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CRACK CONTROL OF CONCRETE STRUCTURES IN SPECIAL CASES

MASTER OF SCIENCE THESIS

Examiner: Professor Ralf Lindberg
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ABSTRACT

In this study, cracking and crack control of concrete structures is explored. Main focus of this study is put on controlling cracking behavior in cases that can be considered as special ones. That comprises massive concrete structures with heavy reinforcement and thick concrete covers that are present e.g. in nuclear power structures.

It was noted in the thesis, that in special cases cracking can be controlled with proper curing, casting arrangements, limiting of compressive stress of concrete and tensile stress of steel, right selection and arrangement of reinforcement, surface reinforcement, transversal reinforcement in the anchorage zone and in case of massive concrete structures minimizing the temperature difference in the structure.

The primary goal of this study is to explore and clarify, whether it is possible to control cracking in special cases by calculating crack widths according to EC2, when concrete cover is thick and surface reinforcement cannot be used. In addition the reliability of calculated crack widths was explored in such situations. This was conducted by calculating crack widths for a sample structure, and comparing the results with the experimental test results found in literature. For comparison crack widths were calculated not only according to EC2 but also three different available methods were used.

The crack width calculations proved that with thick concrete covers and heavy reinforcement calculated crack widths increased significantly. Comparison of calculated results with experimental tests showed relatively good relation between predicted and real crack widths in normal case, but, in special case calculated crack widths seemed to be greater than experimental widths. On the basis of this study crack width calculation according to EC2 will not be suitable for cases with thick concrete covers and heavy reinforcement without distinct lowering of steel stress, even though it was proven that real crack widths will likely remain lower than predicted ones.

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TIIVISTELMÄ

Työssä tutkittiin betonirakenteiden halkeilua yleisesti ja halkeilun hallintaa. Pääpaino työssä oli halkeilun hallinta tilanteissa joita voidaan pitää tyypiltään haasteellisina. Niihin kuuluvat paksut betonirakenteet joiden raudoitus on massiivinen ja suojapeitepaksuudet suuret, joita löytyy muun muassa ydinvoimarakenteista.

Työssä todettiin, että erikoistapauksissa halkeilua voidaan hallita muun muassa hyvällä jälkihoidolla, valujärjestelyillä, rajoittamalla betonin puristusjännitystä ja teräksen vetojännitystä, raudoituksen oikealla valinnalla ja järjestelyllä, pintaraudoituksella, vedetyn alueen poikittaisraudoituksella sekä massiivisten betonirakenteiden tapauksessa rakenteen lämpötilaeron minimoinnilla.

Työn keskeisin tavoite oli tutkia ja selvittää, onko erikoistapauksissa mahdollista hallita halkeilua Eurokoodi 2:n mukaisilla halkeamaleveyslaskelmilla, kun betonipeite on suuri ja pintaraudoitusta ei mahdollisesti voida käyttää. Lisäksi tutkittiin laskettujen halkeamaleveyksien luotettavuutta. Tämä suoritettiin laskemalla halkeamaleveydet esimerkkirakenteelle ja vertaamalla tuloksia kirjallisuudesta löytyneisiin kokeellisiin tuloksiin. Vertailun vuoksi laskelmissa käytettiin EC2:n lisäksi kolmea muuta käytettävissä olevaa laskentamenetelmää.

Laskelmat osoittivat, että laskentamenetelmästä riippumatta suurilla betonipeitepaksuuksilla ja voimakkaasti raudoitetulla poikkileikkauksella laskennalliset halkeamaleveydet kasvoivat huomattavan suuriksi. Laskelmien vertailut koetuloksiin osoittivat, että kaikilla laskentamenetelmillä lasketut halkeamaleveydet olivat normaaleissa tilanteissa lähellä todellisia halkeamaleveyksiä, mutta erikoistapauksissa laskennalliset halkeamaleveydet näyttivät olevan suuremmat kuin todelliset halkeamaleveydet. Eurokoodin mukainen halkeamaleveyden laskenta ei siis tämän tutkimuksen mukaan sovellu hyvin käytettäväksi tilanteisiin, joissa on suuret betonipeitepaksuudet ja massiivinen raudoitus ilman selvää teräsjännityksen rajoittamista, vaikkakin työ osoitti, että todelliset halkeamaleveydet jäävät erikoistapauksissa laskennallisia leveyksiä pienemmiksi.

PREFACE

This study has been carried out in co-operation with Pöyry Finland, Forest and nuclear industry business group.

First of all, I want to express my gratitude to Pöyry for this opportunity to study cracking and crack control of concrete structures. I want to thank also my supervisors, Matti Korhonen and Kevin de Bleser, from whom the advice and feedback I have received during this project. I want to express my gratitude also to Professor Ralf Lindberg for feedback and examining of this study.

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TABLE OF CONTENTS

ABSTRACT II
TIIVISTELMÄ III

1 Introduction 8

2 Cracking of concrete structures 9
2.1 Composition and features of concrete 9
2.2 Formation of the cracks 9
2.2.1 Cracking under axial loading 10
2.2.2 Cracking under bending moment 12
2.3 Types of cracks 13
2.3.1 Loading crack 13
2.3.2 Thermal crack 15
2.3.3 Drying shrinkage crack 15
2.3.4 Plastic shrinkage crack 16
2.3.5 Plastic settlement crack 16
2.3.6 Corrosion crack 17
2.3.7 Alkali-aggregate crack 17
2.3.8 Crazing crack 17
2.3.9 Sulfate attack crack 18
2.3.10 Weathering crack 18
2.4 Autogenous healing of cracks 18
2.5 Reasons for controlling cracking and crack width 19
2.5.1 Durability 19
2.5.2 Appearance 21
2.5.3 Leakage 21
2.6 Methods for crack control 21
2.6.1 Curing 21
2.6.2 Casting arrangements 22
2.6.3 Allowable compression stress of concrete 22
2.6.4 Limiting of crack width 22
2.6.5 Minimum reinforcement for crack control 23
2.6.6 Crack control without direct calculation 25

2.6.7 Surface reinforcement ... 26
2.6.8 Transversal reinforcement in the anchorage zone ... 27
2.7 Crack control of massive concrete ... 28
2.7.1 Mass concrete ... 28
2.7.2 Reasons for mass concrete crack control ... 29
2.7.3 Ways to mass concrete crack control ... 30
2.8 Special cases of structures in terms of crack control ... 32
2.8.1 Wall of a reactor containment structure in nuclear power plant ... 32
2.8.2 Diaphragm wall ... 33

3 Calculation of crack width ... 35
3.1 EC2 ... 35
3.2 RakMK B4 ... 36
3.3 DIN 1045-1 ... 37
3.4 ACI 318 ... 37
3.5 Sample calculation of crack width in a simple case ... 38
3.5.1 Calculation according to EC2 ... 39
3.5.2 Calculation according to RakMK ... 41
3.5.3 Calculation according to DIN 1045 ... 43
3.5.4 Calculation according to ACI 318 ... 44
3.6 Summary of the simple case results ... 45
3.7 Sample calculation of crack width in a special case ... 47
3.7.1 Calculation according to EC2 ... 48
3.7.2 Calculation according to RakMK ... 51
3.7.3 Calculation according to DIN 1045 ... 51
3.7.4 Calculation according to ACI 318 ... 52
3.8 Summary of the special case results ... 53
3.9 Effect of surface mesh on the calculated crack width ... 54

4 Experimental studies of crack width ... 57
4.1 Experimental study of cracking in normal case ... 57
4.1.1 Presentation of the study ... 57
4.1.2 Design codes for test calculations ... 57
4.1.3 Experimental arrangements ... 59
4.1.4 Test result ... 60
4.2 Experimental study of cracking in special case ... 62
4.2.1 Presentation of the study ... 63
4.2.2 Design codes for the calculations ... 63
4.2.3 Experimental arrangements ... 63
4.2.4 Measuring of crack width ... 64

4.2.5 Test results ... 65

4.2.6 Summary of the experimental test ... 66

4.3 Comparison between experimental results with calculated values in special case ... 66

5 Practical difficulties in the interpretation of crack calculation according to EC2 ... 68

5.1 Tension stiffness ... 68

5.1.1 More accurate tension stiffness model ... 68

5.1.2 Local tension stiffness model according to EC2 and flexure ... 69

5.2 Cracking strain and crack width position ... 71

5.2.1 Cracking in the cover zone vs. cracking at the bar surface ... 72

5.2.2 Large concrete cover thickness ... 74

5.3 Multiple rows of steel ... 75

6 Conclusions ... 77

7 References ... 79

1 Introduction

Cracking of concrete structures is a typical phenomenon that will occur in such sections of a concrete member where tensile stresses reach the tensile strength of concrete. Tensile stresses may be induced e.g. by external loading or imposed deformations. Cracking will impair both the durability and also the appearance of a concrete member. Therefore, it is a relevant part of designing concrete structures to control cracking in order to ensure design working life.

This study reveals different ways to control cracking not only in conventional structures but also in structures that are more or less uncommon including massive concrete structures as well. Important way of crack control is limiting of crack width. According to Eurocode unacceptable cracking in terms of appearance and action of structures under the characteristic combination of loads may be assumed to be avoided and no further cracking analysis is not required if steel stress is below 60% of the yield strength [13, chapter 7.2]. This is somewhat non-specific definition which will result crack widths around 0,3mm in normal cross-sections, but, occasionally there are cases in which crack widths must be limited to 0,1 or 0,2mm. Therefore, structures shall be designed so that calculated crack widths will not exceed the limit values according to Finnish National Annex of the Eurocode.

Crack width calculation rules according to Eurocode are basically drafted to be applied into design of normal structures. But, e.g. in massive nuclear industry structures there are occasionally cases, in which concrete cover is large and reinforcement is heavy. Practically this will come into question in a wall of a nuclear containment structure, in which this type of structures can be regarded as special ones, and there are obvious doubts whether the calculation rules according to Eurocode are valid in such occasions. Normal is to use surface reinforcement mesh in order to limit crack widths in which case total sum of crack widths is assumed to be independent of the surface mesh. But, if the surface reinforcement for some reason could not be used, calculation formulas would obviously result in wide crack widths due to increased cover thickness. An open question is how reliable the crack width formulas by Eurocode are then.

This Master of Science thesis focuses on study of crack width calculation in special cases, and will explore the reliability of the calculation rules by presenting experimental data of crack width tests as well. For comparison crack width calculation is carried out for sample structures by EC2, ACI 318 and, recessive calculation methods, RakMK B4 and DIN 1045.

2 Cracking of concrete structures

2.1 Composition and features of concrete

Concrete is a composite material, which consists primarily of a mixture of cement, water and aggregates. It differs from other building materials, like steel and timber, distinctly by its composition and behavior under load. The behavior of concrete under compressive load compared to tensile load is quite different due to its heterogeneous composition. Compressive strength is often considered the main feature of concrete. Its tensile strength is considerably lower; approximately tenth of compressive strength. Therefore, reinforcement is usually embedded in the concrete structures to carry tensile stresses. Plain concrete is used in such cases, when the eccentricity of loading is so small that development of cracks does not risk reliability of a structure against breaking or buckling. [1, pp. 115-116, 120]

Deformations of hardened concrete may be divided in three types. Elastic deformations, resulted from loading or changes in temperature, return to zero after loading is no longer applied. Plastic deformations, resulted from massive loading, do not disappear completely after loading is removed. Last type of deformation is depending on time and climate conditions. Shrinkage is independent of loading and is caused by change of moisture of cement paste in drying concrete. Creep depends on loading and is caused by viscose flow of water in cement paste under pressure caused by loading. [2, p. 11]

2.2 Formation of the cracks

Cracking is typical for concrete and cannot be avoided. It occurs in such parts of a concrete member where tension stress reaches tensile strength of the concrete. Tensile stresses may be developed due to, for example, external loading, imposed deformations or chemical reactions. However, cracking is not always detrimental to concrete. Crack spacing and crack width must be kept small enough by a proper design and construction of a structure. Therefore, the statical behavior, stability in use and appearance of the concrete structure is ensured. [4, p. 73]

Cracking starts when the tensile stress in the concrete reaches the tensile strength of the concrete at some point of the structure. When this occurs, all the force is carried by the reinforcement in this part. By experimental studies, crack width at the level of the bar is clearly lower as that at the surface of the concrete when using reinforcing bars. That is resulted by small cracks developed around the ribs as shown in the Figure 2.1. [2, pp. 188-189]

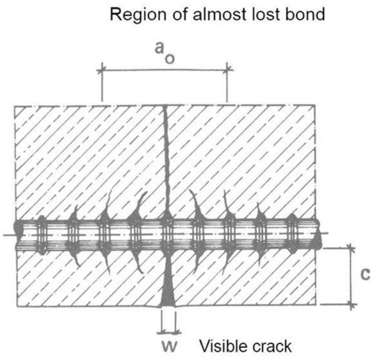


Figure 2.1. Cracking at bar deformations.

2.2.1 Cracking under axial loading

When central tension load is applied to a reinforced concrete structure, first crack develops in the weakest point of the concrete. Tensile stress reaches the tensile strength at this point. When tensile stresses transfer from concrete to reinforcing bars, an impulsive increment occurs in the steel stress as shown in the Figure 2.2. Let us consider crack development under axial tensile loading. When tensile strength of concrete is not yet reached, the steel stress can be calculated as follows:

$$a_{sI} = a_e \sigma_{ct} \quad (2.1)\ [2, p. 189]$$

$$a_e = \frac{E_s}{E_c} \quad (2.2)\ [2, p. 189]$$

$$\sigma_{ct} = \frac{N}{A_c + a_e A_s} \leq f_{ctk} \quad \text{, where} \quad (2.3)\ [2, p. 189]$$

σ_{ct} = tensile stress of concrete
A_c = total cross section minus A_s
A_s = cross-sectional area of steel bar
E_c = Young's modulus for concrete
E_s = Young's modulus for steel

Steel stress instantly after crack development is then:

$$\sigma_{sr} = \frac{N_r}{A_s} \quad (2.4)\ [2, p. 189]$$

Capacity for cracking under axial tension loading can now be calculated from equation:

$$N_r = (A_c + a_e A_s) f_{ctk} \quad (2.5)\ [2, p. 189]$$

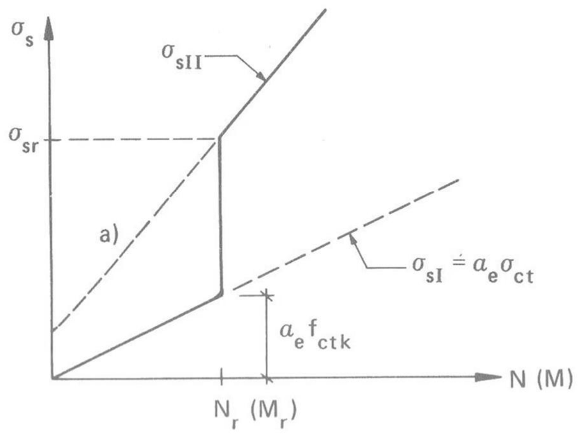


Figure 2.2. Increment in steel stress due to crack development.

The tensile stress of the concrete drops to zero at the place where the first crack occurs. When proceeding forward from the crack, the tensile stress of the concrete increases due to bond between reinforcing bars and concrete until it reaches value by the expression (2.3) at the distance s_0. That stress corresponds to the complete bond (Figure 2.3). The distance s_0, at which the crack has an effect, can be calculated from the following equations:

$$s_0 = \frac{1}{2}a_0 + l_b \qquad (2.6)\ [2, p. 190]$$

$$l_b = \frac{f_{ctk}A_{ct}}{\tau_{bm}\Sigma u_s}\text{, where} \qquad (2.7)\ [2, p. 190]$$

a_0 = length of the zone where is no bond between steel and concrete
l_b = length of the bond
τ_{bm} = average bond stress
A_{ct} = area of the concrete cross section where tension stress acts
u_s = circumference of the bar

The second crack will then develop into the cross section where tensile stresses reach tensile strength at next. The distance from the previous crack to the following one must be at least s_0. Then, the development of cracks can be seen in the Figure 2.3 and the basics of both the stresses in the steel and in the concrete after the development of the second crack are shown in the Figure 2.4. When the development of the cracks has ended, the average crack spacing s_{rm} is supposed to be between the following limits: [2, pp. 189-192]

$$s_0 \leq s_{rm} \leq 2s_0 \qquad (2.8)\ [2, p. 191]$$

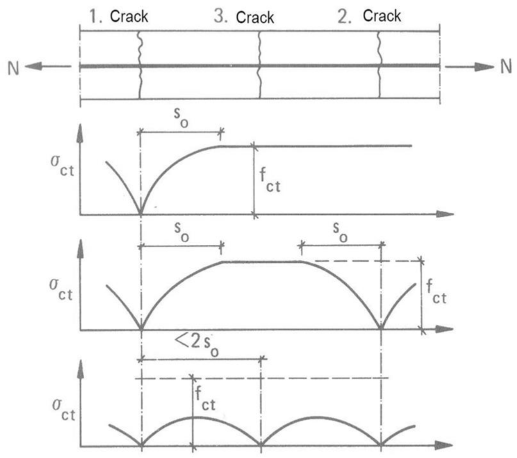


Figure 2.3. Development of cracks under axial loading.

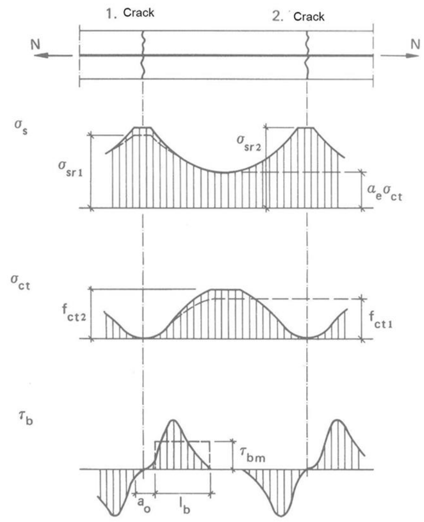


Figure 2.4. Concrete stresses after development of the first cracks.

