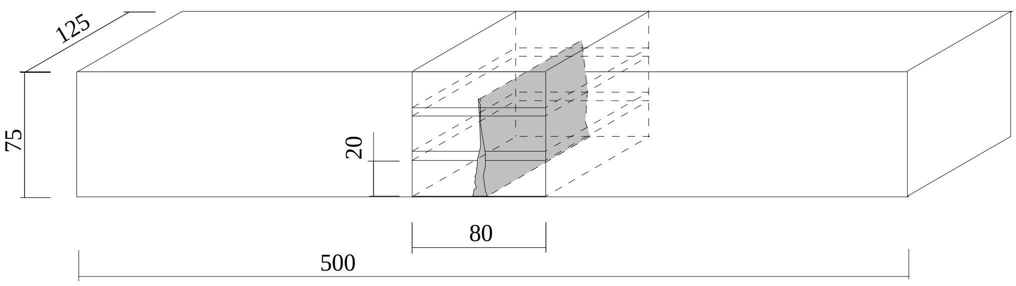

Figure 2: Dismembering of beams (measurements in mm)

To minimize the risk for fibre destruction the plates where subjected to repeated freezing and thawing. To secure a concrete destruction during freezing and thawing the plates where first dried out completely for 24 h in 200 °C. After drying they were put in a vessel for vacuum treatment. For 3 days the concrete was exposed to 98 % vacuum. Before normal pressure was restored the vessel was filled with tap water. When normal pressure was obtained in the vessel the water will be sucked into the plates due to the gradient in air pressure in the plates. The gradient implies that the plates receive a very high degree of saturation that results in a total degradation when exposed to freezing [5]. After saturation the specimen were exposed to a freeze thaw cycle from +20 °C to –30 °C to +20 °C in 24 hours. The exposure continued for approximately 3 weeks.

From the completely degraded concrete plates using a magneto collects the fibres. Only whole fibres where picked out i.e. only fibres crossing cracks are included in the evaluation.

3.3 Corrosion attack

Only fibres crossing the crack are of interest and they are therefore selected by visual examination of the uncovered fibres. These fibres show corrosion in a part surrounded with unaffected steel in the ends. All other fibres are discarded. To make it possible to measure the loss of fibre diameter, the fibres are cauterized to remove all corrosion products by using a “Clark’s”-solution. The solution consists of antimony trioxide (Sb_2O_3) (20 g/l) and tin chloride ($SnCl_2$) (50 g/ml) dissolved in concentrated hydrochloric acid. The solution will make only the corroded part of the steel fibres to be “washed” away. After treatment in “Clark’s”-solution the fibre diameter was measured with a blade micrometer (non-rotating spindle) in the part that has been exposed in the crack. Repeated measurements where made to identify the smallest diameter in the corroded area. The diameter in the corroded area was compared with the unaffected part of the fibre, originally located beside the crack. The loss is measured as percent of the original diameter.

3.4 Chloride content and depth of carbonation

To find out the amount of chlorides penetrating the concrete and the crack, measurements were made with a chloride profile from the exposed surface. The chloride content was also measured at different distances from the crack mouth along the crack surface (Figure 3). Drilling was performed with a 5 mm drill to a depth of 2-4 mm perpendicular to the crack surface. Therefore the measurements in the crack are an average for a depth of 2-4 mm from the crack surface. Drilling for the chloride profile from the exposed surface was made to a depth of 5-10 mm perpendicular to a newly created crack. During drilling the debris was collected and solved in hydrochloric acid before measuring with the RCT-method according to the inventors Germann and Hansen [6]. The RCT-method is based on measuring the chloride content in a solution with a chloride selective membrane electrode.

Carbonation depth was measured by splitting the concrete and detecting the carbonation depth with a phenolphthalein-solution (areas not coloured red are carbonated).

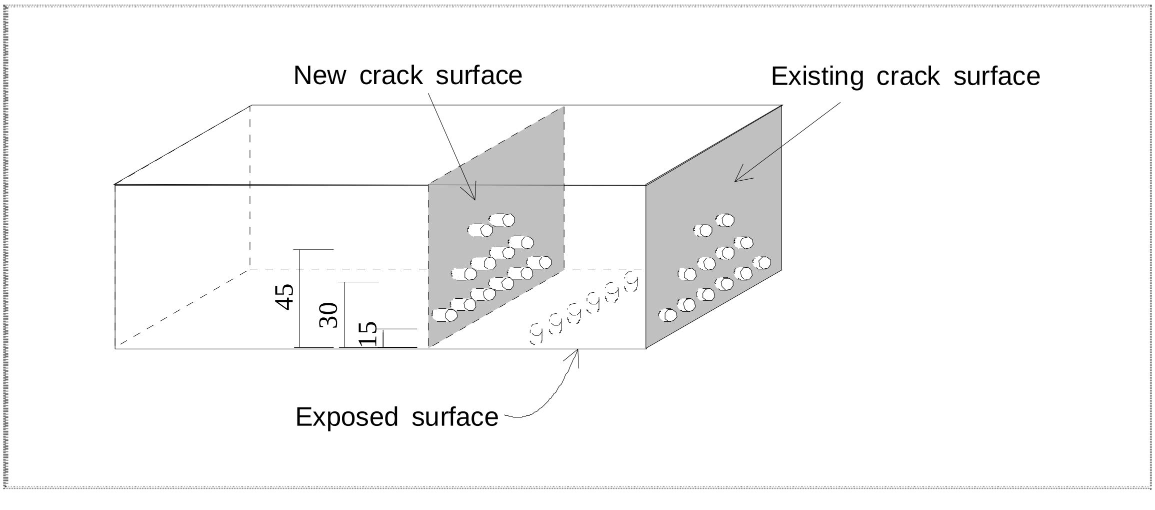

Figure 3: Plan for collection of drilling debris.

