JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

Evaluation of Cracking Limit State Provisions in Different Standard Codes and Numerical Study of Cracking Behaviours of Reinforced Concrete Elements

A Research Thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfilment of the Requirements for the Degree of Masters of Science in Structural Engineering

By: Mohammed Bonso

DECEMBER, 2022

JIMMA, ETHIOPIA

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DECEMBER, 2022

JIMMA, ETHIOPIA

DECLARATION

I, Mohammed Bonso, declared that, this research thesis entitled **"Evaluation of Cracking Limit State Provisions in Different Standard Codes and Numerical Study of Cracking Behaviours of Reinforced Concrete Elements"** is my own original work and all cited were acknowledged.

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EXECUTIVE SUMMARY

Cracking in reinforced concrete structures affects the serviceability and durability of structural members. These effects are mostly occurred due to excessive size of crack width on the sides of RC members. As the crack width increases on the surface of flexural/tensile loading members, corrosion occurrence is probable to the embedded steel reinforcement. To overcome such problems, different standard codes provide analytical expressions to limit the size of crack width. Thus, to have safe and durable structures, comparing the expressions of crack width limits in different standard codes is important to get best practices among the codes.

In this study, the analytical crack width expressions of American, British, and Ethiopian Standard codes were considered. The analytical crack spacing of Ethiopian standard code and Model code (MC2010) were also compared. The study is made through calculation of crack width and crack spacing in the respective codes, comparing it with experimental results in previous studies and Finite Element Numerical modelling in Abaqus. In the comparison, the effects of parameters such as concrete cover, bar diameter, and loading types were discussed.

Firstly, the analytical results of each code crack width limit expression were compared with the experimental crack width results. As a result, while considering the estimation of crack width in different codes, it could be observed that the ACI code gave a relatively good fit to the experimental crack widths than ES-2 and BS codes. On the other hand, for the estimation of crack spacing, the value estimated by MC-2010 gives relatively closer results to the experimental crack spacing than ES-2.

Secondly, to code's crack width and spacing analytical comparisons, the 3D nonlinear finite element model, Abaqus, was used to predict the cracking behaviour of reinforced concrete tensile loading beam. Accordingly, the simulation crack patterns result indicates a good fit to that of the experimental crack patterns result.

In conclusion, when considering the cases of all codes, and based on the percentages of variations, it is observed that the existing codes need to improve the expressions for crack width calculation.

Keywords: Crack Width, Crack Spacing, Type of Loading, Standard Codes, FEA

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ACRONYMS

ACI	American Concrete Institute
BS	British Standard Code
CDPM	Concrete Damage Plasticity Model
EN ES/ES-2	European Norm Ethiopian Standard code (Concrete)
FEA	Finite Element Analysis
RC	Reinforced Concrete Structure
w_k	Characteristic crack width
w_m	Mean crack width
S_{rm}	Average distance between cracks
$S_{r,max}$	Maximum crack spacing
ε_{rm}	Mean strain of reinforcement considering the contribution of concrete in tension
ϕ	bar diameter in mm
A_s	Area of reinforcement contained in effective area, $A_{c,ef}$
$A_{c,ef}$	Section of the zone of the concrete, where the reinforcing bars can influences crack width
σ_{sr}	Steel stress at rupture concrete section
$A_{s,}$	Minimum area of the reinforcement within the tensile zone
A_{ct}	Area of concrete within the tensile zone
σ_s	Yield strength of the reinforcement (service stress)
$f_{ct,}$	Mean value of the tensile strength of the concrete
k	Coefficient allowed for the effect of non-uniform self-equilibrating stresses
c	Cover to the longitudinal reinforcement
$\rho_{p,eff}$	Ratio of effective tension reinforcement
$A_{c,eff}$	Area of effective tension concrete
ε_{sm}	Mean strain under relevant combination of loads and allowing for effects such as tension stiffening or shrinkage
ε_{cm}	Average strain in the solid concrete between the cracks
σ_s	stress in tension reinforcement
$f_{ct,eff}$	The main value of the concrete tensile strength when the cracks can be expected first
n_b	number of tensions reinforcing bars

d_c	distance measured from the centroid of tensile steel to the extreme tensioned fiber
c_{min}	minimum cover to the tension steel;
h	member's total depth;
x	neutral axis depth;
ε_m	average strain at the point of consideration for cracking
a_{cr}	gap from the point to the next longitudinal bar surface
d_b	longitudinal bar diameter;

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CHAPTER ONE

INTRODUCTION

1.1 General

Reinforced concrete structure is a common composite material used in construction. Reinforced concrete is a combination of two dissimilar but complimentary materials, namely concrete and steel. Concrete also known as artificial stone is produced by mixing sand, cement, aggregates, and water. Concrete and steel combined as reinforced concrete (RC) is widely used in construction and other applications. The reinforcement is usually, though not necessarily, steel reinforcing bars and is usually embedded passively in the concrete before the concrete sets. Design methods are formulated based on philosophies, leading to design codes attendant to a particular design method (Huang, 2017).

The use of different design methods and codes will definitely bring about different results in structural analysis and design leading to variability in behavior, costs and durability of structures. It is the duty of the structural engineer to provide designs that would lead to optimum performance and economy by employing the most efficient design method in accordance with a relevant design code available, in order to satisfy the client's requirements. Structural design has gone through various stages, each stage on a design philosophy which tries to produce a structure that will be reversibly safe. It is based on creative ability, imagination and experience, which are all tied to sound scientific principles.

In the structural design, there are two governing criteria for design, ultimate limit state and serviceability limit state, which should be both satisfied (Huang, 2017). ULS focuses on the strength of the structural components to ensure the structural safety, While SLS focus on functionalities of the structure, deflection, cracking and vibration. In SLS, crack width is very important criterion. If the maximum crack width of a structure is larger than 0.3 mm which is the recommended value in EC-2 for reinforcement concrete under quasi-permanent load, the additional reinforcement must be added to control crack width and satisfy SLS criterion(Standard, 2004). For serviceability analysis, the stress transfer between these materials determine the nature of reinforced concrete, development of cracks and the effect of tension stiffening (Jakubovskis *et al.*, 2013).

During serviceability requirements, reinforced concrete structures need a greater concern for behaviours such as excessive deflection and crack width. This is because, durability of reinforced concrete structures to environmental actions is essentially related to controlling crack width under specific environmental conditions, while preventing steel corrosion caused by increasing chloride ions and carbon dioxide (Angst, 2019).

Cracking affect performance of reinforced concrete structures. To minimise these adverse effects, the design of RC structures must ensure that the crack widths under normal service conditions are maintained within acceptable limits under specific environmental conditions. If these cracks are too wide they destroy the aesthetics of the structure and may provoke adverse effect. They also result in steel reinforcement being exposed to the environment causing corrosion of steel. Thus, for RC flexural/tension member, cracking control and cracking widths prediction need important consideration (Abu El Naas, El Hashimy and El Kashif, 2021).

There are different methods to predict the crack width which includes; calculation of crack width in an analytical way by solving the differential equation of bond slip; by semi- analytical equations, by empirical relationships based on fitting of a large number of experimental data, and fracture mechanics models (Balazs, 2005). Considering different factors such as time and cost, Finite Element Analysis can be used to model the behaviour numerically to confirm with these calculations, as well as to provide a valuable supplement to laboratory investigations, particularly in parametric studies(Yadav, 2021).

The design objective for a concrete structure is that, it should be both safe and serviceable, so that the chances of failure during its design lifetime are sufficiently small. The two primary structural design objectives are therefore strength and serviceability. Modern design codes for structures have adopted the limit states method of design, whereby a structure must be designed to simultaneously satisfy a number of different limit states or design requirements, including adequate strength and serviceability(Gilbert, 2011). Minimum performance limits are specified for each of these limit states and any one may become critical and govern the design of a particular member. For each limit state, codes of practice specify both load combinations and methods of predicting the actual structural performance that together ensure an acceptably low probability of failure. In order to satisfy the serviceability limit states, a concrete structure must be serviceable and perform its intended function throughout its working life. Excessive deflection should not impair the function of the structure or be aesthetically unacceptable.

1.2 Statement of Problem

Cracking have significant influence on serviceability, durability and aesthetics of structure. To control such adverse effects on RC structures, different standard codes uses different prescribed limits. In order to have predefined limits, such codes considered different level of severity of environmental conditions. The harassments of environmental conditions are, mostly, coming from industrial countries as they produce large amount greenhouse gases such as carbon dioxide (Berrocal, Löfgren and Lundgren, 2018). As the emission of greenhouse gases are increasing, climate changes are occurring which in turns affect the environmental conditions.

Since RC members are made from concrete and steel, the affected environmental conditions are becoming sensitive to the corrosion of such structures, particularly to that of imbedded steel (Elsener and Angst, 2018).. Such corrosion of steel in RC members are mostly comes through cracking of concrete members. Among different crack patterns, crack width is mostly affecting the performance of such structure as it exposes steel reinforcement to the environment which leads to corrosion (Khorami, Navarro-Gregori and Serna, 2021). As a result, studying the different crack width expression, to limit the width of the crack, in different standard codes and choosing a better expression of crack width analysis, so better standard code, is important.

1.3 Research Question

This research paper has been tried to answer the following question:

- ❖ What are the effects of concrete cover on crack width in different standard codes?
- ❖ What is the effect of bar sizes on the crack width in varies standard codes?
- ❖ What was the effects of bar diameter on crack spacing in different standard code?
- ❖ What are the effects of concrete cover on crack spacing in different standard codes?
- ❖ How cracking pattern of tensile loading member is examined in FE analysis?

1.4 Objectives

1.4.1 General objective

The general objective of this thesis is to investigate the cracking limit states of different standard codes and numerical analysis of cracking behaviours of RC using finite element software.

1.4.2 Specific objective

- To investigate the effect of concrete cover on crack width in different standard codes
- To examine the effect of bar diameter on crack width in different standard codes
- To evaluate effect of bar size on crack spacing in d/t standard codes
- To investigate the effect concrete cover on crack spacing in different standard codes?
- To examine cracking patterns of tensile loading RC beam using Abaqus.

1.5 Research Significance

Evaluation of cracking limit state provisions in different standard codes is important. As a result, different expressions of crack width and crack spacing in different standard codes was evaluated. The goal is to see which expression of the code's best suited to the experimental result, so that, better expression of crack width and crack spacing was suggested. Further, the paper gives some inputs for upcoming researchers while furthers study, particularly, in cracking limit state provision will be considered. Besides to analytical code's crack width evaluations, the findings of this research have tried to tie the cracking behaviour of the simulation result with crack patterns observed from experimental result of reinforced concrete beam subjected to tensile loading. This is taken in to consideration as it saves time, and cost effective in comparison to experimental analysis.

1.6 Scope and Limitation of the Study

This paper was evaluating the crack width and crack spacing of different standard codes for cracking limit state provisions. Standard codes such as Ethiopian, American, British and MC-2010 were taken in to considerations. Parameters such as type of loadings, concrete covers and bars diameter's effect on crack width and crack spacing were examined. Besides, to see cracking behaviours and damage characteristics, sample of experimentally done reinforced concrete beam, which is subjected to tensile loading, was examined using finite element software, Abaqus. And their cracking pattern and propagation have been observed.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

Reinforced concrete structures have been the most common type of structures used in civil engineering construction (Yang *et al.*, 2018). These structures have been widely used for buildings, bridges, and any physical structure to be built. While designing such structures, the structure should be safe and serviceable, so the probability of failure during its design lifetime are sufficiently less. Modern design codes for structures have adopted the limit states method of design, whereby a structure must be designed to satisfy a number of different limit states or design requirements, including adequate strength and serviceability (Gilbert, 2011).

