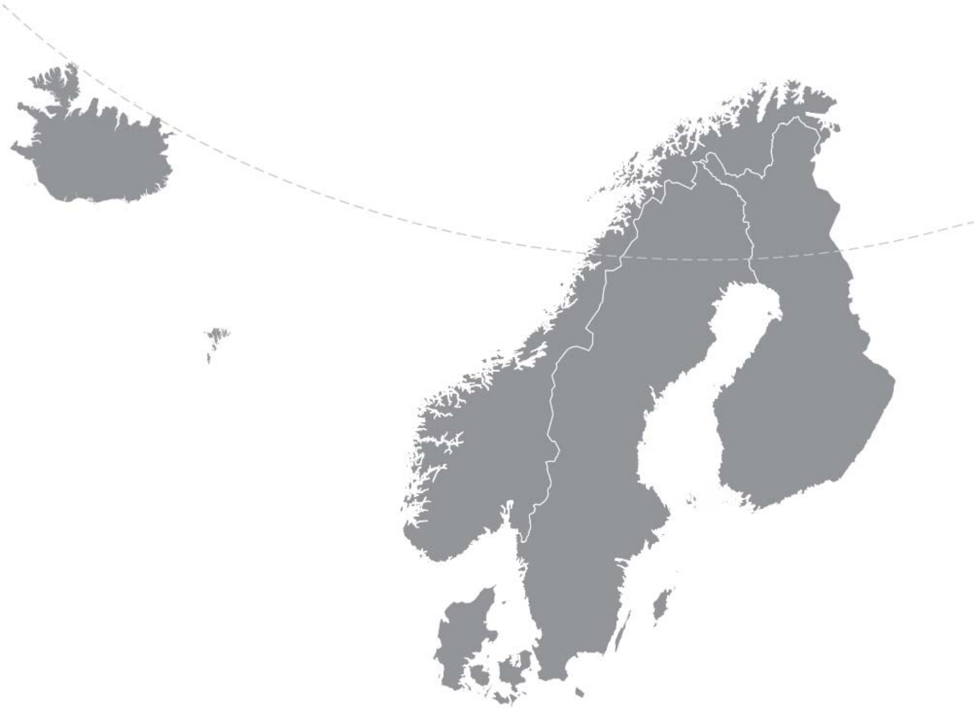
Design and construction of sustainable concrete structures: causes, calculation and consequences of cracks

WORKSHOP PROCEEDINGS FROM A NORDIC WORKSHOP

Oslo Norway, October 16th-17th, 2019



Nordic Concrete Federation

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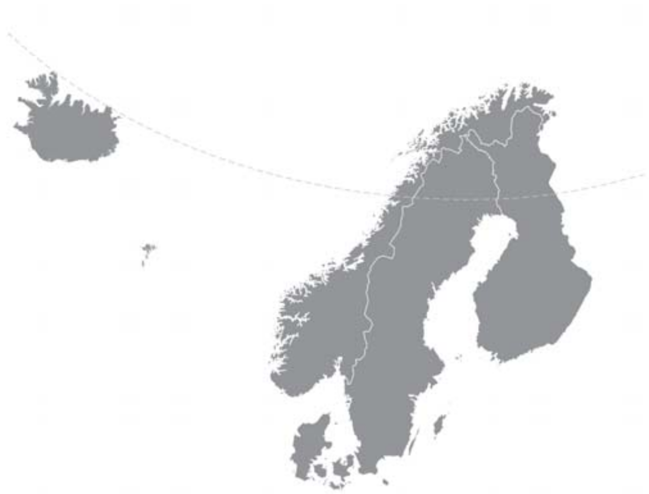

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PREFACE

The Nordic mini-seminar: Design and construction of sustainable concrete structures: causes, calculation and consequences of cracks has been organized under the auspices of the two ongoing Norwegian research projects; The Coastal Highway Route E39[1] and Durable Advanced Concrete Structures (DaCS).

The Coastal Highway Route E39 is carried out under the auspices of the Norwegian Public Roads Administration. The scope of the programme is to improve a stretch of the existing E39 highway between the cities of Kristiansand and Trondheim in Norway, a route that is 1100 kilometres long. Today, the route has seven ferry crossings, the NPRA is considering replacing some of these fjord crossings with alternative structures (i.e. tunnels or bridges) to reduce the travel time along the western coast of Norway. The fjord crossings in question are several kilometres wide, and therefore it is necessary with comprehensive research in order to build structures that are suitable for the fjords in question. The durability and serviceability of the alternative structures are therefore of significant importance.

The research project DaCS looks to increase the knowledge of sustainable and competitive reinforced concrete structures in harsh environment and is funded by The Research Council of Norway, in addition to several industrial partners. Ongoing activities and subtopics related to the aforementioned research projects are i) Crack risk assessment of concrete structures at early ages, ii) Crack width calculation methods for large-scale concrete structures and iii) Impact of cracks and relevance of crack width requirements.

The seminar consisted of keynote lectures on the relevance of the research projects, in addition to presentations within the respective subtopics. The seminar was hosted at Multiconsult in Oslo. The main goal was to increase the understanding and knowledge related to cracking of concrete structures. This booklet documents the collection of extended abstracts of all the given lectures during the seminar. The organizing committee would like to thank all the speakers and contributors at the seminar, and the financial support of the research projects The Coastal Highway Route E39 and DaCS.

Oslo, October 2019.
Kjell Tore Fosså, Kjersti K. Dunham, Terje Kanstad, Mette Geiker, Morten Engen and Reignard Tan (ed.)

[1] The Coastal Highway Route E39 Programme works on development strategies, contracts, societal impacts, safety issues, construction and research. For more information visit www.vegvesen.no/ferjefrie39

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KEYNOTE LECTURES

SEMINAR LECTURES

Controlling crack formation since early ages:
Contributions of COST Action TU1404 and research project IntegraCrete

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ABSTRACT
The present contribution summarizes main activities and conclusions brought about by two funded projects: EU-funded COST Action TU1404 'Towards the next generation of standards for service life of cement-based materials and structures' and Portugal-EU funded project IntegraCrete 'A comprehensive multiphysics and multiscale approach to the combined effects of applied loads and thermal/shrinkage deformations in reinforced concrete structures'.

Key words: Concrete, crack control, early ages, collaborative research.

1. INTRODUCTION
Cracking in concrete structures as perceived by society is influenced by many aspects, most of which endure developments since the early ages of concrete. This happens for example through: residual stresses caused by restraint to heat of hydration induced volumetric changes; non-uniform stress fields caused by drying shrinkage, which affect at crack development since the earliest ages of exposure. Two recently finished funded initiatives on this concern are described hererin: COST Action TU1404 and the Portuguese Research Project IntegraCrete.

2 COST Action TU1404
The COST Action TU1404, entitled 'Towards the next generation of standards for service life of cement-based materials and structures' operated in 2014-2018, with participation of 33 countries, and several integrative research meetings and activities in the scope of the service life of reinforced concrete structures, particularly in concern to cracking behaviour. For more details on the general operation and meetings of COST TU1404, the reader is referred to www.tu1404.eu. One important tool for scientific networking were the Round Robin Testing program, which involved more than 100 ton transport of raw materials and 43 participating labs [1]. Another important tool for this Action was the numerical simulation benchmark, which was composed of several stages [2,3,4]. Both RRT+ and numerical simulation benchmarks interacting with each other, with important exchange of information, namely in regard to parameters for modelling, but also with results of experiments (e.g. TSTM, ring test) made available for validation of simulation approaches [2,3,4].

3 RESEARCH PROJECT INTEGRACRETE
The research project IntegraCrete was taking place 2017 and 2019, funded by a Portuguese Grant (partially supported by EU), and its aims were mostly centred in better understanding the interplay between imposed deformations and applied loads in the crack width formations observed in real structures (see more details on the project in http://civil.uminho.pt/integracrete/). Several initiatives were taken, with some of them interacting directly with COST TU1404. The following initiatives of relevance can be highlighted: the IntegraCrete design challenge for design of

reinforcement for a real case with bending and restraint to deformation [5]; the real-scale laboratory experiment for restraint/bending in reinforced concrete [6]; results obtained through thermo-hygro-mechanical modelling of crack widths in a real sized example [7].

ACKNOWLEDGMENTS

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Cracking of RC structures: Differences between tension and flexure

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ABSTRACT

Dominant cracking models for flexural elements are mostly based on assuming that a virtual tie forms around the reinforcement and the resulting expressions are derived from this assumption. The main difference in existing code formulations is that present in EN 1992-1-1:2004 in which the bond term for crack spacing is affected by factor k_2 which is equal to 0.5 for bending and 1.0 for tension. This paper shows this to be an oversimplification of the problem. An improved proposal is made to account for this effect. But cracking in flexure is different from cracking in tension in other ways. One of them is the effect of curvature on the increase of crack width with cover. It will be shown that this effect, accounted for only in the comments of MC 2010, is very significant. Also, evidence of the effect of bad bond or good bond conditions on crack spacing, affecting mainly flexural members, will be given.

Key words: Cracking, flexure vs. tension, Standard formulations, good/poor bond position

1. INTRODUCTION

In the following paragraphs, some ideas regarding modifications needed in current design code formulations to account for differences between members in flexure and members in tension will be given. These involve the variation of stresses within the effective tensile area, the effect of curvature on the effect on crack width of casting position.

2. EFFECT OF VARIATION OF STRESSES

EN 1992-1-1:2004 recognizes a difference between the flexure and tension in the calculation of crack spacing. Crack spacing is, of course, related to the transfer length, which is a concept referring to the crack formation phase and may be defined as the minimum distance from an already formed crack at which a new crack can form. This distance is governed by the force that needs to be transmitted to the effective tension area around the reinforcing bars to reach a tension stress in the most tensioned fibre equal to the tensile strength (f_{ctm}). At the end of the transfer length behaviour is assumed to be linear elastic. The force transmitted to concrete along the transfer length is limited by the bond strength. In an element in tension, the force would be equal to the effective tension area times f_{ctm}. In bending, however as the stress is variable the force would be smaller. EN 1992-1-1:2004 assumes that the full tensioned area of concrete is affected in this way and proposes a reduction by a factor of 2 in the value of the transfer length in flexure with respect to tension. This is too much, because not all the tensioned area is "effective". A better estimate for this reduction is given in Eq. (1) for a rectangular cross section, where *h* is the section height and $h_{c,ef}$ is the height of effective area.

$$\frac{h-h_{c,ef}}{h} \tag{1}$$

The introduction of this correction results in the limit of (*h*-*x*)/3 to the value of the effective height included in EN 1992-1-1:2004 and MC 2010 to be redundant (*x* is the depth of the neutral axis of the fully cracked section).