4. Results

4.1 Residual strength

To evaluate the change in residual strength two approaches have been developed:
1. The direct comparison approach and 2. The statistical comparison approach.

4.1.1 Direct comparison

The direct approach to detect the change of residual strength with time is based on a comparison of load deflection curves in the four point bending tests before and after exposure. The load at the deflection (differs depending on the crack width) when the flexural load is interrupted, before exposure, is compared with the maximum load achieved at reloading after exposure. In figure 4 a typical test result is shown for a beam with a thin crack (w= 0.1 mm). There it can be stated that the residual strength has increased with 15 % for the sample exposed for one year.

The change of residual strength after 1, 2.5, 5 and 10 years of exposure, according to the direct approach, is shown in figure 5. A slightly higher increase of the residual strength can be seen for crack width 0.1 mm during the first years. For all crack widths the increase seems to have ebbed away or started to decline after 10 years. The influence of type of exposure environment can also be studied. For samples exposed at the Dalälven site, it can be seen that the increase of residual strength is lower and the decrease after 5 years is more obvious. After 10 years of exposure the decrease in residual strength is approximately 30% for w=1.0 mm.

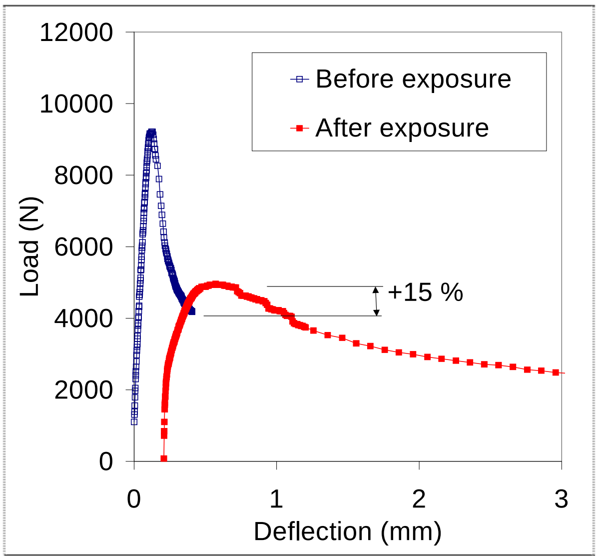

Figure 4: Change of residual strength - direct approach (w= 0.1 mm)

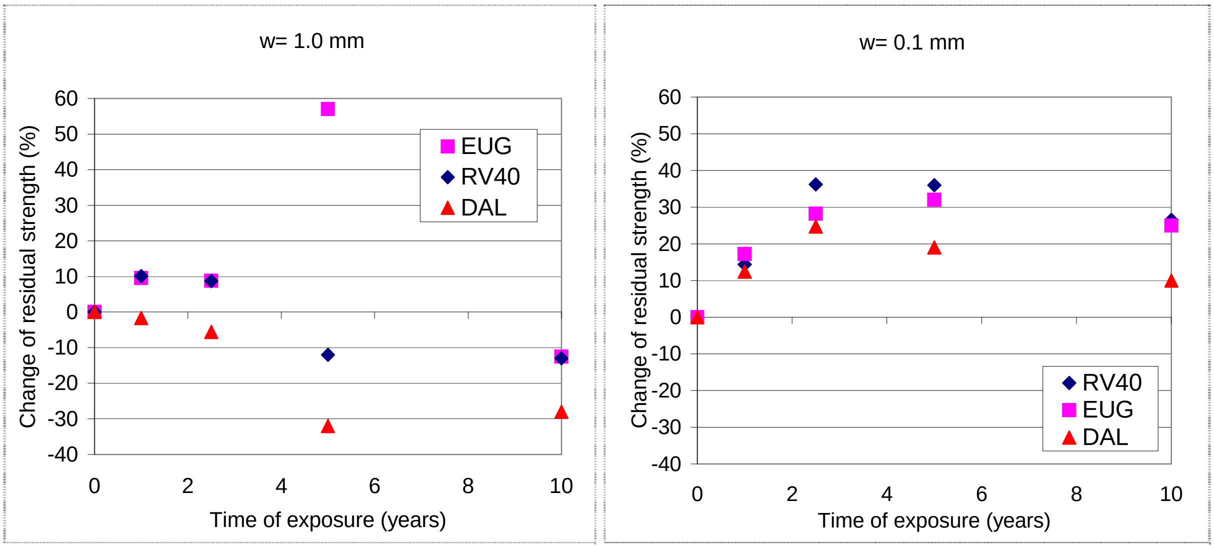

Figure 5: Development of residual strength for samples from mix WA30. Influence of exposure site, w= 1.0 mm (left), w= 0.1 mm (right)

In figure 6 the influence of mix type on the residual strength is shown. The wet-mix samples with 40 mm fibres and accelerator (WA40) did not receive any initial increase of the residual strength and showed a loss of 40 % after 10 years. The residual strength seems to increase slightly more for wet-mix samples with 30 mm fibres and accelerator (WA30) than compared to other mix types. No significant influence of shooting method (dry-mix or wet-mix) can be seen.

