Session A7:

ALKALI-AGGREGATE REACTIONS

Alkali release from typical Danish aggregates to potential ASR reactive concrete

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ABSTRACT
Alkali-silica reaction (ASR) in concrete is a well-known deterioration mechanism affecting the long term durability of Danish concrete structures. Deleterious ASR cracking can be significantly reduced or prevented by limiting the total alkali content of concrete under a certain threshold limit, which in Denmark is recommended to 3 kg/m^3 $Na_2O_{eq.}$. However, this threshold limit does not account for the possible internal contribution of alkali to the concrete pore solution by release from aggregates or external contributions from varies sources. This study indicates that certain Danish aggregates are capable of releasing more than 0.46 kg/m^3 $Na_2O_{eq.}$ at 13 weeks of exposure in laboratory test which may increase the risk for deleterious cracking due to an increase in alkali content in the concrete.

Key words: Alkali-silica reaction, aggregate, alkali content, durability.

1. INTRODUCTION

ASR is a complex physical and chemical reaction between water, alkali in the concrete pore solution and reactive silica minerals in aggregates [1]. The reaction demands an alkaline environment which is found inside the concrete where a natural presence of free calcium hydroxide is found. The product of ASR is a hydroscopic gel which expands in volume resulting in internal pressure which can lead to extensive cracking in concrete structures. ASR cracking affects significantly the durability and the mechanical properties of the concrete. Furthermore, the formation of ASR cracking may enhance the effect of other concrete degradation mechanisms such as carbonation, frost action and reinforcement corrosion.

Preventing deleterious ASR in future concrete structures is an important aspect and can be achieved by controlling the three main components which are necessary to initiate and sustain

ASR in concrete. Controlling the total alkali, $Na_2O_{eq.}$, content is one way of doing so. To prevent ASR in concrete Danish regulations recommend that the total $Na_2O_{eq.}$ content, for a given concrete mix, is kept below 3 kg/m^3 of concrete [2]. However, this threshold limit does not consider an alkali contribution from external sources or internally from the aggregates. The results from previous Danish studies, connected to the construction of the Great Belt Bridge, indicated that some Danish aggregates potentially may release alkali when subjected to an alkaline environment, as in concrete [3]. This present study investigates the release of alkali from typical Danish aggregates according to the RILEM AAR-8 method [4].

2. MATERIALS

Three different Danish aggregate types were tested in accordance to the RILEM AAR-8 method: sand from marine deposits, sand from inland deposits and crushed granite from Rønne, Denmark. The fourth aggregate type, pure quartz sand, is applied as a reference material due to its simple composition and also to validate the test method. According to RILEM AAR-8 method fine material is tested in their natural grain size distribution where particles larger than 4 mm are discarded. Natural coarse aggregates are crushed, graded and combined in a specific grain size distribution described further in RILEM AAR-8 [4].

3. RILEM AAR-8 METHOD

RILEM AAR-8 is a draft method used for estimating the releasable alkali content of aggregates used in concrete. The aggregates of interest are prepared according to [4] and two test solutions are made to study the release of sodium and potassium separately. One solution has an initial high content of sodium and is used to observe the release of potassium. The other solution has an initial high content of potassium and is likewise used to test the release of sodium. Both solutions are saturated with calcium hydroxide which is similar to the concrete pore solution. The release tests are initiated by submerging the aggregates into the two different extraction solutions. The change in alkali content is measured at given predefined time steps by use of Inductively Coupled Plasma (ICP). RILEM AAR-8 suggests that the release of alkali is studied for both an exposure temperature of *38^{o}C and 60^{o}C.*

4. RESULTS AND DISCUSSION

Fig. 1 shows the release of alkali $Na_2O_{eq.}$ as a function of temperature and exposure time. The individual contribution from K_2O and Na_2O to the summarized $Na_2O_{eq.}$ content for Rønne granite after 13 weeks of exposure and a temperature of 60^{o}C is 61.5 % and 38.5 % respectively. The RILEM AAR-8 method suggest a total exposure time of at least 56 weeks or until reaching equilibrium of released alkali. This is not achieved for the present results thus the final releases for some aggregate types are expected to increase with increasing exposure time. Quartz sand and sand from inland deposits are absorbing alkali or having a very low rate of release. However, there seems to be an increase in alkali release from aggregates after 6 weeks and forward. The main composition of quartz is silicium and oxygen hence a release of alkali in the region of zero is expected. The large difference in alkali release for marine and inland sand is unknown, however the results might suggest that marine sand contains larger portions of alkali rich particles than inland sand thus releasing more alkali when tested. All aggregates are firmly washed in distilled water before testing which means that the difference in alkali release is not due to a surplus of marine salts from the surface of the aggregates. The inland sand is known to contain porous opaline or calcareous opaline flint which is highly ASR reactive. The formation of ASR in the test solution with inland sand is therefore plausible. An alkali consumption might take place which lowers the free alkali content in the extraction solution. This hypothesis is however not further studied. Rønne granite is releasing large portions of alkali which is expected due to the presence of alkali rich minerals such as mica and alkali feldspar.

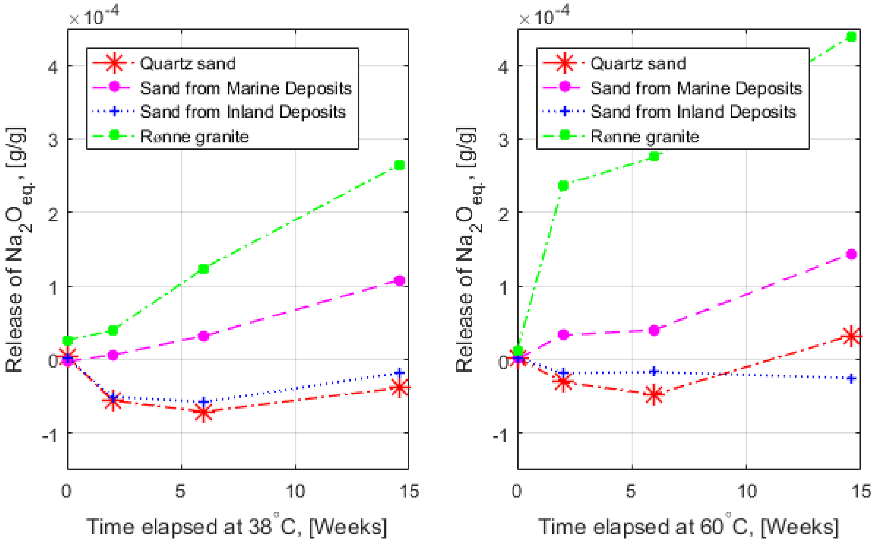

Figure 1 - Release of $Na_2O_{eq.}$ for three typical Danish aggregates and quartz sand according to RILEM AAR-8 method where two different exposure temperatures are used, The result is presented as grams of released alkali pr. gram of aggregate at 38°C (left) and at 60°C (right).

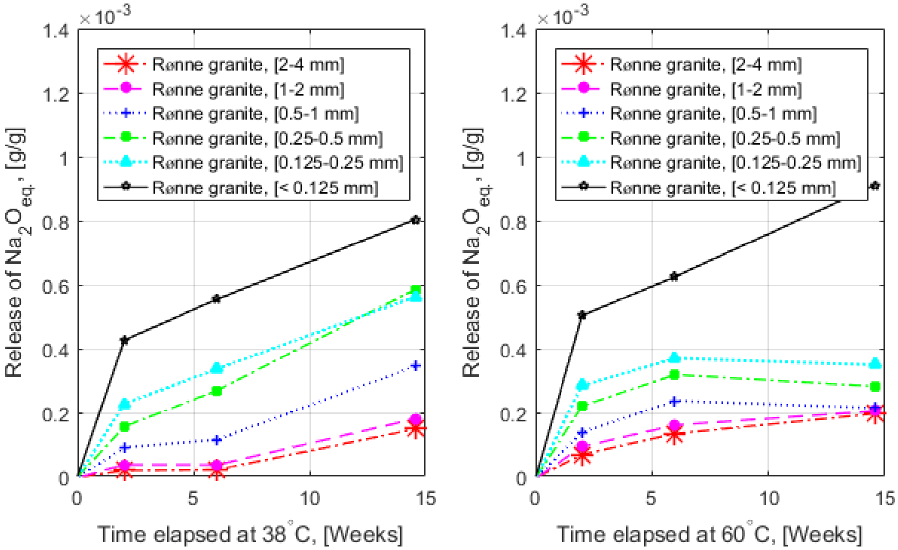

Figure 2 - Release of $Na_2O_{eq.}$ for 6 fractions of Rønne granite according to RILEM AAR-8 method where two different exposure temperatures are used, The result is presented as grams of released alkali pr. gram of aggregate at 38°C (left) and at 60°C (right).

Rønne granite, fraction 4-8 mm, is grinded and divided into six sieve grain sizes to investigate the rate of release and grain size. Fig. 2 shows the release of alkali for the six graduations of

Rønne granite as a function of temperature and exposure time. Smaller grain size are equal to a larger specific surface area. Fig. 2 show that smaller grain sizes result in considerable more release of alkali than larger grain sizes. The contact area between solution and grain is increased with larger specific surface area hence larger amount of released alkali was expected for smaller grain sizes.

Fig. 1 indicate that higher temperature results in higher release of alkali for 13 weeks of exposure time. However, Fig. 2 show contradictory results. Fig. 2 shows that a faster convergence is found for higher temperatures but the total amount of released alkali is larger for lower temperatures and is still increasing after 13 weeks measurement. This implies that exposure temperature is affecting the release of alkali and also that testing aggregates outside their natural temperature range might give a false image of the releasable alkali content. Apparently, increasing the temperature is not always a conservative approach and one should also consider dynamic temperature exposure for more realistic extraction conditions.

The release curves presented in Fig. 1 is used to estimate the expected contribution of alkali to one cubic meter concrete. RILEM AAR-8 method suggests that a standard concrete consist of 700 kg/m^3 fine and 1050 kg/m^3 coarse aggregates. This yields an expected contribution of 0.46 kg/m^3 $Na_2O_{eq.}$ for Rønne granite at an exposure temperature of 60 ^oC. This representation of the alkali release is directly comparable to the alkali threshold limit of 3 kg/m^3. It should be noted that this estimation is based on 13 weeks measurement where the experiments have not reached equilibrium and are still in the process of releasing alkali. However, it may be questioned if the same rate of alkali release could be expected inside the concrete which is a stationary condition surrounded by cement paste. The aggregates tested by the RILEM AAR-8 method are surrounded by liquid implying that the release of alkali may be accelerated for the test method yielding an overestimation of the potential release rates in the concrete**.**

5. CONCLUSION

The scope of this study was to test whether one can expect a release of alkali from typical Danish aggregates or not. RILEM AAR-8 was applied and the results indicate that Rønne granite is capable of releasing more than 0.46 kg/m^3 alkali in only 13 weeks of extraction. The release of alkali from aggregates is dependent on the specific surface area where a larger specific surface area is enhancing both the rate of release and the total release of alkali. The effect of having different exposure temperatures is indicating that it is not always conservative to test aggregates at higher temperatures. This draws the conclusion that aggregates should be tested in a realistic temperature domain for which suits the location of where the aggregates are supposed to be used.

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New method ”Texas.dot” for continuous measurement of potential expansion of ASR reactive concrete

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ABSTRACT

The residual expansion of potential alkali-silica reaction (ASR) concrete is a critical parameter when evaluating the most appropriate remedial action for a concrete structure affected by ASR. A relatively new method in this paper called “Texas.dot” for residual expansion measurements of concrete specimen surrounded by a synthetic pore solution is tested. The method’s name has the origin to Texas A&M Transportation Institute which has developed the method [1, 2]. The equipment consists of a dilatometer with a LVDT connected to a computer. The advantages are 1) continuous monitoring of concrete expansion, and 2) performing test with variable alkali content and/or temperature in the liquid around the test specimens. Based on this method concrete specimens with varying alkali content have reached an expansion above 1 ‰ within 31 days.

Keywords: Mix Design, Testing, Alkali-silica Reaction, Expansion Measurement, Dilatometer, Synthetic Pore Solution.

1. INTRODUCTION

ASR can cause destructive expansions in concrete structures if the exposure conditions are unfavourable. Therefore it is important to investigate the influence of moisture, temperature and alkali content on a potential ASR-concrete. This includes exposure to water and extra alkalis from the surroundings such as de-icing salt (NaCl), seawater or other alkalis sources. If all these parameters are above a certain critical level ASR will cause expansion in the concrete structure which can have influence on the durability of the structure [3].

The potential ASR expansions have been investigated by a relatively new expansion test called ”Texas.dot”. In the experiment four concrete specimens are tested: Three laboratory made cylinders and one drilled core from a Danish bridge foundation. The specimens are installed in a dilatometer where the specimens are surrounded by a synthetic pore solution. The specific alkali content for the pore solution inside the tested concretes is unknown. For the achieved results obtained by expansion due to ASR, the expansion regarding Danish experience is considered as reactive destructive when the value exceeds 1 ‰ [4].

2. MATERIALS AND METHOD

2.1 Materials

The expansion tests are conducted on three laboratory casted cylinders, 100 mm in diameter and 200 mm in height, and a drilled concrete core, 83 mm in diameter and 197 mm in height. The core was drilled from a bridge foundation where no sign of ASR cracking is detected. For the laboratory casted cylinders Danish reactive sand, Øde Hastrup, and non-reactive coarse aggregate, Blå Rønne granite, are used. The Rapid cement had a Na_2O eq. content of 0.64 %. In two casted cylinders the total Na_2O eq. content is raised to 3.5 kg/m^3 of concrete by adding NaOH to the mix water. In the third cylinder the total Na_2O eq. content is 2.4 kg/m^3 of concrete. The cylinders are cured for approximately 24 hours before installed in the experimental setup.

2.2 Experimental setup

Fig. 1 and Fig. 2 illustrate the experimental setup. The experimental setup consists of dilatometer with a LVDT connected to a computer. The dilatometer is placed in a water bath at 38 ^{0}C. Inside the dilatometer the specimen surfaces are exposed to a synthetic pore solution. Table 1 shows the composition of the synthetic pore solution.

Table 1 - Composition of the synthetic pore solution.

	NaOH	K_2SO_4	KOH	$Ca(OH)_2$
Concentration [g/L]	16.0	7.0	18.0	0.15

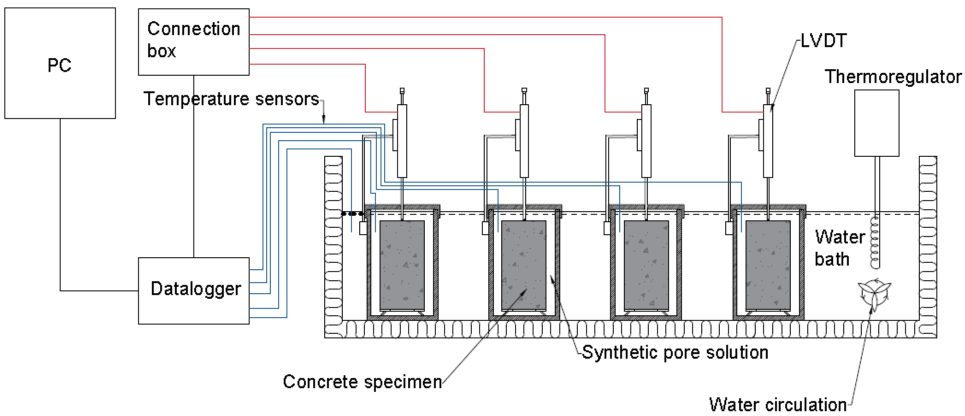

Figure 1 - Experimental setup with four dilatometers for Texas.dot.

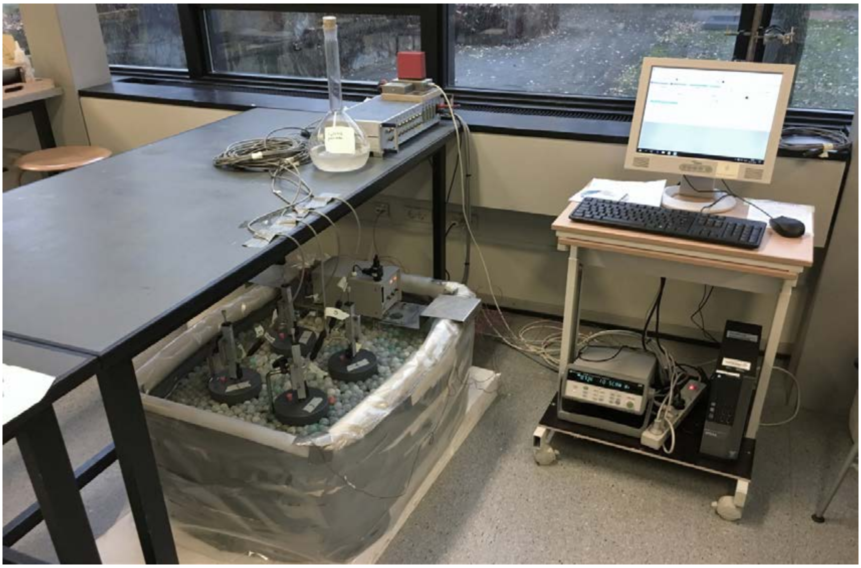

Figure 2 - Experimental setup for Texas.dot

2. RESULTS AND DISCUSSION

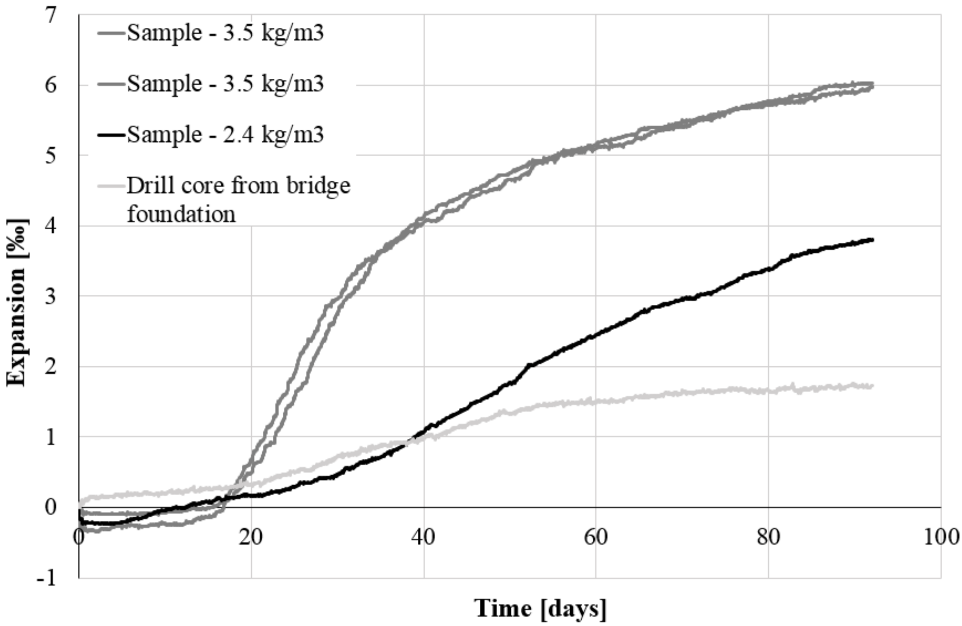

Figure 3 - Expansion over time at 38 ^{0}C. Na_2O eq. in kg/m^3 of concrete.

Fig. 3 shows the measured expansion as function of exposure time. All specimens exceed the Danish recommended critical expansion limit of 1 ‰ after 19, 23 and 31 days, respectively.

Specimens with initial Na_2O eq. content of 3.5 kg/m^3 reach significant higher expansion levels faster than the specimens with Na_2O eq. content of 2.4 kg/m^3. This observation is expected since the total Na_2O eq. content in the specimens with added NaOH exceeded the most commonly recommended threshold value of 3.0 kg/m^3 Na_2O eq..

It is not expected that the specimen without added NaOH, i.e. an alkali content beneath the recommended 3.0 kg/m^3 eq. would exceed the critical limit of 1 ‰, and especially not with an expansion of approximately 4.2 ‰ after 93 days. The alkali content in the synthetic pore solution around the specimens is produced from a general pore solution recipe but not based on the knowledge of the actual composition of the pore solution inside the concrete. This means that it is uncertain if alkali level in the synthetic pore solution is similar to the alkali level in the pore solution inside the tested concrete, i.e. uncertain leach-proof condition.

The expansion result for the drilled core from the bridge foundation exceeds the critical limit of 1 ‰ after 23 days exposure. Therefore the concrete in the foundation possibly will develop damaging ASR over time if the exposure conditions are unfavourable.

3. CONCLUSION

The advantages for this method are continuous monitoring of concrete expansion and performing test with a synthetic pore solution around the test specimens under constant temperature. If the synthetic pore solution has equal chemistry with the pore solution inside the tested concrete a leach-prof condition is obtained. A disadvantage is that the concrete pore solution must be measured beforehand. Based on this method concrete specimens with varying alkali content have reached an expansion above 1 ‰ within 31 days, but the leach-proof condition is uncertain for this test.

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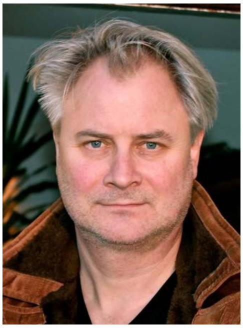
The RILEM approach to mitigate alkali aggregate reactions (AAR) in concrete



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ABSTRACT
Development and assessments of test methods, to avoid deleterious AAR in concrete, have been the focus of three previous RILEM Technical Committees (TC) in the period 1989 to 2014. The 4th RILEM TC on AAR, TC 258-AAA, was established in 2014, and will until 2019 focus on the following Work Packages: WP1- Performance based testing concepts; WP2 - Relationship between results from laboratory and field and the establishment of field exposure sites; WP3 - Testing of potential alkalis released from certain types of aggregates and measurement of internal concrete alkali content; WP4 - Verification of alkalis released from aggregates.

Key words: Alkali Aggregate Reactions, Cement, Concrete, Durability, Testing.

1. INTRODUCTION

1.1 Alkali Aggregate Reactions

Alkali Aggregate Reactions (AAR) can be defined as chemical reactions between the alkalis hydroxides (sodium and potassium) in the pore solution of concrete and certain minerals in the aggregate. The product of the AAR is a hygroscopic gel that expands upon hydration and may introduce cracking in the surrounding concrete, thereby reducing the mechanical properties of concrete and structure service-life and increasing cost for society. The incubation time needed before AAR damage starts ranges from a few months to several decades, much depending on aggregate type, binder type and exposure climate.

1.2 Background of the previous work in RILEM

The 3rd RILEM[1] Technical Committee (TC 219-ACS)[2] on AAR was established in 2007. The TC ended its activities in 2014. Initial work, since 1989, by the first two RILEM TCs, 191-ARP and its predecessor TC 106, concentrated on the assessment of the alkali-reactivity potential of aggregates. However, in recognition that damaging expansion involves interaction between all the main components of a concrete mix, the 3rd TC 219-ACS also focused on the assessment of the effect of the cement/binder on AAR, i.e. performance testing. It was prepared several documents/recommendations, published as:

- **AAR-6.1** (diagnosis & prognosis) – RILEM note book publication [1]
- Literature review on performance testing - SINTEF Report [2]

The central recommendations were published in a special issue of “Material and Structures” [3]:

- **AAR-0** Overview Guide
- **AAR-1.1** Petrographic Examination Method
  - Rapid classification of aggregates based on point counting in thin sections.
- **AAR-2** Accelerated Mortar Bar Test (AMBT; 80°C, 1N NaOH, 14 days)
  - Rapid classification of aggregates based on mortar bar expansion.
- **AAR-3** Concrete Prism Test (CPT; 38°C, 100 % RH, 52 weeks)
  - Potential alkali-reactivity of aggregate combinations based on expansion of concrete prisms (Application 1: AAR-3.1, high alkali content). The method also allows determination of the alkali threshold of an aggregate combination (Application 2: AAR-3.2)
- **AAR-4.1** Accelerated Concrete Prism Test (ACPT; 60°C, 100 % RH, 20 weeks)
  - Potential alkali-reactivity of aggregate combinations based on expansion of concrete prisms with a high alkali content.
- **AAR-5** Screening test for carbonates (AMBT; 80°C, 1N NaOH, 14 days)
  - Rapid screening test for alkali carbonate reactive aggregates.
- **AAR-7.1, 7.2 and 7.3** (ASR specification)
  - ASR specifications for mitigation of ASR.

The "petrographic atlas" (**AAR-1.2**) was published separately as a RILEM book in “Material and Structures” [4].

2 THE NEW RILEM TC 258-AAA – link to a Norwegian R&D project

The new, 4th TC on AAR (TC 258-AAA)[3] started in October 2014, and is chaired by Professor Børge Johannes Wigum (HeidelbergCement Northern Europe, Norway), and the secretary is Dr Jan Lindgård (SINTEF, Norway).

The main purpose of this new RILEM TC is to develop and promote a performance based testing concept for the prevention of deleterious Alkali Silica Reactions (ASR) in concrete (the issue Alkali Carbonate Reaction, ACR, is not included in this TC). Strong emphasis will be put on publishing the new methods develop as RILEM recommendations and on the implementation these methods and recommendations as national- and international standards.

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[1] RILEM – International union of laboratories and experts in construction materials, systems and structures

[2] RILEM Technical Committee 219-ACS. Alkali-Aggregate Reactions in Concrete Structures (2007-2014)

[3] Avoiding alkali aggregate reactions in concrete - Performance based concept (2014-2019).

2.1 The Work Packages (WPs)

WP1 - Performance testing and accelerated testing in laboratory.
Development of performance test methods to document the mitigating effect of supplementary materials such as fly ash or slag etc., or the use of a low alkali levels in the mix. Thus, a much wider selection of aggregates can be used safely while increasing the sustainability of the concrete and aggregate industry. Although some performance tests have been in use for many years, there is still a necessity to improve and validate more reliable test methods, including arranging international inter-laboratory trials. WP1 is headed by Dr Terje F. Rønning, (*HeidelbergCement Northern Europe, Norway*).

WP2 - Performance testing and laboratory vs. field; Exposure site.
An important additional tool in validation of the performance testing concept is to make an assessment of the link between accelerated laboratory results and behaviour of these concrete mixtures in real field structures. One main objective is to establish a link between outdoor exposure sites dedicated to AAR investigations and located in different parts of the world in order to generate an international database on the effect of environmental conditions on the kinetics of AAR. WP2 is headed by Prof. Benoît Fournier (*Université Laval, Québec, Canada*).

WP3 - Performance testing; Assessment of detailed alkali inventory in concrete, including internal alkali release from aggregates, recycling of alkali and external alkali supply.
One important "missing link" is how to measure the amount of potential alkalis that might be released from various aggregate types under accelerated laboratory conditions. It is the intention to finalise and validate such a test method. It is also of importance to evaluate the potential internal alkali recycling in the concrete which in some instances have been reported, in addition to assess any alkali contribution from external sources. WP3 is headed by Dr Esperanza Menéndez Méndez (*Institute of Construction Science, "Eduardo Torroja" (CSIC), Spain*).

WP4 - Verification of alkalis released from aggregates. Results of alkali release from aggregates under accelerated laboratory tests need to be verified and calibrated to what happens in real structures. The aim is to compile results from exposure sites and concrete structures worldwide in order to assess the "true" level of alkali released from various aggregates. WP4 is headed by Professor Børge Johannes Wigum, (*HeidelbergCement Northern Europe, Norway*).

2.2 Norwegian research project (2014-2018)

The Norwegian R&D project 236661 "ASR – Alkali-silica reaction in concrete – reliable concept for performance testing" (*abbreviated: "KPN-ASR"*) (*http://www.sintef.no/prosjekter/alkalireaksjoner-palitelig-konsept-for-funksjonspr/*) covers the same topics as RILEM TC 258-AAA. For several sub-topics, a close research co-operation with members of the RILEM TC, in particular leading researchers in Northern America, is established. Moreover, relevant findings in this project are communicated to members of RILEM TC 258-AAA on TC meetings.

3 STATUS OF RILEM TC 258-AAA SPRING 2017

RILEM TC 258-AAA has a wide international membership, which helps to promote the eventual international use of RILEM methods and recommendations. Physical meetings twice a year will still be the centre of its activities and wherever possible this is co-ordinated with major relevant international conferences to facilitate attendance. Members around the world that are not able to travel to the meetings are following the discussions through the extended minutes of the meetings. All relevant documents, including the minutes from the meetings, are available for TC members at the RILEM internal website.