When structural element becomes unfit for its intended use, it may reach a limit state. The limit states for reinforced concrete structures can be divided into two (James K. Wight, 2012). Ultimate limit states which involves structural collapse of part of the structure. Such a LS should have a very low probability of occurrence, because it may lead to loss of life and major financial losses. The major ultimate limit states are; loss of equilibrium of a part or all of the structure as a rigid body; rupture of critical parts, leading to partial or complete collapse. While, Serviceability limit states involves disruption of the functional use of the structure, but not collapse. Because there is less danger of loss of life, a higher probability of occurrence can generally be tolerated than in the case of an ultimate limit state. It includes, excessive deflections for normal service, excessive crack widths etc.

Reinforced concrete structure have cracked before the reinforcement can function effectively. To reduce such cracking, it is possible to detail the reinforcement to minimize, particularly, the crack widths. Excessive crack widths can cause unsightly and allow leakage through the cracks, corrosion of the reinforcement, and gradual deterioration of the concrete. In order to satisfy the serviceability limit states, a concrete structure must be serviceable and perform its intended function throughout its working life. Excessive deflection should not impair the function of the structure or be aesthetically unacceptable. Cracks should not be unsightly or wide enough to lead to durability problems and vibration should not cause distress to the structure or discomfort to its occupants (Ethiopia Standard Agency, 2015).

Modern limit state design principles of ultimate limit state and serviceability limit state design were introduced in the 1970’s (Basteskår, Engen, Kanstad, Johansen, *et al.*, 2019). In many cases, SLS design increases costs due to increased material consumption for measures to limit cracking. The concept of service life is increasingly used for the design of new structures. To produce concrete suitable for a particular application, required service life, design requirements, and expected exposure environments, should be determined before defining the necessary materials and mixture proportions. The use of good materials and proper mixture proportioning will not ensure durable concrete, but, appropriate placement practices and workmanship are essential to the production of durable concrete(Code and Structures, 2013).

However, concrete deterioration can take place for a variety of reasons such as alkali-aggregate reactivity, and corrosion of the embedded steel reinforcement (Yang *et al.*, 2018). Degradation of reinforced concrete structures with time results in decreased performance of structure. Cracking is a form of deterioration that can be observed on reinforced concrete structures in service due to low tensile strength of concrete. These cracks have significant influence on serviceability, durability and aesthetics of structures. If cracks are too wide they destroy the aesthetics of the structure. To minimise these adverse effects, the design of reinforced concrete structures must ensure that the crack widths under normal service conditions are maintained within acceptable limits (Ethiopia Standard Agency, 2015).

2.2 Reinforced Concrete Cracking

Cracks in reinforced concrete structures reduce the durability of the structure (Hosseini and Nolsjö, 2017). It is formed in reinforced concrete members when the tensile deformations from loads reach the tensile deformation capacity of concrete. These cracks can be formed from different loads such as flexure, tension, shear, and/or from imposed deformations such as shrinkage, and thermal movements (Balazs, 2005). Structural cracks can influence both serviceability and durability of structural members. For serviceability, the reduction of stiffness and increase of deformations, the possible water leakage through the cracks and the aesthetical concerns can be mentioned. From the durability point of view, the possible attack of steel corrosion and the reduced service life of structures can be seen (Borosnyói and Snóbli, 2010).

Cracking of concrete structures due to bending or tension has usually great significance on structural behavior (Basteskår, Engen and Fosså, 2019). It causes a damage in concrete structures and results in huge annual cost to the construction industry. In concrete cracking theory there is a stage between uncracked and cracked concrete which is called the, Fracture Process Zone (Hosseini and Nolsjö, 2017). It is a transit zone between intact continuous material and open discontinuous cracks and consists of micro cracks which are situated near the crack tip. As crack propagates, the micro cracks merge into a single crack.

The cracks in reinforced concrete structures create many adverse effects on the durability, aesthetic and liquid tightness of the structure. To avoid the mentioned adverse effects from cracks, it is necessary to repair the cracks, resulting in high repair costs. It is not possible to control all types of cracks; however, it is preferable to limit cracks at the structural design stage. Depending on the controllability of the cracks at the structural design stage, it is classified as controllable and non-controllable. To minimize the occurrence of cracks due to service load, the 'stress of the tensile steel' has to be limited low (Naotunna and Fosså, 2021).

In the design of concrete structures, it is necessary to check the serviceability of the structure particularly in the post-cracking range. External load results in direct and bending stresses may cause flexural and diagonal tension cracks. Immediately after the tensile stress in the concrete exceeds its tensile strength, internal micro-cracks form. These cracks generate into macro-cracks propagating to the external fiber zones of the element. Immediately after the full development of the first crack in a reinforced-concrete element, the stress in the concrete at the cracking zone is reduced to zero and is assumed by the reinforcement (Nawy, 1991).

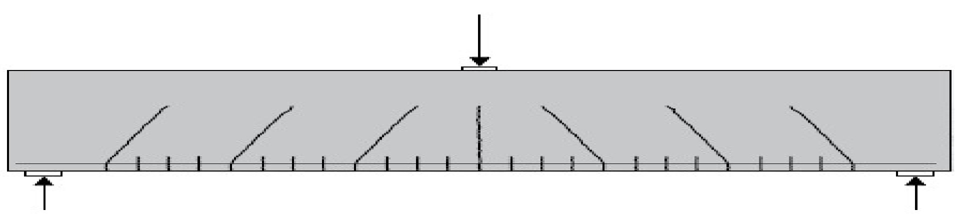
The parameters that might influence cracking are; stress in the reinforcement, surface characteristics of the reinforcement, size of the reinforcing bars, the cover to the bars, the strength of the concrete, the method of loading, degree of concentration of the reinforcement within the tension zone of the beam (Base *et al.*, 1966). Complicated interaction of some of these parameters occurs and it is not possible to investigate each one separately. The occurrence of cracks in reinforced concrete is unavoidable because of the low tensile strength of concrete. The cracking of a reinforced concrete structure at the service load should not be such as to spoil the appearance of the structure or lead to corrosion of the reinforcement but, ensuring that the surface crack widths at service load will not exceed the prescribed limits in different standards.

Crack widths are influenced by shrinkage of concrete and other time-dependent effects and by repeated loading. Crack width measurements are inherently subject to large scatter, even in careful laboratory work. Thus, great accuracy cannot be expected from existing equations for crack widths. The best crack control at the service load is obtained when reinforcing bars are well distributed in the zone of concrete tension. Thus, a number of small-diameter bars are preferable to the use of a few larger-diameter bars.

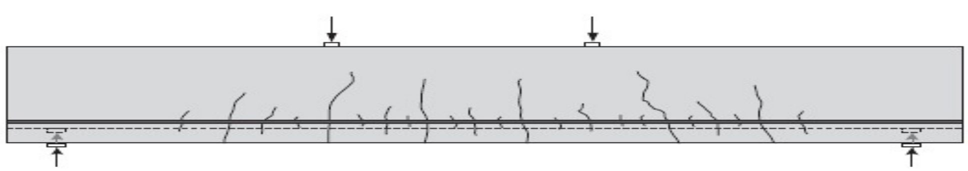
2.2.1 Cracking in flexural members

Reinforced concrete beams and slabs develop flexural cracks wherever the extreme fibre tensile stress reaches the tensile strength of the concrete (Gilbert, 2009). Flexural cracks increase in width as the distance from the tensile reinforcement increases and taper to zero width near the neutral axis. A linear relationship is assumed to exist between the crack width at the side or soffit of a member and the distance from the bar. Many variables influence the width and spacing of flexural cracks in reinforced concrete members includes; the magnitude and duration of loading; the quantity, orientation and distribution of the reinforcing steel, the cover to the reinforcement, the slip between the tensile reinforcement and the concrete in the vicinity of the crack (bond characteristics of the reinforcement); the deformational properties of the concrete (its creep and shrinkage characteristics); and the size of the member (Gilbert and Nejadi, 2008).

Current design procedures to control cracking using conventional steel reinforcement are simplistic and often fail to adequately account for the gradual increase in crack widths with time due to shrinkage. The bonded reinforcement in every reinforced concrete beam or slab provides restraint to shrinkage, with the concrete compressing the reinforcement as it shrinks, and the reinforcement imposing an equal and opposite tensile force on the concrete at the level of the steel. This internal restraining force is often significant enough to cause time-dependent cracking. In a restrained flexural member, shrinkage also causes a gradual widening of flexural cracks and a gradual build-up of tension in the uncracked regions that may lead to additional time-dependent cracking (Gilbert, 2009).



a)



b)

Figure 1: Flexural cracks. a) Three-point loading b) Four-point loading (Gilbert and Nejadi, 2008)

The addition of steel reinforcement that bonds strongly to concrete produces a relatively ductile material capable of transmitting tension and suitable for any structural elements. Reinforcement should be placed in the locations of anticipated tensile stresses and cracking areas. The main reinforcement in a simple beam is placed at the bottom fibers where the tensile stresses develop. There is a proposed design model for crack control in reinforced concrete flexural members based on the tension chord model (Gilbert and Nejadi, 2008).

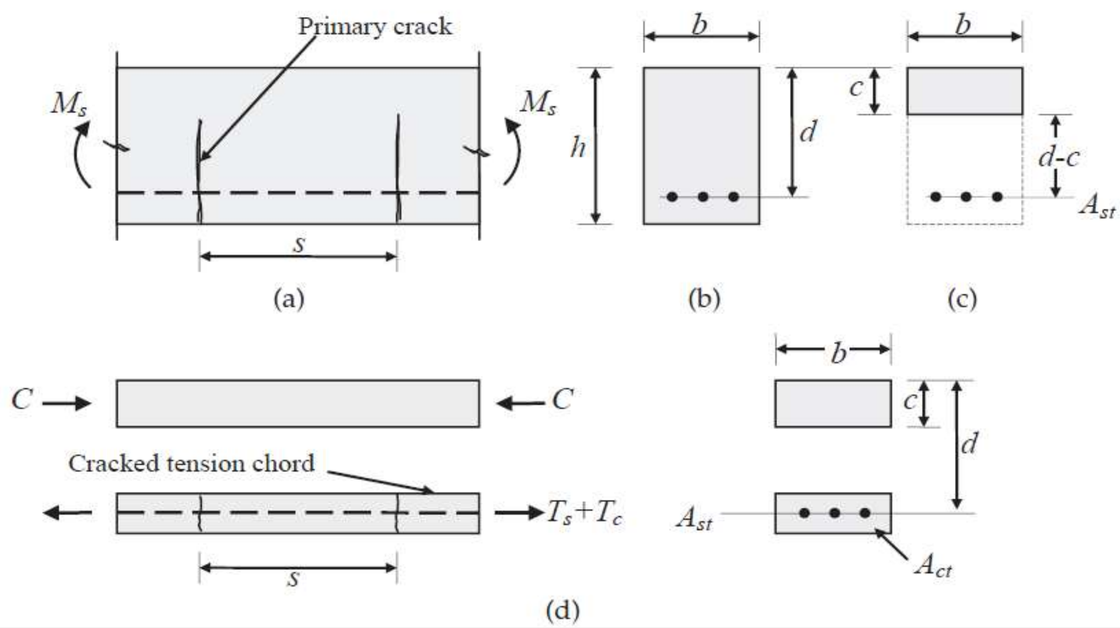


Figure 2: Cracked beam model – (a) beam elevation, (b) uncracked section, (c) cracked section, and (d) idealized compression and tension chord model (Gilbert, 2009)

2.2.2 Cracking in Tension Members

Reinforced concrete is commonly used in civil engineering structures. Since hardened concrete is relatively weak and brittle, it cracks easily under the action of a significant tensile stress. Steel rebars are used to achieve the required tensile strength. The rebars also provide ductility to the structural element under tension and control cracking. When a concrete elements with central longitudinal rebar is subjected to tensile loads at the rebar ends, the load is carried by both the concrete element and the rebar (Yankelevsky, Karinski and Feldgun, 2022). Transfer of the tensile load from the rebar under tension to the surrounding concrete is enabled by the interfacial bond between the rebar and concrete; this phenomenon is enhanced when ribbed rebars are used. Increasing tensile load produces higher tensile stresses in concrete and causes it to crack. Variations in tensile stress along the element determine the location of new cracks and the corresponding cracking loads. This also affects the crack width and the structural member's axial stiffness, which decreases with an increasing number of cracks.