2.2.2 Cracking under bending moment

Development of the cracks due to bending moment is basically of the type shown in the Figure 2.5. The first crack develops when tensile stress reaches tensile strength of the concrete at the zone of the greatest bending moment. This so called cracking moment (M_r) can be calculated as follows:

$$M_r = W_{cp} f_{ctk} \qquad (2.9)\ [2, p. 191]$$

$$W_{cp} = \frac{7}{24} bh^2 \qquad (2.10)\ [2, p. 191]$$

, where W_{cp} is the plastic flexural resistance for a rectangular cross-section when the effect of the reinforcement is not taken into account. [2, pp. 189-192]

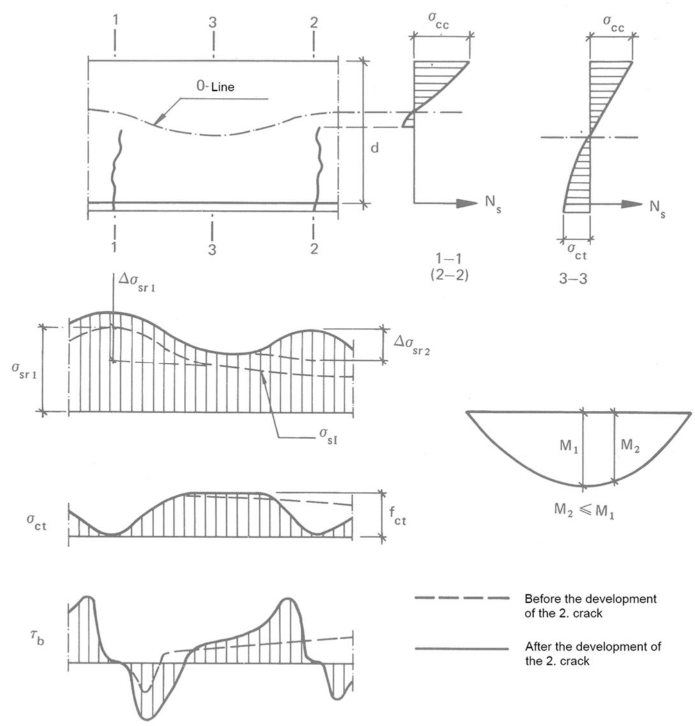


Figure 2.5. Development of cracks under bending moment**.**

2.3 Types of cracks

2.3.1 Loading crack

Types of loading cracks vary depending on the type of loading. Crack patterns caused by tensile stresses induced by external loading are distinctive, as shown in Figure 2.6. In these kinds of cracks caused by loads, the final crack pattern has not generally developed at service load level. There are normally some cracks at points of maximum tensile stress at this load level. If direct tension is applied to a concrete member, cracks are developing through the entire cross section. Spacing of cracks in such cases is approximately 0,75 to 2 times the minimum thickness of the member. If the member, in which direct tension is applied to, is very thick and contains reinforcement in each face, small cracks develop on the surface of the member (Figure 2.6a). These cracks join in the middle of the member. That results the crack width at B which is greater than at A. [3, p.320]

If the member is subjected to bending moments, flexural cracks are developed (Figure 2.6b). These cracks extend vertically almost to the neutral axis of the member. Cracking is relatively closely spaced in the layer of reinforcement in cross sections with high web. Several cracks are then joining or disappearing above the reinforcement, which is shown in the Figure 2.6b. The crack width at B will commonly exceed that at A in this case as well. [3, p.320]

Cracks developed due to shear force have their own characteristic inclined shape (Figure 2.6c). This type of cracks reaches the neutral axis and occasionally compression zone. Torsion cracks are similar to shear cracks. In case where the member is subjected to pure torsion, cracks spiral around the member. When moment and shear also act, cracks tend to be more distinct on the face the direct shear stresses and shear stresses due to torsion add. On the faces stresses counteract, cracks are less distinct or absent. [3, p.320]

Bond stresses around the reinforcement will cause cracks along reinforcement bars as shown in the Figure 2.6e. If load is concentrated, it will occasionally lead to splitting cracks as shown in the Figure. 2.6f. In these kinds of cracks caused by loads, the final crack pattern has not generally developed at service load level. There are normally some cracks at points of maximum tensile stress at this load level. [3, p.320]

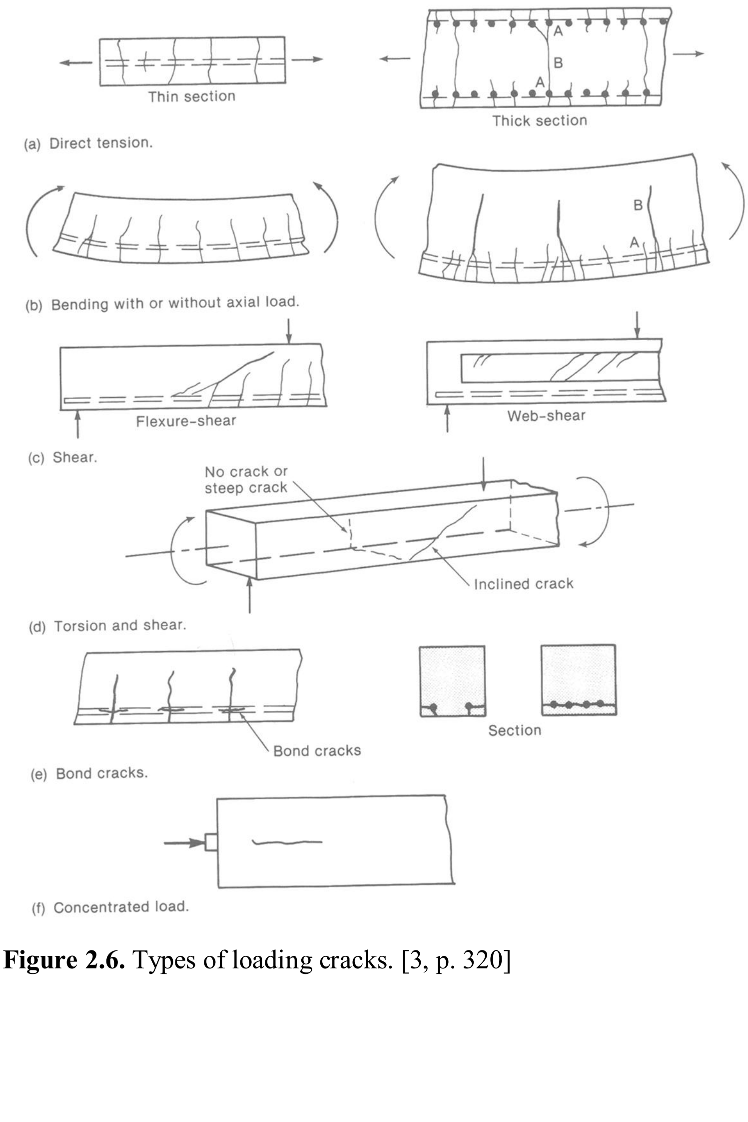


Figure 2.6. Types of loading cracks. [3, p. 320]

2.3.2 Thermal crack

Thermal crack occurs if the temperature differences in concrete or its surroundings grow too excessive. Then, cooler parts of the concrete will contract more than warmer ones resulting tensile stresses. If tensile stresses due to restrained contraction reach the tensile strength of the in-place concrete, thermal crack will appear. [4, p. 1]

The temperature difference is normally resulted from the heat due to hydratation of cement. In hydratation, the generated heat dissipates through the surface of the concrete which induces the difference between the internal and the surface temperature. The surface contraction due to cooling is restrained by the hotter interior concrete that does not contract as quickly as the surface. This restraint creates tensile stresses that can crack the surface concrete. Thermal cracking causes rarely problem in thin section while the heat of hydration dissipates quickly. In mass concrete uncontrolled temperature difference is a real concern. [4, p.1]

Figure 2.7. Thermal crack in a thick slab. [4, p. 1]

2.3.3 Drying shrinkage crack

Concrete has a tendency to contract or shrink as it dries and to expand as it is wettened. The shrinkage, however, is more intensive than the expansion. The change in moisture content of cement paste causes shrinking or swelling. These volume changes are typical for hydraulic-cement concrete. [5, p.3]

Drying shrinkage crack can occur if the contraction due to shrinkage is restrained, for example, by some other structure like previously cast foundation, by another part of the structure or by the reinforcing steel embedded in the concrete. This restrained contraction develops tensile stresses within concrete, which may reach the tensile strength of the concrete leading to development of the cracks as shown in the Figure 2.8. [5, p.2-3]

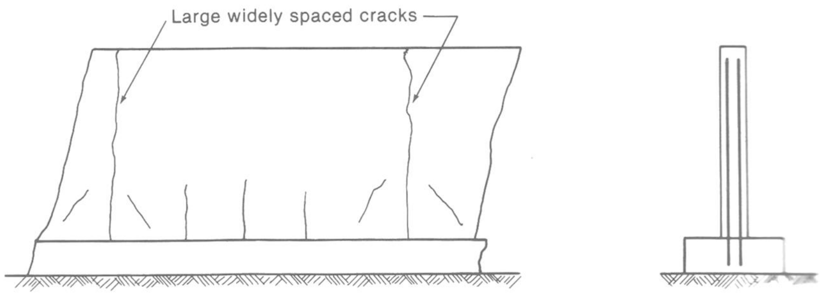


Figure 2.8. Shrinkage cracks in a wall due to restrained contraction. [3, p.322]

2.3.4 Plastic shrinkage crack

Plastic shrinkage cracks occur due to concrete surface shrinkage when water evaporates from the surface of freshly placed concrete faster than it is replaced by bleed water. Tensile stresses develop if the shrinkage is restrained by the concrete below the drying surface, which will result shallow cracks of varying depth in the weak, stiffening plastic concrete. This type of cracks are often quite wide at the surface. [6, pp.16-20]

2.3.5 Plastic settlement crack

After placing of concrete, the solids settle down and the mix water rises up to the surface. When there is no restraint, this only produces a slight lowering of the concrete surface. But, if settling is locally restrained, there is potential for a crack to form over the restraining element that can be, for example, a reinforcing bar, duct or insert. [6, pp.16-20]

Plastic settlement cracks tend to roughly follow the restraining element, e.g. reinforcing bars. Cracks can be quite wide at the surface, tend to extend only to the reinforcement or other restraining element and taper in width to that location. [6, pp.16-20]

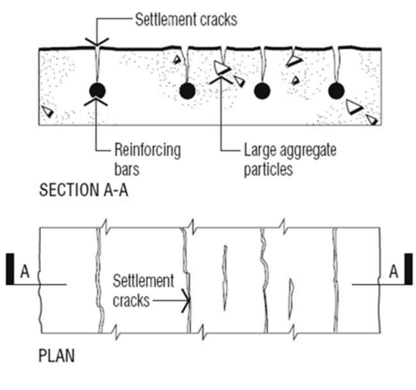


Figure 2.9. Settlement cracks. [6, p. 16]

2.3.6 Corrosion crack

Reinforcing steel embedded in concrete does not normally corrode due to inherently alkaline environment which forms a protective passive layer on its surface. However, if the concrete cover depth is insufficient or the concrete is permeable, concrete may carbonate as deep as the reinforcement and the protective layer may be at risk. The passive layer will be broken down in the presence of excessive amounts of chloride ions as well. [6, p.34]

If the passive layer protecting the steel breaks down, the reinforcing bar is liable to rust or corrode. As this occurs, it can cause the concrete to crack and spall due to expansive process, in which the volume of steel increases. Corrosion cracking is particularly noticeable at corners of beams and columns over the main steel. The pattern of links and stirrups can often be seen as well. [6, p.34]

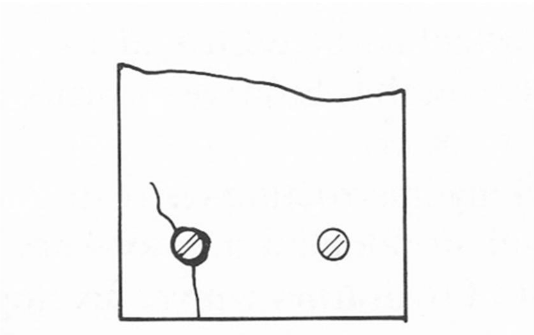


Figure 2.10. Corrosion crack. [3, p. 322]

2.3.7 Alkali-aggregate crack

Alkali-aggregate reaction occurs when the active mineral constituents of some aggregates react with the alkali hydroxides in the concrete. Reactivity occurs in two forms: alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR). Alkali-aggregate reactivity may cause a network of cracks in the surface layer of the concrete with persistent dampness along the edges of cracks, discoloration. [6, pp.35-36]

2.3.8 Crazing crack

Crazing is the phenomenon caused by stresses resulting from differential moisture movement due to either a high moisture concentration gradient or a discontinuity in composition near exposed surface. It generally occurs in the floated or trowelled surface layers of concrete slabs and the formed surfaces of cast stone and concrete. Crazing can lead to formation of map cracking in which the cracks are typically between 0,05 and 0,5mm wide and 2 or 3mm deep. [6, pp.32–33]

2.3.9 Sulfate attack crack

Sulfate attack is phenomenon resulting from two chemical reactions: the combination of sulfates with lime to form gypsum, and the combination of sulfates with hydrated calcium aluminates to form ettringite. Because the final reaction product occupies a larger volume than the original constituents, it may result concrete cracks due to internal expansion. [7, p.16]

2.3.10 Weathering crack

The weathering process includes freezing and thawing, wetting and drying, and heating and cooling. Concrete may crack due to damage from freezing and thawing of water in the paste, in the aggregate, or in both. The cracking is caused by the movement of water to freezing sites and by hydraulic pressure generated by the growth of ice crystals in hardened cement paste. [8, p5]

2.4 Autogenous healing of cracks

Concrete cracks can become self-compacted when fitting substances flow to the crack in suitable circumstances. That improves the tightness of concrete and the cover of reinforcement. As said, proper circumstances are required for the reaction to occur. Firstly, there must be non-hydrated cement and water in the concrete. In addition, crack width must remain constant. If crack width varies due to effect of live load, self-compaction cannot fill the crack. In case when structure is constantly in contact with water, there must be no corrosive substances in the water. Those could dissolve substances resulted during self-compaction. Flow for water must not be too heavy as well, so that formed substances are not eroded. [1, p.140]

Self-compaction occurs when calcium carbonate and calcium hydroxide grains in cement paste transfuse into the crack. Grains are then compacting into the crack and filling the crack while water evaporates. However, self-compaction of cracks must not be taken into consideration when calculating the crack width. It has only diminutive effect to cracks under suitable circumstances. [1, p.140]

2.5 Reasons for controlling cracking and crack width

There are three main reasons for controlling cracking and crack widths: appearance, leakage and corrosion. Wide cracks are unsightly, reduce the quality of a structure, and sometimes lead to concern by owners and occupants. If crack widths grow too wide, the influence of both physical and chemical protection of reinforcement decreases. In addition, more or less strict requirements for permeability may be set for structures containing gases, water and some other liquids. In such cases, cracks can lead to detrimental leakage.

2.5.1 Durability

When designing concrete structures it is important to decide environmental conditions each concrete member is supposed to be exposed to. Exposure conditions may be either chemical or physical. In some cases different surfaces of a concrete member may be assigned to different environmental conditions. For example inner and outer surfaces of external wall and roof may be assigned to different environmental conditions. Different room conditions, especially in industrial buildings, may also cause different environmental conditions in different surfaces of internal concrete members. As a simplification, environmental conditions are divided into classes which are described and shown in tabular form in the Table 2.1.

In order to achieve the required design working life of the structure, sufficient thickness of concrete cover must be chosen to protect reinforcing bars from corrosion. The nominal cover c_{nom} is defined as a minimum cover c_{min}, plus an allowance in design for deviation Δc_{dev}. Minimum concrete cover is defined on the basis of the safe transmission of bond forces, the protection of the steel against corrosion or an adequate fire resistance. The minimum concrete cover values with regard to durability for reinforcement according to Eurocode are shown in Table 2.2. [13, pp.49-50] [22, p. 51]

Table 2.1. Exposure classes related to environmental conditions in accordance with SFS-EN 1992-1-1. [22, p. 48]

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water

Table 2.2. Values of minimum cover, $c_{min,dur}$ requirements with regard to durability for reinforcement steel. [22, p. 51]

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Cracking of the concrete is a deteriorating effect as the formation of a crack in the concrete will allow the intrusion of carbon dioxide (and chlorides if present) and this will break down the passive state of the steel surface and allow corrosion to take place as described in subsection 2.3.6. According to some studies, there is slight relation between corrosion rate and crack widths, though in very beginning of the exposure period only. However, the presence of a crack itself is important regarding corrosion of steel bars in concrete when considering the service life of a structure. Therefore, the importance of crack control is notable in order to fill the requirements set for the service life of a structure. [9, pp. 194-201]

2.5.2 Appearance

Lack of sufficient crack control of concrete structures leading to wide cracks is a real disadvantage when it comes to aesthetics. Some sources suggest that cracks wider than 0,25mm on easily to be observed, clean, smooth surfaces can lead to public concern. If the surfaces are less easy to observe or are not smooth, wider cracks are tolerable to the eye. On the other hand, cracks in exposed surfaces may be accentuated by streaks of dirt or leached materials. According to some sources, it is suggested that crack width in terms of appearance should be related to viewing distance and the “prestige scale” of the structure. [3, p.325] [6, p.14]

2.5.3 Leakage

Crack control of concrete structures must be taken into consideration in some cases to avoid detrimental leakage of liquids or gases. Watertight concrete normally does not allow water to flow through the structure. However, cracks formed into concrete can disable the waterproofness, if those penetrate through the whole structure. According to Eurocode, the maximum crack width for waterproof structure is limited to a value 0,2mm [13, p.], although in certain cases, for example waterproof structures in nuclear industry projects, value of 0,1mm for through-cracks may be required. The ability of cracks to conduct gases is of significant importance for structures subjected to vapor pressure (e.g. confined enclosures of nuclear power plants) as well.

2.6 Methods for crack control

2.6.1 Curing

Cracking shall be limited to an extent that will not impair the proper functioning or durability of the structure or cause its appearance to be unacceptable. An important way for crack control is curing. With proper curing it can be influenced especially on shrinkage, creep, temperature difference and temperature changes of concrete, in which all effect greatly on the crack development.

2.6.2 Casting arrangements

Cracking can be controlled by appropriate casting arrangement and by dividing structures into sufficient small members with construction joints as well. Sometimes impairing of the cross sections may be considered, in which case cracks will develop into certain points without having detrimental effects. [17, p.197]

2.6.3 Allowable compression stress of concrete

Longitudinal cracks, micro-cracks or high levels of creep can result in unacceptable effects on the concrete structure. If the stress level under the characteristic combination of loads exceeds a critical value, longitudinal cracks may appear. That kind of cracking can lead to a weakening of durability. This can be avoided, if possible, by increasing the cover to reinforcement or by adding transversal reinforcement. Alternatively, limiting the compressive stress in the concrete is appropriate to avoid longitudinal cracking according to Eurocode. The compressive stress is then limited to a value $k_1 f_{ck}$ in areas exposed to environments of exposure classes XD, XF and XS. The recommended value of k_1 is 0,6, but, for use in a certain country the value may be defined in its National Annex. [13, p.117]

2.6.4 Limiting of crack width

Major aspect of crack control is limiting the crack width when cracks are caused by flexural or tensile stresses. Factors that affect on the crack width are bar diameter of tension reinforcement, total amount and spacing of tension reinforcement in tension area of concrete, bond characteristics of bars, thickness of concrete cover and type of loading. By taking into consideration of these factors crack width can be affected as well. In order to limit the crack width by reinforcement small bar diameter reinforcement with short spacing must be used. Reinforcing bars shall be applied as near to surface of tension area as possible. This way, the crack width will be limited, but the amount of cracks increases. [12, p.358] [17, p.197]

Allowable crack widths under service loads are listed in each structural design code depending on exposure conditions structure is subjected to. Eurocode presents allowable limit values for crack width, but each country has their own opportunity to define allowable limit values. These values are presented in National Annex to the Eurocode. In

Finnish National Annex design working life is considered differently than in Eurocode. If designed service life is more than 50 years, limiting values for crack width are reduced according to the Figure 2.11. Allowable crack width values according to The Finnish National Annex are shown in Table 2.3. [13, p. 118]

Table 2.3. Recommended maximum crack width values (mm) according to The Finnish National Annex. [13, p. 118]

Rasitusluokka	**Teräsbetonirakenteet ja tartunnattomat ankkurijännerakenteet**	**Tartuntajännerakenteet ja injektoidut ankkurijännerakenteet**
	Pitkäaikainen kuormayhdistelmä	Tavallinen kuormayhdistelmä
X0, XC1	$0{,}4^1$	0,2
XC2,XC3, XC4,XD1,XS1	0,3	$0{,}2^2$
XD2,XD3,XS2,XS3	0,2	Vetojännityksetön tila
Huom. 1 Rasitusluokkien X0 ja XC1 yhteydessä halkeamaleveydellä ei ole vaikutusta säilyvyyteen, ja tämä raja on asetettu, jotta tavallisesti saavutetaan kelvollinen ulkonäkö. Jos ulkonäköehtoja ei aseteta, tätä rajaa voidaan väljentää. **Huom. 2** Näiden rasitusluokkien yhteydessä tarkistetaan myös, ettei vetojännitystä esiinny kuormien pitkäaikaisen yhdistelmän vallitessa.		

