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SUMMARY

Proper design of concrete structures is today important to achieve a cost-effective and sustainable construction. One of the limits which needs to be meet during the design are the allowable crack width diameter.

In total there are 30 crack width formulas which are being presented in this thesis. Each formula has been used to calculate crack widths for a given reinforced concrete slab.

It has been conducted crack width calculations for several different setups for the reinforced concrete slabs. Where four categories for discussion have been looked further into. This has provided a basis for more discussion around what consequences certain design choices have on the crack widths. The results show that there are many aspects which needs to be taken into consideration to achieve a durable reinforced concrete structure.

3 KEYWORDS
Crack Width Calculations
Reinforced Concrete Slab
Axial Tension

Abstract

Proper design of concrete structures is today important to achieve cost-effective and sustainable constructions. One of the limits which needs to be meet during design is the allowable crack width diameter. This thesis focuses on presenting published literature formulas and code provisions and then utilize them on a reinforced concrete slab experiencing axial tension.

The thesis presents what types of cracks that can occur in concrete and their causes Background theory regarding transport mechanisms in concrete and what impact cracking have on these mechanisms are also being evaluated. How crack develops in concrete structures and what rules apply for crack width control is also important topics which are being touched upon in this thesis.

In total there are 30 crack width formulas which are being presented in this thesis. Each formula has been used to calculate crack widths for a given reinforced concrete slab.

It has been conducted crack width calculations for several different setups for the reinforced concrete slabs. Where four categories for discussion have been looked further into. This has provided a basis for more discussion around what consequences certain design choices have on the crack widths. The results show that there are many aspects which needs to be taken into consideration to achieve a durable reinforced concrete structure.

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Symbols

A	penetrated concrete area
a	bar spacing
$A_{c,ef}$	effective concrete area in tension
A_s	area of steel reinforcement bar in tension
b	width of the concrete section
c	concrete cover
$c_1 - c_2$	concentration gradient
D	diffusion coefficient
d	effective depth
E_c	modulus of elasticity of concrete
E_s	modulus of elasticity of steel
f_{cm}	mean compressive strength of concrete
f_{ctm}	mean tensile strength of concrete
f_{yk}	characteristic steel yield strength
h	total height of the concrete section
Δh_w	hydraulic pressure difference
i	exponent which characterise the time development
K_w	coefficient of water permeability in concrete
l	concrete thickness
l_0	length of lost bond
l_{tr}	transfer length
M_w	coefficient of water absorption in concrete
n	number of bars in tension
$R = (h - x_2)/(d - x_2)$	coefficient to obtain the crack width at the bottom of concrete from that at the level of reinforcement
R_{cm}	mean cubic compressive strength of concrete
Q	amount of transported substances
s	slip between concrete and rebar
s_m	average crack spacing
s_{max}	maximum crack spacing

s_{min}	minimum crack spacing
s_k	characteristic crack spacing
t	time
t_e	effective distance between the considered crack and the nearest bar
V	volume of water
w	water absorbed per unit area at time t
w_k	characteristic crack width
w_{k1}	Characteristic crack width for tightness class 1
w_m	mean crack width
x_1	neutral axis depth before cracking
x_2	neutral axis depth after cracking
$\alpha_e = E_s/E_c$	ratio between steel and concrete e-module
$\beta = w_k/w_m$	ratio between characteristic and mean crack width
ε_c	concrete strain
ε_m	average steel strain
ε_{s1}	steel strain before cracking
ε_{s2}	steel strain in cracked phase
ε_{sh}	shrinkage strain in the concrete
ε_{sm}	average difference between steel and concrete strain along two consecutive cracks
$\rho = A_s/(b * d)$	longitudinal reinforcement ratio
$\rho_{ef} = A_s/A_{c,ef}$	effective reinforcement ratio
σ_c	concrete tensile stress
σ_{c1}	concrete tensile stress before cracking
σ_s	steel stress
σ_{s1}	stress in reinforcement before cracking
σ_{s2}	stress in reinforcement after cracking
σ_{sr}	steel stress at cracking of concrete
τ_b	average bond stress
$\tau_{b,max}$	maximum bond stress on the bars
Φ	diameter of reinforcement bars

1 Introduction

1.1 Background and Motivation

For the last couple of years there has been a rising trend in the construction industry to achieve a higher degree of cost efficient and sustainable concrete structures. To achieve this, structures which are being planned to be built are thoroughly analysed during its design phase. For reinforced concrete structures one of the design criteria's which needs to be fulfilled is to minimize the crack widths within a limit.

Cracks which can occur in concrete structures needs to be thoroughly reviewed during the design phase of reinforced concrete structures, no matter the dimension of the structure. If severe cracking is allowed to take place, the structure may experience unplanned deterioration before fulfilling its service life.

Crack width calculations may for some be perceived as a secondary design criterion. Coming in after the calculation of maximum capacity etc. But it is just as important and necessary to maintain the durability of the structure. Especially if the structure is placed in a harsh climate

Therefore, is it desirable to look further into how these crack computations are conducted and what influence the different parameters in the formulas have on crack width calculation.

1.2 Scope of the Thesis

The scope of this thesis has been to look further into how crack width calculations are conducted for reinforced concrete structures. In this case, for a reinforced concrete slab subjected to axial loading. As a whole, the foundation of this thesis is summarized in the bulletins below.

- Present background knowledge about how cracks occur, transport mechanisms in concrete, impact of cracking and crack width control.
- Reviewing published literature formulas and code provisions.
- Conduct crack calculations with the formulas for 4 calculation categories resulting in 12 calculation cases.
- Present the results and discuss important observations

1.3 The Structure of the Thesis

This thesis is divided in to 7 chapters.

Chapter 2 presents theory relevant for the thesis. The chapter starts by presenting mechanisms which may stimulate cracks to start appearing in concrete. The chapter then goes over to describing how liquids, gases and ions gets transported through uncracked concrete, and what effects the transport of the substances may have towards deterioration of concrete. The last part of the chapter dives deeper into what effects cracks may have with respect to the transport mechanisms, and further discusses the consequences the cracking may bring.

Chapter 3 presents the importance of crack width control for reinforced concrete structures. It describes what requirements Eurocode 2 have for crack width calculations. It explains how cracks develops in concrete structures and it discusses some of the parameters which are important for the calculations.

In chapter 4 the crack width calculation formulas are presented. The presentation is based on the journal article “A review of literature and code formulations for cracking in R/C members”, published by Lapi, Orlando and Spinelli. In total 21 literature formulas plus 9 code provisions concerning crack width calculations gets presented in this chapter.

In chapter 5 setup for the calculations conducted for this thesis are presented. The calculations are conducted on a reinforced concrete slab subjected to axial tension. The calculations are conducted in a excel sheet containing all the formulas presented in chapter 4.

The discussion of the results takes place in chapter 6. Where 4 calculations categories are presented and discussed. Each category has three cross-section setups. The discussion is

At last, in chapter 7, a conclusion is drawn for the thesis. Where the chapters and discussion are summarized.

2 Background theory

2.1 Introduction

This chapter presents a summary of theory which is needed to better understand how cracking develops in reinforced concrete structures and what their consequences may be for the concrete structure. The chapter starts with a basic introduction to transport mechanisms for gas, liquid and ions in plain concrete. It then goes over to discussing the impact cracking have on the respected transport mechanisms. It also discusses what impact the various types of cracks will have on the tightness of the concrete.

The chapter is compiled with the help of DaCS Report No. 04. "The impact of cracks on gas and liquid tightness of concrete" Published by Anja Birgitta Estensen Klausen [1].

2.2 Cracking of Reinforced Concrete Structures

There are several different types of mechanism involving cracking which a concrete structure may be exposed to during its life span. Some of these cracking mechanisms and their typical time of occurrence are illustrated in Figure 2.1. The cracking, which the concrete is subjected to, may be disadvantageous when keeping its durability, tightness and aesthetics intact. Since there are various types of cracks which can occur, is there also several reasons why these cracks start to appear. These reasons are coupled to different aspects which the concrete structure may be subjected to during its service life. Summarized, the main reasons for why cracks starts to develop in a concrete structure are as follow; volume changes in the concrete, mechanical loading on the structure, environmental conditions exposed to the concrete and chemical reactions occurring internally in the concrete material [1]. These reasons are discussed further in Chapter 2.2.1 - 2.2.3.

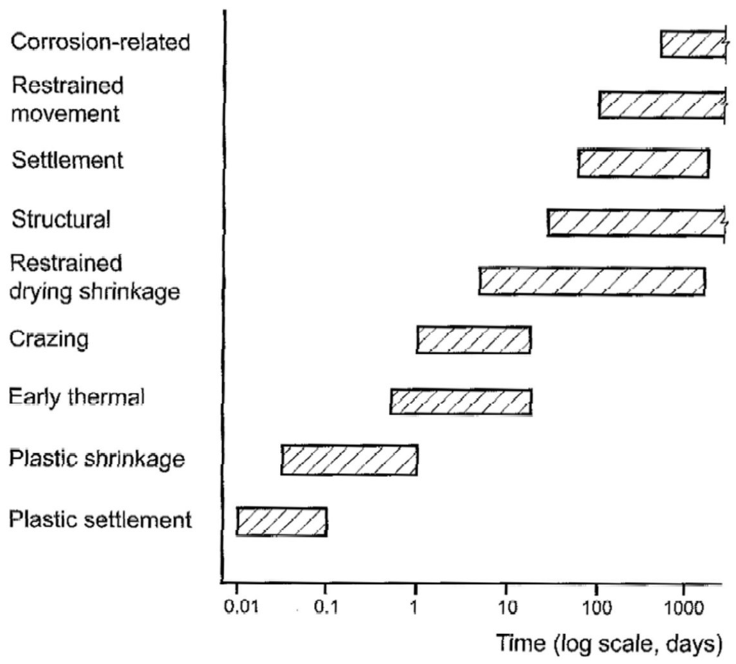

Figure 2.1 Types of cracks which can occur in concrete structures [2]

2.2.1 Cracks due to Volume Changes in Concrete

The different mechanisms which leads to volume changes in concrete are summarized in the following subchapter. The tensile strength of the concrete may be exceeded if theses volume changes gets restrained, thus leading to cracks starting to appear in the concrete structure.

Plastic shrinkage

After casting fresh concrete, the surface of the concrete may start to evaporate water due to humidity differences between the concrete and the surrounding area. This mechanism is known as plastic shrinkage. The evaporation leads to volume contraction due to internal water evaporating from the concrete. The volume contraction can often lead to wide surface cracks in the concrete, often up to 2-3 mm wide. Having a depth equal to the concrete cover, or in some cases even deeper [1].

Plastic settlement

In fresh concrete, solid masses in the concrete mixture, I.E. gravel and sand, may move vertically downward into the cross-section due to gravity affecting the structure. This may lead to an overall settlement of the surface of the concrete. Disturbances internally in the concrete, such as reinforcement or a variation of the dimension of the cross-section may hinder the overall settlement and thus lead to a dissimilar settlement throughout the surface of the concrete. Thus, leading to plastic settlement cracks appearing at the surface of the concrete. The width of the cracks may reach up to 1-3 mm and they may reach as deep as the reinforcement, or even further [1].

Drying shrinkage

Hardened concrete will slowly over time dry out when being exposed to a dry environment. This kind of drying in concrete is known as drying shrinkage. The shrinking will start at the surface of the concrete and over time dive deeper into the cross-section. The shrinkage will contract the concrete and produce stress gradients acting on the whole concretes cross-section. The gradients lead to tensile stresses acting at the surface on the concrete. If severe enough, the concrete tensile strength will be exceeded and thus producing drying shrinkage cracks in the outer perimeter of the concrete cover [1].

Autogenous deformation

Autogenous deformation is a self-produced shrinkage in the concrete. The deformation occurs due to the chemical processes happening in the concrete while curing. The total volume of the reactants (cement and water) taking part in the hydration process are bigger than the volume of the reaction products after that initial curing has concluded. Autogenous deformations will have an impact on the whole cross-section of the concrete. The volume changes due to autogenous deformation, in combination with thermal dilation, can give rise to through-cracking in the concrete-section if restrained during the hardening phase [1]

Thermal dilation

While the concrete is in its fresh state after being cast, hydration will take place internally in the concrete. Cement and water react when they get in contact with each other and produces cement paste. The reaction between the substances is exothermic, which means that it yields excessive heat. The heat can considerably increase the temperature of the concrete, and hence lead to a concrete volume expansion. As the hydration velocity starts to decrease and the concrete begins to go over to a hardened state, the material will contract and go back to its original volume state. The volume changes due to temperature variation can cause through-cracking in the concrete cross-section. In combination with autogenous deformation, which occurs in the concrete at the same time, the cracking can become severe with respect to the structure [1].

2.2.2 Cracks due to Mechanical loading on Concrete

Mechanical loading on reinforced concrete structures, if severe enough, can lead to cracks starting to appear. Depending on the type of loading it may lead to different types of crack development, e.g. flexural cracks, shear cracks and through, cracks. Flexural cracks occur due to the cross-section is being subjected to flexural moment. Shear cracks arises due to shear forces acting internally in the cross-section. Through-cracking may appear due to the axial tensile stresses affecting the structure. Cracking in concrete structures due to mechanical loading is nevertheless not always a problem, and decently designed concrete structures will for the most cases start to develop some types of cracks during their lifespan. Since the reinforcement in the concrete is designed to carry loads, but at the same time reducing the crack widths and ensuring a sufficient distribution of cracks [1].

2.2.3 Cracks due to Environmental Conditions and Chemical Reactions

Both Environmental conditions surrounding the concrete structures and chemical reactions taking place internally in the concrete may cause cracking. A brief summary of the relevant contributors is described in this sub-chapter.

Freeze-thaw deterioration

When free water in moist concrete freezes, its volume expansion will produce a pressure on the internal pore system of the concrete. If the pressure exceeds the tensile strength of the concrete, the pores will rupture. Freeze-thaw may happen cyclic on the concrete placed outside due shifting seasons. The cycles will have an accumulative effect and the rupture in the process can eventually cause concrete volume expansions and cracking. Other damages which can occur due to freeze-thaw deterioration are crumbling and scaling of the concrete [1].

Alkali-Silica reaction

An Alkali silica reaction occurs due to a reaction taking places between hydroxyl ions originating from the cement and reactive silica from the aggregates in the concrete. When this reaction occurs, a gel is produced, which in combination with free water will increase its volume and exert an expanding pressure inside the concrete. This will lead to a volume expansion, which may fracture the concrete structure [1].

Corrosion

Corrosion may take place at the reinforcement inside the concrete structure. The products from the corrosion process are larger in volume compared to the reactants. This will cause a volume expansion at the reinforcement steel, and tensile stresses will start to develop around the reinforcement steel. Once the expansion has propagated enough and the volume expansion exceeds the concrete tensile strength, splitting cracks will begin to develop at the reinforcement level. These cracks will over time propagate to the surface of the concrete resulting in concrete spalling and loss of bond between the reinforcement and concrete [1].

Sulphate attack

Sulphate attack is a deterioration process that start to propagate due to the combination of two chemical reactions taking place in the concrete. The first reaction is based of sulphates reacting with lime from the cement which forms gypsum. The other reaction takes place when sulphates is reacting with hydrated calcium aluminates which forms ettringite. This mechanism also produces products which have a volume that is greater than the initial reactants, The volume expansion exerts expansive pressure internally in the concrete, which may lead to cracks starting to appear in the concrete surface [1].

2.3 Transport Mechanisms in Concrete

The concrete material has the ability to transport gases, liquids and ions through itself. The transport of the substances is supported by various mechanisms, and the most influencing mechanisms are permeation, diffusion, capillary suction and adsorption. These mechanisms can occur simultaneously and therefore make it difficult to describe the full transport process through the concrete [1]. To oppose these mechanisms, the concrete relies on its tightness to deaccelerate the transport of gases, liquids and ions.

The tightness of the concrete is governed by internally permeability. Concrete is a brittle and porous material, and when in a hardened state, the porosity of the material determines the permeability of the actual concrete. The porosity in the concrete is dependent on the pore system of the concrete, which consists of gel pores and capillary pores. The fraction of capillary pores can range between 0 to 40 % of the volume of hardened cement paste While gel pores makes up around 28 % of the volume [1]. Since the transport mechanisms mainly takes place in the pore system, the permeability, which is dependent on the porosity, becomes important.

2.3.1 Permeation

The flow of liquids and gases caused by a pressure gradient acting on the concrete are known as permeation. This flow can over time be modelled and described by Darcy’s law.

$$V = K_w * \frac{A}{l} * \Delta h_w * t \tag{2.1}$$

Where V is the volume of water, K_w is the coefficient of water permeability through the concrete, A is the penetrated area, l is the thickness of the concrete, Δh_w is the pressure difference between the surfaces and t is the time.

The parameters in equation (2.1) are for the most part dependent on the surrounding conditions of the concrete. E.g. dimensions of the material, time and the pressure gradient. The actual concrete is in equation (2.1) only described by the permeability coefficient. The coefficient describes speed, which if the value is high, liquids and gases will consequently be transported faster and easier through the concrete.

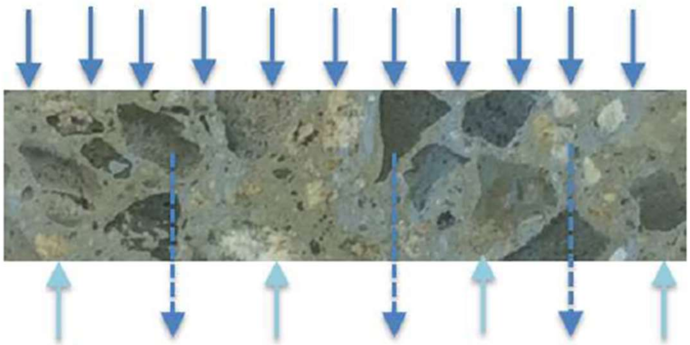

Figure 2.2: Illustration of permeation in concrete [1]

2.3.2 Diffusion

Diffusion describes how mass gets transported through the material. In this circumstance the mass are molecules and ions being set into random motion due to a concentration gradient acting on the concrete. The ions and molecules originate from gases, liquids and dissolved substances which tries to travel through the concrete. The diffusion mechanism is especially important when describing carbon dioxide and chloride ions penetrating the concrete.

The diffusion mechanism can be modelled by Fick's first law of diffusion.

$$Q = D * \frac{c_1 - c_2}{l} * A * t \tag{2.2}$$

Where Q is the amount of transported substances, D is the diffusion coefficient, $(c_1 - c_2)$ is the concentration gradient between the two surfaces, l is the thickness of the penetrated concrete area, A is the area of the penetrated concrete and t is the time.

The flow rate of the diffusion in Equation (2.2) is proportional to the diffusion coefficient D and the concentration gradient $(c_1 - c_2)$. Just as for the permeation transport mechanism, almost all parameters in Equation (2.2) are also dependent on the surrounding conditions. The diffusion coefficient D on the other hand are dependent on the characteristics of the internal structure of the concrete, and are for the most part determined based on its mean compressive strength [1].

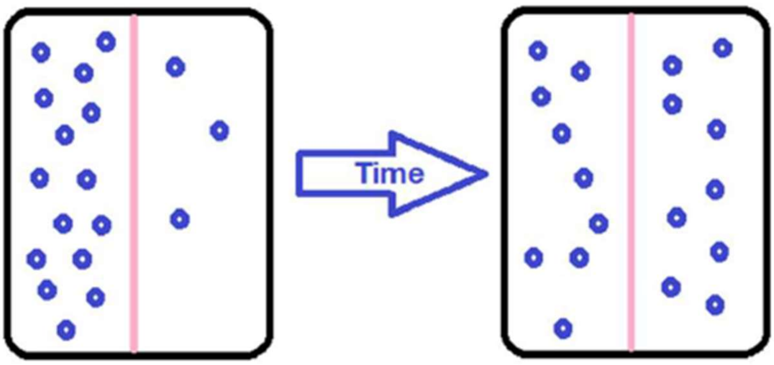

Figure 2.3: Diffusion in concrete [1]

2.3.3 Capillary Suction and Adsorption

Capillary suction is a mechanism which can occur in porous materials, where the capillaries in the material sucks up liquids due to surface tension. Parameters which are influencing the transport are density, viscosity and the surface tension coefficient associated to the particular liquid.

Due to the nature of how concrete reacts when being cast, irregularities within the pore system will occur. This complicates the theoretical approach of describing the occurring mechanism. It has therefore been established empirical relationship describing liquid absorption in concrete occurring due to capillary suction.

$$w = M_w * t^i \tag{2.3}$$

Where w is the water absorbed per unit of area at time t, M_w is the coefficient of water absorption, t is the time and i is an exponent which characterize the time-development of the process.

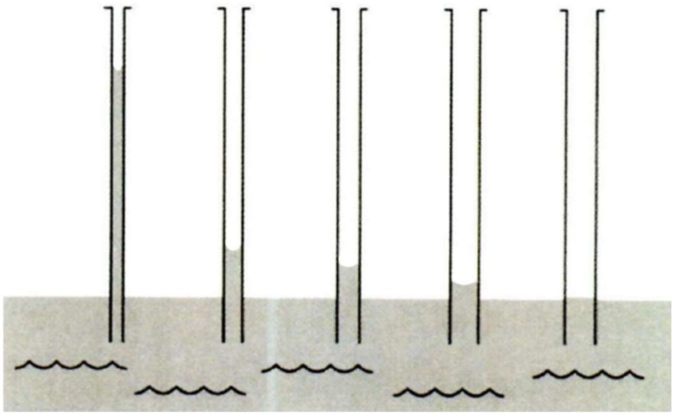

Figure 2.4: Illustration of capillary suction [1]

Just as for the two other transport mechanism, capillary suction also only has one parameter which is described by concrete material. That is the coefficient of the concrete water absorption. This coefficient is also roughly estimated with the help of the mean compressive strength of the concrete. Capillary suction depends on the moisture content internally in the concrete. Which implies that the water adsorption rate decreases when the pore humidity is increased.

2.4 Impact of Concrete Tightness due to Cracking

There are various reasons due to why cracks may start to develop in a concrete structure, as described in Chapter 2.2. Cracks appearing in the concrete may have a direct influence on the transport of gases and liquids through the concrete. The impact is though very dependent on which type of cracking that takes place. Gas and liquids for example, still have to be transported through the uncracked parts of the concrete when the material is exposed to flexural-, surface-, or micro-cracks. Which means that the concrete permeability still is dependent on its porosity. On the other hand, if the concrete is subjected to through-cracking, the substances will then have free room to pass through the concrete, and thus the permeability of the material becomes solely crack-dependent [1].

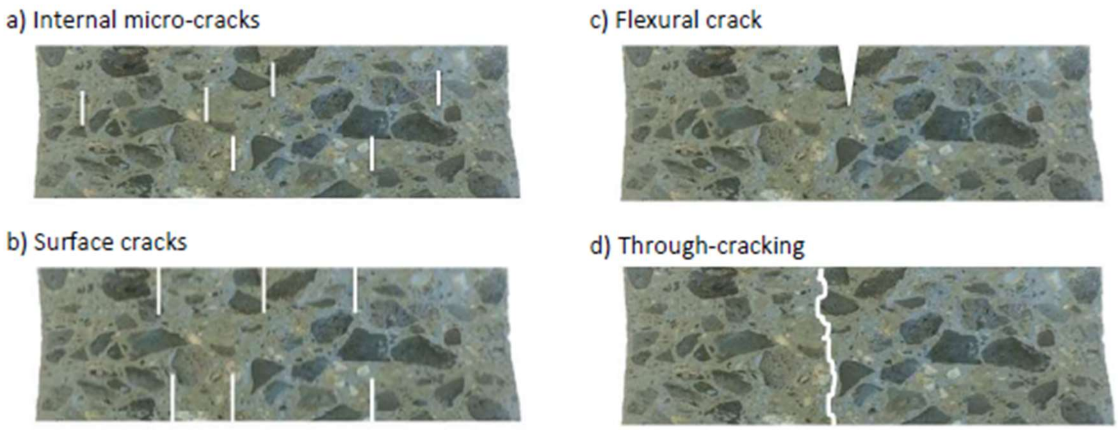

Figure 2.5: Different types of cracks in concrete [1]

2.4.1 Surface-, Flexural- and Microcracks

As mentioned, when a concrete structure starts to develop surface-, flexural- and/or microcracks, its permeability is still dependent on the concrete porosity. But that does not mean that the cracks will not affect the concrete permeability. The cracks will interfere with the pore system of the concrete establishing new flow paths for gas and liquids to transport through, thus increasing the permeability. Over time this means that the accumulation of water transported-or chemical ions penetrating through the concrete will become higher compared to uncracked concrete. This increase can lead to a greater deterioration of the concrete than first expected. This negative chain is illustrated in Figure 2.6.

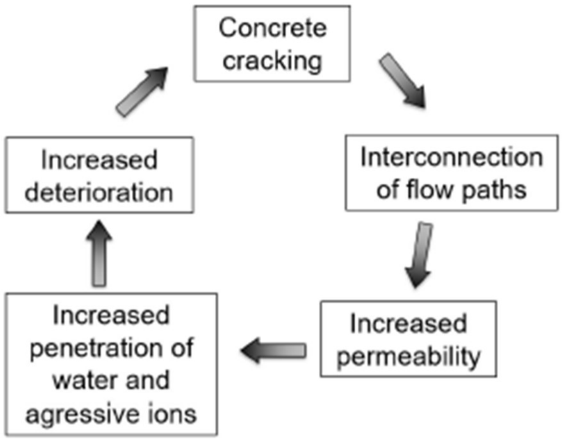

Figure 2.6: Negative chain reaction due to cracking of concrete [1], based on [3].

2.4.2 Through-Cracking

As described earlier, through-cracking will have a direct influence on the permeability of the concrete. The permeability of concrete, which is exposed to through-cracking, are not dependent on porosity, but instead dependent on the through-crack. How easy it is for liquids to flow through cracks in the concrete depends on the width of the crack, the footprint of the crack trough the concrete section, the crack surface roughness and the length of the crack [1].

3 Design Methods Concerning Cracks in R/C

3.1 Introduction

This chapter looks further into what is important for crack width control for concrete structures. It then goes over to describing crack development in the concrete structure based on the journal article “Design for SLS according to fib Model Code 2010”, published by G. Balázs et al [4]. At the end it discusses some of the important parameters for crack calculations.

3.2 Crack Width Control

Cracks in reinforced concrete may in the real-world act rather sporadic and their propagation are dependent on many factors, which makes the behaviour rather complex. When trying to analyse and predict the crack pattern for a concrete structure under design, the analysis is based partly on physical mechanisms in the concrete and partly on experimental results and accumulated experience. The designer usually operates with a design crack width, where the design result originates from an appropriate load combination based on the concept of limit state design. The design crack width is verified against a nominal width limit value, which are being used as a design criterion. This limiting crack value are for most cases conservative compared to the values established based on experience. This to ensure that crack widths will not be a significant problem for the reinforced concrete structure throughout its service life [5]. Appearance, durability and tightness concerning the reinforced concrete structure are reasons for why crack control is so important [6].