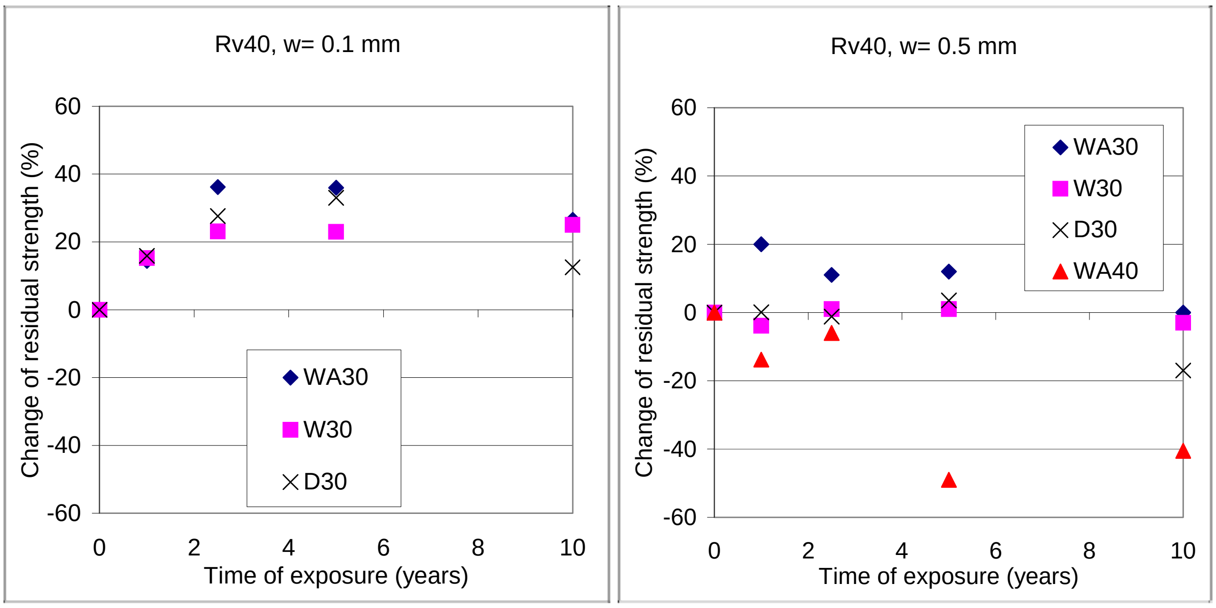

Figure 6: Development of residual strength for all types of mix. Rv40, w= 0.1 mm (left), w= 0.5 mm (right)

4.1.2 Statistical comparison

One weakness of the direct approach is that the load levels are not compared at the same deflection. The load-level when the test is interrupted before exposure is compared with the peak load when continuing the bending test after exposure. These two loads do not occur at the same deflection. In the statistical comparison approach a compilation of 111 beam tests (from routine testing) is used to estimate the expected development of the load-deformation curve, if the test would have continued when the samples where cracked before exposure.

The base for the statistical comparison approach is a large number of standardised beam tests according to the modified ASTM C1018. To estimate the expected load deflection curve the inclination at a deflection interval from 0.35 to 0.45 mm (an interval where there are results from the first loading occasion) is connected to the achieved development up to a deflection of 2 mm. In figure 7 the definition of the inclination (ΔF/Δd) is shown and the results from the compilation of beam tests. In figure 8 the principle for the evaluation of change in residual capacity according to the statistical approach is presented.

The results regarding crack width and exposure site are shown in figure 9 and as can be seen the scatter is very large. This makes it difficult to draw any conclusions regarding the influence of crack width. However, one observed tendency is that samples from the Eugenia tunnel (EUG) initially seemed to perform better than the other samples. After 10 years also these samples show a reduced residual strength.

Influence of mix type on the residual strength at 2 mm, evaluated according to the statistical approach, can be found in figure 8. Also in this approach, the samples with 40 mm fibres and accelerator (WA40) show a lower performance than other mixes. No major influence can be seen between different crack widths after 10 years.

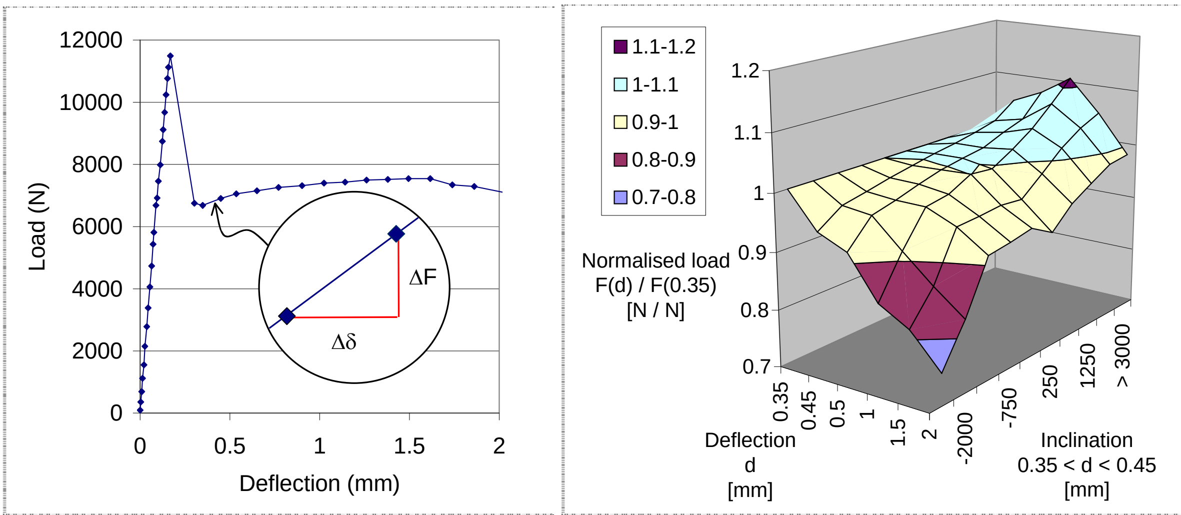

Figure 7: Definition of the inclination in the interval 0.35 < d < 0.45 (left). Average load development up to d= 2 mm due to inclination (right).