In WP1, the main work is concentrating on the performance testing concept using a 38°C Concrete Prism Test (CPT; based on the Norwegian CPT [5]). Previous CPT procedures (e.g. RILEM AAR-3) included testing of alkali-reactivity of aggregate (AAR-3.1) and the determination of the alkali threshold of an aggregate combination (AAR-3.2). The performance testing concept includes applications for performance assessment of combinations of aggregates and cement/binders at various or specific alkali contents.

The initial work in WP2 has included casting of about 80 concrete cubes (300x300x300 mm) for outdoor storage and monitoring at 10 different exposure sites in Europe and North America (Portugal (2), France, Norway (2), Iceland, Germany, Canada (2) & Texas). The concrete mixtures included ordinary Portland cement and addition of fly ash (20 & 30%), along with control mixtures.

In WP3, a Round-Robin test has been initiated in order to evaluate the draft test procedure for measuring potential amount of releasable alkalis from aggregates. In addition, initial work in WP3 includes the preparation of an outline literature review regarding the alkali inventory in concrete.

In WP4, Post-doc activities in the Norwegian "KPN-ASR" project have developed a draft test method to determine the alkali level in concrete facilitating activities to verify the level of potential alkali release from aggregates in real concrete.

In the past 75 years, we have struggled to understand, control and prevent damage from AAR since it was first reported in concrete. This continuing series of RILEM Technical Committees has helped to harness international co-operation in this struggle for the last nearly 30 years and will continue this work in the future.

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Challenges related to structural modelling and assessment of concrete structures affected by alkali-silica reactions

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ABSTRACT

Alkali-Silica Reaction (ASR) in concrete is a chemical process between dissolved alkali and hydroxyl ions from pore solutions in the cement paste, and reactive (amorphous) silica within the aggregates. The product of the chemical reaction is an alkali-silica gel that will increase in volume by water absorption. The result is expansion of the concrete and degradation of material properties. The deteriorating impact of ASR on concrete structures might be reduction in load bearing capacity, durability and service life.

In this paper, the effects of ASR on concrete material are reviewed, and the challenges related to modelling the effects of ASR on the structural scale are addressed. Finally, plans for structural assessment on parts of an existing bridge in Norway are announced.

Key words: Concrete, Alkali-Silica Reactions, Modelling, Structural Assessment

1 INTRODUCTION

Alkali-Silica Reaction (ASR) in concrete is a chemical reaction between alkali and hydroxyl ions (present in the pore solution) and reactive silica within the aggregates, where the product is a hydrophilic gel that will expand by water absorption. The rate of gel production and total gel volume are dependent on the alkali content, moisture, temperature and type of aggregates. As the gel is swelling within a confining matrix, a pressure develops in the gel that is balanced by tensile stresses in the cement paste, eventually leading to micro cracking. Considering a

Representative Volume Element (RVE) of the concrete (a size much bigger than the heterogeneity, but much smaller than the dimension of a typical structure), the ASR will cause expansion (strains) and degradation of material properties: Young's modulus, tensile strength and compressive strength. The spatial variation of ASR induced strains on the scale of a RVE also introduce stresses. From a structural point of view, the material expansion cause displacements that might introduce large inner forces if boundary conditions or other structural parts restrain the displacements. In all aforementioned levels, ASR cause eigen- stresses and strains plus deformation discontinuities as micro or macro cracks. Each level, from meso to a structural level, is important to investigate in order to create a comprehensive model for structural assessment of concrete structures.

The main objectives of this paper is to a) review the mechanical effects of ASR on concrete material level, b) address the challenges related to modelling the effects of ASR on the structural scale and c) announce plans for a structural assessment, with increasing level of sophistication, on parts of an existing bridge in Norway.

2 EXPERIMENTAL EVIDENCE ON THE EFFECT OF ASR

2.1 Anisotropic expansion

Larive [1] observed larger expansion in the casting direction than perpendicular to the casting direction of specimens under load-free conditions. The casting direction may influence the orientation of aggregates and pores within the concrete. This will in turn determine the orientation of the gel pressure load and location of weaker zones prone for micro cracking, explaining the intrinsic anisotropic expansion observed by Larive [1].

The expansion of ASR affected concrete is also dependent on the state of stress, where the expansion is reduced in the restrained/loaded direction and transferred to the stress free directions [2, 3]. Multon [2] concluded that the volumetric ASR induced strain is independent of the state of stress, but recent experiments by Bishnu [4] show reduced volumetric strain (induced by ASR) under tri-axial stress conditions. This might be explained by a) an increased diffusion of gel into the surrounding paste with increasing hydrostatic pressure and b) a reduced damage (micro cracking) of the concrete skeleton due to a triaxial confinement from the external load.

The total expansion/strain of an RVE can be split into elastic (recoverable) strains and inelastic (unrecoverable) due to micro cracks, which are displacement discontinuities on a subscale (meso-level). The deterioration of the material should be related to the micro cracking, which leads to the discussion about ASR induced strain as an appropriate damage/degradation parameter for mechanical properties.

2.2 Degradation of mechanical properties and expansion as a degradation parameter

Esposito [5] carried out an extensive literature survey on the relation of ASR-induced expansion and mechanical properties based on accelerated tests of concrete specimens under free expansion, and from her study, it is clear that compressive strength, tensile strength and Young's modulus all degrade with increasing expansion. This relation is an important input for structural models dealing with the effect of ASR. However, experiments performed by Giaccio [6] of specimens with different reactive aggregates showed that even for the same level of expansion, the degradation of the mechanical properties were different. The reason was explained by the different cracking morphology caused by the different aggregates; the deterioration is mainly attributed to micro cracks in the cement paste and fissures at the Interfacial Transition Zone

(ITZ) rather than the micro-cracks in the aggregates. The different cracking morphologies were related to the location of the gel formation. The aggregates with a reaction rim caused more damage in the cement paste and ITZ compared to aggregates with internal gel production.

One reason for this might be gel flow (in the case of a formed reaction rim) into initial flaws and created micro cracks in the cement paste, which enhance the damage when the gel exert a pressure inside these fissures. Whereas swelling of gel inside an aggregate cause stresses on the cement matrix through the expansion of the aggregate, and as result cause less damage in the cement paste and ITZ.

Further, the state of (average) stress on a RVE of concrete will also influence the degree and orientation of the deterioration of mechanical properties. In analogue to the expansion transfer, the mechanical properties will degrade more in the less compressed directions due to the preferred orientation of micro cracks parallel to the most compressed direction.

In addition, it is believed that stress relaxation in the cement paste around the reactive aggregates will reduce the pressure exerted by the gel and thus the stresses in the cement paste, which in turn, reduce the degree of micro-cracks. This creep effect is a time dependent phenomena, indicating that the rate of gel expansion is important for the development micro-cracks and thus the mechanical properties.

Based on the preceding discussion, one can argue that the ASR induced expansion (only), is not an appropriate parameter to describe the damage of concrete and thus the deterioration of mechanical properties. The ASR induced expansion of a RVE is partly viscoelastic deformation and partly displacement discontinuities (micro cracks), in which the latter results in reduction of the mechanical properties. Even if the overall ASR induced expansion of two specimens are the same, the degree of micro cracks might be different, due to e.g. type of aggregate, ASR kinetics and stress history. Thus, a general relation between ASR induced strain and degree of deterioration of mechanical properties is too ambiguous.

3 CHALLENGES RELATED TO STRUCTURAL MODELING

Many of the models that have been demonstrated and/or validated on a structural scale model the concrete as a homogenous continuum and include the effects of ASR as induced strain, e.g. [7], [8], [9] and [10]. However, as stated by Alnaggar et al. [11], the disadvantage of all aforementioned models is the inability to simulate micro-crack patterns and crack distribution due to ASR, which limits their ability to predict the deterioration of ASR on mechanical properties. In addition, it also limits the ability of such models to explain the relation between concrete state of stress and expansion. Constitutive modelling of homogenized ASR affected concrete is challenging because ASR is a phenomenon on a sub-scale of concrete material level. Consequently, assumptions on relations between e.g. states of stress, ASR induced expansion and change of mechanical properties are necessary.

The current state, i.e. displacements, mechanical properties, stresses etc., of an ASR affected structure is a result of its history of temperature, moisture, alkali-leakage, stresses/forces and strains/displacement, showing the complexity of modelling ASR.

Field measurement of mechanical properties and displacements can be used to “diagnose” the current state. However, numerical simulations [12] of unloaded concrete cores, during swelling, showed rapid cracking due to the vanishing external stresses. This indicates that mechanical properties obtained based on specimens extracted from a structure might not represent the in-situ properties, increasing the challenges to perform a proper structural assessment.

4 STRUCTURAL ASSESSMENT OF AN ASR AFFECTED BRIDGE IN NORWAY

In Norway, there is a need for an increased knowledge about the effects of ASR on the structural level in order to do proper structural assessments of an aging infrastructure. There are several bridges suffering from ASR, e.g. *Elgeseter bru, Tromsøbrua and Tjelsundbrua,* in which the last one will be used as a case in the present PhD project.

It is envisioned to perform a structural assessment of parts on *Tjelsundbrua*. First, a simplified assessment based on the current state of the structure will be carried out. This will be done in a two-step procedure: 1) calculation of change of load actions due to ASR induced displacements and 2) evaluation of cross-sectional load actions against corresponding capacities including effect of ASR. The current configuration of the bridge, obtained from a 3D scan, and tests of mechanical properties of cores extracted from the structure will form the basis for the current state of the structure. Finally, more comprehensive finite element models will be evaluated and applied to predict the evolution of ASR in time.

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Session A8:

SPECIAL APPLICATIONS

Concrete mix designs for tunnel end plugs in nuclear waste repository

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ABSTRACT
A unique self-compacting concrete was developed for use in long-term nuclear waste repository tunnel end plugs. For stability of bentonite clay in the repository for thousands of years, the pH of the concrete leachate is required to be below 11. Two applicable mix designs were developed. A final mix design having a ternary binder composition was selected to be use in a full-scale demonstration tunnel end plug at 450 metres underground. Successful demonstration was performed after adjusting the mix design to fulfil the requirements of a batching plant production.

Keywords: Low-pH concrete, nuclear waste repository, mix design, full-scale demonstration

1 INTRODUCTION

One of the world first long-term nuclear waste repositories will be constructed into Olkiluoto bedrock in Finland [1]. Concrete plugs are deployed as hydraulic and mechanical barriers in repository tunnel sealing at a depth of approximately 450 meters. The service life of the concrete plug is 100 years, yet the plug is one component of the Engineered Barrier System (EBS) that should protect the environment for hundreds of thousands of years during the storage of spent nuclear fuel. The tunnel end plug is a reinforced concrete structure, having dimensions of approximately 4.5 x 6.5 x 6 meters with a total volume of approximately 150m^3 [2]. Concrete in the plug should be self-compacting because only minimal consolidation is possible during casting.

Much of the safety in the long-term nuclear waste repository relies on the stability of bentonite buffer. High pH of traditional concretes endangers the stability of bentonite buffer and therefore the functioning of the total EBS. Low –pH mix designs have been developed to overcome the problem [3].

Two potential mix designs for tunnel end plugs were developed. The first one had an identical binder composition of the Swedish tunnel end plug [4], utilizing Ordinary Portland cement and silica fume. The mix design was labelled as binary mix design. The other developed mix design was named as ternary mix design due to its three-part binder: Ordinary Portland cement, silica fume and fly ash.

2 MATERIALS AND METHODS

Materials and testing methods have been presented in NCR 2014 [5].

3 RESULTS

3.1 Laboratory tests

At the beginning in the Finnish development mix designs were developed in laboratory. This work has been presented in NCR 2014 [5].

3.2 Demo

After VTT laboratory mixture development and performance verification, the final binary and ternary recipes were re-produced in the ready-mix supplier's laboratory and then at their batch plant (Rudus Oy). Three on-site mock-up castings were done at Posiva's ONKALO repository, to verify mix design proportions and construction techniques before the actual plug casting. From these mock-up trials (Hartela Oy), additional quality control samples were also taken for concrete performance verification of strength, watertightness and pH leachate.

Based on these factory and mock-up demonstrations, the final recipe was selected to be the Ternary mix for casting the 150 m3 full-scale tunnel demo end plug. Mix design of the full-scale tunnel end plug with is presented in Table 1. It was decided that the first 20 m3 and last 20 m3 would be cast with a maximum aggregate size of 16 mm, due to the conjected reinforcement around the plug circumference (Figure 1), while the center section of the plug had maximum 32 mm aggregate.

Table 1 - Demo concrete recipes.

		16 mm	32 mm
CEM I 42,5 MH/SR/LA (kg/m3)		107	107
Silica Fume (kg/m3)		85	86
Fly Ash (kg/m3)		89	90
Quartz Filler (kg/m3)		115	116
Local Aggregate (kg/m3)		1810	1850
Effective Water Content (kg/m3)		137	123
Water/Binder Ratio		0,49	0,44
Strength, MPa	91 d	78,5	87,0
Water penetration, mm	91 d	2,0	2,2
pH, in ground water	91 d	10,9	10,9

Figure 1 - Conjected reinforcement in demo structure. (Reinforcement bars are 32 mm diameter)

The full scale concrete tunnel end plug was cast in two sections during July and September 2015. For each casting, there were 20-25 truckloads of 4m3 each delivered at 45 minute intervals over a 10-hour period to 450 m underground. A total of 172 m3 low-pH concrete was used, comprised of 78 m3 in the first plug section and 94 m3 in the second section. The concrete was placed by pumping at increasing high intervals in the plug. No mechanical vibration was used. Figure 2 presents casting work underground.

Figure 2 - Casting demo structure underground.

The plug sections were instrumented with 67 sensors to measure early age and long-term performance, including temperature measurements. Prior to placement, air content, density and temperature and slump flow were measured at both the factory and on-site to ensure consistency and quality. The formwork pressure was measured during emplacement. Concrete samples were taken for measurements of compressive strength, watertightness and pH leachate at 28, 91 and 365 days after casting. 1 m3 quality control cubes were also cast underground beside the plug to be used for further quality control sampling for each mix (16 and 32 mm aggregate sizes, for both plug section castings).

The average slump flow during quality control testing underground was 600mm and air content 2.0%. The maximum temperature of the plug after casting was 43°C at approximately 3 days. No problems were encountered during the casting procedure, with very uniform self-compacting concrete achieved having a very low water-content and meeting demanding conditions for casting. Performance of this end plug is still monitored continuously.

4. CONCLUSIONS

Two mix designs were developed for nuclear waste repository tunnel end plugs. The mix designs were developed using two different mix design approaches. Both mix designs were optimized with laboratory trials and then within factory trials and mock-up castings prior to the full-scale plug emplacement.

Both developed mix designs fulfilled the performance requirements. The mix design with ternary binder (cement, silica fume and fly ash) was selected on full-scale underground demonstration, using two different maximum aggregate sizes of 16 and 32 mm. The mix design was further adjusted, based on batching plant trials. Successful full-scale tunnel end plug demonstration was performed at 450 metres deep underground during July and September 2015, with casting of 172 m3 of self-compacting low-pH concrete.

The results of this work are being used by Posiva Oy, the radioactive waste management organization of Finland, to demonstrate to the safety regulatory authority their readiness to being operation of a spent fuel repository in the early 2020s. The results were the first full-scale in-situ demonstration of the engineered barrier system (EBS) construction at Olkiluoto. The outcomes of the plug concrete development and construction are used to finalize the tunnel end plug design and construction methodologies for safe and reliable management of nuclear waste for permanent geological disposal.

ACKNOWLEDGEMENT

The research leading to these results has received funding from the European Union's European Atomic Energy Community's (Euratom) Seventh Framework Programme FP7/2007-2013 under Grant agreement no 323273, the DOPAS project. [6]

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Reduction of radon gas in concrete – effects and evaluation of effective dose

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ABSTRACT
The second largest cause of lung cancer is related to radon (^{222}Rn) and its progenies in our environment. Building materials, such as concrete, contribute to the production of radon gas through the natural decay of ^{238}U from its constituents. The Swedish Cement and Concrete Research Institute (CBI) has examined two identical concrete recipes where only an additive, X1002 Hycrete hydrophobant corrosion inhibitor was added to one of the recipes as a mean to lower the radon exhalation rate. Measurements were performed with an ATMOS 33 ionizing pulsation chamber at four different occasions for each recipe during 12 months. The results indicate a reduction of the exhalation rate by approximately 30-35 %, meaning roughly 2 mSv per year decrease in effective dose to a human.

Key words: Additives, Admixtures, Building material, Mix design, Radon, Sustainability, Testing.

1 INTRODUCTION AND BACKGROUND

1.1 Ionizing radiation and health

The second largest cause of lung cancer is due to ionizing radiation generated by radon and its progenies [1]. The EU legislation, its Construction Products Regulations [2] and the EU´s Basic Safety Standards (BSS) directive [3] currently put a strong focus on ionizing radiation of building materials and safety for the public. In 2018 the implementation of the current BSS should be fulfilled in the European countries national legislation.

The Swedish bedrock contains in some cases high levels of ^{238}U [4]. Bedrock, as crushed aggregate, often constitutes 70-80 weight % of a concrete recipe. In the natural decay of ^{238}U the progeny ^{226}Ra reduces to ^{222}Rn. ^{222}Rn is more commonly known as radon. Radon is a noble gas that is easily breathable and as such it is inhaled and continuous alpha decay of ^{222}Rn to ^{218}Po as well as ^{214}Bi liberates alpha-particles that stick to the internal organs and cause immediate ionization of human cells within the body through their continued decay [5].

The idea rose from observing an experiment using a hydrophobant admixture where water droplets stayed on the concrete surface. The hindrance to allow transport of water droplets into the concrete, may also have a direct effect of the diffusion rate within the concrete, since diffusion is considered being the driving force of radon gas exhalation?

1.2 Regulations for buildings, earlier works and current study

In Sweden, a national threshold level of radon gas is set at 200 Bq/m^3 for habitants in dwellings, equivalent to 6-7 mSv per year effective dose [6].

Measures to reduce the exhalation rate of radon from concrete as a building material have only to some extent been studied. Chauhan & Kumar [7] showed the potentials of reducing the radon gas exhalation rates from concrete using rice husk. Also Yu et al. [8] and Taylor-Lange et al. [9] demonstrated the possible influences of reducing the exhalation rate of radon gas from different concrete surfaces using alternative binders, such as fly ash or metakaolin.

In this research a study of an additive (hydrophobant), X1002 Hycrete, as an alternative to reduce the radon gas exhalation from the concrete surface was examined.

2 METHODOLOGY

2.1 Radon exhalation rate and radon gas measurements

The principle makes use of a “closed system” as radon builds up within a sealed alumina container [10]. The radon gas exhalation rate, E (Bq/m^{2}h) can be calculated knowing the initial conditions of the radon gas concentration in a given space (volume). The equation for the linear regression model [11] can be described as:

$$E = \frac{\{(\mathrm{C}-\mathrm{Co}) \times V\}}{A\, x\, t} \tag{1}$$

where: E = Exhalation of radon gas (Bq/m^{2}h), C = Concentration of radon gas measured by the radon gas monitor (Bq/m^3), C_0 = Background concentration of radon gas at initiation (Bq/m^3), t = time of duration (h), A = Effective surface area of the sample (m^2), V = Volume of the container including hoses.

The final calculation of radon gas in air within a room is according to guidelines in the Swedish legislation, Swedish National Board of Housing, Building and Planning [12];

$$Cm = \frac{1}{(\lambda+n)}\, x\, \frac{E\, x\, A}{V} \tag{2}$$

Where, Cm = concentration (Bq/m^3), λ = radon decay constant, n = circulations of air/hour, E = Exhalation rate of radon gas (Bq/m^{2}h), A = surface of exhalation within the room (m^2) and V = Volume of the room (m^3).

In the current study a ventilation rate of 0.5 circ./h of air in the room was adopted. The exhalation rate, E, used for calculation of the radon gas level within a room was approximated using the last three readings from each measurement series.

2.2 Assessments

The concrete recipes contained identical constituents (aggregates, cement, water) where the only difference was a contribution of an additive (Hycrete) to one of two recipes. Identical concrete cubes (150 × 150 × 150 mm) were cast and stored in a conditioning room at 23°C and 50 % Relative Humidity (RH) between all measurements. The measurements were conducted during a 12 month period encompassing four separate measurements for each concrete recipe. Table 1 presents the recipes and proportions used.

Table 1 – Recipes and proportions (in kg/m^3 and weight %) of the different constituents in the assessed concrete specimens.

Constituents	Standard recipe		Standard recipe + additive	
	kg/m^3	**Weight (%)**	**kg/m^3**	**Weight (%)**
Cement, CEM II	350	15.3	350	15.3
Crushed aggregate, 0/8 (75 wt %)	1285	55.8	1279	55.9
Crushed aggregate 8/16 (25 wt %)	428	18.6	426	18.6
Water	227.5	9.9	221.1	9.7
Air	0.01 (~1.5%)	0.01	0.01 (~1.5%)	0.01
Superplasticizer (sikament 56)	~1	-	~1	-
Additive, Hycrete X1002	-		6.4	-
Total	**2291**		**2287**	
w/c ratio	0.65		0.65	

3 RESULTS

Figure 1a presents the measured radon exhalation rate of the two concrete recipes investigated. A distinct difference in exhalation rate is evident. A gross reduction of 30-35 % using an additive could be estimated. Figure 1b presents the approximate difference in radon gas within a room (3 × 4 × 2.5 m).

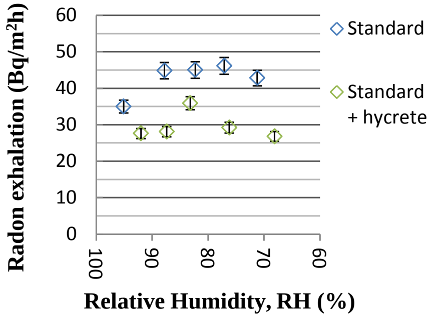

Figure 1a. The radon exhalation rate as a function of Relative Humidity (RH) of the concrete samples investigated. The error bars are ± 5 % [10].

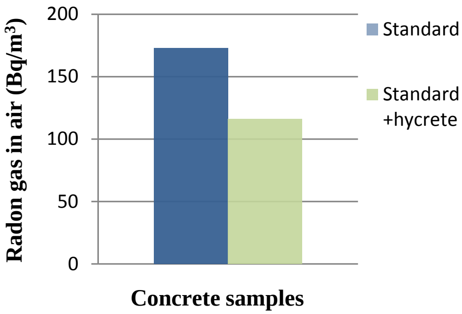

Figure 1b. The difference in radon gas within a standard room using a ventilation rate of 0,5 circ./h.

4 CONCLUDING REMARKS

The effect of using an additive may have a strong impact on the radon gas exhalation rate of building materials such as concrete and consequently the radon gas level within a room. Chauhan & Kumar [7] demonstrated a similar effect using an alternative binder such as rice husk ash in different concrete recipes. In the current study a reduction of 30-35 % of the radon gas exhalation rate from the concrete using an additive estimates a reduced effective dose to the human organs from 5.5 mSv per year to roughly 3.5 mSv per year. This shows the significance of reducing radon levels in building materials as a way to effectively reduce the final total dose to humans. Further and more comprehensive studies are needed as to confirm and validate the initial assessments.

5 ACKNOWLEDGEMENTS

Special thanks to the Swedish Consortium for Financing Fundamental Research in the Concrete Area who has partly funded the research in the concrete area. Special thanks also to my colleagues at CBI with guidance on the concrete castings produced.

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Requirements on concrete floor structures - a comparison of medical imaging facilities

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ABSTRACT
Requirements management in construction projects have a tendency towards production-driven processes and definition of technical solutions. The stakeholders are involved by being asked to comment on defined products which can have consequences on the performance of the end-product. This comparison describes three projects within the Stockholm County where the scope to build new medical imaging facilities with the same requirements on the concrete floor structure. The result shows that the same requirements have resulted in different solutions which could have an impact on the performance of the buildings. Further research regarding tools for systematic requirements management is needed to ensure performance and sustainability of new buildings.

Key words: Execution, Modelling, Structural Design, Sustainability

1. INTRODUCTION

Stakeholder requirements are often non-specific and give room for subjective interpretation from the designers within construction projects. When technical solutions are presented to stakeholders, such as end-users, they have difficulties to understand if these fulfil their requirements in daily work. In the construction business there is a need for a more systematic management of requirements from stakeholders [1, 2]. In the projects there is a tendency towards production-driven processes and definition of technical solutions. The stakeholders are involved by being asked to comment on pre-defined products or technical solution instead of being an active part in the design process. To ensure good performance of the end-product in construction projects focus on requirements are more important than technical specifications the early design stages [2, 3]. Analyses of full life-cost and environmental impact are also necessary to achieve sustainability over time. During the life-time of a building the requirements will likely change which can make the building prematurely obsolete [4]. Processes and tools for requirements management in the construction process of healthcare facilities often vary from project to project. Therefore there is a need for flexibility in the structural systems to reach sustainable solutions, often dominated by prefabricated concrete elements. The need for a systematic tool that connects requirements and technical specifications is here discussed and demonstrated through comparing concrete floor structures in three medical imaging facilities.

2. REQUIREMENTS ON CONCRETE FLOOR STRUCTURES

The requirements on concrete floor structures for new medical imaging facilities with the same functions should be similar but technical solutions often differ. By structured requirements management the technical solutions could be made more similar and lead to sustainability of performance. Stockholm County Council have set requirements to accomplish standardization, flexibility for future functions and facilitate rebuilding based on four types of specifications [5]. The first two are minimum measurements and functionality criterions, as exemplified in Fig. 1. The third and fourth types are specified minimum load carrying capacities and serviceability requirements on e.g. deformations and dynamic behavior. For concrete floor structures the general requirements prescribes an overall load carrying capacity of 6 kN/m^2, a vibration damping capability to ensure performance of medical imaging equipment, practical possibilities to mount heavy equipment in the ceiling and a flexibility regarding placement of future openings due to rebuilding. To meet requirements the general technical solution recommended by the Stockholm County Council is in situ cast concrete floor slabs supported by beams. Anchor rails are recessed or mounted on the underside of the concrete floor structure to enable mounting of equipment in the ceiling.

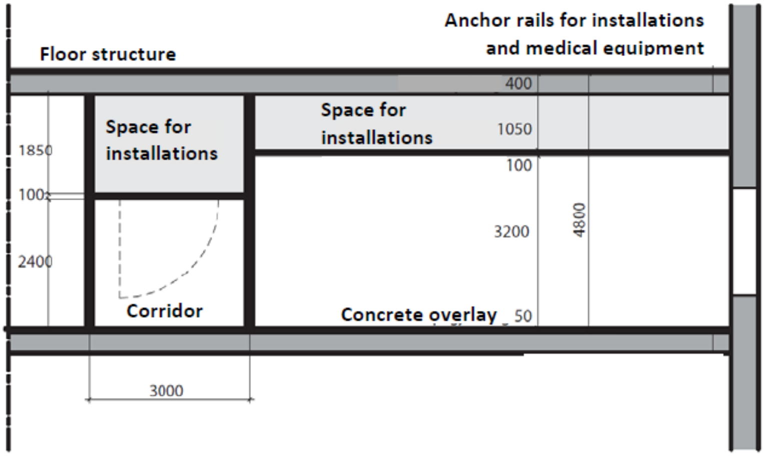

Figure 1 – Measurement requirements on structural systems for new healthcare facilities within the Stockholm County [5].

3. COMPARISON OF FLOOR STRUCTURE

Three medical imaging facilities in new healthcare buildings within the Stockholm County are compared. The three projects and the chosen concrete floor structures are briefly described in the following:

Project 1

A seven storey building with a gross area of approximately 28 500 m^2. One of the storeys is intended for a medical imagine facility. Prefabricated hollow core (HD/F) and massive concrete elements (D/DF) are used in the floor structure with a 100 mm overlay.

Project 2

An eight storey building with a gross area of approximately 19 000 m^2. One of the storeys is intended for a medical imagine facility. Prefabricated hollow core (380 HD/F) elements with 65 mm overlay and in situ cast concrete flooring are used in culverts.

Project 3

A four storey building with a gross area of approximately 28 000 m^2. One of the storeys is intended for a medical imagine facility. Prefabricated hollow core (380 HD/F) elements with 10-130 mm overlay depending on performance requirements. Massive concrete elements (220 RDF) with 210 mm overlay are used in some floor structures.

Medical imaging equipment installed in healthcare facilities is heavy and acquires structural damping of vibrations in order to achieve accuracy of measurements and image sharpness. These requirements are especially strict for magnetic resonance imaging (MRI) equipment. The data in this comparison is taken from technical specifications in system and/or construction plans of each project. In Tab. 1 there is a comparison of capacities for distributed loads, concentrated loads and multiplying factors for vibrations.