3. EFFECT OF CURVATURE

It is well known and experimentally verified [1][2] that crack width increases with cover, and in general with distance to the closest reinforcing bar, due the closure of secondary cracks. This happens both in tension and flexure. In flexure, however, there is an additional increase due to curvature, so that the crack width measured at the concrete surface at the tensioned reinforcement level is smaller than that measured at the most tensioned fibre. This effect can be accurately accounted for by multiplying the crack width computed at the reinforcement level by the factor of Eq. (2), where *d* is the effective height. For large covers and large reinforcing ratios this effect is very significant. As an example, in a $b\times h$=52x60 rectangular beam with 60 mm of cover and 3ϕ40, taken from reference [3], the increase is 23%.

$$\frac{h-x}{d-x} \tag{2}$$

4. EFFECT OF CASTING POSITION

In ULS, it is well recognized that bars cast in poor position need longer anchorage and lap lengths. Even though cracking is also related to bond, to the knowledge of the authors, casting position has never been proposed as a factor affecting crack spacing. Recent investigations [4] show that it is, and corrections should be introduced to account for it. The failure to recognize this effect, may, in fact, be responsible for a significant amount of the scatter observed when comparing experimental data to models since casting position is seldom reported in the literature. This topic needs further investigation. A research program focused in this effect is currently underway at UPM.

5. CONCLUSION

Current code formulations need to be revised in order to account for differences in cracking behaviour in tension and bending as summarized in the paragraphs above. MC 2020 is good opportunity for this.

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Nordic mini-seminar: Design of watertight concrete structures – Can we rely on the selfhealing of cracks?

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ABSTRACT
Structural engineers have commonly used EN 1992-3 "Liquid retaining and containment structures" for the design of watertight concrete structures such as basements, tunnels and water tanks exposed to one-sided water pressure. Following EN 1992-3 one may interpret that for water retaining structures leaking through-going cracks are allowed. In the good faith designers believe that depending on the hydraulic gradient through-going cracks between 0.05 and 0.20 mm are acceptable. However, this is only the case if the through-going cracks heal themselves effectively within a relatively short time. Unfortunately, the selfhealing of cracks does not always occur and the reinforced concrete structures are highly at risk for serious reinforcement corrosion which might even endangers the structural integrity if no other mitigation measures (e.g. injection) are applied.

Key words: Cracking, watertight concrete structures water leakage, selfhealing, corrosion, consequences of cracking.

1. DESIGN OF WATER-RETAINING CONCRETE STRUCTURES

It is common practice when designing water-retaining concrete structures to start with a classification of the structure based on its intended use and the related performance level to define a Tightness Class, leakage and crack limits, see Table 1 below reproduced from EN 1992-3.

Table 1 – Classification of tightness (reproduced from Eurocode EN 1992-3)

Tightness Class	Requirements for leakage
0	Some degree of leakage acceptable, or leakage of liquids irrelevant.
1	Leakage to be limited to a small amount. Some surface staining or damp patches acceptable.
2	Leakage to be minimal. Appearance not to be impaired by staining.
3	No leakage permitted

(111) Appropriate limits to cracking depending on the classification of the element considered should be selected, paying due regard to the required function of the structure. In the absence of more specific requirements, the following may be adopted.

Tightness Class 0. — the provisions in 7.3.1 of EN 1992-1-1 may be adopted.

Tightness Class 1. — any cracks which can be expected to pass through the full thickness of the section should be limited to w_{k1}. The provisions in 7.3.1 of EN 1992-1-1 apply where the full thickness of the section is not cracked and where the conditions in (112) and (113) below are fulfilled.

Tightness Class 2. — cracks which may be expected to pass through the full thickness of the section should generally be avoided unless appropriate measures (e.g. liners or water bars) have been incorporated.

Tightness Class 3. — generally, special measures (e.g. liners or prestress) will be required to ensure watertightness.

NOTE The value of w_{k1} for use in a country may be found in its National Annex. The recommended values for structures retaining water are defined as a function of the ratio of the hydrostatic pressure, h_D to the wall thickness of the containing structure, h. For $h_D/h \leq 5$, $w_{k1} = 0,2$ mm while for $h_D/h \geq 35$, $w_{k1} = 0,05$ mm. For intermediate values of h_D/h, linear interpolation between 0,2 and 0,05 may be used. Limitation of the crack widths to these values should result in the effective sealing of the cracks within a relatively short time.

For tightness classes 0, 1 and 2 water leakages are allowed. Further it specifies the maximum design crack width for the through-going cracks, see clause (111) under the table. Following the classification in EN 1992-3 one may interpret that for water-retaining structures belonging to Tightness class 1 (defined as "leakage to be limited") leaking through-going cracks between 0.05 and 0.20 mm are allowed. Consequently, structural designers take these crack width limits as the governing design crack widths when performing the structural design as they in good faith believe that through-going cracks are allowed for water-retaining reinforced concrete structures. However, designing for through-going cracks is only acceptable if the through-going cracks heal themselves effectively within a relatively short time as stipulated by EN 1992-3. Unfortunately, this condition does not catch every designer's eye as it is hidden in a note under the table in EN 1992-3, see in yellow highlighted text in Table 1.

2. SELFHEALING OF CRACKS

Unfortunately, there are some limitations for achieving a self-healing effect in practice. While limited self-healing may happen, it is highly questionable to purposefully consider that through-going cracks will be fast, completely and effectively sealed. Due to the substantial impact and consequences of whether cracks will ultimately seal themselves by self-healing or not, the role of the self-healing should have been paid more focus in standards and guidelines. Through-going cracks, which are not sufficiently sealed to avoid leaks, are very dangerous from a durability point of view. The risk of reinforcement corrosion and corrosion activity in leaking cracks requires special consideration as it is different to the standard corrosion of reinforced concrete often described in textbooks.

Figure 1 – Left: Successful selfhealing of crack at concrete. Right: Heavy reinforcement corrosion at bars at unhealed cracks.

3. EXPERIENCE FROM PRATICE

The presentation will give some insight in the design of watertight structures, the seafhealing effect of cracks and the presenter's own experience from several projects where relying on the selfhealing of cracks as a design assumption had serious consequences for the structures, both with regard to durability and structural integrity.

Practical curing technology: Practical advice and measures against thermo-cracking in hardening concrete structures


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ABSTRACT
Stress-based curing technology has an academic nature, employed by experts, whereas the solutions are outermost practical and handled by people on-site. Misconceptions and misunderstandings exist regarding detailing and execution of measures to reduce thermal cracking in the hardening phase of concrete structures. An overview of a report-in-progress activity in DaCS WP1.1 is given. The attempt is to narrow the gap between theory and practice by giving examples (FEM-simulations) and recommendations on what to do and what not to do on-site.

Key words: Concrete, hardening phase, thermal cracking, countermeasures, practical solutions

1. OVERVIEW OF REPORT

Within DaCS WP1.1 a report is under progress, finalization late 2019. The effect of different measures (Fig. 1) to reduce the risk of through-cracking (due to thermal and restraint effects) in hardening concrete structures will be illustrated and practical recommendations on execution given. This involves, among others, how to place heating cables and/or cooling pipes. The main chapters of the report are: Problem description, various restraint cases and actual degree of restraint, temperature requirements, counter-measures (Fig. 2) and relative effect on crack risk, recording and documentation. Two concretes, previously mapped experimentally [1], are used in the evaluations:

1. B45 MF40 with a CEM II with 17% fly-ash ("Ref.")
2. B45 MF40 with a CEM II + additional fly-ash, totally 30% fly-ash of binder weight ("LH")

Some wall-on-foundation cases have been selected and the effect of various measures (see Fig.2) will be evaluated with the use of the 2D FEM-program CrackTeSt-COIN (some verification with 3D DIANA is also planned).

Figure 1 –Example of through-cracking and water leakage in a wall. Photo: B.Kristiansen, AF Gruppen

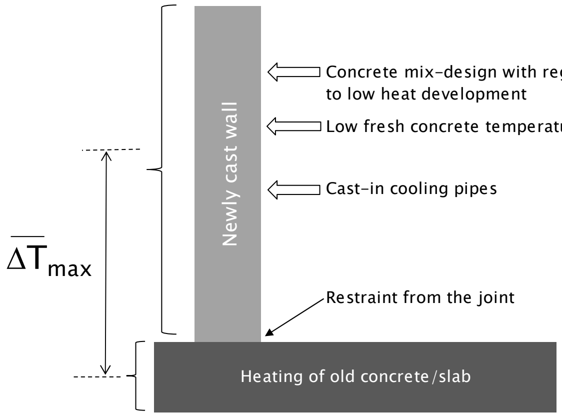

Figure 2 – Measures to limit average temperature difference new vs. old structure

2. EXAMPLES OF SIMULATION RESULTS

The examples in Fig.3 below involves the effect of using concrete 1 (“Ref.”) and 2 (“LH”) in a thick (800 mm) and a thin (400 mm) wall, both cast on a massive foundation. In addition to that the variables are: “Ref” = summer 15 ºC air, fresh concrete temperature 20 ºC. “Cold concrete” = fresh concrete temp. 15 ºC. “Winter” = winter 0 ºC air. A calculated crack index of 1.0 (or above) indicate that through-cracking is very likely to occur. In Fig.3 a crack index=0.75 is indicated, as max. 0.75 is commonly set as maximum where simulations are required. 0.75 takes simulation uncertainty into consideration and has shown to secure moderate or no cracking tendency onsite. The results show that the different cases give quite different crack indices. Both the Ref.- and LH-concrete are prone to cracking in the thick wall for all cases, and additional efforts has to be taken to secure no/low crack risk (mix-design, heating cables, cooling pipes). Relative to the Ref-simulation, the crack index varies within app. ±20% for the cases involving lower fresh concrete and ambient temp. Winter generally leads to cold foundations, which is an unfavourable single factor.