Cracking occurs when stresses in the concrete section exceeds the strain capacity of the concrete material. The crack will arise normal to the stress direction and starts to develop when the stresses in the concrete exceeds the tensile capacity of the material.

As mentioned, there are limits for how wide the crack widths can be, depending on which exposure class the structure falls under and what type of load combination the structure is subjected to. For general structures Eurocode 2 classifies these limits as seen in Table 3.1.

Exposure class	Normally Reinforced Concrete
	Quasi-permanent load combination
X0, XC1	0,4 mm
XC2, XC3, XC4	0,3 mm
XD1, XD2, XS1, XS2, XS3	

Table 3.1: Recommended crack width limits. Based on [7].

For watertight structures, the limits become stricter. These limits area based on tightness classes, which depends on the requirement set for the watertight construction. Table 3.2 summarizes the requirements set by Eurocode 2 Part 3: Liquid retaining and containment structures [8].

Tightness Class	Requirements for leakage
0	Some degree of leakage acceptable
1	Leakage to be limited to a small amount
2	Leakage to be minimal.
3	No leakage permitted

Table 3.2: Classification of tightness. Based on [8]

These classes have their own set rules which need to be followed. The rules for each specific class are summarized below, based on Eurocode 2 Part 3.

Tightness Class 0 – The provisions in Eurocode 2, part 1 may be used

Tightness Class 1 – Any cracks which is to be expected to pass through the full thickness of the concrete section should be limited to w_{k1}.

Tightness Class 2 – Cracks which may be expected to pass through the thickness of the section should be avoided.

Tightness Class 3 – Special measures is required to ensure water tightness.

Usually, class 1 is relevant for concrete slabs, which has the purpose retain water. Which means that the crack width limit is set to value w_{k1}. This value is derived based on the ratio of hydrostatic pressures, h_d acting on the slabs thickness, h. For $h_d/h \leq 5$ the value equals 0,2 mm. When $h_d/h \geq 35$ the limit is set to 0,05 mm. For values in between linear interpolation are used to set the crack width limit [8]. An example of the interpolation is shown in Table 3.3

$\boldsymbol{h_d/h}$	≤ 5	10	15	20	25	30	≥ 35
$\boldsymbol{w_{k1}}$ **[mm]**	0,200	0,175	0,150	0,125	0,100	0,075	0,05

Table 3.3: Crack widths for tightness class 1

3.3 Crack Development

Model Code 2010 uses a prismatic reinforced concrete bar subjected to axial tension as a basis for their crack width calculations. Under increased tensile loading, its behaviour is distinguished into four stages, which are illustrated in Figure 3.1. The stages are as follow

- Uncracked stage (1)
- Crack formation stage (2)
- Stabilized cracking stage (3)
- Steel yielding stage (5)
-

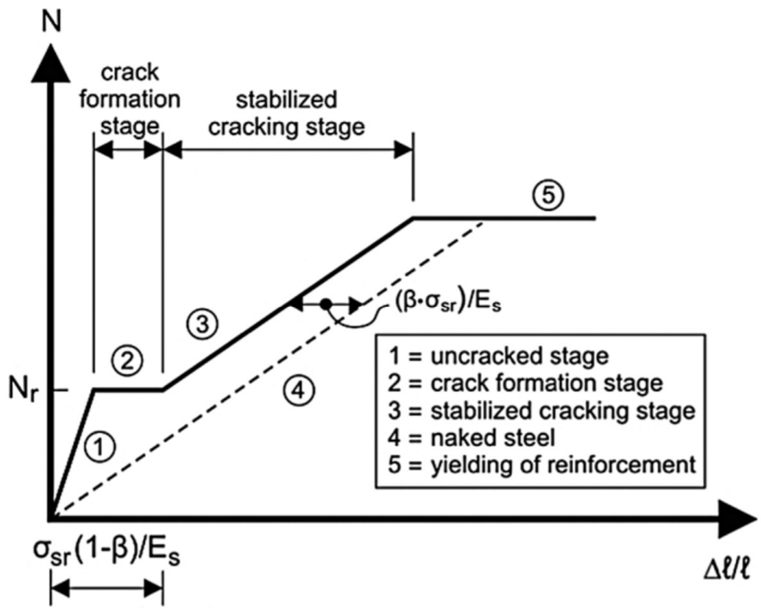

Figure 3.1: Simplified load-strain relationship. [9]

3.3.1 Crack Formation and Stabilized Cracking Stage

The graph in Figure 3.1 illustrates the basic strain behaviour, and its respective stages, of a reinforced concrete prism experiencing an increasing axial deformation. During the crack formation stage (2), the axial tensile force does not increase but instead cracks are starting to appear in the concrete surface. When the number of cracks has reached a point of where there are no undisturbed areas in the prism left, the tensile strength in the uncracked parts of the concrete cannot be reached anymore. Which means that no new cracks can appear. At this point the reinforced prism goes over to the stabilized cracking stage (3). During this stage, while the deformation is increasing, no new cracks will appear in the concrete. Instead, the existing cracks will become wider and the reinforcement will at the same time experience higher stresses. At a certain point, depending on the steel quality, the reinforcement will yield. Which means that the prism no longer can take up any loads. The structure is then at stage (5) and will not function properly anymore, and in worst case scenario collapse [4].

A simplified representation of the load-deformation in the prism during loading is also illustrated in Figure 3.2 and Figure 3.3. When cracks are starting to appear, the reinforcement steel has to carry the load in the cracked cross-sections. The loading is partially transmitted to the concrete on both sides of the crack. This is illustrated as the discontinuity area in Figure 3.2 and Figure 3.3. The concrete behaves as normal again, meaning that it can carry some of the tensile load, on both sides of the crack at a distance equal to $l_{s,max}$ [4].

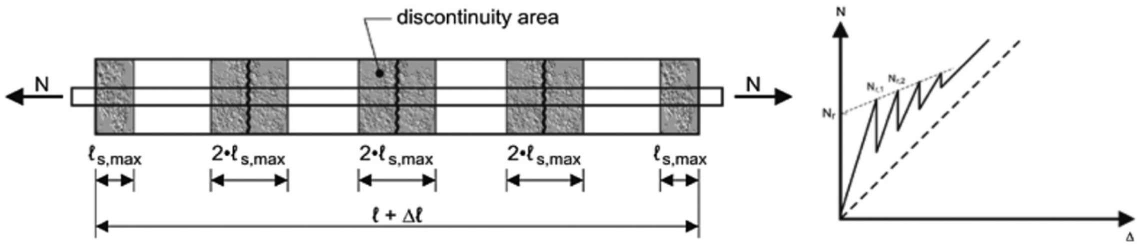

Figure 3.2: Behaviour of a reinforced bar subjected to imposed deformation. [9]

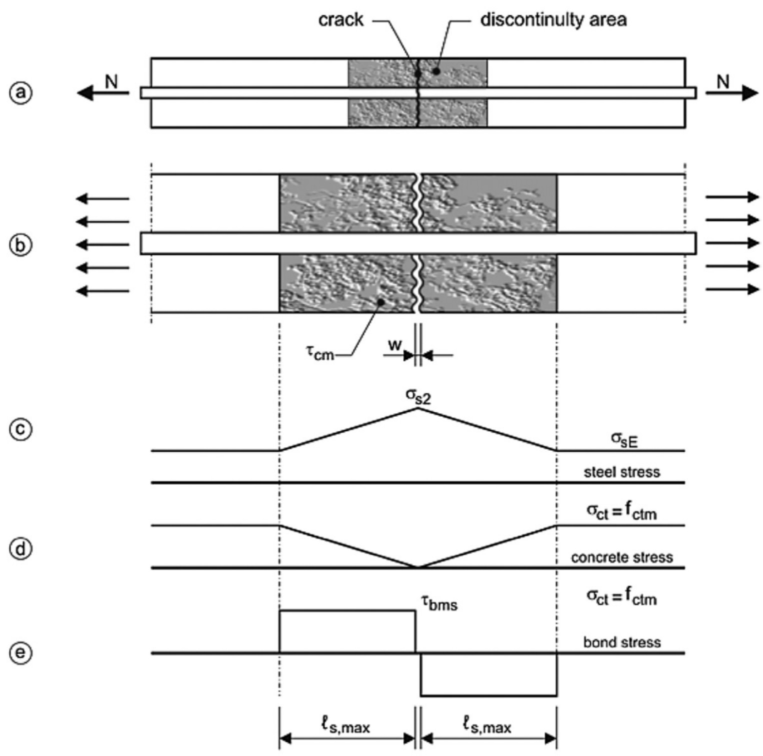

Figure 3.3: Steel, concrete and bond stress during crack formation stage. [9]

3.4 Crack parameters

3.4.1 Tension Stiffening

The tensile forces in concrete is released whenever cracks are formed. Due to bond stresses between the reinforcement and concrete, some of the tensile forces will be transferred from the bars to the concrete after that cracking has occurred. This force transfer lead to decreased reinforcement deformations. The contribution from the concrete, which leads to reduced reinforcement deformations is known as tension stiffening. The tension stiffening becomes insignificant over long term- and repeated loads [4].

3.4.2 Effective Tension Area of Concrete

The effective tension area of concrete, $A_{c,ef}$, has been developed as a calculation parameter used in the calculation of crack widths. The parameter is utilized to transfer the mechanics of reinforced concrete tension ties with distributed reinforcement to more general cases such as concrete members which are thick or in flexure [4].

3.4.3 Crack Width at Reinforcement vs. Crack Width at the Concrete Surface

Cracks in concrete structures are usually observed and measured on the surface of the structure. However, several of the early adopted calculation models calculates the widths at the level of the reinforcement. The differences between crack width values at the reinforcement level and surface level illustrates the deformation of the concrete cover in the cracking zone. Also the difference between the values can be significant for structures with high cover thicknesses [4].

4 Crack Width Calculations Models

4.1 Introduction

In 2018 Lapi, Orlando and Spinelli published a literature review paper which evaluated literature models for crack width formulas, which have been developed during the last couple of decades [6]. The aim of the paper was to assess the progress that have been made within the area and evaluate how effective the current standards are towards estimating cracking in reinforced concrete structures. The review paper presents each model, which have been obtained through available literature, and then discusses the impacts they have had on the formulation of international codes and standards, concerning crack width prediction. Provisions from the standards have also been assessed, where the goal has been to identify which calculation parameters would make the biggest impact on the calculations.

4.2 Literature Formulas

All the literature formulas from the article and the year they were published are summarized in Table 4.1

Saliger (Chapter 4.2.1), Scholz (Chapter 4.2.17) and Debernardi and Taliano (Chapter 4.2.21) will not be included in the calculations due to yielding unfitting results.

Year published	Authors	Model Type
1950	Saliger	Empirical Model
1956	Clark	Empirical Model
1963	Kaar and Mattock	Empirical Model
1965	Broms	Semi-Analytical Model
1965	Broms and Lutz	Semi-Analytical Model
1966	Borges	Semi-Analytical Model
1968	Gergely and Lutz	Empirical Model
1970	Holmberg and Lindgren	Semi-Analytical Model
1977	Leonhardt	Analytical Model
1979	Beeby	Analytical Model
1980	Nawy and Chiang	Empirical Model
1981	Sygula	Semi-Analytical Model
1985	Noakowski	Analytical Model
1986	Janovic and Kupfer	Semi-Analytical Model
1986	Suri and Diliger	Empirical Model
1987	Oh and Kang	Numerical Model
1991	Scholz	Empirical Model
1999	Frosch	Empirical Model
2000	Reynolds and Steedman	Empirical Model
2001	Chowdhury and Loo	Semi-Analytical Model
2016	Debernardi and Taliano	Analytical Model

Table 4.1: Overview of published crack width formulas

4.2.1 1950 – Saliger

In 1950 Saliger published the first formulas for calculating crack widths in reinforced concrete structures. The formulas were developed to comply with concrete with poor mechanical characteristics compared to standards. ($f_{cm} < 15$ MPa). The formula is therefore not applicable on concrete used today since it can yield negative crack width values.

$$W_k = \beta * W_m$$

$$\beta = 2$$

$$W_m = s_m * \varepsilon_{sm} = \left(0{,}157 * \frac{\Phi}{4 * \rho} * \frac{f_{cm}}{\tau_{b,max}}\right) * \left[\frac{\sigma_{s2} - f_{cm} * \left(\frac{0{,}05}{\rho + 2}\right)}{E_s}\right] \tag{4.1}$$

4.2.2 1956 – Clark

Clark developed his formulas for calculating crack widths after gathering data from over 300 experiments conducted on reinforced concrete structures. He developed his formulas by fitting the results from the experiments with already known knowledge on the subject.

$$W_k = \beta * W_m$$

$$\beta = 1{,}64$$

$$W_m = 2{,}27 * 10^{-8} * \frac{h - d}{d} * \frac{\Phi}{\rho} * \left[\sigma_{s2} * 145{,}038 - 56{,}6 * \left(\frac{1}{\rho} + \frac{E_s}{E_c}\right)\right] \tag{4.2}$$

4.2.3 1963 – Kaar and Mattock

Kaar and Mattock published a new set of formulas in 1963, following up on the research conducted by Clark. They conducted several new tests with Clarks equations in mind. Based on their results, they presented their own new set of formulas. The new formulas considered the area of concrete surrounding the reinforcement bars ($A_{c,ef}$) and the stress in reinforcement after cracking (σ_{s2}), which would become important for formulas developed later. They also introduced the coefficient R (Equation (4.4)), which corrects the position of where the width of the crack is being calculated. Instead of calculating width at the level of the reinforcement, the widths would now be calculated at the surface of the concrete.

$$W_k = \beta * W_m$$

$$\beta = 1{,}64$$

$$W_m = 5{,}628 * \sqrt[4]{\frac{A_{c,ef}}{n}} * \sigma_{s2} * R * 10^{-5} \tag{4.3}$$

where:

$$R = \frac{h - x_2}{d - x_2} \tag{4.4}$$

$A_{c,ef} = 2 * b * (h - d)$ (for bending)

$A_{c,ef} = b * h$ (for axial tension)

4.2.4 1965 - Broms

Broms did in 1965 publish a simple formula which calculates the crack width by multiplying the strain in the steel with the spacing of the cracks. Broms also published a new model of stress redistribution for reinforced concrete. Through the model he showed that the maximum crack spacing is equal to the distance between the concrete surface and the center of the reinforcement multiplied by two. When one circles exceeds the tensile strength of the concrete due to tensile stresses, a crack would appear in the structure. Cracks will also appear if the radius of a tensile stress circles will exceed the concrete cover. The cracking mechanism proposed by Broms is illustrated in Figure 4.1.

Broms acknowledged in his paper that the model had some limitations. Equation 4.5 was only applicable for concrete structures with a cover ranging between 32 to 76 mm and that σ_{s2} had to have a value between 138-207 MPa.

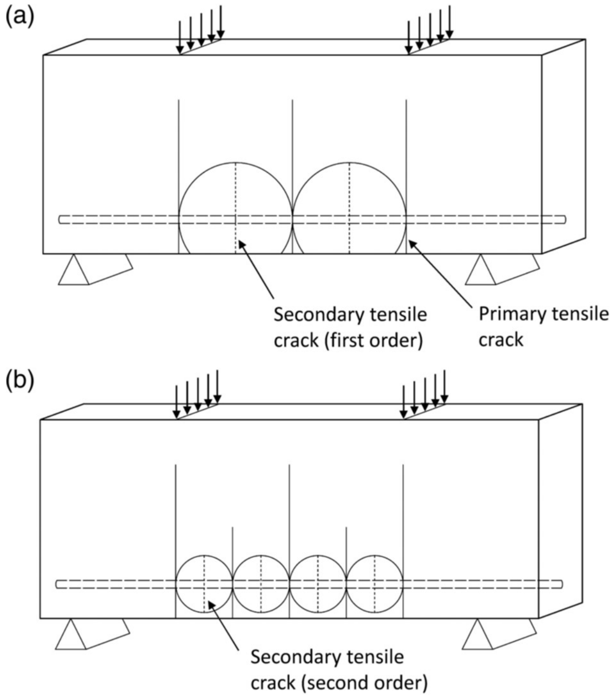

Figure 4.1: Mechanism of tension cracking (a) crack activation, (b) stabilized cracking [6]

$$W_k = \beta * W_m$$

$$\beta = 2$$

$$W_m = s_m * \varepsilon_{sm} = \left[2{,}0 * \left(c + \frac{\Phi}{2}\right)\right] * \frac{\sigma_{s2}}{E_s} \quad (4.5)$$

4.2.5 1965 – Broms and Lutz

During the same year, Broms cooperated with Lutz to improve Broms initial formulas. They introduced the effective cover thickness, t_e, in the formulation of calculating crack widths. The effective cover thickness would replace the original concrete cover, c, to better predict the spacing of the cracks when the distance of the bars was more than 4-5 times bigger than the concrete cover.

$$W_k = \beta * W_m$$

$$\beta = 2$$

$$W_m = s_m * \varepsilon_{sm} = (2{,}0 * t_e) * \frac{\sigma_{s2}}{E_s} \tag{4.6}$$

where:

$$t_e = \sqrt{\left(\frac{a}{4}\right)^2 + (h-d)^2}$$

4.2.6 1966 – Borges

Borges was in 1966 the first person to include the concrete cover c, the bar diameter Φ, and the effective reinforcement ratio (Φ/ρ_{ef}) in the calculation of the crack spacing s_m. As seen in Equation (4.7). This approach would become significant and are still being used todays current code provisions, such as EC2-2004 and Model Code 2010.

$$W_k = \beta * W_m$$

$$\beta = 1{,}66$$

$$W_m = s_m * \varepsilon_{sm} = \left(1{,}5 * c + \frac{0{,}04 * \Phi}{\rho_{ef}}\right) * \left[\frac{1}{E_s} * \left(\sigma_{s2} - \frac{0{,}75}{\rho_{ef}}\right)\right] \tag{4.7}$$

Where:

$A_{c,ef} = b * (h - x_2)$ (for bending)

$A_{c,ef} = b * h$ (for axial tension)

4.2.7 1968 – Gergely and Lutz

Gergely and Lutz developed their own formulas based on an extensive statistical analysis of accumulated experimental data from earlier published papers. The authors identified that the steel stress, σ_{s2}, and the effective concrete area, $A_{c,ef}$, was the two major variables affecting the results for crack width calculation. Through their analysis they also identified how important the coefficient R were, which translates the strain gradient from the level of reinforcement to the concrete surface.

$$W_k = 1{,}1 * 10^{-5} * \sqrt[3]{(h-d) * \frac{A_{c,ef}}{n}} * R * \sigma_{s2} \tag{4.8}$$

where:

$$A_{c,ef} = 2 * b * (h-d)$$

$$R = \frac{h - x_2}{d - x_2}$$

4.2.8 1970 – Holmberg and Lindgren

In 1970 Holmberg and Lindgren published a crack width formula which was focused towards the effects long term- and repeated loads. Through their research they found out that the tension stiffening would reduce, as the loading cycles or loading time would increase, as shown in Figure 4.2. Due to this, they excluded the effect of steel strain from the formulation and instead only focused on the effective concrete surround the reinforcement. Through an extensive statistical analysis of earlier formulas compared to experimental data, they fitted their new equation to better predict the crack spacing.

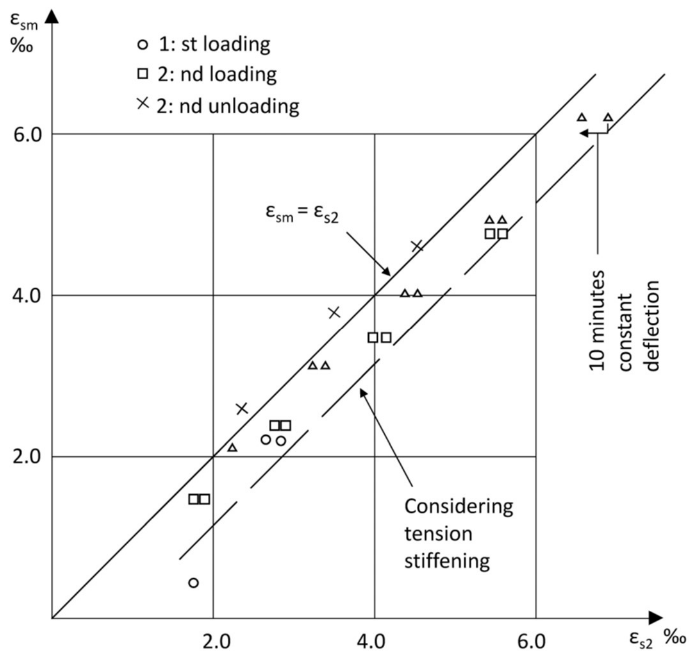

Figure 4.2: Steel strain with/without the contribution of tension stiffening [6]

$$W_k = \beta * W_m$$

$$\beta = 1,70$$

$$W_m = s_m * \varepsilon_{sm} = \left(60 + 0,8 * \sqrt{\frac{c * A_{c,ef}}{\Sigma\Phi}}\right) * \left(\frac{\sigma_{s2}}{E_s}\right) * R \tag{4.9}$$

4.2.9 1977 – Leonhardt

Leonhardt did in 1977 publish an additive formula for crack width calculation. His approach was based on the concept that the bond between concrete and reinforcement is lost in the cracking zone, l_0. See Figure 4.3 The transfer of stresses would therefore only be guaranteed outside of the cracking zone in the concrete, l_{tr}.

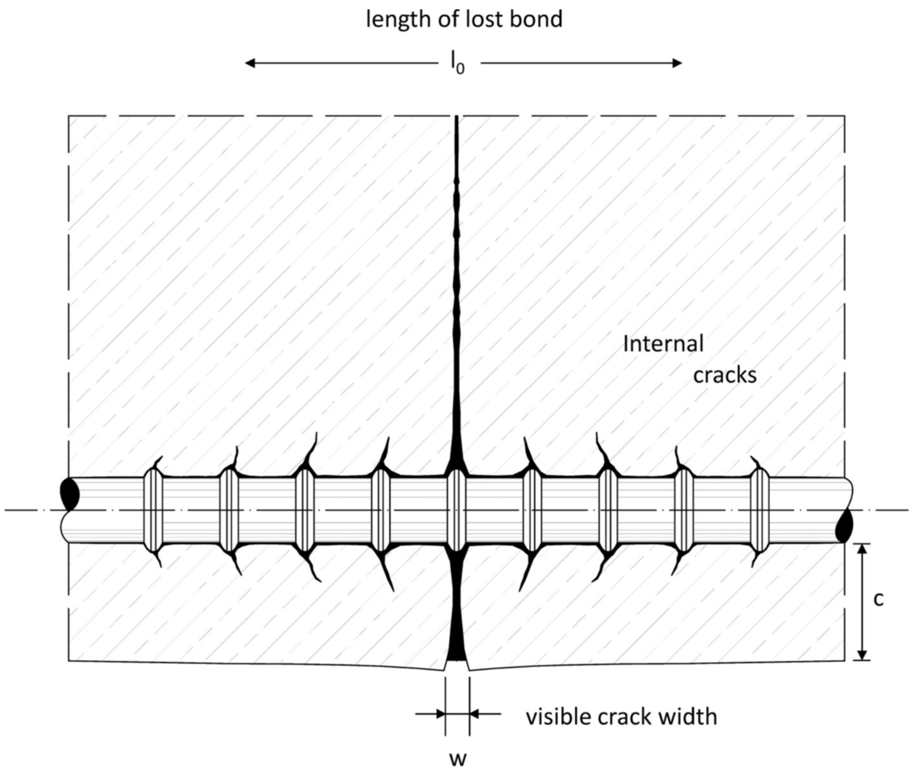

Figure 4.3: Loss of bond in the cracking zone [6]

$$W_k = \beta * W_m$$

$\beta = 1{,}4$ for pure tension,

$\beta = 1{,}6$ for bending.

$$W_m = l_0 * \varepsilon_{s2} + l_{tr} * \varepsilon_{sm} = l_0 * \left(\frac{\sigma_{s2}}{E_s}\right) + l_{tr} * \left[\frac{1}{E_s} * \left(\sigma_{S2} - 0{,}6 * \frac{\sigma_{sr}^2}{\sigma_{s2}}\right)\right] \tag{4.10}$$

$$l_0 = \frac{\sigma_{sr}}{45} * \Phi \tag{4.11}$$

$$l_{tr} = k_1 + 0{,}7 * k_2 * \frac{\Phi}{\rho_{ef}} \tag{4.12}$$

where:

$A_{c,ef} = b * (c + 5 * \Phi)$ bending for slabs,

$A_{c,ef} = b * (c + 7 * \Phi)$ bending for web beam.

k_1 is a coefficient representing spreading-out-length considering cover and bar spacing.

- $k_1 = 1{,}2 * c$ for $a \leq 2 * c$
- $k_1 = 1{,}2 * (c + \frac{a-2*c}{4})$ for $a > 2 * c$

k_2 is a factor depending on the shape of the tensile stress diagram.

- $k_2 = 0{,}25$ for pure tension,
- $k_2 = 0{,}125$ for pure bending.

4.2.10 1979 – Beeby

Beebys analytical approach, which were published in 1979, were completely different compared to that which Leonhardt had earlier provided. Beeby substituted the bond-slip theory with a no-slip theory for the cracked zone of the concrete. In the formula it is assumed that plane sections do not remain plane and that a bond failure between the concrete and reinforcement does not occur at cracking. He also assumed that the distance between crack and the undisturbed part of the concrete were almost equals the concrete cover.

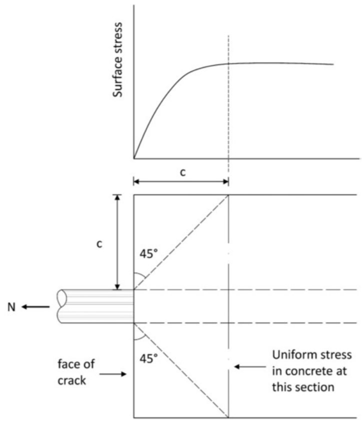

Figure 4.4: No-slip mechanism of cracking [6]

$$W_k = \frac{3 * \varepsilon_m * t_e}{1 + 2 * \left(\frac{t_e - c}{h - x_2}\right)} \tag{4.13}$$

$$\varepsilon_m = \left[\varepsilon_{s2} - \left(\frac{1{,}2 * 10^{-3}}{\rho_{ef} * f_{yk}}\right)\right] * R \tag{4.14}$$

Where:

$$A_{c,ef} = b * h$$

$$t_e = \sqrt{\left(\frac{a}{2}\right)^2 + (h - d)^2} - \frac{\Phi}{2}$$

4.2.11 1980 – Nawy and Chiang

The formulas published by Nawy and Chiang had its focus aimed towards post-tensioned beams. It could also be used for normal reinforced beams, substituting the change of stress in prestressing steel $\Delta\sigma$ with the steel stress σ_{s2}.

$$W_k = 9{,}44 * 10^{-6} * \frac{A_{c,ef}}{\pi * \Sigma\Phi} * \sigma_{s2} * R \tag{4.15}$$

where:

$$A_{c,ef} = 2 * b * (h - d)$$

4.2.12 1981 – Sygula

Sygula published a very simple formula in 1981, where the diameter of the crack width would be directly proportional to the reinforcement ratio of the concrete structure

$$W_k = k_1 * k_2 * \varepsilon_{s2} * 20 * (3{,}5 + 100 * \rho) * \sqrt{\Phi} \tag{4.16}$$

where:

k_1 is a coefficient accounting for the effective stress distribution.