However, flexural members may also be calculated as a tensile member with tensile reinforcement embedded in an equivalent concrete layer where its thickness depends on the concrete cover. As cracking is an inherent characteristic of RC, crack width should be limited considering the functionality, durability, and appearance of the RC element. The maximum allowable crack width depends on the environmental conditions and the risk of corrosion attack. Higher exposure risk to likelihood of carbonation, chlorides, freeze–thaw cycles, and chemical attacks lead to smaller crack width (Yankelevsky, Karinski and Feldgun, 2022).

Reinforced concrete structures that transmit loads primarily by direct tension rather than bending include silos, tanks, shells, ties of arches, bridge, and braced frames and towers (American Concrete Institute, 1997). The cracking behavior of reinforced concrete members in axial tension is similar to that of flexural members, except that the maximum crack width is larger than that predicted by the expressions for flexural members. Cracking occurs when the concrete tensile stress in a member reaches the tensile strength. The load carried by the concrete before cracking is transferred to the reinforcement crossing the crack. For low steel ratios, depending on the grade of steel, yielding occurs immediately after cracking if the force in the member remains the same. The force in the cracked member at steel yield is A_sf_y.

2.3 Types and Development of Cracks in RC Structures

Reinforced concrete is a nonlinear material where the stiffness changes as function of crack development (Nordic concrete federation, 2017). Initially the cross-section will be uncracked and behaves in a linear elastic manner. For an increasing load, the tensile capacity of the concrete will be reached, which will result in the first cracks appearing. With increasing load new cracks will be formed (crack formation stage) until a stabilized cracking stage has been reached. In the stabilized cracking stage, no new cracks will form but the existing cracks will become wider. At a certain point, the reinforcement will start to yield. Tensile stresses induced by loads, moments and shears cause distinctive crack patterns, as shown in Fig.3 below.

Members loaded in direct tension crack through the entire cross section. In the case of a very thick tension member with reinforcement in each face, small surface cracks develop in the layer containing the reinforcement (see thick section from Fig.3 below). These join in the center of the member. Members subjected to bending moments develop flexural cracks, as shown in Fig.

3b. These vertical cracks extend almost to the zero-strain axis (neutral axis) of the member. Cracks due to shear have a characteristic inclined shape, as shown in Fig. 3c. Such cracks extend upward as high as the neutral axis and sometimes into the compression zone. At service loads, the final cracking pattern has generally not developed completely, with the result that there are normally only a few cracks at points of maximum stress at this load level.

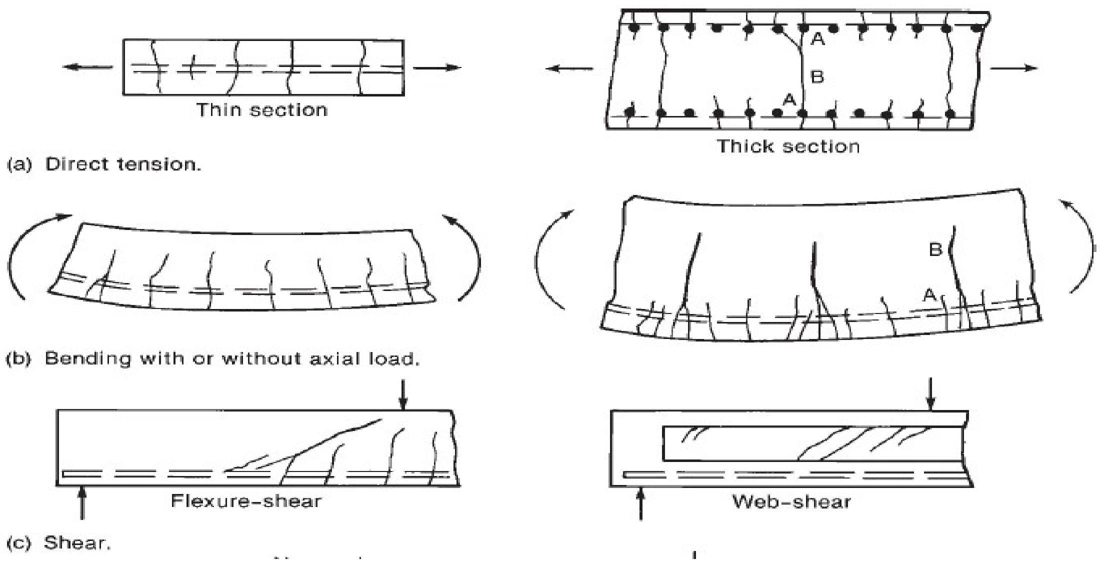


Figure 3: Load-induced cracks (James K. Wight, 2012)

Cracking starts when the tensile stress in the concrete reaches the tensile strength of the concrete at some point in the bar. The distance between stabilized cracks is a function of the overall member thickness, the cover, the efficiency of the bond etc.(James K. Wight, 2012).

2.4 Code Requirement of Cracking Limit State

There is no universally accepted formulation for crack width nor crack spacing prediction for reinforced concrete (Basteskår, Engen, Kanstad and Fosså, 2019). However, different researchers estimate crack width based on experimental behaviour and analytical solutions of the bond-slip relation near the cracks. Most design codes for structural concrete have introduced formulae for the prediction of crack widths together with criteria defining satisfactory service performance in terms of crack-width limits under specified loadings. Besides to crack width calculation formula for the crack width estimation, it was assumed that the concrete quality is adequate, the concrete cover is as intended, the execution is performed according to the requirements in the relevant execution standard, and the design is correct and that no excessive loading or other unintended effects occurs (Standard, 2004), (Code and Structures, 2013).

According to (Ethiopia Standard Agency, 2015) cracking is limited to an extent that will not impair the proper functioning/durability of the structure, cause its appearance to be unacceptable and thus, limiting calculated crack width, $\mathbf{w_{max}}$, have been established.

There are several crack width calculation models available in the literatures. At the design stage, the calculated crack width is limited to an allowable crack width. These calculated crack width models have changed from time to time and differ from region to region (Naotunna, S. S. M. Samarakoon and Fosså, 2021). Furthermore, crack width calculation models in different regions differ from each other, as they are based on different approaches. Empirically based crack width calculation models are found in the American Concrete Institute Code and British Standards Code. Crack width calculation models based on a semi-analytical approach are in Eurocode (Naotunna, S. M. Samarakoon and Fosså, 2021).

Similar to the crack width calculation models, it can be seen that the 'allowable crack width limits' in the widely used codes of practice have changed from time to time and differ from each other. In severe conditions, EC-2 and BS codes limits the crack width to 0.3 mm, while ACI 318 limits the crack width to 0.33 mm. These limits are mainly decided to protect the reinforcement from corrosion. Concrete cover thickness is mainly increased to satisfy the durability aspect, when a long service life is required for an RC structure. Some crack width calculation models have limitations for concrete cover thickness.

Apart from the durability considerations, the thickness of the concrete cover depends on the safe transmission of bond properties and is based on fire resistance. Therefore, even for structures which are not built-in severe exposure classes, there is the possibility to have relatively large covers. If the crack width of a structure with a large concrete cover thickness is controlled to the allowable limits which are prescribed for lower cover thicknesses, additional tensile reinforcement tends to be required. For this reason, it is necessary to identify how the existing allowable limits are decided and what improvements need to be made and apply to structures of higher cover thicknes (Naotunna, S. S. M. Samarakoon and Fosså, 2021).

2.5 Cracking Effect on Durability of RC Structures

The SLS requirements are applied to RC structures to provide their functionality and structure integrity under service conditions (Basteskår, Engen, Kanstad and Fosså, 2019). It is considered by restraining stresses in materials, crack width and spacing restriction, structure element vibration, and long- or short-term deflections. The stress level (tension and compression) should be controlled under service loads in both concrete and steel reinforcement. Under SLS loads, compression stresses are normally limited by design codes to avoid excessive compression stresses, longitudinal cracks and excessive creep deformations.

The durability of RC structures to environmental actions is related to controlling crack width under specific environmental conditions to prevent steel corrosion caused by increasing chloride ions and carbon dioxide(Angst, 2019). While concrete crack, it degrades required performance such as safety and serviceability, due to steel corrosion during the design working life. Water tightness and structure appearance are also important factors for examining cracking and permissible crack width. Both concrete cover and concrete quality are other parameters applied to verify if steel is protected from corrosion due to chloride ion ingress (Khorami, Navarro-Gregori and Serna, 2021)

Widths of cracks should be controlled to a fairly small value (Beeby, 2004). This is because, to reduce the risk of corrosion of the reinforcement; to avoid or limit leakage; and to avoid an unsightly appearance. The influence of the reinforcement layout on the crack pattern and the effect of the crack width and the crack pattern on the corrosion process should well understood. corrosion can be protected through limiting the crack widths by the distribution of minimum reinforcement and allowable steel stress. A main reason for this is that, crack width regulations favour small concrete cover and dense placement of small reinforcing bars that are clearly not good solutions to prevent corrosion (Basteskår, Engen and Fosså, 2019).

Besides to the aesthetic concerns and corrosion of reinforcement, cracking of a reinforced concrete member will cause a significant reduction in the bending stiffness (Allam *et al.*, 2013). Building codes consider the tension stiffening when calculating the crack width of the flexural members. A simple analytical procedure is proposed for the determination of forces, stresses and strains acting on a reinforced concrete section subjected to flexure considering the concrete contribution in tension up to tensile concrete strain corresponding to the cracking strength of concrete. There are two viewpoints regarding the impact of cracking in concrete on reinforcement corrosion (Basteskår, Engen, Kanstad and Fosså, 2019): Cracks significantly reduce the service life of the structure, accelerate corrosion initiation, and provide space for the deposition of the corrosion products, and Corrosion initiation may be accelerated by cracks, but the influence on the subsequent rate of corrosion is minimal and limited to zones where the cracks cross the reinforcement.

Since structural safety is expressed in terms of reliability, which should be maintained over the full-service life of a structure, it is logical that reliability considerations enter the limit state conditions with regard to durability. A limit state of durability is reached when a specified criterion for a certain type of deterioration is reached. For chloride penetration and carbonation these criteria can be reasonably well specified. For a better specification of those criteria, an investigation of older structures with regard to their state of deterioration might be very instructive. Older structures offer an excellent opportunity to calibrate the results of theoretical deterioration models (Code and Structures, 2013).