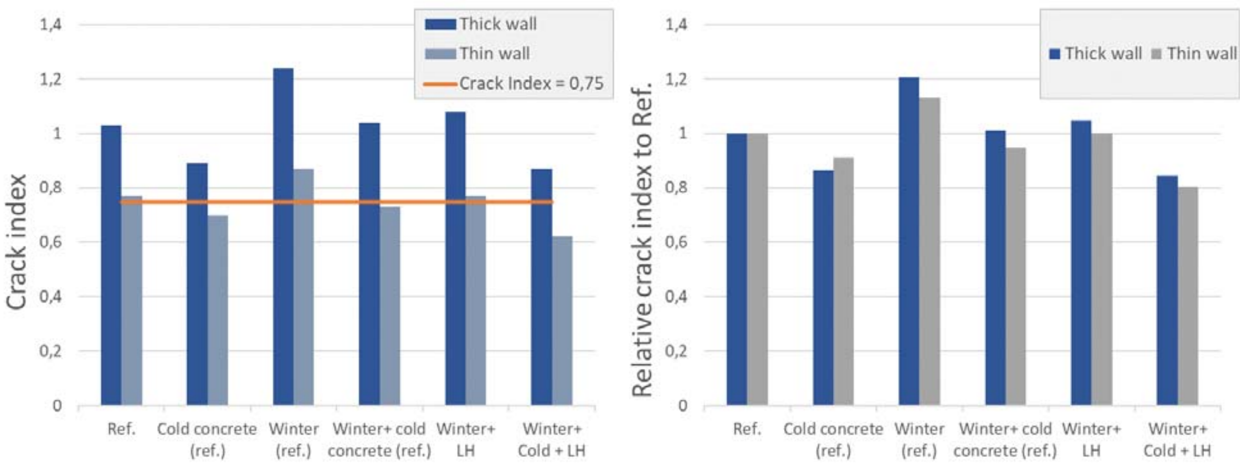

Figure 3 – Example: Effect of some chosen variables on the maximum crack index for a thick and thin wall cast on a massive slab. Crack index; absolute (left) and relative to Ref (right)

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Impact on reinforcement amounts from SLS design of large concrete structures and consequences of design code ambiguity

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ABSTRACT
For most reinforced concrete structures, the critical load combinations in the ultimate limit state (ULS) are decisive for the reinforcement demand, and further checks in the serviceability limit state (SLS) are usually not problematic. However, experience shows that the SLS control of large-scale concrete shell structures can often result in unexpected additional reinforcement amounts. In this paper, this topic is investigated through a parametric study, and a case study where different frameworks for crack width calculations are compared. The results from the case study indicate that the large number of choices that need to be made by the analyst, inevitably causes a variation in the results from different calculation methods. This effect is especially pronounced in case of skew cracks. The results from the parametric study indicate typical additional reinforcement

demands in the SLS as compared to the demand in the ULS. The relative increase of reinforcement amount was shown to be as high as several hundred percent. The underlying reason for the deviations demonstrated in this paper is assumed to be a combination of *i*) different understandings of the crack width limitation requirements, and *ii*) different interpretations of the calculation methods within the engineering community.

Key words: Cracking, shell structures, large concrete structures, crack width calculation, design codes.

The research project “Durable advanced Concrete Structures”

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ABSTRACT
The DaCS (Durable advanced Concrete Solutions) was established together with 13 partners including SINTEF and NTNU in 2015. The DaCS program focus on some identified challenges with regard to design and construction of concrete structures for coastal and arctic regions.

Key words: Cracks, large concrete structures, high performance concrete

1. DURABLE ADVANCED CONCRETE SOLUTIONS (DaCS)

The identified research areas in DaCS was the following work packages [1]:

- WP1 - Early age cracking and crack calculation in design
- WP2 - Production of frost resistant concrete
- WP3 - Concrete ice abrasion
- WP4 - Ductile, durable Lightweight Aggregate Concrete

1.1 DaCS WP1 – “Early age cracking and crack calculation in design”

This WP deals with the ability to predict and limit cracks on short and long term in concrete structures exposed to harsh environment, as well as the impact of cracks on durability, aesthetics and tightness in general. This work package was divided in the following sub-programs:

- Crack risk assessment of concrete structures at early ages
- Calculation of Crack spacing and crack width
- Relevance of crack requirements.

The target for these three programs was (a bit simplified) to obtain more knowledge why cracks occur, how to control it, and what does it mean. Cracks can occur in early or later phase depending on the cause. And quit often, cracks are related to reduced durability and possible corrosion.

1.2 DaCS WP2 – “Production of frost resistant concrete”

The target for WP2 is investigation of requirements to part materials, production and exposure conditions to obtain concrete materials with adequate frost resistance both for onshore and offshore severe exposure. The work package has mainly dealt with sustainable binder materials in durable concrete for production and use under varying kinds of frost exposure. Focus has also been on developing robust admixture and to study common freeze/thaw tests and their evaluation criteria.

1.3 DaCS WP3 – “Concrete Ice abrasion”

WP 3 has the target to increase the understanding of the mechanisms behind concrete-ice abrasion through laboratory experiments of high-performance concrete and repair mortar. The research has been performed with the ice abrasion rig located at NTNU. A non-standard laboratory abrasion rig has been further developed for simulation concrete-ice abrasion. During the project, several types of concrete mixes have been tested: B75, B85, air-entrained concrete B70 5% air, lightweight concrete LB60, Repair Mortar. Concrete with and without frost exposure (ASTM

C666) has been tested to ice abrasion. The focus of this research was on new parameters for concrete-ice abrasion. It was possible to capture different ice fracture modes, variation of coefficient of friction during the test, concrete surface parameters, concrete wear particles and ice wear fragments. Laboratory tests showed an average abrasion depths of 0.01 – 0.35 mm for high-performance concrete after 3 kilometres of sliding ice. Also a prediction model based on laboratory measurements was obtained that includes a pilot lattice model of interaction between ice asperity and concrete surface.

1.4 DaCS WP4 – "Ductile properties in light weight concrete"

The objective of WP4 is documentation and improvement of the ultimate compressive strain in lightweight aggregate concrete. The study also included the influence of different types of confinement, where the confinement effect comes primarily from reinforcement bars. An experimental program are performed to investigate the confinement effects on the compressive ductility. The lower density was obtained by using light weight aggregate, Stalite (expanded slate) from North Carolina, replacing the coarse aggregate in the concrete. The main targets of the work are: a) study the effect of different types of reinforcement configurations on compressive ductility. b) study the effect of strain gradients on the ductility c) derive material models to be used in numerical analysis or design which include confinement effects.

2. WHAT HAVE WE LEARNED?

Kvaerner participates in R&D-programs national and international, in order to be in front of the technology, both within design and execution. A main focus is new technology/solutions that can improve the durability in concrete structures at competitive cost. Essential part of this is also sustainability and environmental-friendly concrete structures.

The DaCS program as described above, have resulted in more knowledge within key areas of the concrete technology. The research areas was defined based on a gap-analyses performed on what research-areas to be in improved. After 4 years, the identified research areas are thoroughly examined, and more knowledge and improved technology have been obtained. New challenges to be addressed are also identified (which was expected).

In general, the DaCS research program has contributed with more knowledge within the selected research areas. Some selected outcome: improved concrete test methods, improved the understanding of the concrete behavior under frost exposure and ice abrasion, studied the correlation between cracks and durability, contributed with input to standard, studied the effect of confinement in lightweight concrete, confirmed that depth and quality of the cover zone are essential for the service life.

3. ACKNOWLEDGMENT

The DaCS project has received financial contribution from the Norwegian Research Council.

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Durability aspects of cracks in concrete: field observations.

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ABSTRACT

The research presented in this paper briefly summarises findings from a project on impact of cracks on reinforcement corrosion within the Norwegian Public Roads Administration's R&D program "Ferry-free coastal route E39". A main part of the project was the collection of long-term field data on the influence of cracks on chloride ingress and reinforcement corrosion.

Key words: Concrete, cracking, ingress, reinforcement corrosion, self-healing.

1. INTRODUCTION

Based on, among others, [1-4] we concluded in [5] that there is a need for detailed long-term data and understanding of mechanisms governing corrosion propagation of steel reinforcement embedded in cracked concrete. In the literature, there is consensus on the promoting influence of concrete cracking on the initiation of reinforcement corrosion. However, corrosion propagation in cracked concrete is not yet fully understood. Contradicting conclusions are reached based on short- and long-term observations. Short-term investigations (up to a few years) indicate that corrosion rates are enhanced by cracks and mainly depend on cover depth and concrete quality rather than on crack width. The few undertaken long-term studies indicate that small cracks have limited influence on corrosion propagation. However, we do at present not have sufficient background for such a general statement.

2. FIELD OBSERVATIONS

2.1 Ingress and self-healing

The impact of cracks on chloride ingress was found to depend on, among others, crack width, exposure (road vs marine) and surface orientation [6]. Many of the observations may be explained by the self-healing ability. Extended self-healing was observed in marine exposure, especially in the splash and submerged zones [7]. The mechanism of self-healing was independent of the binders investigated [7]. Results from a long-term field test combining fatigue and cathodic protection (CP) suggest a potential beneficial impact of temporary CP, limiting chloride ingress and facilitating self-healing [8].

2.1 Reinforcement corrosion

The impact of cracks on corrosion was found to vary [6, 8, 9]. Among others, we observed a strong impact of other potential defects on corrosion development. In 25 years old marine exposed elements, severe corrosion was observed in connection with plastic spacers, whereas none or negligible corrosion was observed in the vicinity of cracks [9]. Numerical simulations support the hypothesis that the corrosion at the "weakest link" (here the severely exposed plastic spacers) protects the steel in other areas including cracks.

Results of more than 5 years of continuous *in situ* monitoring of a cracked concrete element exposed to splash and submerged conditions in Rødby Havn Denmark [10], showed unexpected behaviour: (i) Corrosion potentials indicating active corrosion were only observed after almost two years of exposure. This is in strong contrast to many laboratory investigations that have reported corrosion initiation in cracked concrete within days. (ii) Cycles of corrosion potentials indicating depassivation and repassivation were observed with a duration of several years. The varying state of corrosion underlines the importance of monitoring to understand the mechanisms of corrosion initiation and propagation and ultimately the service life of structures.