Table 1 –Comparison between technical specifications for concrete floor structures regarding load carrying capacities and multiplying factors for vibrations for three medical imaging facilities.

Project	Distributed load (kN/m^2)	Concentrated load – equipment (kN)	Concentrated load - transportation (kN)	Multiplying factors [a)]
1	6,0	150	145	0,125 [d)] 0,200 [c)]
2	6,0 [b)] 10,0 [c)]	150	-	0,200
3	7,5	130	90	< 0,250

a) according to British standard [6] b) within concentrated load area c) generally d) for MRI

Eurocode standards EN 1990, EN 1991 and EN 1992 have been used for design of the structural frames and specification of distributed and concentrated loads. Multiplying factor for continuous vibrations is calculated according to British Standard [6], defined as:

$$\text{Multiplying factor} = \frac{\text{root mean square acceleration} \times \text{frequency weighting of Wg}}{\text{base value}}$$

Frequency weighting should be Wg and the base value is 0.005 m/s^2 [6].

4. CONCLUSIONS FROM THE COMPARISON

The three projects described in the comparison show that the performance of the concrete floor constructions differs. The capacity for distributed and concentrated loads varies which may limit future choice of new equipment. The ability to reduce vibrations is also different. Since the development of medical imaging equipment aims for more accurate measurements and image sharpness, its sensitivity to vibrations increases. If the requirements with respect to future medical imaging equipment must be changed and the flexibility of the floor structure is not enough, it could mean that some of the buildings in the comparison could become obsolete. Even though the requirements for most medical imagine facilities are similar, it was here shown that there are different technical solutions for the concrete floor performance. Reasons could here be non-specific requirements from the Stockholm County Council, open for interpretations. Stakeholders, such as end-users, often lack the technical knowledge to set requirements for e.g. the concrete floor structures.

5. FUTURE RESEARCH

The comparison shows that same requirements regarding concrete floor structure can lead to different technical solutions and performance of the building. Research regarding requirements management in construction projects often focus on converting demands from stakeholders to technical specifications. The software and product development business has done the opposite, requirement tools are based on technical information needed by developers. The tools often consist of questions that are easily understood by stakeholders and give correct information for formulation of accurate technical specifications [7]. Future research is needed to explore if a similar tool for the construction projects can be used to minimize the functionality discrepancy between requirements and the end-product. A tool for requirements management should be empirically validated to avoid subjectivity and bias.

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A study on recycling of concrete in Sweden

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ABSTRACT
Sweden's recycled concrete waste is used up in low-utility construction such as backfilling and seldom as a substitute for natural aggregates in new concrete. To foster such high-utility recycling, a literature study was conducted on the regulatory instruments, building standards, production and properties of recycled concrete aggregates and the resulting new concrete for Sweden and other forerunner countries.
Results urge statistics to quantify recycled concrete; regulations such as source sorting of waste and selective demolition could potentially optimize recycled aggregate production. Also, the adhered mortar was found to govern the properties of the recycled concrete aggregate and new concrete.

Key words: Aggregate, Reuse and Recycling, Sustainability

1. BACKGROUND ON CONCRETE RECYCLING

The Swedish Environmental Protection Agency estimated that Sweden utilized 670,000 tons of concrete waste arising from the construction and demolition operations for constructing landfill covers and road backfilling in 2014. Additionally about 510,000 tons of this waste was estimated landfilled.

Contrary to this, Germany in 2003 used Recycled Concrete Aggregates (RCA) to produce Recycled Aggregate Concrete (RAC) which contributed to about 4% of the total precast and ready mixed concrete produced in Germany that year [1]. This comparison emphasizes the scarcity of high-utility concrete recycling in Sweden and its exclusion from national statistical estimations.

2. CONCRETE WASTE TO RCA

The regulation on landfill ban has played an important role in diverting concrete waste to produce RCA in Germany and Netherlands [2]. However the landfill taxation in Sweden amounting to 435 SEK/ton is not seemingly effective for the continuous production of RCA for use in RAC.

Regulations like the source-sorting of concrete waste can create cost savings at the separation stages in the RCA production process as has happened in Portugal; where source sorting is obligatory [3]. Other regulations include selective demolition such as in Denmark that render the concrete waste free of environmentally hazardous contaminants and impurities like harmful expansion causing gypsum in RAC. Source-sorting as a sole concrete fraction in Sweden is non-existent and selective demolition is not largely implemented. Fig.1 presents the RCA production process that could be optimized by aforementioned regulations.

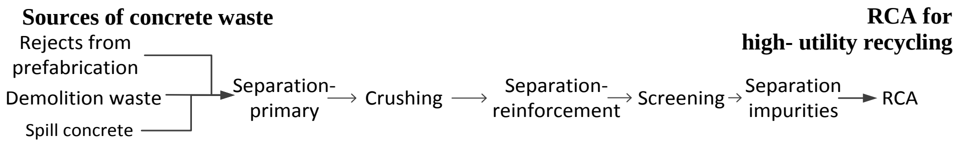

Figure 1 –A Generic representation of RCA production and the reach of Swedish statistics

To enable a quality control over the RCA, Building standards and technical guidelines such as RILEM and GEAR (from Spain) have classified RCA based on its constituents such as mortar, stone, cement, masonry or bricks. A unified European classification has been adopted by the Swedish standard SS-EN 12620+A1:2008 where coarse recycled aggregates are classified into Type A and B.

The amounts of type A and B recycled aggregates that could replace natural aggregates in concrete are regulated by the Swedish standard SS-137003:2015 based on RAC exposure classes. The standard prescribes the highest replacement percentage of 50% for recycled aggregates for un-reinforced concrete members indoors.

3. PHYSICAL PROPERTIES OF RCA - WATER ABSORPTION AND DENSITY

RCA comprises partly of crushed aggregate, original aggregate and adhered cement mortar. The adhered mortar causes the water absorption of RCA to be higher and the density of the RCA to be lower than that of the natural aggregate [4]. As a consequence, the mechanical and durability properties of RAC are influenced as well [5]. Thus the building standards such as aggregates for concrete SS-EN 12620+A1:2008 limits the RCA density to 2100 kg/m^3 and 1700 kg/m^3 for Type A and B respectively.

Further investigations on the adhered mortar revealed a relation between the compressive strength of the parent concrete and the porosity of the RCA, see Table 1. Concluding that the parent concrete of higher strength yield more porous RCA than parent concrete of normal strength as the bond between the mortar and the aggregate is stronger and harder to separate [6].

The results for particle density for the Swedish study [7] were difficult to relate to the parent concrete because of the inconsistent results for particle density at different RCA sizes. The particle density of the RCA sourced from hollow-core exceeded the RCA sourced from railway sleepers for 8-16 mm but for 4-8 mm RCA fraction it was vice-versa.

Table 1 – Water absorption and density values for RCA from different parent concrete [6]

Compressive strength (MPa)	37	50	58
Water absorption [%weight for 24 h]	3.65	4.1	4.86
Particle density [kg/m^3]	2520	2510	2480

3 MECHANICAL PROPERTIES OF RAC - COMPRESSIVE STRENGTH, ELASTIC MODULUS, DRYING SHRINKAGE

Besides the water/cement ratio of the parent concrete and the RAC, the RCA replacement percentage additionally affects the **compressive strength**.

Fig.2 shows the results for Li's investigation of RAC's compressive strength with varied water/cement ratios and coarse RCA replacement percentages [1]. RAC like conventional concrete shows decreased compressive strength with increasing water/cement ratio but this claim is not supported by the 50% replacement. Further investigations are required to adequately determine this relationship.

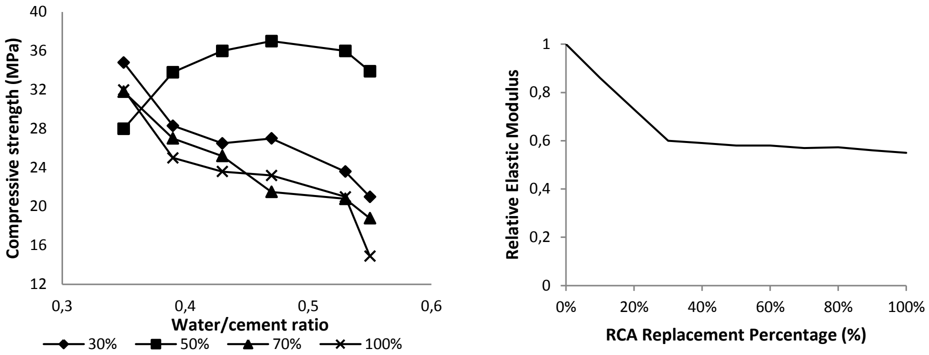

Figure 2 – Left: Compressive strength of RAC. Right: Relative Elastic modulus of RAC.

The **elastic modulus of RAC** is always lower than that of conventional concrete because of the adhered mortar of low elastic modulus attached around the RCA [8]. This would imply that as the RCA content increases the RAC elastic modulus must decrease. This was investigated by Xiao [1] and represented as relative elastic modulus; defined by the ratio of the elastic modulus of RAC to the elastic modulus of the reference concrete. As seen in Fig. 2, the elastic modulus decreases to 60% of that of reference concrete when RCA replaces 30% of the aggregate mass; it is then nearly constant for higher replacement percentages.

Researchers investigated that the **drying shrinkage** increased between 6-7% at increased coarse RCA replacements (30%, 50%, and 100%) in high-strength concrete samples compared to their control concrete. They attributed this increased shrinkage to the increased cementitious content in the RAC owing to the richness of the mix and the old cement adhered to RCA [5].

4 DURABILITY OF RAC- RESISTANCE TO FREEZE AND THAW

RAC has been speculated for being non-resistant to freeze and thaw due to its high porosity, where an increase in RCA content could increase the porosity resulting in non-durable RAC. However, with varying replacement percentages of coarse RCA and 5.5% entrained air, researchers found that RAC showed highest durability factors at 100% RCA replacement [5].

Similarly, the Swedish researchers designed RAC with varied RCA replacement percentages: with a water/cement ratio of 0.4 and 4.5% air-entrainment [7]. The scaling test results were well within the threshold value of 0.1 kg/m^2; proving that the designed RAC was freeze-thaw resistant. They concluded that RCA sourced from pre-cast and pre-fabricated concrete could produce freeze-thaw resistant RAC when designed at low water/cement ratios.

5 CONCLUSIONS

- Swedish statistics include estimations on quantities of mineral waste fraction including concrete and subsequently the percentage recycled. However, a more conclusive reporting could develop the market potential by ensuring the steady supply of RCA to RAC manufacturers.
- Introducing regulatory instruments such as selective demolition and source-sorting provides concrete waste with consistent quality suitable for RAC production. Cost optimizations are additionally achieved in the RCA production process.
- Literature points out that the adhered mortar content makes the RCA porous which influences eventually the mechanical properties and durability of the RAC. More research could be conducted towards finding the relationship between adhered mortar and RAC properties to enable a forecasting of the latter.

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Thermal conductivity based mix design of cementitious materials

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ABSTRACT
While cement-based materials are the most consumed materials in the construction industry, low or high thermal conductivity may be desirable for these materials depending on the application purposes such as embedded floor heating systems, building envelope or using as structural elements. The procedure of presenting prediction models for thermal conductivity of cementitious composites by considering different variables such as constituent materials, porosity and moisture content is described in this paper. The prediction model can be used for thermal conductivity based mix design of cementitious materials. Based on the desired accuracy, two methods for predicting the thermal conductivity are presented.

Key words: Mix Design, Modelling, Thermal Conductivity, Cement-based Materials.

1. INTRODUCTION

Thermal conductivity is an important material property in the energy design process of buildings. While cement-based materials are the most consumed materials in the construction industry, a wide range of thermal conductivity may be desirable for these materials depending on the application purposes. Indoor surfaces such as embedded floor heating systems or cementitious materials mixed with phase change materials, may demand high thermal conductivity. On the other hand, materials with low thermal conductivity may be desirable for using as a part of heat insulation or for thermal bridge calculations as well as structural elements.

Moisture content, porosity and constituent materials are the main parameters affecting thermal conductivity of cement-based materials. Thermal conductivity of water is more than 20 times bigger than thermal conductivity of the stagnant air and replacement of air by water can make a significant change in the thermal conductivity of porous materials. While changes in constituent materials and porosity may be neglected after concrete curing for thermal conductivity determination, the moisture content is expected to have considerable changes during lifetime of most cementitious materials. This means that considering one certain value for thermal conductivity of such types of materials may give low accuracy when considering the material performance during the service life of the material. Calculating thermal conductivity as a function of main effective variables such as moisture content, porosity and constituent materials based on semi-empirical models can be a practical solution to this challenge. The thermal conductivity of dry material can be adjusted in the mix design based on the concrete technology knowledge on porosity and constituent materials. Variations in this material property due to moisture content can be estimated based on the saturation degree. Moreover, the water sorption can be controlled by modifying the pore structure as well as internal or surface hydrophobation [1,2]. The proposed model can for example be introduced to building physic tools, where the thermal conductivity can be updated based on the existing climate conditions.

2. PARTICLE-MATRIX MODEL FOR THERMAL CONDUCTIVITY DETERMINATION

2.1 Matrix

The main factors affecting thermal conductivity of the matrix can be considered as variables in a multiphase composite model. Baghban et al. [3] presented a three-phase model for predincting thermal conductivity of hardened cement pastes (hcps):

$$\lambda^n = m\lambda_w^n + (\varepsilon_{tot} - m)\lambda_a^n + (1 - \varepsilon_{tot})\lambda_s^n \tag{1}$$

Where λ, λ_w, λ_a and λ_s are the thermal conductivity of the hcp, water, air, and solid structure of the hcp, respectively. ε_{tot} is the total porosity, m is the volumetric moisture content and n is a constant value. While λ_w and λ_a are known, a proper estimation needs to be done for λ_s and n based on experimental investigation.

The graph on the left in Fig. 1 illustrates the thermal conductivity of plain hcps at different total porosities and volume fraction of water. Unknown parameters are determined based on experimental investigations and introduced to Eq. 1, which is in agreement with the results obtained from the laboratory (See Fig. 1- right). Since thermal conductivity of solid structure of the matrix, λ_s, may vary due to changes in constituent materials such as presence of pozzolanic materials, fibers or changes in the cement chemistry, λ_s can be determined as a function of these variables by laboratory research. Furthermore, changes in the thermal conductivity of the fluid phase due to variations in the pore structure or different fluid chemistries can also be investigated by the same procedure.

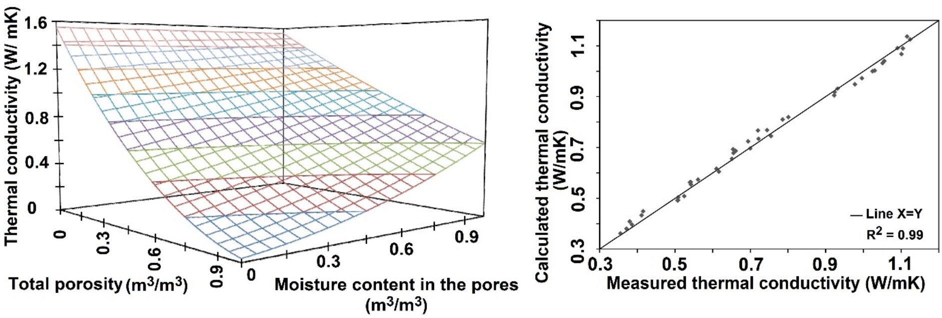

Figure 1 – Thermal conductivity of plain hcps at different total porosities and volume fraction of water calculated from Eq. 1(left), comparison of the measured and calculated thermal conductivities (right)[3]

2.2 Particle

Stone aggregates are the most commonly used particle types in cementitious composites. These aggregates have usually a low porosity and the effect of moisture sorption may be neglected for many practical applications. On the other hand, multiphase prediction models can also be presented for the particles in case of using aggregates with considerable porosity, such as using light weight aggregates.

Fine particles in the size range of the matrix particles can be considered as a part of the matrix. Moreover, the coupling effects such as effect of interfacial transition zone can also be defined as a function of the surface area of the particles in the mix.

2.3 Prediction models for cementitious composites

Semi-empirical model

Individual determination of the thermal conductivity of the particle and the matrix phases makes it possible to determine the thermal conductivity of the cementitious composites using a two-phase model. Furthermore, the accuracy of the model can be adjusted based on the considered accuracy in predicting the thermal conductivity of individual phases.

$$\lambda^n_{composite} = \upsilon_1 \lambda^n_{matrix} + \upsilon_2 \lambda^n_{particle} \tag{2}$$

υ_1 and υ_2 are the volume fractions and n is a constant value determined by experimental investigation.

Simplified estimation using Hashin-Shtrikman bounds

While the above mentioned semi-empirical model can be used for thermal conductivity based mix design as well as estimation of thermal conductivity of existing cement-based composites with a reasonable accuracy, a simplified method can be used for predicting the upper and lower limits of this material property. The Hashin-Shtrikman (H-S) lower (λ_l) and upper (λ_u) bounds for two material phases with $\lambda_1 \geq \lambda_2$, are given by [4]:

$$\lambda_l = \lambda_1 + \frac{v_2}{\frac{1}{\lambda_2 - \lambda_1} + \frac{v_1}{3\lambda_1}} \tag{3}$$

$$\lambda_u = \lambda_2 + \frac{v_1}{\frac{1}{\lambda_1 - \lambda_2} + \frac{v_2}{3\lambda_2}} \tag{4}$$

As the difference between thermal conductivity of matrix- and particle phases becomes lower, the two-phase H-S bounds become tighter and a reasonable estimation of thermal conductivity of cement-based composites is possible without conducting experimental investigation for predicting n value in Eq. 2. The same procedure can be used for predicting the thermal conductivity of matrix- or particle phases separately, for example in the case of submerged hcps where most of the air which has low thermal conductivity is replaced by water which has a thermal conductivity value closer to the thermal conductivity of solid structure of the hcp (Fig. 2).

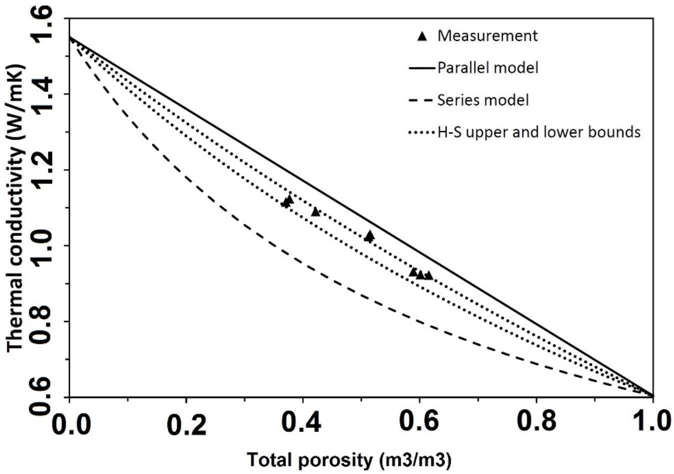

Figure 2– Measured thermal conductivity of plain hcps submerged in water and analytical bounds [3]

The expected porosity and moisture content of the cement-based materials can be estimated and adjusted by using the knowledge of concrete technology and building physics. Consequently, by introducing appropriate constituent materials, a particle-matrix model in the form of semi-empirical model or two-phase H-S bounds can be used for thermal conductivity based mix design of cementitious materials with desirable accuracy.

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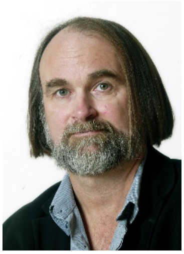
Characterization of cements for injection



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ABSTRACT
The three most common cements for rock injection in Norway were characterized in terms of grout flow properties, stability and initial set for w/c = 1.2, 1.0, 0.8 and 0.6 at 8 and 20°C. The fineness was characterized by Blaine, BET and particle size distribution (PSD). The test methods were bleeding, consistency (ring and Marsh Cone), setting by Vicat and temperature evolution in insulated cup. Additional rheology tests with parallel plate rheometer and isothermal calorimetry for hydration evolution were performed for mixes at 20°C only. Only one of the three cements tested could be utilized at all w/c levels.

Key words: Cement, Hydration, Injection, Rheology, Setting

1. INTRODUCTION

The objective of the study was to characterize different cement used for rock injection in the Nordic countries, and provide basic data for the behavior of fresh grout prepared from these cements [1]. A total of seven cements were selected for initial testing; the two most commonly used in Norway and 5 other cements. It was emphasized to make initial tests on relevant cements within a wide fineness range, and then select 3 cements for testing in cement paste. Limited data for the 3 selected cements on hydration and rheology are presented here, while additional data is published earlier by Skjølsvold and Justnes [2].

2. EXPERIMENTAL

2.1 Cement characterization

All cements were initially tested for 1) Density by Accupyc 1330 helium pycnometer, 2) Particle size distribution by Colter LS Particle Size Analyzer, 3) Blaine fineness (EN 196-6) and 4) BET fineness by nitrogen absorption . The test results are given in Table 1 and the particle size distributions are given in Figure 1.

Table 1 - Test results, characterization of cements

Cement	A	B	C
Density, g/cm^3	3.17	3.16	3.10
Blaine, cm^2/g	729	541	706
BET, m^2/g	1.88	1.58	1.93
D_{95}, µm	17	18	25

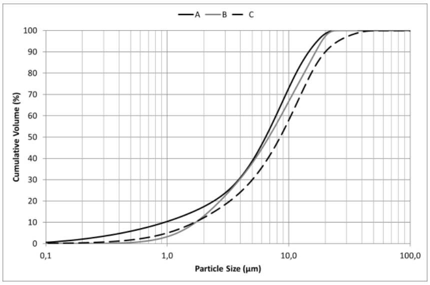

Figure 1 – The particle size distributions of the cements

2.2 Injection cement performance

The 3 cements were blended in a Waring blender with water to achieve 4 different w/c = 1.2, 1.0, 0.8 and 0.6. The resulting pastes were characterized in terms of rheology, setting and heat of hydration. The paste consistency was measured by a standard Marsh Cone with 4.5 mm outlet (27.6 sec for 1 liter water) and spread by the use of a cylinder (diameter 39 mm, height 60 mm) according to EN445. Bingham viscosity and dynamic yield point was also tested by a Physica MCR 300 parallel plate rheometer. Paste was placed between two parallel serrated plates in the rheometer where temperature is controlled to 20°C by a Peltier element. The speed was varied stepwise from 2 till 150 s^{-1} and back to 2 s^{-1} again. The Bingham viscosity (µ in mPa·s) and yield point (τ_{dyn} in Pa) was calculated from the ≈ linear part of the descending curve.

Setting time was determined at 20^0C by Vicat apparatus according to EN196-3 and by temperature evolution in insulated cups (i.e. semi-adiabatic) at 8 and 20°C taking a 2°C increase above base line as criterion for initial set. The hydration development was followed by a TAM Air isothermal calorimeter set at 20°C.

3. RESULTS

The initial consistency data from Marsh funnel and spread are given in Table 2, while Bingham viscosity and yield point are listed in Table 3. The setting times from Vicat needle tests and cups are reported in Table 4. The hydration heat development curves are depicted in Figures 2 and 3.

Table 2 - Initial consistency at 8 and 20 ºC

Cement	Cement A				Cement B				Cement C			
w/c	1.2	1.0	0.8	0.6	1.2	1.0	0.8	0.6	1.2	1.0	0.8	0.6
Room temperature	20^0C											
Paste temperature	20	19	19	19	19	20	20	20	19	19	19	19
Spread (EN445), mm	290	255	229	143	284	263	222	127	295	255	220	150
Marsh Cone, sec	30.3	32.6	36.4	52.1	30.4	32.6	39.2	88.4	31.0	33.9	39.6	-*
Room temperature	8^0C											
Paste temperature	7	7	6	6.5	6	6	6	7	7	7	7	6
Spread (EN445), mm	295	290	250	205	280	280	255	190	285	260	215	160
Marsh Cone, sec	32.0	33.5	36.1	50.4	30.3	31.5	36.0	51.1	31.5	34.3	39.6	75.6

* The paste was too stiff for Marsh Cone testing

Table 3 - Initial Bingham viscosity and yield stress 15 minutes after water addition at 20°C

w/c	1.2	1.0	0.8	0.6	1.2	1.0	0.8	0.6
	Viscosity, μ, mPa·s				Yield stress, τ_{dyn}, Pa			
Cement A	31.4	38.9	75.1	183	0.4	1.4	3.7	17.0
Cement B	19.7	31.3	56.9	139	0.0	1.6	4.9	23.2
Cement C	27.1	44.4	72.2	293	1.2	2.9	7.3	24.9

Table 4 - Initial set results by Vicat needle at 8 and 20°C

Cement	Cement A				Cement B				Cement C			
w/c	1.2	1.0	0.8	0.6	1.2	1.0	0.8	0.6	1.2	1.0	0.8	0.6
Room temperature	20^{0}C											
Vicat, initial set, hours-min	-*	0-40	0-35	0-30	11-40	11-00	8-35	6-10	9-15	8-10	6-35	5-45
Time to 2°C T-rise, hours-min	0-30	0-25	0-20	0-20	6-30	4-55	6-15	5-20	3-50	3-50	3-15	3-45
Room temperature	8^{0}C											
Time to 2°C T-rise, hours-min	0-45	0-45	0-45	0-45	-**	23-55	20-40	17-10	14-35	13-00	11-15	9-50

* Initial set occurred quickly, but was not recorded as the paste did not withstand the needle
** 2^{0}C temperature rise was not achieved

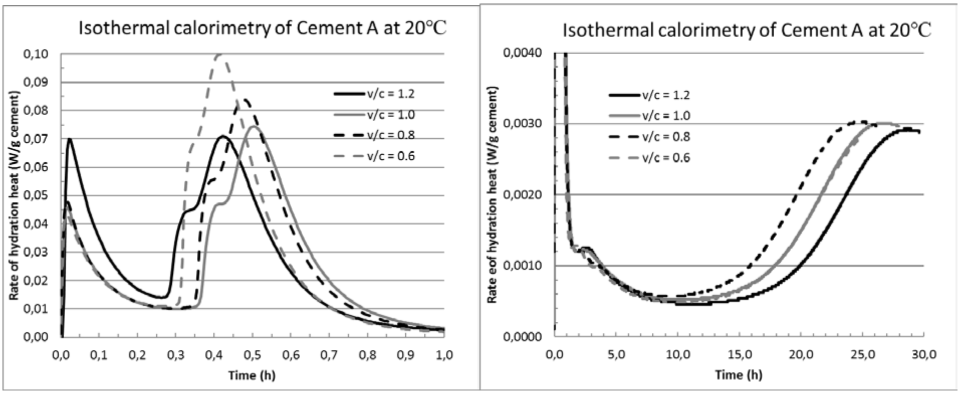

Figure 2 – Rate of hydration heat for cement A as a function of w/c (left; unusual early peak).

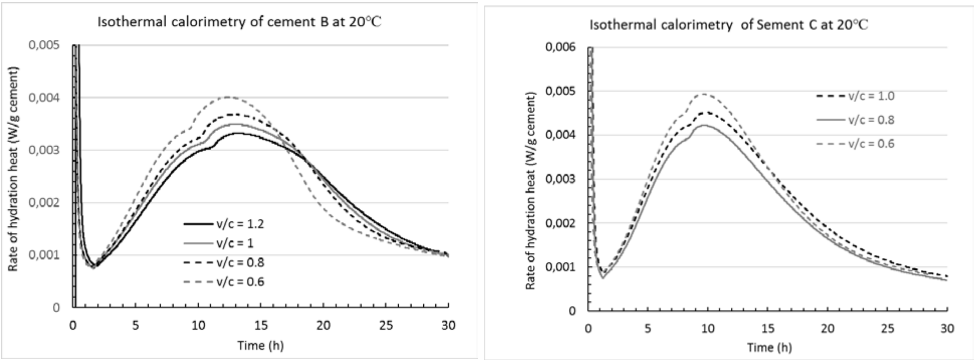

Figure 3 – Rate of hydration heat for cement B (left) and cement C (right)

4. DISCUSSION

All cements had similar initial spread and Marsh funnel flow, which correlated well with the viscosity measured by the rheometer as shown in Figure 4. All cements gave poor flow at the lowest w/c and unsatisfactory bleeding at the highest w/c (exception for cement A). The reason for reduced bleeding for Cement A can be understood from the early hydration development curve in Figure 2 showing an unusual early hydration heat between 0.25 and 0.75 h that would lead to a premature weak set before the second major hydration start at around 11 h at 20°C. This is also reflected in the very early setting by vicat and semi-adiabatic cup for this cement. The heat of hydration curves of cements B and C in Figure 3 looks like what one would expect from a fine portland cement and their difference reflects the difference observed for the setting times. Vicat is not well suited for testing at much higher w/c than 0.5 as the cement paste sets without development of sufficient strength to withstand the needle from penetration. Determination of temperature development at low temperature was also difficult at high w/c as the heat development is too slow compared to the temperature loss from the cup.