- $k_1 = 1$ for bending and axial tension,
- $k_2 = 1{,}2$ for pure tension.

$k_2$is a coefficient accounting for the load type.

- $k_2 = 1$ for accidental loads
- $k_2 = 1.5$ for permanent loads

4.2.13 1985 – Noakowski

In 1985 Noakowski proposed way of calculating crack widths by managing to solve the differential equation of bond-slip, taking place when the concrete section cracks.

$$W_k = \beta * W_m$$

$$\beta = 1{,}50$$

$$W_m = s_m * \varepsilon_{sm} = 2{,}33 * \left[\frac{\left(\frac{k_1 * f_{ctm}}{\rho}\right)^{0{,}88}}{R_{cm}^{0{,}66}} * \Phi \right]^{0{,}89} * \left[\frac{\left(\sigma_{s2} - 0{,}42 * \frac{k_1 * f_{ctm}}{\rho}\right)}{E_s} \right] \quad (4.17)$$

where:

k_1 is a coefficient accounting for the effective stress distribution.

- $k_1 = 0{,}22$ for bending,
- $k_1 = 0{,}50$ for pure tension.

4.2.14 1986 – Janovic and Kupfer

Unlike previous formulas published, Janovic and Kupfer did in 1986 publish their formula where the crack spacing would be dependent on the reinforcement spacing a. They also considered the tension stiffening by reducing ε_{s2} with 20 %

$$W_k = \beta * W_m$$

$$\beta = 1{,}50$$

$$W_m = s_m * \varepsilon_{sm} = (50 + 0{,}75 * a) * \left(0{,}8 * \frac{\sigma_{s2}}{E_s}\right) \quad (4.18)$$

4.2.15 1986 – Suri and Diliger

The equation which Suri and Diliger proposed in 1986 were specifically meant for partially prestressed concrete beams. They did this by rearranging the formula proposed by Gergely and Lutz, which was published in 1968. They also included a coefficient, k_1, which accounted for different types of reinforcement being used in the concrete structure. The coefficients were worked out through the help of statistical analysis.

$$W_k = \frac{k_1 * \sigma_{s2} * (h-d)}{\sqrt{\rho_{ef}}} \tag{4.19}$$

where:

k_1 is a factor accounting for the type of reinforcements.

- $k_1 = 2{,}55 * 10^{-6}$ for deformed bar-strand,
- $k_1 = 3{,}51 * 10^{-6}$ deformed bar-wire,
- $k_1 = 2{,}56 * 10^{-6}$ for strands only,
- $k_1 = 4{,}50 * 10^{-6}$ for wires only.

$$A_{c,ef} = b * (h - x_2)$$

4.2.16 1987 – Oh and Kang

Oh and Kang based their formula on fracture mechanics. Their numerical model, which was published in 1987, was set up against over more than 700 experimental values and showing good agreement.

$$W_k = \Phi * \left[159 * \left(\frac{h-d}{h-x_2}\right)^{4,5} + 2{,}83 * \left(\frac{A_{c,ef}}{n * A_s}\right)^{\frac{1}{3}}\right] * (\varepsilon_{s2} - 0{,}0002) * R \tag{4.20}$$

where:

$$A_{c,ef} = b * \frac{(h-x_2)^3}{9 * (d-x_2)^2}$$

4.2.17 1991 – Scholz

In 1991 Scholz published a simple empirical model, which was meant for pre-stressed beams and only being used for engineering purposes.

$$W_k = 2{,}4 * 10^{-4} * \frac{\sigma_{s2}}{\rho_{ef}} * R \tag{4.21}$$

where:

$$A_{c,ef} = b * (h - x_1)$$

4.2.18 1999 – Frosch

The formula proposed by Frosh was a direct improvement of Broms and Lutz equation which was published and 1965.

$$W_k = 2 * \frac{\sigma_{s2}}{E_s} * [1 + 0{,}0031 * (h - d)] * t_e \tag{4.22}$$

where:

$$t_e = \sqrt{\left(\frac{a}{2}\right)^2 + (h - d)^2}$$

4.2.19 2000 – Reynolds and Steedman

Reynolds and Steedman based their equation on the formula published by Beeby back in 1979. They rearranged Beeby's equation so that the probability of exceeding the calculated crack width would be reduced from 20 % to 5 %.

$$W_k = \frac{9 * \varepsilon_m * t_e}{2 + 5 * \left(\frac{t_e - c}{h - x_2}\right)} \tag{4.23}$$

$$\varepsilon_m = \varepsilon_{s3} * R - \left(\frac{7 * b * h * 10^{-4}}{A_s * f_y}\right) \tag{4.24}$$

where:

$$t_e = \sqrt{\left(\frac{a}{2}\right)^2 + (h - d)^2} - \frac{\Phi}{2}$$

4.2.20 2001 – Chowdhury and Loo

Chowdhury and Loo proposed a new equation in 2001. The formula was suited for predicting the average crack width for both partially prestressed- and normally reinforced concrete beams. They did not consider the tension stiffening effect in their proposed equation.

$$W_k = \beta * W_m$$

$$\beta = 1{,}50$$

$$W_k = s_m * \varepsilon_{sm} = \left[0{,}60 * (c - a) + 0{,}1 * \frac{\Phi}{\rho}\right] * \left(\frac{\sigma_{s2}}{E_s}\right) \tag{4.25}$$

4.2.21 2016 – Debernardi and Taliano

Debernardi and Taliano did in 2016 publish a new formula for predicting crack widths. The model was an improvement of Model Code 2010. They managed to improve this formula by solving the differential equation of bond-slip. With the help of this solution they rearranged the formula from the model code, by replacing the constant bond capacity with the new variable τ_b.

$$W_k = s_k * \varepsilon_{sm} = 2 * \left(\frac{1}{4} * \frac{f_{ctm}}{\tau_b} * \frac{\Phi}{\rho_{ef}}\right) * \varepsilon_{sm} \tag{4.26}$$

$$\varepsilon_{sm} = \frac{\sigma_{s2} - 0{,}45 * \frac{f_{ctm}}{\rho_{ef}} * \left(1 + \alpha_e * \rho_{ef}\right)}{E_s} \tag{4.27}$$

Where the average bond stress is taken equal to:

$$\tau_b = \frac{\left(f_{ctm}^2 * \frac{\Phi}{\rho_{ef}^2}\right)^{\frac{\alpha}{1+\alpha}}}{4 * \left(\frac{S_1^{\alpha}}{K^{1-\alpha}}\right)^{\frac{1}{1+\alpha}}} \tag{4.28}$$

4.3 Code Formulas

The code provisions presented in [6] are summarized in Table 4.2. The provisions are presented in the following subchapter in chronological order with SI units.

CBE (Chapter 4.3.1) will not be included in the calculations due to yielding unfitting results.

Year published	Code Provision	Publisher
1960	CBE	Commission 4a of the European Concrete Committee.
1966	Cement and Concrete Association	Cement and Concrete Association
1978	CEB-FIP Model Code 1978	Comité Euro-International du Béton
1990	Model Code 1990	Comité Euro-International du Béton
1992	ENV 1992	European Committee for Standardization
2004	EC2 2004	European Committee for Standardization
2007	JSCE	Japan Society of Civil Engineers
2010	Model Code 2010	Fédération Internationale du Béton
2016	Review proposal of EC2	European Committee for Standardization

Table 4.2: Overview of Crack width code provisions

4.3.1 1960 – CBE

In 1960 the European Concrete Committee published a general theory for crack width calculations supposed to represent the literature published up to that point.

$$W_k = \left(4{,}5 + \frac{0{,}40}{\rho_{ef}}\right) * \frac{\Phi * \sigma_{s2}}{327500} \tag{4.29}$$

Where:

$$A_{c,ef} = 2 * b * (h - d)$$

4.3.2 1966 – Cement and Concrete Association

The Cement and Concrete Association published their equation for crack width calculation in 1966. Their equation was based on results from over 100 experimental tests performed on concrete beams.

$$W_k = 3{,}3 * c * \frac{\sigma_{s2}}{E_s} * R \tag{4.30}$$

4.3.3 1978 – CEB-FIP Model Code 1978

One of the first milestone for crack width calculations, seen on an international level, occurred when the Model Code 1978 were proposed by the Comité Euro-International du Béton. This provision sat the premises for crack width calculations for the years to follow, and with minor changes it has also been followed by the latest versions of Eurocodes and Model Codes. This code was also the first code provision which explicitly introducing shrinkage strain ε_{sh} in the formula.

$$W_k = \beta * W_m$$

$$\beta = 1{,}50$$

$$W_m = s_m * (\varepsilon_{sm} - \varepsilon_{sh}) \tag{4.31}$$

$$s_m = 2 * (c * a) + k_1 * k_2 * \frac{\Phi}{\rho_{ef}} \tag{4.32}$$

$$\varepsilon_{sm} = \left[1 - \beta_1 * \beta_2 * \left(\frac{\sigma_{sr}}{\sigma_{s2}}\right)^2\right] * \varepsilon_{s2} \geq 0{,}4 * \varepsilon_{s2} \tag{4.33}$$

where:

k_1 is a coefficient defining the influence of the bond properties of the bars,

- $k_1 = 0{,}4$ for high bond bars,
- $k_1 = 0{,}6$ for ribbed prestressing wires,
- $k_1 = 0{,}8$ for plain bars and plain, indented or crimped prestressing wires.

k_2 is a coefficient depending on the distribution of tensile stress within the section,

- $k_2 = 0{,}125$ for pure bending,
- $k_2 = 0{,}25$ for pure tension.

β_1 is a coefficient accounting for the bond quality of the reinforcing bars,

- $\beta_1 = 1$ for high bond bars,
- $\beta_1 = 0{,}5$ for smooth bars.

β_2 is a coefficient representing the influence of the duration of application or repetition loading,

- $\beta_2 = 1$ for first loading,
- $\beta_2 = 0{,}5$ for repeated loads.

4.3.4 1990 – Model Code 1990

The next version of the Model Code introduced several changes. The crack spacing, S_k, would no longer be dependent on the concrete cover and instead being fitted up against experimental results. This would become problematic for concrete structures with covers distances exceeding covers used in the experiments. A new approach for calculating the steel strain was also proposed, see Equation (4.38), this formula is still being used in today's code provisions.

$$W_k = s_k * (\varepsilon_{sm} - \varepsilon_{sh}) \tag{4.34}$$

$$s_k = \frac{\Phi}{3,6 * \rho_{ef}} \text{ for stabilized cracking} \tag{4.35}$$

$$s_k = \frac{\sigma_{s2}}{2 * \tau_b} * \frac{\Phi}{1 + \alpha_e * \rho_{ef}} \text{ for single crack formation} \tag{4.36}$$

If the criteria in Equation (4.37) is satisfied it is assumed that the condition for stabilized cracking has been meet, and ε_{sm} can thus be calculated with Equation (4.38).

$$\rho_{ef} * \sigma_{s2} > f_{ctm} * (1 + \alpha_e * \rho_{ef}) \tag{4.37}$$

$$\varepsilon_{sm} = \varepsilon_{s2} - \beta^* * \frac{f_{ctm}}{\rho_{ef} * E_s} * \left(1 + \alpha_e * \rho_{ef}\right) \tag{4.38}$$

$$A_{c,ef} = 2,5 * b * (h - d) < b * (h - x_2)/3$$

β^* is a coefficient account for the average strain between two consecutive cracks. See Table 4.3.

	Single crack formation		Stabilized cracking	
	β^*	τ_b	β^*	τ_b
Short term/instantaneous loading	0,6	$1.8\ f_{ctm}$	0,6	$1.8\ f_{ctm}$
Long term/repeated loading	0,6	$1.8\ f_{ctm}$	0,38	$1.8\ f_{ctm}$

Table 4.3: Coefficients for steel strain calculation. Model Code 1990

4.3.5 1992 – ENV 1992

The first formulation published by the European Committee for Standardization was revealed in 1992. It was directly based on Model Code 1978, with only minor changes on the calculation of the crack spacing. See Equation (4.39)

$$W_k = \beta * W_m$$

$$\beta = 1{,}70$$

$$W_m = s_m * \varepsilon_{sm}$$

$$s_m = 50 + 0{,}25 * k_1 * k_2 * \frac{\Phi}{\rho_{ef}} \tag{4.39}$$

$$\varepsilon_{sm} = \left[1 - \beta_1 * \beta_2 * \left(\frac{\sigma_{sr}}{\sigma_{s2}}\right)^2\right] * \varepsilon_{s2} \tag{4.40}$$

where:

k_1 is a coefficient defining the influence of the bond properties of the bars,

- $k_1 = 0{,}8$ for high-bond bars,
- $k_1 = 1{,}6$ for smooth bars

k_2 is a coefficient depending on the distribution of tensile stress within the section,

- $k_2 = 0{,}5$ for pure bending,
- $k_2 = 1$ for pure tension.

β_1 is a coefficient accounting for the bond quality of the reinforcing bars,

- $\beta_1 = 1$ for high bond bars,
- $\beta_1 = 0{,}5$ for smooth bars.

β_2 is a coefficient representing the influence of the duration of application or repetition loading,

- $\beta_2 = 1$ for first loading,
- $\beta_2 = 0{,}5$ for repeated loads.

$$A_{c,ef} = 2{,}5 * b * (h - d) < b * (h - x_2)/3$$

4.3.6 2004 – EC2 2004

The second provision from the European Committee for Standardization was published in 2004 and is still being used today as part of the current Eurocode 2. The formula set is based on both Model Code 1978 and Model Code 1990. Where the crack spacing formula, s_k, is calculated according to Model Code 1978 with minor changes. The steel strain, ε_{sm}, is calculated according to Model Code 1990.

$$W_k = s_k * \varepsilon_{sm}$$

$$s_k = 3{,}4 * c + 0{,}425 * k_1 * k_2 * \frac{\Phi}{\rho_{ef}} \tag{4.41}$$

$$\varepsilon_{sm} = \varepsilon_{s2} - \beta^* * \frac{f_{ctm}}{\rho_{ef} * E_s} * \left(1 + \alpha_e * \rho_{ef}\right) \geq 0{,}6 * \varepsilon_{s2} \tag{4.42}$$

where:

k_1 is a coefficient defining the influence of the bond properties of the bars,

- $k_1 = 0{,}8$ for high-bond bars
- $k_1 = 1{,}6$ for smooth bars

k_2 is a coefficient depending on the distribution of tensile stress within the section,

- $k_2 = 0{,}5$ for pure bending
- $k_2 = 1$ for pure tension.

β^* is a coefficient accounting for the average strain between two consecutive cracks,

- $\beta^* = 0{,}6$ for short term and instantaneous loading
- $\beta^* = 0{,}4$ for long term loads.

$$A_{c,ef} = 2{,}5 * b * (h - d) < b * (h - x_2)/3$$

4.3.7 2007 – JSCE

The formulas published by the Japan Society of Civil Engineers in 2007 was rather simple and is not being used as much in Europe.

$$W_k = s_k * (\varepsilon_{sm} - \varepsilon_{sh})$$

$$s_k = 1{,}1 * c * k_1 * k_2 * k_3 * [4 * c + 0{,}7 * (a - \Phi)] \tag{4.43}$$

$$\varepsilon_{sm} = \frac{\sigma_{s2}}{E_s} \tag{4.44}$$

where:

$k_1$is a coefficient accounting for the bond properties of the bars,

- $k_1 = 1$ for deformed bars
- $k_1 = 1{,}3$ for plain bars and prestressing steel

$$k_2 = \frac{15}{f_{cd} + 20} + 0{,}7$$

$$k_3 = \frac{5 * (n_l + 2)}{7 * n_l + 8}$$

$n_l$is the number of tensile reinforcement layers

4.3.8 2010 – Model Code 2010

Model Code 2010 introduced a new way of calculating the crack width spacing, $2 * l_{tr}$, by solving the differential bond-slip equation and assuming constant bond stress between the reinforcement and concrete. This version also reintroduced the dependence of concrete cover in the crack spacing calculation, while the steel strain, ε_{sm}, is still calculated according to Model Code 1994

$$W_k = 2 * l_{tr} * (\varepsilon_{sm} - \eta_r * \varepsilon_{sh}) \tag{4.45}$$

$$l_{tr} = c + \frac{1}{4} * \frac{f_{ctm}}{\tau_b} * \frac{\Phi}{\rho_{ef}} \tag{4.46}$$

$$\varepsilon_{sm} = \varepsilon_{s2} - \beta^* * \frac{f_{ctm}}{\rho_{ef} * E_s} * \left(1 + \alpha_e * \rho_{ef}\right) \tag{4.47}$$

where:

β^* is a coefficient account the average strain between two consecutive cracks (Table 4.4)

η_r is a coefficient for considering the shrinkage contribution.

$$A_{c,ef} = 2{,}5 * b * (h - d) < b * (h - x_2)/3$$

	Single crack formation			Stabilized cracking		
	β^*	η_r	τ_b	β^*	η_r	τ_b
Short term/instantaneous loading	0,6	0	$1.8\ f_{ctm}$	0,6	0	$1.8\ f_{ctm}$
Long term/repeated loading	0,6	0	$1.35\ f_{ctm}$	0,4	1	$1.8\ f_{ctm}$

Table 4.4: Coefficients for steel strain calculation. Model Code 2010

4.3.9 2016 – Review proposal of EC2

The review proposal for Eurocode 2 published in 2016 were based on the provisions from the current Eurocode, with minor changes. The new introduction in this proposal is that the shrinkage strain from the concrete is included for the calculations.

$$W_k = s_k * (\varepsilon_{sm} - \eta_r * \varepsilon_{sh}) \tag{4.48}$$

$$s_k = 2{,}0 * c + 0{,}35 * k_1 * \frac{\Phi}{\rho_{ef}} \tag{4.49}$$

$$\varepsilon_{sm} = \varepsilon_{s2} - \beta^* * \frac{f_{ctm}}{\rho_{ef} * E_s} * \left(1 + \alpha_e * \rho_{ef}\right) \geq 0{,}6 * \varepsilon_{s2} \tag{4.50}$$

where:

k_1 is a coefficient defining the influence of the bond properties of the bars,

- $k_1 = 0{,}8$ for high-bond bars
- $k_1 = 1{,}6$ for smooth bars

$$A_{c,ef} = 2{,}5 * b * (h - d) < b * (h - x_2)/3$$

5 Calculations

5.1 Slab Setup

The calculations are conducted on a reinforced concrete slab. The slab has a length equal to 12 meter and is 9 meter wide. The concrete quality used is C30/37 and the quality of the reinforcement used is B500NC. Reinforcement are placed near both faces of the slab.

Dimensions	
Length	12 meters
Breadth	9 meters
Thickness	0,2 meters

Table 5.1: Dimensions for the concrete slab

Material Properties	
$\boldsymbol{f_{ck}}$	$30\ N/mm^2$
$\boldsymbol{f_{ctm}}$	$2{,}9\ N/mm^2$
$\boldsymbol{E_c}$	$30\ 000\ N/mm^2$
$\boldsymbol{f_{yk}}$	$500\ N/mm^2$
$\boldsymbol{E_s}$	$200\ 000\ N/mm^2$

Table 5.2: Material properties

The slab has been modelled in FEM-Design, which have been used to obtain σ_{s2} which is a crucial parameter for the calculations. The shrinkage contribution in the concrete has been neglected for these calculations. This due to shrinkage not being included as a calculation parameter for most of the formulas, except for some of the code provisions. To achieve a better comparison between the results it was therefor decided to exclude shrinkage.

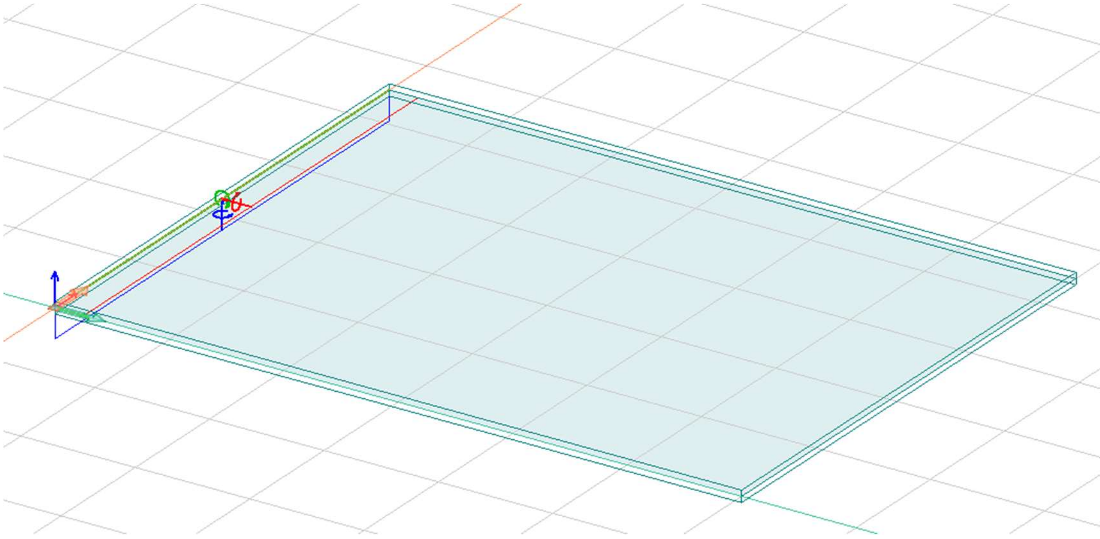

Figure 5.1: Reinforced concrete slab

5.2 Tension Loading Case

For the calculations in this thesis it has been decided to aim our focus towards axial tension. To achieve this, the reinforced concrete slab is restrained with a line support group at one of its edges. The support group restrains the edge in all of the principal directions. At the other edge, a line load is applied. The loading magnitude for the line load is adjusted depending on the reinforcement amount placed the cross-section. The dead weight of the construction is not considered. Which means we will get a structure that is only affected by axial tension.

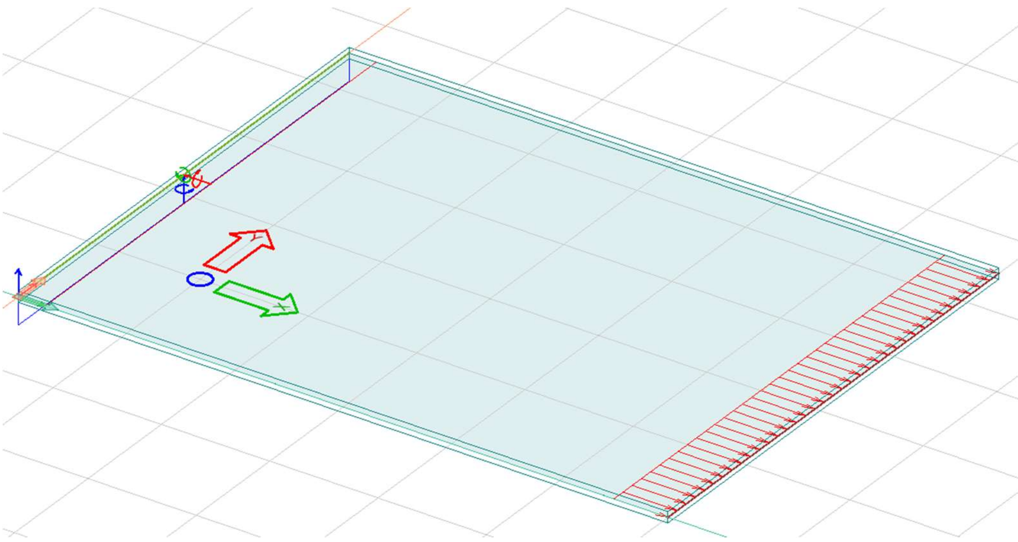

Figure 5.2: Reinforced concrete slab with line load

The line loading has been setup this way to facilitate similar effects of what a slab would endure when it is subjected to restrain. One could imagine that there is another line support group placed at the edge where the current line loading is placed. Instead a line loading is placed there to simulate the restraint forces a support would induce on the structure if for it for example were exposed to shrinkage strains.

The crack widths are calculated on each face of the slab on a 1-meter width of the slab. Both faces achieve the same crack width due to the loading being evenly distributed between the faces. As mentioned earlier, σ_{s2}, the stress in the reinforcement after cracking, is important for the calculations. It is also used to depict the capacity of the structure for these calculations. Which means that when the steel reaches $500\ MPa$, which is the yield threshold value for the reinforcement, we assume that the structure have reached its capacity.

5.3 Calculation Cases

In total twelve cases have been evaluated and calculated. These twelve cases are separated into four categories, where the results for each category is trying to show what impact their differences will have towards the crack width. The categories and the setup for each case is summarized in Table 5.3.

	Rebars	Reinf. area [mm^2]	Cover [mm]	Height [mm]
Varying rein. layout	Ø18/130 mm	1957	50	200
	Ø16/103 mm	1952	50	200
	Ø14/79 mm	1949	50	200
Varying concrete cover	Ø18/130 mm	1957	50	200
	Ø18/130 mm	1957	40	200
	Ø18/130 mm	1957	30	200
Varying concrete thickness	Ø18/130 mm	1957	50	200
	Ø18/130 mm	1957	50	300
	Ø18/130 mm	1957	50	400
Increasing bar diameter	Ø18/130 mm	1957	50	300
	Ø25/130 mm	3776	50	300
	Ø32/130 mm	6187	50	300

Table 5.3: Overview of setup for the calculations cases

The cracks which occur due to loading are evenly spread depending on crack width distance in FEM-Design, as can be seen in Figure 5.3.

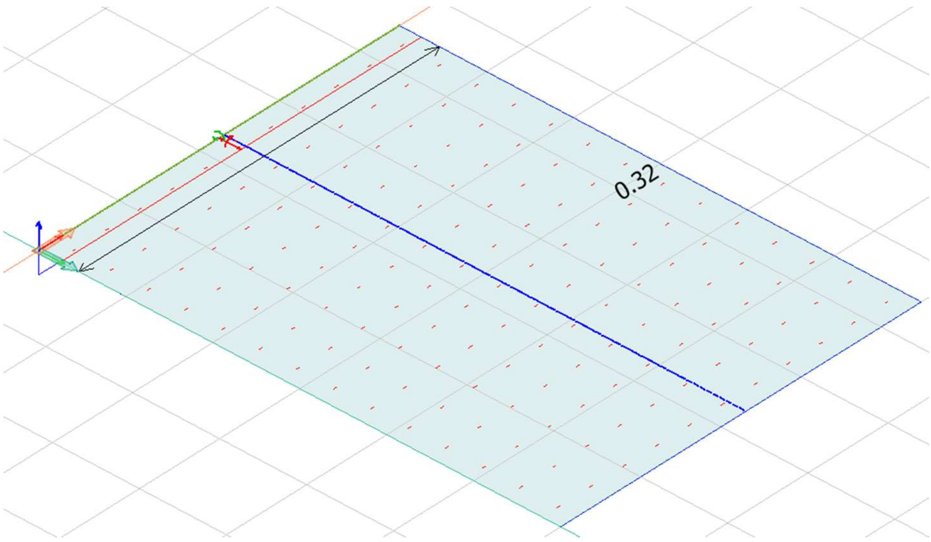

Figure 5.3: Example of a cracked concrete slab

6 Discussion

6.1 Introduction

The discussion in this thesis is based on the results from a own produced excel sheet, which includes all the formulas for crack width calculations presented in Chapter 4. In total, twelve cross-sections were assessed, were cracks occurs due to axial tension. The setups were compared towards each other in four main categories, as listed below.