3. SUMMARY AND ACKNOWLEDGEMENTS

In summary, depending on possible self-healing and corrosion initiation at other weak areas, cracks may or may not lead to corrosion.

We should like to acknowledge funding from the NPRA R&D program "Ferry-free coastal route E39" and the DaCS project. Access to data from Femern Belt field exposure station, and contributions by Andres Belda Revert and Ulla Hjorth Jakobsen are also appreciated.

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Chloride-induced reinforcement corrosion: the influence of cracks, concrete moisture, and exposure duration

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ABSTRACT
Chloride-induced reinforcement corrosion in cracked concrete is one of the major challenges in civil engineering. The service life for structures in chloride-bearing environments is generally described in two stages: the time to corrosion initiation and the corrosion propagation phase.

It is generally agreed that cracks reduce the time to corrosion initiation. This is due to the faster ingress of chlorides and presumably local damage at the steel-concrete interface for mechanical cracks. Concerning the corrosion propagation phase, however, recent experiments and a literature review reveal that corrosion propagation is not only influenced by the presence of the crack but also by the moisture conditions (ambient relative humidity) and exposure duration.

Key words: Cracking, chlorides, reinforcement corrosion.

1. INTRODUCTION

Chloride-induced reinforcement corrosion in cracked concrete is one of the major challenges in civil engineering causing high costs to society. These costs will increase in the forthcoming years as the age of infrastructure is continuously increasing. Understanding corrosion phenomena of steel in concrete is thus important.

This presentation discusses the influence of cracks on the corrosion initiation and propagation phase by a literature review and with results of our own experiments.

2 LITERATURE REVIEW

2.1 Corrosion initiation

For the influence of cracks on corrosion initiation, around 40 studies have been analysed. The vast majority of those studies concluded, that corrosion initiation is accelerated by the existence of cracks. However, no consistent answer on critical crack widths could be found. [1]

2.2 Corrosion propagation

26 studies have been reviewed, which report experimental results about the acceleration of the corrosion rate by the presence of cracks. The following figure shows that the conclusion of an individual study was strongly dependent on exposure time and exposure condition. The influence of cracks on corrosion propagation ("accelerating corrosion") was stronger in submerged exposure compared to spraying conditions. Moreover, the influence of cracks on corrosion rate became less apparent with increasing experimental time (short-term vs. long-term studies).

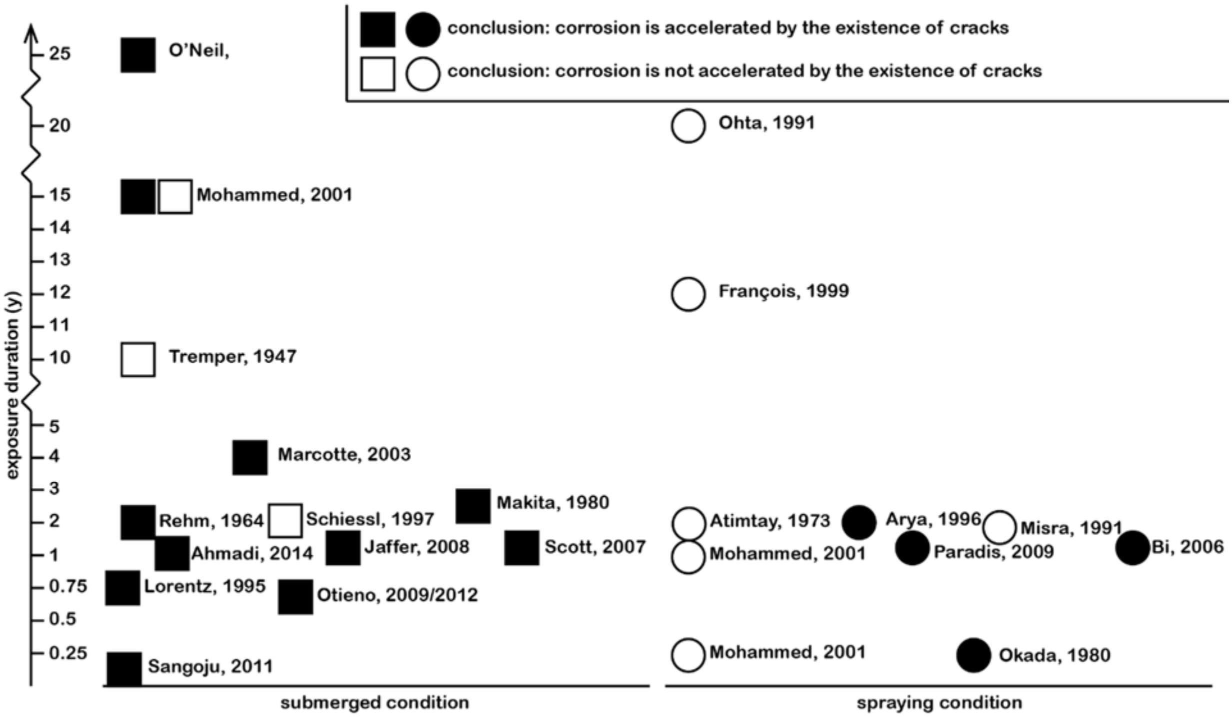

Figure 1 – Summary of conclusions about the effect of cracks on corrosion rate.

3 EXPERIMENTAL RESULTS

In a laboratory experiment, cracked reinforced concrete beams were exposed to periodic chloride spraying, followed by exposure to different relative humidities. The corrosion rate between the wetting events was considerably higher for high RH conditions (75%) than for low RH conditions (40%). These results are in agreement with the literature study.

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Chloride-induced corrosion in cracked concrete at early ages: maintenance, monitoring and prediction.

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ABSTRACT
Reinforced concrete structures form the basis of our infrastructure. However, its serviceability is often impaired due to chloride-induced corrosion of the reinforcement. In cracked concrete the situation is even more severe due to a shorter period of corrosion initiation. The lesson learnt from the past is that as soon as a structure in chloride-contaminated environment shows cracks, corrosion measures need to be applied. This presentation shows the results and discusses the conclusions of both - laboratory and on-site tests of possible corrosion measures for structures cracked at an early age.

Key words: Chloride-induced corrosion, cracks, concrete structures, monitoring, repair

1. EXTENDED ABSTRACT

Civil engineering structures are very costly to build and therefore long service lives are required. However, reinforcement corrosion is a major reason to shorten their service life. Chloride ions from marine environment or de-icing salts penetrate through the porous concrete and as soon as a critical chloride concentration reaches the reinforcement corrosion becomes likely. Moreover, in cracked concrete the transport of chlorides to the reinforcement is even faster and corrosion initiation is unavoidable if no corrosion protection measures are applied. Main purpose of the protection measure is to prevent further chloride and moisture ingress. Such measures could be e.g. the application of a crack bridging concrete coating once early cracking is completed according to EN 1504-09 procedure 8.3 [1]. The repair principle aims to increase the concrete resistive in order to impede the ion transport in the concrete and to hinder corrosion processes. However, during the early age cracking phase, which can take up to six month, the concrete is exposed to chlorides already. Thus, reinforcement corrosion could have been initiated in vicinity of the cracks before the concrete coating is applied.

In an extensive laboratory study [2] and on-site study [3] this situation was reproduced to check whether corrosion will be initiated and if so, if the corrosion rate slows down to a negligible rate. One of the two main findings of the experimental laboratory study was that due to the coating the corrosion process was stopped in all specimens. However, it was found that the coating did not have the expected influence on the electric resistivity of the concrete. Instead, the anodic polarization resistance was identified as the decisive controlling factor of the corrosion system, from which it was inferred that the diminishing corrosion activity resulted from a change of anode kinetics and herewith a reduction of the anodic sub-process because of the changing exposure conditions (chlorides ingress without coating, no chlorides ingress with coating). In a numerical study [4] this time-dependency of the anodic behaviour was considered in order to be able to simulate the corrosion process after the coating application and to extrapolate the corrosion development beyond the experimental investigation. The numerical study was in very good agreement with the experimental results and confirmed that the anodic polarization resistance is the dominant control factor in the corrosion system according to the repair measure 8.3 given in EN 1504-9 [1]. Contrary to what is written in the standard the repair principle is not successful due to the increase of the concrete resistivity. Its applicability is based on the increase of the anodic polarization resistance. The reasons for the increase of the anodic resistance could be the lack of supply of chloride ions and/ or the formation of an iron oxide layer close to the corrosion pit. Further research is going on to clarify these assumptions. Moreover, it remains to be unknown, which amount of chlorides in the concrete are acceptable before the application of the concrete coating to exclude on-going reinforcement corrosion reliably.

The EN 1504-9 demands additional measure to control the effectiveness when applying the repair principle 8.3. The additional measure can be a corrosion monitoring system. In parallel to the laboratory experiment a corrosion monitoring has been installed in structures with comparable condition [3]. The specification B12 “Corrosion Monitoring of reinforced and pre-stressed concrete structures” [5] from the German Society for Non-Destructive Testing gives further guidance of the selection, application and evaluation of corrosion monitoring systems. After a monitoring period of about several months the corrosion current decreased in all monitoring spots while the anodic polarization resistance increased constantly. The monitoring proved the effectiveness of the concrete coating however, the monitoring period can take years before the corrosion activity stops. Additionally, the acceptable corrosion rate until the repair measure shows its effectiveness needs to be discussed with the structural engineer in charge. However, there are probably plenty of application where this repair measure could be an option.

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Experience with operation and maintenance of 30-year-old offshore concrete structures

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ABSTRACT

- Equinor`s offshore concrete portofolio overview
- Historic overview, from 80`s until now
- Problem descriptions
- Inspection methods for Equinor`s offshore concrete structures
- Inspection findings, and causes
- Rehabilitation offshore
- Summary and conclusion

Key words: offshore concrete structures, GBS, inspection, rehabilitation

Early age stress development – experimentally based parameter study performed within DaCS

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ABSTRACT
The present document contains a brief overview of a comprehensive test program performed within the research project DaCS (Durable advanced Concrete Solutions) on early age cracking of concrete structures. The program includes material property characterization and corresponding restrained stress tests in a Temperature-Stress Testing Machine, and the work has been performed in order to investigate how parameters such as shrinkage reducing admixtures (SRA), fly ash content, cement batch and aggregate type influence the early age cracking risk of concrete.