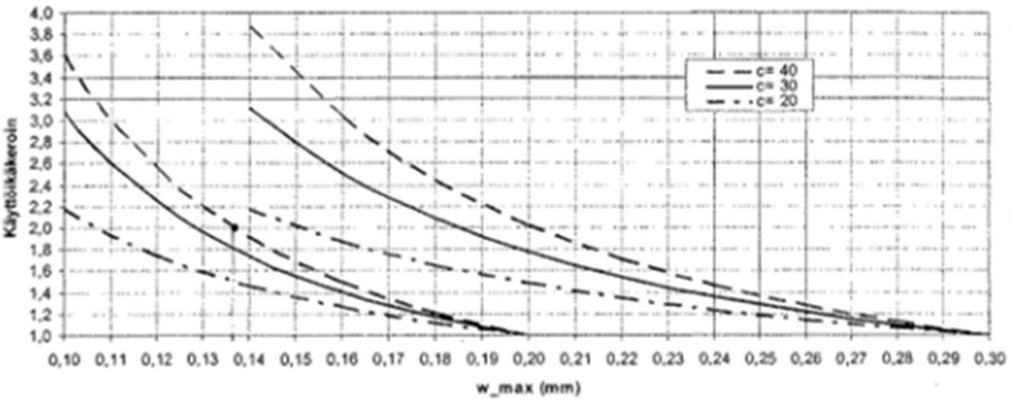


Figure 2.11. Graph for reducing factor of the maximum crack width when service life is more than 50 years. [13, p. 119]

2.6.5 Minimum reinforcement for crack control

In case crack control is required, minimum amount of reinforcement is needed to control cracking in areas where tension is expected. The required amount of reinforcement may be estimated from equilibrium between the tensile force in concrete just before cracking and the tensile force in bars at yielding or at a lower stress if necessary to limit the crack width. Unless a more accurate calculation shows lesser areas to be adequate, the required minimum areas of reinforcement may be calculated by the following equation. [13, pp. 118-119]

$$A_{s,min}\sigma_s = k_c k f_{ct,eff} A_{ct}$$ (2.11) [13, expression 7.1, p. 118]

where $A_{s,min}$ is the minimum area of reinforcing steel within the tensile zone, A_{ct} is the area of concrete within tensile zone which is that part of the section calculated to be in tension just before formation of the first crack, σ_s is the absolute value of the maximum stress permitted in the reinforcement immediately after formation of the crack, $f_{ct,eff}$ is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur, k is the coefficient which allows for the effect of non-uniform self-equilibrating stresses which lead to a reduction of restraint forces. The value of coefficient k is 1,0 for webs with $h \leq 300$ mm or flanges with widths less than 300 mm, and 0,65 for webs with $h \geq 800$ mm or flanges with widths greater than 800 mm. Coefficient k_c in the equation of minimum area takes account of the stress distribution within the section immediately prior to cracking and of the change of the lever arm. The value of k_c is 1,0 for pure tension. For rectangular sections and webs of box sections and T-sections subjected to bending or bending combined with axial forces, the value of k_c may be calculated from the following equation. [13, p. 119]

$$k_c = 0{,}4 \cdot \left[1 - \frac{\sigma_c}{k_1\left(\frac{h}{h^*}\right) f_{ct,eff}}\right] \leq 1$$ (2.12) [13, expression 7.2, p. 119]

, in which σ_c is the mean stress of the concrete acting on the part of the section under consideration obtained from the following expression:

$$\sigma_c = \frac{N_{Ed}}{bh}$$ (2.13) [13, expression 7.4, p.119]

, in which N_{Ed} is the axial force at the serviceability limit state acting on the part of the cross-section under consideration. It should be determined considering the characteristic values of prestress and axial forces under the relevant combination of actions. Factor h^* equals h when $h < 1{,}0$ m and 1,0m when $h \geq 1{,}0$ m. k_1 is a coefficient considering the effects of axial forces on the stress distribution. The numerical value of k_1 is 1,5 if N_{Ed} is a compressive force, and $\frac{2h^*}{3h}$ if N_{Ed} is a tensile force. [13, p. 119]

For flanges of box sections and T-sections, coefficient k_c can be calculated from the following equation.

$$\mathrm{k_c} = 0{,}9 \cdot \left[\frac{\mathrm{F_{cr}}}{\mathrm{A_{ct} f_{ct,eff}}}\right] \geq 0{,}5$$ (2.14) [13, equation 7.3, p.119]

, in which F_{cr} is the absolute value of the tensile force within the flange immediately prior to cracking due to the cracking moment calculated with $f_{ct,eff}$. [13, pp. 119-120]

2.6.6 Crack control without direct calculation

Control of cracking without direct calculation is an effective and expeditious method for limiting the crack width values within allowable limiting values. The crack width calculation rules according to Eurocode, which are shown later in section 3.1 may be presented in a tabular form by restricting the bar diameter or spacing as a simplification, as shown in Table 2.4 and 2.5.

When the minimum reinforcement given in subsection 2.6.5 is provided, crack widths are unlikely to be excessive for cracking caused dominantly by restraint, if the bar sizes given in Table 2.4 are not exceeded and where the steel stress is the value obtained immediately after cracking. For cracks caused mainly by loading, either the provisions of Table 2.4 or the provisions of Table 2.5 are complied with. The steel stress which is the main factor of this tabular method should be calculated on the basis of a cracked section under the relevant combination of actions. [13, p.121]

For pre-tensioned concrete, in which cracking is mainly restrained by tendons with direct bond, Tables 2.4 and 2.5 may be used with a stress that is obtained by the difference between the total stress and prestress. In case of post-tensioned concrete, where crack control is provided primarily by ordinary reinforcement, the tables may be used with the stress in this reinforcement calculated with the effect of prestressing forces included. [13, p.121]

Table 2.4. Maximum bar diameter ϕ^*_s for crack control. [22, p. 123]

Steel stress[2] [MPa]	Maximum bar size [mm] w_k= 0,4 mm	w_k= 0,3 mm	w_k= 0,2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 2.5. Maximum bar spacing for crack control. [22, p.123]

Steel stress[2] [MPa]	Maximum bar spacing [mm]		
	w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

The values given in Tables 2.4 and 2.5 to provide crack control are obtained by the calculations with assuming the concrete cover as 25 mm and concrete strength as 2,9MPa. In addition numerical value $0.5*h$ is used for coefficient h_{cr}; $0,1h$ for (h-d); 0,8 for k_1: 0,5 for k_2; 0,4 for k_c; 1,0 for k; 0,4 for k_t and 1,0 for k_4. [13, p.121]

In case of bending (at least part of section in compression) the maximum bar diameter ϕ^*_s in Table 2.4 should be modified as follows:

$$\phi_s = \phi_s^*\left(f_{ct,eff}/2.9\right)\frac{k_c h_{cr}}{2(h-d)}$$ (2.15) [13, equation 7.6N, p.122]

, and for tension (uniform axial tension) with the following formula:

$$\phi_s = \phi_s^*\left(f_{ct,eff}/2.9\right)h_{cr}/\left(8(h-d)\right)$$ (2.16) [13, equation 7.6N, p.122]

, where ϕ_s is the adjusted maximum bar diameter, ϕ^*_s is the maximum bar size given in the Table 2.4, h is the overall depth of the section, h_{cr} is the depth of the tensile zone immediately prior to cracking and d is the effective depth to the centroid of the outer layer of reinforcement. [13, p.121]

2.6.7 Surface reinforcement

Surface reinforcement is required in structures where the main reinforcement consisting of bars with diameter greater than 32 mm or bundled bars with equivalent diameter greater than 32 mm. Surface reinforcement should also be used in cases where concrete cover is greater than 70 mm and also in cases in which control of cracking is not achieved by reasonable amount of main reinforcement. Additional surface reinforcement, which consists of wire mesh or small diameter bars, should be embedded into such structures in order to resist spalling. This kind of reinforcement has to be placed outside the links of main bars, which is shown in the Figure 2.12. [13, p.221]

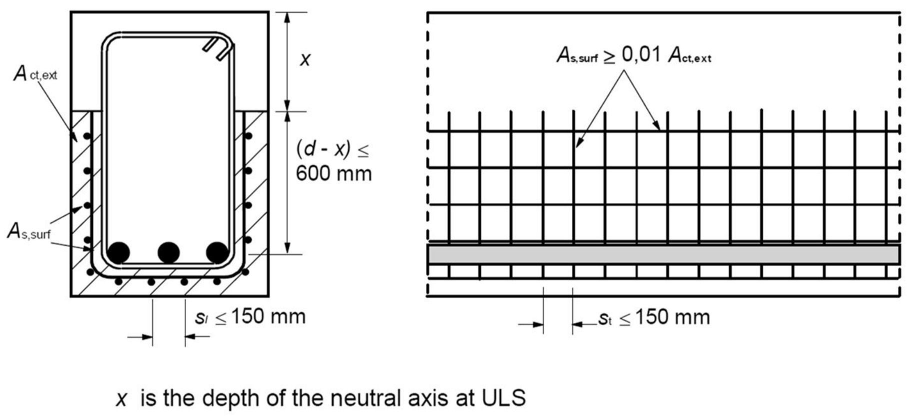


Figure 2.12. Example of surface reinforcement. [22, p. 221]

The area of surface reinforcement, that is marked as $A_{s,surf}$, should not be less than $A_{s,surfmin}$ in the two directions parallel and orthogonal to the tension reinforcement in the concrete beam. The value of $A_{s,surfmin}$ is found in the National Annex of the Eurocode and the recommended value is $0,01A_{ct,ext}$, in which $A_{ct,ext}$ is the area of the tensile concrete external to the links, shown in the Figure 2.12. [13, p.221]

For enhanced durability similar surface reinforcement should be used in cases where the concrete cover to reinforcement is greater than 70mm as well. The minimum value for the reinforcement in such cases is $0.005A_{ct,ext}$ in each direction. [13, p.221]

2.6.8 Transversal reinforcement in the anchorage zone

In addition to surface reinforcement and crack width calculations anchoring of large diameter bars with diameter greater than 32mm should be taken into consideration to achieve required crack control as well. As splitting forces are higher and dowel action is greater with use of large diameter bars, anchoring of the bars must be ensured with sufficient transversal reinforcement in the anchoring zone. Large bars should be anchored with mechanical devices, but, they may be anchored as straight bars as well, when links are used as confining reinforcement. [13, pp. 142-143]

Lapping of large diameter bars is not recommendable with the exception of sections with a minimum dimension 1.0m or where the stress is less than 80% of the design ultimate strength. In addition of shear reinforcement, transverse reinforcement should be embedded in the anchorage zones where transverse compression is not present. That additional reinforcement should fulfill the following requirements for the minimum amount of reinforcement area. In direction parallel to the tension face the value of transversal reinforcement should not be less than:

$A_{sh} = 0{,}25 A_s n_1$ (2.17) [13, equation 8.12, p.142]

, and in the direction perpendicular to the tension face:

$A_{sv} = 0{,}25 A_s n_2$ (2.18) [13, equation 8.13, p.142]

, in which A_s is the area of an anchored bar, n_1 is the number of layers with bars anchored at the same point in the member, and n_2 is the number of bars anchored in each layer. Added transverse reinforcement in the anchorage zone should be uniformly distributed and the spacing of bars should be less than 5 times the diameter of the longitudinal reinforcement. An example in an anchorage for large diameter bars is shown in the Fig. 2.13. [13, pp. 142-143]

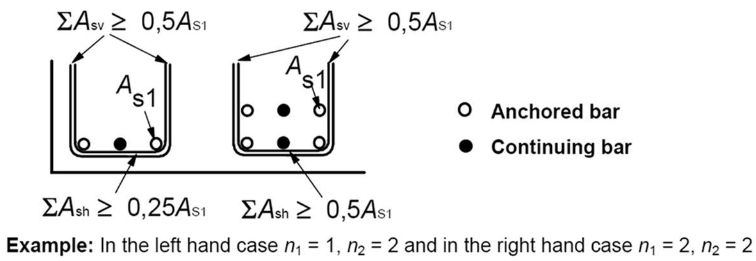


Figure 2.13. Additional reinforcement in an anchorage for large diameter bars where there is no transverse compression. [22, p.143]

2.7 Crack control of massive concrete

Massive concrete structures are one special case in terms of crack control and special attention has to be put on to avoid detrimental formation of cracks. The main reason is the heat rise due to hydration. In massive structures, the heat does not transfer from the internal concrete to the surface as fast as it does in normal structures. In this section, the crack control of massive concrete is discussed.

2.7.1 Mass concrete

Massive concrete is defined differently depending on the source. The American Concrete Institute suggests that mass concrete is considered as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking". The definition is somewhat nonspecific because the concrete mix design, the dimensions, the type of the placement, and the curing methods all affect whether or not cracking will occur. By same source, a minimum dimension structure has to be considered as a mass concrete is equal or greater than 900mm. Other source suggests that, if thermal difference between the middle part of the structure and external temperature exceeds 20°C, it is considered as a mass concrete. The minimum dimension of the structure is then at least 1m. The internal heat in a wall structure with different thickness after placing is shown in the Figure 2.14. [10, p.2] [11, p.529-530]

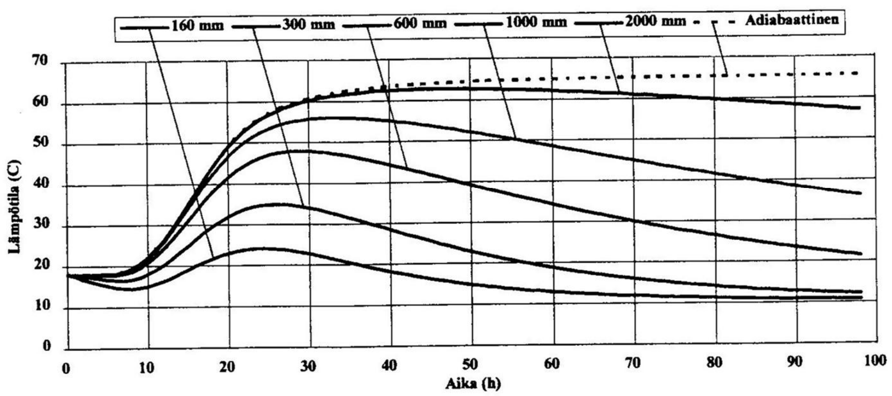


Figure 2.14. Internal temperature of a wall structure after placing in case of external temperature of 10oC. [11, p. 529]

2.7.2 Reasons for mass concrete crack control

There are two primary reasons for mass concrete crack control: thermal cracking and delayed ettringite formation. The primary reason, thermal cracking, is described in subsection 2.3.2. In mass concrete, the heat due to hydration does not transfer from the internal concrete to the surface as fast as it does in normal structures. As a result, a great temperature difference develops between the interior and the surface, which generates thermal stresses in the concrete. Therefore, the main focus in mass concrete has to be on the heat control during setting so that temperature difference does not grow too excessive. [10, pp.2-3]

The other concern, delayed ettringite formation, results from the concrete getting too hot. The cement hydration reactions change due to high temperatures. At temperatures above 70°C, unstable hydration products develop in some concretes. In concrete where DEF (delayed ettringite formation) occurs, the unstable hydration products can eventually begin to expand within the concrete. This is a long-term effect that may not occur for months or years after the time of construction. In its worst form, DEF can cause significant cracking. The basic rule for preventing DEF is to keep the concrete temperature below 70°C. [10, pp.2-3]

2.7.3 Ways to mass concrete crack control

As described above, the maximum temperature and the maximum temperature difference are important to be limited sufficiently to prevent formation of cracking. Crack control can be performed by an optimal mix design, insulation or concrete cooling either before or after placement.

The most effective and easiest way to crack control is using an optimal mix design. Using low-heat cement decreases the heat of hydration, though the effect is slight and also many cement manufacturers do not provide heat of hydration data in their normal documentation. More effective way to limit the maximum temperature is using of fly ash or slag cement. The heat of hydration in fly ash is even half of that of cement and it is typically used to replace 25 to 40 percent of the cement. Slag cement is often used 50 to 75 percent of the cement and the heat hydration typically 50 to 75 percent of the cement. Both slag cement and fly ash decrease the early age strength of the concrete, but can greatly increase the long-term strength. The influence of slag cement on the maximum heat is demonstrated in the Figure 2.15 where slag cement is used to replace 0%, 20%, 40%, 60% and 80% of the Rapid cement in a structure 2 meters of a thickness. [10, pp.4-5] [11, pp.529-530]

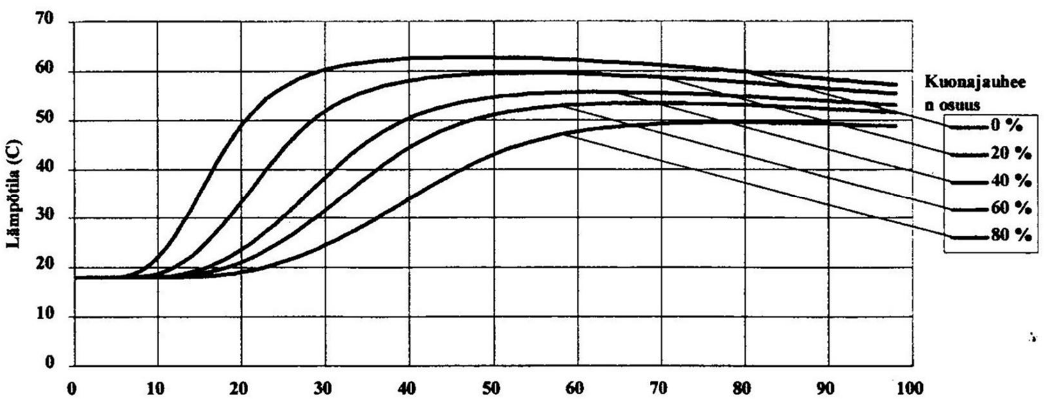


Figure 2.15. Internal temperature of a wall structure when Rapid cement is replaced with slag and external temperature is 10°C. [11, p. 531]

However, the total cementitious materials content has the most effective influence on the heat generation in concrete. It is always advisable to minimize the amount of cement as low as possible in mass concrete for the required compressive strength. Mix design can be performed so that the age of accepted strength is, for example, 42 or 56 days in place of 28 days resulting minimized heat energy and temperature after placement. Possible problems with pumping and placing can be solved by choosing as coarse aggregates as possible, dense composition and by using plasticizers. In the Figure 2.16, the effect of Rapid cement content in a 2 meters thick wall after placing is shown. One option to reduce the amount of cementitious materials needed to achieve a particular strength is to use larger and better graded aggregates. Aggregates such as limestone, granite, or basalt should be used, as well, to reduce the thermal expansion and potential for thermal cracking. [10, pp.4-5] [11, pp.531-532]

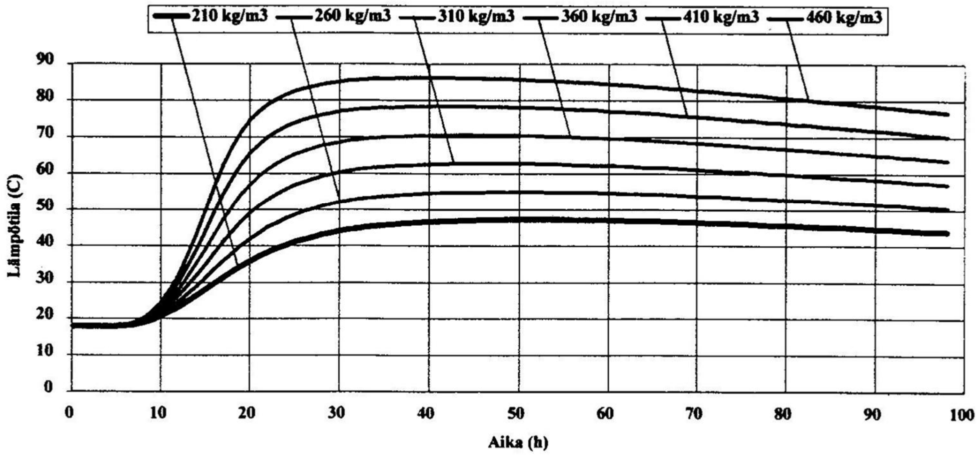


Figure 2.16. Internal temperature of a wall structure after placing with different cement contents. [11, p. 532]

Composition of concrete cannot always be chosen so that there wouldn't be a risk of cracking due to temperature differences. In such cases, thermal insulation of mass concrete is a functional solution, although it may seem counter-intuitive. The purpose is to warm the concrete surface and reduce the temperature difference. Insulation increases, however, the maximum temperature in the middle parts of the concrete. But, the influence is rather slight deep in the concrete. Concrete insulating blankets are generally used; however, virtually any insulating material is often acceptable.

When using insulation to prevent thermal cracking, insulation should be kept in place until the hottest portion of the concrete cools to within the temperature difference limit of the average air temperature. For example, if a 45°C allowed temperature difference is specified and the average air temperature is 10°C, insulation should not be removed until the hottest portion of the concrete cools down to 55°C. This may require insulation to be kept in place up to several weeks (especially on thicker placements). During this time, insulation can possibly be temporarily removed to perform work, if that is done for a window of time when temperature difference in the concrete is less than the specific limit. [10, pp.4-5] [11, p.532]

Concrete cooling, which can be used both before and after placement, is a method to reduce concrete temperature that can be used both before and after placement. Normally, temperature of concrete delivered to the site is about 10°C warmer than the average air temperature. Delivered concrete can be precooled prior to placement to reduce its temperature. Approximately, every 1°C of precooling reduces the maximum temperature by a similar amount after placement.

To precool the concrete by about 5°C, one option is to use chilled water. Shaved or chipped ice can be substituted for up to about 75 percent of the mix water to reduce the concrete temperature by up to 15 to 20°C. In case when extreme precooling is needed, liquid nitrogen can be used to precool the concrete mix by any amount. This method requires highly specialized equipment to safely cool concrete and can be expensive. However, it is a notable option for the contractor as it can be done at the site or at the ready-mix plant.

Reducing the maximum temperature of the concrete after placement is almost impossible. If insulation is removed, it cools the surface, which increases the temperature difference and the likelihood of thermal cracking. In fact, moisture retention curing methods should be used to avoid artificially cooling the surface, which actually can increase the liability of thermal cracking.

2.8 Special cases of structures in terms of crack control

Calculation of crack width according to Eurocode is mainly designed for customary structures. However, there are some special cases of structures when the accuracy of the formulae is highly doubted due to certain aspects. Thick concrete cover, regardless of the structural design code, increases the maximum calculated crack width directly. Normally, the reinforcing bars are covered with 20 to 50mm layer of concrete in which case reasonable results are obtained from crack width calculations. The excess cover may be required sometimes, which induces difficulties with crack width calculations. The reinforcement is another subject when it comes to problems with the accuracy of the crack width calculations. There are also structures that are heavily reinforced occasionally, which means that large diameter reinforcing bars are embedded into multiple layers. Two cases of structures are now shown in an effort to visualize these questions.