- Varying reinforcement layout. Reinforcement area kept constant.
- Varying concrete cover to reinforcement.
- Varying concrete thickness
- Increasing bar diameter

The examples in the discussion are for the most part using the calculations based on Eurocode 2. This is done due to Eurocode being that code provision being used by the majority for designing reinforced concrete structure.

For the results, the crack widths are compared towards N/N_{max}. Where N is the tension load applied to the 1-meter width cross-section, and N_{max} is the axial tension capacity of the cross-section. Since the axial force is applied on the whole height of the cross-section and cracks can occur on both faces of the structure, which have reinforcement placed near them, we have to multiply the capacity with 2.

One curious observation seen when conducting the calculations is the high scatter between the results. Since the formulas are differently arranged is to be expected some variances in the results.

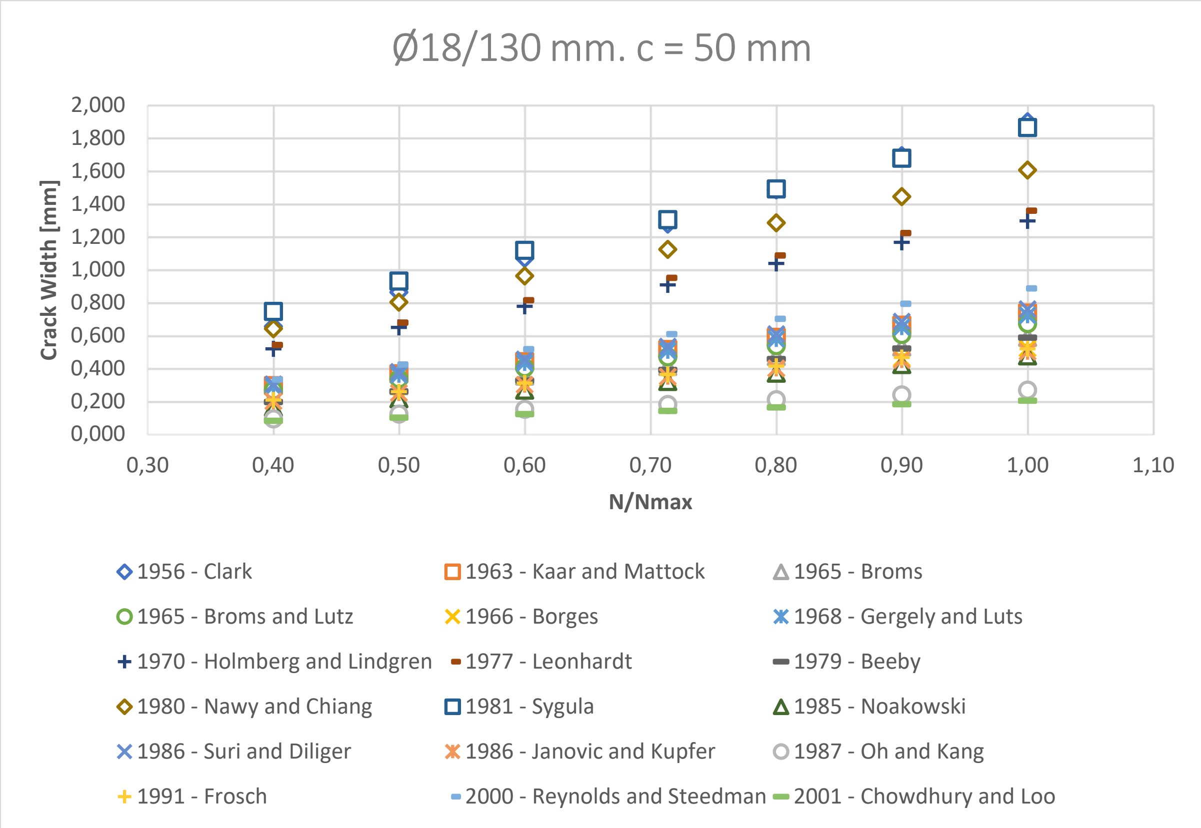

Figure 6.1: Example of high scattering in results

Another important observation has been how hard it is to stay within the crack width limitations, as discussed in Chapter 3.2. For many of the calculation cases, the crack width limitation set by Eurocode 2 are exceeded when N/N_{max} passes 0,5. This means that construction still has capacity towards mechanical loading. But will fall short, due to severe cracking opening up for more rapid deterioration of the construction.

6.2 Reinforcement Layout

For the category for varying reinforcement layouts 3 cases are evaluated. The reinforcement amount is almost equal for each case, but the bar diameter and the bar spacing is varying. All other parameters, such as cover thickness and concrete height have been kept constant for all cases.

- Ø18/130 mm. $A_s = 1957\ mm^2$
- Ø16/103 mm. $A_s = 1952\ mm^2$
- Ø14/79 mm. $A_s = 1942\ mm^2$

N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
Ø18/130	0,288	0,359	0,431	0,503	0,575	0,647	0,719
Ø16/103	0,270	0,338	0,406	0,473	0,541	0,608	0,676
Ø14/79	0,253	0,316	0,380	0,443	0,506	0,569	0,632

Table 6.1: Comparison of crack width when varying the reinforcement layout. EC2

As can be seen in Table 6.1, the crack widths tend to be lower for layouts with small bar diameters and shorter reinforcement spacing. Even though the effective reinforcement ratio in the cracking region, ρ_{ef}, is nearly the same for all 3 cases, and the loading also are kept the same. This is due to the spacing of the cracks are dependent on the ratio between bar diameter and the effective reinforcement ratio, Φ/ρ_{ef}. Even though the reinforcement diameter is smaller, the area of the reinforcement is kept constant. Which means that Φ/ρ_{ef} yields a smaller value. This ratio is used in the crack spacing calculations for both the Eurocode 2 (Equation 4.41) and Model Code 2010 (Equation 4.46).

This observation would indicate that keeping the bar diameters low and space the bars closely would keep the cracks at a moderate level. But then again you could run into issues where the bars are too closely spaced and a lot more labour has to be conducted on the structure.

6.3 Concrete Cover Distance

The cover distance from the reinforcement level to the surface of the concrete also makes a contribution to the predictions of crack widths through calculations. In this category 3 new cases were assessed. Where the only varying parameter was the cover distance. Everything else, such as reinforcement area, height of cross-section and reinforcement layout were kept the same.

- Ø18/130 mm. $c = 50\ mm$
- Ø18/130 mm. $c = 40\ mm$
- Ø18/130mm. $c = 30\ mm$

N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
c = 50 mm	0,288	0,359	0,431	0,503	0,575	0,647	0,719
c = 40 mm	0,262	0,327	0,393	0,458	0,523	0,589	0,654
c = 30 mm	0,236	0,295	0,354	0,413	0,472	0,531	0,590

Table 6.2: Comparison of crack width when increasing the cover thickness. EC2

Table 6.2: Comparison of crack width when increasing the cover thickness. EC2Table 6.2 summarizes the results from the Eurocode 2 calculations. We can here clearly see that small covers yields smaller crack widths. The difference becomes even more significant when the loading becomes greater. The concrete cover distance makes a direct contribution in the calculation of crack width spacings. As again can be seen in Equation 4.41 for Eurocode 2 calculations and Equation 4.46 for Model Code 2010 calculations. In the equations we see that small cover distances will produce low crack width distances, which again contributes to small crack width diameters.

This observation would indicate that by having small cover distances in the concrete cross-section you can achieve structures with low crack width diameters. Unfortunately, you would then run into problems concerning the durability of the concrete structure. Due to corrosion of reinforcement being

6.4 Concrete Thickness

One of the bigger observations made during the calculations were that the thickness of the concrete slab would make a big impact on the crack width results. For this category, the reinforcement layout Ø18/130 mm was once again used. With a cover distance equal to 50 mm. The height of the cross section was raised, ranging from 200- to 400 mm divided into three cases.

- Ø18/130 mm. $H = 200\ mm$
- Ø18/130 mm. $H = 300\ mm$
- Ø18/130 mm. $H = 400\ mm$

N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
H =200 mm	0,288	0,359	0,431	0,503	0,575	0,647	0,719
H = 300 mm	-	0,443	0,550	0,642	0,734	0,825	0,917
H = 400 mm	-	-	-	0,774	0,892	1,003	1,115

Table 6.3: Comparison of crack widths for increasing concrete thickness EC2

As we can see from the results in Table 6.3, a greater axial loading is needed to crack the structures with higher thicknesses. A problem which then arises is that the width of the cracks occurring are rather high. This is due to effective reinforcement ratio, ρ_{ef}, becoming lower when the height of the cross-section is increased. At the same time, the reinforcement area and the bar diameter are kept constant. This means that more loading is transferred to the reinforcement after cracking has occurred.

The quality of the concrete used for the calculations have been C30/37. This concrete quality has a tensile strength capacity, f_{ct}, equal to 2,9 MPa. Which means that when the thickness of the concrete cross-section is higher, its cracking threshold also raises. As shown in the calculations below.

$$Crack\ treshhold = f_{ct} * A_c$$

$H = 200\ mm$: $\quad 2{,}9\ MPa * 1000\ mm * 200\ mm = 580\ kN$

$H = 300\ mm$: $\quad 2{,}9\ MPa * 1000\ mm * 300\ mm = 870\ kN$

$H = 400\ mm$: $\quad 2{,}9\ MPa * 1000\ mm * 400\ mm = 1160\ kN$

It necessary to place more reinforcement in the cross-section to maintain crack widths at a moderate level when having a greater concrete thickness. This would then not be optimal seen from a cost perspective, due to that more reinforcement being needed to withstand the same loading compared to a cross-section with lower thickness.

Another problem one may run in to when having concrete slabs with high thicknesses is that the transferred forces from the concrete to reinforcement may be severe at cracking. This means that the reinforcement may instantly yield when the cracking occurs. This will also leave us with bigger crack widths, since the reinforcement cannot take any more loading. In the end this leaves us with structure which has exceeded its capacity.

6.5 Increasing Bar Diameter

The last category evaluated was the influence of increasing bar diameters. This increase leads to a higher reinforcement area for the cross-section, which again raises the tension capacity of the structure. The spacing of the bars are kept constant for all the cases. To make the comparison optimized it was decided to keep the axial loading equal for all the cases in this category.

- Ø18/130 mm. $A_s = 1957\ mm^2$
- Ø25/130 mm. $A_s = 3776\ mm^2$
- Ø32/130 mm. $A_s = 6187\ mm^2$

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Ø18/130	-	0,443	0,550	0,642	0,734	0,825	0,917
Ø25/130	-	-	0,233	0,272	0,311	0,350	0,389
Ø32/130	-	-	0,120	0,148	0,166	0,187	0,208

Table 6.4: Comparison of crack widths for increasing bar diameter EC2

The results in Table 4.1 shows us that the increase of bar diameters directly influence the size of the crack width. By applying reinforcement bars with bigger diameter the effective reinforcement ratio, ρ_{ef}, gets increased. This increase will influence ε_{sm}, lowering the result due to the placement of ρ_{ef} in the denominator. This can be seen in Equation 4.41 for Eurocode 2 and Equation 4.47 for Model Code 2010.

Another interesting observation in this category for calculations is the development of steel stress in the reinforcement after cracking. As can been seen for the three cases in Figure 6.2. As the loading progressively increases the steel stress does the same. The slope on the other hand is different for each loadout. The higher bar diameter is, the slower the steel stress increases. The initial steel stress after cracking is also much lower for higher bar diameters, which means that they yield much lower crack width results, as seen in Table 6.4.

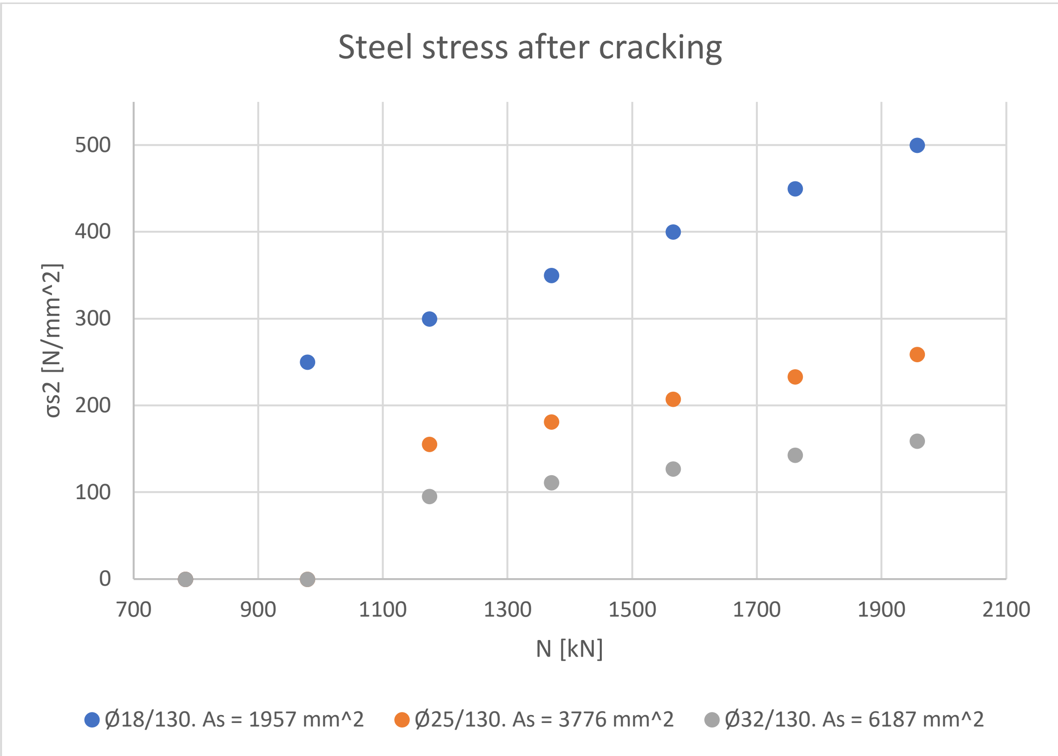

Figure 6.2: Steel stress after cracking

The increase of bar sizes is though not necessarily a good idea. Given this case for example where the spacing of the bars are kept constant. The increased bar diameter will then take up much more place in the construction, making it harder to assemble the reinforcement before casting the concrete. It is also harder to handle bigger reinforcement bars at the construction site.

6.6 Minimum Reinforcement

An observation which were made during the calculations were the great amounts of reinforcement needed to contain the crack widths at a moderate level. The minimum required reinforcement for the slab used in the calculations are around $500\ mm^2$, according to Eurocode 2 and FEM-Design.

For example, when having a reinforcement layout equal to Ø18/130 mm. The reinforcement area on one face for a 1-meter strip slab equals to $1957\ mm^2$. This is almost four times higher than the required minimum reinforcement. In Table 6.5 we can observe that already at $N/N_{max} = 0{,}4$ that the crack width is getting at a impactful point.

N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1
Ø18/130 mm	0,288	0,359	0,431	0,503	0,575	0,647	0,719

Table 6.5: Loading case example

This could suggest that the crack width calculations could be dominant when deciding which reinforcement layout the structure should have. If not properly handled, you could run into a situation where the

7 Summary and Conclusion

As presented in this thesis, there have been a great development for calculating crack widths in reinforced concrete structures during the last couple of decades. The formulas presented are varying in complexity, where some are easily formulated and are based on coefficients derived through experimental works. While others are more complex, where the goal have been to stay as analytical as possible.

The calculations conducted have shown that there are many obstacles which needs to be dealt with while designing reinforced concrete structures with respect to crack widths. Overall, 4 categories for the calculations have been conducted. Where the cases consists of varying reinforcement layout, different cover distances, different concrete thicknesses and increasing bar diameters.

By having reinforcement bars with small diameters and short distances between the bars one can achieve lower crack widths for the structure. This may though lead to closely spaced reinforcement layouts which are not achievable at the construction site.

Smaller cover distances between the reinforcement and the concrete surface does also have a positive contribution on the calculations, yielding lower crack width values. But this decrease makes the reinforcement less protected since there are less concrete between itself and the surroundings. Which means deterioration processes on the structure may take place earlier than expected.

One of the more interesting findings have been how significant the thickness of the concrete section is towards crack propagation. When having thick cross-sections, the tensile capacity of the concrete becomes greater. Which means that more loading has to be applied on the structure before it starts to crack. When the crack occurs, the loading amount gets transferred over to the reinforcement. If the amount of reinforcement is not sufficient enough, then the crack widths will become severe.

As seen in the results, the best way to limit the crack width sizes is to use reinforcement with a big bar diameter. When the tensile capacity of the concrete is exceeded. The bigger bars will handle the loading better, producing crack widths which are within the limits. As can be seen for the case having a Ø32/130 mm layout. If this achievable at the construction site is another matter.

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Appendix

A Varying Reinforcement Layout

A.1 Ø18/130 mm

N [kN]	782,8	978,5	1174,2	1396,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,40	0,50	0,60	0,7	0,8	0,9	1,0
1956 - Clark	0,655	0,863	1,071	1,279	1,487	1,694	1,902
1963 - Kaar and Mattock	0,295	0,369	0,442	0,516	0,590	0,664	0,737
1965 - Broms	0,236	0,295	0,354	0,413	0,472	0,531	0,590
1965 - Broms and Lutz	0,269	0,337	0,404	0,471	0,539	0,606	0,673
1966 - Borges	0,152	0,214	0,275	0,337	0,399	0,460	0,522
1968 - Gergely and Luts	0,290	0,362	0,435	0,507	0,579	0,652	0,724
1970 - Holmberg and Lindgren	0,519	0,649	0,779	0,909	1,038	1,168	1,298
1977 - Leonhardt	0,542	0,679	0,815	0,951	1,088	1,224	1,360
1979 - Beeby	0,196	0,261	0,327	0,392	0,457	0,522	0,587
1980 - Nawy and Chiang	0,643	0,803	0,964	1,125	1,285	1,446	1,607
1981 - Sygula	0,746	0,933	1,120	1,306	1,493	1,679	1,866
1985 - Noakowski	0,163	0,216	0,268	0,320	0,373	0,425	0,477
1986 - Suri and Diliger	0,304	0,380	0,456	0,532	0,608	0,684	0,760
1986 - Janovic and Kupfer	0,201	0,251	0,301	0,351	0,401	0,451	0,501
1987 - Oh and Kang	0,093	0,122	0,151	0,180	0,209	0,239	0,268
1991 - Frosch	0,208	0,260	0,311	0,363	0,415	0,467	0,519
2000 - Reynolds and Steedman	0,332	0,425	0,517	0,610	0,702	0,795	0,887
2001 - Chowdhury and Loo	0,082	0,102	0,123	0,143	0,163	0,184	0,204

Table A 1: Literature formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

N [kN]	782,8	978,5	1174,2	1396,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1
1960 - CBE	4,765	4,831	4,898	4,964	5,030	5,096	5,163
1966 - Cement and Concrete Association	0,234	0,292	0,351	0,409	0,468	0,526	0,585
1978 - Model Code 1978	0,713	0,893	1,072	1,252	1,431	1,610	1,789
1990 - Model Code 1990	0,138	0,181	0,223	0,266	0,309	0,351	0,394
1992 - ENV 1992	0,292	0,366	0,439	0,513	0,586	0,660	0,733
2004 - EC2 2004	0,288	0,359	0,431	0,503	0,575	0,647	0,719
2007 - JSCE	0,306	0,383	0,459	0,536	0,612	0,689	0,765
2010 - Model Code 2010	0,207	0,275	0,342	0,410	0,478	0,545	0,613
2016 - Review proposal of EC2	0,208	0,276	0,344	0,412	0,480	0,548	0,616

Table A 2: Code formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

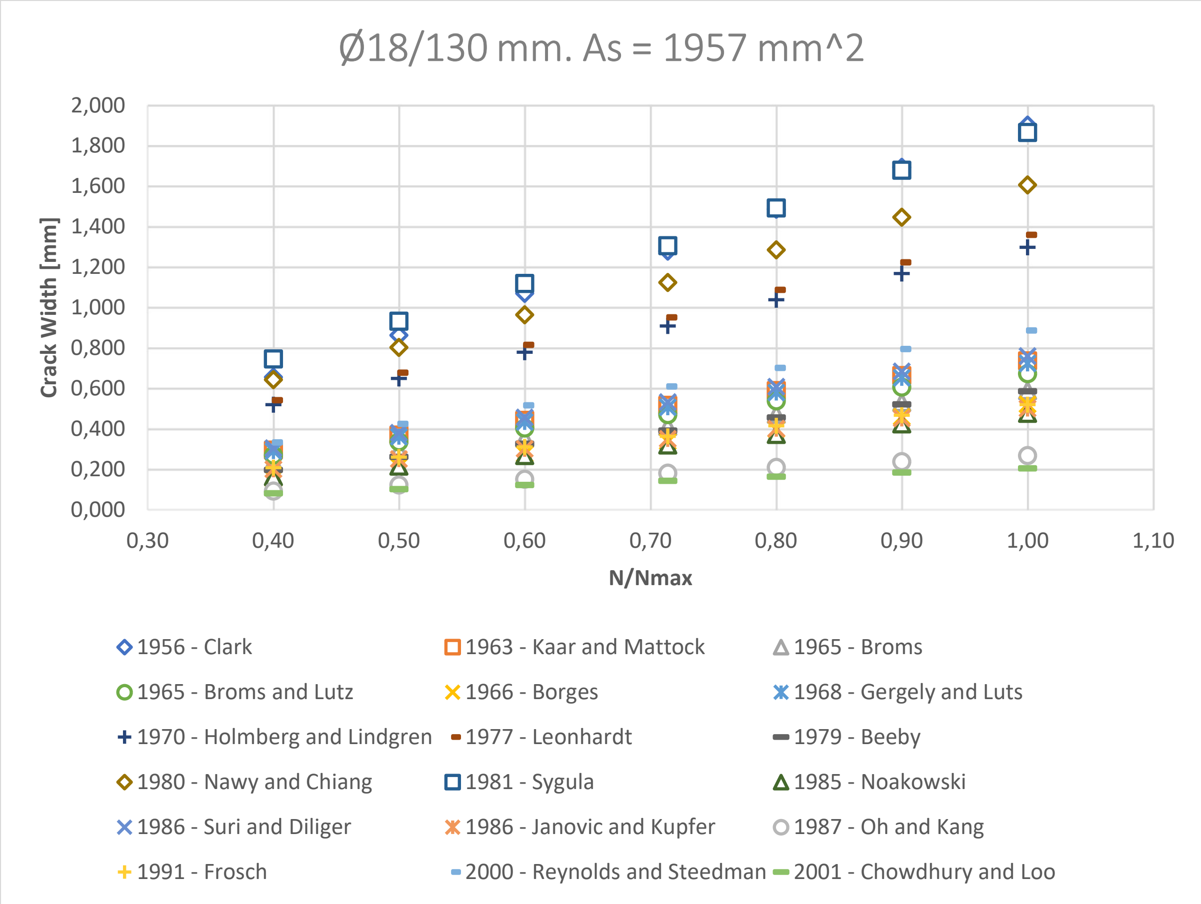

Figure A 1: Literature formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

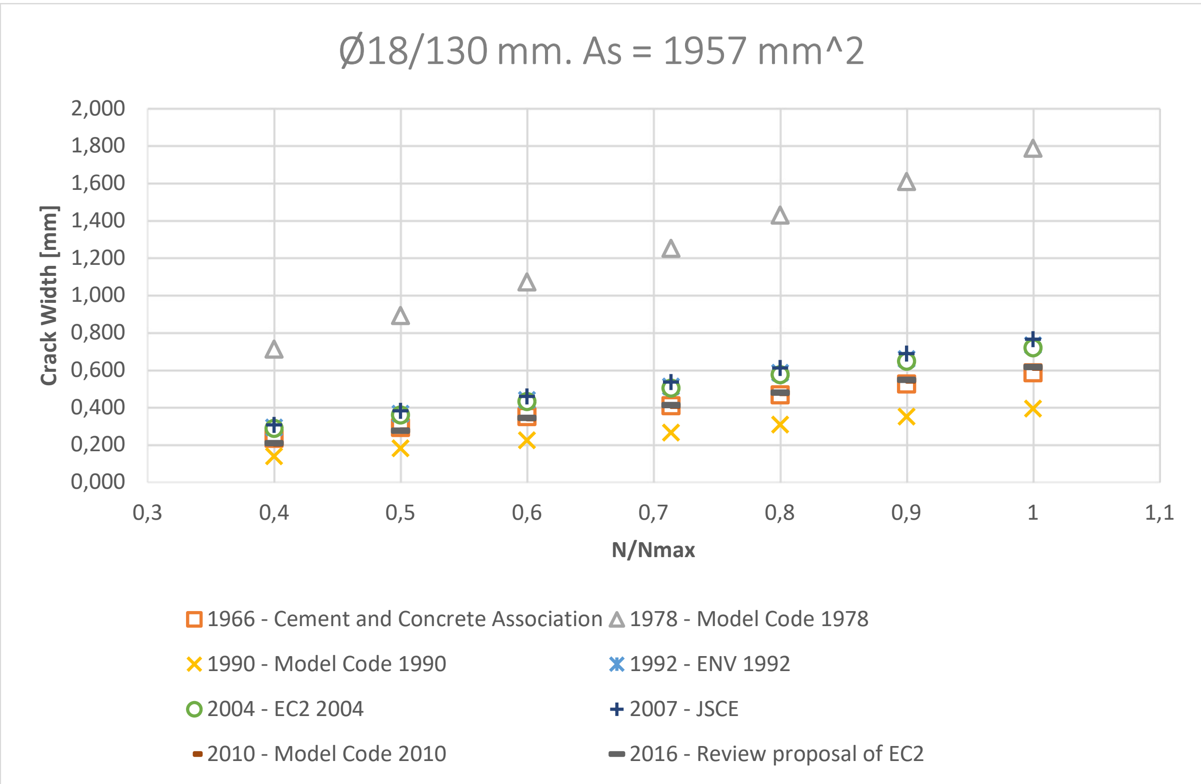

Figure A 2: Code formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

A.2 Ø16/103 mm

N [kN]	780,8	976	1171,2	1336,4	1561,6	1756,8	1952
Nmax (σs2=500 Mpa)	1952	1952	1952	1952	1952	1952	1952
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
1956 - Clark	0,434	0,563	0,691	0,819	0,948	1,076	1,205
1963 - Kaar and Mattock	0,278	0,347	0,417	0,486	0,556	0,625	0,695
1965 - Broms	0,232	0,290	0,348	0,406	0,464	0,522	0,580
1965 - Broms and Lutz	0,254	0,317	0,381	0,444	0,508	0,571	0,635
1966 - Borges	0,144	0,202	0,260	0,319	0,377	0,435	0,494
1968 - Gergely and Luts	0,265	0,332	0,398	0,464	0,531	0,597	0,663
1970 - Holmberg and Lindgren	0,496	0,620	0,744	0,868	0,992	1,116	1,240
1977 - Leonhardt	0,492	0,616	0,740	0,864	0,988	1,111	1,235
1979 - Beeby	0,185	0,247	0,308	0,370	0,431	0,493	0,554
1980 - Nawy and Chiang	0,573	0,716	0,860	1,003	1,146	1,289	1,433
1981 - Sygula	0,702	0,877	1,053	1,228	1,404	1,579	1,755
1985 - Noakowski	0,148	0,195	0,243	0,290	0,338	0,385	0,433
1986 - Suri and Diliger	0,299	0,374	0,449	0,524	0,599	0,674	0,749
1986 - Janovic and Kupfer	0,173	0,216	0,260	0,303	0,346	0,389	0,433
1987 - Oh and Kang	0,076	0,100	0,124	0,148	0,172	0,196	0,219
1991 - Frosch	0,183	0,229	0,275	0,320	0,366	0,412	0,458
2000 - Reynolds and Steedman	0,318	0,407	0,495	0,584	0,672	0,761	0,850
2001 - Chowdhury and Loo	0,085	0,106	0,127	0,148	0,169	0,190	0,211