Key words: Early age cracking, large concrete structures

1. INTRODUCTION

The present document contains a brief overview of a comprehensive test program performed within the research project DaCS (Durable Advanced Concrete Solutions) on early age cracking (EAC) of concrete structures. EAC is induced by volume changes in the hardening phase caused by autogenous deformation and thermal dilation. In addition, EAC is strongly dependent on material and geometrical properties such as hydration heat development, tensile strength, E-modulus, creep, cross-sectional dimensions and degree of restraint. Several of these parameters are known to be complex and dependent on for instance mix-design, w/c-ratio, time, degree of hydration and curing temperature. Despite this, material parameters used in early age crack calculations are often determined from generalized models found in guidelines and codes. The currently performed test program investigates how parameters such as addition of shrinkage reducing admixtures (SRA), fly ash content, aggregate type and cement batch influence EAC of concrete.

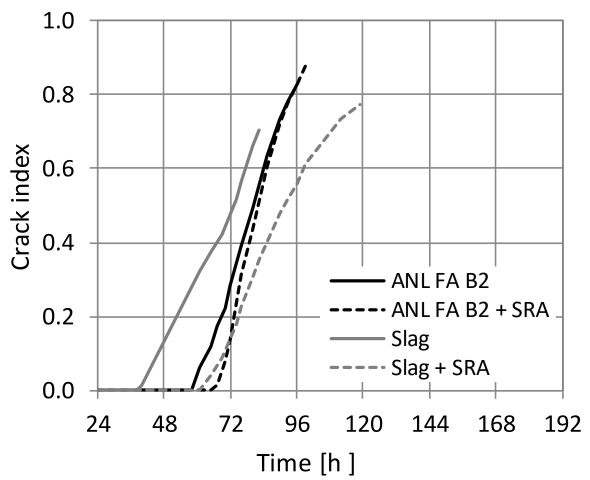
2. CONCRETES AND EXPERIMENTAL TEST PROGRAM

The currently tested concretes were based on two types of cement: one slag cement "slag" and three different batches of a fly ash cement "ANL FA B1", "ANL FA B2" and "ANL FA B3" containing approximately 16% inter-ground fly ash. The variation in the concrete mixes included addition of 1% SRA, cement-by-fly ash replacement and change of aggregate (from Aardal to Ansit). The investigated concretes and the mix design are described in [1]. All concretes contained 5% silica fume (by weight of cement + FA).

The current property characterization includes development of heat, compressive strength, tensile strength, E-modulus in tension and compression and autogenous deformation, as well as restrained stress development in the Temperature-Stress Testing Machine (TSTM), which is designed to directly simulate the stress development over time for a given section of a concrete structure.

3. TEST RESULTS

The cracking tendencies of the investigated concretes were evaluated based on the crack index, which is the occurring tensile stress measured in the TSTM divided by the corresponding tensile strength development. The obtained crack indexes are presented in Fig. 1, and they represent the cracking risk for a predefined 800-mm-thick wall on a slab [1]. Fig. 1 (left) shows that the addition of 1% SRA had a very beneficial effect on the cracking risk for the slag concrete, however it did not have much effect over time on the ANL FA concrete. Fig. 1 (right) shows that ANL FA Batch 2 and 3 gave quite similar cracking risk, while ANL FA Batch 3 reduced the cracking risk for the given structure. Increasing the fly ash content to 45% by cement-by-fly ash replacement reduced the cracking risk considerably. For the currently exemplified 800-mm-thick wall, changing the aggregate type did not have a big effect on the cracking risk. It should however be noticed that although the obtained cracking risks could give a general indication of the effect of the investigated parameters, they are only valid for the given structural case. The variation in the cracking risk with the investigated parameters shows that an accurate characterization of the development of relevant material properties for the concrete in question is of great importance when it comes to EAC design.



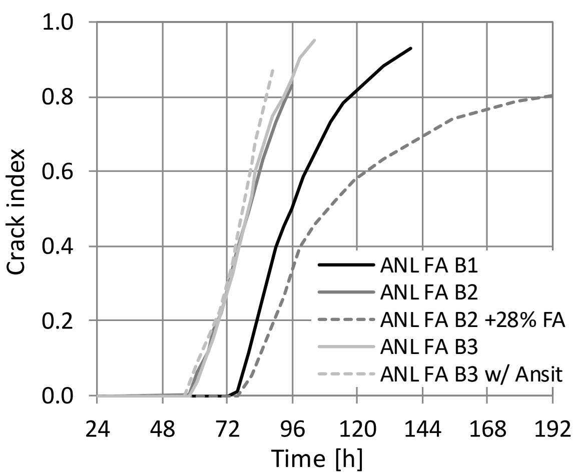


Figure 1 – Crack index. Left: Effect of SRA. Right: Effect of cement batch, aggregate and fly ash.

ACKNOWLEDGEMENTS

The work has been performed within the User-driven Research-based Innovation project DaCS (Durable advanced Concrete Solutions, 2015 - 2019) and COIN (Concrete Innovation Centre, 2007 – 2014, https://www.sintef.no/en/projects/coin/coinp/, a Centre for Research-based Innovation established by the Research Council of Norway).

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Comparison of the experimental crack spacing behavior with the theoretical predictions.

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ABSTRACT
Cracks in reinforced concrete structures are controlled during the service life, by limiting the calculated crack width. The spacing of cracks is an important parameter for both calculating the crack width and to predict the number of cracks that can occur. An experimental study have carried out to study the crack spacing behaviour. The maximum crack spacing results gives a good agreement with the Model code 2010 model. Adaption of the upper and lower fractile values of the tensile strengths of concrete to the Model code 2010 model, gives a better prediction on the range of crack spacing results.

Key words: Crack spacing, axial tension test, tensile strength of concrete

1. INTRODUCTION

The 'crack spacing models' proposed by each codes have different approaches. In order to identify the model, that gives a better agreement with the experimental results, an experiment was conducted for reinforced concrete beams under axial tension load.
It is important to introduce a criteria to control the number of cracks, while controlling the crack width. As the crack spacing denotes the number of cracks, identification of the distribution of the crack spacing is another objective of this experiment.

2 CRACK SPACING MODELS OF MODEL CODE 2010 AND EUROCODE 2

Both Model code 2010 and Eurocode 2 crack spacing models are based on the theoretical approach of Beeby, A. (2001) [1]. The equation (1) represent the relationship, where the maximum crack spacing ($S_{r,max}$) is β (a value between one and two) times the minimum crack spacing ($S_{r,min}$).

$$S_{r,max} = \beta\, S_{r,min} \tag{1}$$

Table 1. Crack spacing models in EC2 and MC 2010

Eurocode 2	Model Code 2010
$s_{r,\,max}$ =1.7 [2.c + (1/4) . k_1. k_2. (φ / $\rho_{p,eff}$)] where β_1 =1.7	$s_{s,\,max}$ = 2 [1.c + (1/4) .(f_{ctm}/ τ_{bms}) (φ_s / $\rho_{s,ef}$)] where β_2 =2

3 MATERIAL AND METHOD

Axial tensile tests were carried out for three similar specimens, up to the stabilized cracking stage. Figure 1, shows the details of the specimens.

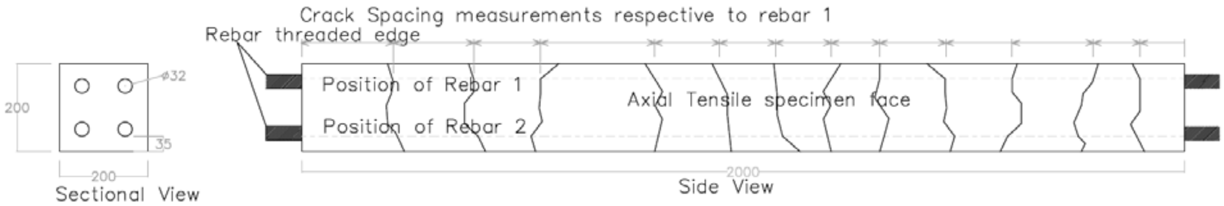

Figure 1 – Cross section details and the side view of specimen (all the dimensions are in mm).

4 RESULTS AND DISCUSSION

Table 2. Experimental results and predictions from EC2 and MC 2010.

Description	Axial tension experiment		Tan et al, (2018) (diameter 32, cover 35)	
	minimum crack spacing (mm)	maximum crack spacing (mm)	minimum crack spacing (mm)	maximum crack spacing (mm)
Experimental Data	79	192	89	301
EC2 prediction	150	254	239	407
MC2010 prediction	90	180	150	301
Strength fractile to MC 2010	73	213	117	367

The 'minimum to maximum crack spacing' ratio deviates from the theoretical prediction of two. The main reason can be the inhomogeneous properties of concrete [3]. In order to represent the effect, the fractiles of the tensile strength properties were included to the MC2010 model.

4 CONCLUSIONS

The experimental readings of maximum crack spacing values, gives a better agreement with the MC 2010 model than the EC2 model. By considering the lower and upper fractile values of concrete tensile strengths to the MC 2010 model, gives a better prediction of the range of crack spacing.

ACKNOWLEDGEMENTS

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Simulations in CrackTeSt COIN with CEM II/B-M

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ABSTRACT
The government funded research project COIN (2007-2014), yielded the FEM based calculation software, CrackTeSt COIN. With the software, it is possible to simulate the curing process of concrete with regard to temperature and risk of cracking.

The focus has traditionally been to examine how curing temperature can be reduced by adding fly ash as a substitute for cement. Norway has some 200 different concrete plants spread all over the country [1], not all of them have the silo storage for fly ash, nor a low-heat cement. In the following chapters, concrete with CEM II/B-M [2] is simulated and measures to reduce thermal cracking are investigated.