2.8.1 Wall of a reactor containment structure in nuclear power plant

Typical nuclear reactors in US and Europe are enclosed with containment structures to prevent radioactive leakage in case of serious internal accident due to increased pressure on the containment. Most of the containments in US and Europe consist of two parts: an outer bearing concrete structure and inner sealing which is a tight-welded steel liner. In this type of containments, the outer concrete is the load-bearing part and is normally prestressed in case of pressurized water reactor. In case of boiling water reactor the outer concrete is usually non-prestressed structure. The function of steel liner is to secure the tightness and it has no intended bearing function. The liner is normally attached to the concrete by some type of discrete connector, which is welded to the liner and cast into the concrete. Connectors may be, for example, studs, vertical L-sections or tubes. A principal sketch of a prestressed concrete containment with steel liner is shown in the Figure 2.17. [18, p.1]

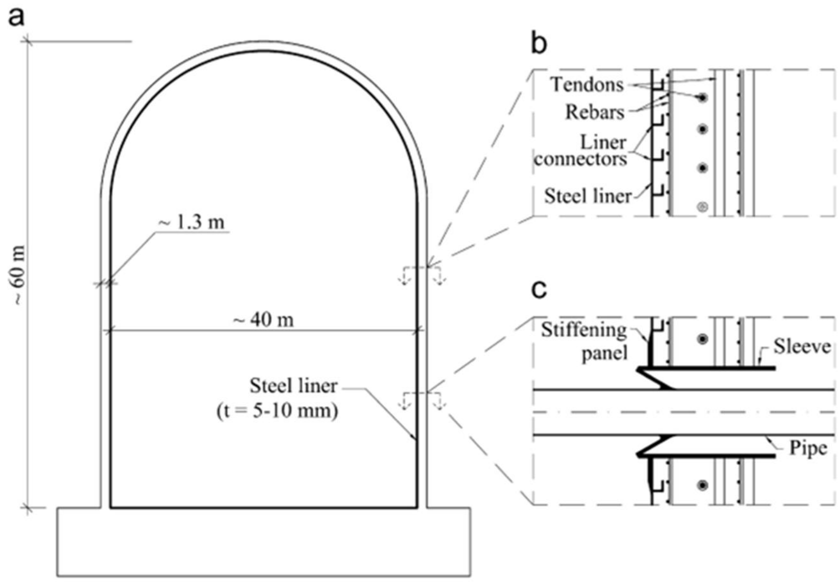


Figure 2.17. Principal sketch of a prestressed concrete containment with steel liner. (a) Vertical section, (b) Horizontal section and (c) pipe penetration horizontal section. [18, p.1]

Some problems related to crack control exist in this type of containment structure. First of all, most of the concrete members are considered as a mass concrete. Therefore, requirements for mass concrete control, discussed in section 2.7 have to be taken into consideration in order to prevent non-structural crack formation already at the construction phase.

Discrete connectors welded to the steel liner require certain anchoring length, which increases the concrete cover thickness for reinforcing bars sometimes up to 150mm. Most crack width models indicate that increasing concrete cover results in increased crack spacing and hence increased crack width. Surface mesh is usually embedded in an effort of limiting the crack spacing in such cases. However, usage of surface mesh may sometimes be difficult or impossible. Wall of a containment structure is, as well, heavily reinforced, which means large diameter bars in multiple layers. Both the excess of concrete cover and heavy reinforcement combined with thick concrete section induce problems with calculation of crack width.

2.8.2 Diaphragm wall

Diaphragm walls are used to construct underground stations in city centres, multi-level underground car parks, road junctions and underpasses, and open cut and cut & cover rail tunnels. In deep shafts for tunnel ventilation, intervention shafts and water treatment plants can as well be constructed by using diaphragm walls. The walls are usually located in confined inner-city areas where space is at a premium and those extend typically to a depth of 20m to 50m. Standard widths of diaphragm walling equipment are 600, 800, 1000, 1200 and 1500mm although larger can be provided. [19, p. 1]

Diaphragm walling is constructed of vertical walls by means of deep trench excavations. Stability of the excavation is maintained by the use of a drilling fluid, usually a bentonite suspension. The walls consist of discrete panels, lengths ranging typically between 2,5m and 7,0m. Purpose built grabs or, in appropriate circumstances, milling machines are used. [19, p. 1]

Figure 2.18. Diaphragm wall. [19, p. 1]

Diaphragm walls are another special case of a structure where controlling of crack width is more or less difficult. Like in case of a containment structure, wall panels are heavily reinforced as massive loading due to soil pressure requires multiple rows of steel. Large concrete cover thickness is as well typical for diaphragm walls due to severe exposure conditions.

3 Calculation of crack width

As noted the crack width must be limited to a certain value, depending on the case, to fulfill the requirements for the structure. To calculate maximum crack width to a certain structure subjected to loading, there are several different types of methods universally from which four are considered in this chapter. Eurocode, The National Building Code of Finland, German DIN-code all are based on the CEB/FIB approach in which it is presumed that crack width is governed by the relative slip between the concrete and steel. Therefore, these three methods are, more or less, similar in appearance and calculation formulas include same kind of factors. ACI-code, on the contrary, is based on different approach in which major factor contributing to the crack width is strain release in the concrete in the vicinity of the crack. At first, these four calculation methods are presented in this chapter. After that, not only a simple example but also an example of a special case of crack width is calculated by all of these calculation methods.

3.1 EC2

Eurocode 2 presents the formula for calculating a crack width, which is:

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm})$$ (3.1) [13, expression 7.8, p.123]

, in which

$$s_{r,max} = k_3 c + k_1 k_2 k_4 \frac{\phi}{\rho_{p,eff}}, \text{ when bar spacing} \leq 5\left(c + \frac{\phi}{2}\right)$$ (3.2) [13, exp.7.11, p.124]

$$s_{r,max} = 1{,}3(h - x), \text{ when bar spacing} > 5\left(c + \frac{\phi}{2}\right)$$ (3.3) [13, expression 7.11, p.124]

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \frac{f_{ct,eff}}{\rho_{p,eff}}(1 + \alpha_e \rho_{p,eff})}{E_s} \geq 0{,}6\frac{\sigma_s}{E_s}$$ (3.4) [13, expression 7.9, p.123]

In the expression 3.1, $s_{r,max}$ is the maximum crack spacing and calculation of it depends on the main reinforcing bar spacing. If the bar spacing is equal or less than $5(c+ \phi/2)$, there will be taken into consideration concrete cover c, the diameter of the main bars ϕ and reinforcing ratio $\rho_{p,eff}$ in the calculation. In addition, there is a bunch of factors in the equation, in which e.g. bond characteristics of reinforcing bars and the type of loading are taken into consideration. In case main reinforcing bar spacing is more than $5(c+ \phi/2)$, crack spacing is depending on the height of the cross section and the location of the neutral axis. In the crack width equation, the average strain of the main bars ε_{sm} and the concrete ε_{cm} are taken into consideration as well. To the difference between these strains is given a separate formula, which takes into account a.o. the stress of the main bars, the duration of the loading and the effective tensile stress during the loading. In case the concrete has obtained the full strength, the effective tensile strength $f_{ct,eff}$ needed in the equation is equal to $f_{ctm,28}$. The time of the loading is taken into account by the factor k_t, which equals 0,6 in case of the short term loading, and 0,4 in case of the long term loading. In other words long term loads induce wider cracks than short term loads. The lower limit is given to the difference $\varepsilon_{sm} - \varepsilon_{cm}$, which is shown in the expression 3.4. [13, chapter 7] [12, p.361]

It can be noticed from the equations above that thickness of the concrete cover, in particular, has a great influence on the crack width as it effects on the crack spacing greatly. Crack width then, as can be seen, is proportional to crack spacing directly. [12, p. 361]

3.2 RakMK B4

The formula for calculating crack width according to the National Building code of Finland is:

$$w_k = \varepsilon_s \left(3{,}5c + k_w \frac{\phi}{\rho_r}\right) \qquad \text{(3.5) [14, expression 2.81, p.32]}$$

, in which

$$\varepsilon_s = \frac{\sigma_s}{E_s}\left[1 - \frac{1}{25k_w}\left(\frac{\sigma_{sr}}{\sigma_s}\right)^2\right] \geq 0{,}4\frac{\sigma_s}{E_s} \qquad \text{(3.6) [14, p.32]}$$

As seen, the expression 3.6 corresponds to the maximum crack spacing $s_{r,max}$ by the Eurocode, and the form of it is pretty similar as well. The differences are mainly dissenting factors. The bond characteristics of the main bars are taken, as well, into account by the factor k_w in the National Code of Finland. Unlike the Eurocode, there are no factors depending on the type of loading in the formula. Additional difference to the Eurocode is the average strain of the concrete which is not included in the formula. Only the strain in the main bars ε_s at the location of the crack is included in the crack width formula. The parameter, indicative of the cracking, involved in the formula of ε_s is the stress in the main bars at the time of the crack development σ_{sr}. The minimum limit is determined for the steel strain in The National Code of Finland too. The appearance of it is like that of the EC2, but the difference is smaller coefficient 0,4.

The concrete cover plays, like in the EC2, an important role when calculating the crack width. However, the minimum value c_{min} is used as the thickness of the concrete cover instead of the nominal value in the crack width formula. It can be seen as well that the reinforcement reduces the crack width due to the term ρ_r. Greater diameter of the bar seems to increase the crack width according to the formula as well. [14, p.361]

3.3 DIN 1045-1

The German Institute for Standardization (DIN) presents the following formula for the calculation of the crack width:

$$w_k = s_{r,max}(\mathcal{E}_{sm} - \mathcal{E}_{cm})$$ (3.7) [15, expression 135, p.133]

, in which

$$s_{r,max} = \frac{d_s}{3{,}6 \cdot eff\rho} \leq \frac{\sigma_s \cdot d_s}{3{,}6 \cdot f_{ct,eff}}$$ (3.8) [15, expression 137, p.133]

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - 0{,}4 \frac{f_{ct,eff}}{eff\rho} \cdot (1 + \alpha_e \cdot eff\rho)}{E_s} \geq 0{,}6 \frac{\sigma_s}{E_s}$$ (3.9) [15, expression 136, p.133]

The general basis of the formula by DIN is similar to that of EC2, as seen, when comparing the expressions 3.1 and 3.7. In expression 3.9, the difference between the average steel and concrete strain, corresponds to the Eurocode. DIN, as well as RakMK, do not take into account the time of loading, which is seen as a single factor of 0,4. The most significant differences compared with not only EC2 but also RakMK are seen in the equation for the maximum crack spacing $s_{r,max}$. The thickness of the concrete cover, for example, seems not to have as substantial effect on the maximum crack spacing, because concrete cover has not been included in equation 3.8 directly.

3.4 ACI 318

The American Concrete Institute (ACI) has a different approach, as said, to the model of cracking. Therefore, the formula of the maximum crack width is totally different than formulas above, which are based on the CEB model. The formula for calculation of crack width in inches by ACI is:

$$\omega = 0.076\beta f_s \sqrt[3]{d_c A}$$ (3.10) [5, expression 4-2a, p.18]

Although the formula differs significantly from the formulas by EC2, RakMK and DIN, there are some similarities as well. As a common factor, the steel stress is taken into account in the ACI formula with a factor f_s that is given in kilopound per square inch (ksi). Also the concrete cover affects directly the maximum crack width. It is taken into account by a factor d_c which is thickness of concrete cover from the outermost tension fiber to the center of the closest bar in inches. In addition, the ACI formula takes, as well, into account the area of concrete subjected to tensile stresses by a factor A. That is the area of concrete symmetric with reinforcing steel divided by number of bars in square inches. There is also a geometric factor β that is ratio of distance between neutral axis and tension face to distance between neutral axis and reinforcing steel. [5, p.18]

As it can be seen, the most important variable is the steel stress. It is noticeable that the reinforcing bar diameter does not have an effect on the maximum crack width as it did in the formulas by EC2, RakMK and DIN. When comparing the ACI formula with the others, it is noticed, as well, that there are no variables, which would take into account the average concrete strain.

3.5 Sample calculation of crack width in a simple case

Let us next consider the calculation of crack width according to each four methods in a simple case of a reinforced concrete structure. An example structure is 200mm thick concrete slab, in which 12mm reinforcing bars are embedded with spacing 150mm (A_s = 754 mm^2) at the tension face (Fig 3.1). In the calculations the slab is considered as a beam of a width of 1000mm. The thickness of the concrete cover c_{min} is 25mm. Concrete class is C35/45 and reinforcing steel B500C1. The slab is subjected to pure bending. The position of the neutral axis for cracked concrete will be calculated by using the elastic modulus of concrete for short-term loading.

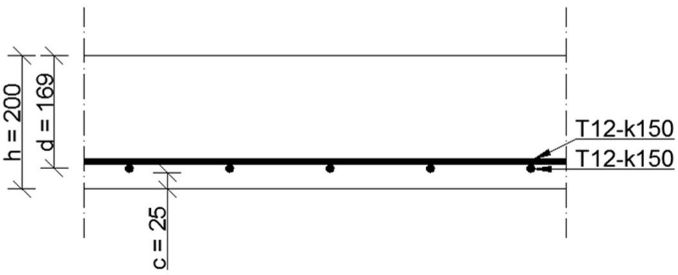


Figure 3.1. Cross section of the sample slab**.**

Let us first calculate the cracking moment for the cross-section in order to evaluate steel stress caused by pure bending in case cracking occurs. The cracking moment M_{cr} can be evaluated from the geometry with assuming the effective tensile strength $f_{ct,eff}$ as f_{ctm} (=3,2MPa for concrete C35/45):

$$M_{cr} = f_{ct,eff}\frac{bh^2}{6} = 3{,}2MPa \cdot \frac{1000mm \cdot (200mm)^2}{6} = 21{,}3kNm$$

When the cracking moment is evaluated, we can assume in the calculations the structure to be subjected to bending moment of 40kNm in order to ensure the cracking will occur.

3.5.1 Calculation according to EC2

The calculation of the crack width according to EC2 begins with determining the height of the neutral axis in the serviceability limit state. The height is needed when calculating the effective area of the concrete subjected to tension. There are found prepared graphs in the literature for defining the position of the neutral axis, when the ratio between the elastic modulus of steel and concrete, and reinforcement ratio are known. Now, the position of the neutral axis in the cracked cross section at the service limit state is, however, performed computationally as follows.

Firstly, we have to calculate parameter α, which is ratio between the elastic modulus of steel and concrete, and parameter ρ, which is reinforcement ratio:

$$\alpha = \frac{E_s}{E_c} = \frac{200000MPa}{22 \cdot \left(\frac{35+8}{10}\right)^{0,3} MPa} = 5{,}869$$

$$\rho = \frac{A_s}{d \cdot b} = \frac{754mm^2}{169mm \cdot 1000mm} = 0{,}00446$$

Then, ratio between steel and concrete strains can be calculated from the equation:

$$\frac{\varepsilon_c}{\varepsilon_s} = \rho\alpha + \sqrt{\rho\alpha(2+\rho\alpha)} \qquad (3.11)\ [16, p.163]$$

When $\rho\alpha = 5{,}869 \cdot 0{,}00446 = 0{,}0262$, ratio $\frac{\varepsilon_c}{\varepsilon_s}$ can be evaluated:

$$\frac{\varepsilon_c}{\varepsilon_s} = 0{,}0262 + \sqrt{0.0262(2+0.0262)} = 0{,}257$$

Now, the distance to the neutral axis x (see Figure 3.2) can be calculated:

$$x = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s} \cdot d = \frac{\varepsilon_c/\varepsilon_s}{\varepsilon_c/\varepsilon_s + 1} \cdot d = \frac{0,257}{1,257} \cdot 169mm = 34,5mm$$

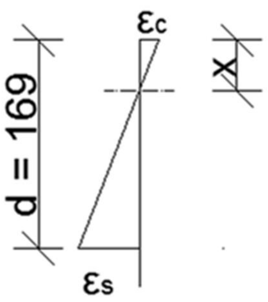


Figure 3.2. Strain graph and position of the neutral axis.

Next, the effective tension area, shown in the Figure 3.3, needs to be calculated. The height of the area is:

$$h_{c,ef} = min\left(2,5(h-d); \frac{h-x}{3}; \frac{h}{2}\right) \quad (3.12)\ [13,\ p.120]$$

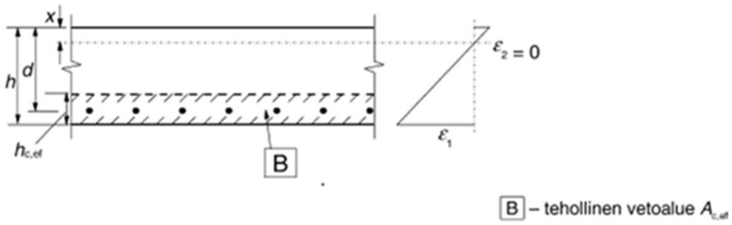


Figure 3.3. The effective area in tension zone. [13, p.121]

When the value of x calculated above is input to the expression 3.12, the height $h_{c,ef}$ is obtained:

$$h_{c,ef} = min\left(2,5(200-169); \frac{200-34,5}{3}; \frac{200}{2}\right) = 55,17mm$$

The effective tension area $A_{c,eff}$ is then:

$$A_{c,eff} = 55166mm^2$$

Before calculating the maximum crack spacing, $\rho_{p,eff}$ is calculated from the equation:

$$\rho_{p,eff} = \frac{A_s}{A_{c,eff}} = \frac{754mm^2/m}{55166mm^2/m} = 0,0137 \quad (3.13)\ [13,\ equation\ 7.10]$$

Now, the maximum crack spacing can be evaluated from the expression (3.2). The factors used in the equation are:

k_1 = 0,8 (bars are assumed to have good bond characteristics)
k_2 = 0,5 (in case of bending)
k_3 = 3,4 (recommended value)
k_4 = 0,425(recommended value)

$$s_{r,max} = 3{,}4 \cdot 25mm + 0{,}8 \cdot 0{,}5 \cdot 0{,}425 \frac{12mm}{0{,}0137} = 234mm$$

As we know the position of the neutral axis, we can evaluate the steel stress caused by the chosen bending moment M (=40kNm) which is greater than cracking moment. The steel stress in the bars equals:

$$\sigma_s = \frac{M}{A_s(d - {}^{x}\!/_{3})} = \frac{40kNm}{754mm^2 \cdot (169mm - {}^{34{,}5mm}\!/_{3})} = 337MPa$$

When the value 0,4 is given to the factor k_t due to long term loading, the difference between the main strains of steel and concrete can be calculated from the expression 3.4:

$$\mathcal{E}_{\mathrm{sm}} - \mathcal{E}_{\mathrm{cm}} = \frac{337\mathrm{MPa} - 0{,}4 \cdot \frac{3{,}2\mathrm{MPa}}{0{,}0137}(1 + 5{,}869 \cdot 0{,}0137)}{200000\mathrm{MPa}} = 0{,}00118$$

Finally, the crack width according to EC2 can be calculated from the formula 3.1:

$$w_k = 234mm \cdot 0{,}00118 = 0{,}276mm$$

3.5.2 Calculation according to RakMK

At first, let us calculate the parameter ρ_r, which is ratio between the area of reinforcing steel and tension area of the concrete. The height of the tension area is at the distance of 7,5 times the bar diameter from the centroid of the bar. [14, p.32]

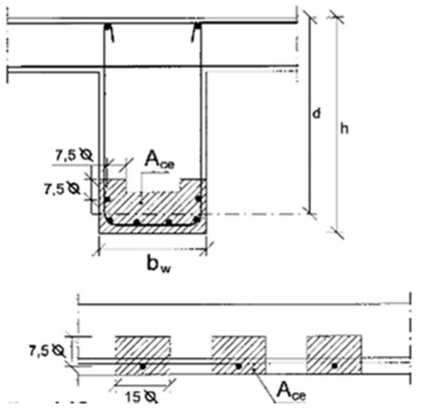


Figure 3.4. The effective tension area according to RakMK. [14, p.32]

$$\rho_r = \frac{A_s}{A_{ce}} = \frac{754\text{mm}^2}{1000\text{mm}\cdot(25\text{mm}+8\cdot12\text{mm})} = 0.00623 \qquad (3.14)\ [14, p.32]$$

The tensile stress in the bars at time when the first crack develops needs to be calculated next. The position of the neutral axis *x* was already calculated in the subsection 3.5.1, and that result can be used now. The tensile stress σ_r can be calculated from the equations:

$$M_r = 1{,}7W_{cc}f_{ctk} \qquad (3.15)\ [14, p.32]$$

$$z = d - (\tfrac{1}{3})x \qquad (3.16)\ [16, p.163]$$

$$\sigma_{sr} = \frac{M_r}{z\cdot A_s} \qquad (3.17)\ [14, p.32]$$

$$\sigma_{sr} = \frac{1{,}7\cdot\left(\frac{1000mm\cdot(200mm)^2}{6}\right)\cdot 3{,}2MPa}{\left(169mm - \left(\frac{1}{3}\right)\cdot 34{,}5mm\right)\cdot 754mm^2} = 179{,}6MPa$$