Cracks provide easy access for oxygen, moisture, and chlorides, and thus, surface cracks can create a condition in which corrosion and cracking are accelerated. The risk of excessive corrosion in concrete structures containing embedded steel can be minimized to promote long service lives. Concrete protects against corrosion of embedded steel because of the highly alkaline environment provided by the pore fluid of the Portland cement paste. The adequacy of the protection depends on the depth of concrete cover, the quality of the concrete, the details of the construction, the degree of exposure to chlorides from concrete component materials and from the environment, and service environment (Angst, 2019).

2.5.1 American Concrete Institute (ACI 318M-19)

The Code addresses concrete durability on the basis of exposure categories and exposure classes. The assigned exposure classes, which are based on the severity of exposure, are used to establish the appropriate concrete properties to include in the construction documents. The Code does not include provisions for especially severe exposures, such as acids or high temperatures. The Code addresses four exposure categories that affect the requirements for concrete to ensure adequate durability (ACI, 2019):

Exposure Category F: Concrete exposed to moisture and cycles of freezing and thawing.

Exposure Category S: Concrete in contact with soil or water containing deleterious sulfate ions.

Exposure Category W: Concrete in contact with water, absorb water from surrounding soil

Exposure Category C: Non-prestressed and prestressed concrete exposed to conditions that require additional protection against corrosion of reinforcement.

Severity of exposure within each category is defined by its own classes with increasing numerical values representing increasingly severe exposure conditions (ACI, 2019). American Concrete Institute (ACI, Committee, 2014) decided to greatly simplify crack control requirements due to increased evidence suggesting a reduced correlation between crack width and reinforcement corrosion (Allam *et al.*, 2012). Beeby (Beeby, 2004) showed that corrosion does not correlate with surface crack widths in the range normally found with reinforcement stresses at service load levels.

2.5.2 British Standard Institution (BS8110, 1997)

To produce a durable structure, it requires the integration of all aspects of design, materials and construction. The environmental conditions to which the concrete will be exposed should be defined at the design stage. Consideration may also be given to the use of protective coatings to either the steel or the concrete, or both, to enhance the durability of vulnerable parts of construction. Concrete should be of the relevant quality; this depends on both its constituent materials and mix proportions. There is a need to avoid some constituent materials which may cause durability problems and, in other instances, to specify particular types of concrete to meet special durability requirements. Good workmanship, curing, dimensional tolerances and the levels of control and inspection of construction should be specified.

A durable concrete element is designed and constructed to protect embedded metal from corrosion and to perform satisfactorily in the working environment for the life-time of the structure. The structural form and cover to steel are considered at the design stage and this involves consideration of the environmental conditions. The main characteristics influencing the durability of concrete are the rates at which oxygen, carbon dioxide, chloride ions and other potentially deleterious substances can penetrate the concrete, and the concrete’s ability to bind these substances. These characteristics are governed by the constituents and procedures used in making the concrete. The factors influencing durability include: the cover to embedded steel, the exposure conditions, the type of cement, the type of aggregate, and water/cement ratio of the concrete etc. The Code addresses six exposure conditions that affect the requirements for concrete to ensure adequate durability (British Standards Institution, 1997):

Mild Environment: Concrete surfaces protected against weather or aggressive conditions.
Moderate Environment: Exposed concrete surfaces but sheltered from severe rain or freezing.
Severe Environment: Concrete surfaces exposed to severe rain, alternate wetting and drying.
Very Severe Environment: Concrete surfaces exposed to sea water spray or de-icing salts.
Most Severe Environment: Concrete surfaces frequently exposed to sea water spray or de-icing.

Abrasive Environment: Concrete surfaces exposed to abrasive action, e.g. machinery, metal tyred vehicles or water carrying solids.

So, according to this code, the widths of flexural cracks at a particular point on the surface of a member depend primarily on three factors (British Standards Institution, 1997): distance from point considered to reinforce bars perpendicular to the cracks; neutral axis distance to the point considered, and average surface strain at the given point. The equation that gives a relationship between the crack width and these three major variables which results in accurate acceptability under most common design conditions have been discussed in chapter three of this paper.

2.5.3 Ethiopian Standard Code (EN ES-2, 2015)

Accordingly, the environmental conditions shall be identified at the design stage so that their significance can be assessed in relation to durability and adequate provisions can be made for protection of the materials used in the structure. The degree of any deterioration may be estimated on the basis of calculations, experimental investigation, experience from earlier constructions, or a combination of these considerations. In order to achieve an adequately durable structure: the intended use of the structure; the expected environmental conditions; properties and performance of the materials; the quality of workmanship and the level of control; the particular protective measures; the intended maintenance during the design working life should be considered (Ethiopia Standard Agency, 2015).

A durable structure should meet the requirements of serviceability, strength and stability throughout its design working life, without significant loss of utility or excessive unforeseen maintenance. The required protection of the structure shall be established by considering its intended use, design working, maintenance programme and actions. The possible significance of direct and indirect actions, environmental conditions and consequential effects shall be considered. Corrosion protection of steel reinforcement depends on density, quality and thickness of concrete cover and cracking. The cover density and quality are achieved by controlling the maximum water/cement ratio and minimum cement content and may be related to a minimum strength class of concrete. The Code addresses six exposure conditions that affect the requirements for concrete to ensure durability(Ethiopia Standard Agency, 2015):

No Risk of Corrosion or Attack: dry Concrete inside buildings with very low air humidity etc.

Corrosion Induced by Carbonation: Concrete inside buildings with low air humidity etc.

Corrosion Induced by Chlorides: Concrete surfaces exposed to airborne chlorides etc.

Corrosion Induced by Chlorides from Sea Water: Structures near to or on the coast etc.
Freeze/Thaw Attack: Moderate water saturation, without de-icing agent etc.
Chemical Attack: Aggressive chemical environment, Natural soils and ground water etc.

Severity of exposure class within each category can be seen by its own sub-classes representing severe exposure conditions (Ethiopia Standard Agency, 2015). According to Eurocode 2, corrosion protection depends on the nominal cover to reinforcement and the cracking of concrete. Further, the density of concrete, water/cement ratio, strength class, etc. can be considered as some of the other factors connected with the durability of the concrete. Environmental conditions are the starting factor considered in selecting the nominal cover to the reinforcements. There are subcategories under each main category depending on the changes in the environment. There are separate articles on this website that discussed the carbonation, corrosion by chlorides, chemical attacks, and Free/Thaw attacks.

Classification of the structures according to the Eurocode is done based on the expected design life, the type of structure or importance of the structure. Classification is solely based on the durability of the structure. It specifies the design life for the structures depending on its type and design life. After selecting the structural class, durability requirements can be finalized. The following table indicates the classification of the structures depending on the type and durability.

Table 1: Classification of Structural Class Based on Types and Durability

Structural Class	Assumed Years	Design Working Life Example Structure
1	< 10	Temporary structures
2	10 to 25	Replaceable parts; gantry, girders, bearings
3	15-30	Agricultural and similar structures
4	50	Building structures and other similar structures
5	100	Monumental buildings, bridges, and others
6	> 100	Special structures

2.6 Corrosion of Steel in RC Structures

Degradation of reinforced concrete due to corrosion of reinforcement is a widespread problem that mostly affects structures in global sense (Berrocal, Löfgren and Lundgren, 2018). The most common causes of reinforcement corrosion are the loss of alkalinity in the concrete cover due to carbonation and the local break-down of the steel passive layer due to the ingress of chloride ions. Cracks are present in most RC members due to mechanical loading, temperature gradients, shrinkage, etc. Such cracks are known to have a negative impact on reinforcement corrosion as they provide a preferential path for external agents to penetrate into the concrete. Although general agreement exist that corrosion of reinforcement initiates earlier for larger crack widths, the role of crack width during the corrosion propagation phase has been a subject of debate among researchers (Berrocal, Löfgren and Lundgren, 2018). However, codes of standards dictate minimum cover depth requirements and crack width limitations to minimize corrosion of reinforcement (ACI, 2019).

As reinforced concrete structures are aging, it start to show signs of degradation due to the corrosion of the reinforcing steel when chloride ions from seawater, deicing salts, or carbonation destroy the protective passive film on the steel (Elsener and Angst, 2018). As a result, these structures need repairing. However, the required repair work affects the environment due to consumption of materials and energy. Besides, corrosion of the infrastructures leads to very high costs. In industrialized countries, the major part of infrastructure has reached or are reaching a critical age where major maintenance work has to be performed. The continuous aging of these existing assets will lead to an increased need of repair. While, in emerging countries, where the civil engineering infrastructure still has to be expanded and built, the main challenge is that these new structures have to be designed and constructed durable. The main problem of the cement industry is the intrinsic emission of CO_2 when producing clinker (Elsener and Angst, 2018).

Reinforcing steel in concrete is protected by a thin oxide film. "Tuutti diagram" shows the evolution of degradation of reinforced concrete over time: at the beginning, no degradation occurs. When the initiation time is reached, the passive film is destroyed (the steel is de-passivated) and corrosion of the reinforcement can start in presence of oxygen and water; this is called the propagation state. The requirements given in the codes of practice, a high cover depth, good quality concrete, and long curing, intend to reach a long-service life of reinforced concrete, thus to preserve the passive state as long as possible by avoiding or at least retarding the ingress of aggressive agents such as chloride ions or CO_2 (Elsener and Angst, 2018).

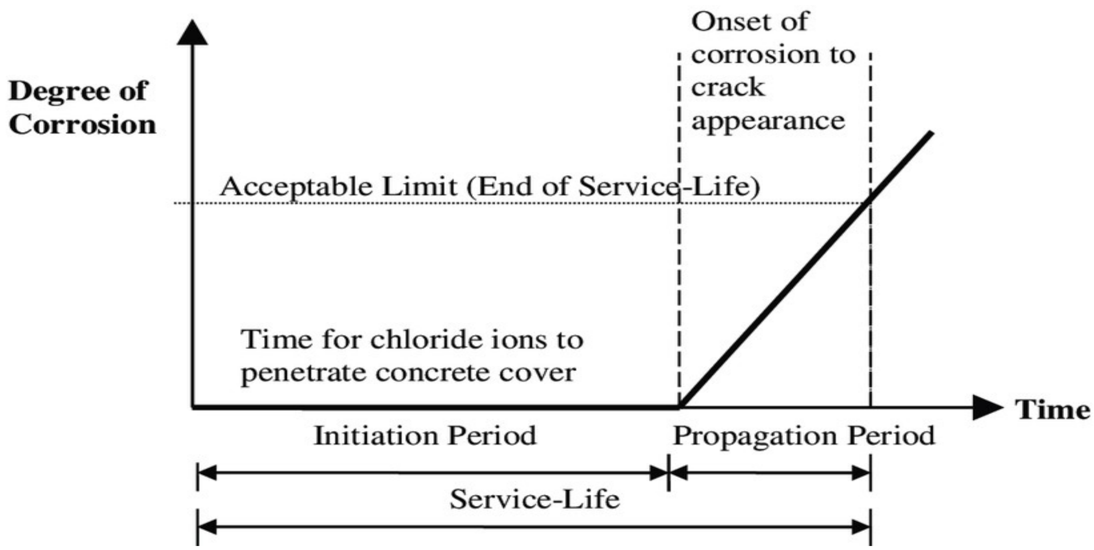


Figure 4: "Tuutti diagram" of RC Structures service life showing different phase (Elsener and Angst, 2018).