Key words: FEM, concrete curing analysis, thermal cracking of concrete

1. INTRODUCTION
In the FEM based software CrackTeSt COIN (CTC), there is limited public material data sheets available. This is one of the reasons why the use of the software is not currently widespread when casting large concrete structures. The database can be extended by doing laboratory tests that documents heat and material properties.

In projects owned by the Norwegian Public Road Authorities (NPRA), there is an option to use computer software to assess the risk of thermal cracking and hence possibly reduce the need for measures, i.e. reduced cost for the contractor. If this option is to be used, a 1m^3 well-insulated curing box needs to be cast prior to the software simulations. Temperatures are logged, and from this, values for heat development can be calculated based on the principle of maturity. These values can be implemented in CTC to calculate the heat generated in the structure.

Below, a virtual curing box has been used to obtain the heat development for a concrete with a CEM II/B-M, which is the most common cement in Norway. The virtual curing box is a 1D computer software provided by the cement producer [3]. An existing material data sheet in CTC is adapted and modified to fit the temperature curve of the virtual curing box. The curing box is modeled in CTC and the values are iterated to obtain the best correlation between the virtual curing box and CTC.

2. METHOD
To investigate the method, previous data from a 1m^3 cast sample were compared with the virtual curing box where the same temperatures were applied. The two curves correlated well and the

same was done using a mix design with CEM II/B-M. One of the most important parameter in CTC is Q [kJ/kg], the values for Q is linked together with the maturity age of the concrete. Values for Q is calculated using a spreadsheet developed for the purpose, using temperature data from the virtual curing box, see *Fig.1*.

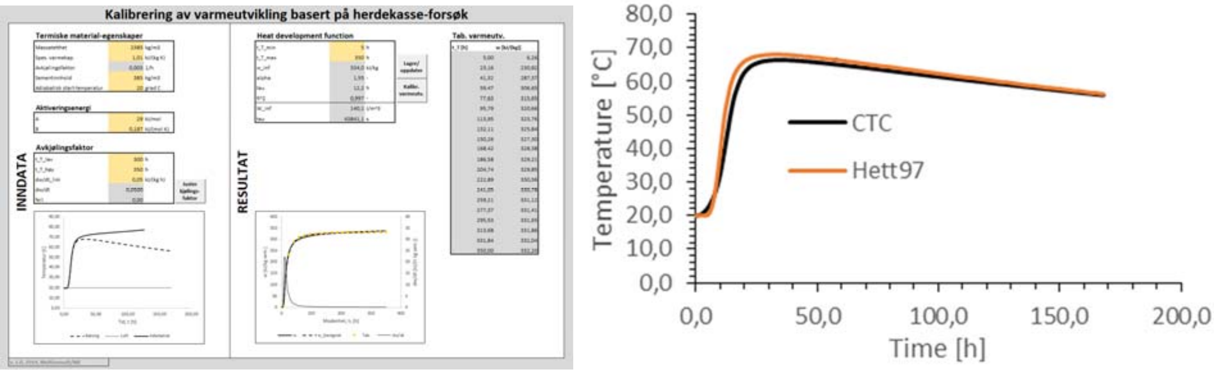

Figure 1 – Left: Showing calculated values for Q. Right: Showing the correlation between the simulations between the 1D software and CTC, simulated for 7 days according to the NPRA method description [4].

Values extracted from the new material data sheet in CTC are applied to a simple structure to investigate the differences between a mix design using the CEM II/B-M and a semi-low-heat cement.

3. RESULTS

Simulations are performed on a 0,5 m wall casted onto a slab of matured concrete, that again is casted on rock. As expected, the temperature is higher with the CEM II/B-M and the wall reaches its maximum temperature at an earlier point in time, see *Fig.2*. From these results, we can simulate the crack index in the wall, based on the temperature simulations and empirical mechanical properties.

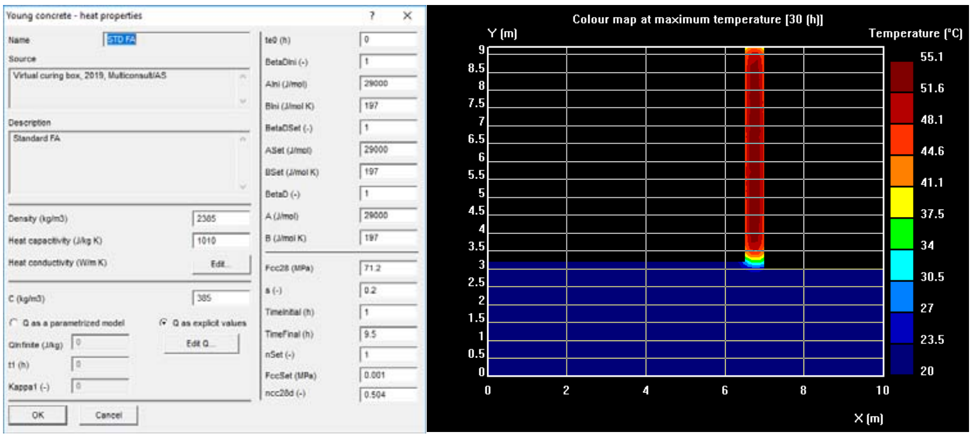

Figure 2 – Left: Shows the input parameters of the new material data sheet in CTC. Right: Showing a color map of the wall at maximum temperature.

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Cracking in High Volume Fly Ash Concrete specimens during the European salt-frost slab test: dilatometry measurements and consequence for surface scaling

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ABSTRACT
The European salt-frost slab test CEN/TS 12390-9 measures the resistance of concrete to surface scaling under the combined attack of frost and deicer salt. Frost damage can also be measured as internal cracking. For that length change measurements with invar steel dilatometers equipped with LVDTs in a low – temperature chamber were taken during repeated freeze-thaw cycles in the slab test. Some preliminary results are shown on the effect of environment (minimum temperature, liquid uptake during cycling) and material (air voids, fly ash, water-binder ratio) on cracking expressed as residual length change and its consequence for surface scaling damage.

Key words: Frost testning, salt scaling, internal cracking, fly ash

1 INTRODUCTION
Concrete exposed to freezing and thawing can be tested in accordance with very different freeze-thaw testing procedures used in different parts of the world. Alongside with harsh freeze-thaw test in freshwater ASTM C666 procedure A, which is often used for offshore and bridge concretes to identify internal cracking susceptibility, concrete can also be tested for resistance to surface damage in the presence of deicing salt [1]. The latter test was modified by using in-house-developed unique leakage-proof preparation system adapted for simultaneous measurements of internal cracking and surface damage to investigate the consequence of cracking for surface damage in concrete with high amount of fly ash.

2 METHODS
Table 1 shows some relevant properties of hardened concrete. The mix code is translated as follows: 0.45-35 **A** means w/b=0.45, FA/b = 0.35 and air-entrained (0.45-35 **0** is a code for non-air-entrained mix). All mixes contain 4% of SF in a binder. The w/b = 0.293 mixes were made to represent w/c = 0.45 for zero pozzolan hydration, i.e. in comparison with non-FA 0.45-0 A. All the mixes were exposed to standard salt-frost sclaing slab test. Mixes 0.293-35 were in addition exposed to standard slab salt-frost testing [1] and "Arctic" test (36h long "Arctic" cycles - +20C°…-52C° with heating and cooling velocities from the slab test) after 4 months of storage in lime water.

Table 1. Properties of hardened concrete

Mix	Air content, hardened [%]	Air void spacing factor [mm]	PF [%]	Comp. strength [MPa] 28d	91d
0.293-35 A	5,9	0,20	36,1	81,5	93
0.293-35 0	1,8	0,63	17,6	98,8	114,9
0.45-35 A	4,1	0,18	32,5	50,4	63,3
0.45-35 0	2,0	0,68	17,0	67	81,9
0.45-0 A	6,2	0,30	31,7	77,2	

3 RESULTS

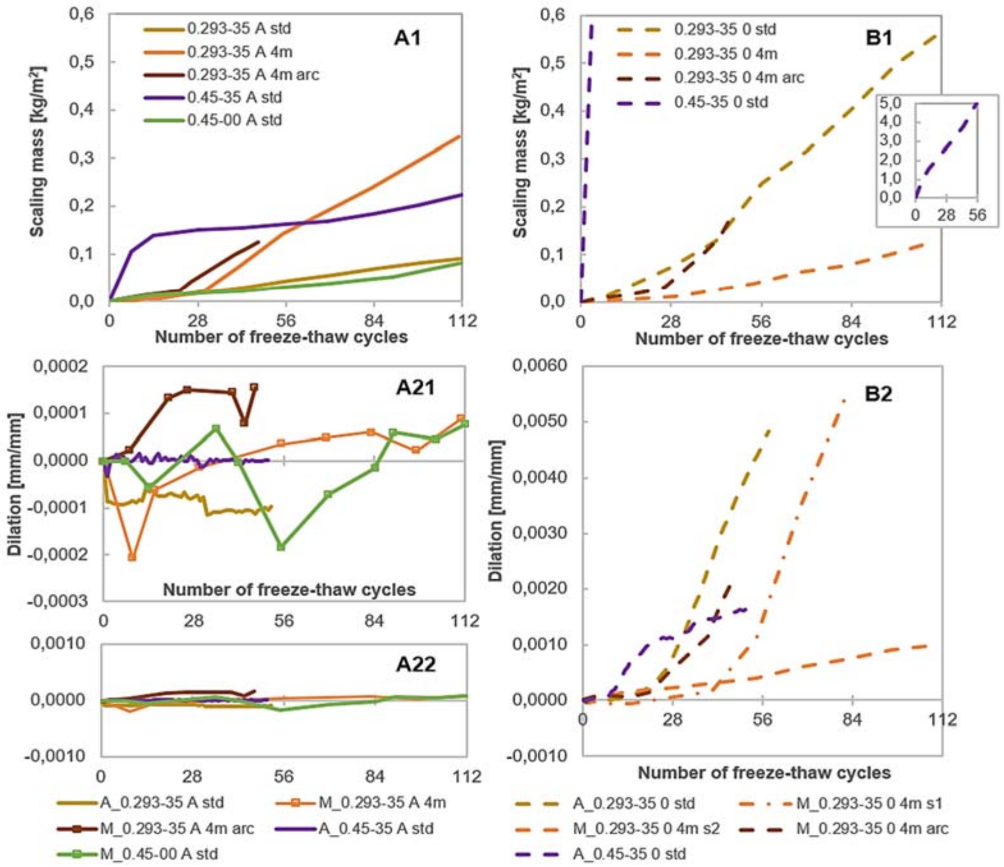

Figure 1 – Surface damage, length change, and water uptake in salt-frost scaling test:
***A** and **B** divide the concretes to air-entrained and non-air entrained; A1,B1 – surface scaling; A21,A22,B2 – dilation, where A22 shows A21 in the same scale as B2.*

**Note: In the legend for A21, A22, B2 there M and A letters in front of the mix code mean that the measurements were taken Manually and Automatically (continuously). Manual measurements of a length change of the specimens with LVDT, comparing to a length of the invar reference rod. The fluctuations of length change on A21, A22 can be related to a difference in temperature of the specimens between the measurements.*

To assess possible consequences of cracking for surface scaling damage we looked at the curvature of the accumulated scaling as a function of number of cycles in salt-frost testing. The acceleration is taking place if the ratio between mass at 56 and 28 (as per standard [1]) or at 112 and 56 cycles is more than 2. Grand average acceleration factor (GAAC) of all tests shows that non-air entrained concrete (GAAC 3,3) has larger tendency to accelerated scaling than air-entrained concrete (GAAC 2,2). Figure 1 shows scaling and internal cracking for a selection of the tested concretes.