Now, the average steel strain and the final maximum crack width can be calculated from the equations 3.5 and 3.6:

$$\mathcal{E}_s = \frac{337MPa}{200000\text{MPa}}\left[1 - \frac{1}{25\cdot 0{,}085}\left(\frac{179{,}6\text{MPa}}{337\text{MPa}}\right)^2\right] = 0{,}00146$$

$$w_k = 0{,}00126\left(3{,}5\cdot 25mm + 0{,}085\cdot\frac{12mm}{0{,}00623}\right) = 0{,}367mm$$

3.5.3 Calculation according to DIN 1045

The calculation according to DIN begins with calculating the effective tension area. The position of the neutral axis can, again, be utilized here.

$$h_{eff} = min\left(2{,}5(h-d), \frac{h-x}{2}\right) \qquad (3.18)\ [15, p.133]$$

$$\mathrm{h_{eff}} = \min\left(2{,}5(200\mathrm{mm} - 169\mathrm{mm}), \frac{200\mathrm{mm} - 34{,}5\mathrm{mm}}{2}\right) = 77{,}5\mathrm{mm}$$

$$\mathrm{A_{c,eff}} = 1000\mathrm{mm} \cdot 77{,}5\mathrm{mm} = 77500\mathrm{mm}^2$$

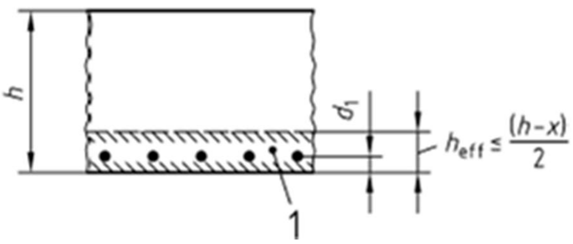


Figure 3.5. The effective tension area according to DIN. [15, p.133]

$$eff_p = \frac{A_s}{A_{c,eff}} = \frac{754mm^2}{77500mm^2} = 0.00973 \qquad (3.19)\ [15, equation\ 133]$$

The effective tensile strength of the concrete needed for the expression 3.8 and 3.9 in this case depends on the age and the class of the concrete. Now, the concrete is at an age older than 28 days so the average tensile strength f_{ctm} is used as the effective tensile strength. However, f_{ctm} for the concrete C35/45 is 3,2MPa which is greater than the minimum limit 3MPa. Therefore, 3,2MPa is the final value given to the effective tensile strength used in the formulas. [15, p.128]

Now, the maximum crack spacing $s_{r,max}$ and the average strain difference $\mathcal{E}_{\mathrm{sm}}$ - $\mathcal{E}_{\mathrm{cm}}$ can be calculated from the expression 3.8 and 3.9:

$$s_{r,max} = \frac{12mm}{3{,}6 \cdot 0{,}00973} < \frac{337MPa \cdot 12mm}{3{,}6 \cdot 3{,}2MPa} = 342{,}6mm$$

$$\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}} = \frac{337\mathrm{MPa} - 0{,}4\frac{3{,}2\mathrm{MPa}}{0.00973} \cdot (1 + 5{,}869 \cdot 0{,}00973)}{200000\mathrm{MPa}} > 0{,}6\frac{337\mathrm{MPa}}{200000\mathrm{MPa}}$$

$$\varepsilon_{sm} - \varepsilon_{cm} = 0{,}001$$

As a final result, the numerical value of the crack width for the simple case example according to the DIN 1045 can be obtained from the expression 3.7:

$$w_k = 342{,}6mm \cdot 0{,}00101 = 0{,}346mm$$

3.5.4 Calculation according to ACI 318

The calculation of the maximum crack width according to ACI 318 starts with calculating the parameter A. That corresponds to the effective tension area calculated above.

The area of concrete symmetric with reinforcing steel divided by number of bars can be calculated as follows:

$$A = \frac{2 \cdot d_c \cdot b}{n} = \frac{2 \cdot 1{,}22in \cdot 39{,}37in}{\frac{39{,}37in}{5{,}869in}} = 14{,}3in^2 \qquad (3.20)\ [5,\ p.18]$$

, in which
d_c = concrete cover from bottom of the slab to center of bar (in)
b = width of a slab
n = amount of bars

Next, the parameter β, which is ratio of distance between neutral axis and tension face to distance between neutral axis and reinforcing steel, is needed. The position of neutral axis is already calculated and equals 1,36 inches. Ratio β is then:

$$\beta = \frac{7{,}874\text{in} - 1{,}36\text{in}}{7{,}874\text{in} - 1{,}22\text{in} - 1{,}36\text{in}} = 1{,}23$$

As tensile stress 337MPa is converted to kilopound per square inch, the crack width in inches can be calculated from the equation 3.10:

$$f_s = 337MPa \cdot \frac{1{,}450377}{100} \frac{ksi}{MPa} = 48{,}877ksi$$

$$\omega = 0{,}076 \cdot 1{,}23 \cdot 48877 \sqrt[3]{1{,}22\text{in} \cdot 14{,}3\text{in}^2} = 0{,}0119\text{in}\ (0{,}304\text{mm})$$

3.6 Summary of the simple case results

Let us consider the results of the calculation of the maximum crack width next. In normal cases, the smallest crack width values seem to be obtained by the calculation according to the Eurocode, as seen in the Table 3.1. To confirm that conjecture, three additional tables are added in an effort to achieve information about the influence of the steel stress, concrete cover, bar diameter and bar spacing on the crack width in the structure of the simple case.

Table 3.1. The results of the simple case calculations.

	EC2	RakMK	DIN 1045	ACI 318
Crack width (mm)	0.276	0.367	0.346	0.304
Difference compared to EC2	0 %	33 %	25 %	10 %

In the table 3.2, the crack widths are calculated in different steel stresses. As seen, lowering the stress retains the values by Eurocode as the smallest one. The effect of lowering the steel stress seems to effect on the crack width by RakMK almost as intensive as in case of Eurocode. The greatest influence of the steel stress on the crack width is in case of calculation by DIN. That can be explained when taking a look at the formulas of DIN. Both the equation of the maximum crack spacing and the equation of the average strain difference include the parameter of the steel stress. Lowering the steel stress decreases the maximum crack spacing directly, which leads to smaller crack widths.

Table 3.2. Crack widths in the simple case with different steel stresses.

Steel stress	**Crack width**			
	EC2	**RakMK**	**DIN 1045**	**ACI 318**
337 MPa	0,276	0,367 (33 %)	0,346 (25 %)	0,304 (10 %)
300 MPa	0,233	0,313 (34 %)	0,281 (21 %)	0,269 (15 %)
260 MPa	0,186	0,253 (36 %)	0,211 (14 %)	0,233 (25 %)
220 MPa	0,155	0,190 (23 %)	0,151 (-2 %)	0,197 (27 %)

The thickness of the concrete cover effects on the crack width quite similar in cases of Eurocode, RakMK and DIN. In the results by ACI, there are seen more intensive growth in crack width compared to the other results. This can be explained by the fact that the concrete cover is indirectly involved in two parameters in the formula of the crack width.

Table 3.3. Crack widths in the simple case with different cover thicknesses.

Cover thickness	**Crack width**			
	EC2	**RakMK**	**DIN 1045**	**ACI 318**
20 mm	0,256	0,334 (31 %)	0,315 (23 %)	0,260 (2 %)
25 mm	0,276	0,367 (33 %)	0,346 (25 %)	0,304 (10 %)
30 mm	0,296	0,398 (34 %)	0,355 (20 %)	0,348 (17 %)
35 mm	0,317	0,428 (35 %)	0,355 (12 %)	0,394 (25 %)

When taking a look at the effect of bar diameter and crack spacing on the crack width, shown in the table 3.4, it can be seen generally that both the bar diameter and the bar spacing are increasing the crack width regardless of the calculation method. However, it is notable that crack width results by Eurocode are growing to a lesser extent than the results calculated by the other methods.

Table 3.4. Crack widths in the simple case with different bar diameters.

Bar diameter and spacing	**Crack width**			
	EC2	**RakMK**	**DIN 1045**	**ACI 318**
10-100 mm (A_s = 785mm^2/m)	0,245	0,298 (22%)	0,275 (12%)	0,258 (5%)
12-150 mm (A_s = 754mm^2/m)	0,276	0,367 (33%)	0,346 (25%)	0,304 (10%)
16-250 mm (A_s = 804mm^2/m)	0,328	0,497 (52 %)	0,460 (40 %)	0,381 (16 %)

Summary of the results shows the fact that the lowest crack width values are obtained by Eurocode. However, comparing of the results is not so evident due to the location of the calculated crack width, which is explained next.

It is unclear, which value the calculated crack width according to EC2 is presenting. The basis of the crack width equations is similar to that of the Model Code 1990, which gives rise to assumption that the calculated value signifies crack width either on the surface or on the centroid of the reinforcing bar because it is not calibrated in any way to be transferred into the outermost tension face of a structure. In order to maintain equivalence to Finland's National Building Code calculated crack width according to Eurocode must be transferred into concrete surface linearly by scaling. [12, p.362]

$$w_{kt} = r_t w_{ks} = \frac{h-x}{d-x} w_{ks} \qquad (3.21)\ [12,\ p.\ 362]$$

In order to compare results between Eurocode and Finland's National Building Code the crack width value from the simple case example is applied to the formula. Then the crack width at tension face of the structure is obtained from the equation 3.21:

$$w_{kt} = \frac{200mm - 34{,}5mm}{169mm - 34{,}5mm} \cdot 0{,}276mm = 0{,}340mm$$

When comparing the calculated result above to the crack width result according to RakMK (0.367mm) it can be noticed that values are fairly close to each other.

3.7 Sample calculation of crack width in a special case

In this section, the calculation of the maximum crack width is performed to a special case of a reinforced concrete structure according to each method. An example structure is 2000mm thick and heavily reinforced concrete wall that demonstrates the wall of a containment structure enclosing a nuclear reactor. As a simplification we will consider it as a beam of a width of 1000mm. Reinforcement is embedded into four layers, in which each consists of two orthogonally set 40mm diameter bars with bar spacing 300mm. Spacing of two parallel bars in different layers is 100mm. A principle sketch of the cross section is shown in the Figure 3.6, in which tension bars are shown at the plane of the figure. The thickness of the concrete cover *c* used in the calculations is now 100mm. Concrete class is C35/45, and reinforcing steel B500C1. The structure is subjected to pure bending.

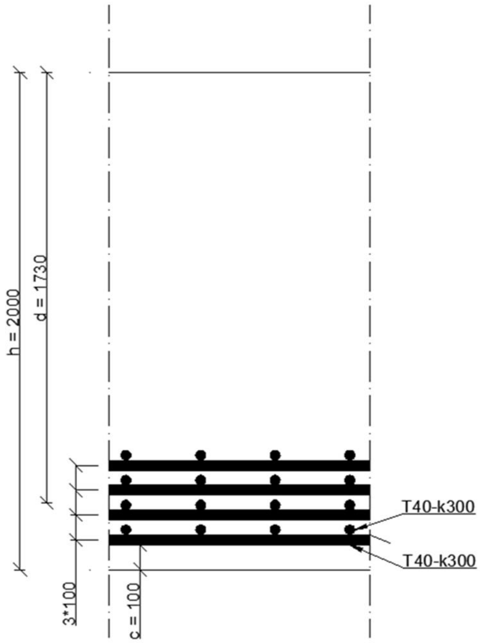


Figure 3.6. Cross section of the special case structure.

Next, we are calculating the cracking moment for this cross-section in order to evaluate steel stress caused by pure bending in case cracking occurs. The cracking moment M_{cr} can be evaluated from the geometry with assuming the effective tensile strength $f_{ct,eff}$ as f_{ctm} (=3,2MPa for concrete C35/45):

$$M_{cr} = f_{ct,eff}\frac{bh^2}{6} = 3{,}2MPa \cdot \frac{1000mm \cdot (2000mm)^2}{6} = 2133{,}3kNm$$

When the cracking moment is evaluated, we can assume in the calculations the structure to be subjected to bending moment of 8000kNm in order to ensure the cracking can occur.

3.7.1 Calculation according to EC2

The progress of the crack width calculation is similar to that of the simple case. It starts with calculating the position of the neutral axis. At first, the following values need to be calculated:

$$\alpha = \frac{E_s}{E_c} = \frac{200000MPa}{22 \cdot \left(\frac{35+8}{10}\right)^{0.3} MPa} = 5{,}869$$

$$\rho = \frac{A_s}{d \cdot b} = \frac{16755mm^2}{1730mm \cdot 1000mm} = 0{,}00969$$

$$\rho\alpha = 5{,}869 \cdot 0{,}00969 = 0{,}0568$$

$$\frac{\varepsilon_c}{\varepsilon_s} = 0{,}0568 + \sqrt{0{,}0568(2 + 0{,}0568)} = \ 0{,}399$$

Now, the position of the neutral axis is:

$$x = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s} \cdot d = \frac{\varepsilon_c/_{\varepsilon_s}}{\varepsilon_c/_{\varepsilon_s} + 1} \cdot \mathrm{d} = \frac{0{,}399}{1{,}399} \cdot 1730mm = 493mm$$

After that, the height of the effective tension area is:

$$h_{c,ef} = \min\left(2{,}5(2000 - 1730); \frac{2000 - 493}{3}; \frac{2000}{2}\right) = 502\mathrm{mm}$$

The effective tension area $A_{c,eff}$ is then:

$$A_{c,eff} = 502267\mathrm{mm}^2$$

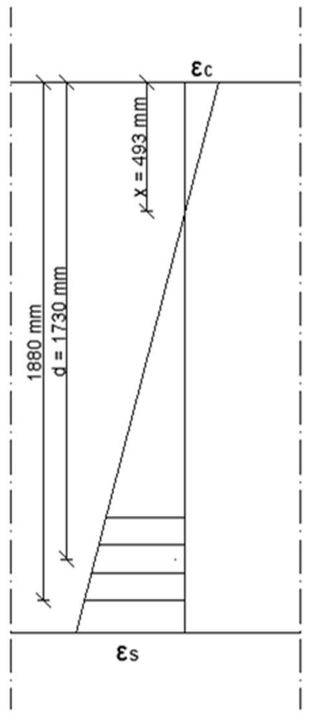


Figure 3.7 Steel and concrete strains and position of the neutral axis.

Now that we know the position of the neutral axis, we can evaluate the steel stress caused by the chosen bending moment M (=8000kNm) which is greater than cracking moment. The average steel stress in the bars equals:

$$\sigma_s = \frac{M}{A_s(d - {}^{x}/_{3})} = \frac{8000kNm}{16755mm^2 \cdot (1730mm - {}^{493mm}/_{3})} = 305MPa$$

The maximum steel stress in the bar layer closest to concrete surface in the tension area is obtained from the figure 3.7 due to geometry when the average stress in the bars equals 305MPa:

$$\sigma_s = 305MPa \cdot \frac{(1880mm - 493mm)}{(1730mm - 493mm)} = 342MPa$$

Prior to calculate the maximum crack spacing, $\rho_{p,eff}$ is calculated from the expression 3.13:

$$\rho_{\mathrm{p,eff}} = \frac{16755\mathrm{mm}^2}{502267\mathrm{mm}^2} = 0{,}0334$$

Now, the maximum crack spacing can be evaluated from the expression (3.2). The factors used in the equation are:

k_1 = 0,8 (bars are assumed to have good bond characteristics)
k_2 = 0,5 (in case of bending)
k_3 = 3,4 (recommended value)
k_4 = 0,425 (recommended value)

$$s_{r,max} = 3{,}4 \cdot 100mm + 0{,}8 \cdot 0{,}5 \cdot 0{,}425 \frac{40mm}{0{,}0334} = 523mm$$

When the value 0,4 is given to the factor k_t due to long term loading, the difference between the main strains of steel and concrete can be calculated from the expression 3.4:

$$\mathcal{E}_{\mathrm{sm}} - \mathcal{E}_{\mathrm{cm}} = \frac{342\mathrm{MPa} - 0{,}4 \cdot \frac{3{,}2\mathrm{MPa}}{0{,}0334}(1 + 5{,}869 \cdot 0{,}0334)}{200000\mathrm{MPa}} = 0{,}00148$$

Finally, the crack width can be calculated according to the expression 3.1:

$$w_k = 523mm \cdot 0{,}00148 = 0{,}775mm$$

3.7.2 Calculation according to RakMK

At first, the ratio between the area of reinforcing steel and tension area of the concrete is calculated. The height of the area reaches the level, which is at the distance of 7.5 times the bar diameter from the innermost bar.

$$\mathrm{A_{ce}} = 1000\mathrm{mm} \cdot \left(100\mathrm{mm} + \frac{40\mathrm{mm}}{2} + 3 \cdot 100\mathrm{mm} + 7{,}5 \cdot 40\mathrm{mm}\right) = 720000\mathrm{mm}^2$$

$$\rho_\mathrm{r} = \frac{16755\mathrm{mm}^2}{720000\mathrm{mm}^2} = 0{,}023$$

The tensile stress in the bars at time of the first crack development needs to be calculated next. The position of the neutral axis was already calculated in subsection 3.7.1. The tensile stress σ_{sr} can be calculated from the expression 3.17:

$$\sigma_{\mathrm{sr}} = \frac{1{,}7 \cdot \left(\frac{1000\mathrm{mm} \cdot (2000\mathrm{mm})^2}{6}\right) \cdot 2{,}2\mathrm{MPa}}{\left(1730\mathrm{mm} - \left(\frac{1}{3}\right) \cdot 493\mathrm{mm}\right) \cdot 16755\mathrm{mm}^2} = 95{,}0\mathrm{MPa}$$

Now, the average steel strain and the final crack width can be calculated from the expression 3.5 and 3.6:

$$\mathcal{E}_s = \frac{342MPa}{200000MPa}\left[1 - \frac{1}{25 \cdot 0{,}085}\left(\frac{95{,}0MPa}{342MPa}\right)^2\right] = 0{,}00165$$

$$w_k = 0{,}00165\left(3{,}5 \cdot 100mm + 0{,}085 \cdot \frac{40mm}{0{,}023}\right) = 0{,}818mm$$

3.7.3 Calculation according to DIN 1045

The calculation according to DIN begins, again, with calculating the effective tension area from the equation 3.14:

$$h_{eff} = min\left(2{,}5(2000mm - 1730mm), \frac{2000mm - 493mm}{2}\right) = 675mm$$

$A_{ceff} = 1000mm \cdot 675mm = 675000mm^2$

After that, the reinforcement level can be evaluated from the expression 3.15:

$$eff_Q = \frac{16755mm^2}{675000mm^2} = 0{,}0248$$

Now, the maximum crack spacing $s_{r,max}$ and the average strain difference ε_{sm} - ε_{cm} can be obtained from the expression 3.8 and 3.9:

$$s_{r,max} = \frac{40mm}{3{,}6 \cdot 0{,}0248} < \frac{342MPa \cdot 40mm}{3{,}6 \cdot 3{,}2MPa} = 447{,}6mm$$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{342MPa - 0{,}4\frac{3{,}2MPa}{0{,}0248} \cdot (1 + 5{,}869 \cdot 0{,}0248)}{200000MPa} < 0{,}6\frac{342MPa}{200000MPa}$$

$$\varepsilon_{sm} - \varepsilon_{cm} = 0{,}00142$$

As a final result, the crack width for this special case according to the DIN 1045 can be evaluated from the equation 3.7:

$$w_k = 447{,}6mm \cdot 0{,}00142 = 0{,}633mm$$

3.7.4 Calculation according to ACI 318

Lastly, the maximum crack width in special case is calculated by ACI 318. Tension area is calculated from the equation 3.16 at first:

$$A = \frac{2 \cdot d_c \cdot b}{n} = \frac{2 \cdot (78{,}74 - 68{,}5)in \cdot 39{,}37in}{\frac{39{,}37in}{11{,}81in} \cdot 4} = 60{,}45in^2$$

When taking into account the previously calculated position of the neutral axis 19.3in, parameter β gets the value of:

$$\beta = \frac{78,74\text{in} - 19,42\text{in}}{68,1\text{in} - 19,42\text{in}} = 1,218$$

When the tensile stress in the bars is assumed to be 342MPa that equals 49602 pounds per square inch, the crack width in inches can be calculated from the equation 10:

$$\omega = 0,076 \cdot 1,218 \cdot 49603\sqrt[3]{(78,74 - 68,1) \cdot 62,78} = 0,0401\text{in}\ (1,027\text{mm})$$

3.8 Summary of the special case results

The results of the crack widths calculated for the special case are summarized in this section in order to obtain information about crack widths in uncommon cases.

Table 3.5. Crack width results in the special case.