Table A 3: Literature formulas. Ø16/103 mm. h = 200 mm. c = 50 mm

N [kN]	780,8	976	1171,2	1336,4	1561,6	1756,8	1952
Nmax (σs2=500 Mpa)	1952	1952	1952	1952	1952	1952	1952
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
1960 - CBE	4,732	4,790	4,848	4,906	4,965	5,023	5,081
1966 - Cement and Concrete Association	0,232	0,290	0,349	0,407	0,465	0,523	0,581
1978 - Model Code 1978	0,611	0,764	0,918	1,072	1,225	1,378	1,532
1990 - Model Code 1990	0,123	0,161	0,199	0,237	0,275	0,313	0,351
1992 - ENV 1992	0,270	0,338	0,405	0,473	0,541	0,609	0,677
2004 - EC2 2004	0,270	0,338	0,406	0,473	0,541	0,608	0,676
2007 - JSCE	0,287	0,359	0,430	0,502	0,574	0,646	0,717
2010 - Model Code 2010	0,193	0,256	0,319	0,382	0,445	0,508	0,571
2016 - Review proposal of EC2	0,194	0,257	0,320	0,384	0,447	0,510	0,573

Table A 4: Code formulas. Ø16/103 mm. h = 200 mm. c = 50 mm

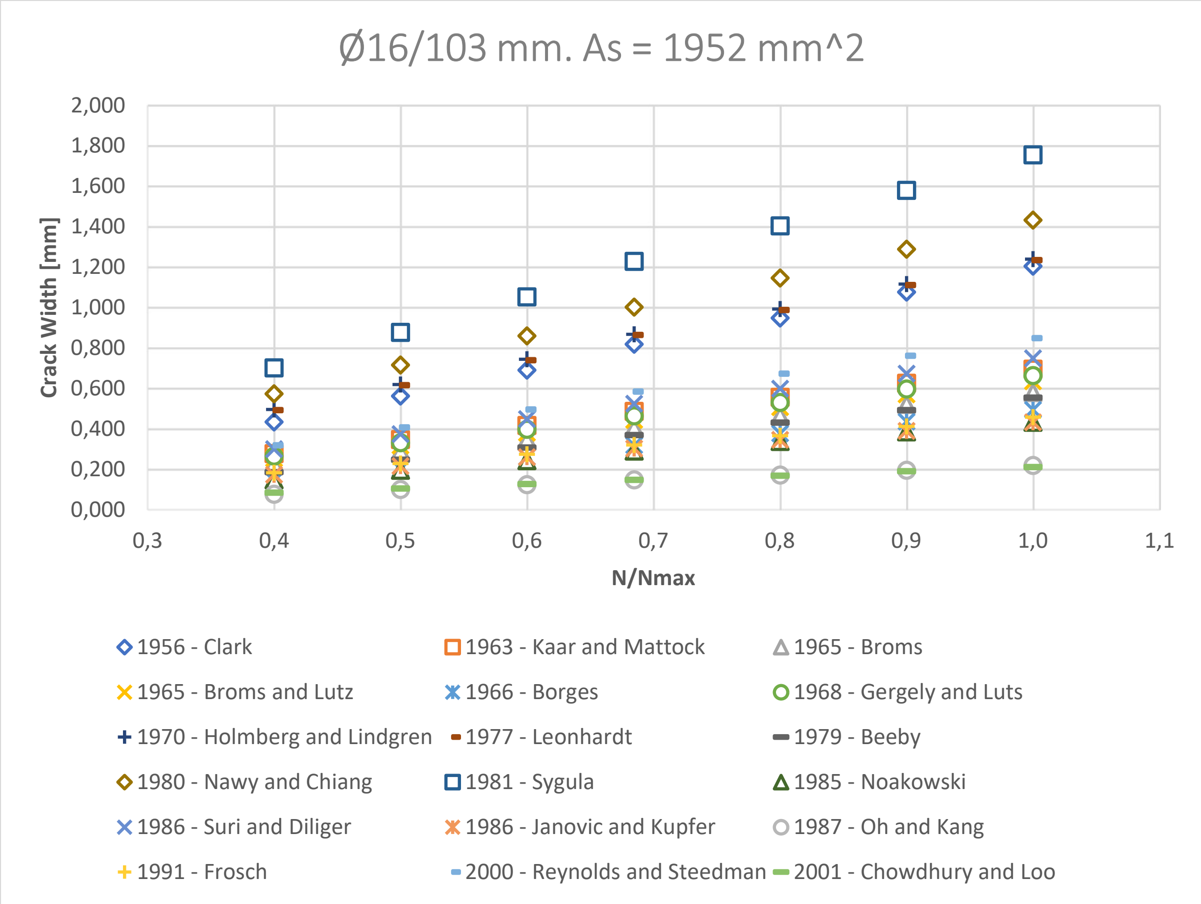

Figure A 3: Literature formulas. Ø16/103 mm. h = 200 mm. c = 50 mm

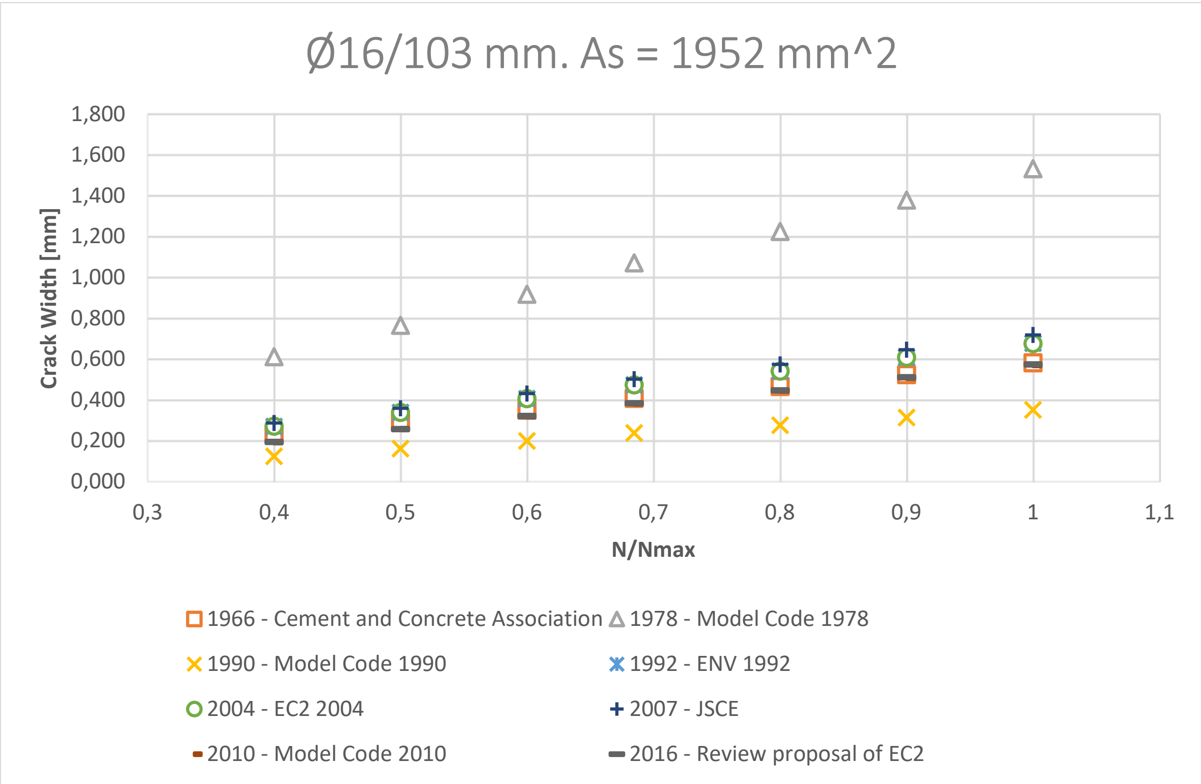

Figure A 4: Code formulas. Ø16/103 mm. h = 200 mm. c = 50 mm

A.3 Ø14/79 mm

N [kN]	779,6	974,5	1169,4	1364,3	1559,2	1754,1	1949
Nmax (σs2=500 Mpa)	1949	1949	1949	1949	1949	1949	1949
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
1956 - Clark	0,374	0,484	0,596	0,705	0,816	0,927	1,037
1963 - Kaar and Mattock	0,260	0,325	0,390	0,455	0,520	0,585	0,650
1965 - Broms	0,228	0,285	0,343	0,399	0,456	0,513	0,570
1965 - Broms and Lutz	0,241	0,302	0,363	0,422	0,483	0,543	0,603
1966 - Borges	0,135	0,190	0,246	0,300	0,355	0,410	0,465
1968 - Gergely and Luts	0,240	0,300	0,361	0,420	0,481	0,541	0,601
1970 - Holmberg and Lindgren	0,471	0,589	0,708	0,825	0,942	1,060	1,178
1977 - Leonhardt	0,442	0,553	0,665	0,775	0,886	0,997	1,109
1979 - Beeby	0,176	0,234	0,293	0,350	0,408	0,467	0,525
1980 - Nawy and Chiang	0,501	0,627	0,753	0,877	1,002	1,128	1,253
1981 - Sygula	0,655	0,819	0,985	1,147	1,310	1,474	1,638
1985 - Noakowski	0,132	0,174	0,217	0,259	0,302	0,344	0,387
1986 - Suri and Diliger	0,295	0,368	0,443	0,515	0,589	0,663	0,736
1986 - Janovic and Kupfer	0,149	0,186	0,223	0,260	0,297	0,334	0,372
1987 - Oh and Kang	0,061	0,080	0,099	0,118	0,137	0,156	0,175
1991 - Frosch	0,163	0,204	0,245	0,286	0,326	0,367	0,408
2000 - Reynolds and Steedman	0,305	0,390	0,476	0,560	0,645	0,730	0,815
2001 - Chowdhury and Loo	0,085	0,107	0,128	0,149	0,171	0,192	0,213

Table A 5: Literature formulas. Ø14/79 mm. h = 200 mm. c = 50 mm

N [kN]	779,6	974,5	1169,4	1364,3	1559,2	1754,1	1949
Nmax (σs2=500 Mpa)	1949	1949	1949	1949	1949	1949	1949
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
1960 - CBE	4,700	4,750	4,801	4,850	4,900	4,950	5,000
1966 - Cement and Concrete Association	0,231	0,289	0,347	0,404	0,462	0,519	0,577
1978 - Model Code 1978	0,518	0,649	0,780	0,909	1,039	1,169	1,300
1990 - Model Code 1990	0,108	0,141	0,175	0,208	0,241	0,274	0,308
1992 - ENV 1992	0,247	0,309	0,372	0,433	0,495	0,557	0,619
2004 - EC2 2004	0,253	0,316	0,380	0,443	0,506	0,569	0,632
2007 - JSCE	0,270	0,338	0,406	0,473	0,540	0,608	0,675
2010 - Model Code 2010	0,179	0,237	0,296	0,353	0,412	0,470	0,528
2016 - Review proposal of EC2	0,179	0,238	0,297	0,355	0,414	0,472	0,531

Table A 6: Code formulas. Ø14/79 mm. h = 200 mm. c = 50 mm

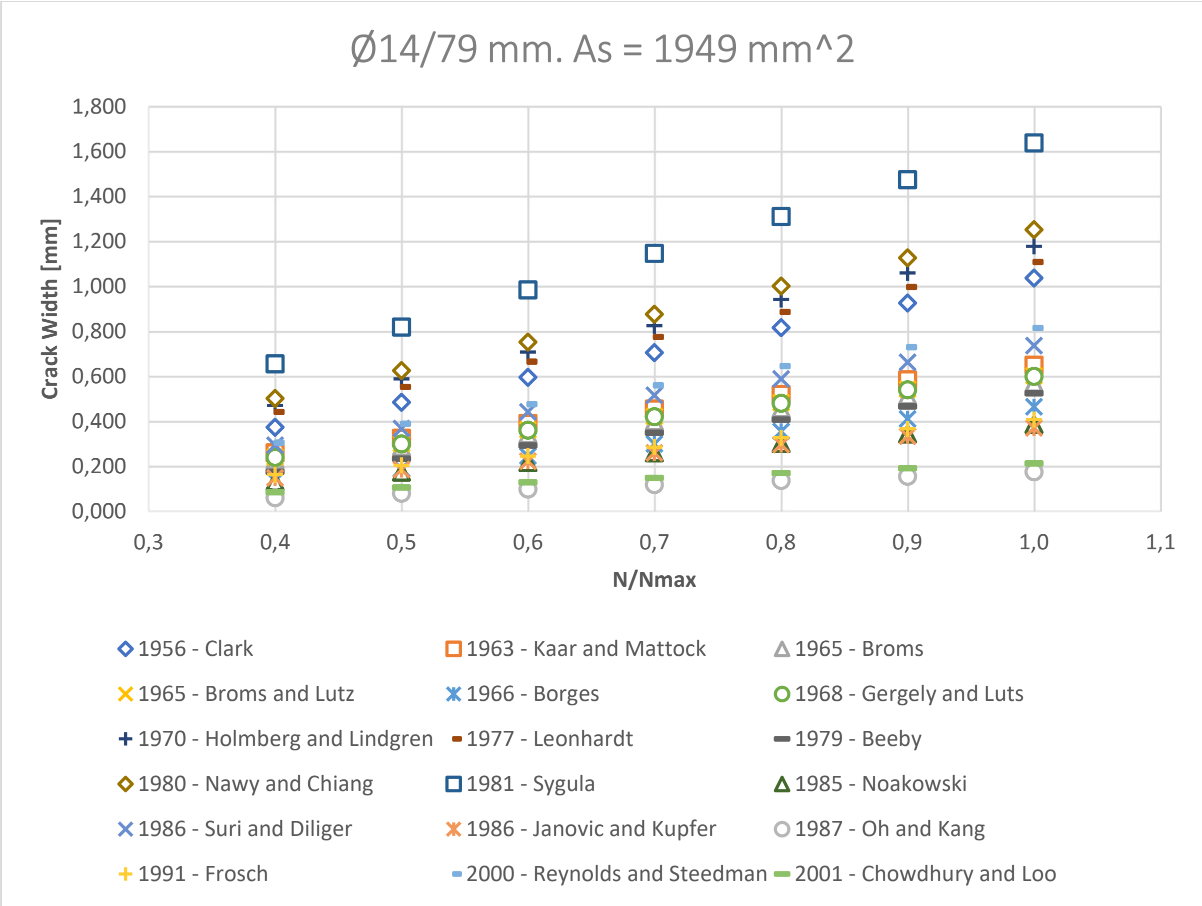

Figure A 5: Literature formulas. Ø14/79 mm. h = 200 mm. c = 50 mm

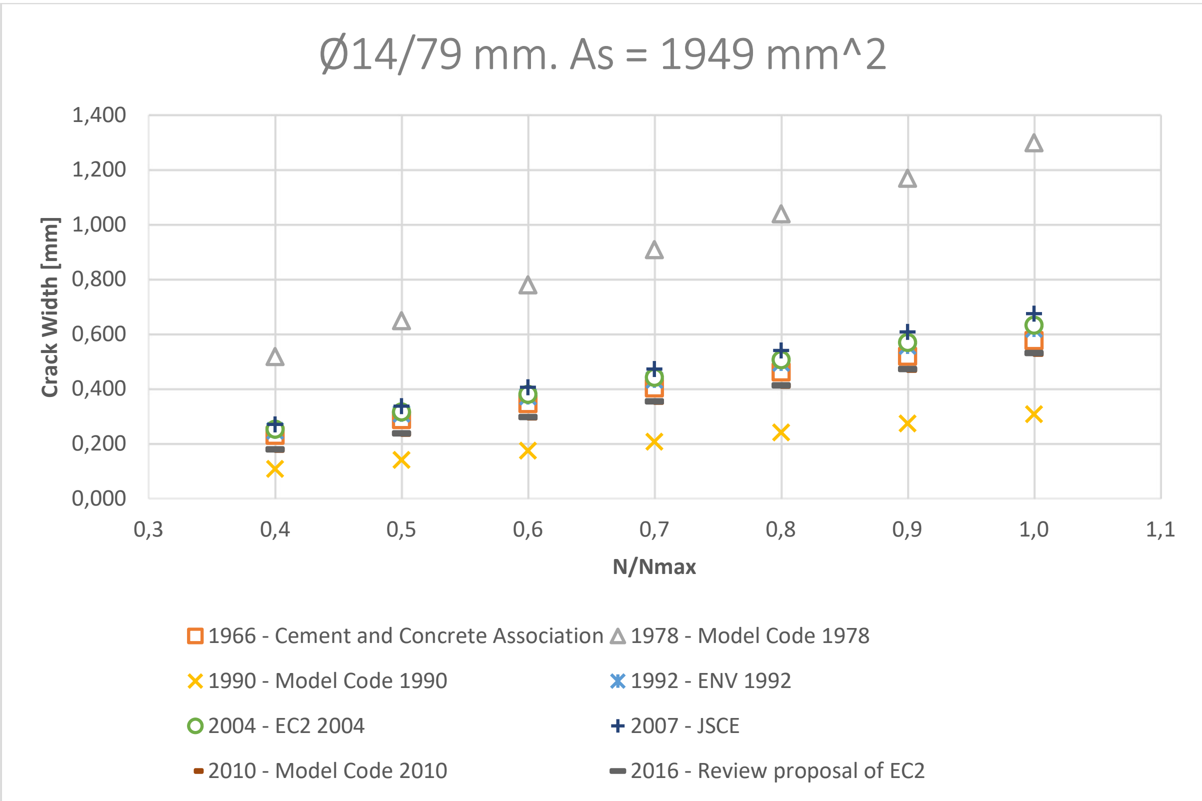

Figure A 6: Code formulas. Ø14/79 mm. h = 200 mm. c = 50 mm

B Varying Concrete Cover

B.1 Ø18/130 mm. C=50 mm

N [kN]	782,8	978,5	1174,2	1396,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,40	0,50	0,60	0,7	0,8	0,9	1,0
1956 - Clark	0,655	0,863	1,071	1,279	1,487	1,694	1,902
1963 - Kaar and Mattock	0,295	0,369	0,442	0,516	0,590	0,664	0,737
1965 - Broms	0,236	0,295	0,354	0,413	0,472	0,531	0,590
1965 - Broms and Lutz	0,269	0,337	0,404	0,471	0,539	0,606	0,673
1966 - Borges	0,152	0,214	0,275	0,337	0,399	0,460	0,522
1968 - Gergely and Luts	0,290	0,362	0,435	0,507	0,579	0,652	0,724
1970 - Holmberg and Lindgren	0,519	0,649	0,779	0,909	1,038	1,168	1,298
1977 - Leonhardt	0,542	0,679	0,815	0,951	1,088	1,224	1,360
1979 - Beeby	0,196	0,261	0,327	0,392	0,457	0,522	0,587
1980 - Nawy and Chiang	0,643	0,803	0,964	1,125	1,285	1,446	1,607
1981 - Sygula	0,746	0,933	1,120	1,306	1,493	1,679	1,866
1985 - Noakowski	0,163	0,216	0,268	0,320	0,373	0,425	0,477
1986 - Suri and Diliger	0,304	0,380	0,456	0,532	0,608	0,684	0,760
1986 - Janovic and Kupfer	0,201	0,251	0,301	0,351	0,401	0,451	0,501
1987 - Oh and Kang	0,093	0,122	0,151	0,180	0,209	0,239	0,268
1991 - Frosch	0,208	0,260	0,311	0,363	0,415	0,467	0,519
2000 - Reynolds and Steedman	0,332	0,425	0,517	0,610	0,702	0,795	0,887
2001 - Chowdhury and Loo	0,082	0,102	0,123	0,143	0,163	0,184	0,204

Table B 1: Literature formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

N [kN]	782,8	978,5	1174,2	1396,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1
1960 - CBE	4,765	4,831	4,898	4,964	5,030	5,096	5,163
1966 - Cement and Concrete Association	0,234	0,292	0,351	0,409	0,468	0,526	0,585
1978 - Model Code 1978	0,713	0,893	1,072	1,252	1,431	1,610	1,789
1990 - Model Code 1990	0,138	0,181	0,223	0,266	0,309	0,351	0,394
1992 - ENV 1992	0,292	0,366	0,439	0,513	0,586	0,660	0,733
2004 - EC2 2004	0,288	0,359	0,431	0,503	0,575	0,647	0,719
2007 - JSCE	0,306	0,383	0,459	0,536	0,612	0,689	0,765
2010 - Model Code 2010	0,207	0,275	0,342	0,410	0,478	0,545	0,613
2016 - Review proposal of EC2	0,208	0,276	0,344	0,412	0,480	0,548	0,616

Table B 2: Code formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

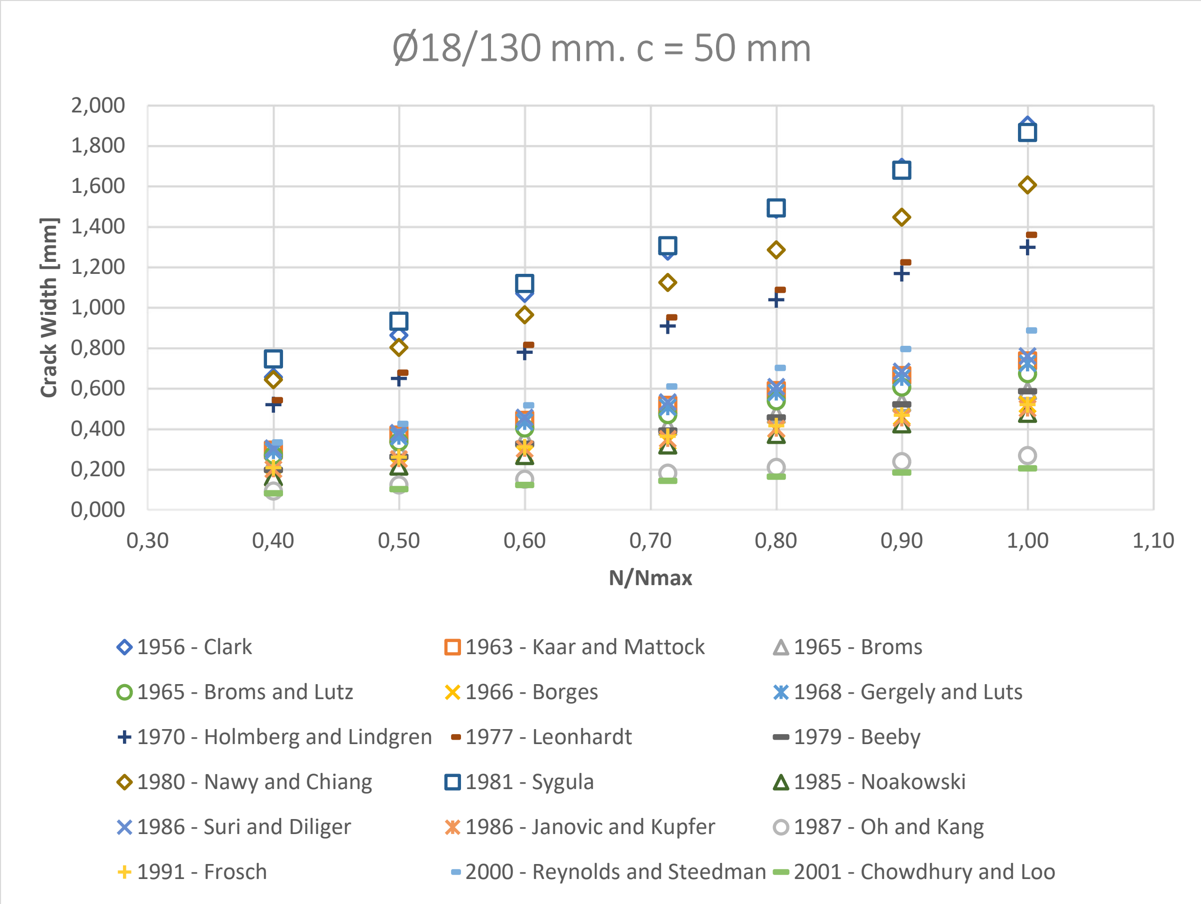

Figure B 1: Literature formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

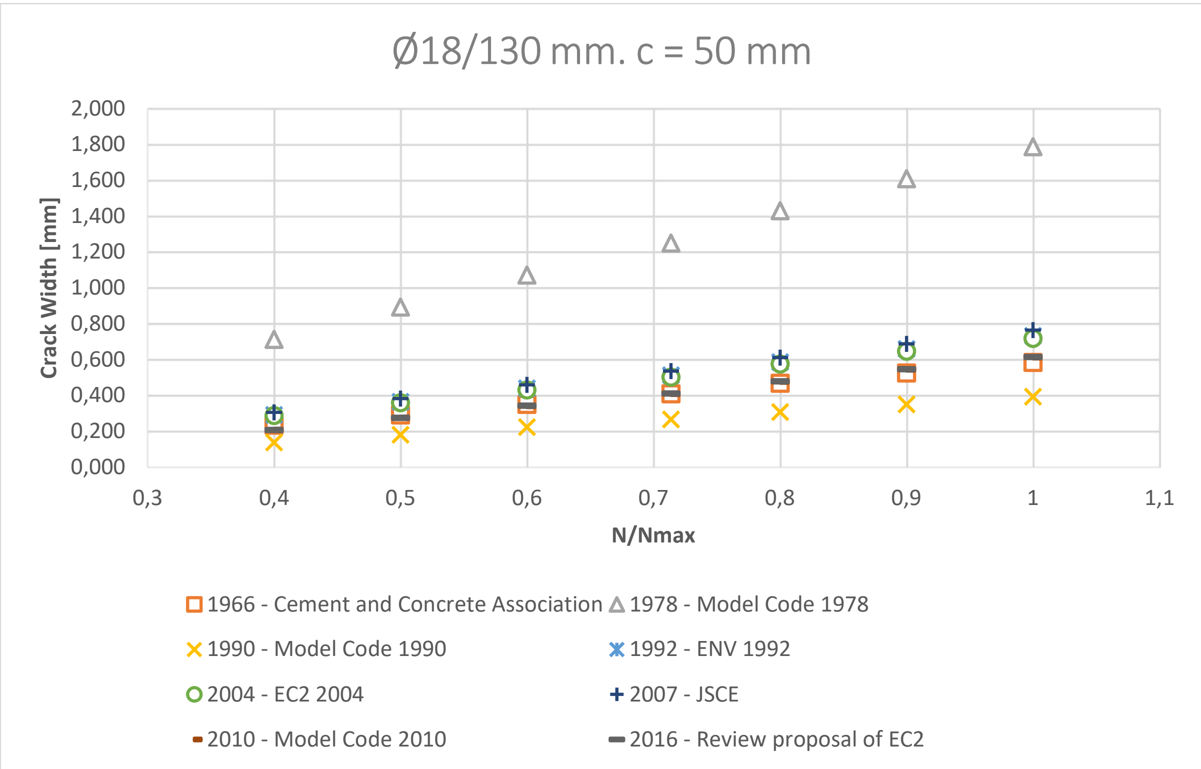

Figure B 2: Code formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

B.2 Ø18/130 mm. C=40 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1791,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
1956 - Clark	0,408	0,529	0,651	0,772	0,894	1,016	1,137
1963 - Kaar and Mattock	0,275	0,344	0,413	0,482	0,551	0,620	0,688
1965 - Broms	0,196	0,245	0,294	0,343	0,392	0,441	0,490
1965 - Broms and Lutz	0,235	0,294	0,353	0,411	0,470	0,529	0,588
1966 - Borges	0,137	0,192	0,248	0,303	0,358	0,414	0,469
1968 - Gergely and Luts	0,239	0,299	0,359	0,418	0,478	0,538	0,598
1970 - Holmberg and Lindgren	0,420	0,525	0,630	0,735	0,840	0,945	1,050
1977 - Leonhardt	0,525	0,658	0,790	0,922	1,054	1,186	1,318
1979 - Beeby	0,164	0,218	0,273	0,327	0,381	0,435	0,490
1980 - Nawy and Chiang	0,498	0,623	0,748	0,872	0,997	1,122	1,246
1981 - Sygula	0,732	0,915	1,099	1,282	1,465	1,648	1,831
1985 - Noakowski	0,169	0,224	0,279	0,334	0,390	0,445	0,500
1986 - Suri and Diliger	0,253	0,316	0,379	0,442	0,505	0,568	0,631
1986 - Janovic and Kupfer	0,201	0,251	0,301	0,351	0,401	0,451	0,501
1987 - Oh and Kang	0,077	0,100	0,124	0,148	0,172	0,196	0,220
1991 - Frosch	0,187	0,234	0,281	0,328	0,375	0,422	0,469
2000 - Reynolds and Steedman	0,301	0,385	0,469	0,553	0,638	0,722	0,806
2001 - Chowdhury and Loo	0,085	0,106	0,127	0,149	0,170	0,191	0,212