Figure B2 shows that the non-air samples have a tendency to internal damage whereas Figure A21 and A22 show low / no internal damage for air-entrained concrete as expected. There is a tendency that concretes with the lowest w/b-ratio and a reference concrete (with “poor” air void system) show expansion to a level near strain capacity of concrete. B2 also shows that initiation of internal damage is delayed as effect of lowered w/b-ratio and prolonged curing time.

Comparing internal damage (B2) and scaling (B1) it seems that very high and accelerated scaling correlates to dilation (=internal cracking) for all non-air entrained samples. For air-entrained samples Figure A1 shows that the 2 concretes with accelerated scaling (or delayed onset of scaling?) also have larger length change but the scaling level is low and scatter of length change is possibly high. In addition to scaling and internal cracking measurements, we are looking into liquid uptake in the salt-frost testing and freshwater rapid freeze-thaw testing known as ASTM C666 procedure A for rather broader variation of concretes and curing ages.

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Simplified cracking analysis of RC ties taking account of the effect of secondary cracks

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ABSTRACT
A simplified model based on the assumption that bond stresses are linearly distributed along the transmission length is here set up to study the cracking behaviour of an RC tie subjected to monotonic loading, taking into account the effects of the so-called Goto cracks, or internal secondary cracks. At the stabilized cracking stage, under the assumption that the crack spacing is maximum, the relative mean strain formula proposed in *fib* Model Code 2010 and Eurocode 2 is modified by introducing a coefficient η, depending only on the acting axial force-to-cracking force ratio, which allows the effect of internal secondary cracks to be considered.

Key words: average bond stress, cracking, maximum and minimum crack width, crack spacing.

1. INTRODUCTION

A simplified approach, named linear method, is here formulated to study the stabilized cracking stage of an RC tie. The method avoids the need of iterative procedures for the stabilized cracking stage. It is based on the assumption that bond stresses are linearly distributed along the transmission length, L_s. Here, only the situation of maximum crack spacing is considered, equal to twice the transmission length, but the study can be performed also in case of minimum crack spacing [1]. The formula for the calculation of the maximum crack width in the stabilized cracking stage is, then, obtained. Finally, the theoretical results are compared with the experimental data concerning tests on RC ties.

2 SIMPLIFIED ANALYSIS OF CRACKING BEHAVIOUR

Assuming a linear elastic behaviour of the materials and on the basis of the equilibrium conditions of forces acting on the concrete and steel, the following second-order differential equation of the slipping contact between steel and concrete can be obtained:

$$\ddot{s}_s(x) = \frac{4\cdot\tau_{bs}}{E_s\cdot\phi_s}\cdot\left(1+\alpha_e\cdot\rho_{s,ef}\right) \qquad (1)$$

Two stages can be distinguished. First, a crack formation stage takes place. When the axial force increases, other cracks occur randomly along the axis of the member till a certain situation, named stabilized cracking stage, in which the crack pattern stabilizes.
With the general method [2], the solution of the second-order differential equation (1) can only be expressed in closed form for the crack formation stage, while an iterative procedure is needed to solve the problem when the crack pattern stabilises.
In order to avoid iterative calculations, a simple as possible approach for the direct calculation of the crack width at the stabilized cracking stage, based on the assumption that the bond stresses are

linearly distributed along the transmission length, taking account of the effect of internal secondary cracks, is here proposed. Bond stresses and slips are correlated through the *fib* Model Code 2010 bond law only at the section where bond stresses reach the maximum value. In the other sections, bond stresses and slips are in compliance with equation (1).
When the crack distance from the adjacent cracks on each side of the crack is equal to twice the transmission length, the crack presents an upper bound width, w_{max}:

$$w_{\max}^{(stab.cracking)} = s_{r\,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) = \frac{2 \cdot L_s}{E_s} \cdot \left[\sigma_{s2} - \frac{2}{3} \cdot \frac{f_{ct}}{\rho_s} \cdot (1 + \alpha_e \cdot \rho_s) \cdot \left(1 - \frac{\ell_{sc}}{2 \cdot L_s} \right) \right] \quad (2)$$

In equation (2) the term in square brackets assumes a form that is similar to the formula of the relative mean strain proposed in *fib* Model Code 2010 or in Eurocode 2 for the calculation of the crack width. This similarity occurs unless for the term $(1 - \eta / 2)$, in which $\eta = \ell_{sc} / L_s$ and ℓ_{sc} is the length of secondary cracks. It results that, when the axial force increases, the secondary cracks reduce the effect of tension stiffening on the relative mean strain, ε_{sm}-ε_{cm}, through a factor $(1 - \eta / 2)$ which depends only on the axial force, diminishes from 1 and 0.5 as the axial force increases, and is independent on bar diameter, reinforcement ratio and concrete strength.

3 COMPARISON BETWEEN THEORETICAL AND EXPERIMENTAL DATA

The linear method can be applied for a comparison with the experimental data, concerning short-term tests on RC ties, available in the literature. It results that the experimental data are mostly smaller than the theoretical upper bounds of crack width.

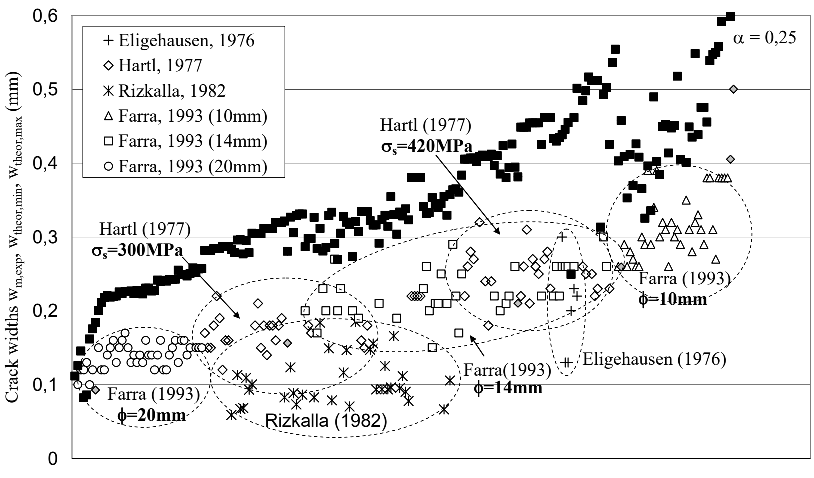

Figure 1 – Comparison between experimental results and theoretical values of maximum crack width (square black boxes).

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Calculation method for predicting skew cracks in RC structures

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ABSTRACT
Skew cracks occur in most reinforced concrete structures and should be accounted for appropriately in Serviceability Limit State design. A proposal for predicting skew cracks is presented in this paper due to its inadequacy in the current design codes. This paper seeks to contribute to the new revisions of the design codes currently being undertaken.

Key words: Cracking, skew cracks, tension stiffening, multiaxial stress states.

1. INTRODUCTION

Provisions for determining skew cracks are currently incomplete in design codes such as Eurocode 2 (EC2) [1] and *fib* Model Code 2010 (MC2010) [2], meaning they provide guidelines for determining the maximum crack spacing for skew cracks but not the difference in mean strains. Skew cracks occur typically in members subjected to multiaxial stress states, such as two-way bearing slabs, flat slabs, RC membranes and RC shells. Such members are present in most RC structures thus amplifying the significance of having complete provisions for predicting skew cracks in such cases. This paper aims to improve the inadequacy in the aforementioned design codes by proposing an expression for the difference in mean strains for skew cracks based on the work of [3].