	EC2	**RakMK**	**DIN**	**ACI**
Crack width (mm)	0,775	0,818	0,633	1,027
Difference compared to EC2	0 %	5 %	-18 %	33 %

As it can be noticed the maximum crack widths are way out of the limiting values set for the maximum crack width 0,2mm, 0,3mm or 0,4mm, depending on the exposure class. Calculation by DIN seems to result the lowest crack width values. Value according to EC2, almost, equals to that of the RakMK, which is notable. As stated in previous section 3.5, in case of normal structure crack width values calculated by EC2 are about 30% lesser than values according to RakMK. The effect of steel stress and cover thickness is explored by performing two different tables, in which the mentioned parameters are varied.

Table 3.6. Crack widths in the special case with different steel stresses.

The maximum steel stress	**Crack width**			
	EC2	**RakMK**	**DIN 1045**	**ACI 318**
342 MPa	0,775	0,818 (5 %)	0,633 (-18 %)	1,027 (33 %)
300 MPa	0,665	0,709 (7 %)	0,539 (-19 %)	0,901 (32 %)
260 MPa	0,506	0,542 (7 %)	0,387 (-24 %)	0,691 (37 %)
220 MPa	0,409	0,447 (9 %)	0,307 (-25 %)	0,585 (43 %)

Table 3.7. Crack widths in the special case with different cover thickness.

Cover thickness	Crack width			
	EC2	**RakMK**	**DIN 1045**	**ACI 318**
50 mm	0,470	0,448 (-5 %)	0,453 (-4 %)	0,746 (59 %)
60 mm	0,513	0,501 (-2 %)	0,470 (-8 %)	0,773 (51 %)
70 mm	0,556	0,554 (0 %)	0,486 (-13 %)	0,801 (44 %)
80 mm	0,598	0,606 (1 %)	0,502 (-16 %)	0,829 (39 %)
90 mm	0,640	0,658 (3 %)	0,517 (-19 %)	0,857 (34 %)
100 mm	0,775	0,818 (5 %)	0,633 (-18 %)	1,027 (33 %)

3.9 Effect of surface mesh on the calculated crack width

It is recommended to use a reinforcement mesh close to the concrete surface in such cases, where the concrete cover is very thick and crack widths tend to exceed the values of limiting calculated crack widths w_{max}. The surface mesh shall be designed so that it reduces the maximum crack spacing which leads to acceptable crack widths.

Let us next consider the crack width calculation according to Eurocode in the special case example by adding the surface mesh in the structure in order to achieve information about the decreasing effect on the calculated crack widths in case of thick concrete cover of main bars. The purpose is that surface mesh is only taken into consideration at the serviceability limit state when calculating the maximum crack width. Tensile stress in the bars is assumed to be obtained by extrapolation from the stresses in the main bars. In purpose to find out the influence of surface mesh on crack widths we add the following surface mesh into our special case. Reinforcing bars in the surface mesh are 12mm of a diameter with spacing of 150mm in both perpendicular directions. The mesh is embedded into surface of concrete with nominal concrete cover of 30mm.

Firstly, the tensile stress in the surface mesh is calculated with assuming the maximum tensile stress in the main bar as 342MPa, according to the example in section 3.7. Since the increase of the total reinforcing steel after adding the mesh is slight, some simplifications can be done. The graph below (Figure 3.7), which is based on the graph in the

Figure 3.1, shows now the average steel and concrete strains. According to the geometry in the graph, the average stress in the mesh can be solved by extrapolation:

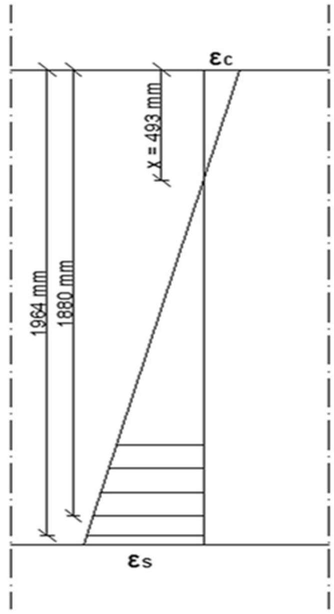


Figure 3.7. Strain graph for the surface reinforcement.

$$\sigma_{mesh} = \frac{342MPa \cdot (1964mm - 493mm)}{1880mm - 493mm} = 353MPa$$

Despite the fact that the total amount of reinforcement steel has increased slightly due to adding of the surface mesh, the values of the effective tension area $A_{c,eff}$ and the effective reinforcement ratio $\rho_{p,eff}$ needed for the calculations can be taken directly from the example in the subsection 3.7.1 without making a significant mistake in terms of final crack width result.

When the reinforcement in cross section consists of two differently sized reinforcing bar layers, a formula for calculating the equivalent bar diameter is presented in the Eurocode. According to the definition, the equivalent bar diameter ϕ_{eq} for a cross section, is obtained from the equation 3.22, in which the amount of ϕ_1 diameter bars is n_1 and the amount of ϕ_2 diameter bars is n_2: [13, p.124]

$$\phi_{eq} = \frac{n_1\phi_1{}^2 + n_2\phi_2{}^2}{n_1\phi_1 + n_2\phi_2} \quad (3.22)\ [13, \text{expression } 7.12]$$

$$\phi_{eq} = \frac{\frac{4 \cdot 1000mm}{300mm} \cdot (40mm)^2 + \frac{1000}{150} \cdot (12mm)^2}{\frac{4 \cdot 1000mm}{300mm} \cdot 40mm + \frac{1000}{150} \cdot 12mm} = 36{,}3mm$$

The maximum crack spacing is now obtained from the expression 3.2 when the cover thickness c is 30mm and factors k_1-k_4 are taken from the example in the subsection 3.7.1 and the average strain difference can be solved from the equation 4:

$$s_{r,max} = 3,4 \cdot 30mm + 0,8 \cdot 0,5 \cdot 0,425 \cdot \frac{36,3mm}{0,0334} = 286mm$$

$$\mathcal{E}_{sm} - \mathcal{E}_{cm} = \frac{353MPa - 0,4\frac{3,2MPa}{0,0334}(1 + 5,869 \cdot 0,0334)}{E_s} = 0,0015$$

Finally, the maximum calculated crack width for the special case example after adding the surface mesh can be evaluated:

$$\mathrm{w_k} = 286\mathrm{mm} \cdot 0,0015 = 0,439\mathrm{mm}$$

As can be seen the adding of surface reinforcement in case of thick concrete cover decreases the calculated crack width closer to allowable value. By trial and error such surface mesh can be found which leads to acceptable crack width. In our special case, reducing the calculated crack width by adding the amount of main reinforcement would be in most similar cases impossible, and, if it was possible, it would be much more expensive than reducing the crack width by adding a suitable surface mesh.

4 Experimental studies of crack width

After the study and calculations of crack width it is essential to achieve information about the reliability of not only the Eurocode but also the other presented codes. The aim is to search for experimental data of crack width tests in the literature and try to study the relationship of measured and calculated crack widths according to different codes. At first, the comparison between measured and calculated values is performed in cases that can be considered as normal ones. However, the main focus is put on the special cases. This means to obtain answers to the question, how reliable the Eurocode is in cases of a thick concrete cover and heavy reinforcement combined with massive cross-section, as in the wall of a containment structure of nuclear reactor for example.

4.1 Experimental study of cracking in normal case

In order to study the accuracy of the crack width calculation in normal cases according to Eurocode experimental research by Marzouk et al. [21, pp. 282-287] is presented and analyzed next. Their research was focused on evaluating the crack widths and crack properties of thick two-way reinforcement slabs and plates.

4.1.1 Presentation of the study

A comprehensive investigation by Marzouk et al. consisted of experimental and analytical parts. The numerical study focused on the available code prediction models for estimating the crack width of concrete plates and the aim was to study the suitability of available crack width expressions. The experimental work included the investigation of the cracking behavior, such as examining the effect of increasing concrete cover and bar spacing on crack width properties. The crack widths were measured electronically for three series of test specimens. The test results were tabulated to compare test results with the available code expression for calculating crack widths.

4.1.2 Design codes for test calculations

Numerical calculations of crack width for the test specimens were conducted by using five different model codes: ACI 318-05, Norwegian code NS 3473 E, The Canadian offshore code CSA-S474-04, The CEB-FIP (1990) code and Eurocode 2 (BS EN 1992-1-1:2004). The American ACI 318 code and the Eurocode for calculation of crack width are presented in sections 3.1 and 3.4. Therefore, a brief look is taken at the other codes used in the numerical investigation in the research.

The Norwegian Standard NS 3473 E provides the following equation for calculating the crack width. The factor r is used to account for the tension stiffening effect.

$$w_k = 1{,}7w_m \qquad \text{(4.1) [21, p. 283, expression 5]}$$
$$w_m = r\varepsilon_1 S_{rm} \qquad \text{(4.2) [21, p. 283, expression 6]}$$

,where S_{rm} is the average crack spacing (mm); ε_1 is the average concrete tensile strain in the effective embedment zone; w_k is the maximum characteristic crack width (mm); w_m is the average crack width at the concrete surface (mm); and r is the tension stiffening modification factor. NS 3473 E and other European codes define the characteristic crack as the width that only 5% of the cracks will exceed and S_{rm} is the average crack spacing. This characteristic crack width is taken as 60 to 70% more than the average crack width. [21, p283]

The Canadian offshore code CSA-S474-04 recommends that the average crack width may be calculated as the average crack spacing times the total average tensile concrete strain after considering the contribution of the tension stiffening. Both NS 3473 E and CSA-S474-04 provide similar expressions for calculating crack spacing. CSA-S474-04 estimates the crack width at the surface of the member. However, NS 3473 E calculates the crack width at the level of steel reinforcement. CSA-S474-04 provides the following expression for calculating the crack spacing:

$$S_{rm} = 2{,}0(C + 0.1S) + k_1 k_2 d_{be} h_{ef} b / A_s \qquad \text{(4.3) [21, p. 283, expression 7]}$$

, where S_{rm} is the average crack spacing (mm); C is the concrete cover (mm); S is the bar spacing of the outer layer (mm); k_1 is the coefficient that characterizes bond properties of bars; k_2 is the coefficient to account for strain gradient; d_{be} is the bar diameter of the outer layer (mm); h_{ef} is the effective embedment thickness as the greater of $(c + d_{be}) + 7.5d_{be}$ not greater than the tension zone or half slab thickness (mm); b is the width of the section (mm); A_s is the area of reinforcement within the effective embedment thickness (mm2); and ε is the concrete tensile strain in the effective embedment zone h_{ef}.

The CEB-FIB code (1990) gives the following equation for calculating the characteristic crack width:

$$w_k = l_{s,max}(\varepsilon_{s2} - \beta\varepsilon_{sr2} - \varepsilon_{cs}) \qquad \text{(4.4) [21, p. 284, expression 10]}$$

, where w_k is the characteristic maximum crack width (mm); w_m is the average crack width (mm); ε_{s2} is the steel strain of the transformed section in which the concrete in tension is ignored; ε_{cs} is the free shrinkage of concrete, generally a negative value; ε_{sr2} is the steel strain at a crack, under a force causing stress equal to f_{ctm} within A_{cef}; and β is an empirical factor to assess average strain within $l_{s,max}$.

4.1.3 Experimental arrangements

The experimental investigation in the research by Marzouk et al. included the testing of eight reinforced two-way concrete slabs. The tested slabs were square with a side dimension of 1900mm in both directions and were simply supported along all four edges with the corners free to lift. A central load was applied on the slab through a 250x250mm column stub. Typical test specimen is shown in Figure 4.1. The slabs were cast with normal and high strength concrete of 35 MPa and 70 MPa, respectively. [21, pp. 284-285]

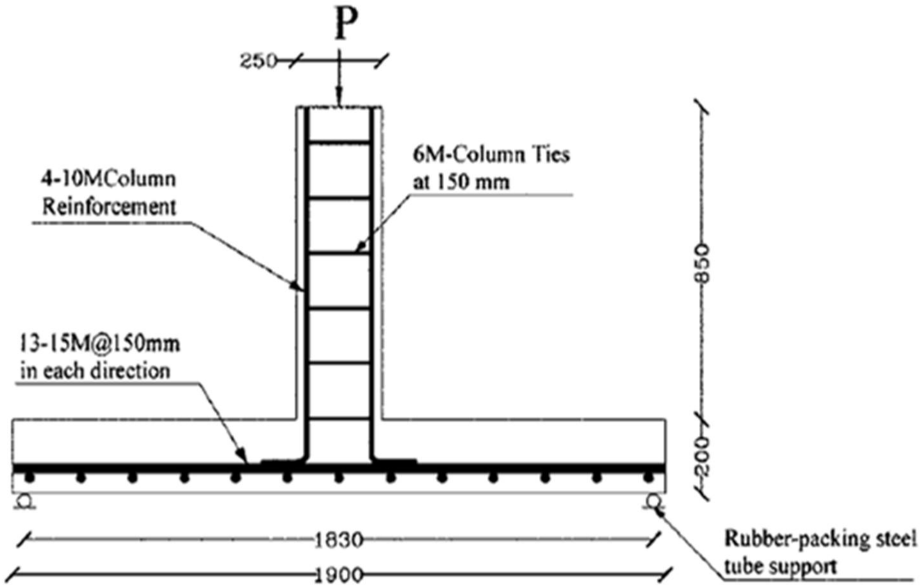


Figure 4.1. Typical details of test specimen. [21, p. 284]

The test specimens were divided into three groups. The first series included three slabs designated as Specimens NSC1, HSC1 and HSC2. Each slab had equal thickness of 200mm, same bar spacing of 150mm, and bar diameter of 25mm with different concrete covers that were 30mm, 40mm, 50mm and 60mm. Series II-Specimens HSC3 and HSC4 had the same concrete thickness of 200mm, the same concrete cover of 30mm, the same bar diameter of 25mm, and different bar spacings of 150mm, 200mm, 210, 240mm and 250mm. The first and second series were designed to represent heavily reinforced concrete walls that normally fail under the punching failure mode as is the case for most offshore structures. The third series, however, was designed to investigate the effect of pure flexural failure. The third group consisted of Specimens HSC5, NSC2, and NSC3. All tested specimens are summarized in Table 4.1. [21, pp. 285-286]

At first, the test specimens were loaded at 5 to 8% of the expected ultimate load until the first cracks initiated. Then crack displacement transducers were mounted to concrete surface cracks after which the loading was increased step by step to the failure load. Each slab was carefully inspected at each load step and the maximum visible crack width was measured. [21, pp. 285-286]

Table 4.1. Group specimens' details [21, p.285]

Series no.	Slab no.	f_c', MPa	Bar size, mm	Bar spacing, mm	Concrete cover, mm	Slab thickness, mm	Steel ratio ρ, %
Series I	NSC1	35	25M	150	30	200	2.17
	HSC1	69	25M	150	50	200	2.48
	HSC2	70	25M	150	60	200	2.68
Series II	HSC3	67	25M	200	30	200	1.67
	HSC4	61	25M	250	30	200	1.13
Series III	HSC5	70	15M	100	30	150	1.88
	NSC2	33	15M	240	30	200	0.52
	NSC3	34	10M	210	40	150	0.40

4.1.4 Test result

A large amount of test data was recorded in the investigation and the main focus was put on the crack width at the serviceability limit state. The crack widths for all test specimens at the serviceability level at a steel stress level of 250MPa are given in Table 4.2 as well as all the results of the crack width prediction equations by different codes.

Table 4.2. Comparison of test results with predictions of other international codes. [21, p.286]

Series no.	Slab no.	Experiment, mm	ACI, mm	CSA, mm	NS, mm	CEB-FIP, mm	EC2
Series I	NSC1	0.406	0.261	0.227	0.269	0.107	0.135
	HSC1	0.772	0.311	0.351	0.354	0.114	0.142
	HSC2	0.950-	0.341	0.438	0.397	0.115	0.143
Series II	HSC3	0.486	0.329	0.252	0.314	0.143	0.160
	HSC4	0.483	0.399	0.287	0.361	0.174	0.183
Series III	HSC5	0.327	0.258	0.248	0.294	0.133	0.165
	NSC2	0.248	0.376	0.324	0.430	0.236	0.249
	NSC3	—	0.348	0.425	0.473	0.185	0.268

For series I and II, Figures 4.2 to 4.5 indicate that both BS EN 1992-1-1:2004 and CEB-90 are very similar in results for maximum predicted crack width and the calculated results are only around 25% of the experimental results. The CSA-S474-04 and NS 3473 E predict closer crack widths than the BS EN 1992-1-1:2004 and CEB-90, but less than the experiment results. For series III, as shown in Table 4.2, these codes can provide a good prediction for maximum crack width for normal strength concrete specimens with small concrete covers of 30 to 50 mm cover. Therefore, it can be summarized that the EC2 and the CEB-90 codes are applicable to be used for buildings with small concrete cover rather than for infrastructure facilities like offshore and nuclear containment structures. The results of ACI 318-05, CSA-S474-04, and NS 3473 E can provide a reasonable estimate for crack widths of such structures.

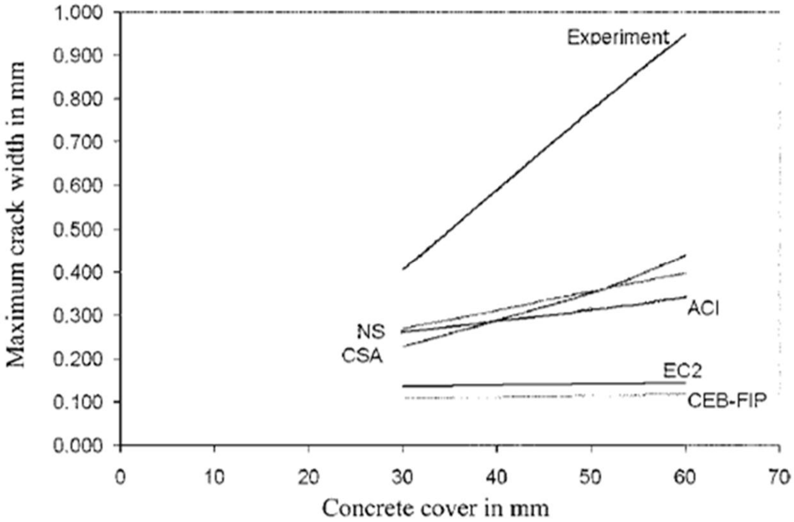


Figure 4.2. Comparison of maximum crack width for Series I. [21, p. 286]

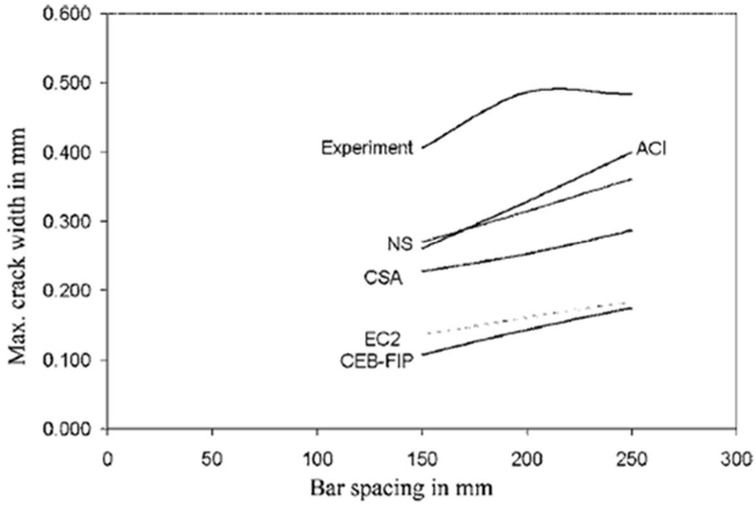


Figure 4.3. Comparison of crack width for series II. [21, p.286]

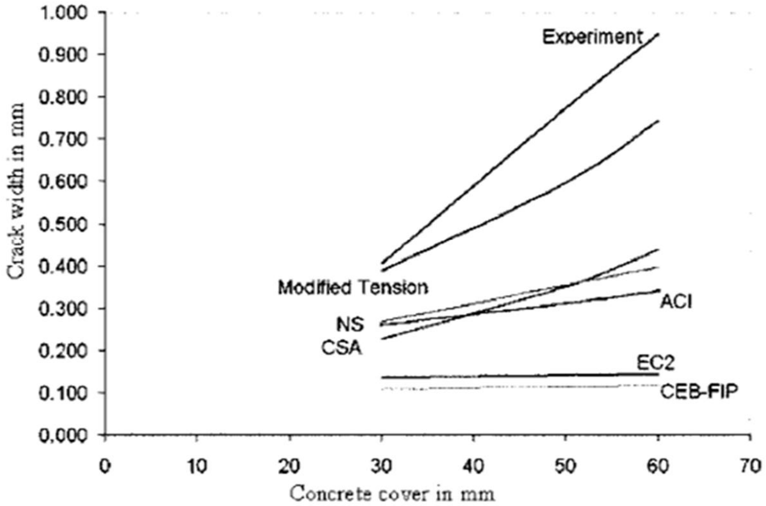


Figure 4.4. Comparison of crack width for Series I. [21, p. 287]

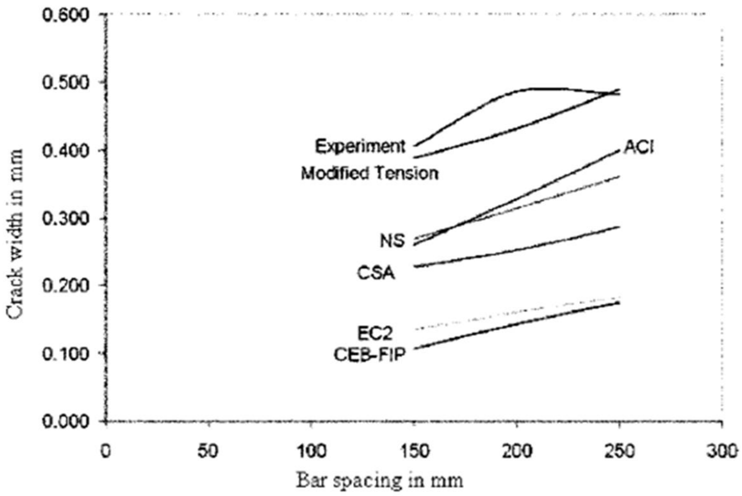


Figure 4.5. Comparison of crack width for Series II. [21, p. 287]

4.2 Experimental study of cracking in special case

The increasing use of thick concrete covers due to durability issues in offshore and nuclear containment applications has raised a universal interest for crack control in special cases and some experimental investigations are found in literature. Based on the fact that according to most crack width models increasing concrete cover results in increased crack spacing and hence increased crack width, an extensive experimental study by Rizk et al. [20, pp. 1501-1510] is presented and analyzed now.