To keep in mind is that the compilation of standardised tests is made solely for 30 mm fibres. If a similar compilation had been made for 40 mm fibres the fictive development would have given a higher load level at 2 mm deflection. This is because longer fibres give a higher pull-out resistance and therefore a higher residual strength. Thereby the reduction showed in figure 10 should have been even larger for the WA40 samples.

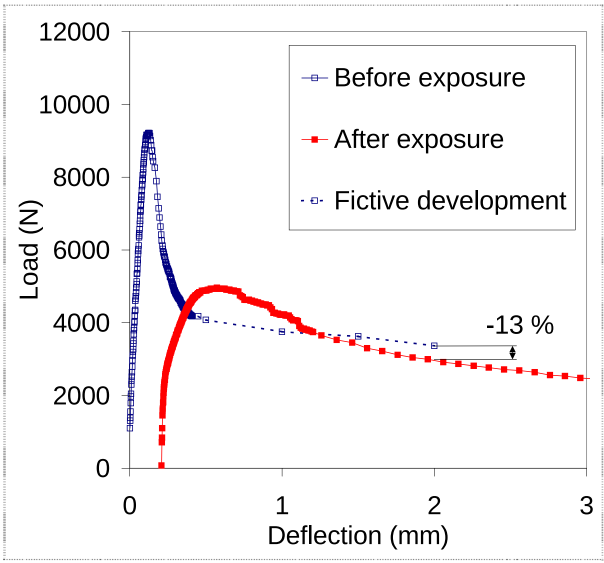

Figure 8: Change of residual strength - statistical approach. Development of residual strength at 2 mm deflection for samples from mix WA30.

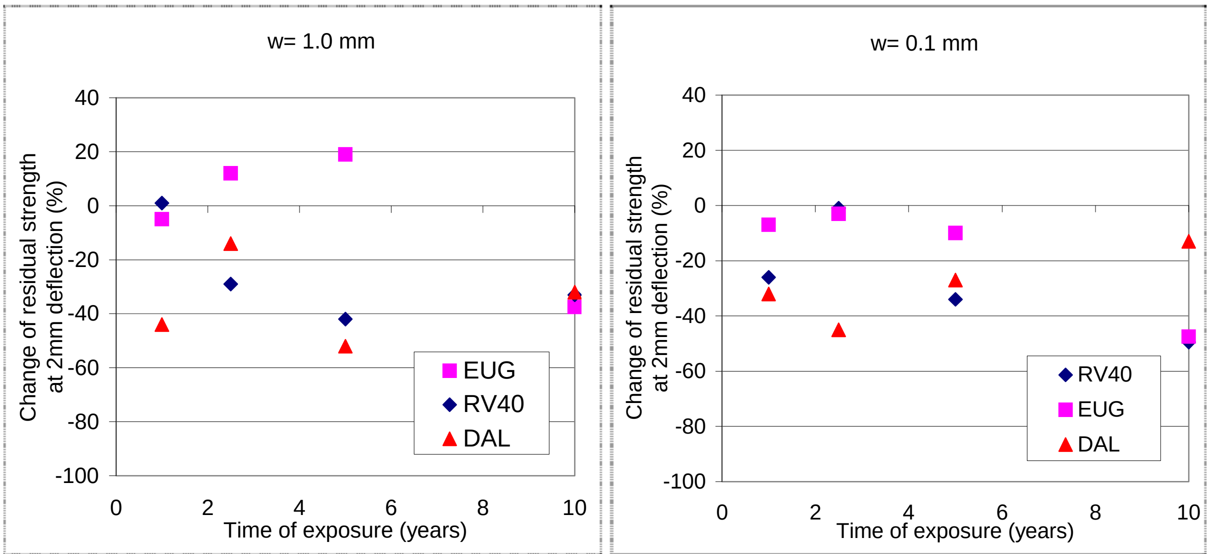

Figure 9: Development of residual strength at 2 mm deflection for samples from mix WA30. w= 1.0 mm (left), w= 0.1 mm (right). (Comparison with fictive values!)