2 DERIVATION OF MEAN STRAINS

The crack width is in general determined as

$$w_{\mathrm{k,cal}} = S_{\mathrm{r,max}}(\varepsilon_1 - \varepsilon_{\mathrm{c1}}) \tag{1}$$

where $S_{\mathrm{r,max}}$ is the maximum skew crack spacing, ε_1 are maximum principal strains including for tension stiffening and $\varepsilon_{\mathrm{c1}}$ are maximum principal concrete strains. Expressions for determining ε_1 and $\varepsilon_{\mathrm{c1}}$ can be found using Mohr's circle of strains at which the difference in mean strains can be derived as

$$\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}} = \frac{(\varepsilon_{\mathrm{smx}} - \varepsilon_{\mathrm{cmx}}) + (\varepsilon_{\mathrm{smy}} - \varepsilon_{\mathrm{cmy}})}{2} + \sqrt{\left[\frac{(\varepsilon_{\mathrm{smx}} - \varepsilon_{\mathrm{cmx}}) - (\varepsilon_{\mathrm{smy}} - \varepsilon_{\mathrm{cmy}})}{2}\right]^2 + \left(\frac{\gamma}{2}\right)^2} \tag{2}$$

where $(\varepsilon_{smx} - \varepsilon_{cmx})$ and $(\varepsilon_{smy} - \varepsilon_{cmy})$ are differences in mean strains in the x and y directions of the reinforcement respectively, making them compatible with any other tension stiffening model for uniaxial loading. The shear strains γ must in general be determined from equilibrium iterations while accounting for strain compatibility and tension stiffening. However, conservatively neglecting tension stiffening yields a simplified expression

$$\gamma = (\varepsilon_x - \varepsilon_y)\tan(2\theta) \qquad (3)$$

where $\varepsilon_x = \sigma_{s,x}/E_s$ and $\varepsilon_y = \sigma_{s,y}/E_s$ at which $\sigma_{s,x}$ and $\sigma_{s,y}$ are steel stresses at the crack, while E_s is the Young's modulus. Here, the steel stresses can be determined as $\sigma_{s,x} = (\sigma_x + \tau_{xy}\tan\theta)/\rho_x$ and $\sigma_{s,y} = (\sigma_y + \tau_{xy}\tan\theta)/\rho_y$ according to the compression field theory, at which σ_x, σ_y and τ_{xy} are inflicted stresses to the member while ρ_x and ρ_y are reinforcement ratios. Furthermore, θ is the angle between the reinforcement in x direction and the direction of the maximum principle strains ε_1. Conservatively, θ can be determined from linear elastic analysis. A better approximation can be obtained by solving the following equation

$$\tan^4\theta + \frac{\sigma_x}{\tau}\tan^3\theta - \frac{\sigma_y}{\tau}\frac{\rho_x}{\rho_y}\tan\theta - \frac{\rho_x}{\rho_y} = 0 \qquad (9.20)$$

3 SUMMARY

A proposal to predict tension stiffening for skew cracks was provided in this paper to improve the inadequacy in the current design codes. The expression was derived while accounting for equilibrium iterations, strain compatibility and material laws for tension stiffening. However, simplifications showed that the expression conservatively also could be applied without having to solve for equilibrium in an iterative manner. This means that for a set of inflicted in-plane stresses, skew cracks can be predicted analytically using simple hand calculations.

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Digitalisation in modern concrete bridge design: Evaluation of various approaches used for SLS-design in selected computer programs

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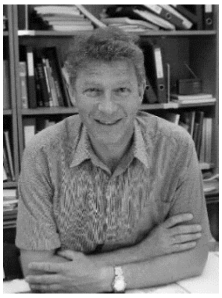

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ABSTRACT
This paper aims to investigate the range of approaches that are applied in available commercial computer programs as a first step towards better and more sustainable design. The questions which will be considered and answered are which theory is used by the software for calculating the time-dependent effects due to construction processes, strength development, creep, shrinkage and relaxation. I.e. Linear viscoelasticity for aging materials, Rate of creep method, or Effective E-modulus method. Which models are being used for describing creep, shrinkage and relaxation, which methods are being used for crack width calculation, and which subjective choices have been made by the software suppliers. The investigated computer programs are CSiBridge, Sofistik and Novaframe frequently used among practitioners, and the well-known Finite Element program systems Diana and Abaqus used for linear and non-linear analysis.

Key words: concrete structures, sustainable design, digitalization, serviceability limit states, time-dependent effects, construction processes,

1. INTRODUCTION

Today increased degree of digitalization is an important trend in the societal development, and related to civil engineering, structural analysis and design of concrete structures has become increasingly dependent building information models (BIM) and advanced computer programs. The engineers are using advanced software to find optimal solutions. Applying the advanced digital tools, the engineering community must be vigilant to avoid systematic errors, and that the analysis and dimensioning of complex structures becomes so “simple” to handle that the understanding of the structural behaviour may disappear.

Different commercial software applies often various solution methods when it comes to calculation of structural effects. Timedependent effects are often decisive in the Serviceability Limit State (SLS) design of concrete bridges, and which method that is being used to calculate them effects is not always clear without thorough studies of the software’s user manual and reference checks. The terminology is not always consistent, and two seamlessly similar analysis software’s might give the impression that they use of the same theory but in fact, they can be vastly different. To know which theory is being applied is important for the engineer to create a safe structure and do a sustainable design.

This paper is a part of a recently started PhD-project and presents the preliminary results of an investigation of the various approaches and methods for SLS analysis and design used by well-known computer programs as Novaframe, CSiBridge and Sofistik. The general-purpose finite element program systems Diana and Abaqus, frequently used both in academia and by practicing engineers are also included in the investigation.

2. APPROACHES APPLIED IN THE VARIOUS COMPUTER PROGRAMS

CSiBridge, Sofistik, Novaframe, Diana and Abaqus all have the possibility of modelling a staged construction analysis and account for time-dependent effects. To be able to calculate these effects, consistently, the software is highly dependent on the structures stress histories. These effects can be calculated i.e. by using different theories like: Effective E-modulus method (EEM), Age adjusted effective E-modulus method (AEMM), Linear viscoelasticity for aging materials and Rate of creep method. Diana and Abaqus can use all theories mentioned above but the software’s requires a great deal of knowledge among the engineers who use them, [1][2], and the authors have the impression that these are not widely used by engineering consultants at least not in bridge design. Sofistik doesn’t give a clear impression through the user and design manuals which theory is being applied or what user choices can be made. The authors have approached the company, but they seem reluctant to answer and refers to the manuals. Our impression is that the software can calculate creep behaviour by using the rate of creep method. This method can be explained as a simplified solution of linear viscoelasticity. Novaframe uses the rate of creep method as Sofistik but unlike the other software’s it is not a finite element program but applies 3D beam theory [7].

Concerning the cross-sectional analysis in ULS and SLS (dimensioning), the program systems also makes various choices that to a large extent may influence the results. A mapping of this is ongoing and will be presented at the seminar.

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Comparison of crack width predictions by three different models with experimental results on webs of beams


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ABSTRACT
This paper presents and compares three different approaches for calculating crack widths in reinforced concrete (RC) members subjected to predominant in-plane loading (plane stress). The crack widths predicted by the approaches are compared with experimental results and discussed.

Keywords: Crack width calculation methods, tension stiffening, compression field theory, nonlinear finite element method, pull-out model, tension chord model, cracked membrane model.

1. INTRODUCTION

A realistic calculation of crack widths in RC members is essential to ensure durability and to estimate the reserve capacity of existing structures. However, the knowledge on the crack behaviour for complex elements and/or stress states is still limited. This paper focuses on the evaluation of cracks in webs of beams. Three strategies with different levels of complexity are presented and compared to experimental results [1].

2. APPROACHES FOR CALCULATING CRACK WIDTHS

Linear-elastic approach: The stress resultants are computed based on a conventional beam analysis and linear-elastic (LE) compression field approach [2]. The crack widths are calculated considering now tension stiffening, using the Tension Chord Model (TCM) [3] and the Pull-Out Model (POM) [4] for $\rho_z \geq \rho_{min,z}$ and $\rho_z < \rho_{min,z}$, respectively (ρ=amount of reinforcement).

The other two approaches use nonlinear finite element analysis (NLFEA). Both approaches formulate equilibrium at stress-free cracks and consider tension stiffening in the reinforcement (concrete tensile stresses are considered for stiffness, but not for strength).

Compatible Stress Field Method (CSFM) [4]: A discrete modelling of the reinforcement is used and every reinforcing bar is treated as a tension chord with an effective concrete area. Tension stiffening and crack width calculation based on TCM and POM for $\rho_z \geq \rho_{min,z}$ and $\rho_z < \rho_{min,z}$, respectively.

Cracked Membrane Model (CMM) [2, 5]: The third approach considers a smeared modelling of the reinforcement. Tension stiffening (using TCM) and crack width calculations are coupled for the different reinforcing directions. It is assumed that the horizontal crack spacing in webs without a longitudinal reinforcement is the same as in vertical direction and that the crack spacing calculation remains valid also for $\rho_z < \rho_{min,z}$.

3. COMPARISON WITH TEST RESULTS

The results of the three presented approaches are compared to the measured crack widths in the webs of the T-beam tests TA11 and TA12 from Leonhardt and Walther [1]. The results in Fig. 1b-c show that the LE approach clearly overestimates the crack widths and gives conservative results. The other two approaches provide reasonable results for crack widths in the serviceability range (before yielding). The predicted yield load of the stirrups by CSFM and CMM differs from the experimental one; this leads to significant deviations between the experimental and the predicted crack width after the onset of yielding, which is, however, irrelevant for serviceability checks.

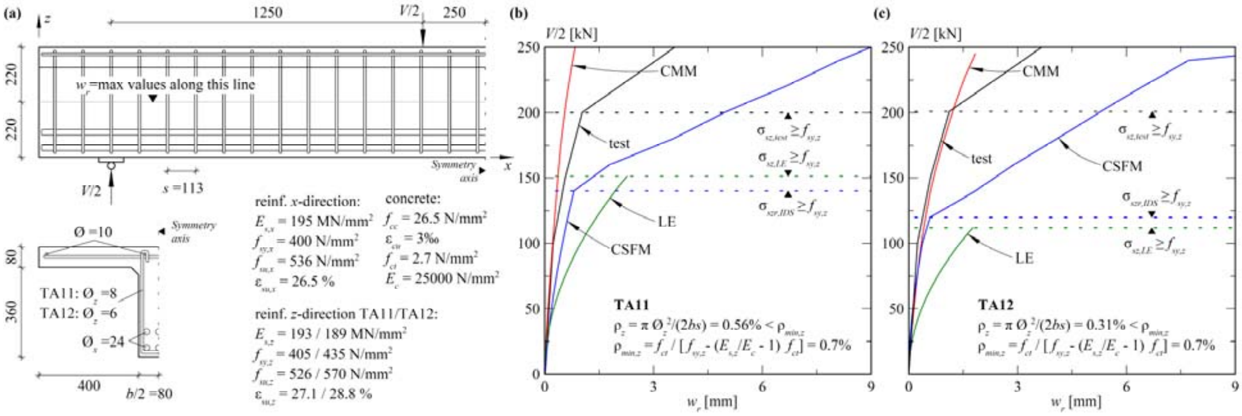

Figure 1 – Comparison with test results from Leonhardt and Walther [1]: (a) Test setup, reinforcement arrangement and material properties; (b) TA11; (c) TA12.

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