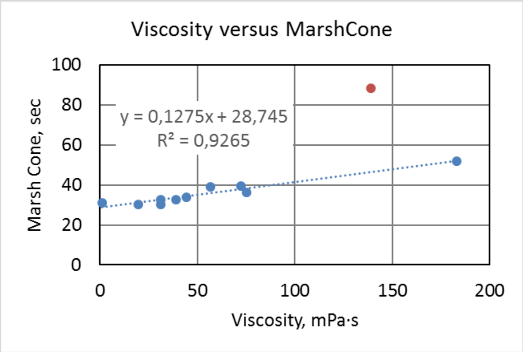

Figure 4 - Viscosity after 15 min versus Marsh Cone values. The open circle is w/c = 0.6 for cement B (w/c = 0.6 for cement C did not give any Marsh Cone result)

5. CONCLUSION

Only cement A among the three tested cements were suitable at the whole w/c range in terms of rheological properties, but had a very early setting and unusual hydration heat evolution.
The setting time roughly triples when going from 20 to 8°C (typical rock temperature), which is worth noting for tunnel injections. The tests performed do not indicate the suitability for grout penetration into small cracks and therefore additional tests on penetrability properties (e.g. fluid loss by filter pump) are performed by other project partners and will be published elsewhere.

ACKNOWLEDGEMENT

This paper is a part of the research project “True Improvement in Grouting High pressure Technology for tunnelling (TIGHT)" financed by the Research Council of Norway (project no. 236676/O30).

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Concrete pavements' resistance to studded tyres

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ABSTRACT
There are few Concrete Pavements in Sweden. Today there are only 68 km, built between 1991 and 2006. The earlier concrete pavements had a problem with much rutting due to the Swedish use of studded tyres during the winter months. The earliest road stretches in the 1990's were therefor constructed with a focus on wear resistance [1-2] with high quality concrete and aggregate. Measurements after the first few years showed minimal rutting [2]. The two newest pavements, built 1999 and 2006 had lower quality aggregates and less margins on the concrete quality, barely meeting the requirements. They also show severe rutting at 10-15 years of age. This work looks into the relationships between concrete quality, aggregate quality and rutting.

Key words: Pavements, Durability, Wear, Aggregate, Studded Tyres, Long Term Testing, Sustainability, Surfaces.

1. INTRODUCTION

Several concrete roads were built in Sweden in the middle of the 20th century, but the life span of these was shorter than projected and the roads were rather soon repaved with asphalt. An improved construction was introduced during the 1970's and the two stretches of road that were built in southern Sweden then both had a life span of over 30 years. In the 1990's there were further improvements to the design of a couple of concrete pavement stretches, and these together with one road built 2006 make up today's Swedish concrete road network of 68 km. One continuing issue with concrete pavements in Sweden is the use of studded tyres which causes rutting and drives the need for early and costly rehabilitation (see Figure 1).

Figure 1 A concrete pavement with rutting in Sweden. Photograph: Fredrik Hellman.

2. CASE STUDIES

The 68 km of concrete pavements on the Swedish highway network consists of five different stretches. These stretches were measured for rutting and drilled cores were taken and tested for concrete quality through compressive strength. Aggregate qualities were collected from construction reports and procurement requirements.

The Arlanda pavement was built 1991 as a Jointed Plain Concrete Pavement (JPCP) on a cement bound base course and with a concrete quality of K80 (corresponding to 80 MPa compressive strength) and a strong aggregate [2].

The northern part of the Falkenberg pavement was built 1993 as a JPCP with a concrete quality of K60 (corresponding to 60 MPa compressive strength) and a strong aggregate. About half of the pavement was on an asphalt bound base course and the other half on cement bound base course [1].

The southern part of the Falkenberg pavement was built 1996 as part of a larger research project. It is the same concrete quality as the northern Falkenberg pavement and mostly the same aggregate. It is mostly designed as JPCP with one test section being Continuously Reinforced Concrete Pavement (CRCP). Aggregate type and size are also varied on different sections [3].

The Eskilstuna pavement was built in 1999 as a JPCP with requirements of a K60 pavement and a strong aggregate, however, there is no data on what aggregate was actually used.

The Uppsala pavement was built in 2006 as a JPCP with a K60 quality and lower aggregate strength than the other pavements.

3. MEASUREMENTS

The measurements that were made were comparisons on the aggregate strength from construction reports and other documentation from the construction of the different pavements, compressive strength from three drilled cores from each pavement or test section and measured rutting with RST Laser.

3.1 Aggregates

No reasonable method of extracting and measuring the strength of the aggregates was found and therefore it was decided to only use the data given in construction reports and procurement requirements. Different methods were used in different reports, Kulkvarnsvärde (Kkv) is the Nordic test to determine the resistance to wear by abrasion from studded tyres (low values are better than high ones).

Arlanda had a flint called X100 with a strength corresponding to Kkv 6 (measured with an old method). Both Falkenberg pavements hade the Norwegian aggregate called Durasplit with a Kkv of 6,9. Eskilstuna had a requirement of Kkv 6 in the procurement, but no data was found on the actual aggregate used. Uppsala had a requirement of Kkv 8,5 and had a stronger and a weaker aggregate mixed to match the required value.

3.2 Compressive strength

Flexural strength would be the preferred test method since the design is based on flexural strength, but sawing beams out of each pavement was not financially feasible in the scope of this project. Instead, it was decided to take drilled cores and test the compressive strength. The results are shown in Figure 2. It is noticeable that the Arlanda pavement was designed for 80 MPa, and the rest for 60 MPa.

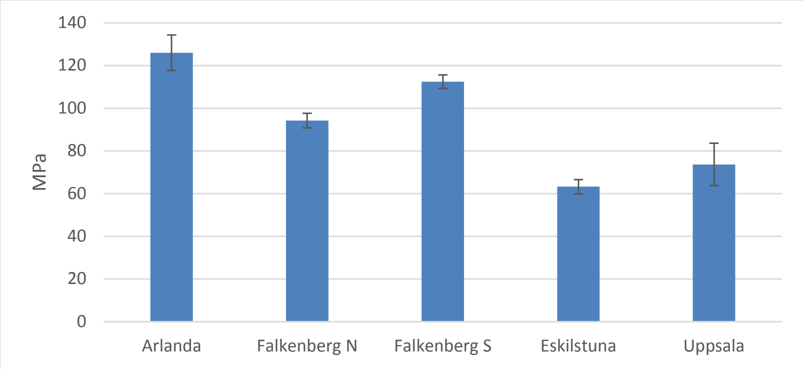

Figure 2 Compressive strength 2015.

3.3 Rutting

All pavements were measured with VTI's RST Laser car during summer season of 2015. The results are shown in Figure 3, but remember that the different pavements vary in age from 9 to 25 years.

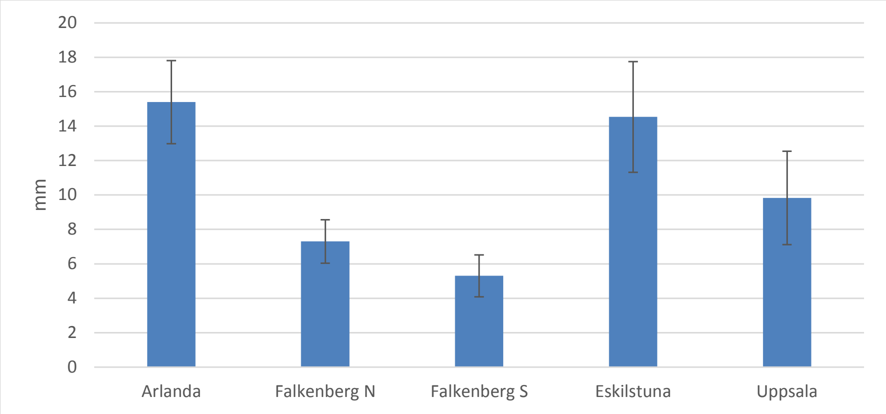

Figure 3 Rutting measured 2015.

To be able to compare the rutting between the different pavements, a wear coefficient was calculated for each pavement. The wear coefficient consists of the factors:

- Amount of traffic, that is AADT with the heavy traffic removed since they don't use studded tyres. Data from the national road database https://pmsv3.trafikverket.se/.
- Usage of studded tyres, presented by the National Road Administration [4]. Use of studded tyres have historically been higher than today, but the change is assumed to be roughly the uniformal nationwide. Therefore the latest data is used, not historical average.
- Length of winter season, four or five winter months depending on geographical location.
- Correction factor due to steel studs being used before the year 2000, but only light weight studs being allowed after that.

The measured rutting was then divided by the wear factor, results are shown in Figure 4.

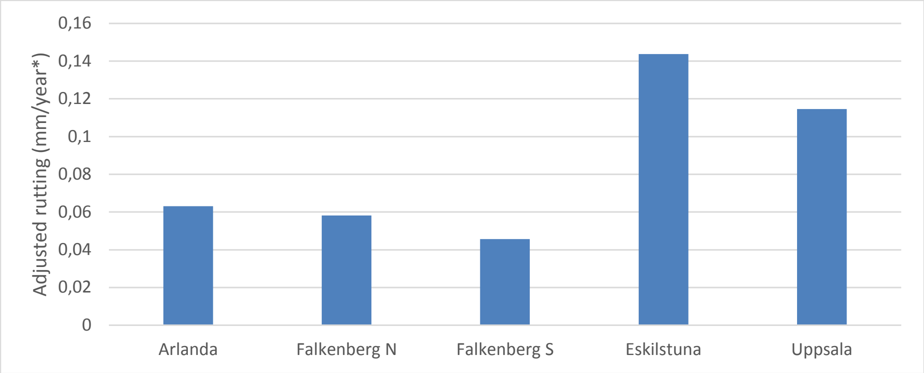

*Figure 4 Rutting per total average day wear. *Weighed for age, traffic, use of studded tyres and change of stud material over time.*

4. CONCLUDING REMARKS

The results of these measurements show that both the aggregate strength and the concrete quality play their part in the pavements' resistance to wear by studded tyres.

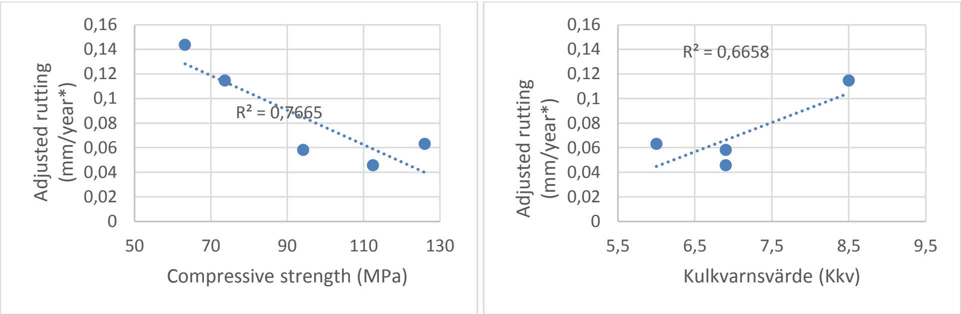

Figure 5 a) Rutting correlated to compressive strength and b) aggregate strength (without Eskilstuna).

To get a well performing concrete pavement it is important to set the right requirements for both concrete and aggregate quality and it is important to follow up that the requirements are met.

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Session B1:

STRUCTURAL DESIGN

Challenging the limits for beam bending designs

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ABSTRACT
The traditional design limits of beams in bending have been challenged by testing from very under-reinforced design to over-reinforced and strengthened over-reinforced designs in order to investigate if the current limits could be abolished. The ductility of normally reinforced beam depends significantly on the amount of reinforcement and an over-reinforced design can be modified to behave as a normally reinforced design, but with extreme ductile behaviours, but may requires stirrups beyond the codes requirements for columns. The ductility of under-reinforced beams may exceed that of some normally reinforced designs.

Key words: Modelling, Reinforcement, Structural Design, Testing.

1. INTRODUCTION

The traditional beam designs use normally reinforced cross-sections, in order to have a good warning before failure, a ductile and plastic behaviour near the peak load and at the same time achieve a good economy in the design, since such a design utilizes the reinforcement (the most expensive material) to its full capacity. The normally reinforced cross-section is defined as the tensile reinforcement yielding, but not rupturing before the beams failure (peak load):

$$\varepsilon_y \leq \varepsilon_s \leq \varepsilon_u$$

The reinforcement strains are here defined as

ε_s tensile strain,
ε_y yield strain,
ε_u ultimate tensile strain capacity.

The design of prefabricated elements requires, however, often very low height of build-in beams in wall elements (e.g. over window and door holes), but requires at the same time a high strength and a high stiffness. This is not always possible with a normally reinforced beam.

In other designs, there is plenty of height for the beams and the minimum reinforcement, required for achieving the normally reinforced state, leads to a capacity well beyond the required. It would be of high interest to explore the possibility to actually only use the reinforcement, required for obtaining the necessary loadcarrying capacity and the required stiffness.

It was therefore decided to look into the effects of the reinforcement degrees on the beam behaviours in two projects, with help from the prefab producer EXPAN/CRH, who would produce and deliver the beams and relevant samples to DTU for testing.

2. TEST PROGRAM

The first project [1] designed and tested a number of beams (see Table 1) starting with normally reinforced beams and increasing the tensile reinforcement until the over-reinforced cross-section was reached. The theory predicted that an increase of the compressive reinforcement in an over-reinforced cross-section should change the design back to a normally reinforced cross-section. The project has an initial test program (beam A to H) and later a second, additional program (beam K to M).

The second project [2] designed and tested beams N to U (see Table 1), starting from with the normally reinforced cross-section (where beam design N and A should be as identical as possible) and decreasing the reinforcement or increasing the cross-section until a seriously under-reinforced cross-section was obtained.

Table 1 – Test programs [1],[2]. No is number of beams, dimensions are in mm and strengths in MPa. Note NR denotes normally reinforced cross-section, OR over-reinforced, NR normally reinforced due to the effect of additional compressive reinforcement and UR under-reinforced.*

			Comp. reinf.		Tensile reinf.		Stirrups	Design			
Ref	Serie	No	Area	f_{ym}/f_{um}	Area	f_{ym}/f_{um}	Ø/s	Width	Height	f_{cm}	Note
[1]	A	2	2Ø8	564/690	2Ø12	600/694	Ø8/100	200	200	43	NR
[1]	B	3	2Ø8	564/690	2Ø16	584/670	Ø8/100	200	200	43	NR
[1]	C	2	2Ø8	564/690	2Ø20	585/666	Ø8/100	200	200	43	NR
[1]	D	2	2Ø8	564/690	3Ø20	585/666	Ø8/100	200	200	43	NR
[1]	E	2	2Ø8	564/690	4Ø20	585/666	Ø8/100	200	200	43	OR
[1]	F	3	2Ø12	600/694	4Ø20	585/666	Ø8/100	200	200	43	OR
[1]	G	2	2Ø16	600/694	4Ø20	585/666	Ø8/100	200	200	43	NR*
[1]	H	2	2Ø20	585/666	4Ø20	585/666	Ø8/100	200	200	43	NR*
[1]	K	3	2Ø8	564/690	3Ø25	543/668	Ø8/50	200	200	34	OR
[1]	L	3	2Ø25	543/668	3Ø25	543/668	Ø8/50	200	200	34	NR*
[1]	M	3	4Ø25	543/668	3Ø25	543/668	Ø8/50	200	200	34	NR*
[2]	N	3	2Ø8	552/700	2Ø12	586/679	Ø8/100	200	200	55	NR
[2]	O	2	2Ø8	552/700	2Ø10	630/714	Ø8/100	200	200	55	NR
[2]	P	2	2Ø8	552/700	2Ø8	552/700	Ø8/100	200	200	55	NR
[2]	Q	2	2Ø8	552/700	2Ø8	552/700	Ø8/100	200	300	55	NR
[2]	R	2	2Ø8	552/700	2Ø8	552/700	Ø8/100	200	400	55	UR
[2]	S	2	2Ø8	552/700	2Ø8	552/700	Ø8/100	250	400	55	UR
[2]	T	2	2Ø8	552/700	2Ø8	552/700	Ø8/100	300	400	55	UR
[2]	U	2	2Ø8	552/700	2Ø8	552/700	Ø8/100	350	450	55	UR

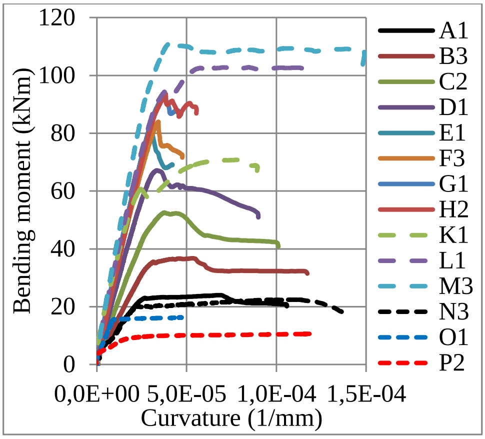
A total of 44 beams were produced by EXPAN/CRH in three steps and delivered to DTU Civil Engineering along with samples of the longitudinal reinforcement types and concrete cylinders.

3. RESULTS.

All beams were tested in four-point bending (see Figure 1), with registration of loads, deformations and crack distribution and also videorecording of all tests (available at Youtube [3]). All longitudinal reinforcement types and concrete cylinders from the three productions were tested as well and their average strengths listed in Table 1.

Figure 1 – Test setup with beam Q2 after failure [2].

It was observed [1] during testing of designs A to H, that the ductility of the cross-sections (see Figure 2) was less than what would have been expected from the theory and that it varied a lot within the range of the normally reinforced beams. Over-reinforced beams were, as expected, less ductile, but the attempt to increase the compressive reinforcement in beams G and H was not successful. This may have been due to the buckling of the compressive reinforcement, where the stirrups had been placed with 100 mm distance, sufficient for the design of a column.



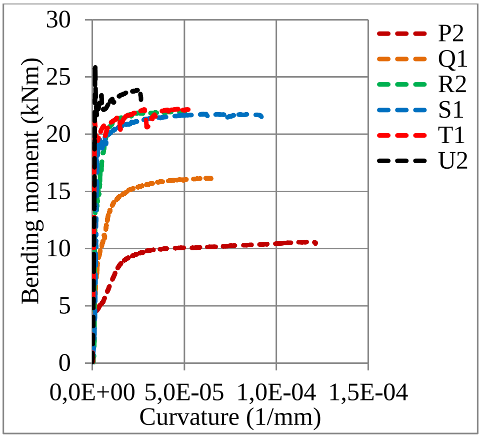


Figure 2– Representative bending moment-curvature curves for beams with constant cross-section 200 x 200 mm (left) and with varying cross-section (right).

The second part doubled therefore the number of stirrups and increased the compressive reinforcement in some designs, after which very good ductility and strength was, partly due to a confinement effect of the reinforcement. The ductility was at least equal to the ductility of a normally reinforced cross-section and the testing even had to be stopped before failure, due to the large deformations.

The second project [2] showed that the ductility of a number of the under-reinforced designs was as good or better than that of the normally reinforced designs near the upper reinforcement limit. The most under-reinforced designs had as predicted a bending moment capacity of the cracked cross-section below the uncracked cross-section, but the only slightly under-reinforced cross-sections actually had signs of a plastic plateau and a reserve after cracking (see Figure 2).

4. CONCLUSIONS

The ductility in a cross-section depends significantly on the reinforcement degree, even for normally reinforced designs.

Over-reinforced designs can be changed back to normally reinforced designs by adding compressive reinforcement, however, the design must then consider the possible buckling of the compressive reinforcement, which may require additional amount of stirrups beyond the codes requirements for stirrups for columns.

Over-reinforced designs can be changed back to normally-reinforced designs, by adding additional compressive reinforcement, however, this may require stirrups beyond the codes requirements. Such designs should at the moment be verified by testing.

Under-reinforced designs may have a ductile behaviour, however, the designer must for some of these designs distinguish between designs, where the beams is exposed to a forced deformation and those where it is exposed to an increased moment.

Under-reinforced designs should only be used after serious considerations, as their cracked capacity may be below the uncracked capacity (see Figure 2), but similar problems with the moment-curvature relation can be observed for normally reinforced cross-sections near the limit for over-reinforced designs.

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Strut and tie method – A powerful method to use in continuity and discontinuity regions in reinforced concrete structures in the design process

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ABSTRACT
The strut and tie method (STM) is an effective tool which illustratively describes how a given cracked reinforced concrete structure carries a specific load. Particularly, it has been used for design and structural analysis of discontinuity regions. It has a good potential to be used as a practical tool in the early state, optimizing a structure, in simple design and to check the reasonableness of the resulting load effects in complicated linear FE-analyses. This project aims to investigate the method´s suitability regarding resistance, ductility, serviceability and economic reinforcement arrangement. The paper presents the knowledge obtained in the project until today.

Key words: Strut and Tie Method, Structural Design, Nonlinear Finite Element Analyses, Knowledge and Teaching, Modelling.

1. BACKGROUND

Advanced computer aided analysis using the finite element method (FEM) is increasingly used for design of reinforced concrete structures. It is important that the designer makes his/her own verification of the reasonability assessment of the calculations and in the same way the reviewer has to make his/her rough estimates concerning the correctness of the calculations, see Ekström, et al. [1]. STM is a useful tool to assess the reasonableness of the results but when using the method it is important for the designer to have insight into what effect various parameters have on the structural response. This because the assumed stress field, out of many, must be possible to reach by means of stress redistribution without premature failure.

The idea of using strut and tie modeling was based on work done by German civil engineer Wilhelm Ritter [2] in the late 1890’s. Ritter introduced the idea that concrete beams could be designed by using truss analogy where tensile forces are carried by reinforcing steel wires or bars and compressive forces are carried by the concrete. In the early 1900’s Mörsch [3], [4], developed the theory. In the 1980’s Schlaich et al. started working with a way to design entire beams and structures using the truss analogy. They proposed a generalization of the truss analogy in order to consistent apply it in the form of Strut and Tie models to every part of any structure and their work is extensively presented in Schlaich, et al. [5] and in Schlaich and Schaefer [6]. A fundamental work to mention is also Muttoni, et al. [7] who emphasised stress field design. STM is based on theory of plasticity and simulates the stress field in cracked reinforced concrete in the ultimate limit state. STM can be considered as a truss idealization with struts carrying compression and ties carrying tension. The struts and the ties are connected in nodes. STM has in recent years been introduced in Eurocode 2, EN 1992-1-1 [8], as a method that may be used for design and detailing of both continuity and discontinuity regions. Other important references are CEB-FIP Model Code 1990 (MC 90) [9], FIP Recommendations 1999 “Practical Design of Structural Concrete” [10] and also Schaefer [11] and Schaefer [12] where design with STM are summarized in an extensive review for MC 90.

2. PURPOSE AND AIM

Reinforced concrete always has a nonlinear performance. Therefore it is always questionable regarding stress redistributions due to cracking and plasticizing. The purpose of this project is to create basis for assessing the consequences of a certain choice of a Strut and Tie model or a combination of Strut and Tie models in different stages. How can the freedom in choosing the model be used in optimizing the structure considering practical and economical reinforcement arrangement. The gained knowledge will also lead to applications for using Strut and Tie models in various stages of the design process, such as early stages, in simple designs and for verification of linear Finite Element Analysis (FEA).

The overall aim is to give recommendations concerning developing suitable strut and tie models for use in ultimate limit state (ULS) and for verifications in serviceability limit state (SLS) and to implement the STM in the design process. Answers to the following overall research question lead forward to the fulfilment of the objective:

- How will the stress redistribution take place in a reinforced concrete structure when it is subjected to loads above the:
    - Crack load and up to serviceability limit state,
    - Serviceability limit state and up to ultimate limit state,

  with respect to the variation of parameters as geometry, materials, reinforcement content and arrangement, minimum reinforcement, etc. and the fulfilments of requirements in ULS and SLS.
- What tools and procedures are necessary to implement STM in the design process?

3. SCIENTIFIC APPROACH

Initial literature studies has been made to acquire knowledge in the subject. Discontinuity regions in reinforced concrete structures, where a number of specified input parameters will be varied, will be analysed. The examples will be chosen based on simple and practical reinforcement arrangements. Failure load will be analysed for different Strut and Tie models for which the load effects will be calculated by statics. The models will advantageously be hold simple and easy to understand, regarding how the load is carried, with small number of struts and ties and also minimizing the ties. Then nonlinear FE-analyses will be performed under increasing load and response like cracking, stress redistribution, ductility and failure load will continuously be followed and registered. The consequences of the choice of Strut and Tie model for different part of the structure will be assessed based on the response of the FE-analyses.

4. PRELIMINERY FINDINGS

Until now the study has been focused on deep beams in reinforced concrete structures with low reinforcement ratio. Figure 1 shows three alternative Strut and Tie models for the same loaded structure. The difference lies in the choice of internal lever arm. Figure 2 shows the distance from the bottom of the structure to the compression resultant in the midspan versus total load on the structure in question for a nonlinear FEA. As can be seen from the figure it is obvious that the compression resultant redistributes under load increase from a linear position to a position near the top of the structure. For the studied cases it is clear that Strut and Tie models following the linear elastic solution or following a recommended value for the angle between the strut and the tie in the lower edge of the concrete around 60 degrees, compared to nonlinear FE-analyses, underestimates the maximum load bearing capacity for the structure and overestimates the stresses in the reinforcement. It is also clear that there is large stress redistributions due to cracking before the reinforcement yields. This is also a well-known phenomenon when the reinforcement content is low, se FIB Bulletin 61 [13] and Leonardt and Walther [14].

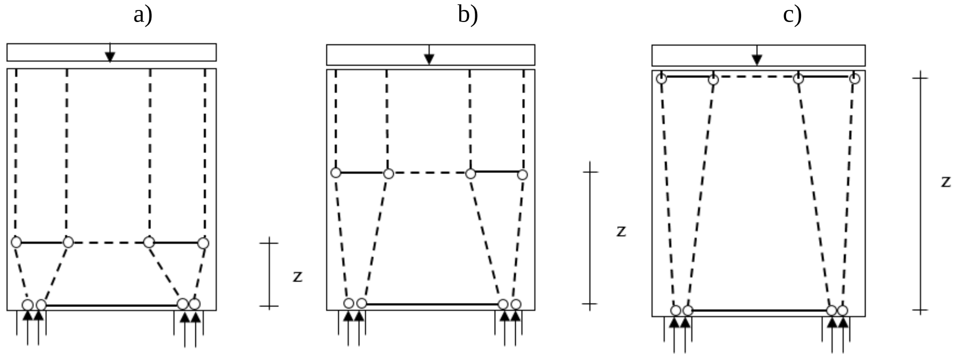

Figure 1 – (a) Strut and Tie model for 60 degrees, (b) Strut and Tie model for linear stress field, (c) Strut and Tie model for f_{cc} in horizontal strut.

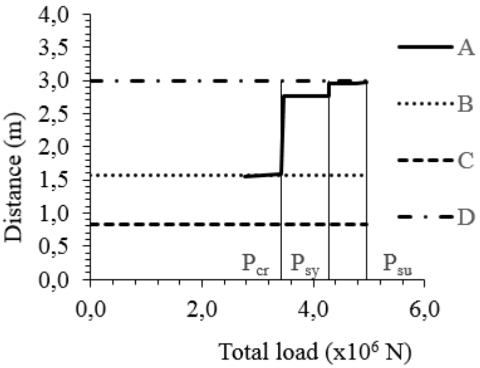

Figure 2 – Distance from the bottom of the structure to the compression resultant in the midsection versus total load on the structure. Curve A to D show the distance for: A=Nonlinear FEA, B=Linear FEA, C=STM for 60 degrees, D=STM for f_{cc} in horizontal strut. P_{cr}=Concrete cracking load, P_{sy}=Reinforcement yield load, P_{su}=Reinforcement ultimate load.

5. EXPEXTED OUTCOME

The project will result in knowledge about STM that:

- From a choice of same or different stress fields in the ULS and SLS will provide the basis for recommendations in terms of how much one can allow the stress redistribution to deviate from the linear elastic stress field and still meet requirements on the performance in serviceability state and ductility in ultimate state.

- Facilitates the application of the method in the design process.
- Creates incentive for knowledge and competence development regarding understanding the flow of the stresses and the stress redistribution in a reinforced concrete structure.

6. CONCLUDING REMARKS AND FURTHER WORKS

Looking ahead, STM and comparison with nonlinear FE-analyses will be performed for deep beams with a larger reinforcement amount to study what effect this will have on the development of the stress redistribution for increased loading up to failure. Analyses will also be carried out for other types of reinforced concrete structures where use of STM can be of interest.