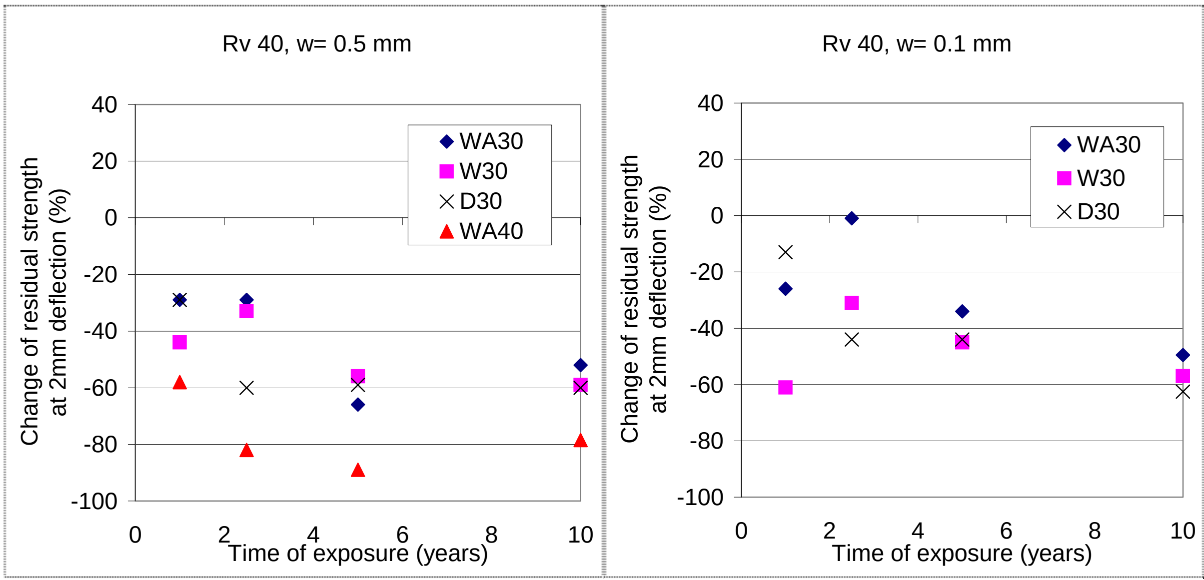

Figure 10: Development of residual strength at 2 mm deflection for all mix types. Rv40, w= 0.5 mm (left), w= 0.1 mm (right). (Comparison with fictive values!)

4.2 Chloride content

Generally, despite some few values, the measured chloride content was quite low (<0.1% Cl/kg cement) after year 1. After 10 years of exposure the chloride content has increased successively. From figure 11 it can be seen that there is no large influence from crack width on the chloride concentration along the crack surface. The chloride profiles made in homogenous concrete (figure 12) show a higher chloride concentration on samples exposed in the Eugenia tunnel.

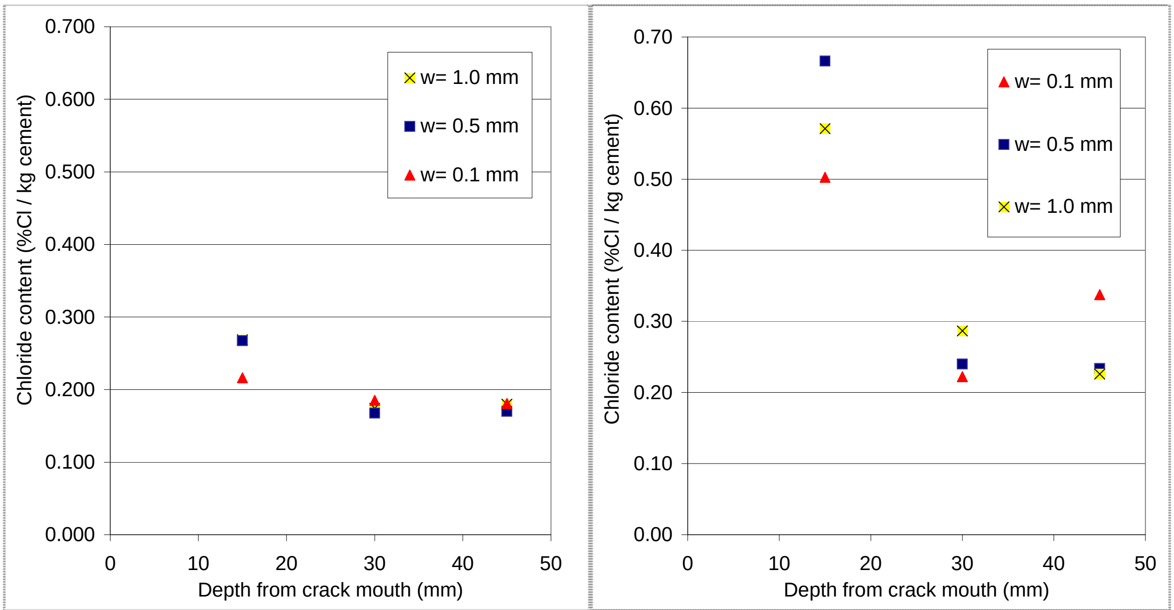

Figure 11: Chloride content at crack surface for WA30-samples after 10 years of exposure. Crack widths. Rv40 (left). Eugenia tunnel (right).

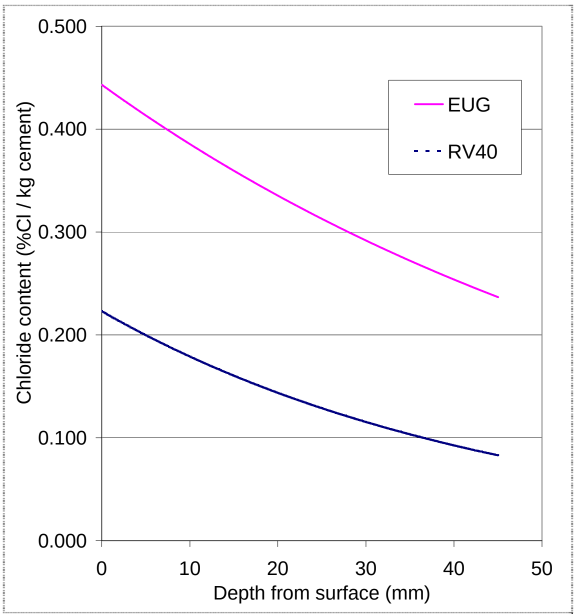

Figure 12: Average chloride profile from homogenous samples after 10 years of exposure. Exposure sites: Eugenia and RV40.