2.7 Control of Cracking in RC Members

Crack formation and crack controls are the most important considerations in designing systems of reinforced concrete structures (Abu El Naas, El Hashimy and El Kashif, 2021). It is provided by calculating the probable crack width and proportioning structural elements so that the computed width is less than some predefined value. The basis for codes of practice to limit service-load cracking is rooted in equations to predict crack widths. Number of equations have been proposed for predicting crack widths in RC members; such reviewed in (ACI 224R-01, 2001) and in listed references such as (Basteskår, Engen, Kanstad and Fosså, 2019).

Cracking in reinforced concrete members may be of two forms (Zhao *et al.*, 2012): Cracking due to restraint provided by structure to volume change, and cracking due to applied loads. The purpose of crack control in RC structures were: for durability, aesthetics, protecting rebars from corrosion, and to have water tightness structures (Angst, 2019). To control cracking, ACI considers bar spacing, concrete cover, and bar stress to limit crack widths in proposed equation. However, as per EC2, there are three principal elements in the provisions of crack control (Zhao *et al.*, 2012): The provision of minimum reinforcement area; Method for calculating design crack width, and Simplified rules which will avoid the necessity for explicit calculation of crack width in most normal situations.

Cracks of large widths lead to corrosion, degradation of the surface and thus harm the overall quality of life-being of the structure. More important parameters for corrosion protection are concrete cover and concrete quality. Reinforcement surrounded by concrete will not corrode until an electrolytic cell can be established. This will occur when carbonization of the concrete reaches the steel or when chlorides penetrate through the concrete to the bar surface. The time taken for this to occur will depend on a number of factors: whether the concrete is cracked, the environment, the thickness of the cover, and the permeability of the concrete. If the concrete is cracked, the time required for a corrosion cell to be established is a function of the crack width.

Excessive crack width would reduce the structure's existence by allowing for faster penetration of high humidity corrosive factors, repeated moisture absorption, salt water and chemical-related gases, in ordered to achieve reinforcement. Moreover, structural performance, including rigidity, capacity and ductility, was impaired by cracks in reinforced concrete structures. For RC flexural members' usability; prediction and the cracking control and cracking widths is important. It has been realized that the most practical means to protect the steel against corrosion is by increasing concrete cover (Abu El Naas, El Hashimy and El Kashif, 2021).

2.8 Crack Width of RC Members

Crack width that develop during the loading of flexural members should satisfy the serviceability criteria (Dash, Markandeya Raju and Mishra, 2021). It is evaluated when loading occurs at the tension or compression face of an RC beam. Due to low tensile strength, cracks occur in the RC beam. So, control of cracking is important for obtaining long-term durability for concrete structures, especially for those that are subjected to aggressive environments. When the crack width is increasing, the service life of the structure will be decreasing by allowing more rapid penetration of chlorides to reach the reinforcement causing corrosion. Various factors such as high humidity, repeated loads, and gases with chemicals may cause corrosion of reinforcement and spalling of concrete. Besides, cracking in reinforced concrete structures affects structural performance including stiffness, energy absorption, capacity etc.

In RC structures, minimum reinforcement is needed for two conditions (Pérez Caldentey, Garcia and Corres Peiretti, 2018). One is crack control in serviceability limit state and another is to provide strength in ultimate limit state. The first condition is derived from assuming a linear elastic response of materials (concrete linear in compression with no tensile strength, steel linear both in compression and tension) to the forces producing cracking of the cross section. It addresses cases of pure bending or pure axial forces but, deal only with cracking from imposed strains. Thus, a certain limit to the crack width have been established.

The second condition is aims to guarantee that all cross sections can resist, once they are cracked, the forces that produce cracking. This condition is aimed at avoiding a brittle rupture due to concrete cracking and how robustness reinforcement can be optimized by consideration of fracture mechanics, and its implications on the size effect, and are mostly centred on bending without axial force. The possibility of reducing this minimum reinforcement by taking advantage of the fact that cracks are better controlled near the surface reinforcement and that internal cracks can be allowed to have larger widths.

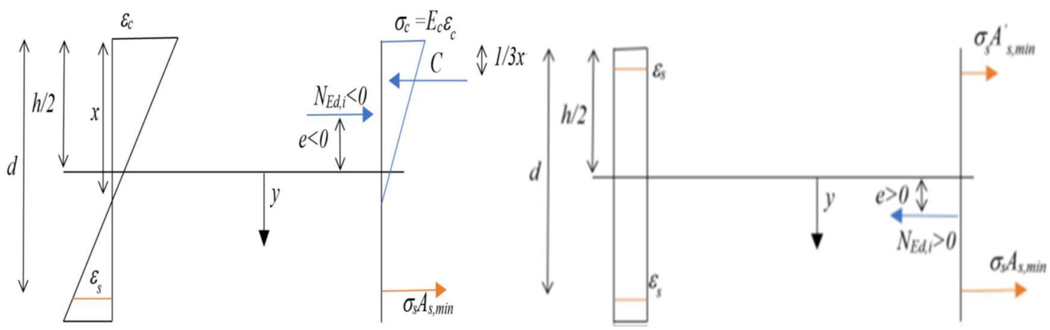


Figure 5: Strains and Stresses (cracked cross section)- a) assuming that the rectangular section part is partially in compression; b) assuming all the section part is in tension (Pérez Caldentey, Garcia and Corres Peiretti, 2018)

For reinforced concrete, the two limit states of cracking are (Ethiopia Standard Agency, 2015): limit state of crack formation and limit state of crack widths. The particular limit state to be checked is chosen on the basis of the requirements for durability and appearance. The requirement for durability depends on the condition of exposure and sensitivity of the reinforcement to corrosion. Adequate protection against corrosion may be assumed to be provided that the minimum concrete covers are complied with and provided that the characteristics crack widths w_k, do not exceed the limiting values given in different codes (Basteskår, Engen and Fosså, 2019). Checking of the limit state of flexural crack widths is generally not necessary for reinforced concrete where at least minimum reinforcement given per code is provided, the reinforcement consists of deformed bars and their diameter does not exceed the maximum allowable values provided in code.

Cracks in reinforced concrete structures are caused by externally applied loads and/or imposed Deformations (Nordic concrete federation, 2017). The crack width caused by externally applied loads will be a function of the magnitudes of the load. The strain level will correspond to the stabilized crack stage where the final number of cracks have been formed. The crack width due to imposed deformations (creep, temperature and shrinkage) is caused by restrained movements. The strain level, cracking stage, is between the formation of the first crack and the stabilized cracking stage.

The occurrence of cracks in reinforced concrete elements is expected under service loads, due to the low tensile strength of concrete. Control of cracking is important for obtaining acceptable appearance and for long-term durability of concrete structures, especially those subjected to aggressive environments. Excessive crack width may reduce the service life of the structure by permitting more rapid penetration of corrosive factors such as high humidity, repeated saturation with moisture, vapour, and gases with chemicals, to reach the reinforcement. The width of a crack depends on the quantity, orientation and distribution of the reinforcing steel crossing the crack and the cover to the reinforcement. It also depends on the bond characteristics between the concrete and the reinforcement bars at and in the vicinity of the crack (Gilbert and Nejadi, 2008).

Requirements related to the maximum crack width are usually related to susceptibility of reinforced concrete structures for the corrosion of the embed steel. Corrosion of a metal is an electrochemical process that requires an oxidizing agent, moisture, and electron flow within the metal. Although concrete provides cover and alkaline environment to the rebar, with carbonation of the concrete cover and cracks on the surface, which enables penetration of hazardous substances, such as chloride ions to the reinforcement, the corrosion may still occur. Therefore, the usual requirements in concrete structures include limiting the maximum allowable crack width due to durability reasons. In more aggressive environment, the structure will be more susceptible to rebar corrosion and the max. allowable crack widths are smaller.

Cracking in reinforced concrete structures has an effect on structural performance including stiffness, energy absorption, capacity, and ductility. Consequently, there is an increased interest in the control of cracking by building codes and scientific organizations. With the use of ultimate strength methodology and high strength reinforcement steel, researchers and designers recognized the need for providing a mechanism by which crack width would be minimized. The crack width of a flexural member is obtained by multiplying the maximum crack spacing by the mean strain of the flexural steel reinforcement. Therefore, the crack width depends on the nature and the arrangement of the reinforcing steel crossing the cracks and the bond between the steel bars found in the tension zone of concrete (Allam *et al.*, 2012).

In engineering design, the crack width calculation in RC beams is based on the assumptions of stress equilibrium between cracked and uncracked cross-sections. The stress is transferred from concrete to reinforcement through the bond. A crucial assumption in this model is the intensity and shape of the bond stress distribution, which determines the crack spacing and the crack width (Rimkus *et al.*, 2020). However, the simplifying assumptions, such as the constant bond stress distribution, and the effective concrete area resisting in tension, make this model valid in average but inadequate in many specific cases.

The tension reinforcement is important when calculating crack widths due to flexural, i.e. bending. According to Euro Code (Standard, 2004) the characteristic crack width, w_k, is calculated based on the difference between the reinforcement and concrete elongation as well as the maximum crack spacing. An accurate crack width checking is a crucial and complicated problem in the design and testing of RC structures (Beeby, 2004). Therefore, a correct establishment of the crack width checking approach for RC structures is theoretically significant and offers a practical value in structural engineering. Existing formulas for crack width checking in RC beams differ among countries, they are inconsistent, and can be roughly divided into two types. The first type is semi-theoretical and semi-empirical formulas, which are based on the analysis of the cracking mechanism, and theoretically derived on the basis of mechanical models. But, some of the coefficients in them are determined from experiments or experience. Formulas belonging to the first type can be found in the US codes, ACI318-99

2.8.1 Crack Width Calculation Models

Different crack controlling methods have been limit the calculated crack width to predefined allowable crack width. There are various types of crack width calculation models, based on different approaches. These includes, the American Concrete Institute (ACI) and the British Standards Institute (BS) models are based on an empirical approach. The governing standards in Europe, which is Euro Code (EC-2) are based on a semi-analytical approach. The mentioned semi-analytical model predicts the crack width by integrating the differences in strain between reinforcement and concrete between two cracks. Therefore, these models identify the crack width by multiplying the crack spacing with the mean strain difference between reinforcement and concrete (Naotunna, Samarakoon and Fosså, 2020).

The theoretical concept of crack width is the integration of the actual strain difference of reinforcement and concrete between two cracks (Naotunna and Fosså, 2021). Due to the nonlinear behaviour of strain variation in both concrete and reinforcement between two cracks, obtaining the crack width explicitly is a complicated process. Therefore, in order to make the crack width calculation model less complicated or more user-friendly, many codes use simplified or semi- analytical approaches. Examples of such models are found in EC2, MC2010. On the other hand, codes like ACI and BS use crack width calculation models based on empirical approaches. The models are developed by considerable amount of tested data.

The theories used for deriving the crack width formulas are based on conceptually two completely different approach (Tan, Hendriks and Kanstad, 2010): The slip theory and no-slip theory. Semi-analytical models predict the crack width by multiplying the crack spacing with the strain difference between reinforcement and concrete. Many studies have identified that it is vital to improve the 'crack spacing' model, in order to improve the crack width calculation models (Naotunna, Samarakoon and Fosså, 2020).