Table B 3: Literature formulas. Ø18/130 mm. h = 200 mm. c = 40 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1791,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
1960 - CBE	4,720	4,775	4,830	4,885	4,940	4,995	5,050
1966 - Cement and Concrete Association	0,175	0,218	0,262	0,306	0,350	0,393	0,437
1978 - Model Code 1978	0,679	0,850	1,021	1,192	1,363	1,534	1,704
1990 - Model Code 1990	0,138	0,181	0,223	0,266	0,309	0,351	0,394
1992 - ENV 1992	0,292	0,366	0,439	0,513	0,586	0,660	0,733
2004 - EC2 2004	0,262	0,327	0,393	0,458	0,523	0,589	0,654
2007 - JSCE	0,262	0,328	0,393	0,459	0,524	0,590	0,655
2010 - Model Code 2010	0,192	0,255	0,317	0,380	0,442	0,505	0,567
2016 - Review proposal of EC2	0,193	0,256	0,319	0,382	0,445	0,508	0,571

Table B 4: Code formulas. Ø18/130 mm. h = 200 mm. c = 40 mm

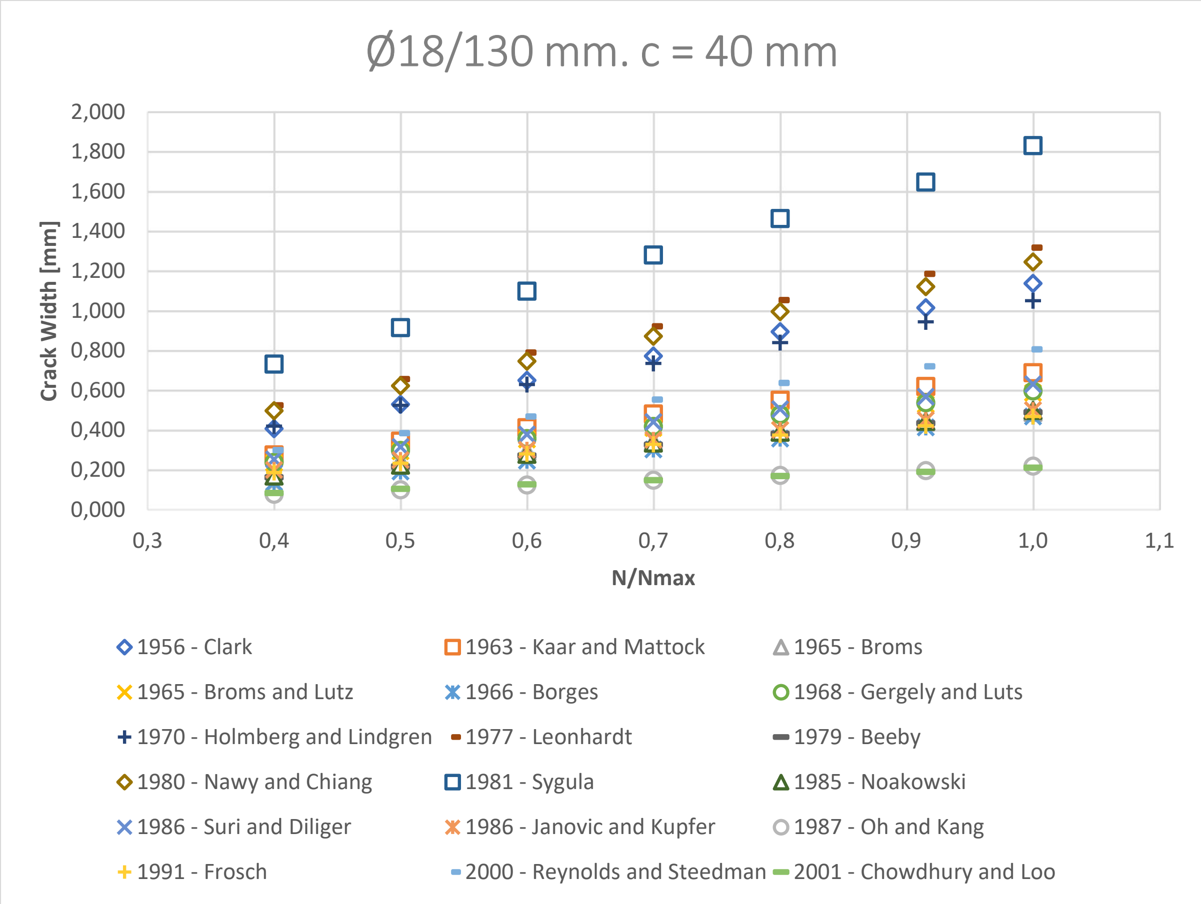

Figure B 3: Literature formulas. Ø18/130 mm. h = 200 mm. c = 40 mm

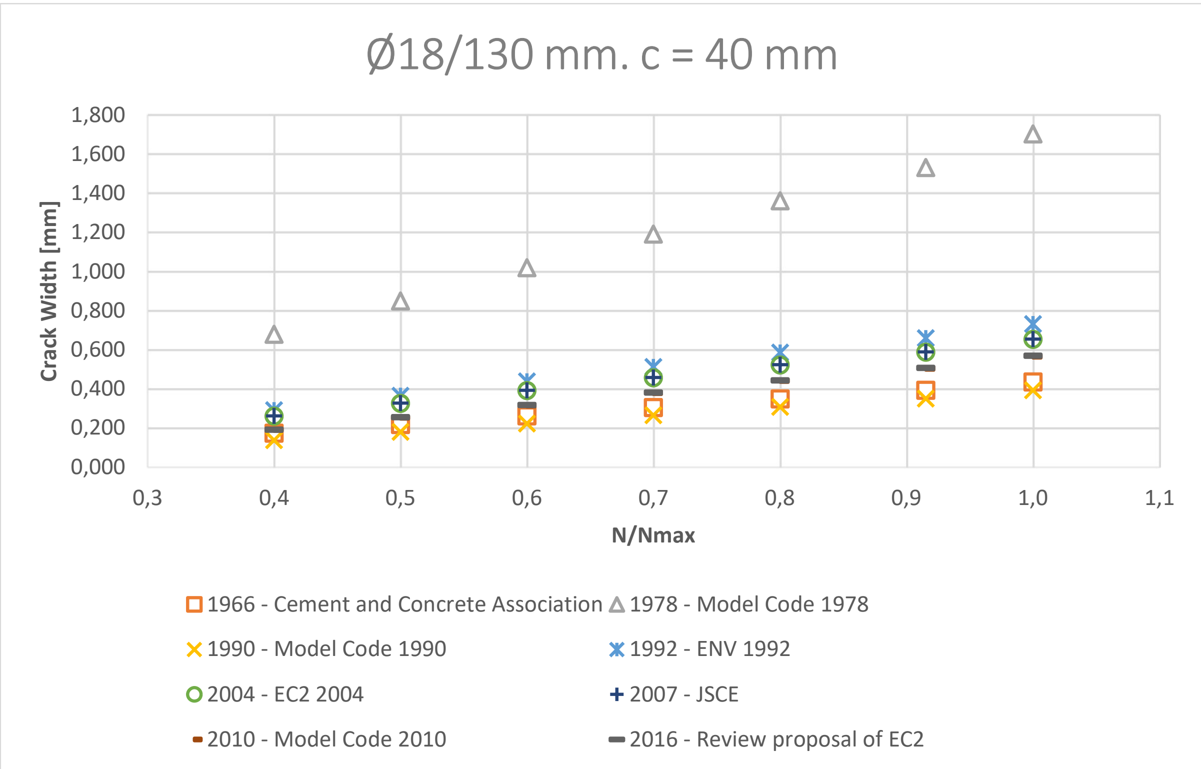

Figure B 4: Code formulas. Ø18/130 mm. h = 200 mm. c = 40 mm

B.3 Ø18/130 mm. C=30 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
1956 - Clark	0,321	0,417	0,514	0,611	0,708	0,805	0,901
1963 - Kaar and Mattock	0,258	0,323	0,387	0,452	0,517	0,581	0,646
1965 - Broms	0,156	0,195	0,234	0,273	0,312	0,351	0,390
1965 - Broms and Lutz	0,203	0,254	0,305	0,355	0,406	0,457	0,508
1966 - Borges	0,121	0,171	0,220	0,269	0,318	0,367	0,417
1968 - Gergely and Luts	0,193	0,241	0,289	0,337	0,385	0,433	0,481
1970 - Holmberg and Lindgren	0,333	0,417	0,500	0,583	0,666	0,750	0,833
1977 - Leonhardt	0,508	0,637	0,765	0,893	1,020	1,148	1,276
1979 - Beeby	0,137	0,183	0,228	0,274	0,319	0,365	0,410
1980 - Nawy and Chiang	0,372	0,465	0,558	0,651	0,744	0,837	0,930
1981 - Sygula	0,720	0,900	1,080	1,260	1,440	1,620	1,800
1985 - Noakowski	0,174	0,232	0,290	0,348	0,406	0,464	0,522
1986 - Suri and Diliger	0,201	0,251	0,302	0,352	0,402	0,452	0,503
1986 - Janovic and Kupfer	0,201	0,251	0,301	0,351	0,401	0,451	0,501
1987 - Oh and Kang	0,066	0,086	0,107	0,127	0,148	0,168	0,189
1991 - Frosch	0,170	0,212	0,255	0,297	0,340	0,382	0,425
2000 - Reynolds and Steedman	0,273	0,350	0,427	0,504	0,582	0,659	0,736
2001 - Chowdhury and Loo	0,088	0,110	0,132	0,154	0,176	0,198	0,220

Table B 5: Literature formulas. Ø18/130 mm. h = 200 mm. c = 30 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
1960 - CBE	4,675	4,719	4,763	4,807	4,850	4,894	4,938
1966 - Cement and Concrete Association	0,123	0,154	0,184	0,215	0,246	0,277	0,307
1978 - Model Code 1978	0,646	0,808	0,970	1,133	1,295	1,457	1,619
1990 - Model Code 1990	0,138	0,181	0,223	0,266	0,309	0,351	0,394
1992 - ENV 1992	0,292	0,366	0,439	0,513	0,586	0,660	0,733
2004 - EC2 2004	0,236	0,295	0,354	0,413	0,472	0,531	0,590
2007 - JSCE	0,218	0,273	0,327	0,382	0,436	0,491	0,545
2010 - Model Code 2010	0,177	0,234	0,292	0,349	0,407	0,465	0,522
2016 - Review proposal of EC2	0,178	0,236	0,294	0,351	0,409	0,467	0,525

Table B 6: Code formulas. Ø18/130 mm. h = 200 mm. c = 30 mm

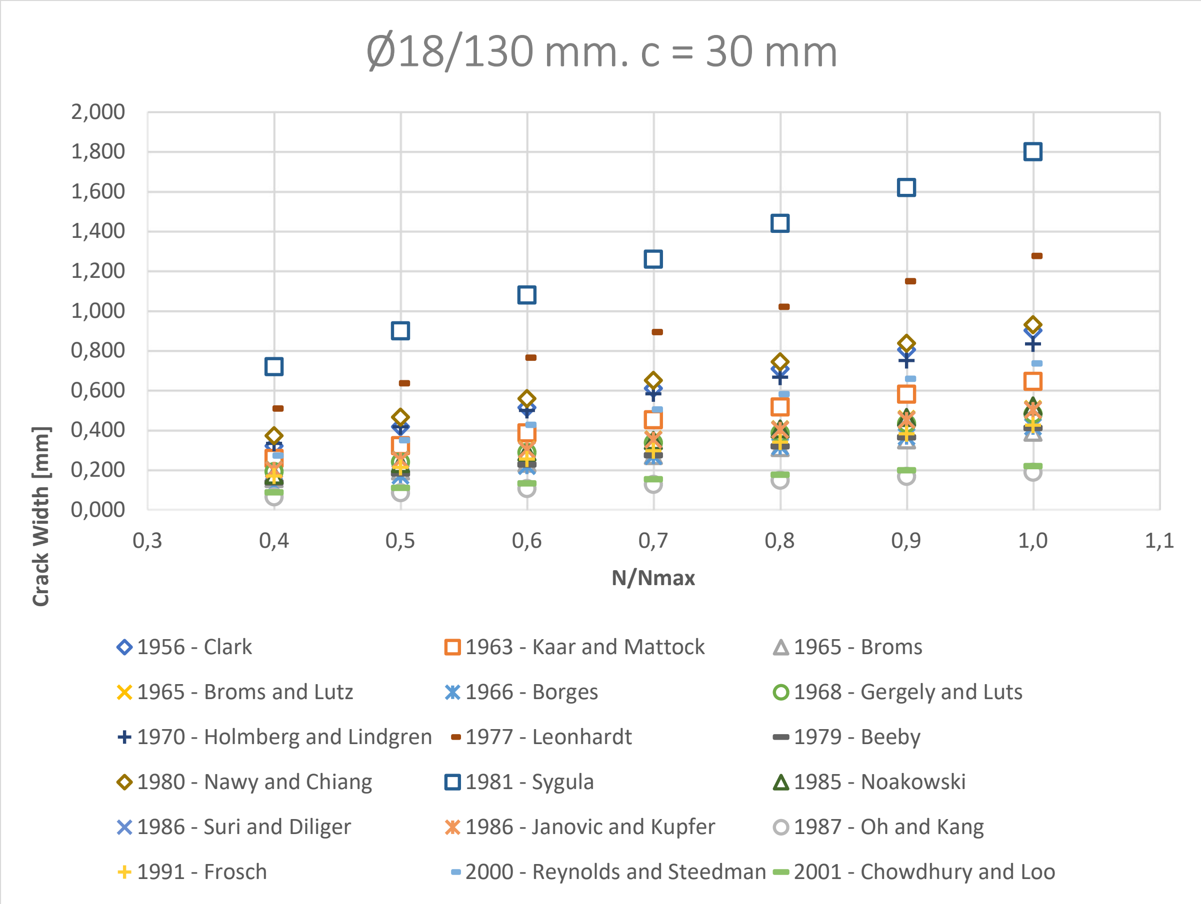

Figure B 5: Literature formulas. Ø18/130 mm. h = 200 mm. c = 30 mm

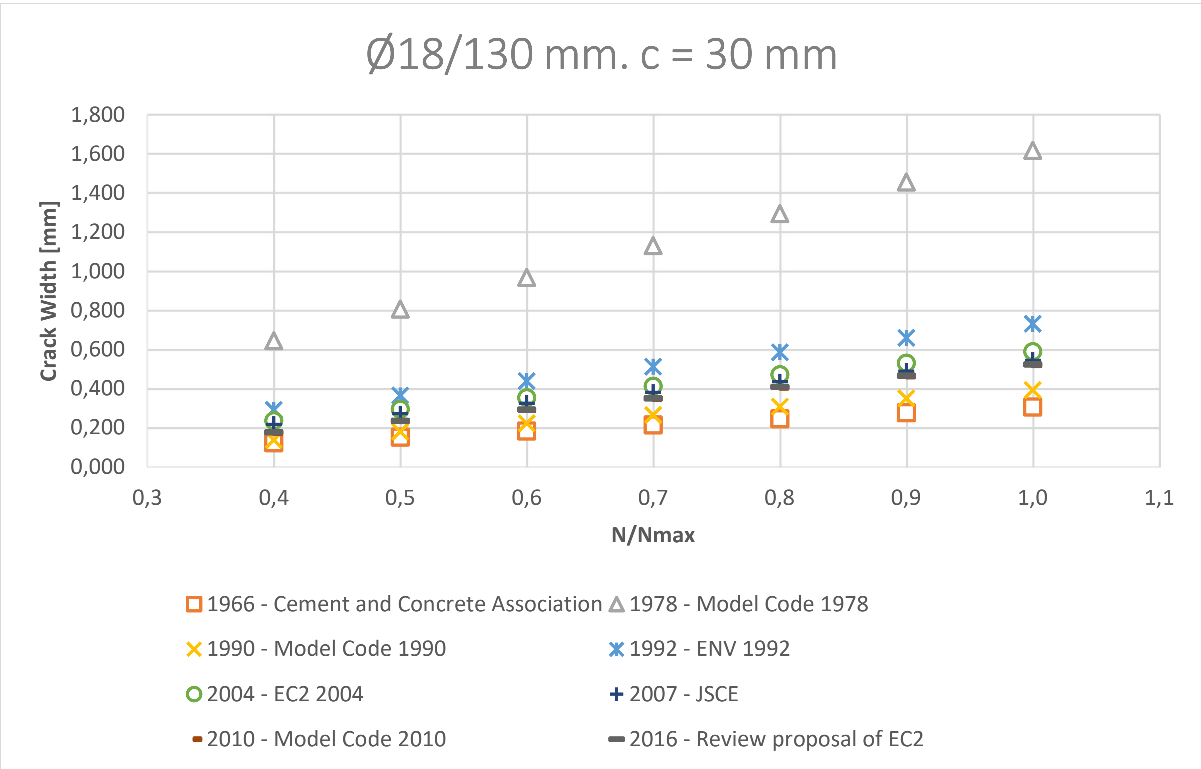

Figure B 6: Code formulas. Ø18/130 mm. h = 200 mm. c = 30 mm

C Varying Concrete Thickness

C.1 Ø18/130 mm. H=200 mm

N [kN]	782,8	978,5	1174,2	1396,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,40	0,50	0,60	0,7	0,8	0,9	1,0
1956 - Clark	0,655	0,863	1,071	1,279	1,487	1,694	1,902
1963 - Kaar and Mattock	0,295	0,369	0,442	0,516	0,590	0,664	0,737
1965 - Broms	0,236	0,295	0,354	0,413	0,472	0,531	0,590
1965 - Broms and Lutz	0,269	0,337	0,404	0,471	0,539	0,606	0,673
1966 - Borges	0,152	0,214	0,275	0,337	0,399	0,460	0,522
1968 - Gergely and Luts	0,290	0,362	0,435	0,507	0,579	0,652	0,724
1970 - Holmberg and Lindgren	0,519	0,649	0,779	0,909	1,038	1,168	1,298
1977 - Leonhardt	0,542	0,679	0,815	0,951	1,088	1,224	1,360
1979 - Beeby	0,196	0,261	0,327	0,392	0,457	0,522	0,587
1980 - Nawy and Chiang	0,643	0,803	0,964	1,125	1,285	1,446	1,607
1981 - Sygula	0,746	0,933	1,120	1,306	1,493	1,679	1,866
1985 - Noakowski	0,163	0,216	0,268	0,320	0,373	0,425	0,477
1986 - Suri and Diliger	0,304	0,380	0,456	0,532	0,608	0,684	0,760
1986 - Janovic and Kupfer	0,201	0,251	0,301	0,351	0,401	0,451	0,501
1987 - Oh and Kang	0,093	0,122	0,151	0,180	0,209	0,239	0,268
1991 - Frosch	0,208	0,260	0,311	0,363	0,415	0,467	0,519
2000 - Reynolds and Steedman	0,332	0,425	0,517	0,610	0,702	0,795	0,887
2001 - Chowdhury and Loo	0,082	0,102	0,123	0,143	0,163	0,184	0,204

Table C 1: Literature formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

N [kN]	782,8	978,5	1174,2	1396,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1
1960 - CBE	4,765	4,831	4,898	4,964	5,030	5,096	5,163
1966 - Cement and Concrete Association	0,234	0,292	0,351	0,409	0,468	0,526	0,585
1978 - Model Code 1978	0,713	0,893	1,072	1,252	1,431	1,610	1,789
1990 - Model Code 1990	0,138	0,181	0,223	0,266	0,309	0,351	0,394
1992 - ENV 1992	0,292	0,366	0,439	0,513	0,586	0,660	0,733
2004 - EC2 2004	0,288	0,359	0,431	0,503	0,575	0,647	0,719
2007 - JSCE	0,306	0,383	0,459	0,536	0,612	0,689	0,765
2010 - Model Code 2010	0,207	0,275	0,342	0,410	0,478	0,545	0,613
2016 - Review proposal of EC2	0,208	0,276	0,344	0,412	0,480	0,548	0,616