4.3 Carbonation

It could be stated that all concrete types were carbonated only a few millimetres. The outdoor conditions and the short time (in relation to common carbonation rates) of exposure is the reason to this. High quality of the shotcrete giving a low permeability to carbon dioxide is another important factor.

4.4 Corrosion of fibres

Measurements of the loss of fibre diameter give an indication about the rate of corrosion since measurements are made after different times of exposure. To take into consideration that only fibres with a corrosion pit in combination with unaffected steel on both sides are selected, all other fibres crossing the crack are assumed to be unaffected. Since the exact number of fibres crossing the crack is unknown an estimated average is used. From [4] the average number of fibres are 27. The decrease of fibre diameter presented below is therefore an average of found corroded fibres and non-corroded fibres up to a total of 27 fibres. The influence from crack width and exposure site on the corrosion attack is shown in figures 13 & 14. At the exposure site Rv40 the samples with crack width 1.0 mm corrode more than the 0.1 and 0.5 mm samples. The difference in corrosion attack after 10 years is small between w= 0.1 mm and w= 0.5 mm. The corrosion attack for samples in the Eugenia tunnel has increased significantly since the evaluation after 5 years. The attack is even more severe than at Rv40.

In figure 15 the influence from mix type can be studied. The mix with longer fibres (WA40) shows a higher loss of fibre diameter significantly after 10 years of exposure. The degree of attack on the 40 mm fibres is twice as much compared to the 30 mm fibres. The samples from the Dalälven river show almost no corrosion attack. Noticeable is that also the dry-mix samples have started to corrode to a large extent at Rv40. The attack is in the same range as for WA40 samples.

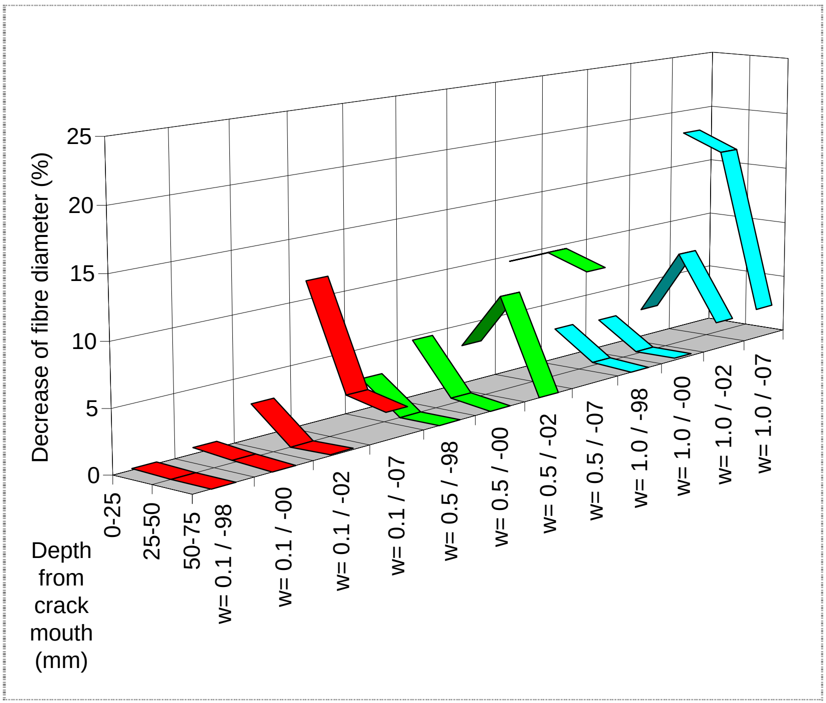

Figure 13: Decrease of fibre diameter for samples at different time of exposure. Rv40, Crack widths, Mix= WA30.