7. ACKNOWLEDGEMENT

This industrial PhD research project is funded by ÅF-Infrastructure AB. The author also wishes to thanks Scanscot AB for their sponsoring.

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Seismic performance of reinforced concrete frame for the risk assessment process

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ABSTRACT
Risk assessment is a suitable method for general decision making in the field of seismic safety. The aim of seismic risk assessment is to calculate the probability of detrimental economic and social effects in a particular region due to earthquake. Performing risk assessment requires information about seismic performance and response of the structure. In this paper, the seismic performance of moment resisting reinforced concrete frame with shear wall is studied using incremental dynamic analysis. 20 far-field earthquake records are used and the fragility curve is presented. The results indicate considerable effect of structural stiffness on the probability of limit states violation.

Keywords: Structural Design, Reinforcement, Seismic Performance, Risk Assessment

1. INTRODUCTION

Urban development and the high density of buildings especially in large cities have made it necessary to consider effects of earthquake on structural design. Performing seismic reliability assessment requires a powerful tool for seismic analysis and one of the most modern tools at present is incremental dynamic analysis (IDA) [1]. IDA can also provide the possibility of performing probabilistic analysis. Using this method requires an accurate structural modelling by taking into account environmental and loading conditions [2].

2. PERFORMANCE BASED ASSESSMENT

Performance based method that is developed at the Pacific Earthquake Engineering Research Centre (PEER), includes four types of random variables and its stages are shown in Figure 1. These four mentioned random variables are considered as Intensity Measure (IM), Engineering Demand Parameter (EDP), Damage Measure (DM) and Decision Variable (DV) [6].

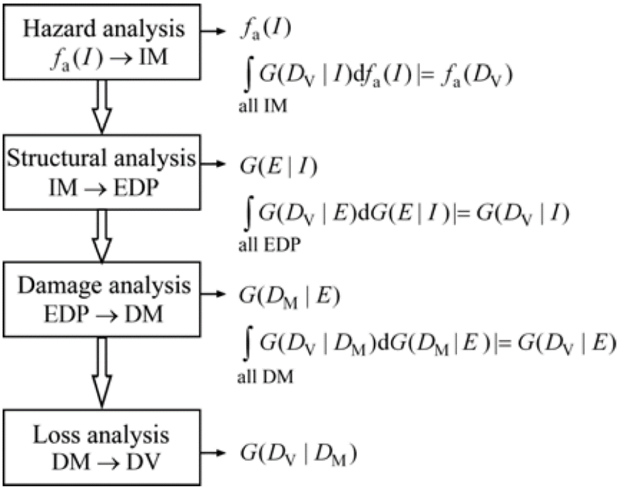

Figure 1 – Four stages of performance based assessment [6]

3. THE MODEL

The modelled structure is an eight-storey moment resisting reinforced concrete frame with shear wall, which is designed according to the ACI-318 code. The period of the first mode of this structure is T1=0.92 s. This frame is located in an area of high seismicity with gravitational acceleration of 0.3g.

In order to perform incremental dynamic analysis, a selection of ground motion records is required. 20 records that are used by Cornell and Vamvatsikos are considered in this study [5].

4. RESULTS

The IDA curves are illustrated in Fig 2 and 3. The horizontal axis corresponds to Maximum Interstory Drift Ratio (MIDR) and the vertical axis pertains to spectral acceleration values in the period of structure's first vibration mode considering a damping coefficient of 5% (S_a(T1,5%)).

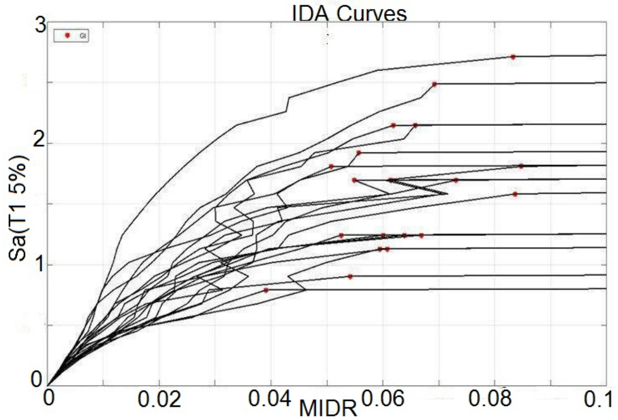

Figure 2 – IDA curves of earthquake records

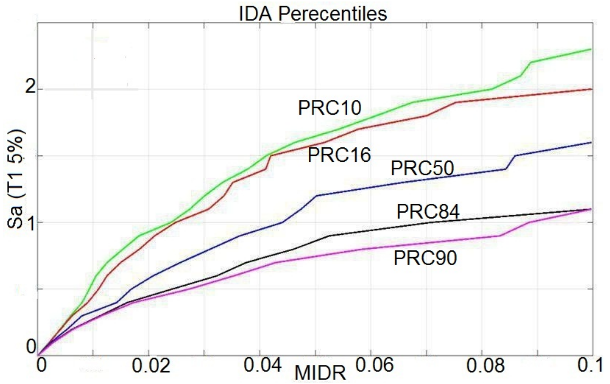

Figure 3 – IDA percentile curves (10%, 16%, 50%, 84% and 90%)

Fig. 4 presents MIDR fragility curve in initial levels of spectral acceleration (0.3g) which is obtained using IDA curves. In this figure, the horizontal axis is related to maximum interstory drift ratio values and the vertical axis corresponds to the probability cumulative distribution function.

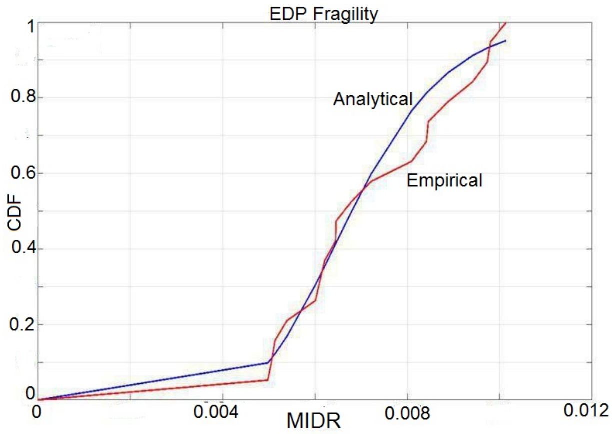

Figure 4 – MIDR fragility curve

The results show that, considering the structural stiffness, the probability of violating limit states in the dual structural system is considerably low. Furthermore, according to the IDA percentile curves, earthquake records damage the structure uniformly.

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Shear capacity of lightweight aggregate concrete beams without shear reinforcement

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ABSTRACT

The main disadvantages of lightweight aggregate concrete (LWAC) compared with normal weight concrete are its brittleness at the material level in compression and uncontrolled crack propagation. This experimental investigation consists of beams with lightweight concrete with Stalite as aggregate. Main goal were to investigate beams without shear reinforcement subjected to shear in four-point bending test and compare those results with previous experimental work. The main test parameter was the shear span length to effective height ratio (a/d). Tested beams did not show brittle behaviour. Beams were more ductile than expected, and cracking was similar as for normal weight concrete beams.

Keywords: Lightweight Aggregate Concrete, Testing, Shear Reinforcement, Shear capacity, Ductility.

1. INTRODUCTION

This investigation was part of the ongoing research programme, “Durable advanced concrete structures (DACS)”. One part of this programme is to investigate the structural behaviour of lightweight aggregate concretes (LWAC), i.e. concretes with an oven-dry density below 2000 kg/m^3. A general characteristics of LWAC is its very high degree of brittleness at the material level in compression, which result in sensitivity to stress concentrations and rapid crack/fracture development. This influences the behaviour of concrete where its tensile strength is important, as for instance with its shear and bond strength. To investigate the brittleness of LWAC, beams without shear reinforcement were subjected to shear in a four-point bending test. The main test parameter was the shear span length to effective height ratio (a/d). For all beams, the shear loads at diagonal cracking and at failure were plotted as a function of the a/d ratio and compared with previous results. For comparison, tested beams were the same size and with the same reinforcement as in earlier shear tests on other normal density (ND) and lightweight aggregate (LWA) concretes [1]. To produce the concrete, a lightweight aggregate Stalite was used to achieve an oven-dry density of about 1800 kg/m^3 and a compressive strength of about 60 MPa.

2. EXPERIMENTAL TEST PROGRAM AND RESULTS

The experimental program consist of four reinforced LWAC beams without shear reinforcement, which were subjected to a four point bending test, to produce a constant moment in the middle of the beam. The program and results are given in Table 1. The beams measured (width x height x length) 150 x 250 x 2900 mm. Longitudinal and cross section details of the beam specimens are shown in Figure 1.

Table 1-Test parameters and results for diagonal cracking, failure loads and calculated shear strengths in accordance with NS3473[2]

Beam	a [mm]	d [mm]	a/d	P_{cr} [kN]	P_u [kN]	P_{calc} [kN]	P_{cr} / P_{calc}	P_u / P_{calc}
1	876	219	2.3	45	92.3	44.6	1.01	2.07
2	876	219	2.3	44.5	127.2	44.6	1.00	2.85
3	504	219	4	36.8	44.4	44.6	0.82	0.99
4	504	219	4	33	62	44.6	0.78	1.39

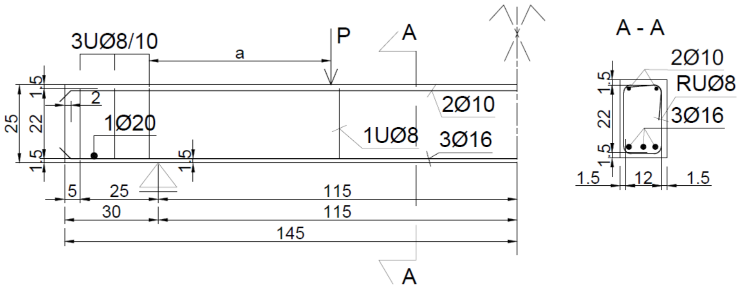

Figure 1 – Reinforcement and cross section details for the beams

The shear capacity of the beam has been calculated in accordance with NS 3473 [2] and Eurocode 2 [3], and these values are 42.8 kN and 44.6 kN, respectively.

3. DISCUSSION

The cracking and failure mechanism in beams without web shear reinforcement, which is usual in NWC, is that cracks will appear in the shear span with increasing load. The term diagonal cracking load is defined in this paper as the load when the specific shear crack is formed that goes on to lead to shear failure. The load at which the beam collapses is the ultimate load or the load-carrying capacity. Due to the brittle nature of LWAC, the diagonal cracking load is usually equal to the ultimate load. In this experiment, however, the beams could carry increasing load after the diagonal cracks formed. The cracks propagated almost horizontally along the tensile reinforcement and diagonally into the compression zone. In the final stage, the shear cracks opened wide as the diagonal cracks spread along the beam and this resulted in the crushing of the concrete close to the loading point.

Similar tests have been carried out previously for high-strength concrete classes of normal density concretes ND65 and ND95 and lightweight aggregate concretes LWA75, LWA40 and LWA_Leca_mix. Table 2 shows these tests compared with the present investigation on beams with a/d = 2.3 and a/d = 4.0, respectively. The test conditions were the same, including the rig, the cross section, and the amount of reinforcement [1]. For all of these concretes, capacity was calculated in accordance with NS3473 (P_{calc}).

Table 2 - Comparison of the shear strengths for beams with a/d = 2.3 and 4.0

Beam/Aggregate	a/d	Cylindar Strength [MPa]	P_{cr} [kN]	P_u [kN]	P_{calc} [kN]	P_{cr} / P_{calc}	P_u / P_{calc}
1/Stalite	2.3	67.5	45	92.3	44.6	1.01	2.07
2/Stalite	2.3	67.5	44.5	127.2	44.6	1.00	2.85
ND65/Årdal	2.3	54	62.2	71.6	55.1	1.13	1.30
ND95/Årdal	2.3	78	66.7	103.5	57.3	1.16	1.81
LWA75/Liapor 8	2.3	58	47.1	126.1	52.0	0.91	2.43
LWA40/Leca	2.3	37	46.6	77.9	39.3	1.19	1.98
LWA_Leca_mix	2.3	42.7	34.3	102.9	42.1	0.81	2.44
3/Stalite	4	67.5	36.8	44.4	44.6	0.82	0.99
4/Stalite	4	67.5	35	62	44.6	0.78	1.39
LWA40/Leca	4	37	38.2	38.2	39.30	0.97	0.97
LWA_Leca_mix	4	42.7	29.4	44.1	42.10	0.70	1.05

The results of main interest in this comparison are the ratio between the observed shear diagonal cracking load (P_{cr}) and the calculated capacity (P_{calc}), and the ratio between the observed failure load (P_u) and the calculated capacity (P_{calc}).

For beams with a/d = 2.3, the ratio between P_{cr}/P_{calc} was larger or equal to 1.0 for all concretes except LWA75 and LWA_Leca_mix. This indicates that they are on the safe side. For LWA75 and LWAC_Leca_mix, this ratio was less than 1.0, indicating a higher drop in shear strength than predicted by NS3473. The ratio between P_u/P_{calc} for all LWA concretes was almost 2 or higher, which shows that LWA concretes in general can stand more loading than predicted. For normal density concretes, this ratio was lower.

The ND65, ND95 and LWA75 concretes were not tested with ratio a/d = 4.0. For beams with a/d = 4.0, the ratio between P_{cr}/P_{calc} for both concretes tested was below 1.0 – indicating that beams with a/d = 4.0 have a certain drop in capacity [1,2].

For tested LWA concrete with Stalite as aggregate, the diagonal cracking load was close to other lightweight aggregate concretes, while the failure load was higher, especially in the case of a/d = 4.0.

In comparison with normal density concretes ND65 and ND95, the diagonal cracking load for normal concretes ware approximately 30% higher, while failure load for LWAC with Stalite was significantly higher. In general, here is actually from high importance the fact that after shear diagonal crack were formed beams can stand increase of load from 30 to 50 %.

4. CONCLUSIONS

For all concretes tested in this research, the shear stress at inclined cracking of the beams decreased with an increase in the shear span to effective height ratio (a/d). Cracking propagation in the tested beams showed they were more ductile than expected, which should promote increased investigation and structural use of this type of LWAC. According to this experimental investigation, the design strength for shear in beams and slabs without shear reinforcement should be based on inclined cracking loads. Comparison with similar tests on other types of lightweight concrete and normal density concretes showed that the ratio between the load observed at diagonal cracking and the predicted strengths was in the same range. However, the ratio between the load at failure observed and the strengths predicted was significantly higher for the lightweight concrete used in this investigation.

ACKNOWLEDGEMENT
This research forms part of the DACS project (Durable Advanced Concrete Solutions). The DACS partners are Kværner AS (project owner), Axion AS (Stalite), AF Gruppen Norge AS, Concrete Structures AS, Mapei AS, Multiconsult AS, NorBetong AS, Norcem AS, NPRA (Statens vegvesen), Norwegian University of Science and Technology (NTNU), SINTEF Byggforsk, Skanska Norge AS, Unicon AS and Veidekke Entreprenør AS. The project gratefully acknowledges the Norwegian Research Council for financial contribution.

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Comparison of punching shear design according to the new Finnish national annex for Eurocode 2 and the former national method

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ABSTRACT
In the beginning of 2017 the design method for punching shear in Finland was changed. The method presented in Eurocode 2 was adopted with some nationally determined parameters and rules. In autumn of 2016 a computational analysis was conducted in order to compare the previous national design method and the new one. Two comparison setups were created in which different parameters were varied. The results were presented as line charts. The focus was to find out how the predicted punching resistances differ between the two methods. It was discovered that the differences are significant and can be almost 60% in some cases.

Key words: Punching, Eurocode, Comparison, National annex, Structural Design

1. INTRODUCTION

In the former national annex (NA) of Finland for Eurocode 2 (EC2) [1] it was stated that the punching design method in its default form was forbidden due to observed discrepancies between predicted and experimental results. It was instructed that the method presented in The National Building Code of Finland (part B4) [2] was to be used instead. This changed in the beginning of 2017 when the new national annexes for Eurocodes were released and the old ones abolished. The new NA for EC2 instructed to use the punching design method of EC2 but with a couple of national rules.

A computational analysis was conducted to compare these two methods: the former national one and the new EC2-based one. Object of the study was to find out how much do they differ and to gain information on the effect of this change to practical concrete design.

2 COMPARISON SETUPS

Two setups were chosen to compare results. The following properties apply to both:

- Concrete class C30/37
- Reinforcement class B500B
- Square column 380mm * 380mm
- Distance from reinforcement centre of gravity to the edge of concrete slab was 40mm
- Safety factor for concrete $\gamma_c = 1.5$
- Reduction factor for compressive strength of concrete $\alpha_{cc} = 0.85$
- Reduction factor for tensile strength of concrete $\alpha_{ct} = 1.00$
- Safety factor for reinforcement $\gamma_s = 1.15$

In addition, in both setups the eccentricity factor β (relating to EC2) was included as two values: 1.0 and 1.15. This factor takes non-symmetrical shear force distribution into account. The bigger the eccentricity of the punching force the lower the punching resistance. Factor β increases as the eccentricity increases. Only unreinforced cases were examined. In other words, no shear reinforcement was used.

2.1 Varying reinforcement ratio (setup #1)

First setup was chosen to represent the effect of slab's reinforcement ratio to punching resistance. A constant slab thickness of 300mm was adopted and reinforcement ratio was varied between 1,51‰ and 25‰. The lower limit represents the minimum reinforcement requirement due to bending and the upper one is chosen to be a bit over the maximum reinforcement ratio of EC2 regarding punching resistance. Eurocode 2 gives a limit value of 20‰ after which an increase in reinforcement ratio is no longer effective to increase punching resistance.

2.2 Varying ratio of column side length and effective slab thickness (setup #2)

The most notable change in Finnish national annex in relation to default Eurocode 2 is the factor $C_{Rd,c}$ (coefficient derived from tests affecting shear resistance) which is a constant value in default EC2 but a variable (considering column side length and the effective slab thickness as factors) in the Finnish NA. This is the most profound difference between the two EC2 methods and also the primary reason why the default EC2 method was forbidden in Finland. Being so noticeable a factor it was only a natural choice to include this ratio as a variant. The reinforcement ratio was kept as a constant 7‰.

3 RESULTS

Comparison line charts were chosen as the main form of displaying results due to their clarity and versatility.

3.1 Varying reinforcement ratio (setup #1)

It can be observed that there are distinguishable differences between the two methods, see figure 1. Resistances according to EC2+NA are lower than according to the National Building Code (B4) only with very low reinforcement ratios. This is true on both eccentricity factors β.

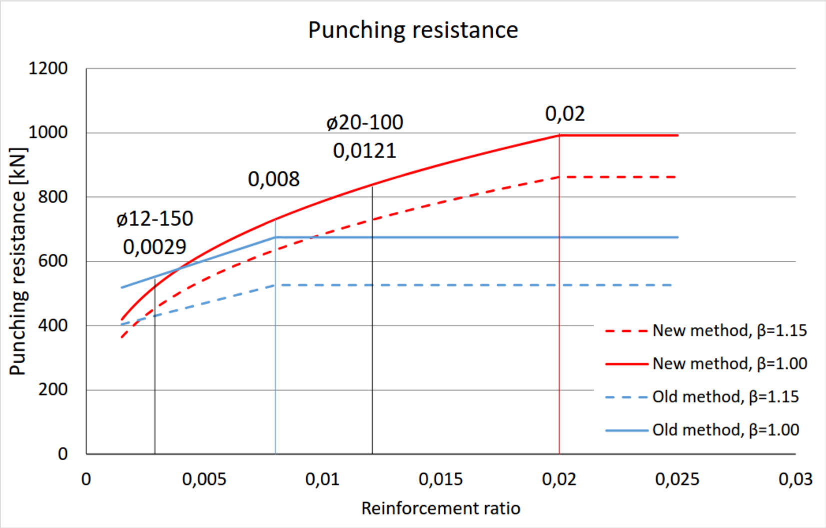

Figure 1 – Punching resistance in setup #1 [3].

3.2 Varying ratio of column side length and effective slab thickness (setup #2)

With setup #2 it is also clear that EC2+NA gives higher resistances than the national B4 method, see figure 2. The difference decreases as the ratio c/d (ratio of square column side length and the effective slab thickness) increases.

Figure 2 – Punching resistance in setup #2 [3].

To further clarify how big the relative differences are, also another set of line charts were generated for both setups. In example, see figure 3, which shows the relative ratio of EC2+NA and B4. It can be observed that the differences can be at least 50%. Regarding setup #1 values of over 60% were discovered.

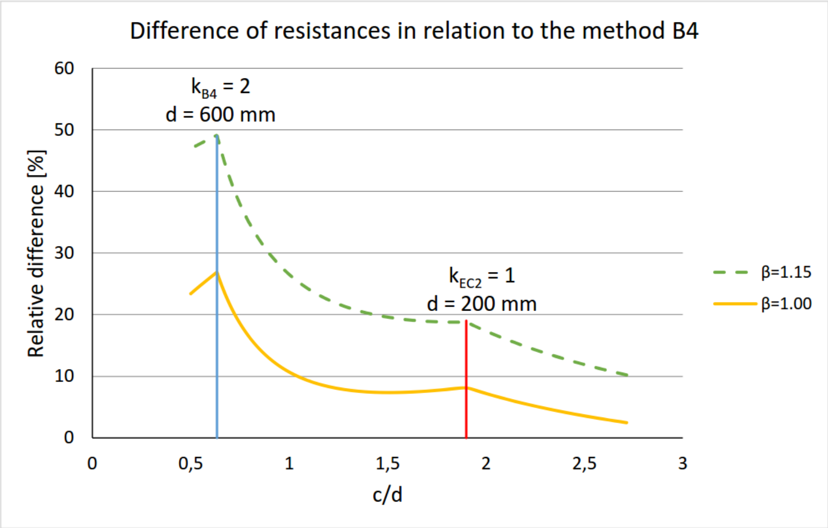

Figure 3 – Comparison chart (setup #2) representing the relative difference of resistances according to the two methods [3].

4 CONCLUSIONS

Based on these results it is apparent that the new method implemented in Finland as a national annex for Eurocode 2 gives very different results in some cases compared to the former B4 method. It is worth mentioning that the old method is no longer applicable to new building projects in Finland.

According to the new method, bigger reinforcement ratios can be utilized with an upper limit of 20‰ whereas with the old B4 method the limit was only 8‰. It is also noticeable that the eccentricity of the punching force affects the resistances considerably. It was shown that the difference increased as the eccentricity increased.

A general conclusion can be made that the new method gives higher punching resistances. Slab thicknesses compared to the past are likely to change due to this difference. Concrete compressive strength affects the results significantly: the higher the strength, the smaller the relative difference between the two methods.

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Evaluation and improvement of crack width calculation method for large concrete structures

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ABSTRACT
The current crack width calculation methods according to Eurocode 2 and *fib* Model Code 2010 have been evaluated. The physical nature of the equations is found to be inconsistent, which limits the generalization of the formulas. One of the inconsistencies is related to the cover term, which is currently being investigated in an experimental study of reinforced concrete prisms. An improvement of the formulas is suggested by looking to more physical realistic assumptions in a tension chord model and the cracked membrane model.

Key words: Crack widths, calculation, Eurocode 2, *fib* Model Code 2010, tension chord model, cracked membrane model, cover, concrete structures.

1. INTRODUCTION

The Ferry-free coastal route E39 is an ongoing research project under the auspices of the Norwegian Public Roads Administration (NPRA). It is an 1100 km long coastal high way along the west coast of Norway, where large bridge superstructures are intended to replace ferries crossing the fjords today. Among others, crack width calculation methods are of primary importance with respect to evaluating the serviceability limit state of concrete structures along this route.

2. EVALUATION OF CURRENT CRACK WIDTH CALCULATION METHODS ACCORDING TO EUROCODE 2 AND *fib* MODEL CODE 2010

There exists several proposals for crack width calculation methods in the literature. Borosnyói and Balász [2] provides a thorough overview. In the present work, special attention is given to the semi-empirical formulation according to Eurocode 2 [3] and *fib* Model Code 2010 [5], which often are straightforward in use for relatively simple geometries, e.g. *conventional* beam and slab dimensions. The formulas can in its most simplistic form be presented as

$$w_k = (\varepsilon_{sm} - \varepsilon_{cm})S_{r,max} \tag{1}$$

where ε_{sm} and ε_{cm} are mean steel and concrete strains respectively and $S_{r,max}$ is twice the maximum transfer length $l_{t,max}$ to each side of a crack, which is composed by a term related to the cover c and the ratio between the bar diameter and the effective reinforcement ratio $\frac{\varphi}{\rho_{s,ef}}$ as

$$S_{r,max} = 2l_{t,max} = 2k_{0.95}\left(k_c c + \frac{1}{4}\frac{f_{ctm}}{\tau_{bm}}\frac{\varphi}{\rho_{s,ef}}\right) \tag{2}$$

The difference in mean deformations of steel and concrete over the maximum crack distance yields the characteristic crack width w_k, which in order should not exceed a limit value for the crack width. Tan et al. [9] revisited the theory and the origin of the formulas to investigate the range of applicability of the formulas. It was found that the formulas stem from the slip and no-slip theory, which are based on completely opposite assumptions. The slip theory, which forms the basis for the expression $\varepsilon_{sm} - \varepsilon_{cm}$ in (1) and the second term in (2), assumes that a physical slip occur due to the deformation incompatibility between steel and concrete when a *primary* crack forms. An important assumption in this theory is that plane sections remain plane over the transfer length. In the no-slip theory, which forms the basis for the first term in (2), deformation compatibility between concrete and steel is ensured at the rebar level, thus generating no-slip between the materials. However, plane sections no longer remain plane and the concrete at the rebar level is deformed greater than the concrete surface. While the slip theory is based on equilibrium, compatibility and material laws for steel and concrete and the interface between the materials, the no-slip theory is based on an engineering practice claiming that strain compatibility is adjusted over a transfer length proportional to the cover. Neither theory was able to describe the physical reality related to cracking adequately, which in turn resulted in pragmatically merging the theories. Furthermore, terms in the expression for the mean strains $\varepsilon_{sm} - \varepsilon_{cm}$ in (1) and the products in front of c and $\frac{\varphi}{\rho_{s,ef}}$ in (2) are adjusted empirically, highlighting the semi-empirical formulation.

The nature of the formulas leads to physical inconsistencies that opposes the basic principles in solid mechanics, and become somehow peculiar from a structural engineering point of view. Also, the notion that the empirical adjustments are based on test results from relatively small specimen limits a further generalization of the formulas and care should be taken in its application to large concrete structures.

3. TENSION CHORD MODEL

The discussion above suggests that a more consistent crack width calculation model should be established. A first step in improving the current formulas is by understanding the physical reality related to cracking better, where the bond interaction between steel and concrete is essential. The significance of bond can be elucidated by considering a concrete bar pulled in the reinforcement steel. At relatively low loads, the bar behaves according to the no-slip theory with adhesion governing the bond interaction. When the load increases, the adhesion eventually breaks down and a mechanical bond due to the interaction between the bar ribs and the surrounding concrete takes over. If the localized concrete strains adjacent to the steel caused by bond stresses exceeds the tensile strength of concrete, small internal cracks arises in an inclination towards the primary crack, see Figure 1. Goto [7] was the first to discover the phenomenon and several authors have later also confirmed the acclaimed behaviour.

Considering equilibrium, compatibility and linear elastic material laws for steel and concrete in an infinitesimal element of the discussed concrete bar, a second order differential equation for the slip can be derived for the tension chord model. To account for the non-linearity acting in the interface between concrete and steel, a local bond-slip curve according to *fib* Model Code 2010 [5] is utilized in this study. One of the main advantages of such a formulation is that the "uncoupled" terms in (1) now are completely interrelated for an inflicted steel strain at a crack. This provides a considerably more consistent formulation with respect to the basic principles in solid mechanics than the approach adopted by Eurocode 2 [3] and *fib* Model Code 2010 [5]. However, the analytical solution of the second order differential equation for the slip cannot be obtained in a closed form due its mathematical nature and must be replaced with an infinite series [9].

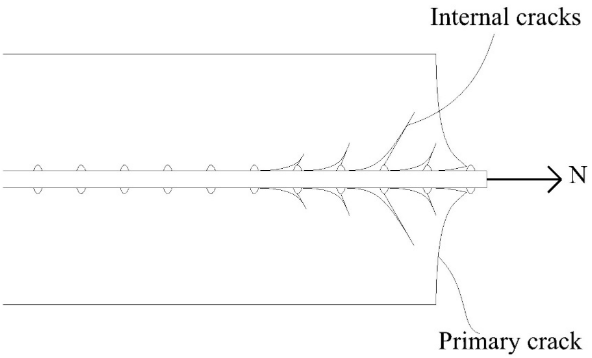

Figure 1 – Formation of internal cracks [11].