Table C 2: Code formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

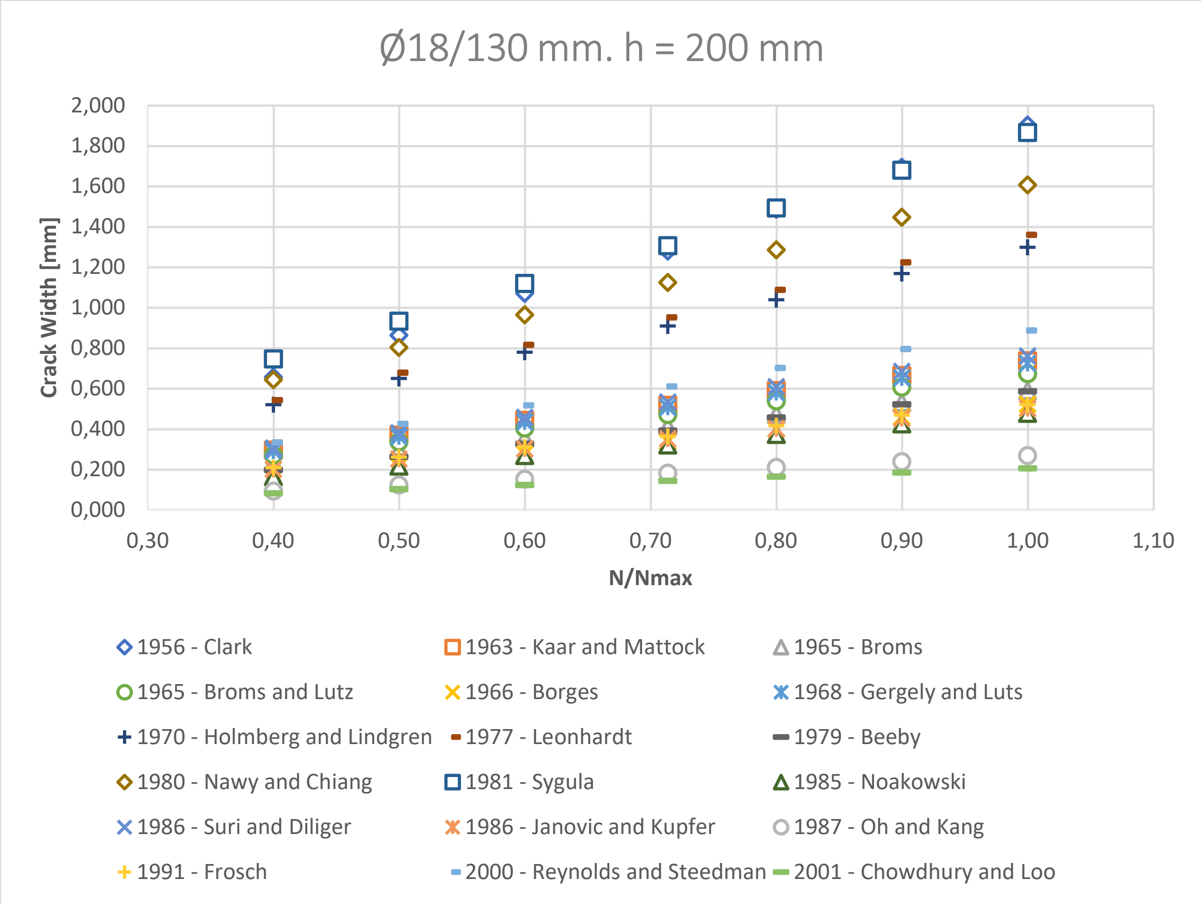

Figure C 1: Literature formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

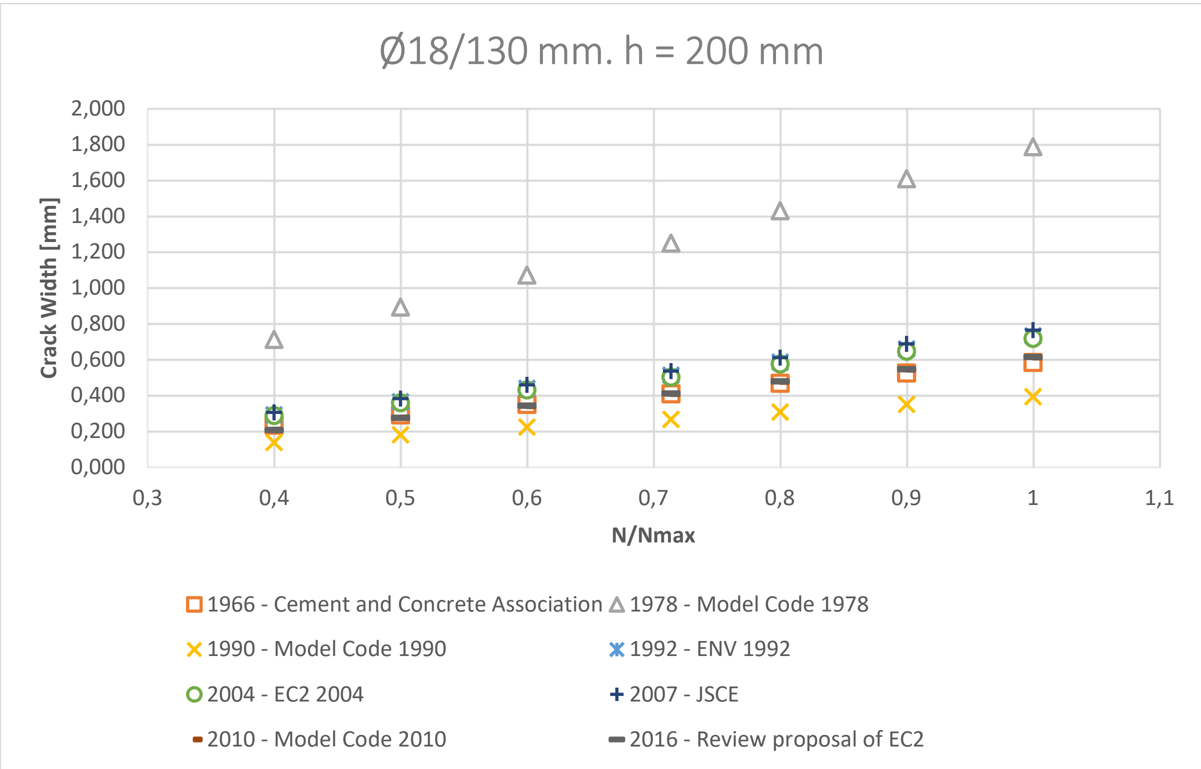

Figure C 2: Code formulas. Ø18/130 mm. h = 200 mm. c = 50 mm

C.2 Ø18/130 mm. H=300 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
σc1	2,45						
1956 - Clark	-	0,585	0,731	0,878	1,024	1,171	1,317
1963 - Kaar and Mattock	-	0,358	0,430	0,501	0,573	0,644	0,716
1965 - Broms	-	0,295	0,354	0,413	0,472	0,531	0,590
1965 - Broms and Lutz	-	0,337	0,404	0,471	0,539	0,606	0,673
1966 - Borges	-	0,208	0,285	0,361	0,438	0,515	0,592
1968 - Gergely and Luts	-	0,318	0,381	0,445	0,509	0,572	0,636
1970 - Holmberg and Lindgren	-	0,570	0,683	0,797	0,911	1,025	1,139
1977 - Leonhardt	-	0,959	1,152	1,345	1,538	1,730	1,923
1979 - Beeby	-	0,218	0,279	0,341	0,403	0,464	0,526
1980 - Nawy and Chiang	-	0,705	0,846	0,987	1,128	1,269	1,410
1981 - Sygula	-	0,823	0,988	1,152	1,317	1,482	1,646
1985 - Noakowski	-	0,279	0,358	0,438	0,517	0,597	0,677
1986 - Suri and Diliger	-	0,466	0,559	0,652	0,745	0,838	0,931
1986 - Janovic and Kupfer	-	0,251	0,301	0,351	0,401	0,451	0,501
1987 - Oh and Kang	-	0,099	0,122	0,146	0,169	0,193	0,217
1991 - Frosch	-	0,260	0,311	0,363	0,415	0,467	0,519
2000 - Reynolds and Steedman	-	0,383	0,472	0,561	0,650	0,739	0,828
2001 - Chowdhury and Loo	-	0,217	0,260	0,304	0,347	0,391	0,434

Table C 3: Literature formulas. Ø18/130 mm. h = 300 mm. c = 50 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
σc1	2,45						
1960 - CBE	-	4,831	4,898	4,964	5,030	5,096	5,163
1966 - Cement and Concrete Association	-	0,257	0,308	0,359	0,411	0,462	0,513
1978 - Model Code 1978	-	0,958	1,150	1,343	1,535	1,727	1,919
1990 - Model Code 1990	-	0,247	0,311	0,375	0,439	0,503	0,567
1992 - ENV 1992	-	0,496	0,595	0,695	0,795	0,894	0,993
2004 - EC2 2004	-	0,443	0,550	0,642	0,734	0,825	0,917
2007 - JSCE	-	0,383	0,459	0,536	0,612	0,689	0,765
2010 - Model Code 2010	-	0,326	0,415	0,504	0,593	0,682	0,771
2016 - Review proposal of EC2	-	0,328	0,418	0,507	0,596	0,686	0,775

Table C 4: Code formulas. Ø18/130 mm. h = 300 mm. c = 50 mm

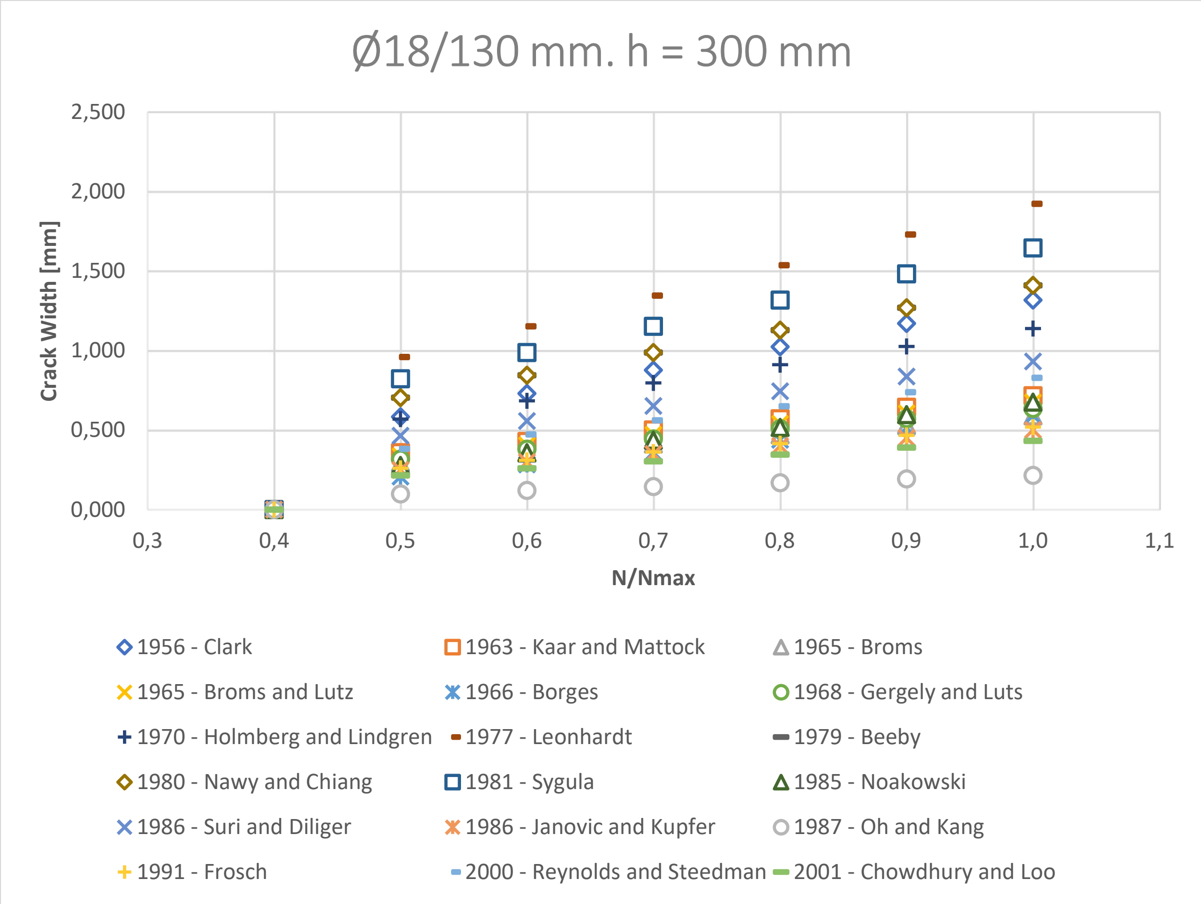

Figure C 3: Literature formulas. Ø18/130 mm. h = 300 mm. c = 50 mm

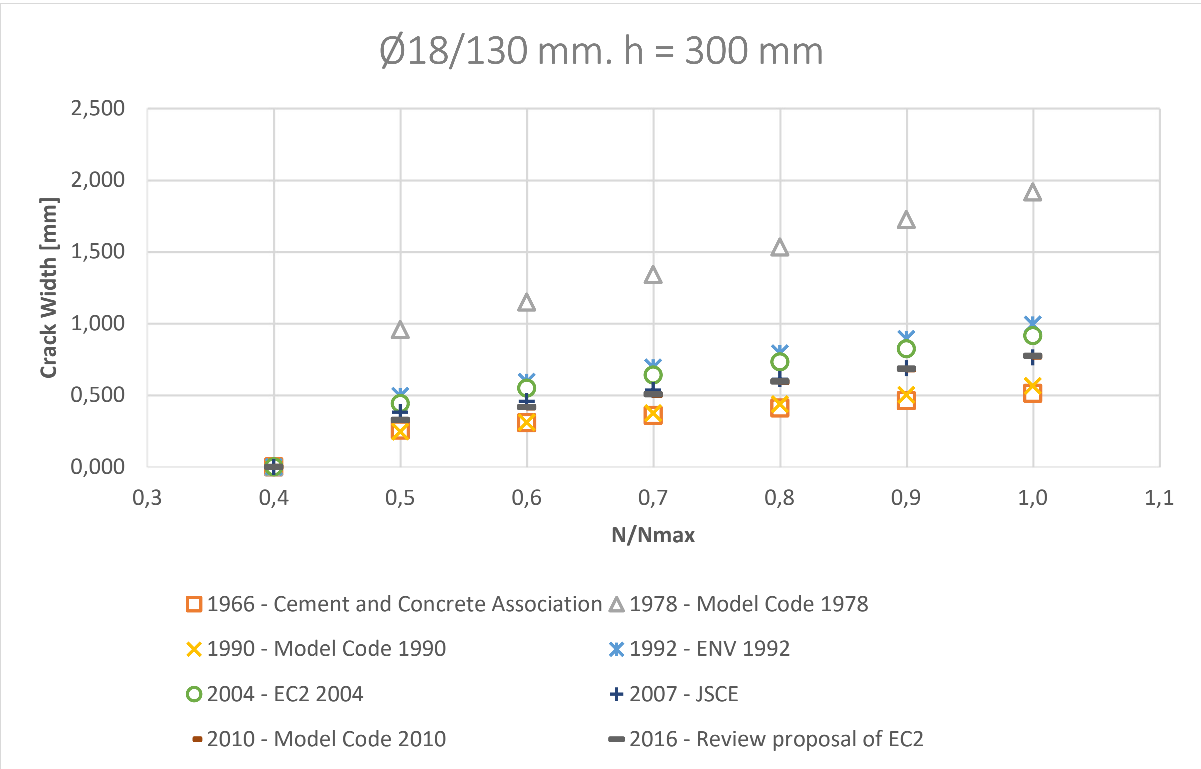

Figure C 4: Code formulas. Ø18/130 mm. h = 300 mm. c = 50 mm

C.3 Ø18/130 mm. H=400 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
σc1	1,87	2,34	2,80				
1956 - Clark	-	-	-	0,819	0,966	1,112	1,259
1963 - Kaar and Mattock	-	-	-	0,508	0,580	0,653	0,725
1965 - Broms	-	-	-	0,413	0,472	0,531	0,590
1965 - Broms and Lutz	-	-	-	0,471	0,539	0,606	0,673
1966 - Borges	-	-	-	0,363	0,455	0,547	0,639
1968 - Gergely and Luts	-	-	-	0,419	0,479	0,539	0,599
1970 - Holmberg and Lindgren	-	-	-	0,751	0,859	0,966	1,073
1977 - Leonhardt	-	-	-	1,739	1,988	2,237	2,486
1979 - Beeby	-	-	-	0,305	0,366	0,426	0,487
1980 - Nawy and Chiang	-	-	-	0,930	1,063	1,196	1,329
1981 - Sygula	-	-	-	1,089	1,244	1,400	1,555
1985 - Noakowski	-	-	-	0,510	0,614	0,718	0,823
1986 - Suri and Diliger	-	-	-	0,753	0,860	0,968	1,075
1986 - Janovic and Kupfer	-	-	-	0,351	0,401	0,451	0,501
1987 - Oh and Kang	-	-	-	0,143	0,166	0,189	0,212
1991 - Frosch	-	-	-	0,363	0,415	0,467	0,519
2000 - Reynolds and Steedman	-	-	-	0,531	0,619	0,707	0,795
2001 - Chowdhury and Loo	-	-	-	0,465	0,531	0,598	0,664

Table C 5: Literature formulas. Ø18/130 mm. h = 400 mm. c = 50 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
σc1	1,87	2,34	2,80				
1960 - CBE	-	-	-	4,964	5,030	5,096	5,163
1966 - Cement and Concrete Association	-	-	-	0,339	0,387	0,435	0,484
1978 - Model Code 1978	-	-	-	1,434	1,639	1,844	2,049
1990 - Model Code 1990	-	-	-	0,468	0,553	0,638	0,724
1992 - ENV 1992	-	-	-	0,877	1,003	1,128	1,254
2004 - EC2 2004	-	-	-	0,774	0,892	1,003	1,115
2007 - JSCE	-	-	-	0,536	0,612	0,689	0,765
2010 - Model Code 2010	-	-	-	0,581	0,691	0,802	0,912
2016 - Review proposal of EC2	-	-	-	0,585	0,696	0,807	0,917

Table C 6: Code formulas. Ø18/130 mm. h = 400 mm. c = 50 mm

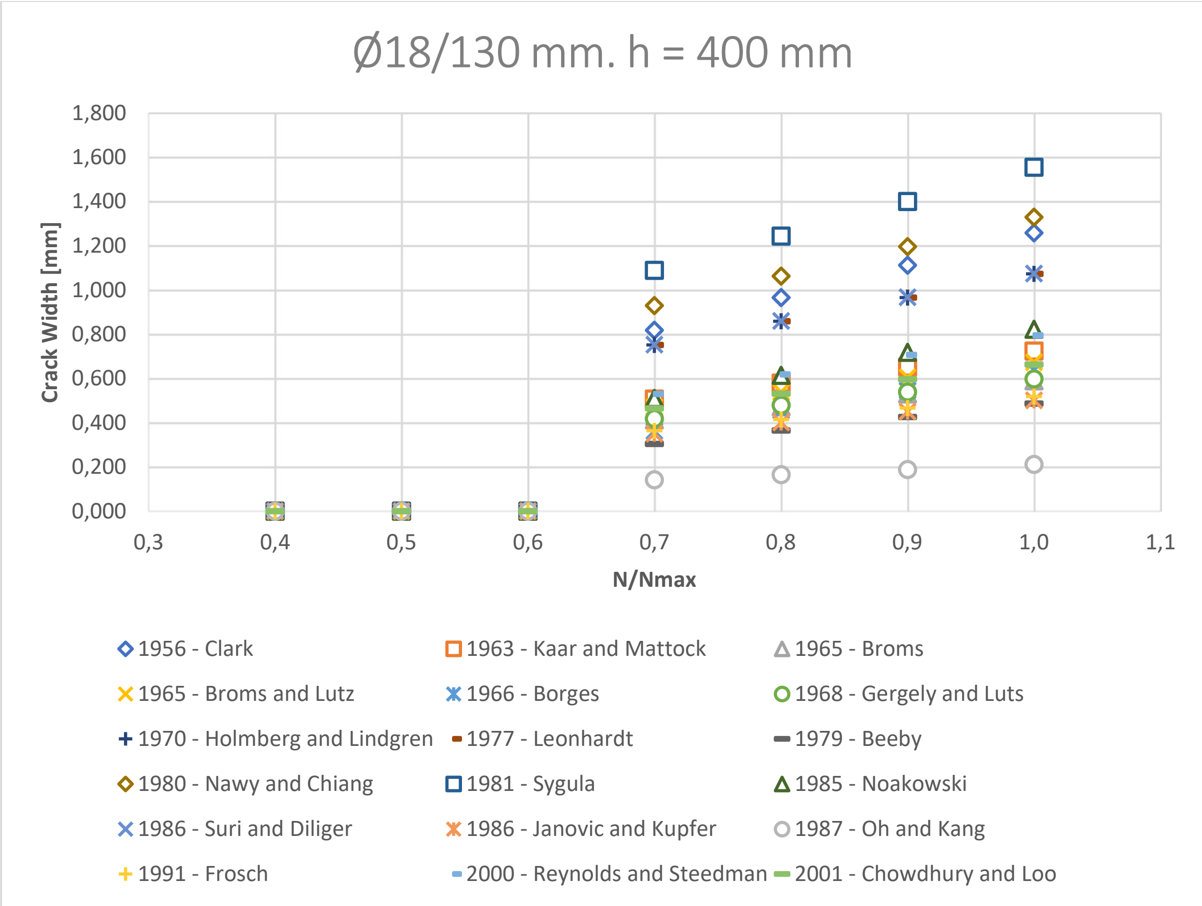

Figure C 5: Literature formulas. Ø18/130 mm. h = 400 mm. c = 50 mm

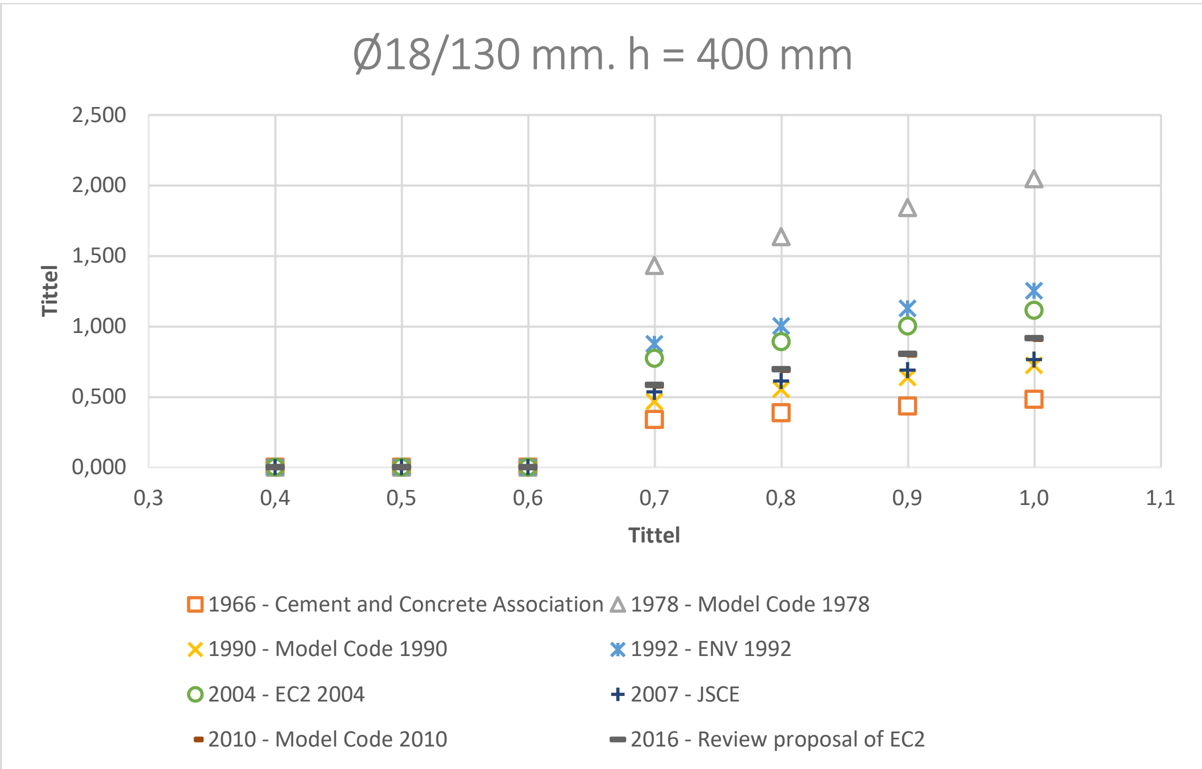

Figure C 6: Code formulas. Ø18/130 mm. h = 400 mm. c = 50 mm

D Increasing Bar Diameter

D.1 Ø18/130 mm. As=1957 mm^2

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
σc1	2,45						
1956 - Clark	-	0,585	0,731	0,878	1,024	1,171	1,317
1963 - Kaar and Mattock	-	0,358	0,430	0,501	0,573	0,644	0,716
1965 - Broms	-	0,295	0,354	0,413	0,472	0,531	0,590
1965 - Broms and Lutz	-	0,337	0,404	0,471	0,539	0,606	0,673
1966 - Borges	-	0,208	0,285	0,361	0,438	0,515	0,592
1968 - Gergely and Luts	-	0,318	0,381	0,445	0,509	0,572	0,636
1970 - Holmberg and Lindgren	-	0,570	0,683	0,797	0,911	1,025	1,139
1977 - Leonhardt	-	0,959	1,152	1,345	1,538	1,730	1,923
1979 - Beeby	-	0,218	0,279	0,341	0,403	0,464	0,526
1980 - Nawy and Chiang	-	0,705	0,846	0,987	1,128	1,269	1,410
1981 - Sygula	-	0,823	0,988	1,152	1,317	1,482	1,646
1985 - Noakowski	-	0,279	0,358	0,438	0,517	0,597	0,677
1986 - Suri and Diliger	-	0,466	0,559	0,652	0,745	0,838	0,931
1986 - Janovic and Kupfer	-	0,251	0,301	0,351	0,401	0,451	0,501
1987 - Oh and Kang	-	0,099	0,122	0,146	0,169	0,193	0,217
1991 - Frosch	-	0,260	0,311	0,363	0,415	0,467	0,519
2000 - Reynolds and Steedman	-	0,383	0,472	0,561	0,650	0,739	0,828
2001 - Chowdhury and Loo	-	0,217	0,260	0,304	0,347	0,391	0,434

Table D 1: Literature formulas. Ø18/130 mm. h = 300 mm. c = 50 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	1957	1957	1957	1957	1957	1957	1957
N/Nmax	0,4	0,5	0,6	0,7	0,8	0,9	1,0
σc1	2,45						
1960 - CBE	-	4,831	4,898	4,964	5,030	5,096	5,163
1966 - Cement and Concrete Association	-	0,257	0,308	0,359	0,411	0,462	0,513
1978 - Model Code 1978	-	0,958	1,150	1,343	1,535	1,727	1,919
1990 - Model Code 1990	-	0,247	0,311	0,375	0,439	0,503	0,567
1992 - ENV 1992	-	0,496	0,595	0,695	0,795	0,894	0,993
2004 - EC2 2004	-	0,443	0,550	0,642	0,734	0,825	0,917
2007 - JSCE	-	0,383	0,459	0,536	0,612	0,689	0,765
2010 - Model Code 2010	-	0,326	0,415	0,504	0,593	0,682	0,771
2016 - Review proposal of EC2	-	0,328	0,418	0,507	0,596	0,686	0,775