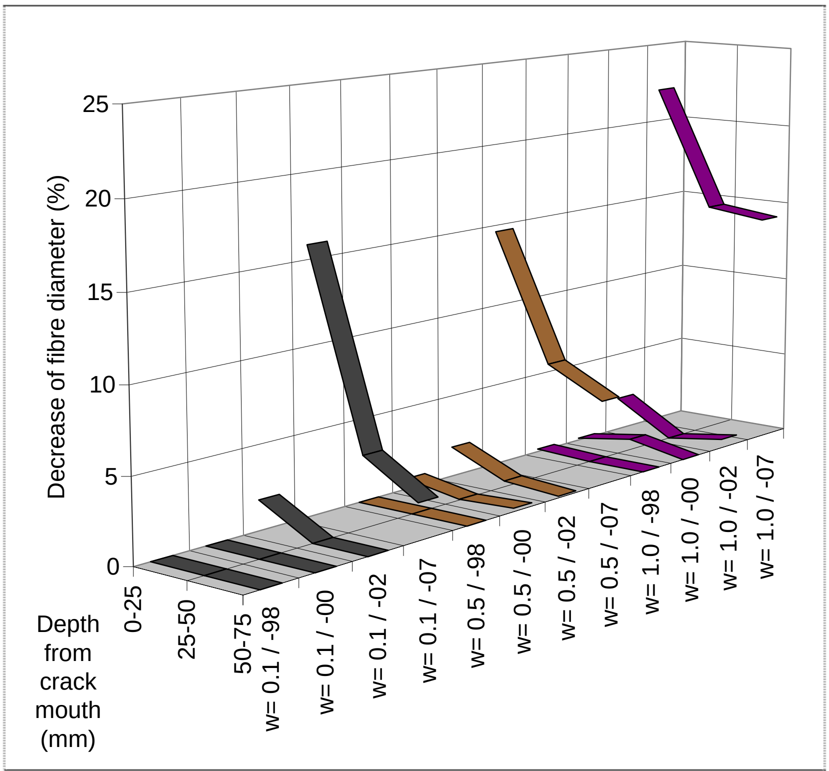

Figure 14: Decrease of fibre diameter for samples at at different time of exposure. Eugenia tunnel, Crack widths, Mix= WA30.

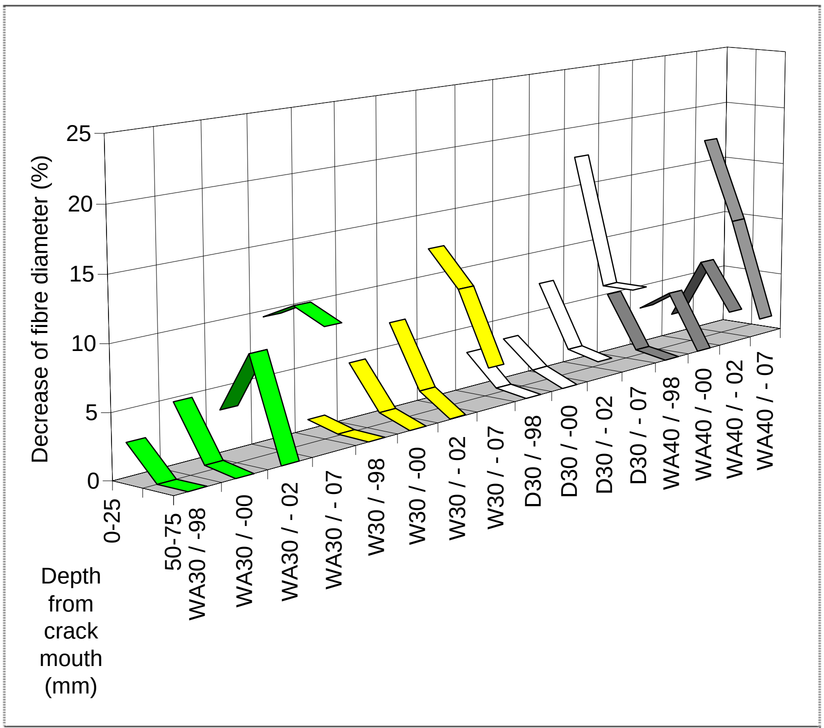

Figure 15: Decrease of fibre diameter for samples at Rv40 at different time of exposure. Mix types, Crack width w= 0.5 mm.

5. Discussion

A factor not taken into consideration in the evaluations is that the samples are placed on slightly different levels (+/- 250 mm) above ground (except the Dalälven exposure). This could influence the microclimate by slightly different humidity conditions (lower level samples should be exposed to slightly higher humidity and higher degree of exposure to de-icing salts). According to measurements by Luping presented in Wirje & al. (1996) [7] a change of height with 0.5 m can change the chloride flux with 50% (at a constant distance of approximately 1 m from the road).

5.1 Residual strength

The initial increase of residual bending strength after exposure, seen in the direct approach, is probably caused by continued hydration of the concrete, resulting in increased anchorage strength between fibre and concrete. This gives an increased residual strength especially at low levels of deflection, where the comparison is made in the direct approach. The strength increase is higher for smaller crack widths and the effect of self-healing could be an additional reason for that effect. It was namely observed that some of the thinner cracks were completely filled with deposits from self-healing. Looking on the statistical approach and the change in residual strength at higher deflections it is not possible to see any increase for the smaller crack widths. The increase must therefore only be valid for a very small pullout distances (fibre slip) of the fibres.

For samples with ongoing corrosion of fibres the increase of residual strength must be expected to be temporary. Actually, the fibres should turn to show a brittle tensile failure instead of a ductile pullout failure at some critical degree of corrosion. The trend after 10 years is that the increase of residual strength already has slowed down for all samples according to the direct approach.

Influence from exposure environment is obvious for the Dalälven river samples, which show a larger loss of residual strength in the direct approach than samples stored at Rv40 and Eugenia. One explanation can be that the samples have been partly immersed in water all the time leading to a high degree of saturation. This resulted in higher degradation of the concrete matrix due to frost action. Another explanation could be a limited self-healing due to conditions with more constant humidity and without splashing or a lower ability to self-heal due to other water characteristics in the river. Continuous flow of river water could also flush the cracks. Ice formation in the cracks or pressure from ice between the beams could make the crack width vary over the year and therefore diminish the possibilities for self-healing. In the statistical approach the Dalälven and the Rv40 samples perform in a quite similar way (larger difference in direct approach). The reason for the loss of residual strength is however probably different. To summarize the discussion, the two evaluation methods show the residual strength loss at different deflections. For the Dalälven samples the loss can be due to frost action and for the Rv40 and Eugenia tunnel samples the loss is more likely to be corrosion. The reason is that the concrete quality is more important at small deflections and the fibres are more important at higher deflections. The influence from concrete quality is therefore better shown in the direct approach and the influence from loss of fibres is better shown in the statistical approach.