4. SKEW CRACKS TO AN ORTHOGONAL REINFORCEMENT

Both the Eurocode 2 [3] and the *fib* Model Code 2010 [5] proposes a method for calculating the distances between skew cracks, but do not give any further provisions in calculating the difference in mean steel and concrete strains normal to a skew crack. A possible way of calculating skew cracks is by utilizing the Cracked Membrane Model (CMM) [8]. One of the main advantages of the CMM is that it is based on a chosen tension chord model to account for the tension stiffening between skew cracks. Hence, with a proper tension chord model, provisions for calculating skew cracks can be provided.

5. THE IMPACT OF COVER

One of the inconsistencies in the formulas according to Eurocode 2 [3] and *fib* Model Code 2010 [5] seems related to the cover term in (2). There is in general no consensus in the literature regarding the relevance of the cover term. While some authors argue that the cover term should be governing [1], others claim that it should be abandoned [4]. Hence, to increase the understanding of the problem, an experimental study involving tensile tests of four prisms with dimensions 400x400x2000 mm has been initiated. Two of the prisms have the same reinforcement ratio (8ϕ20), with cover 40 mm and 90 mm, respectively. The other two cross sections have the similar variation in cover, however, with a larger reinforcement ratio (8ϕ32).

6. CONCLUSION

Current crack width calculation methods according to Eurocode 2 and *fib* Model Code 2010 are based on a formulation that leads to physical inconsistencies, This, limits their general applicability. One of the inconsistencies seems related to the impact of cover, and an experimental study of reinforced concrete prisms has been initiated to investigate its relevance. A study to improve the formulas is also currently underway by looking to the tension chord model using a bond-slip relationship according to the *fib* Model Code 2010 and solving the resulting differential equation for the slip. With a proper tension chord model, provisions for calculating skew cracks can further be provided.

ACKNOWLEDGEMENT

The work presented in this paper is part of an ongoing PhD study funded by the Norwegian Public Roads Administration as a part of the Ferry-free coastal route E39 project. Reignard Tan would like to express his outmost gratitude to the financers, supervisors and colleagues at Multiconsult ASA for contributions and making this PhD study possible.

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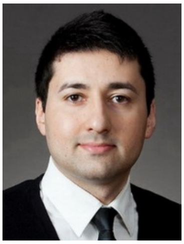
Tensile breakout capacity of cast-in-place headed anchors in concrete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ABSTRACT
The influence of member thickness, size of anchor head, and orthogonal surface reinforcement on the tensile breakout capacity of cast-in-place headed anchors was studied both numerically and experimentally. The aim of this paper is to form a background for developing improved methods for the design of new fastening systems as well as the assessment of the current anchorage systems in practice. Numerical and experimental results showed that the tensile breakout capacity of anchor bolts increases by increasing the member thickness or if surface reinforcement is present. Furthermore, the anchorage capacity increases with increasing the anchor head size.

Key words: Modelling, Pullout Testing, Concrete Cone Breakout, Concrete Splitting, Anchor Bolt, Headed Anchor, Fastening System, Member Thickness, Anchor-Head Size, Surface Reinforcement, Structural Design.

1. INTRODUCTION

Fasteners of different kinds are often used to anchor loads in concrete structures. The cast-in-place anchors have been used ever since reinforced concrete was introduced around 1900. The post-installed anchors started to be used in the 1960ies with the advances in drilling technology of concrete structures. An overview of the technology is given in [1].

Design models for fastening systems are often conservative and based on the simplifying assumptions that the size of anchor head, the surface reinforcement in concrete walls and slabs, and the thickness of concrete member have negligible influence on the anchorage capacity and performance. A need for more refined and reliable models has been called for in connection with a planned increase of the loads in Swedish power plants. For this purpose, an extensive numerical Finite Element (FE) study and a series of experiments were carried out to systematically evaluate the influence of member thickness, size of anchor head, and orthogonal surface reinforcement on the tensile breakout capacity of headed anchors in concrete.

2 REVIEW OF CURRENT DESIGN METHODS

The current methods for predicting the concrete tensile breakout capacity of fastening systems are very simple and do not explicitly consider the influence of member thickness, surface reinforcement and size of anchor head. The mean concrete tensile breakout capacity of a single anchor, far from concrete free edges or adjacent anchors, in uncracked concrete can be evaluated by the Concrete Capacity (CC) method [1] as follows:

$$N_{u,m} = 16.8\sqrt{f_c}\,h_{ef}^{1.5} \tag{1}$$

where $N_{u,m}$: mean concrete cone breakout capacity of a single anchor [N], f_c: concrete cylinder compressive strength [MPa], h_{ef}: anchor effective embedment depth [mm]. The CC method is derived based on a large number of anchor pullout tests at various embedment depths. The tested anchors were embedded in un-reinforced and reinforced concrete members with various thicknesses. In addition, tested headed anchors also had various head sizes and thus showed wide scatter in the obtained capacities.

It is believed that it is possible to refine the CC method if the influence of member thickness, amount of surface reinforcement and size of anchor head is taken into account.

3 NUMERICAL EVALUATIONS

For the purpose of this research, an extensive numerical study was carried out using the FE code MASA [2,3]. Cast-in-place headed anchors at various embedment depths (h_{ef}=50–500 mm) were simulated in plain and reinforced concrete members of various member thicknesses (H=1.5–5.0·h_{ef}). The simulated anchors had also various head sizes (i.e. small, medium and large heads). In addition, the concrete slabs were considered to be lightly-reinforced (ρ=0.3%) and over-reinforced (ρ>0.5%) to also evaluate the influence of reinforcement-content on the anchorage capacity. In addition, the length (L) and width (W) of concrete slabs, for all embedment depths of anchors, were scaled systematically and were proportional to the anchor embedment depth (L=W=6.0·h_{ef}). The typical geometry of specimens and the discretized FE model are shown in Fig 1. Due to a symmetrical geometry, only a quarter of the specimens were simulated by introducing double symmetry boundary conditions to save the time of analyses. For a complete description of the FE simulation and procedure see Nilforoush et al. [2,3].

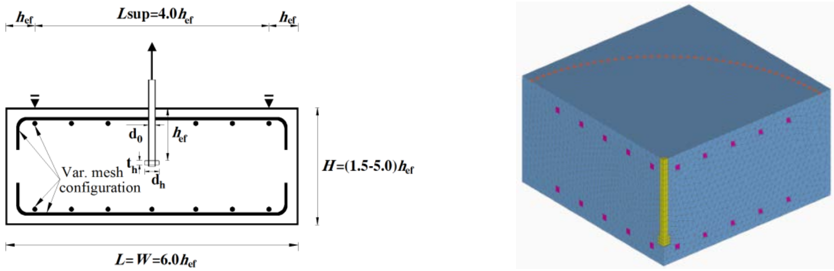

Figure 1 – Left: typical geometry of specimens. Right: FE discretized model.

4 EXPERIMENTAL STUDY

To verify the numerical findings and better clarify the influence of member thickness, size of anchor head, and orthogonal surface reinforcement on the anchorage capacity and performance, a supplementary experimental study was carried out [4]. A total of nineteen anchor bolts cast-in unreinforced and reinforced concrete slabs were tested under monotonic tensile loading. The testing parameters were the same as the numerical study. However, only one anchor size was tested due to financial limitations. The test setup is shown in Fig. 2. The effective embedment depth of tested anchors was h_{ef}= 220 mm. The length and width of concrete blocks for all specimens were identical (L=W=1300 mm), whereas the height of concrete blocks varied (H= 330, 440 and 660 mm). The reinforcement-content for the reinforced slabs was (ρ≈0.3%). For anchor pullout loading, the vertical reaction was taken up by a stiff circular steel ring with an inner diameter of L_{sup}= 880 mm. The anchor pullout loading was displacement-controlled by applying incremental deformations on the top of anchor shaft. The load was applied by means of a 100-ton hollow cylinder hydraulic jack.

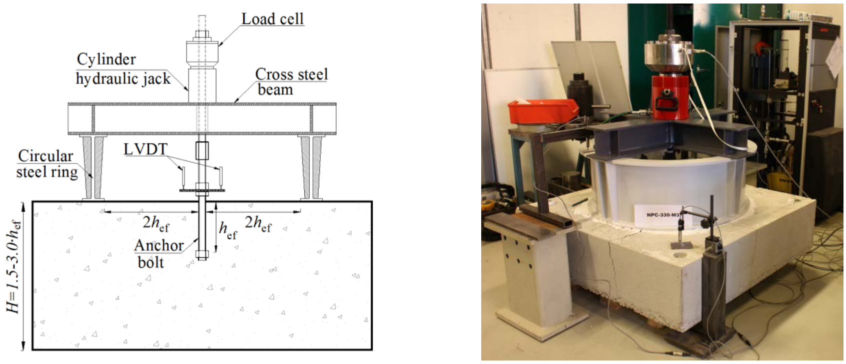

Figure 2 – Left: schematic view of test setup. Right: view of test apparatus at experiment.

5 RESULTS AND DISCUSSION

A selection of the results of this study is presented in the followings and additional results are given in Nilforoush et al. [2-4]. In unreinforced concrete slabs, it was found that the tensile breakout capacity of anchors increases up to 20% with increasing the member thickness (see the numerical and experimental results in Fig. 3 on the left and right, respectively). The anchorage is governed by concrete bending/splitting cracks in thin unreinforced concrete members H<2.0h_{ef} while it is governed by concrete cone breakout in thick members H≥2.0h_{ef} .

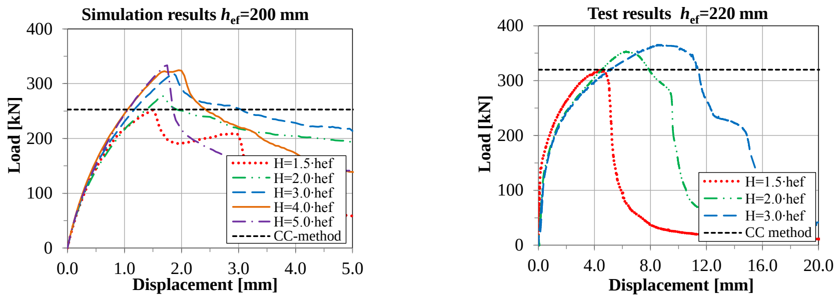

Figure 3 – Load-displacement curves for an anchor bolt in unreinforced concrete members of various thicknesses. Left: numerical results (h_{ef}=200 mm). Right: experimental results (h_{ef}=220 mm).

A comparison of load-displacement curves of anchor bolts in unreinforced and reinforced concrete members are shown in Fig. 4. The numerical and experimental results showed that the anchorage capacity and ductility increases if a light amount of orthogonal surface reinforcement is present (i.e. ρ=0.3%). Compared to the lightly-reinforced members, the over-reinforced members do not seem to affect the anchorage capacity and performance (see the numerical results in Fig. 4 on the left). The failure of all tested and simulated anchor bolts in reinforced concrete members was concrete cone breakout. In fact, the observed concrete bending/splitting failure in thin unreinforced concrete is prevented by surface reinforcement.

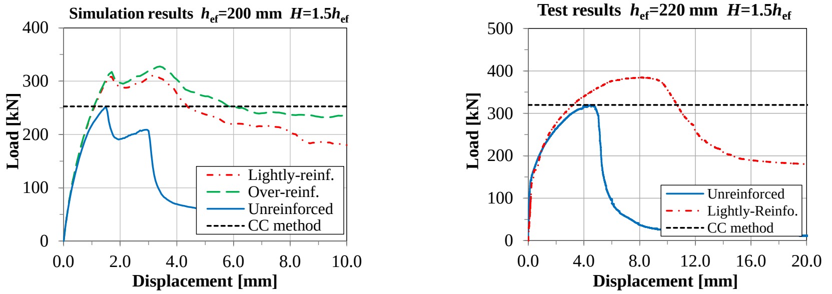

Figure 4 – Comparison of load-displacement curves of anchor bolts in unreinforced and reinforced members. Left: numerical results (h_{ef}=200 mm). Right: experimental results (h_{ef}=220 mm).

The experimental and numerical results of anchor bolts with various head sizes showed that the anchorage capacity and stiffness increases by increasing the head size (see Fig. 5). However, the post-peak anchorage behavior becomes brittle with increasing head size.

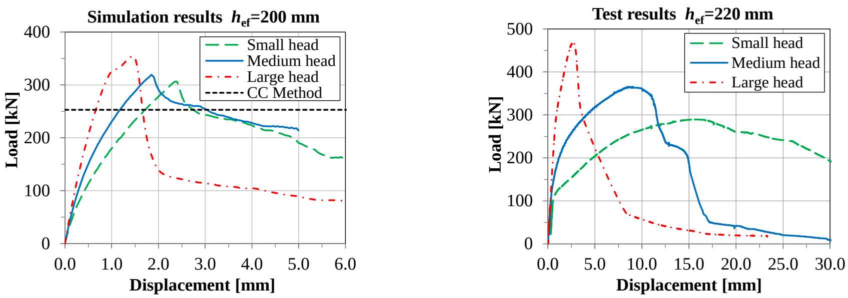

Figure 5 – Load-displacement curves of anchor bolts with various head sizes. Left: numerical results (h_{ef}=200 mm). Right: experimental results (h_{ef}=220 mm).

6 CONCLUSIONS

The present methods for predicting the tensile breakout capacity of anchor bolts can be refined if the influence of member thickness, surface reinforcement, and size of anchor head is considered.

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Session B2:

EXISTING STRUCTURES (I)
CONDITION ASSESSMENT,
ESTIMATION OF LOAD-CARRYING CAPACITY & REPAIR

Preventive bridge maintenance in Sweden - introduction to a PhD-project

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ABSTRACT
In Sweden there has been a small change of direction from using all funds in corrective bridge maintenance, to increasing the funds for the preventive bridge maintenance in the last decade.

The effectiveness of the current preventive bridge maintenance however, has not been examined. Neither has the optimum frequency or time period for performing preventive bridge maintenance. A PhD research project has therefore been initiated to try and answer these questions. This article will describe the current status of the project and the reasons for starting it.

Keywords: Preventive bridge maintenance, washing, sustainability, chlorides, testing.

1. INTRODUCTION

A PhD project focusing on preventive bridge maintenance in Sweden was initiated in September 2016 at the Swedish Cement and Concrete Research Institute in association with the Royal Institute of Technology. This paper will present the background reasons for the project, the current research activities and planned future activities.

2. BACKGROUND

According to EN 13306 [1] the definition of preventive maintenance is “maintenance carried out at predetermined intervals or according to prescribed criteria and intended to reduce the probability of failure or the degradation of functioning of an item”. The definition is suitable for this project but not perfect as it’s also too wide in the sense that it could include procedures that occur less periodically than the ones examined in this project.

In Sweden, there are approximately 30 000 bridges (span length exceeding 2 m, culverts are included) [2]. The Swedish Transport Administration (STA) is responsible for 20 900 of these bridges, of which about 16 500 are road bridges [3]. In total, 50 percent of these are built between 1950 and 1980 and have, thus, reached an age at which the need of maintenance and repair usually increases [4]. A large percentage (86 %) of these bridges are concrete bridges. During the last decade, STA has transferred resources from corrective to preventive bridge maintenance [5]. Presently, 10 to 15 percent of the budget is devoted to preventive maintenance whereas corrective maintenance, repair, and reconstruction comprise the remaining 85 to 90 percent. This reallocation has resulted in considerable efficiency gains but further savings are likely to be large. Preventive maintenance aims at measures to maintain the function of the bridge structure. Frequent measures include water washing, vegetation removal, crack repairs, material refill, and stretching of bridge railings. STA has defined a series of technical and operation procedural requirements to harmonize the preventive bridge maintenance. The scientific basis for the relationship between the requirements and the function of the bridge structure is unknown or weak. The aim of the preventive bridge maintenance is to prevent or delay the deterioration processes in order to maintain the function and prolong the service life of the bridge.

Demands from STA

The demands on bridge maintenance specified from the STA are either technical requirements or operation procedural requirements [6]. The technical demands concern the removal of growth and allowed crack widths. The operation procedural requirements concern the washing, where the water pressure and distance from the object are specified. For the cleaning of the drainage system, a combination of both technical and operation procedural requirements are used. In a previous version of the requirements, there were only technical requirements. The main reason for introducing the operation procedural requirements was to improve the performance of the maintenance since the technical requirements were difficult to interpret and verify.

3. AIM AND HYPOTHESIS

The overall aim of the project is to improve the preventive bridge maintenance in Sweden. Bridges are washed, debris and growth are removed, drainage systems are cleaned and small repairs are done as part of the yearly time controlled maintenance. The belief is that it will have a positive effect on the durability of the bridge. This thought is shared with the authors but the frequency and the time period of performed maintenance are not believed to be optimal in the current maintenance and could be improved. The hypothesis is that preventive maintenance improves the durability of the bridge and that a more frequent preventive maintenance performed at an optimal time period closer to the exposure period of de-icing salts can increase its positive effects. The initial goal is to obtain information about the current preventive bridge maintenance in Sweden. There is a need to analyse both the theoretical and practical aspects of the current preventive bridge maintenance. An underlying hypothesis is that there is a difference in performance and opportunities, both practical and economical ones, in different locations in the country. By identifying differences in practices and results of the preventive bridge maintenance it would be possible to establish a best practice.

4. CURRENT ACTIVITIES

The project is still in the early stages and the major objective is therefore to gather information and a literature review of the current research and practice concerning preventive bridge maintenance. The literature review can be divided into two parts.

1. Literature from science/research, which consist of books and articles about bridge management, deteriorating mechanisms and more.
2. Literature from bridge maintenance industry. Documents used by the people managing the bridges. Documents and contracts from STA and the city of Stockholm describing the demands, procedures and follow-ups concerning the preventive bridge maintenance.

The exploration of the Bridge and Tunnel Management system (BaTMan) has recently been started. In the database, there are a lot of important information about bridges, inspections, repairs and a lot more. It is a system used by the STA, many municipals and contractors to store important information about the bridges. It is considered a very important and useful instrument in the maintenance of the Swedish bridges. However, a lot of the preventive bridge maintenance is not registered in BaTMan but registered and controlled in other ways.

A series of interviews with people from different parts of the maintenance industry have been initiated. The interviews are about the interviewee's part in the process and generally about the maintenance and its requirements. The goal is to interview people whose knowledge covers all the aspects of maintenance, both theoretical and practical.

A survey to the municipals has started with questions about their bridge maintenance. The aim is to compare municipals and the STA as well as identify differences and similarities in size of population, geographical and economical influences and more.

5. FUTURE ACTIVITIES

The future activities cover continuation of the current ones as well as the further development of them. Interviewing different people in industry and in different regions of the country within the STA is to collect and identify differences and similarities in the procedures used for the preventive bridge maintenance.

One future activity that is of great importance is to study and evaluate the preventive maintenance steps in practice. This means following inspections and the yearly maintenance when the contractors do the washing, removing debris and cleaning the drainage system on the bridges. The goal is to get the practical knowledge about the actual procedures that influence the maintenance.

A field experiment is planned to start in the autumn of 2017. The aim is to examine the effect of washing on the durability of the concrete samples. The reason for selecting concrete is the fact that the majority of Swedish bridges are concrete bridges or a combination of steel and concrete. Depending on the possibilities at the site there may also be a possibility of testing different frequencies and periods of washing after application period of de-icing salts.

There is a wish to conduct a comparison study between two similar bridges with similar condition, exposure and bridge structure where the only difference is the maintenance. One of the bridges has been maintained and the other one not. The goal is to compare the long term effects of using preventive maintenance. The main problem however is to find suitable objects.

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Reliable engineering assessments of corroded concrete structures

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ABSTRACT
Corrosion of steel reinforcement is a large and increasing problem for reinforced concrete structures. Simple and reliable assessment methods are required to use the full capacity of existing infrastructure. In this paper, a reliable engineering assessment method is outlined. A model for anchorage has been developed and verified against a large database of bond tests. Also the influence of corrosion on the bending and shear capacity is to be included, and a probabilistic model will be established. The outcome of this work will enable practising engineers to perform reliable assessments of concrete structures with corroded reinforcement.

Key words: Corrosion, Reinforcement, Concrete, Bond, Modelling, Assessment, Sustainability.

1. INTRODUCTION
This paper presents an overview of the development of a bond model for corroded reinforcement in concrete structures meant for engineering purposes. In this introductory

section, first a general introduction is given, followed by a summary of previous work on the topic and the section is ended with a description of how the development is to be performed.

1.1 General

Concrete structures under service conditions are unavoidably exposed to processes that with time may affect their ability to fulfil the structural requirement, corrosion of steel reinforcement being the most common [1]. Although many of society's large investments, e.g. bridges in reinforced concrete, already show significant corrosion damage with cover cracking and spalling of the concrete cover, the deterioration is expected to become more severe in the future due to climate change [2]. The demands for load-bearing capacity are nevertheless often increasing with time. In order to meet future needs in an environmentally friendly and economic way, without unnecessary re-constructions, advances are needed in the methods for structural assessment of existing structures.

Main consequences of reinforcement corrosion in concrete structures

The reinforcement in structural concrete is originally covered by a passivating layer, due to the alkalinity of the surrounding concrete. The corrosion process can begin only once the passivation is broken, due to e.g. ingress of chloride ions or carbonation [3]. Reinforcement corrosion affects a concrete structure in several ways, the most important being:

- Loss of reinforcement bar cross section
- Loss of reinforcement ductility and strength
- Loss of bond between reinforcement and surrounding concrete
- Cracking and spalling of the concrete cover

On the structural level, the local effects mentioned above reduce the load-bearing capacity for shear forces and moments, influences the tension stiffening of the member and thus also the deflection and crack widths. Furthermore, the capacity for plastic rotation is affected, consequently also the moment redistribution in indeterminate structures, the robustness and seismic resistance [4]. Reinforcement corrosion may lead to an abrupt collapse instead of the desired ductile failure sought in the design of structures. For example, bond failures can occur in anchorage zones and curtailment ends of bridge beams, leading to an abrupt collapse as a consequence.

1.2 Previous developments

In previous work, a simple analytical model for the assessment of anchorage in corroded reinforced concrete structures was established based on the local bond-slip relationship in *fib* Model Code 1990 [5]. Later verifications include test results from naturally corroded specimens, and 3D NLFE analyses and experiments of highly corroded specimens with spalled off cover [6]. The practical importance of the model, denoted ARC1990, has been shown in a pilot study of two bridges [7]. Approximately €3 million was saved by avoiding unnecessary strengthening for these two bridges alone, indicating that more widespread use of simple models for assessment can lead to enormous cost savings for society.

1.3 Approach

The development of a reliable model for assessing the structural effects of reinforcement corrosion and evaluating the remaining service-life of bridges with corroding reinforcement has been divided into three main points:

- Enhancing and validating the ARC1990 model for structural assessment of concrete brides with corroded reinforcement for engineering purposes;

- Establishing a probabilistic framework by incorporating uncertainties to enable reliability based structural assessments and calibrating safety factors for deterministic analysis;
- Demonstrating the use of the model through a case study.

2. DEVELOPMENT OF STRUCTURAL ASSESSMENT MODEL

In a recent work, the analytical model for bond strength assessment, mentioned in Section 1.2, has been further developed; *fib* Model Code 2010 has been implemented and the model has been verified against a database of 500 bond tests [8]. A comparison between the relative average bond strength (i.e. corroded divided by uncorroded strength) from the database compared to the developed model, denoted ARC2010, can be seen in Figure 1 for cases with stirrups.

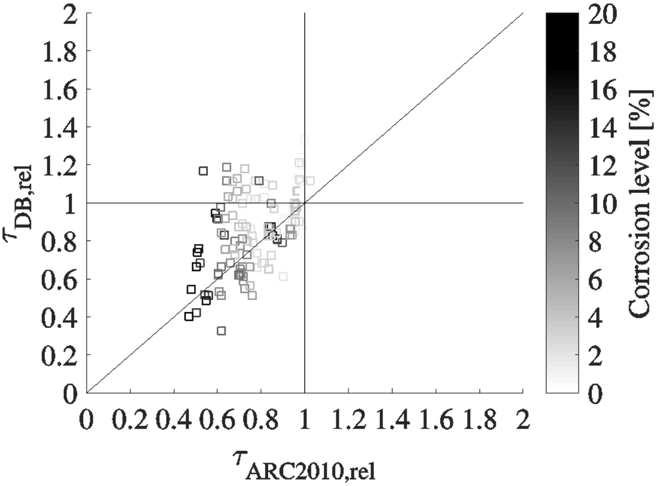

Figure 1: Comparison between bond test database and ARC2010 in terms of relative bond strength.

Current work includes studying the confining effect of stirrups dependent on the position of the bars, corner versus middle positions, as well as the reinforcement layer in a multilayer rebar configuration. Furthermore, the effect of corroding reinforcement on the shear capacity is investigated, mainly focusing possible changes of the admissible shear angle and the effect of stirrups carrying additional tensile stresses induced by corroding main reinforcement bars.

3. PROBABILISTIC MODELLING

The design of new structures as well as analysis of existing ones are affected by many uncertainties; both the applied load and the load-carrying capacity of a structure are uncertain [9]. As a consequence, the probability of failure for a structure will never be zero; instead a finite probability of failure should be met.

The analytical model presented in Section 2 will be established within a probabilistic framework, where uncertainties of the input parameters are accounted for. Uncertainties can be categorized as e.g. physical uncertainty, statistical uncertainty and model uncertainty. Physical uncertainty refers to the randomness in nature e.g. the variation in concrete strength; statistical uncertainty describes the uncertainty in parameter estimation based on available data; and model uncertainty refers to the fact that the model itself is an imperfect representation of reality.
The uncertainties of the basic variables for the model (physical and statistical uncertainties) will be described using data from literature, e.g. JCSS Probabilistic Model Code [10]. The modelling

uncertainty of the calculation model will be estimated using the bond test database mentioned in Section 2. Use of Monte Carlo simulations will enable establishing the distribution function of the load-bearing capacity. The probabilistic model will be used for calibrating modification factors for the deterministic resistance model for use within a semi-probabilistic safety concept, e.g. Eurocode. The probabilistic model will also be used for service life predictions. A schematic view of the probabilistic model can be seen in Figure 2.

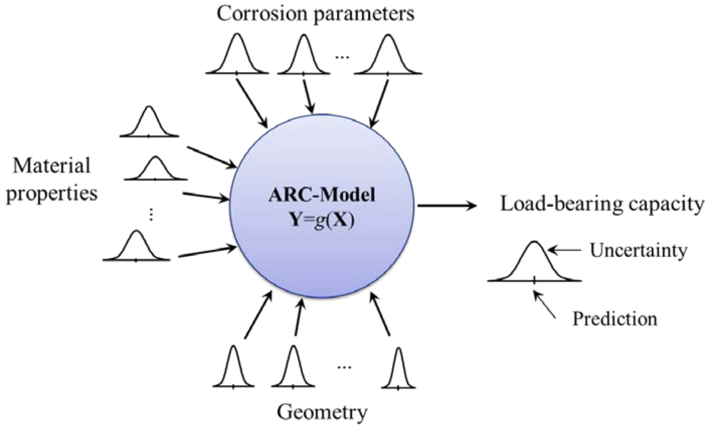

Figure 2: Illustration of the uncertainties of the input variables and the model output as a distribution

4. CASE STUDY

The relevance of the model in a practical context will be verified by application in a case study of an actual corroded concrete bridge. The study will include analysis of inspection data, existing drawings and previous calculations, and the assessment method will be applied to both estimate load-bearing capacity and to make a service-life evaluation of the studied bridge. The case study will give valuable information concerning application of the assessment method, and life cycle analyses will be used to quantify the method’s environmental and economic impacts.

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Quality control plans for European concrete road bridges
Experiences from cooperation within COST Action TU 1406

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ABSTRACT
The main objective of the COST Action TU1406 is to develop a guideline for the establishment of Quality Control plans for roadway bridges, by integrating the most recent knowledge on performance assessment procedures. 36 European countries are working together in this endeavor during 2015-2019.

Key words: Cracking, Corrosion, Execution, Modelling, Performance Indicators, Repair, Reuse and Recycling, Shrinkage, Structural Design, Sustainability, Testing, Quality Control Plans, Quality Specifications.

1. INTRODUCTION

1.1 General

During the implementation of asset management strategies, maintenance actions are required in order to keep assets at a desired performance level. In case of roadway bridges, specific performance indicators are established for their components. These indicators can be qualitative or quantitative based, and they can be obtained during principal inspections, through a visual examination, a non-destructive test or a temporary or permanent monitoring system.