4.2.1 Presentation of the study

The aim of the study was to assess the effect of thick concrete covers on crack width. One of the purposes of that investigation was, as well, to estimate the accuracy of design codes' models in case of thick reinforced concrete plates having thick concrete covers. [20, pp. 1501-1502]

The study consisted primarily of calculating and measuring of crack spacing and crack width for different reinforced test slabs with concrete covers up to 70mm and slab thicknesses up to 400mm. Calculations were conducted using different design codes. In addition, a new proposed crack width model was presented by the authors, and it was also tested in order to compare it with other models. [20, p. 1502]

4.2.2 Design codes for the calculations

Crack spacing and crack widths for test slabs were calculated using five different approved design codes: ACI 318-08, Canadian offshore code CSA-S474-04, Norwegian code NS 3473E, CEB-FIB (1990) model code and Eurocode EC2.

The ACI approach and the Eurocode model are already considered in the sections 3.1 and 3.4. The rest of the codes were described in the subsection (4.1.2) as well.

4.2.3 Experimental arrangements

The experimental tests were conducted to nine reinforced concrete plates by varying the concrete cover and bar spacing in the structural lab at Memorial University of Newfoundland. Overall six high strength concrete slabs (HS) and three normal strength concrete slabs (NS) ranged from 250 to 400 mm were selected for the investigation of the cracking behavior study as detailed in Table 4.3. Typical test specimen is shown in Figure 4.6. [20, pp. 1505-1506]

The first group of test specimens (Group I) consisted of five different slabs with two different slab thicknesses, 250 and 300 mm, two thick concrete covers, and three different bar sizes, 15, 20 and 25 mm. Each slab had the equal bar spacing of 368 mm. The cover thickness of the first two slabs was 67.5 mm (60 mm clear cover), and 82.5 mm (70 mm clear cover) of the rest of slabs. The purpose of this first group was to investigate the effect of concrete cover, concrete strength and corresponding change in steel ratio for the same bar spacing on the crack width. [20, p. 1506]

The second group (Group II) included four different slabs nominated as HS4, HS5, NS3 and HS6, and was designed to investigate the effect of bar spacing on the crack spacing and crack width. Slab thicknesses ranged from 350 to 400 mm, and each slab except slab HS4 had high reinforcement ratio. All the slabs had the same thick concrete cover 70 mm, but various bar diameters, 25 and 35 mm, as well as different bar spacing of 217, 289 and 368 mm. [20, p. 1506]

Table 4.3. Details of test specimens [20, p.1506]

Group no.	Slab no.[a]	Compressive Strength f_c' (MPa)	Bar size (mm)	Bar spacing (mm)	Concrete cover C_c (mm)	Slab thickness (mm)	Depth (mm)	Steel ratio (ρ%)
I	NS1	35.0	15	368	60	250	182.5	0.35
	HS1	70.0	15	368	60	250	182.5	0.35
	NS2	35.0	25	368	70	300	217.5	0.73
	HS2	64.7	25	368	70	300	217.5	0.73
	HS3	70.0	20	368	70	300	220.0	0.43
II	HS4	76.0	25	368	70	350	267.5	0.50
	HS5	65.4	35	289	70	350	262.5	1.42
	NS3	40.0	35	217	70	400	312.5	1.58
	HS6	60.0	35	217	70	400	312.5	1.58

[a] *NS* normal strength slabs, *HS* high strength slabs

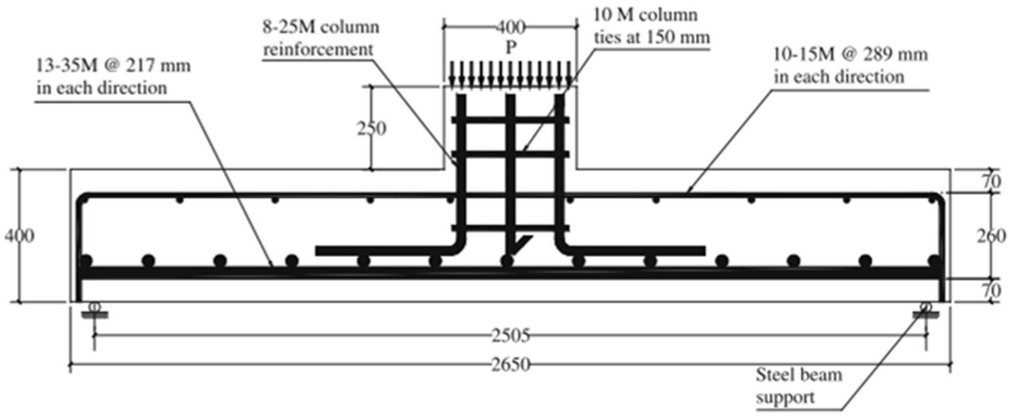


Figure 4.6. Details of typical test specimen HS6. [20, p. 1506]

4.2.4 Measuring of crack width

The measuring of crack width was a multistage process, in which the loading of the specimen was increased step by step. At first, the test specimens were loaded up to 10% of the ultimate load. Then Crack Displacement Transducers (CDT) were installed using epoxy glue on the concrete surface of the first, second and third visible cracks on the tension surface of the slab to measure the crack opening displacement. Before resuming the loading of the specimen the gauges were left for 1 hour in order to enable the epoxy to dry. After that the loading was released and reapplied using selected load increment of 44 kN. Each slab was carefully inspected and cracks were marked manually after mapping all the cracks at each load step. [20, p. 1507]

4.2.5 Test results

Crack widths were measured at each load stage at different locations on the test slab. The widths of the primary cracks were explored in the two sets of test specimens to define the effect of concrete cover, bar diameter and bar spacing on the maximum crack width measurements. The results of the measurements at a steel stress level of 267 MPa are shown in table 4.4. It can be noticed from the data that as the cover thickness was increased in the first group, the crack width increased as well. Data shows that increasing the concrete cover from 60 to 70 mm for the same bar spacing increases the maximum crack width as much as 28%. When comparing that with calculated values a 0% increase according to ACI 318-08 and EC2 is found, 3% decrease estimated according to CSA-S474-04 and NS 3474 E and a 22% increase calculated according to CEB-FIB (1990) model code. This means that for the same bar spacing, increasing the concrete cover by about 17% resulted in increasing the crack width by about 28%. [20, pp. 1508-1510]

Table 4.4. Comparison between the calculated crack width values using code formulae with the measured experimental values. [20, p. 1510]

Table 2 Comparison between the calculated crack width values using code formulae with the measured experimental values

Group no.	Slab no.[a]	Experiment S_m (mm)	Experiment w_k (mm)	Proposed eq. w_k (mm)	ACI w_c (mm)	CSA w_m (mm)	NS w_m (mm)	CEB-FIP w_k (mm)	EC2 w_k (mm)
I	NS1	245	0.465	0.724	0.470	0.638	0.649	0.413	0.661
	HS1	263	0.402	0.775	0.470	0.634	0.649	0.294	0.661
	NS2	261	0.607	0.705	0.470	0.619	0.630	0.493	0.663
	HS2	246	0.596	0.767	0.470	0.615	0.630	0.415	0.663
	HS3	247	0.590	0.935	0.470	0.689	0.705	0.326	0.745
II	HS4	221	0.581	0.927	0.470	0.663	0.679	0.431	0.740
	HS5	264	0.435	0.835	0.370	0.514	0.526	0.376	0.605
	NS3	250	0.439	0.733	0.278	0.471	0.480	0.366	0.587
	HS6	210	0.469	0.852	0.278	0.469	0.480	0.344	0.587

[a] *NS* normal strength slabs, *HS* high strength slabs

Test specimens HS2 and HS3 were similar except reinforcement ratio. Both slabs had the same bar spacing, the same concrete cover but with different bar diameters. Slab HS2 had a reinforcement ratio 0.73% with a bar diameter of 25mm while slab HS3 had a reinforcement ratio equal to 0.43% with a bar diameter of 20mm. The experimental results show that the effect of changing the bar diameter was negligible. It should be noted as well that both specimens had the same average crack spacing.

Two test specimens HS2 and HS3 of Group II were designed and specifically tested to study the effect of increasing bar spacing on crack width while keeping concrete cover constant. It can be noticed, in general, that the maximum crack width is increased as the bar spacing increases. In addition, it seems that the effect of increasing the bar spacing on the crack width is more profound than the effect of increasing the concrete cover. The data of Group II showed that for the range of bar spacing tested, the maximum crack width can be influenced by as much as 50% when the bar spacing was increased from 217 mm to 368 mm. This means that for the same concrete cover increasing the bar spacing by about 70% resulted in increasing the crack width by about 50%.

Table 4.4 shows that each code seems to neglect the effect of concrete strength on crack width. The CEB-FIB (1990) model code is the only code that takes into account the effect of concrete strength when calculating crack width. Data of group I indicate that increasing the concrete strength from 35 MPa to 70 MPa resulted in about 10-15% decrease in crack width.

Test specimens of Group II contained four thick slabs with the same 70 mm thick concrete cover. Test results show that crack control in this case can still be achieved by limiting the spacing of the reinforcing steel despite using thick concrete cover.

4.2.6 Summary of the experimental test

The experimental results in the study by Rizk et al. can be summarized in the following way. The results of the test specimens in Group I show that as the concrete cover increases, the maximum crack width increases. The data showed that the maximum crack width can be influenced by as much as 28% when the concrete cover was increased from 60 to 70 mm for the same bar spacing

The data of Group II showed that increasing the bar spacing from 217 mm to 368 mm, the maximum crack width can be influenced by as much as 50 %. This means that for the same concrete cover increasing the bar spacing by about 70% results in increasing the crack width by about 50%. The analytical investigation revealed that the crack widths calculated using CSA-S474-04 and NS 3473 E (1989) were relatively close. EC2 seems to overestimate the crack width about 25 to 65 % in case of thick concrete cover.

4.3 Comparison between experimental results with calculated values in special case

As can be seen from the experimental test in case of thick concrete cover EC2 tends to overestimate both the rate of crack width increase as well as the value of crack width the more the cover thickness increases. From that point of view it may be assumed that calculated values in the special case example are 25% to 65% higher than the crack width in reality. That means for calculated value for 100mm cover thickness real crack width would be in the range of 0.376mm and 0.5mm for steel stress of 342MPa. That conjecture is also supported by the experimental test conducted to concrete beams varying the cover thickness, which showed that doubling the cover thickness induces only a slight increase of crack width in real life. That can be seen in the Figure 4.7 below. [24, pp. 257-265]

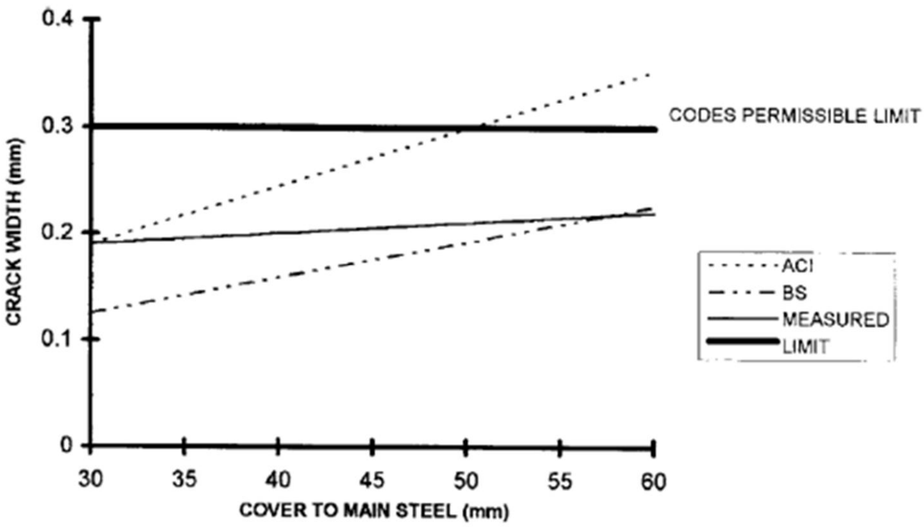


Figure 4.7. Variation of service load crack width with increasing concrete cover for tested concrete beam. [24, p. 261]

5 Practical difficulties in the interpretation of crack calculation according to EC2

The interpretation of rules of crack width calculation according to Eurocode is more or less unclear in some special cases, which may be the factor resulting into too large crack widths in cases of thick concrete cover or heavy reinforcement with large diameter bars in multiple rows. The main problems with the EC2 calculation method are discussed in this chapter in order to achieve information and clarification about the reliability and feasibility of the present method for the special cases.

5.1 Tension stiffness

The cracking strain in the crack width calculation, expression 3.4, is based on the tension stiffness model, which is a rectangular tension stress block of height marked as $h_{c,ef}$ in the equation, with a constant stress of $k_t f_{ctm}$ ($=0.4f_{ctm}$). It is proven to give accurate results for pure tension situations and deep bending elements with small concrete covers and high tension strains. However, in cases of general bending additional interpretation is required as this stress block and resulting tension stiffness strain remain constant regardless of loading. Experimental studies of real sections have demonstrated that present tension stiffness model is not always applicable. [23, p.3] [24, pp. 141-143]

5.1.1 More accurate tension stiffness model

The calculation of deflection in EC2 is based on a triangular stress block, which can be applied to predict the strain difference ε_{sm} - ε_{cm} as a proportion of the fully cracked strain plane. Then the strain difference can be estimated as:

$$(\varepsilon_{sm} - \varepsilon_{cm}) = \zeta \varepsilon_{cr} \qquad (5.1)\ [23,\ p.\ 3]$$

, in which the ratio ζ varies with load and can take account of previous higher or early age loading. This tension stiffness model results in an instant jump on cracking from uncracked section stiffness to 0,5 of the cracked model followed by a quick drop to mobilize ~0,8 of the fully cracked result and only 0,2 of the uncracked result. This type of tension stiffness model is roughly equivalent to a triangular concrete tension stress block with an extreme fibre stress of 0,7MPa for typical loads and C40 concrete. The model has also been tested against experimental results with success. [23, p. 3]

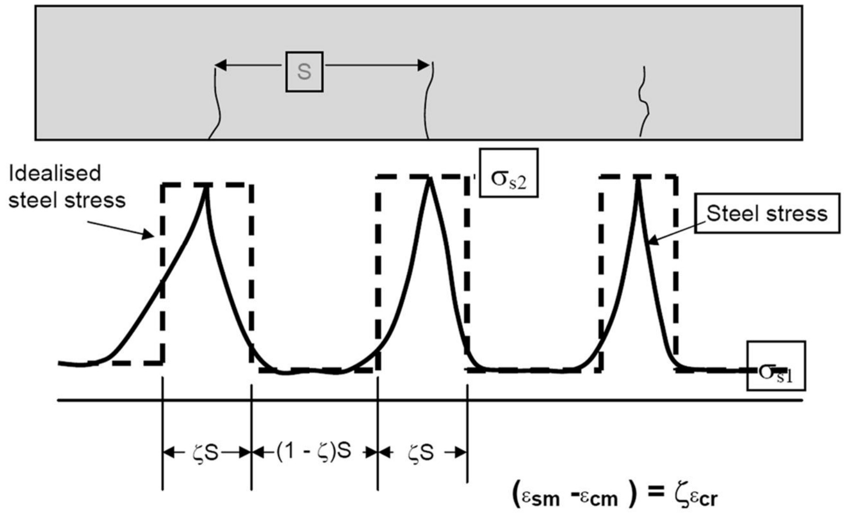


Figure 5.1. Distribution of steel stress at and between cracks. [23, p. 4]

5.1.2 Local tension stiffness model according to EC2 and flexure

When using the expression of strain difference for flexure there seem to be difficulties associated with the interpretation of $\rho_{p,eff}$ and k_t. First of all increasing $\rho_{p,eff}$ means that tension stiffness decreases, which increases crack width. However, increasing $\rho_{p,eff}$ causes decrease on crack spacing as well. These influences are not taken into consideration resulting in the crack width formula to be insensitive to bar layout, which is more or less against current understanding. [23, p.5]

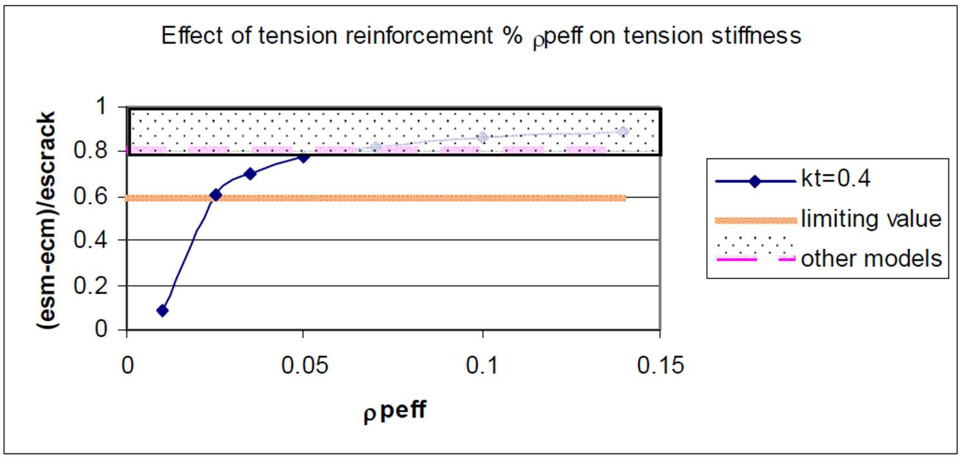


Figure 5.2. Effect of $\rho_{p,eff}$ on tension stiffness. [23, p. 5]

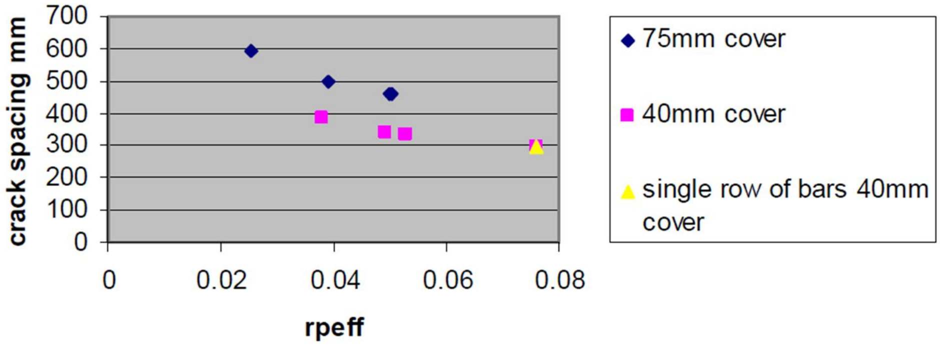


Figure 5.3. Effect of $\rho_{p,eff}$ on crack spacing. [23, p. 6]

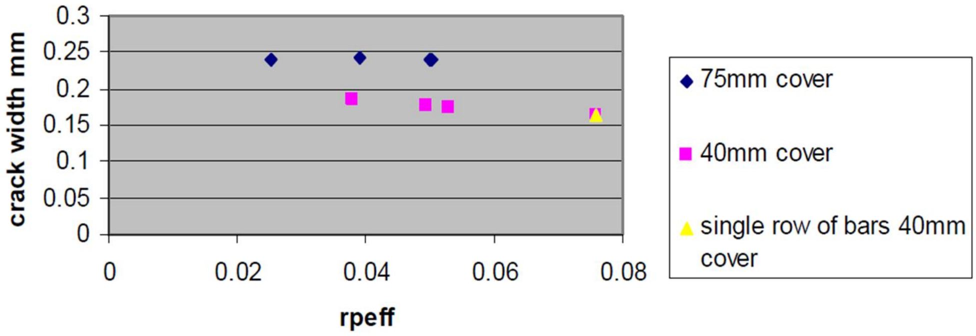


Figure 5.4. Effect on $\rho_{p,eff}$ on crack width. [23, p. 6]

The results above can be explained with an example of a section with a fixed bar depth *d*, where cover is increased by adding depth to the tension zone while retaining the depth of steel from the compression face. Tension stiffness model in expression 3.4 will reduce the crack strain while the cover increases up to a limiting value of *(h-x)/3*. This indicates that the bar extends its effective zone over the concrete both internally and externally as the cover increases. The change in calculated tension stiffening will be nearly relative to the change in cover.