Table D 2: Code formulas. Ø18/130 mm. h = 300 mm. c = 50 mm

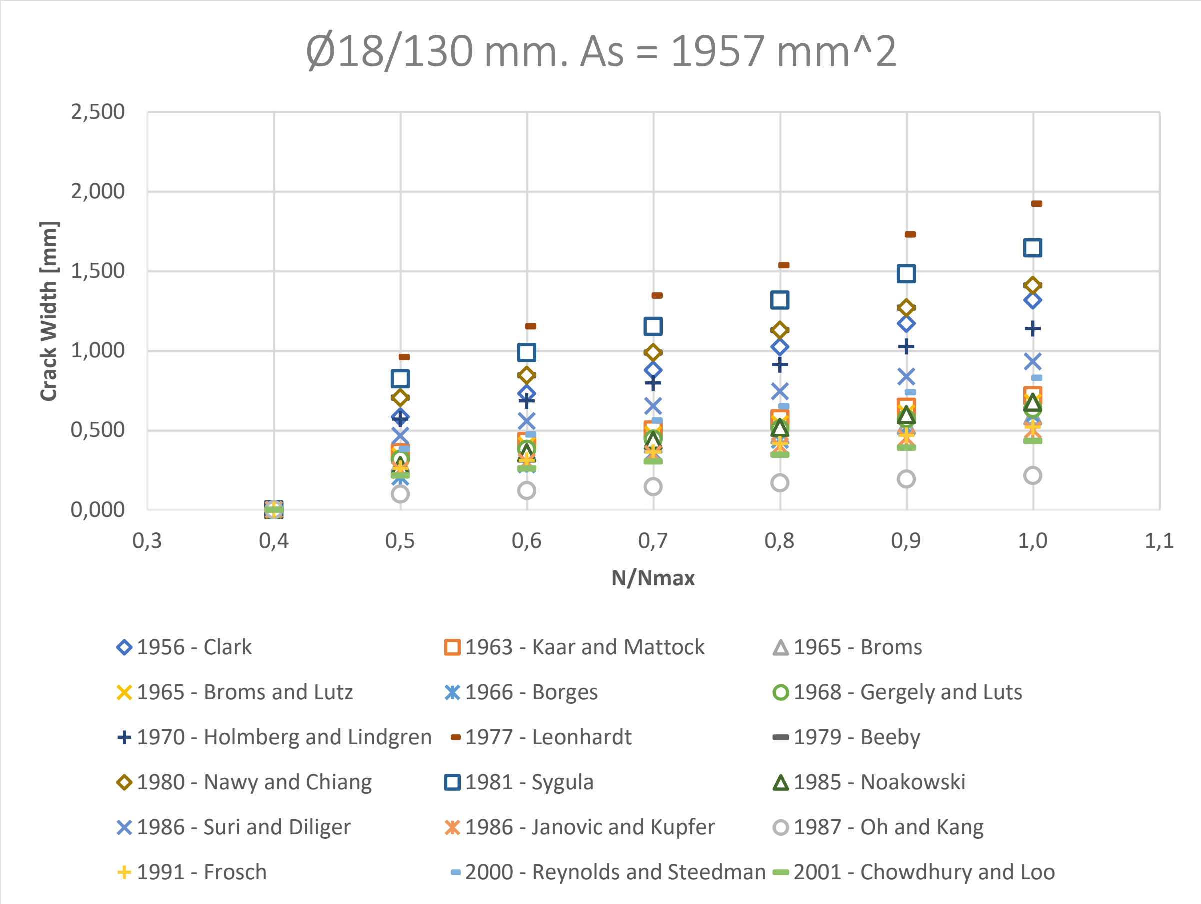

Figure D 1: Literature formulas. Ø18/130 mm. h = 300 mm. c = 50 mm

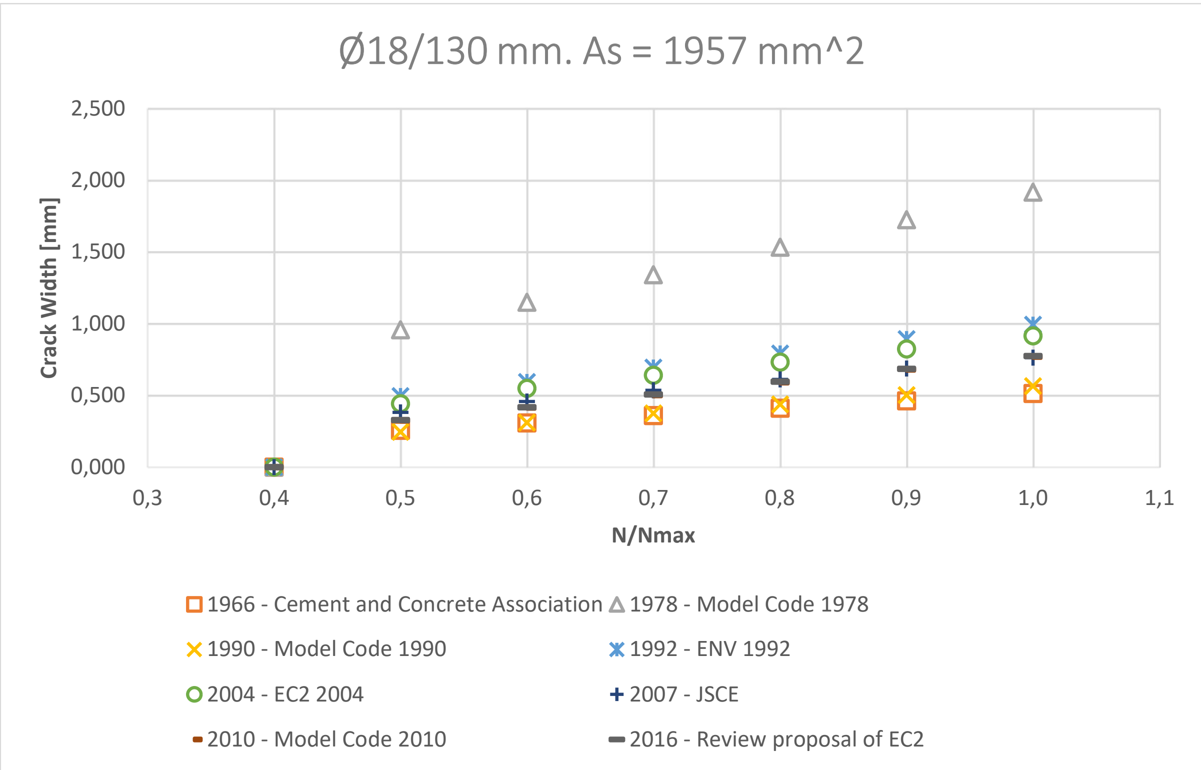

Figure D 2: Code formulas. Ø18/130 mm. h = 300 mm. c = 50 mm

D.2 Ø25/130 mm. As=3776 mm^2

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	3776	3776	3776	3776	3776	3776	3776
N/Nmax	0,21	0,26	0,31	0,36	0,41	0,47	0,52
σc1	2,29	2,81					
1956 - Clark	-	-	0,287	0,345	0,403	0,461	0,519
1963 - Kaar and Mattock	-	-	0,226	0,264	0,301	0,339	0,377
1965 - Broms	-	-	0,194	0,227	0,259	0,292	0,324
1965 - Broms and Lutz	-	-	0,219	0,256	0,292	0,329	0,365
1966 - Borges	-	-	0,123	0,156	0,189	0,223	0,256
1968 - Gergely and Luts	-	-	0,208	0,243	0,278	0,313	0,347
1970 - Holmberg and Lindgren	-	-	0,327	0,381	0,436	0,490	0,544
1977 - Leonhardt	-	-	0,451	0,527	0,603	0,679	0,755
1979 - Beeby	-	-	0,146	0,178	0,210	0,242	0,275
1980 - Nawy and Chiang	-	-	0,339	0,396	0,453	0,509	0,566
1981 - Sygula	-	-	0,712	0,831	0,950	1,068	1,187
1985 - Noakowski	-	-	0,148	0,180	0,213	0,246	0,278
1986 - Suri and Diliger	-	-	0,221	0,258	0,294	0,331	0,368
1986 - Janovic and Kupfer	-	-	0,156	0,182	0,208	0,234	0,260
1987 - Oh and Kang	-	-	0,063	0,077	0,091	0,106	0,120
1991 - Frosch	-	-	0,167	0,195	0,223	0,251	0,279
2000 - Reynolds and Steedman	-	-	0,247	0,294	0,340	0,387	0,433
2001 - Chowdhury and Loo	-	-	0,085	0,099	0,113	0,127	0,142

Table D 3: Literature formulas. Ø25/130 mm. h = 300 mm. c = 50 mm

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	3776	3776	3776	3776	3776	3776	3776
N/Nmax	0,21	0,26	0,31	0,36	0,41	0,47	0,52
σc1	2,29	2,81					
1960 - CBE	-	-	4,657	4,683	4,710	4,736	4,762
1966 - Cement and Concrete Association	-	-	0,162	0,189	0,216	0,243	0,270
1978 - Model Code 1978	-	-	0,560	0,654	0,748	0,843	0,937
1990 - Model Code 1990	-	-	0,116	0,140	0,164	0,188	0,211
1992 - ENV 1992	-	-	0,240	0,280	0,320	0,361	0,401
2004 - EC2 2004	-	-	0,233	0,272	0,311	0,350	0,389
2007 - JSCE	-	-	0,234	0,273	0,312	0,351	0,390
2010 - Model Code 2010	-	-	0,167	0,204	0,241	0,277	0,314
2016 - Review proposal of EC2	-	-	0,168	0,205	0,242	0,279	0,316

Table D 4: Code formulas. Ø25/130 mm. h = 300 mm. c = 50 mm

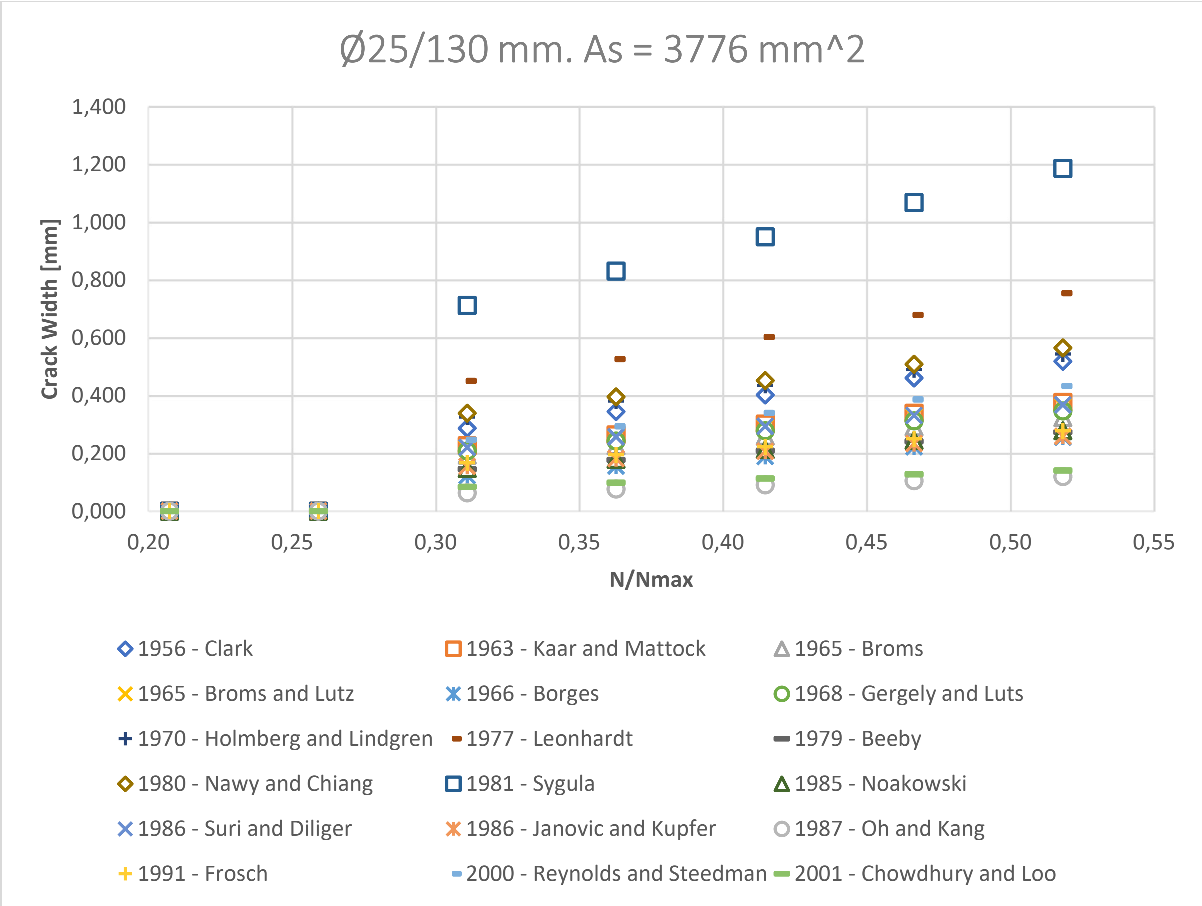

Figure D 3: Literature formulas. Ø25/130 mm. h = 300 mm. c = 50 mm

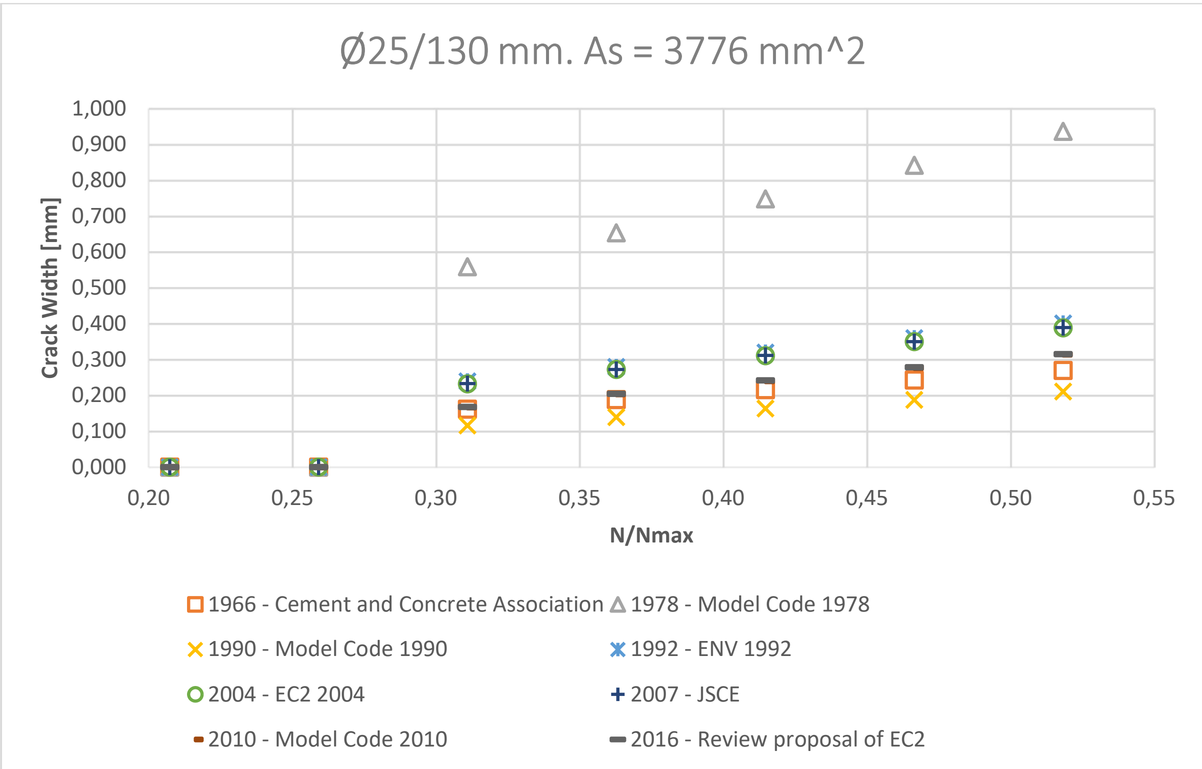

Figure D 4: Code formulas. Ø25/130 mm. h = 300 mm. c = 50 mm

D.3 Ø32/130 mm. As = 6187 mm^2

N [kN]	782,8	978,5	1174,2	1396,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	6187	6187	6187	6187	6187	6187	6187
N/Nmax	0,13	0,16	0,19	0,23	0,25	0,28	0,32
σc1	2,11	2,64					
1956 - Clark	-	-	0,143	0,177	0,202	0,231	0,260
1963 - Kaar and Mattock	-	-	0,140	0,167	0,187	0,210	0,233
1965 - Broms	-	-	0,125	0,149	0,167	0,188	0,209
1965 - Broms and Lutz	-	-	0,140	0,166	0,186	0,209	0,233
1966 - Borges	-	-	0,067	0,087	0,103	0,121	0,139
1968 - Gergely and Luts	-	-	0,134	0,159	0,179	0,201	0,223
1970 - Holmberg and Lindgren	-	-	0,189	0,225	0,253	0,284	0,316
1977 - Leonhardt	-	-	0,224	0,268	0,301	0,340	0,378
1979 - Beeby	-	-	0,090	0,112	0,129	0,149	0,169
1980 - Nawy and Chiang	-	-	0,173	0,206	0,231	0,260	0,289
1981 - Sygula	-	-	0,594	0,706	0,792	0,891	0,990
1985 - Noakowski	-	-	0,076	0,095	0,109	0,126	0,142
1986 - Suri and Diliger	-	-	0,111	0,132	0,148	0,167	0,185
1986 - Janovic and Kupfer	-	-	0,095	0,113	0,127	0,143	0,159
1987 - Oh and Kang	-	-	0,034	0,045	0,054	0,063	0,073
1991 - Frosch	-	-	0,106	0,126	0,141	0,159	0,176
2000 - Reynolds and Steedman	-	-	0,153	0,185	0,210	0,238	0,267
2001 - Chowdhury and Loo	-	-	0,035	0,041	0,046	0,052	0,058

Table D 5: Literature formulas. Ø32/130 mm. h = 300 mm. c = 50 mm

N [kN]	782,8	978,5	1174,2	1396,9	1565,6	1761,3	1957
Nmax (σs2=500 Mpa)	6187	6187	6187	6187	6187	6187	6187
N/Nmax	0,13	0,16	0,19	0,23	0,25	0,28	0,32
σc1	2,11	2,64					
1960 - CBE	-	-	4,579	4,594	4,606	4,619	4,632
1966 - Cement and Concrete Association	-	-	0,100	0,119	0,134	0,151	0,167
1978 - Model Code 1978	-	-	0,326	0,390	0,439	0,494	0,550
1990 - Model Code 1990	-	-	0,055	0,068	0,078	0,089	0,101
1992 - ENV 1992	-	-	0,122	0,145	0,163	0,184	0,205
2004 - EC2 2004	-	-	0,120	0,148	0,166	0,187	0,208
2007 - JSCE	-	-	0,140	0,167	0,187	0,210	0,234
2010 - Model Code 2010	-	-	0,084	0,106	0,123	0,142	0,161
2016 - Review proposal of EC2	-	-	0,085	0,107	0,123	0,143	0,162

Table D 6: Code formulas. Ø32/130 mm. h = 300 mm. c = 50 mm

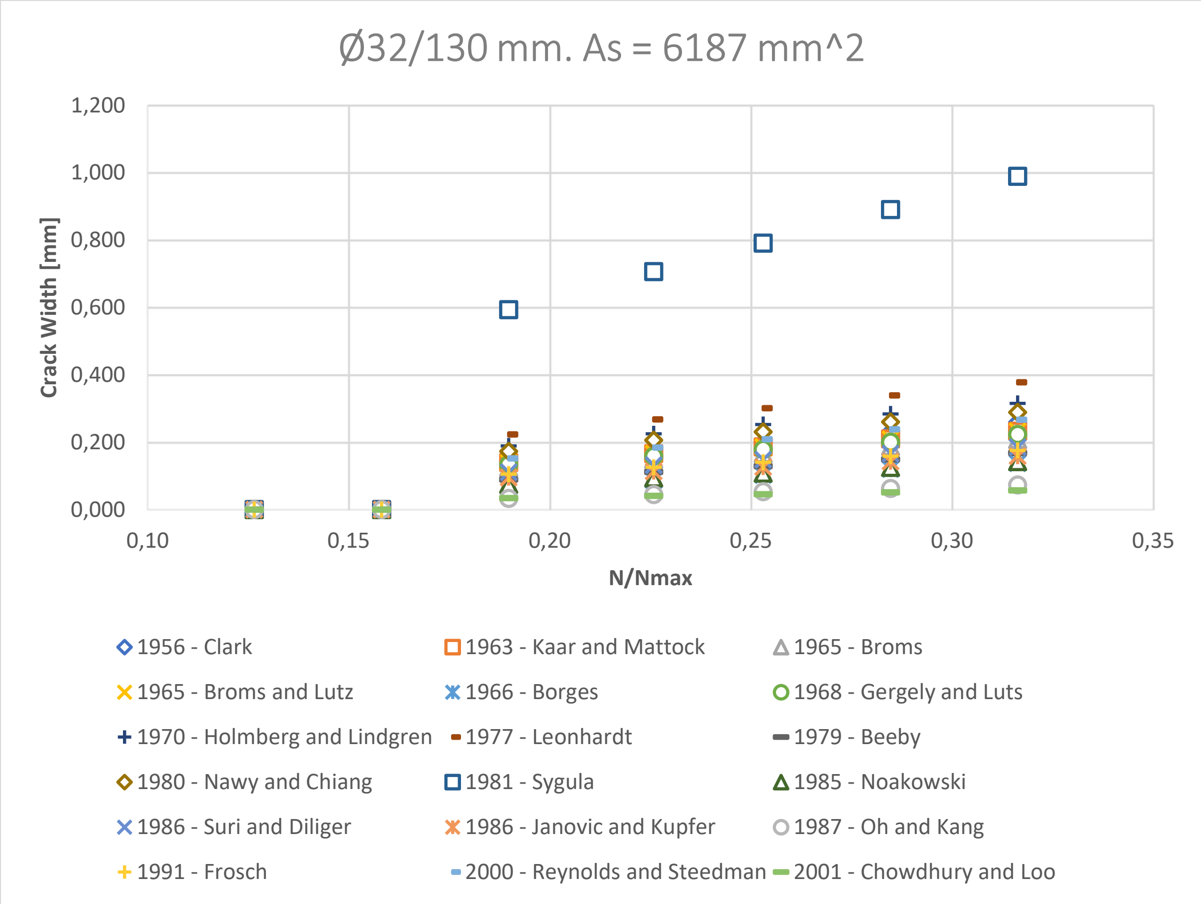

Figure D 5: Literature formulas. Ø32/130 mm. h = 300 mm. c = 50 mm

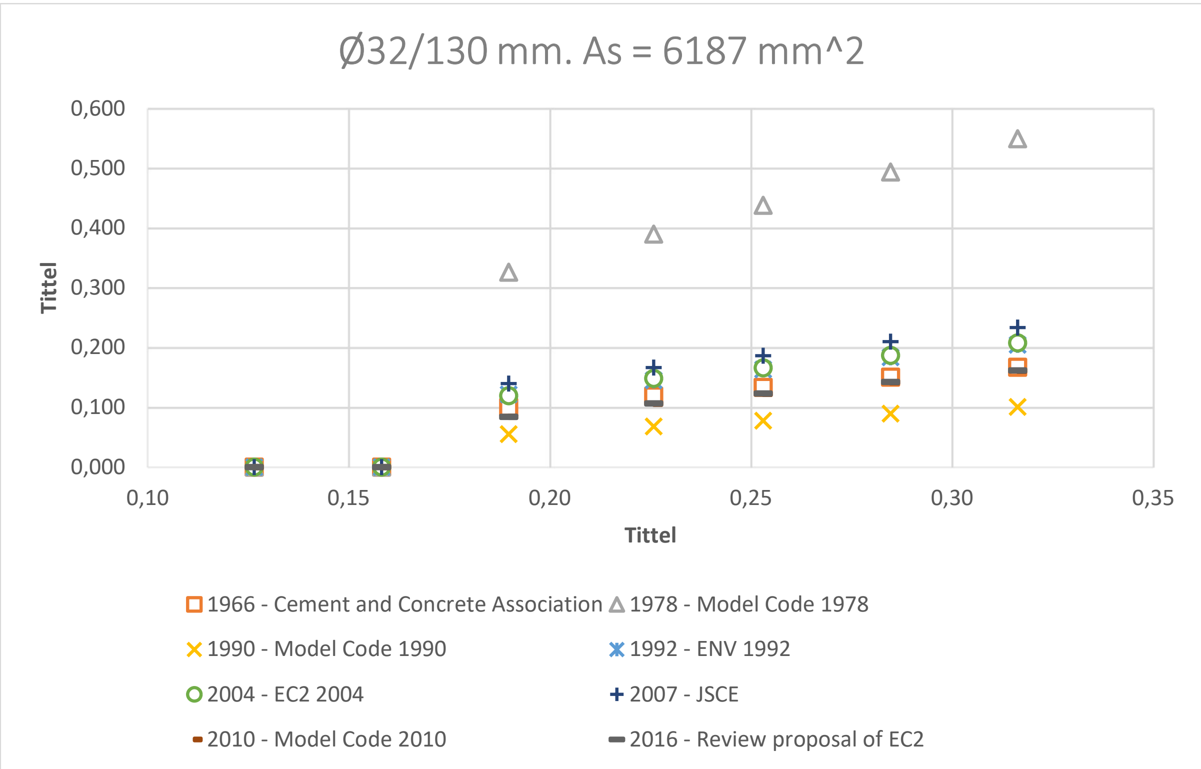

Figure D 6: Code formulas. Ø32/130 mm. h = 300 mm. c = 50 mm

D.4 Steel stress after cracking

N [kN]	782,8	978,5	1174,2	1369,9	1565,6	1761,3	1957
Ø18/130. As = 1957 mm^2	-	250	300	350	400	450	500
Ø25/130. As = 3776 mm^2	-	-	155,48	181,40	207,31	233,22	259,14
Ø32/130. As = 6187 mm^2	-	-	95,35	111,25	127,14	143,03	158,92

Table D 7: Steel stress after cracking. Varying reinf. layout. h = 300 mm. c = 50mm

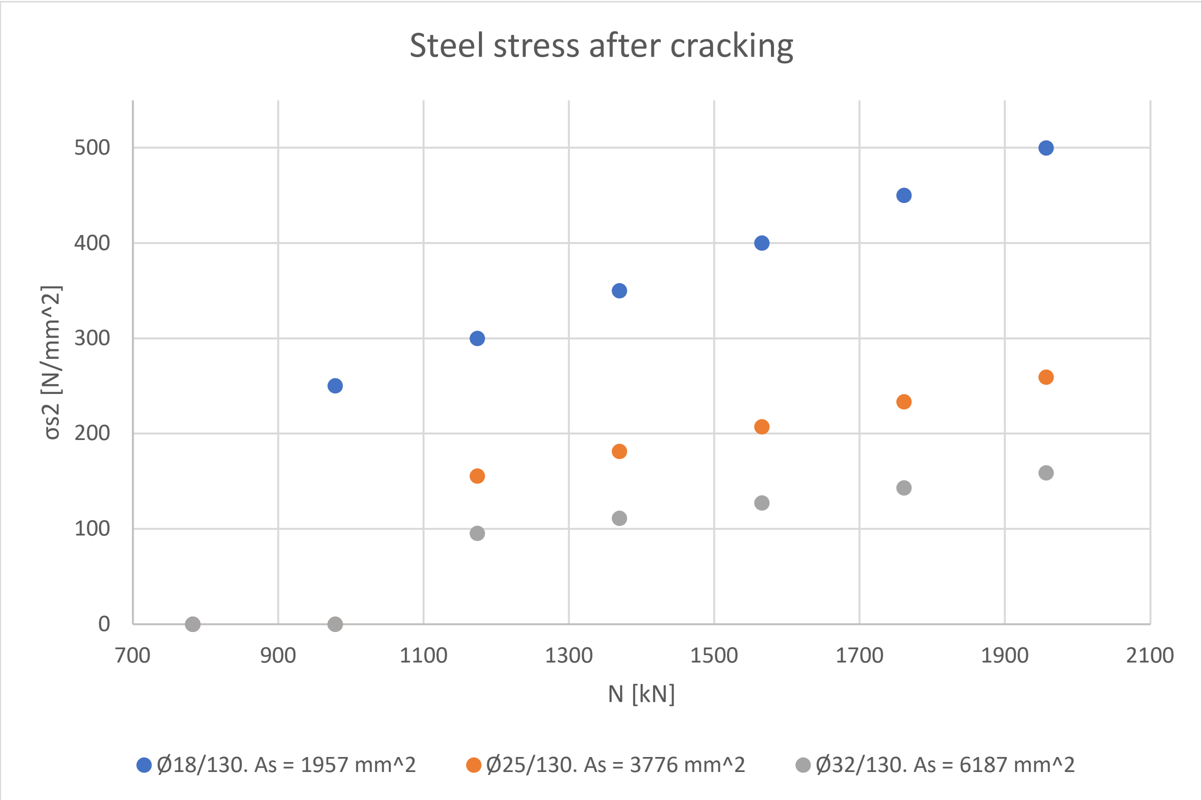

Figure D 7: Steel stress after cracking. Varying reinf. layout. h = 300 mm. c = 50mm