The crack spacing models in the semi-analytical models are based on the two main theories (Naotunna and Fosså, 2021): 'bond-slip' and 'no-slip' theories. According to the bond-slip theory, since a 'slip' is assumed at the reinforcement-concrete interface, a bond-stress would generate. Therefore, the governing crack spacing parameters from this theory are bond parameter ø/ρ, bond stress and others. However, in no-slip theory, the governing crack spacing parameters can be considered to be concrete cover thickness and the distance between tensile reinforcement (thickness of the surrounding concrete of the tensile reinforcement). The EC2 and MC2010 models are based on the combined theory, which considers the cracking behaviour to be based on the combination of the two theories.

There are different crack width prediction models in the codes of practice. These prediction models have been selected, as they can represent the different regions of the world. As an example, EC-2 has been selected, as it was the governing codes of practice in Europe, from which Ethiopian code have been derived. And the crack width prediction models in American code, and British code have been also considered. When examining the models proposed by these codes, it can be observed that different governing parameters were mentioned in crack width prediction models. The crack width governing parameters have been categorized based on the mechanical properties of concrete and reinforcement, properties of interface, cross-sectional properties of the RC member and loading conditions.

The EC-2 code models, which are based on a semi-analytical approach, consider a higher number of parameters than the empirically based ACI and BS code models. Further, it is clear that, although the mentioned models have been developed based on different approaches, the concrete cover thickness parameter is included in every model. The calculated crack width from these models causes the crack width to increase with the increase in concrete cover. The models in EC-2 specifically mention their applicable limitations for concrete cover thickness. The models in the ACI and BS codes do not mention such limitations. The main reason could be that the commonly used concrete cover thickness in the period of developing the code might not be as large as the current requirement. It is important to note that the empirically based crack width calculation models developed by the ACI and BS codes have considered test specimens with concrete cover thicknesses of 84 mm and 89 mm, respectively.

2.8.2 Crack Width Analysis Methods

There was different approach of crack width evaluation as mechanical and empirical (Tue *et al.*, 2021). Mechanical model provides a solid basis for design tools, such as tables for maximum bar diameter or maximum bar spacing, and developing design model for minimum reinforcement to limit the crack width. Essential features of the mechanically based approach are to consider the crack width as a local problem and clearly differentiate between crack stages (single crack or stabilized crack stage). In case of single cracks, the concrete cross section's entire tensile area is activated during cracking and the force to be applied via the bond can be determined. For stabilized crack stages, part of the concrete tensile area is involved in the crack formation in the case of high members with a reinforcement concentration in the surface zone, and the force to be applied via the bond depends on the crack spacing and bond stiffness.

The bond force possible at stabilized crack stages is smaller than the cracking force of the tensile area in the uncracked state, because the concentration of the bond force in the locality of the reinforcement leads to the formation of the so-called secondary cracks with smaller crack depths. Secondary cracks resulting from the bond force always occur when the bond force reaches the cracking force of the concrete tensile area affected by the reinforcement. The relevant concrete tensile area in the stabilized crack stage can be limited to $A_{c,eff}$, whereas in the single crack stage, the entire concrete cross-sectional area must always be considered.

Another method of calculating crack widths uses an empirical approach with statistical analyses of experiments. On this basis, equations are established, which predict observations made in experiments with an accepted exceedance probability. For crack width calculations, these models, however, lose focus on the actual member conditions as soon as the stress level deviates or no stabilized crack stage is available at all. As for the derivation of design tools, such as tables for maximum bar diameter or maximum bar spacing, and for developing practice-oriented design models for minimum reinforcements to limit crack widths, calibrated design models implicate uncertainties. Another problem always arises when the concrete tensile area affected by reinforcement ($A_{c,eff}$) is determined via a multiple of the bar diameter. It was outlined that determining the acceptable bar diameter as a calculation result requires an iterative solution since the bar diameter would also be the input parameter of the calculation.

2.8.3 Effects of Different Parameter on Crack Width

2.8.3.1 Effect of Reinforcement

Reinforced concrete structures are relative brittle and therefore usually contain tensile reinforcement. When the concrete cracks the tension reinforcement carries the tensile forces instead of the concrete. With higher reinforcement content, the difference between the cracking load and the ultimate load is expected to increase (Hosseini and Nolsjö, 2017). Cracks increase the risk for penetration of dangerous substances which accelerate the degradation of both concrete and reinforcement. It is difficult to avoid the appearance of the cracks; however, they can be limited by using proper reinforcement. One alternative is to combine the tensile reinforcement with crack reinforcement to minimize the crack width. The function of the reinforcement is to distribute the cracks over the cross section; many smaller cracks occur instead of fewer, wider cracks. Small cracks are seen as less of a problem compared to large cracks since larger cracks reduce the durability significantly (Hosseini and Nolsjö, 2017). Therefore, the design of tension reinforcement is important since the serviceability should be retained even after the structure cracks.

2.8.3.2 Effect of Tension Stiffening

Tension stiffening is the effect of concrete acting in tension between cracks on the stress of steel reinforcement (Allam *et al.*, 2013). It is the average contribution of the concrete in tension. At a crack, all the internal tensile force is carried by the reinforcement, whereas between cracks some amount of the tensile force is transferred through bond to the surrounding concrete, which results in a reduction in the reinforcement stresses and strains, and causes the reinforcement strain at uncracked zone to be less than the reinforcement strain at the cracked sections. However, the tension stiffening is reduced due to the creep effect and the cyclic loading, which induces an additional excessive slip between the steel and concrete.

Tension stiffening in a reinforced concrete member arises from tensile stresses carried by the concrete (Gilbert, 2015). It contributes significantly to the stiffness of the member and is an important consideration when designing for deflection and crack control at the serviceability limit states. Tension stiffening increases with an increase in tensile stress in the concrete. Conversely tension stiffening decreases when the tensile stress in the concrete drops and, under constant load, this is caused either by cracking, by tensile creep or by a time-dependent deterioration of bond. Cracking can be caused by external loads or by restraint to imposed deformations, such as drying shrinkage (Gilbert, 2015).

The stiffness of the concrete decreases after cracking but does not drop to zero because the uncracked concrete between adjacent cracks is still able to carry some tensile stresses to contribute to the overall stiffness of the member (Ng, Lam and Kwan, 2010). It occurs in both axial and flexural members. In axial members, such as struts subjected to uniaxial loads and panels subjected to biaxial loads, tensile stresses are induced in the concrete between cracks mainly by the stress transfer through the steel reinforcement–concrete bond. In flexural members, tensile stresses are induced in the concrete between cracks not only by the stress transfer through the steel reinforcement–concrete bond but also by the shearing action of the curvature of the flexural member.

The most popular concept is that, by the long-term loading, the tension stiffening value reduces to approximately half its initial value (ACI 224R-01, 2001). Concrete tension stiffening is not affected by the steel stress when the stabilized crack pattern has been formed. This means that in the cracked stage, the tension stiffening is a constant value and is affected only by the concrete tensile strength and the bond nature between the steel and concrete at the tensioned region of the section, regardless the steel stress level. Building codes consider the tension stiffening when calculating the crack width of the flexural members. A simple analytical procedure is proposed for the determination of forces, stresses and strains acting on a reinforced concrete section subjected to flexure considering the concrete contribution in tension up to tensile concrete strain corresponding to the cracking strength of concrete. This analytical method gives the minimum value (lower bound) of tension stiffening.

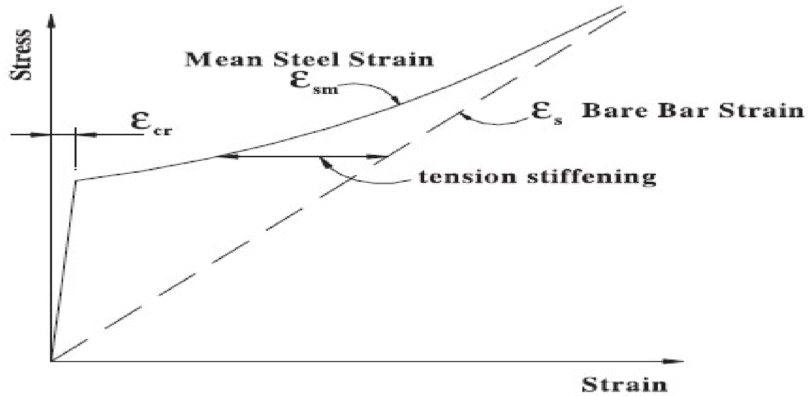


Figure 6: Mean and bare bar steel stress–strain relationship (Allam *et al.*, 2013)

2.8.3.3 Effect of Concrete Cover

Cover is the shortest distance between the surface of a concrete member and the nearest surface of the reinforcing steel. The concrete cover protects the steel reinforcement against corrosion in two ways: —providing a barrier against the ingress of moisture and other harmful substances and forming a passive protective (calcium hydroxide) film on the steel surface. The cover provides corrosion resistance, fi re resistance, and a wearing surface and is required to develop the bond between reinforcement and concrete. It should exclude plaster and any other decorative finish. Too large a cover reduces the effective depth and is prone to cracking, whereas too less may lead to corrosion due to carbonation of concrete.

2.9 Crack Spacing

Crack spacing has been identified as an important parameter in predicting the crack widths in reinforced concrete structures (Naotunna, Samarakoon and Fosså, 2021). Different experimental programs have been conducted to investigate the crack spacings when reinforced concrete beams are subjected to both axial tension and flexure. The existing crack spacing prediction models have been also selected based on different theoretical approaches, namely bond-slip, no-slip, and combined approaches.

Crack spacing has an influence on the corrosion rate of the reinforcement (Naotunna, Samarakoon and Fosså, 2020). Therefore, identifying a good crack spacing prediction model can be advantageous in other ways than just estimating the crack width. When building structures in adverse environmental conditions, large concrete cover thicknesses are required. Therefore, as the cracks do not coincide with the stirrup positions in large covers, a good crack spacing prediction model can be used to predict the distribution of cracks.

The theoretical approach to crack spacing estimation has mainly been based on two approaches (Naotunna, Samarakoon and Fosså, 2020): bond-slip approach, and no-slip approach. Both approaches are based on the two assumptions that a crack is formed when the stress in concrete reaches its tensile strength and at the crack, total force is carried only by the reinforcement. The main idea of bond-slip approach is that the slip occurs between reinforcement and concrete. It has been considered that the slip is largest at the crack and decreases when moving away from the crack. Due to this slip, the concrete strain is not similar to the reinforcement strain at the crack. When the slip is zero between two cracks (at the transfer length away from the crack), the concrete strain can be considered similar to the reinforcement strain. According to this approach, the theoretical crack width is considered similar along the concrete cover thickness. In no-slip approach, a perfect bond is assumed between reinforcement and surrounding concrete. Therefore, it does not allow slip to occur, and, theoretically, the crack width at the reinforcement is zero (Naotunna, Samarakoon and Fosså, 2020).