Then, obtained indicators are compared with performance goals, in order to evaluate if the quality control plan is accomplished. There is a large disparity in Europe regarding the way these indicators are quantified and how such goals are specified. Therefore, this action brings together both research and practicing community from 36 countries in Europe in order to accelerate the establishment of a European guideline in this subject [1]. The project is supported by COST, the longest running European framework supporting transnational cooperation among researchers, engineers and scholars in Europe [2]. The COST Action TU1406 on Quality Control Plans for Bridges was started in 2015 and is planned to be working during four years. It has an estimated economic dimension of 128 M€ and will receive about 650 k€ from EU. This paper will focus on applications for bridges of concrete.

1.2 Background

In engineering, quality control (QC) is related to systems development in order to ensure that products or services meet or exceed the expectations and needs of users and the wider community. Concerning road infrastructures, it can be said that asset management and QC are two sides of the same coin. There is an increasing need of developing strategies to ensure the quality of the entire system, with the aim of reducing the risk of unexpected costs.

For this purpose, the authorities need to produce an asset management plan, which should not only define the goals to be achieved by exploiting the roadway bridge network, but that should also identify the investment needs and priorities based on a life cycle cost criteria. In addition, a proper condition assessment of these assets must be conducted to support the decision-making process regarding their preservation. A set of maintenance operations, carefully planned and executed at proper time, is then established through this process. This will allow to reduce the risk of further deterioration, minimize costs and, simultaneously, ensure the quality of delivered service.

The identification of maintenance needs is more effective when developed in a uniform and repeatable manner. This process can be accomplished by the evaluation of performance indicators, improving the planning of maintenance strategies, see Figure 1.

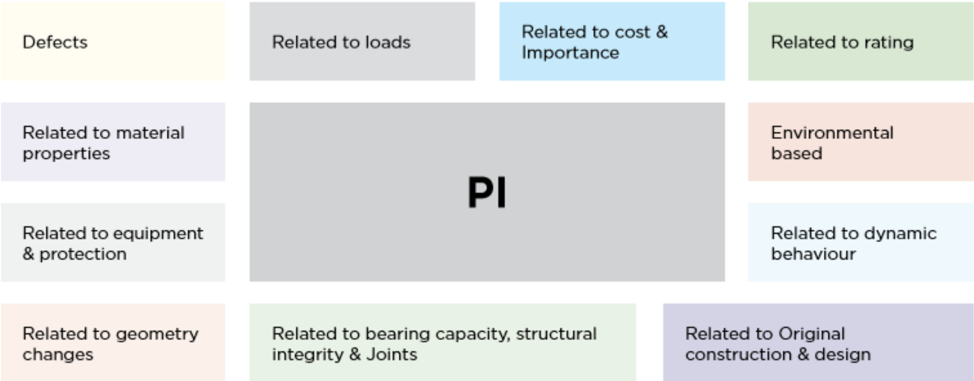

Figure 1 - Main clusters of Performance Indicators related terms, [3].

In this context, a first step would be the establishment of specific recommendations for the assessment of roadway bridges, namely, used methods for the quantification of performance indicators. A second step would be the definition of standardized performance goals.

2. ONGOING WORKS

The following specific objectives/deliverables are in progress: (i) to systematize knowledge on Quality Control (QC) plans for bridges; (ii) to collect and contribute to up-to-date knowledge on performance indicators; (iii) to establish a wide set of quality specifications through the definition of performance goals, aiming to assure an expected performance level; (iv) to develop detailed examples for practicing engineers on the assessment of performance indicators; (v) to create a data basis from COST countries with performance indicator values and respective goals; (vi) to develop a webpage with information about the Action; (vii) to support the development of technical/scientific committees; (viii) to disseminate activities, such as Short-Term Scientific Missions (STSM), training schools and other teaching activities (e.g. e-lectures), for practicing engineers and researchers, regular workshops and conferences.

A first report on Performance Indicators has been issued [3]. Main sub-systems have been discussed, Figure 2 and main approaches in damage detection, Figure 3, see e.g. [4], [5].

Figure 2 - Three main sub-systems: substructure, superstructure and roadway/equipment, [3].

Figure 3 - Main approaches in damage detection: visual inspection, non-destructive testing, probing and structural health monitoring (SHM), [3].

3. SOME PRELIMINARY CONCLUSIONS AND OUTLINE OF FUTURE DEVELOPMENTS

At the level of an Operational Database, more work is necessary to identify key performance indicators. Further extension of the Operational Database with the Research-based one should help in the following two main tasks:

- to select the most important Performance Indicators for achieving Performance goals that are crucial for optimal Quality Control Plan within bridge management
- to allocate them with appropriate weights (importance level).

In order to select the most important Performance Indicators the following steps should be followed:

- Define crucial Performance Goals (for example: safety, serviceability, reliability, durability, availability, maintainability, ...)
- Categorise Performance indicators in relation to Performance Goals (at different levels: component, system, network; taken into account different aspects: technical, sustainability, socio-economic),
- Answer following questions: − Is it measurable? − Is it quantifiable? − Is target value available? − Is it valid for ranking purposes? − Does it allow decision with economic implications?

The overall database should include the most important indicators for achieving the goals crucial for optimal quality control.

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[5] MAINLINE. A European FP 7 research project 2011-2014 with 19 partners with the full name: MAINtenance, renewaL and Improvement of rail transport iNfrastructure to reduce Economic and environmental impacts. A project summary and 27 deliverables are available at www.mainline-project.eu

Session B3:

EXISTING STRUCTURES (II)
CONDITION ASSESSMENT,
ESTIMATION OF LOAD-CARRYING CAPACITY & REPAIR

Assessment of the load-carrying capacity of existing structures with corroded smooth reinforcement bars

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ABSTRACT
Reinforced concrete bridges are common in Sweden and often damaged by reinforcement corrosion that reduces the safety of the structure. This issue has been addressed in several research projects with, however, a strong focus on ribbed bars, while further knowledge about smooth bars is still needed. Edge beams from a bridge in Gullspång with naturally corroded smooth bars will be studied so as to provide benchmark data for the assessment of the load-carrying capacity of existing structures with corroded smooth bars. Structural tests are carried out on several beams, with varying amounts of corrosion damage, measuring anchorage properties such as applied load and end-slip.

Key words: Cracking, Corrosion, Modelling, Bond, Anchorage, Smooth Reinforcement, Testing.

1. INTRODUCTION
Nowadays, a lack of models capable of assessing the load-carrying capacity of existing structures with corroded smooth reinforcement bars, may result in unnecessary strengthening measures or even in the replacement of the structures.

Reinforced concrete bridges are common in Sweden and often damaged by reinforcement corrosion, the most common cause of deterioration [1]. Corrosion of reinforcement affects the structure in two ways: a) volume expansion of corrosion products, which affects the bond between reinforcement and concrete, causing the concrete cover to crack and spall, and b) area reduction and change of ductility of the reinforcement bars. Both effects reduce the safety of the structure. Many existing structures already show significant corrosion damages; Wang [2] analysed the impacts of climate change and showed that the deterioration of concrete structures is expected to become even worse than today. In addition, the demand for load-carrying capacity often increases over time. Thus, there is a growing need for reliable methods to assess the load-carrying capacity and remaining service life of existing structures. The issue of reinforcement corrosion has already been addressed in several research projects, such as Duracrete [3], Contecvet [4], and Sustainable Bridges [5]. Existing research has, however, a strong focus on ribbed bars, while structures built before 1940 have smooth bars. Ribbed bars started to be used towards the end of the 1940s, and for many years, ribbed and smooth bars were used in combination. Very few experiments on the bond of corroded smooth reinforcement exist; the only ones found in literature are [6-9], all of which used accelerated/artificial corrosion processes set up in laboratory conditions. A major disadvantage with almost all research available on the subject today is, in fact, that it is based on artificially corroded specimens, through the application of electric current. There are several uncertainties concerning how well that corresponds with corrosion that takes place in real structures. Accordingly, experiments with naturally corroded specimens are needed. Furthermore, the results from previous research show a strong dependency of the bond capacity on the presence of transverse reinforcement that needs to be investigated.

2 EXPERIMENTAL STUDY
Edge beams from a bridge in Gullspång with naturally corroded smooth bars will be tested to produce benchmark data and to assess the load-carrying capacity of existing structures with corroded smooth bars.

2.1 The Gullspång bridge
The bridge was built in 1935, and torn down in February 2016 due to heavy corrosion damages. The edge beams taken from the bridge were carefully inspected and documented before the bridge was demolished. The edge beams were then cut off, into 4-metre-long segments, and kept

for research purposes, resulting in a total of 20 beams with varying amounts of corrosion damage (see Fig.1). The specimens are naturally corroded and spalling, crack patterns, and crack widths of the surrounding concrete on the bridge were observed and documented.

Figure 1 – On the left, the bridge in Gullspång before removal, photo from below showing the bottom side of the edge beam. On the right, a close-up of an edge beam and one of the edge beams after removal, note the hooks.

The specimens are made of concrete with a required strength of 30 MPa and smooth reinforcement bars with end hooks, as typical of the time of the construction of the bridge; similar materials are used in numerous structures. The cover, which is about 30 mm, is common for structures of that age as well. The edge beams have been exposed to deicing salts and freezing, which is typical environment for bridges, and parking garages. Furthermore, harbour structures are exposed to salt as well; the specimens are thus relevant for many more structures than this specific bridge, making the results of this study of a general nature.

2.2 The experimental set-up

A four-point bending, asymmetrical and indirectly supported, test configuration (as can be seen in Fig.2), is being taken into consideration for the performance of structural tests on the beams, similar to the set-up used for testing of edge beams from Stallbacka bridge [10]. The aim is to reach anchorage failure after the development of a shear crack. The set-up allows to avoid gripping of the bars, that could potentially modify the bond-slip capacity, as well as to avoid external transverse pressure acting in the anchorage region: for a thorough discussion see [10]. The asymmetry of the set-up is due to the need of knowing the anchorage failure side, on which DIC will be used for monitoring the bars slips. This test setup is being confronted with several other possible solutions. The edge beams have one reinforcement bar with 16 mm diameter in each corner, and stirrups that are open on one side: some beams will be tested in the same direction as when placed on the bridge; others will be tested upside down. This enables the study of varying confinement effects of the stirrups and the effect of top-cast and bottom-cast bars. Furthermore, the smooth bars have hooks at the ends, as was common to improve anchorage (as can be seen in Fig.1). For specimens with less corrosion damage, X-ray scanning will be used to locate the position of the hooks before cutting the specimens, to test specimens both with and without end hooks. Before testing, the edge beams will be grouped per similar damage pattern, in terms of cracking and cover spalling. During the tests, the applied load, deflection and end-slip will be measured and the crack pattern carefully documented. After structural testing of the beams, material tests will be performed on undamaged concrete, as well as on the corroded reinforcement bars. The aim is to investigate anchorage capacity of the corroded smooth bars, and, ultimately, to link bar properties, such as anchorage capacity, corrosion level, tensile capacity and ductility, to the documented visible damage, so as to provide guidance on how to obtain the load-carrying capacity of structures from visual inspection.

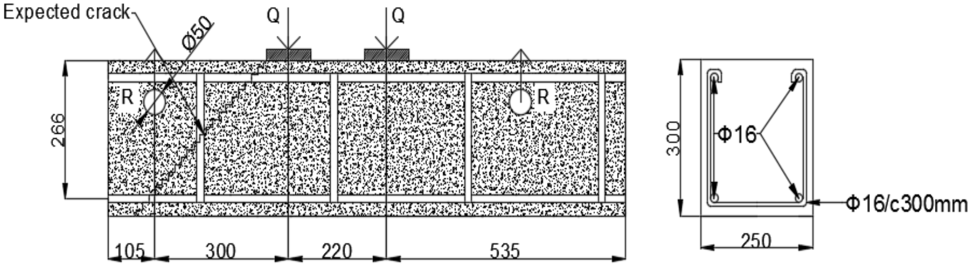

Figure 2 – Left: Planned test set-up with expected crack. Right: Cross-section of the edge beams. Measurements in mm.

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The effect of reinforcement corrosion on the structural behaviour of prestressed bridges in the Norwegian coastal regions

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ABSTRACT

Prestressed concrete (PC) is one of the main structural materials used in Norway for bridge structures. Prestressed and precast concrete beams were even used for building the historical Atlantic Road, which has the status of a National Tourist Route and is of strategic importance for the infrastructure. Bridges located along the Norwegian coast are exposed to an aggressive marine environment, containing chlorides, which can be the cause of reinforcement corrosion. Severe deterioration of prestressed concrete structures may lead to serious consequences, including sudden and brittle failure. Reliable assessment of existing damages, as well as prediction of residual behaviour and service life is crucial for PC.

Key words: Pre-stressed concrete, Deterioration, Corrosion, Stress Corrosion Cracking

1. INTRODUCTION

In the past few decades, prestressed concrete has been widely used in Norway for construction of bridges. According to the Norwegian bridge management system Brutus [1], more than 100 bridges have been built in the coastal areas using the popular prestressed and precast concrete beams type NIB (normalized I-beams), with more than 20 of them in a harsh coastal environment. Some of them are listed in the Table 1. Apart from NIB beams other types (NOB, NOT and MOT) were used in bridges. In these beam types, the reinforcement is pretensioned before the concrete is cast and their steel strands are lying naked in the concrete (without ducts). Moreover, increased use of prestressed elements in new-build structures can be motivated by several advantages of pre-stressing like significantly increased span length, reduced material, costs and time-to-build of the bridge. However, deterioration of prestressed structures can lead to serious consequences affecting structural safety. In marine environment, PC is exposed to seawater and deicing salts, both containing chlorides. These aggressive ions may penetrate the concrete cover, reaching finally the reinforcement depth. When a certain chloride concentration (the chloride threshold value) is obtained on the steel level, localized corrosion (pitting) may initiate and propagate, causing, in worst case, sudden and brittle failure of the prestressing wires. Although prestressing reduces, or even fully prevents, flexural cracking of the structural elements in comparison with traditional reinforced concrete, in older structures cracks may occur due to increasing service loads, concrete aging and deterioration. These main cracks will facilitate chloride ingress and might increase risk of corrosion initiation. Considering the age of existing bridges with prestressed beams, their current state, any possible deterioration and its effect, as well as the residual service life, need to be carefully assessed.

Table 1 –Pre-stressed, NIB beam bridges in Norway, in harsh coastal environment [1].

No.	Bridge	Location	Year of construction	Length [m]
1	Rong II	Hordaland	1972	22.7
2	Skrubbholmsundet	Hordaland	1989	25.4
3	Dampleia	Møre og Romsdal	1986	161.0
4	Hulvågbrua	Møre og Romsdal	1987	208.0
5	Myrbærholmbrua	Møre og Romsdal	1987	87.0
6	Hestøysundet bru	Møre og Romsdal	1987	69.5
7	Geitøybrua	Møre og Romsdal	1988	45.0
8	Storlauvøybrua	Møre og Romsdal	1988	45.5
9	Litllauvøybrua	Møre og Romsdal	1988	116.5
10	Askjesundet	Rogaland	1991	170.0
11	Herdlevær	Hordaland	1979	103.0
12	Søre Vetterhusstraumbrua	Nord-Trøndelag	1986	23.0
13	Mastadsvaet	Sør-Trøndelag	1992	153.5

2 EFFECT OF CORROSION ON PRESTRESSED AND ORDINARY REBARS

Prestressed concrete beams usually consist of both, ordinary reinforcement (stirrups, additional longitudinal rebars) and prestressing high strength steel loaded in tension [2]. Prestressed reinforcement is mostly arranged as strands consisting of several small-diameter wires, twisted around each other. Considering the rather small size of a single wire (typically 4 mm), corrosion will rapidly lead to a relatively great reduction of the cross-sectional area, when comparing to large-diameter, ordinary bars [9], and thereby to a local increase of the already high stresses in the wire. Cross-section loss, particularly localised, will have significant impact on mechanical performance of both, prestressed and ordinary rebars.

Chloride-induced corrosion changes the steel reinforcement behaviour from ductile to more brittle, and this effect was observed for both types of reinforcement, ordinary and prestressed [3, 5, 6, 9]. The main cause of the reduced ultimate strain for ordinary bars is a substantial local loss of the cross section (pits). Risk of brittle failure is even more pronounced for corroded high strength steel, where fracture can occur due to different mechanisms than the usual chloride-induced corrosion. Sudden failure of the prestressing wire can be the result of stress corrosion cracking, hydrogen-induced cracking or hydrogen-induced stress corrosion cracking [3, 7]. That kind of damage in high strength steel takes place only if its surface has some defects: notches or sharp cracks [7]. Initiated pits can be considered as one of the material defects resulting in stress corrosion crack development.

In addition to a significant decrease in ultimate strains, degradation of other mechanical parameters of corroded, stressed wires can be observed [3]. Increased corrosion results in decreased yield and ultimate stresses, which are calculated from the applied load and residual cross-section in a pit location [3]. This phenomenon was not observed for true yield and ultimate stresses in ordinary reinforcement. While the nominal yield and ultimate strength for regular bars reduces with the degree of reinforcement corrosion, the true yield strength of all corroded and control bars had the same value, meaning that corrosion hardly alters the yield behavior of the steel [8]. Another parameter of prestressed steel affected by corrosion is the apparent Young's modulus [3]. A decrease in the elastic modulus and yield limit was explained by material damage due to the evolution of microcracks and microvoids in wire exposed to stress corrosion cracking.

The mechanical behaviour of corroded prestressing reinforcement, as well as its maximum mass loss, is strongly dependent on the stress level. According to [3], for wires loaded up to 80% of the yield strength f_y, the loss of mass was 10-15% higher than for unstressed ones. In addition, a reduction of the elastic modulus and yield strength was observed exclusively for stressed wires, while for unstressed wires only loss of ductility was detected [3]. Moreover, only for high levels of stress, brittle failure of prestressed wires was induced by stress corrosion cracking [3]. Experiments in [3] revealed, that for wires loaded up to 70% of f_y no brittle failure occurs. Similarly in [9], no brittle fracture was observed for stresses equal to 63% of ultimate tensile strength (less than 80% of f_y), although the failure mode changed to less ductile. It seems that the stress level has a high influence on the mechanical performance of corroded prestressed reinforcement, and its increase leads to a decrease of the wire's service life [3]. It needs to be mentioned, that except environmental conditions and stress levels, the behaviour of prestressed reinforcement depends also on its metallurgical properties [7]. That is why mechanical properties need to be carefully assessed for relevant types of steel.

Nevertheless, severe chloride-induced corrosion, under certain loading conditions, may result in sudden, brittle failure of prestressing wire, leading to a reduction of prestressing force and bending stiffness of the element [3, 4]. Moreover, as local pitting does not affect the overall tension in the wire [3], corrosion damage may not be detectable in structural members before a brittle failure occurs [4]. In addition to the loss of cross section and the degradation of mechanical properties of corroded prestressed steel, also a degradation of the bond need to be taken into consideration when assessing structural behavior of PC.

3 HULVÅGBRUA AS PC BEAM BRIDGE IN NORWAY – CASE STUDY

Hulvågbrua is one of the eight road bridges belonging to Atlantic Road, which is part of Norwegian national road 64 (Rv 64) in Møre og Rømsdal. The bridge was built in 1987 [1], and opened for traffic in 1989. Hulvågbrua is placed between small islands in Norwegian Sea, only four meters above the waterfront, and is exposed to extremely aggressive marine environment. The main structure of the bridge consists in total of thirty six prestressed and precast longitudinal NIB beams (4 in cross-section), with maximum span length 26.56 m [1]. A reinforced concrete (RC) plate is placed on top of the NIB beams and designed as a composite. The I-shaped PC beams are made of C55 concrete, high strength prestressed strands, and ordinary reinforcement [2].

When assessing the load-bearing capacity of the PC, both concrete and steel deterioration need to be reliable evaluated, based on inspection and tests data. Degradation of concrete, bond, loss of the steel cross-section and possible reduction of mechanical properties of prestressing and ordinary steel need to be taken into account. Stresses in steel wires depend on loading conditions. Moreover, they are changing in time due to steel relaxation, concrete creep and shrinkage, possible increase in service loads, and concrete deterioration (particularly cracking). The stress level is also one of the major parameters affecting cross-section loss (due to corrosion) and mechanical behaviour of corroded prestresses steel. The relation between time-dependent environmental, material and loading conditions makes modelling of PC long-term performance complex. That is why the prediction of the residual service life and load-bearing capacity will be analysed with nonlinear finite element modelling.

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Assessment of concrete bridges - Structural capacity. Experiences from full-scale testing to failure of a bridge in Kiruna


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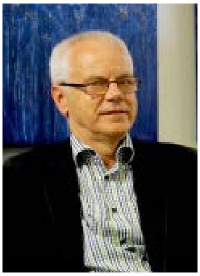
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ABSTRACT

To calibrate methods for condition assessment of prestressed concrete (PC) bridges, tests were carried out on a 55 year old five-span bridge with a length of 121 m in Kiruna in northern Sweden. Both non-destructive and destructive full-scale tests were performed. This paper presents results regarding methods for assessment of the structural capacity of concrete bridges.

Key words: Assessment, Cracking, Corrosion, Creep, Modelling, Pre-stress, Reinforcement, Structural Design, Sustainability, Testing.

1. INTRODUCTION

1.1 General

Assessment, repair and strengthening of existing bridges are required in order to meet current and future demands on sustainability of existing infrastructure. For instance, a survey carried out within the project MAINLINE (2015) [1], indicated a need in Europe for strengthening of 1500 bridges, replacement of 4500 bridges and replacement of 3000 bridge decks. It is believed that for some of these structures replacement can be avoided if more accurate assessment methods are used, see e.g. Sustainable Bridges (2007) [2], Nilimaa et al. (2016) [3] and Paulsson et al.(2016) [4].

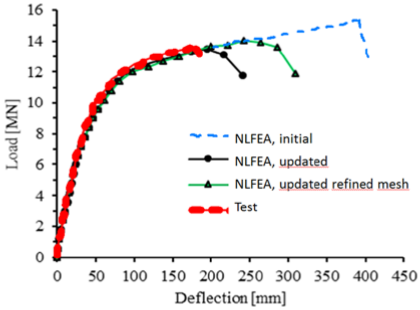

Figure 1. The Kiruna Bridge. Left: Photo with road E10 in the foreground behind birches, the earlier location of the Iron Ore Railway line and the LKAB iron mine in the background. Right: Load-deflection diagrams for the tested span (No 2 from the right): dashed thin blue - initial Non Linear Finite Element Analysis (NLFEA); black circles – updated NLFEA; black triangles with green line – updated NLFEA with refined mesh; and dashed thick red line – test results. Bagge (2017) [5].

1.2 The Kiruna Bridge

The test object, located in Kiruna, Sweden, was a viaduct across the European road E10, see Figure 1. It was constructed in 1959 as a part of the road connecting the city centre and the LKAB mining area. Due to underground mining, the entire area underwent excessive settlement. The owner of the bridge, LKAB, closed it in October 2013 and planned to tear it down in September 2014. Before demolition, the bridge was tested in a research project which will be discussed below [5]-[7].

2 ASSESSMENT METHODS

Structural analysis has a crucial role in bridge assessments. To support rational improvements of this analysis, a multi-level strategy has been developed, Bagge (2017) [5]. By gradually increasing the complexity, the structural response and the load-carrying capacity can be more accurately estimated. This methodology, see Figure 2, is an extension of the strategy developed for bridges in SB-LRA (2007) [8], MAINLINE 2014 [1], UIC 2009 [10], ISO 2015 [11], fib Model Code 2010 (2013) [12] and a procedure specialised for deck slabs by Plos et al. (2016) [9]. Various degrees of degradation and/or fatigue must also be taken into consideration, see e.g. Elfgren (2015) [14].

In order to enable a better representation of the structural behaviour in the intermediate Phase 2 of the assessment, three sublevels have been defined by Bagge (2017) [6]:

2A - Linear elastic analysis
2B - Linear elastic analysis with limited redistribution
2C - Plastic analysis

These methods utilize well-established procedures. For instance, methods for linear elastic analysis with limited redistribution are provided by design codes as e.g. the fib Model Code 2010 [12] and for plastic analysis by K.W.Johansen and M.P. Nielsen [13].

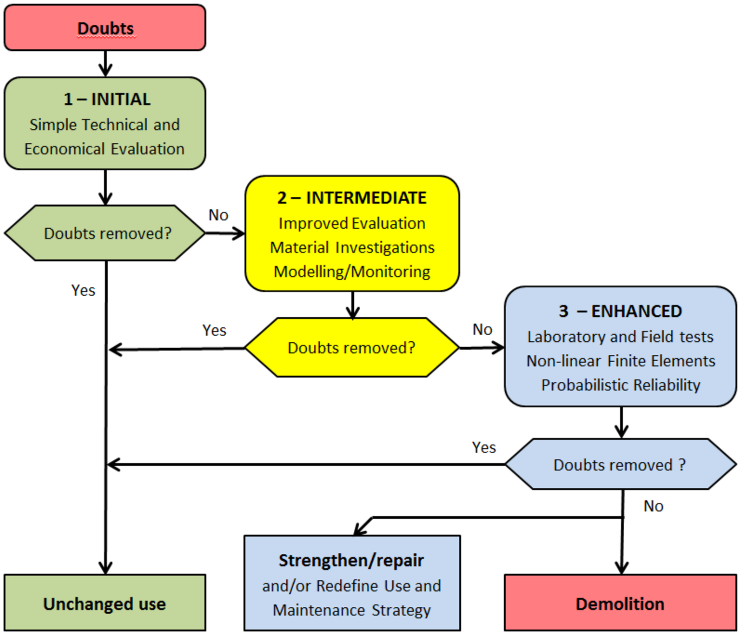

Figure 2. Assessment procedure in three phases: (1) Initial, (2) Intermediate and (3) Enhanced. Based on Sustainable Bridges 2007 [2], [8], Mainline 2014 [1], UIC 2009 [10], ISO 2015 [11], and Paulsson et al (2016) [4].

For structural assessment at enhanced levels, Phase 3, the FE method is a powerful tool. However, depending on the idealisations of the structure, different types of failures can be captured.. Since different modelling choices can necessitate varying computational demands, it is useful to define three levels of approximation for the enhanced structural analysis in Phase 3:

3A - Nonlinear finite elements analysis NLFEA able to calculate the capacity related to flexure.
3B - As 3A but also able to calculate shear-related failures
3C - As 3B but also able to calculate anchorage related failures.

In Figure 1 (right) four load-deflection diagrams are shown for the tested span 2 in the bridge. A Phase 3A initial Non Linear Finite Element Analysis (NLFEA) is shown with a thin dashed blue line. The model was updated with improved material and boundary conditions (partially built in columns) in Phase 3B, shown with black circles. Here a shear failure was captured which did not show up in Phase 3A. Smaller elements were also tested shown with black triangles and a green line. Finally the obtained test result is shown with a thick dashed red line. The Phase 3B with elements of ordinary size (black circles) showed the best correspondence to the test results.

4 SUMMARY AND CONCLUSIONS

A stepwise procedure is recommended for assessment with non-linear finite element calculations. Further research on assessment methods and life-cycle analysis of bridges should be carried out in order to get a more sustainable management.

ACKNOWLEDGEMENT
The authors gratefully acknowledge financial support from Trafikverket/BBT, LKAB/HLRC, SBUF and LTU. They also thank colleagues in the Swedish Universities of the Built Environment (Chalmers University of Technology in Göteborg, the Royal Institute of Technology (KTH) in Stockholm and Lund Institute of Technology (LTH) in Lund) for fruitful cooperation. The experimental work and a previous monitoring campaign were carried out in cooperation with staff of Complab at Luleå University of Technology.

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Assessment of concrete bridges - Prestress forces
Experiences from full-scale testing to failure of a bridge in Kiruna


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ABSTRACT
To calibrate methods for condition assessment of prestressed concrete (PC) bridges, tests were carried out on a 55 year old five-span bridge with a length of 121 m in Kiruna in northern Sweden. Both non-destructive and destructive full-scale tests were performed. This paper presents results regarding the residual forces in the prestressed reinforcement.

Key words: Assessment, Cracking, Corrosion, Creep, Modelling, Pre-stress, Reinforcement, Structural Design, Sustainability, Testing.