In the statistical approach the samples from the Eugenia tunnel show a large reduction of residual strength in the period since the evaluation after 5 years. Corrosion of fibres is the most probable explanation.

The influence of fibre length can be seen from the reduction of residual strength according to the statistical approach where the samples with 40 mm fibres show a larger reduction than samples with 30 mm fibres. This goes in line with the reduction of fibre diameter, where the longer fibres loose more diameter than the shorter ones in the same period.

5.2 Chloride content

The evaluation after one year showed a very low content of chlorides in the samples. At following evaluations the levels have increased. The general chloride contents at both Rv40 and the Eugenia tunnel are quite high (> 0.20 %/kg cement) in the outer 10 mm of the concrete. In the Swedish Road Administration regulations Bro 2002 [8] the upper limit for non-tensioned reinforcement is 0.3 %/kg cement. For homogenous concrete, the use of a chloride threshold value is common. For a cracked concrete like in these tests, the corrosion mainly takes place in the cracked region and therefore the relationship between free OH^- and Cl^- ions is supposed to be less important than the environment (humidity, temperature, oxygen concentration) in the crack.

Surprisingly the samples in the Eugenia tunnel show the highest chloride concentrations. They are placed higher up from the ground than at Rv40 and therefore the chloride exposure should be lower. The lower chloride concentrations at Rv40 could be explained by the fact that the samples there are exposed to rain and direct splashing from the road during the summer period. Chlorides close to the crack mouth are then washed away. Samples exposed in the Eugenia tunnel are sheltered from rain. One uncertainty with the chloride levels in the Eugenia tunnel is that the tunnel surface generally is washed with water applied at high pressure once a year (late summer) and it is not clear whether the exposed beams have been washed or not.

5.3 Corrosion on fibres

Surprisingly corrosion had initiated after only one year of exposure at Rv40. At the same time no or very small amounts of corrosion could be seen in samples exposed in the Eugenia tunnel or at the Dalälven River. After 10 years the corrosion attack continues to propagate at Rv40. In the Eugenia tunnel a rapid change of corrosion attack is noted. The attack is now more severe than at Rv40. One uncertainty is that the tunnel samples have been moved outside the tunnel where they were not sheltered from rain for 6-12 months. This could possibly have given conditions to accelerate the corrosion. On the other hand only 6-12 months should not be that important in a 10-year-perspective. The Dalälven samples only show small amounts of corrosion.

The influence of crack width shows that fibres in 1.0 mm cracks corrode more than in 0.1 and 0.5 mm cracks. It can also be seen that the corrosion attack on fibres decreases with increased depth in the crack. The reason to this should be decreased crack width and therefore lower access for oxygen due to higher RH for longer periods than in the parts close to the crack mouth.

From the limited amount of samples it seems like the 40 mm fibres show a more severe damage from corrosion than the 30 mm fibres. The loss of fibre diameter on the 40 mm fibres is twice the loss on the 30 mm fibres. This supports the hypothesis that the effect of the anode to cathode area to be an important factor. This could also be an underestimation of the attack. Since an ocular method for selection of fibres is used, only fibres not ruptured can be selected. If a fibre has broken due to heavy corrosion or if it has broken due to ice formation in the crack during the freeze-thaw cycles, used to disintegrate the fibres from the concrete matrix, they will not be found in the selection of fibres. The extreme situation would be if all fibres where broken due to heavy corrosion, no fibres would be found.

A correlation between corrosion attack and the behaviour in the residual strength tests can also be made where the samples with 40 mm fibres have lost the majority of their ductility at a deflection of 2 mm.

When it comes to concrete mix type, no significant influence on the corrosion can be seen.

In general the rate of corrosion is much higher than expected in the Rv40 and Eugenia tunnel environment. If the rate of corrosion in this environment is constant, there will be a great loss of residual strength in a couple of years for cracks larger than 0.5 mm. No larger cracks can therefore be accepted in a structure placed in this type of environment, if fibres are used as structural reinforcement.

6. Conclusions

After 10 years of field exposure tests the following conclusions can be drawn:

- Possible continued hydration and therefore increased anchorage strength for the steel fibres gives an initial increase in residual strength at small deflections. The increase is temporary since loss of fibre diameter by corrosion or lowered bond strength between fibre and concrete, due to frost action, will lower the ductility and reduce this effect.

Self-healing of thin cracks (w=0.1 mm) can also be a contributing effect for small crack widths. At higher deflections, in the re-load after exposure, the influence from both corrosion (Rv40, Eugenia tunnel) and frost action (Dalälven river) give rise to reduced load bearing capacity.

- The chloride content along the crack surface increases closer to the crack mouth. Generally there is an accumulation with increasing exposure time. The accumulation seems to be dependent on the access to precipitations, since the tunnel samples (sheltered from rain) have a larger increase of the chloride concentration than the motorway samples (not sheltered from rain).

- A high degree of exposure to de-icing-salts during the wintertime at Rv40 and Eugenia tunnel gives a larger corrosive attack than at the Dalälven River. Samples along the motorway (Rv40) show quite extensive corrosion after only five years of exposure and in a similar way after 10 years in the Eugenia tunnel. The importance of crack width seems to decrease with time after initiation. Longer fibres corrode more than shorter ones at the same crack width. Fibre length seems therefore to be more important than crack width, when w > 0.5 mm.

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