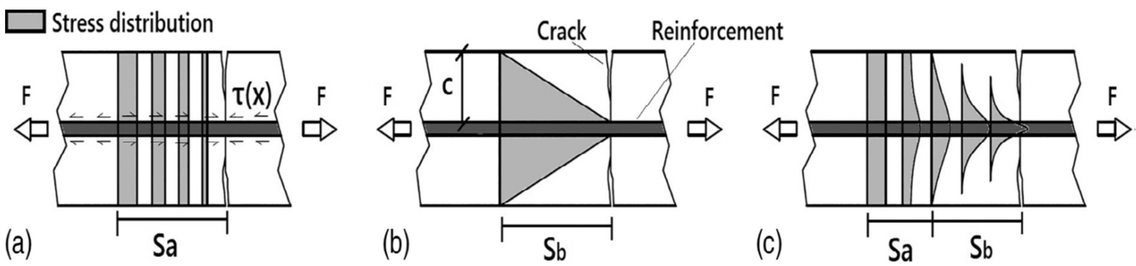


Figure 7: Estimation of crack spacing (a) bond-slip approach; (b) no-slip approach; (c) combined approach (Naotunna, Samarakoon and Fosså, 2020)

Crack spacing parameter can be identified as an important factor in crack controlling criteria. In some aforementioned codes, the concrete cover thickness and $\emptyset/\rho_{p,ef}$ parameter have identified as the two most governing factors of the crack spacing model. In order to assured the existing models, this paper have studied the behavior of aforementioned parameters with the help of available experimental literature. The concrete cover parameter and $\emptyset/\rho_{p,ef}$ parameter is available in the existing crack governing models are due to the 'no-slip theory' and 'bond-slip theory'. The 'no-slip theory' assumes a perfect bond between reinforcement and surrounding concrete. The 'bond-slip theory' considers that a slip occurs between the reinforcement-concrete interface.

Increase of concrete cover, cause to increase crack spacing. The higher number of smaller diameter bars consist of higher concrete-reinforcement interface area than the few numbers of large diameter bars arrangement. When bond area becomes large, transfer length will be low. In theory, the RC specimens with similar cross-sectional area and similar steel area, the larger bar diameter has the higher $\emptyset/\rho_{p,ef}$ value. According to EC-2 crack spacing model, this cause to predict larger crack spacing values than the specimens with smaller bar diameter.

Moreover, Beeby (2004) concluded that the $\emptyset/\rho_{p,ef}$ parameter does not influence on crack width or crack spacing.

There are different reasons why the previous experimental findings shows that $\emptyset/\rho_{p,ef}$ parameter (which appears from the 'bond-slip approach') does not influence on crack spacing. The bond per surface area of every reinforcement is not similar, due to the different rib geometry. Moreover, when the diameter of a bar increases, the bond strength increases, due to the increase in rib area and height relative to the smaller bar diameters. Therefore, the assumption made on developing the existing crack spacing models that the bond stress is similar among every bar diameter have to be reconsidered. The rib pattern or height are considered as the governing factors of the bond-strength and bond-stiffness of a reinforcement.

The investigations show that there is a direct connection between the crack spacing in tensile members and secondary crack spacing in the flexural members because both can be described from the same two parameters; cover and φ_s/ρ_s-ratio. However, the empirical coefficients are slightly different for the two different structural members. The Eurocode suggests that the difference should be a reduction in the coefficient of the φ_s/ρ_s-term from tensile members to the $\varphi_s/\rho_{s,eff}$-term for flexural members. This, however, means that a beam with the same cover and $\varphi_s/\rho_{s,eff}$-ratio as a tension bar has a smaller crack spacing than the tension bar, which is not seen in the comparison. Instead it was concluded that while the influence of the $\varphi_s/\rho_{s,eff}$-term should be reduced in flexural members compared to tensile members, the influence of the cover-term should be increased simultaneously. Moreover, from the comparison, there was no indication that the $\varphi_s/\rho_{s,eff}$ should be reduced as much as the Eurocode dictates.

2.10 Finite Element Analysis

Experimental testing is commonly carried out to study specific structural elements and the concrete strength under different loading conditions. This method gives the actual behaviour of the structure. However, this takes time and is quite expensive. Recently, the finite element analysis method has been used for the evaluation of structures, providing an accurate prediction of the structural elements subjected to different types of structural loading. Since the FEA

method is much faster than the experimental work, this method has becoming the favoured to study the behaviour of concrete elements (Halahla, 2019).
However, the comparison and application of the existing FE models are difficult due to the differences in the adopted modelling strategies, which involve a considerable number of options such as (Yang *et al.*, 2018): regarding the concrete constitutive models, critical parameters, bond between the concrete and the steel reinforcement, and numerical analysis procedures. Damage of structures is significantly influenced by the properties of the material used. Concrete is a quasi-brittle material and its tensile stress slowly decreases after it reaches ultimate value. But, the tensile strain remains to increase. The concept of strain softening develops from plasticity in which the post ultimate decline of the tensile stress is considered as a gradual fall of the tensile strength.

Finite element analysis is effective for investigating the nonlinear behaviour of reinforced concrete structures and performing parametric studies at lower costs than experimental tests (Mathern and Yang, 2021). The source of nonlinearity of reinforced concrete structures is mainly divided into three categories (Sihua, Ze and Li, 2015) : Material nonlinearity, Geometric nonlinearity and Boundary conditions nonlinear. Material nonlinearity refers not only to consider the elastic properties of the linear phase, but also consider the nonlinear stage of its plastic properties when we analyse the mechanical properties of the steel and concrete.

In ABAQUS, achievement of nonlinear characteristics of the materials is through definition of steel and concrete constitutive model. In the elastic stage, we enter elastic modulus and Poisson's ratio of the two materials. But the definition of the plastic stage is different: Stress-Strain relationships of materials have entered. However, three available models can be chosen in the concrete plastic stage (ABAQUS, 2014): Concrete Smeared Cracking, Concrete Damaged Plasticity and Cracking Model for Concrete in ABAQUS/Explicit. The Plasticity model of concrete damage has certain advantages: it can be used in the individual load, cyclic loading, and dynamic loading. It has good convergence, so plasticity model of concrete damage for concrete plastic definition is commonly used.

The failure of structures is significantly influenced by the properties of the material used. Concrete is considered to be a quasi-brittle material, in which the tensile stress gradually decreases after it reaches its peak value while the tensile strain continues to increase. As such, the stiffness degradation behaviour is called strain softening. The concept of strain softening evolves from plasticity in which the post peak decline of the tensile stress is considered as a gradual decrease of the tensile strength, i.e., softening. Concrete exhibits the strain softening behaviour because there is an inelastic zone developed ahead of its crack tip, often referred to as fracture process zone (FPZ). When a crack propagates in concrete, the cracked surfaces may be in contact and are tortuous in nature, resulting from various toughening mechanisms, such as aggregate bridging, void formation, or micro crack shielding(Yang *et al.*, 2018).

2.10.1 Concrete Damaged Plasticity Model

Finite element analysis (ABAQUS) provides tools for simulating damage in concrete using one of the crack models for RC: Smeared crack concrete model, Concrete damage plasticity (CDP) model, and Brittle crack model. The CDP model was chosen in this study for simulating concrete. It allows the definition of inelastic behaviour of concrete in compression and tension stiffening in tension, including damage characteristics in both tension and compression. The CDP model can be used in applications in which concrete is subject to static and cyclic loading (Shamass, Zhou and Alfano, 2015).

The nonlinear behaviour of concrete is attributed to the process of damage and plasticity (Tao and Chen, 2015). The plasticity behaviour can be characterized by several phenomena such as strain softening, progressive deterioration and volumetric expansion. These lead to the reduction of the strength and stiffness of concrete. Damage is usually characterized by the degradation of stiffness. An isotropic scaled damage model from the continuum damage mechanics is introduced in FE Software (ABAQUS, 2014) to describe the stiffness degradation, which can be represented by under uniaxial loading:

$$\sigma = (1 - d) * E_0 * \left(\varepsilon - \varepsilon^{pl}\right)$$

Where; σ, ε, and ε^{pl} represent respectively; the stress, total strain and plastic strain; E_0 is the initial (undamaged) elastic stiffness and d the damage factor, which characterize the degradation of the elastic stiffness and has values in the range between 0 (undamaged) to 1 (fully damaged). The current degraded stiffness E is defined as:

$$E = (1 - d) * E_0$$

If no damage is considered in the concrete (d = 0), stress is reduced to:

$$\sigma = E_0 * \left(\varepsilon - \varepsilon^{pl}\right)$$

Stiffness degradation models can be classified into two types according to the presence of irreversible deformation/plastic strain (Tao and Chen, 2015): Elastic degradation models and

Plastic degradation models. The elastic degradation models are associated with the total strain, implying that no plastic strain exists. The concept of elastic degradation is associated with the total deformation but without the necessity of a damage criterion.

$$\sigma = (1 - d) * E_0 * \varepsilon$$

A plastic degradation, in which the stiffness degradation is associated with the plastic deformation instead of the total deformation, was introduced to overcome the weaknesses of the elastic degradation model. It means that irreversible deformation/plastic strain exists after damage has occurred (so the plastic strain with stiffness degradation in this case can be rewritten as:

$$\sigma = (1 - d) * E_0 * \left(\varepsilon - \varepsilon^{pl}\right)$$

2.10.2 Mechanical behaviors

CDP model is a continuum, plasticity-based, damage model for concrete. It assumes that the main two failure mechanisms are tensile cracking and compressive crushing of the concrete material (ABAQUS, 2014). The evolution of the yield (or failure) surface is controlled by two hardening variables, and, linked to failure mechanisms under tension and compression loading, respectively.

a) Uniaxial tension and compression stress behavior

The model assumes that the uniaxial tensile and compressive response of concrete is characterized by damaged plasticity. Under uniaxial tension the stress-strain response follows a linear elastic relationship until the value of the failure stress is reached. The failure stress corresponds to the onset of micro-cracking in the concrete material. Beyond the failure stress the formation of micro-cracks is represented macroscopically with a softening stress-strain response, which induces strain localization in the concrete structure. Under uniaxial compression the response is linear until the value of initial yield. In the plastic regime the response is typically characterized by stress hardening followed by strain softening beyond the ultimate stress. This representation, although somewhat simplified, captures the main features of the response of concrete. It is assumed that the uniaxial stress-strain curves can be converted into stress versus plastic-strain curves. (This conversion is performed automatically by Abaqus from the user-provided stress versus "inelastic" strain data).