Tension stiffness model presented in subsection 5.1.1 and models in UK codes as well will not submit such a marked effect. The tension stiffening will be a function of the entire depth of the tension zone *(h-x)*. The cover is then only a small proportion of this. Although stiffening model according to Eurocode is more suited to direct tension, rather than flexural cracking it presents a simple model that is attractive to hand calculation and will submit reasonable results for deep bending elements with small covers and high tension strains. The Eurocode-model fits other tension stiffening models better if the constraint k_t in Expression 3.4 is reduced from 0,4 to 0,3 or 0,2. [23, p.6]

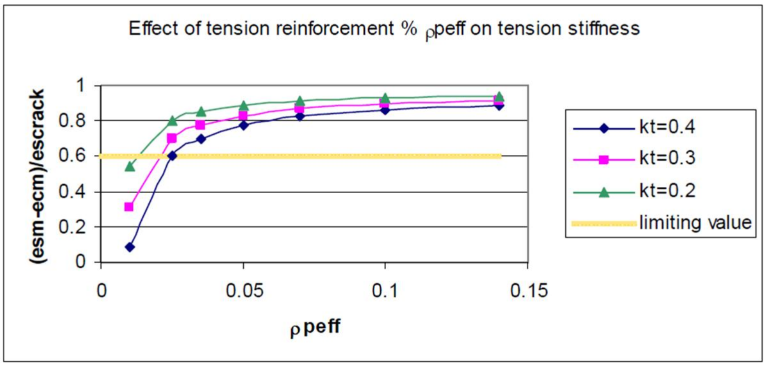


Figure 5.5. Effect of reducing k_t. [23, p.7]

The unsuitability of the Expression 3.4 to general flexural situations was known at the time of compiling the code and that is why limits of *(h-x)/3* and *2,5(h-d)* were determined on the effective height of the local tension block h_{ceff}. Both these limits induce problems of interpretation due to lack of clarity in the code. The *(h-x)/3* limit has been noticed to dominate in many practical calculations and is discussed more specifically later in section 5.3. A simple example of problems which occur is a cross-section with reinforcement distributed up the side of the section. Some interpretations which meet the code can induce large areas of unreinforced concrete contributing to the tension stiffness at a high constant stress of $0,4f_{ctm}$. As well problem is encountered when there are no local bars to control the cracking as clearly it is not possible to define ε_{sm}. An alternative tension stiffening model is needed for this situation. [23, pp.7-9]

5.2 Cracking strain and crack width position

The Eurocode does not clearly name the position for calculation of crack strain and crack width. Although ε_{sm} is clearly at the bar it could be more logical to consider ε_{cm} at the surface of the concrete. While the formulae may include adjustments to compensate, the crack width and crack strain position do not necessarily need to be identical. If tension stiffness is to be calculated another way than according to Eurocode the code needs to be clarified with regard to position for crack strain (ε_{sm}- ε_{cm}). In case of durability issue the crack width in the bar location may be the main concern, whilst for aesthetics the crack width at the surface would be the design parameter. [23, p.9]

The crack spacing formula Expression 3.2 is understood to include a combination of slip at the bar and crack opening in the cover zone. When taking a look at the terms in the formula it is assumed that the calculated crack spacing occurs at the concrete surface. However the code does not clarify if the constant terms in the formula are calibrated using the assumption that the strain (ε_{sm}- ε_{cm}) is calculated at the bar or the concrete surface.

5.2.1 Cracking in the cover zone vs. cracking at the bar surface

In order to achieve information of the influence of the slip at the bar and crack opening in the cover zone the expression for maximum crack spacing 3.2 can be divided into two separate components. First term containing factor k_3c is called "Term A" and second term containing factor $k_1k_2k_4\phi/\rho_{p,eff}$ is called "Term B". "Term A" represents cover zone cracking and "Term B" cracking at bar. [23, p.10] In tables below are shown experimental results, in which the two components of the maximum crack spacing formulae are taken individually to see if the first term "Term A" correlates with the change in crack width between bar and surface, and the second term "Term B" represents the crack width at the bar.

Table 5.1. Comparison between calculated and measured crack widths at surface. [23, p. 11]

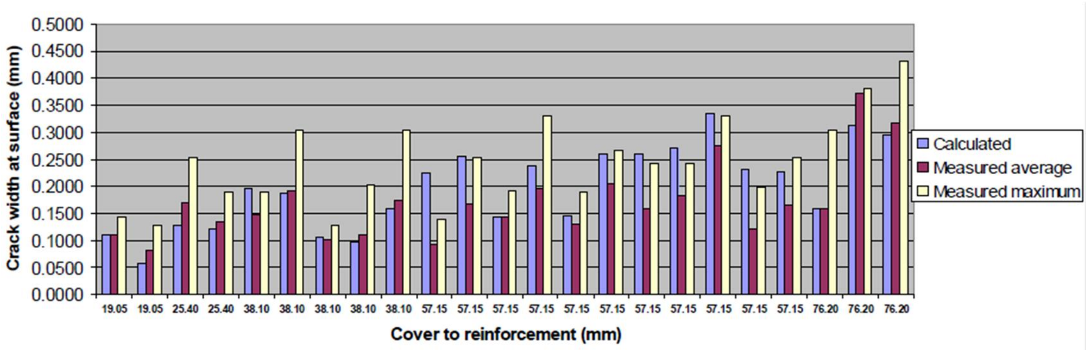


Table 5.2. Comparison between calculated and measured crack widths at bar. [23, p.11]

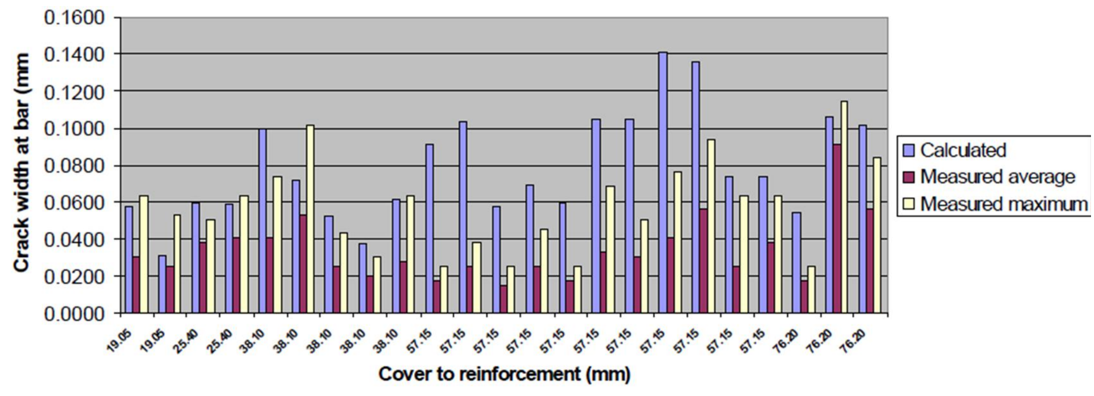


Table 5.3. Comparison between calculated and measured crack increase through cover. [23, p. 12]

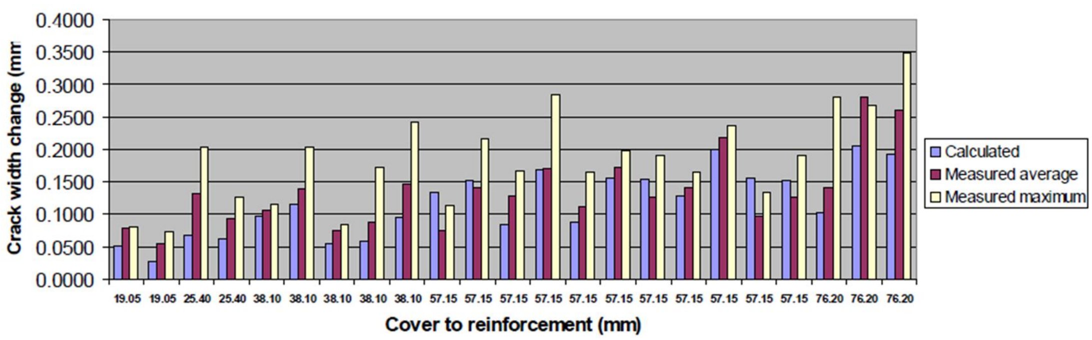
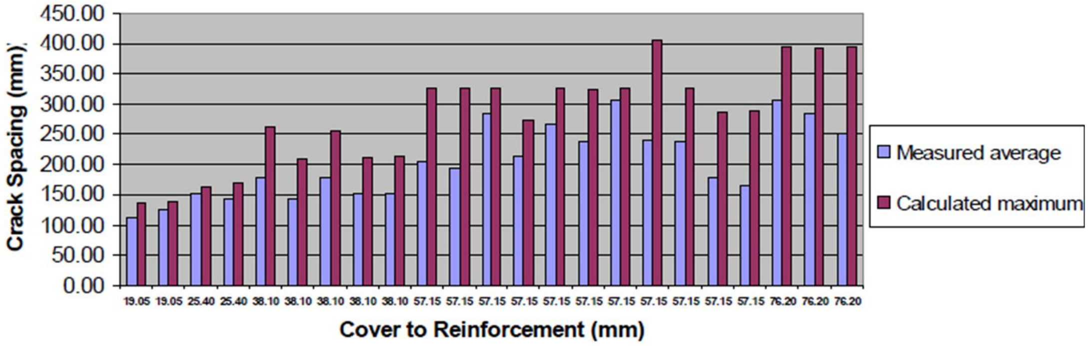
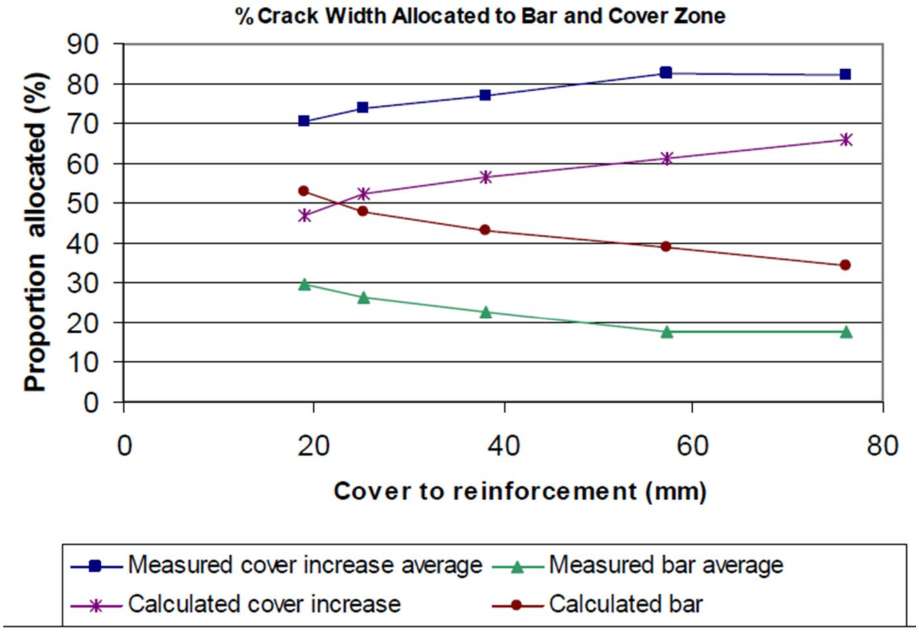


Table 5.4. Comparison between calculated and measured crack spacing. [23, p. 12]



Comparison between the measured data and EC2 cracking calculation provides that calculated crack width at the bar is higher than measured. Also the calculated crack width growth from bar to concrete surface fits the average data fairly well but is lower than the maximum values. The following table also shows that the relative strength of the cover term “A” in crack spacing formula is lower than in practice and the bar slip term “B” is stronger than observed.

Table 5.5. Ratio of cracking at bar to cracking in over zone. [23, p. 12]



It must also be noted that in calculations the effective tension zone is limited to a height of *(h-x)/3*. Otherwise the effects observed would be more marked due to fact that the cover *c* features in both terms of the crack spacing formula in case h_{ceff} is defined by *2,5(h-d)*. As a conclusion it seems that, at least partly, “Term A” is related to the difference in strain in the concrete at the bar and the concrete at the surface.

5.2.2 Large concrete cover thickness

Debate around limiting the crack width for sections with large cover thicknesses usually centers on the need to limit cracking for durability reasons. In the background there is the paradox of large cover thickness which on the other hand ensures better protection for reinforcement against corrosion, but, again, will lead to increased crack widths. When considering the calculation according to EC2 it is not evident if the methods in the code have been calibrated for use with very large covers, or if those can predict cracking at notional positions between bar and surface, for example at the minimum cover position. [23, p.13]

For instance bridge design in the UK is based on the required cover for durability regardless of any excess cover provided. In order to see if the EC2 formulae could be used in a similar manner the case has been investigated, which proved that increasing cover will:

- increase the crack spacing with the term k_3c leading to increased crack width
- increase the crack spacing by reducing $\rho_{p,eff}$ leading to increased crack width
- reduce the crack strain by reducing $\rho_{p,eff}$ leading to reduced crack width

Remarkable is that, if the limit *(h-x)/3* dominate the calculation the last two changes will not perform, which leads to increased crack width as a net result. Experimental tests, in which various combinations of including and excluding the excess cover in any or all three parts of the cracking formulae was studied, show also that these interpretations will stretch the crack width calculation according to EC2 beyond its intended use. [23, p.13]

Because of the variation in crack width between bar and surface it has been suggested to change the value of the limiting crack width for durability as a function of C_{nom}/C_{dur} in Table 2.3 or reduce the calculated crack width by a factor related to C_{nom}/C_{dur} before comparison with the limiting value. This limit would be a nationally set parameter and way easier to change than the spacing formula. [23, p.13]

5.3 Multiple rows of steel

Problems with the interpretation and use of EC2 crack width calculation have been arisen also generally in cases of reinforcement with multiple rows of steel. Such cases are associated with two problems which occur when $h_{c,eff}$ is limited at the value of *(h-x)/3*. [23, p.15]

First aspect can be demonstrated with section with two rows of steel. Then it is fairly possible for the inner one to fall just inside, or just outside $A_{c,eff}$. Different interpretations can result in doubling or halving the value of $\rho_{p,eff}$. Though, there is no physical reason to assume that change in rows of bars would affect such significantly on crack spacing in reality. [23, pp.14-15]

Second problem can be seen when comparing the sections below. As the value of $h_{c,eff}$ is limited by *(h-x)/3* the two different bar arrangements give nearly same values for $\rho_{p,eff}$ and crack spacing.

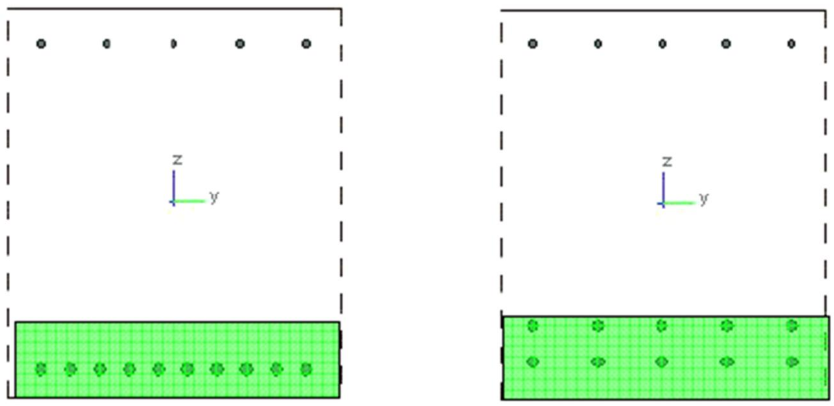


Figure 5.6. Effective tension area governed by *(h-x)/3*. [23, p. 15]

These two problems can be solved by using the definition of *d* as the effective depth from the bar layer closest to concrete surface. In addition $h_{c,eff}$ should be limited by the adjacent row of bars to the value of *(h-d) + row spacing/2*. This might seem to ignore the concrete that could be mobilized by the inner row of bars, but, it will provide the similar ratio of $\rho_{p,eff}$, when expecting that the inner row of bars has a similar area of concrete associated with it. It is also supported by the theory behind EC2 approach, which is based on a local area surrounding a single bar with a constant tension stress of lesser than f_{ctm}. [23, p.15]

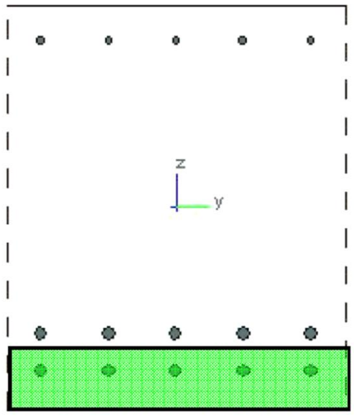


Figure 5.7. Alternative proposed effective tension area. [23, p. 17]

6 Conclusions

The present study has explored and clarified cracking and crack control of concrete structures in special cases. In addition to normal methods such as curing and casting arrangements significant ways of crack control in practice are limiting compressive strength of concrete and crack width, in which detrimental cracking can be avoided. Limiting of crack width without direct calculation according to Eurocode is meant for simple structures and cannot be used for special cases, which is why crack widths are mainly to be limited by calculation. It is noted that requirement for minimum amount of reinforcement must be fulfilled then.

It was stated in the study that special attention must be paid to construction of massive concrete structures due to two primary reasons: thermal cracking resulting in increased heat of hydration, and delayed ettringite formation. The study revealed that mix design, insulation or concrete cooling are the most effective ways to minimize cracking in massive structures.

An extensive study of calculation of crack width for sample structures in the simple and special cases according to Eurocode was carried out in this thesis. In the crack width calculations for a sample structure in the simple case, it was discovered rather slight difference between the results of the used methods, and according to experimental results the reliability of Eurocode, and other methods as well, can be said to be good in calculation of crack width for normal structures.

The study of calculation of crack width for a sample structure in the special case was carried out in order to achieve information about the reliability of the Eurocode in case of thick concrete cover, combined with heavy reinforcement and massive cross-section. The intention was also to clarify, whether it is possible to control cracking in such case without having to use surface reinforcement. The crack widths for cover thickness of 100mm were quite large in the calculations regardless of the method and satisfying crack width of about 0,4mm was received by lowering the steel stress to 220MPa in that case, when steel stress for cracking moment equaled about 95Mpa. DIN 1045 method proved to give smallest crack widths, while the results of ACI 318 method were much higher than those given according to other methods. This is odd even if the writers of the code are highly doubting its reliability for cover thicknesses greater than 70mm. However, high crack widths were obtained already for 50mm of concrete cover in the special case according to ACI method. While the sample calculation in the normal case showed only a 10% of difference between the Eurocode and ACI method, it can be doubted whether the ACI code gives accurate results for cross-sections with multiple rows of steel.

Another interesting point must be raised in the calculations. In normal case crack widths according to Eurocode are smaller than those according to other methods. The difference between the Eurocode and RakMK is about 30% which is because the Eurocode is assumed to predict crack width values to occur either on the surface or on the centroid of the reinforcing bar, while RakMK predicts it into outermost tension face of a structure. However, in the special case this difference is disappeared and EC2 and RakMK give almost equal results, while crack width results according to DIN turned out to be the smallest. This cannot be explained by anything else than the fact that reliable results are not obtained according to EC2 for a cross-section of the special case.

The experimental results of crack width tests were studied as well in this thesis. Experimental test for special case, in which the influence of thick concrete cover and large diameter bars were under investigation. It turned out that that EC2 seems to overestimate the crack width about 25 to 65 % in case of thick concrete cover. Meanwhile the ACI underestimates the predicted crack width. This will support the suggestion of the unreliability of the crack width calculation rules according to Eurocode for thick concrete covers. The experimental results were close to crack widths obtained according to Canadian CSA-S474-04 and Norwegian NS 3474 E methods. More information would have been received in this thesis, if additional crack width calculations for a sample structure in the special case had been performed according to these methods.

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