1. INTRODUCTION

1.1 General
Assessment, repair and strengthening of existing bridges are required in order to meet current and future demands on sustainability of existing infrastructure. For instance, a survey carried out within the project MAINLINE (2015) [1], indicated a need in Europe for strengthening of 1500 bridges, replacement of 4500 bridges and replacement of 3000 bridge decks. It is believed that for some of these structures replacement can be avoided if more accurate assessment methods are used, see e.g. Sustainable Bridges (2007) [2], Nilimaa et al. (2016) [3] and Paulsson et al.(2016) [4].

Figure 1. The Kiruna Bridge with road E10 to the right and Iron Ore Railway line to the left (2010-03-23).

1.2 The Kiruna Bridge

The test object, located in Kiruna, Sweden, was a viaduct across the European road E10, see Figure 1. It was constructed in 1959 as a part of the road connecting the city centre and the LKAB mining area. Due to underground mining, the entire area underwent excessive settlement. The owner of the bridge, LKAB, closed it in October 2013 and planned to tear it down in September 2014. Before demolition, the bridge was tested in a research project which will be discussed below [5]-[11].

2 PRESTRESSING FORCES

There are some 850 prestressed concrete bridges in Sweden and they represent about 5% of the total amount of bridges maintained by Trafikverket [12]. When assessing prestressed concrete bridges, it is essential to take the current condition of the prestressing system into account. For instance, the quality of reinforcement protection (e.g. grout), steel corrosion and residual prestress force are all aspects that are crucial and require special attention, SB-LRA (2007) [13]. The residual prestress force influences the structural response both at the service-load and ultimate-load levels. By preventing cracks or limiting their formation, prestressing also reduces environmental exposure and, consequently, has a favourable impact on structures in harsh environments. However, there are often many uncertainties associated with the residual prestress force, especially after a longer time in service and, therefore, it can be useful to calibrate theoretically-based methods using experimental data from the assessed structure.

Due to its expected use on full-scale bridge members reinforced with post-tensioned tendons, the saw-cut method was further investigated in Bagge (2017) [6]. The principle of the method is to measure the development of longitudinal strain at the surface (top or bottom) of a member when a block of concrete is isolated from the loads acting on it. The isolation is carried out gradually by introducing transverse saw-cuts on each side of the position of measured strains and the concrete block is regarded as isolated when increasing the depth of saw-cuts does not cause further changes in the strains at the measured surface. The saw-cutting can be simulated in a FE model by gradually removing FE elements corresponding to the saw-cuts in the experiments (see Figure 2). Therefore, using this method, it is possible to avoid any damage to the structure which might be difficult to repair.

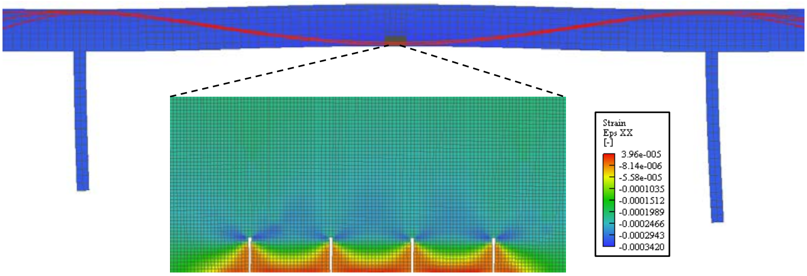

Figure 2. A part of a FE model for simulation of the strain distribution as saw-cuts are introduced transversally at the base of a concrete member (Bagge 2017) [6].

Figure 3 shows the results from the measurements on the south girder in Sections A close to midspan 1, indicating consistent measurements and also an incomplete isolation of the concrete blocks from the acting stresses, which would be characterised by a plateau in the response. In the tests the remaining prestress force varied between some 10 to 85% of the original prestress force of some 85% of the yield stress (a yield stress f_y = 1600 MPa for the 32 wires of 6 mm in a cable in the BBRV System gives an intial prestress force of some 1300 kN/cable).

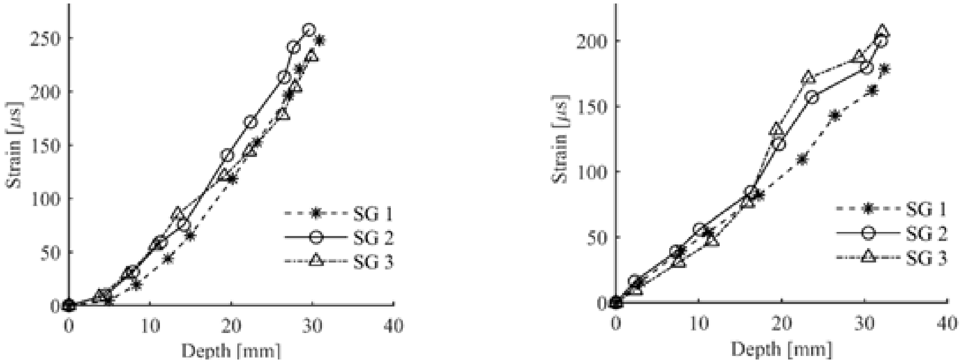

Figure 3. Measured development of strains as function of saw-cut depth for non-destructive determination of the residual prestress force in the south girder, Left: Section A close to midspan 1. Right: Section D at midspan 3. Bagge (2017) [6].

4 SUMMARY AND CONCLUSIONS

Tests have been carried out to check the remaining prestress level in 55 year old bridge. The level was found to vary between some 10 and 85 % of the original force of about 1300 kN/cable. Further research on assessment methods and life-cycle analysis of bridges should be carried out in order to get a more sustainable management.

ACKNOWLEDGEMENT

The authors gratefully acknowledge financial support from Trafikverket/BBT, LKAB/HLRC, SBUF and LTU. They also thank colleagues in the Swedish Universities of the Built Environment (Chalmers University of Technology in Göteborg, the Royal Institute of Technology (KTH) in Stockholm and Lund Institute of Technology (LTH) in Lund) for fruitful cooperation. The experimental work and a previous monitoring campaign was carried out in cooperation with staff of Complab at Luleå University of Technology.

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Multi-level assessment of a field tested RC bridge deck slab

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ABSTRACT
This study proposes a Multi-level Assessment Strategy for reinforced concrete bridge deck slabs. The proposed methods were used for the analysis of previously a tested 55-year old existing bridge deck slab subjected to a shear type of failure, loaded with concentrated loads. The case studies show that the proposed assessment strategy and the analysis methods are feasible and yield reasonable estimates of the load-carrying capacity and structural behaviour such as arching action and load distribution.

Key words: Bridge deck slabs, Multi-level Assessment, FE analysis, Shear distribution

1. INTRODUCTION

In order to provide a systematic approach for the assessment of RC slabs, Plos et al. [1] has developed a “Multi-level Assessment Strategy” which provides recommendations for the assessment of RC slabs using analytical and finite element (FE) models; see Figure 1. The strategy is based on the principle of successively improved evaluation in structural assessment. Accordingly, the assessment of the load-carrying capacity with associated structural response can be conducted through the following levels and methods: (I) Simplified analysis (II) 3D linear (FE) analysis (III) 3D non-linear shell (FE) analysis (IV) 3D non-linear FE analysis with continuum elements and fully bonded reinforcement (V) 3D non-linear FE analysis with continuum elements including the slip between reinforcement and concrete. The aim of this study was to examine the Multi-level Assessment Strategy [1] and modelling methods developed by Shu el al. [2][3] and to investigate the response of a real structure in engineering practice.

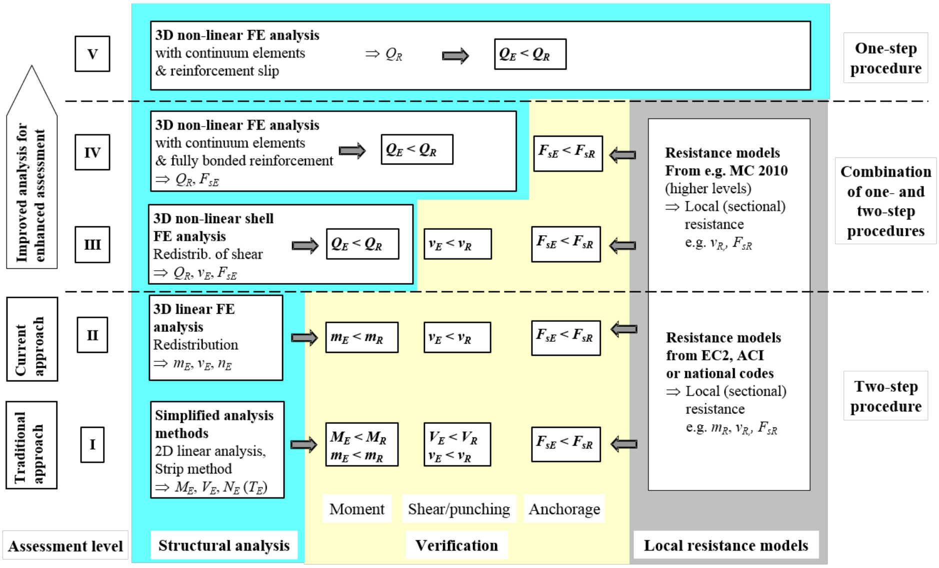


Figure 1: Multi-level Assessment Strategy of RC slabs; from Plos et al. [1].

2. FE ANALYSES OF FIELD TESTED BRIDGE DECK SLAB

To examine the Multi-level Assessment Strategy, this study was conducted by applying a the strategy to a 55-year old RC bridge deck slab subjected to concentrated loads near the main girder in a field failure test. More information about the field test can be found in Bagge et al [4]; see Figure 2. The shear and punching capacity $Q_{u.cal}$ of the deck slab calculated is compared to the failure load $Q_{u.exp}$ from the experiment in Figure 3. At levels I, II and III, one-way shear capacity and punching shear capacity were calculated according to different resistance models based on EC2. At level IV, the load-carrying capacity was obtained from the continuum non-linear FE analysis directly.

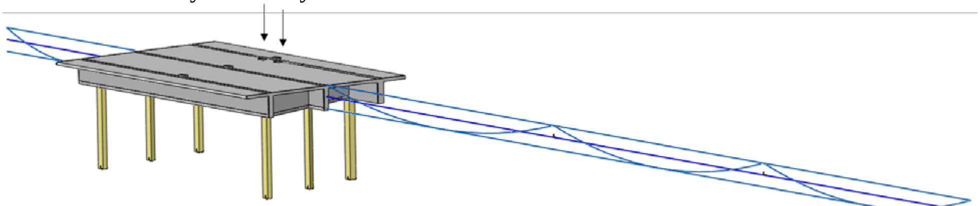
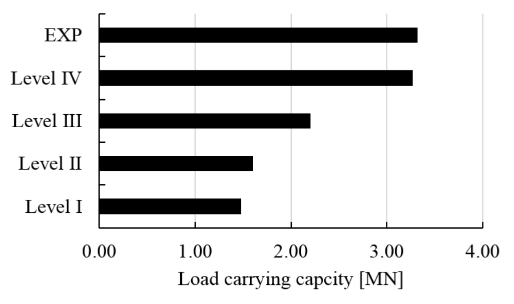


Figure 2: Level IV analysis: non-linear FE model of the tested bridge, showing supports.

3. RESULTS AND CONCLUSIONS

As observed in Figure 3, the shear resistance calculated based on EC2 at level I largely underestimated the real capacity. This indicates that the level I model does not fully represent the behaviour of the tested bridge deck slab. For instance, the influence of prestressing and boundary conditions was not fully taken into account, but are essential for the actual structure. By upgrading the level of approximation, the accuracy of calculated capacity increases. Level II gives similar results as level I, indicating that improved representation of the geometry when determining the load effect is not sufficient. Level III analysis provides a notably higher, still considerably underestimated, load carrying capacity just by representing the non-linear bending response more correctly. Finally, the continuum non-linear FE analysis at level IV provides a load-carrying capacity which is close to that obtained in the experiment.



Capacity	$Q_{u,cal}/Q_{u,exp}$	Control failure mode
3.32 MN		One-way shear & Punching
3.27 MN	0.98	One-way shear & Punching
2.20 MN	0.66	One-way shear
1.60 MN	0.48	One-way shear
1.48 MN	0.45	One-way shear

Figure 3: Load-carrying capacity calculated based on Multi-level Assessment Strategy and comparison to experiment.

In the bridge test, the distance from the edge of the loads to the edge of the girder were only 1.09*d* and 0.6*d* for load plate 1 and load plate 2, respectively. To study the influence of arching action, the loads were gradually moved further away (100 mm per step) from the girder in the level IV model (see Figure 4 (*a*)). The load on position 1 was the same as in the field test. The nominal shear strength (excluding influence of b_w, d & f_{cm}) were calculated assuming a pure one-way shear failure and then the values were compared to laboratory test results obtained by Natario et al. [5] and Lantsoght et al. [6]; see Figure 4 (*b*). From the analysis results, it was observed that the shear capacity decreased when loads were moved further away from the support. When the loading plates were placed in position 4, the failure mode even changed from shear to bending failure.

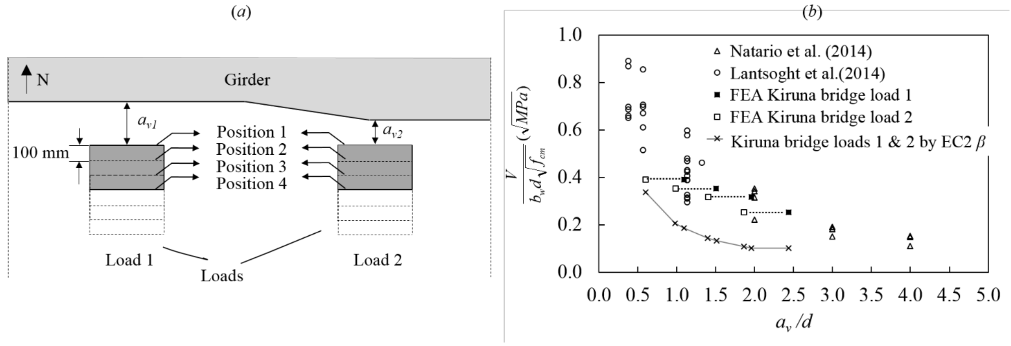


Figure 4: (a) Variation of load positions in the FE analyses; (b) nominal shear capacity of the slab subjected to loads at different positons, with comparison to literature [5][6].

The shear force distribution obtained from the FE analysis at level IV was investigated. In Figure 5, the shear force per unit length along a line in the longitudinal direction of the bridge close to the girder is presented for different load levels (Q/Q_u = 0.2, 0.4, 0.6, 0.8 and 0.95). As expected, force applied to the area of the loading plates is distributed over a larger width closer to the girder. A clear shear force redistribution was observed for the shear force near loading plate 1; the shear force close to loading plate increased fast as the applied load increased at low load levels ($Q/Q_u \leq 0.8$), but stopped to increase at higher load levels ($Q/Q_u > 0.8$). Instead, the shear force in the adjacent region increased faster. However, close to loading plate 2, the phenomenon of shear force redistribution was not as clear. Possible explanations for this are that

(1) there is not enough space for shear force redistribution since loading plate 2 is much closer to the girder and (2) the change in distance to the support due to the changing girder width clearly influenced the shear flow.

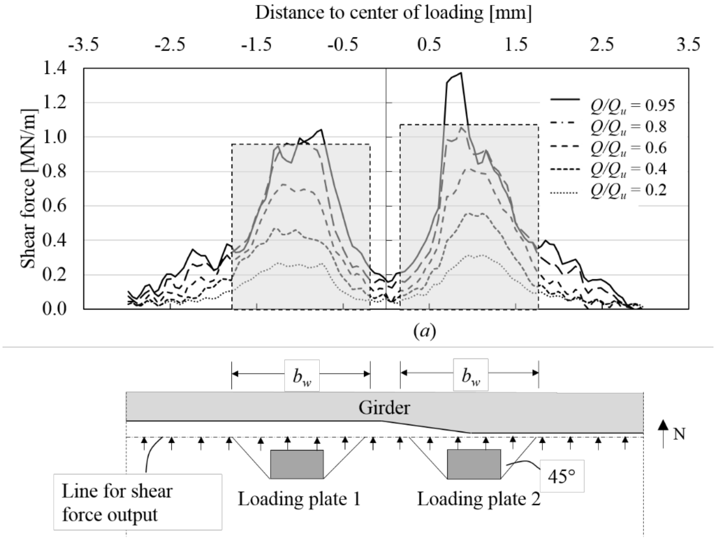


Figure 5: Shear force per unit length across a line parallel to the girder, from FE analysis

4. CONCLUSIONS

It can be learned that existing models in building codes for shear and punching can be underestimated. The analysis method based on the “Multi-level Assessment Strategy” is a straight forward approach to evaluate the load-carrying capacity of existing RC bridge deck slabs. By upgrading the level of assessment, the accuracy of the calculated capacity increases and the continuum non-linear FE analysis at level IV provided a shear capacity very close to the experiment. The shear force distribution is influenced by applied load levels and the failure mode is affected by factors such as boundary conditions and the locations of concentrated load.

ACKNOWLEDGMENT

The presented research was funded by the Swedish Transport Administration (Trafikverket).

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Session B4:

WORKSHOP ON TEACHING

Project families: A new concept for student thesis activities

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ABSTRACT
The students' activities during their final thesis work have been organised in project families, i.e. a group of individual student project organized in a shared learning environment. The aim is more efficient supervision and support, simultaneously to improved learning. DTU Byg have now tested this concept for 100+ students with experimental activities and found a major improvement in their learning, grades, interaction and behaviour in the laboratories, just as they now provides a strong support for the supervisors' research. The use of resources for the supervision and the support in the laboratories has also been significantly reduced.

Key words: Knowledge and Teaching, Testing.

1. INTRODUCTION

The Technical University of Denmark offers many different engineering educations and the Department of Civil Engineering (DTU Byg) is much involved in teaching courses and supervising student's thesis works. The number of students, doing thesis work at DTU Byg has grown over the years to app. 350 annually and usually with experimental activities included (app. 100 students used the concrete and mortar lab during their thesis work in 2016).

It is the university's policy to strengthen the experimental activities in the educations. This has resulted in four new laboratory buildings for DTU Byg being completed, construction being started or bid being asked for these new buildings in 2016 [1]. The experimental activities will thus not be reduced but increased in the near future according to DTU' strategy, "*DTU will assure and expand the students' access to experimental facilities and activities designed to train engineers*" [2], just as the number of project students are expected to increase.

The growing number of students increases the pressure on our resources, but also challenges our ability to increase the number of new project ideas and cooperation with industry. We are, at the same time, under pressure to maintain or even increase our scientific production and this creates a situation, where we are highly motivated to rethink our organisation of students projects with experimental activities, so the resources are utilized optimally within both teaching and research.

2. THE PROJECT FAMILY CONCEPT

The project family concept is to group individual projects, so they have a common focus. The individual projects can be a mixture of BEng, BSc and MSc projects, where a project has one or two participants (form the same education). The family have a supervisor team with a leading supervisor and the required number of cosupervisors. The supervisor team decides the semesters focus for the project family and ensures that the required info, materials, finances and test facilities will be available.

The students' supervision and instructions in the laboratory will be given to all students in the project family at the same time, but is supplemented with the required individual supervision. These may even include E-learning activities such as the general safety instructions and its electronic exam (used by over 700 in the first year), video recording of instructions (by students, by supervisors or by professionals), so the students can go over the instructions again (if required).

The students will often share the same lab facilities, test setup and even use each other's results (with proper references) and will have a joint start-up meeting and also a midterm presentation of their initial results. This presentation will often be attended by one or several industrial partners.

The students will when possible be given a common room with work places during their project period in order to promote cooperation and peer interaction and to stress the “family” aspect. DTU Byg has over the last years established a number of such project rooms, where all members of one or more project family can work and where lockers are available for students (for storage of PC, personal stuff etc.).

3. RESULTS FROM THE PROJECT FAMILIES

DTU Byg has tested the concept for a period of six years and have started over 100 project students in experimental families in the fields of ZeroWaste Byg (upgrading of waste to a valuable resource), Strengthening with CFRP (Carbon Fibre Reinforced Polymers) to increased loadcarrying capacities and Glass Structures (glass as a load-carrying material). This is only 10 % of the project students at DTU Byg during the period, but we can already identify some effects based on the results with these students.

The grades document (see Figure 1) shows that the students in the project families perform better, as their average grade for the thesis (DK:10,9) is half a grade higher than all the remaining thesis students at DTU Byg in the same period (termed the reference group with a thesis average of DK:10,0), although the students in the project families have the same average grades in their education as the others. The supervisor’s experience is also that far fewer thesis lack the vital information required for later use of the results, which have often been a problem (unless the supervisor coached the students). This change has turned the project families into a substantial help for the supervisors research, as a project family may put in e.g. 5000 hours of work during a semester, focused on the specified problem or development.

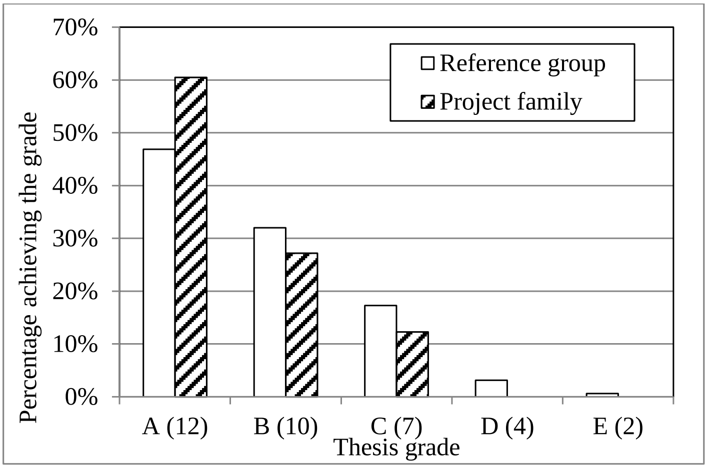

Figure 1 – Grades in student thesis at DTU Byg according to the international ECTS Scale [3] and Danish grade in brackets, (A(12): For an excellent performance; B(10): For a very good performance; C(7): For a good performance; D(4): For a fair performance; E(2): For an adequate performance; Fx and F: Failed).

The students appreciate the project family concept, as the period where the students do their thesis work can be a very stressful and lonely period for students, especially if they work individually. This problem seems to be eliminated with the project family concept, as they will always have a number of peers to discuss with. The concept has also established some amount of peer pressure, which is good for the somewhat relaxed students, but which have also improved their behaviour in the laboratories significantly. The supervisors have experimented with the size of the project family and have found that the optimal size will normally be 7 to 10 students, for the concept to work optimally.

Figure 2 – Students in a project family at their midterm presentation.

The supervisors and the laboratory staff have also noticed that they spend less time on boring, repetitive activities, but also that the project families requires a better planning prior to the projects initiation. If the planning is done properly, the discussions between students and supervisors reach a higher scientific level, than possible with stand-alone projects. The students' behaviour in the laboratories and the quality of their thesis, their presentations and their ability to answer at the examinations has all improved.

4. CONCLUSION AND FUTURE WORK

The concept of project families have been a good and strong success for the thesis works with experimental activities, as it has improved the student performance, the students experience and responsibility during the project, while at the same time reducing the resources needed for supervising and supporting the projects.

The use of project families without experimental work will be tested in the near future and some initial activities have already been initiated.

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Green concrete workshops for students

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ABSTRACT
Sharing of knowledge is an important part of the project “Green transition of cement and concrete production”. To ensure that the future users of green concrete gains access to this knowledge, approximately 200 students have participated in workshops at the Danish Technological Institute. The students have had the opportunity to get "hands on" and gain experience about the challenges and opportunities involved in the designing of green concrete that is also workable in practice.

Key words: Cement, Mix Design, Rheology, SCC, Surfaces, Teaching, Testing.

1. INTRODUCTION

The Innovation Consortium "Green Transition of Cement and Concrete Production” is a project supported by the Danish Innovation Fund - colloquially the project is called "Green Concrete II". The project aims to create the foundation for a green transition of cement and concrete production in Denmark. The cement production contributes to 5% of the total man-made CO_2 emissions worldwide and it is estimated that the demand for cement and concrete, will be twice as large in 2050 as in 2010 [1]. Therefore, focus is now on how to reduce CO_2 emissions from cement and concrete production. In this project, the focus is on the use of supplementary cementitious materials as partial replacement for cement clinker and how this affects rheology, mechanical performance, and durability. Thus, the project aims to provide basis for the use of alternative binder compositions in order to ensure that future concrete structures can still be realized with the necessary durability and minimal need for maintenance.

An important part of the dissemination and education strategy in the project has been to develop workshops about green concrete technology. The aim of the workshops is to give future concrete practitioners an understanding and insight into the challenges and opportunities of concrete mix design, with a special focus on the environmental aspects and how this affects workability and stability of fresh concrete. The use of alternative binders often results in reduced water to binder ratios leading to concrete with increased viscosity and stickiness, which may be seen as a hindrace to a more wider spread industrial implementation of green concrete technology [2].The workshops combine "hands on" lab exercises and theory. The target group has been students within building and construction at bachelor level. The workshops have been developed together with Copenhagen School of Design and Architecture (KEA), Lillebaelt Academy, Zealand Institute of Business and Technology, VIA University College, Technical University of Denmark, Technical University of Denmark and the Danish Concrete Association. Currently, approximately 200 students from these civil engineering and constructing architect programs have participated in the workshops at the Concrete Centre at the Danish Technological Institute.

3. WORK SHOPS

3.1 Green concrete mix design

Before entering the workshop, the students are given an introduction to basic concrete technology including concrete properties, testing procedure, and mix design optimization procedures. As part of the preparations, the students are divided into groups and the task for each group is to design a concrete with the lowest possible CO_2 emission, which fulfils certain requirements to workability (either slump or slump flow for SCC), For each group, a certain number of mix design parameters are fixed. For instance, one group might be given fixed values of water to cement ratio and aggregate type and relative proportions. The students are then allowed to adjust the paste content and binder composition e.g. reducing cement content by fly ash addition. For each workshop, the restrictions and freedoms of choice are selected in such way that the students will be able to compare the findings between groups. For instance, one group might work with crushed aggregate and another group with rounded aggregates as the only difference.

At the workshop the students mixed concrete after their own homemade recipes, often figuring out that the concrete was completely unusable in practice as much of the focus during the desktop mix design was to reduce the CO_2 emission resulting in concretes with extreme low amounts of paste. To many, this was an eye-opener into the practice of concrete mix design. The next step was then to optimize the mix design during the rest of the day based on additional information and theory about composition of aggregate, paste volume and paste composition, and the effect of chemical additives. On the basis of their new knowledge the students optimized their recipes and tested them continuously in the laboratory to learn how the adjustments affect their concrete in practice. After 3-4 iterations, most groups ended up with concretes that fulfilled the required workability performance. During the day, all results including pictures of the concrete were reported on a wall in the lab. At the end of the workshop, each group presented their results and experiences during the day. This gave rise to many good discussions and the groups started to get a feel for the science of mix design and the challenges and opportunities involved in finding a good balance between technical performance and environmental impact.

Figure 1 – Students at Green Concrete workshops at the Danish Technological Institute


Figure 2 – Test of the student's green concrete at the work shop

3.2 Design of Mock up

For some of the workshops, an additional exercise was included. Each group were given the task to explore the potential of playing with the aesthetics of the concrete surface in terms of roughness, reliefs, and geometry. The surface should illustrate the groups vision for a concrete wall designed for an industrial and commercial building. Some groups explored the opportunities lying within digital fabrication of formwork applying the industrial robot facilities at the Concrete Centre at the Danish Technological Institute. Other groups decided for a more handmade formwork solution. Some of the mock ups were exhibited at the "Sustainable Concrete Conference 2015" in Tivoli Hotel and Congress Center in Copenhagen.


Figure 3 – Students are preparing mock ups before casting

Figure 4 – Some of the mock ups after demoulding

3.3 Service life of green concrete

In addition to the work described in the above, it was possible to include further learnings about the mechanical properties and durability. For instance, in one workshop, the civil engineering students cast cylinders during the workshop with their own concretes in order to carry out supplementary testing at their university. They tested and compared the strength development and the chloride migration, to see how the cement/fly ash – content affect the concrete properties.

3. CONCLUSION

During the project “Green transition of cement and concrete production”, workshops on green concrete technology have been developed. A total of six workshops have been held so far and the learnings from each workshop has been carried on to the next in order to optimize the learning process and work procedures during the workshop. According to the wishes of educational institutions, the workshop can be designed as a one or two-day workshop. The students have experienced that a unilateral focus on a single parameter, in this case CO_2, is not possible in practice, since the concrete must also fulfill other requirements as workability, strength and durability. Both teachers and students from the educational institutions have been very satisfied with the outcomes and learnings from the workshops. Danish Technological Institute is very open to continue the workshops are the end of the project for those education institutions interested in this.

ACKNOWLEDGEMENTS

Innovationsfonden for funding of the project.

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