As shown in Figure-8 below, when the concrete specimen is unloaded from any point on the strain softening branch of the stress-strain curves, the unloading response is weakened: the elastic stiffness of the material appears to be damaged (or degraded). The degradation of the elastic stiffness is characterized by two damage variables, d_t and d_c, which are assumed to be functions of the plastic strains, temperature, and field variables

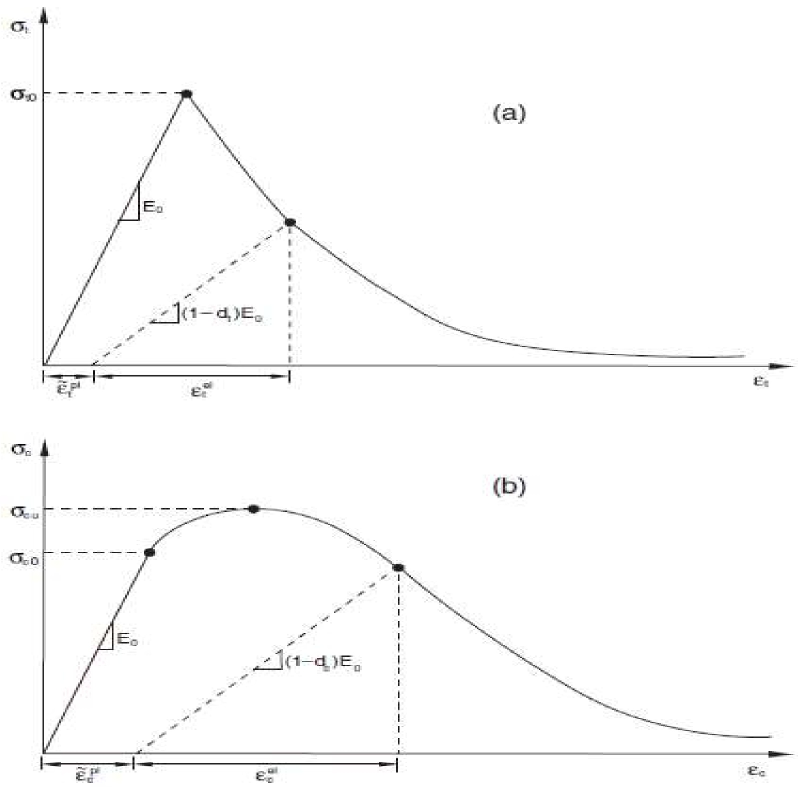


Figure 8: Response of concrete loading in tension (a) and compression (b) (ABAQUS, 2014)

b) Reinforcement Behaviours

In Abaqus, reinforcement in concrete structures is typically provided by means of rebars, which are one-dimensional rods that can be defined singly or embedded in oriented surfaces. Rebars are typically used with metal plasticity models to describe the behaviour of the rebar material and are superposed on a mesh of standard element types used to model the concrete. With this modelling approach, the concrete behaviour is considered independently of the rebar. Effects associated with the rebar/concrete interface, such as bond slip and dowel action, are modelled approximately by introducing some “tension stiffening” into the concrete modelling to simulate load transfer across cracks through the rebar. Defining the rebar can be tedious in complex problems, but it is important that this be done accurately since it may cause an analysis to fail due to lack of reinforcement in key regions of a model (ABAQUS, 2014)

c) Defining tension stiffening

The post-failure behaviour for direct straining is modelled with tension stiffening, which allows to define the strain-softening behaviour for cracked concrete. This behaviour also allows for the effects of the reinforcement interaction with concrete to be simulated in a simple manner. Tension stiffening is required in the concrete damaged plasticity model. We can specify tension stiffening by means of a post-failure stress-strain relation or by applying a fracture energy cracking criterion (ABAQUS, 2014). In reinforced concrete the specification of post-failure behaviour generally means giving the post-failure stress as a function of cracking strain. The cracking strain is defined as the total strain minus the elastic strain corresponding to the undamaged material. Tension stiffening data are given in terms of the cracking strain. When unloading data are available, the data are provided to Abaqus in terms of tensile damage curves. Abaqus automatically converts the cracking strain values to plastic strain values using the relationship

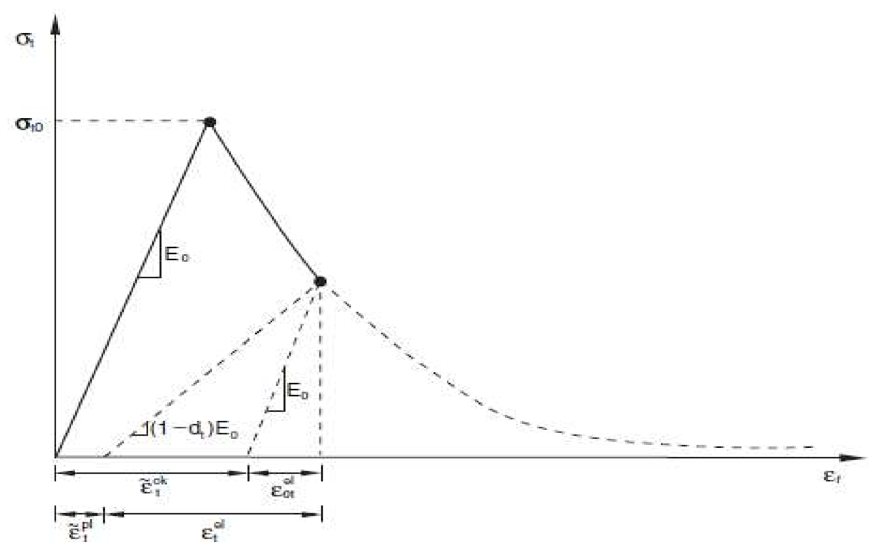


Figure 9: Definition of the cracking strain, ε_t^{-cr}, for the definition of tension stiffening data (ABAQUS, 2014)

CHAPTER THREE

METHODOLOGY

3.1 General

Cracks have significant influences on durability, aesthetics and leakage of structure. Among different cracks scenarios, crack width and crack spacing are mostly affecting the performance of the structure as they expose steel reinforcement to the environment which leads to corrosion. To counteract these effects and others, different regions produce different mechanism of crack width calculation and limitations. However, widely used crack width calculation models and allowable crack width limits have changed from time to time and differ from region to region. In this paper, the applicability of current crack width calculation formulas and the allowable crack width limits have been compared by considering different parameters with experimental result of crack width. The effects of parameters such as concrete cover, bar diameter, and type of loading have been considered.

As sample of study, the background of crack width calculation model of Ethiopian code, American code and British code have been examined. Experimentally determined crack spacing and crack width have been selected for both axial tension and bending specimen from literature to compare with the aforementioned code's expression. Further, the cracking behavior of reinforced concrete beam subjected to tensile loading was investigated through Abaqus analysis to see its cracking patterns.

3.2 Analytical Investigation of Crack Width in Different Standard Codes

There are different crack width prediction models in the codes of practice. These prediction models have been selected, as they can represent the different regions of the world. In this paper, Eurocodes, which was the governing code of practice in Europe and from which Ethiopian codes have been derived is selected for assessing crack width. And the crack width prediction models in American code, and British code have been also considered. When examining the crack width proposed by these codes, it can be observed that different governing parameters were mentioned in prediction models. Moreover, crack widths arising from flexure and direct tension in RC member have somewhat different expressions in aforementioned codes.

3.2.1 Ethiopian Standard Code

Ethiopian Standard Code analysis the crack width by the following Equation:

$$w_k = S_{r,max} * (\varepsilon_{sm} - \varepsilon_{cm}) \leq w_{k,max}$$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \left(f_{ct,eff} / \rho_{p,eff} \right)(1 + \alpha_e \rho_{p,eff})}{E_s} \geq 0.6 \left(\frac{\sigma_s}{E_s} \right)$$

In situations where, bonded reinforcement is fixed at reasonably close centres within the tension zone (spacing (s) $\leq 5(c + \frac{\emptyset}{2})$, the maximum final crack spacing may be calculated from Expression:

$$S_{r,max} = k_3 * c + k_1 * k_2 * k_4 * \left(\emptyset / \rho_{p,eff} \right)$$

Where the spacing of the bonded reinforcement exceeds $5(c + \frac{\emptyset}{2})$; or where there is no bonded reinforcement within the tension zone, an upper bound to the crack width may be found by assuming a maximum crack spacing:

$$S_{r,max} = 1.3 * (h - x)$$

k_1 Coefficient reflecting the bonded reinforcement properties

= 0.8 for bars which are deformed

= 1.6 for bars which are smooth/plain

k_2 Coefficient representing the effect on the cross-sectional strain distribution

= 0.5 for the pure bending member

= 1.0 for the axial tension section

k_3 = 3.4; k_4 = 0.425

α_e $= \frac{E_s}{E_{cm}}$

K_t Factor depend on the duration of the load

= 0.6 for loading on short term

= 0.4 for loading on long term

$\rho_{p,eff}$ $= \frac{A_{st}}{A_{c,eff}}$; where, $A_{c,eff}$ = lesser of $\left\{ 2.5(h - d); \frac{(h-x)}{3}; \frac{h}{2} \right\} * b$

Where, $A_{c,eff}$ is the effective tension area, the area of concrete surrounding the tension reinforcement of depth, $h_{c,eff}$; h_{eff} = Lesser of $\left[2.5 * (h - d); \left(\frac{h-x}{3} \right); \frac{h}{2} \right]$

3.2.2 American Concrete Institute

Crack control requirements in ACI were based on the z-factor method. The equation proposed by this code took the following form:

$$w_{max} = 0.011\beta * f_s * \sqrt[3]{d_c * A_{0*}} 10^{-3} mm \qquad \text{(Flexural Loading)}$$

$$w_{max} = 0.0145 * f_s * \sqrt[3]{d_c * A_{0*}} 10^{-3} mm \qquad \text{(Direct Tension Loading)}$$

Where:

β	1.20 in beams used to compare the crack widths in flexure and axial tension.
A_o	A_e/n_b, the area of concrete surrounding each reinforcing bar
A_e	$2*d_c*b$, the effective area of concrete in tension and can be defined as the area of concrete having the full width of the beam and having the same centroid of the main reinforcement;

3.2.3 British Standard Code

The code informs that crack width is not exceeding values specified for different environmental conditions, and concrete and reinforcement stresses are maintained below a safe limit. The maximum surface crack width suggested by code is 0.3 mm. For water tightness, values of 0.2/ 0.1 mm may be required. Analysis will be required at service loads to determine the moments and stresses. The formula is designed to give a width of crack which has an acceptably small chance of being exceeded. Thus, an occasional crack slightly larger than the predicted width should not be considered as cause for concern.

Design surface crack width:

$$w_{kd} = \frac{3a_{cr}*\varepsilon_m}{1+2*\left(\frac{a_{cr}-c_{min}}{h-x}\right)}; \qquad \text{(Flexural crack width)}$$

$$w_{xd} = 3 * \alpha_{cr} * \varepsilon_m \qquad \text{(Tension crack width)}$$

$$a_{cr} = \sqrt{\left(\frac{s}{2}\right)^2 + {d_c}^2} - \frac{d_b}{2}; \quad \text{and } \varepsilon_m = \varepsilon_1 - \varepsilon_2$$

$$\varepsilon_m = \varepsilon_1 - \frac{b*(h-x)*(a-x)}{3*E_s*A_s*(d-x)} \qquad \text{(Flexural mean strain)}$$

$$\varepsilon_m = \varepsilon_1 - \frac{2b*h}{3*E_s*A_s} \qquad \text{(Tension mean strain)}$$

d_c effective cover = $c_{min} + d_b/2$

a distance from the face of the compression to the level at which the width of the crack is determined;

= h, when measuring the width of the crack at a soffit;

ε_1 at the chosen point, strain based on a cracked sectional analysis:

= $\varepsilon_s (a - x)/(d - x)$; for flexure

3.3 Study Variables for Crack Width Analysis in different standard codes

There are different influential parameters that affect crack width during examining using different codes. For this thesis different parameters such as the effect of concrete cover, bar diameter, and type of loading have been examined during analysing crack width in different codes. The crack width obtained by using different codes have been compared with experimentally determined crack width from previous studies.

3.3.1 Experimental Description for Crack Width Analysis

3.3.1.1 Crack Width Subjected to Tension Loading

To see the parametric effects on crack width, three bars having different diameters [10,12,16] mm would have been considered. Further, concrete cover (15,30,40,50) mm was assumed as per conducted test. The conducted test program from (Gribniak *et al.*, 2020) was taken.

Table 2: Specimen description from test results (Gribniak *et al.*, 2020)- Tensile Loading Test

Specimen Type	Reinforced Concrete Tension Beam
Specimen Length	150*150*1200 (w*d*l) (in mm)
Load Level	100.4kN
Concrete Strength (f_{cm})	42.51MPa (Mean Compressive Strength)
Tensile Strength (f_{ctm})	3.18Mpa (Mean Tensile Strength)
Modulus of Elasti. (E_{cm})	34GPa (Mean Modulus of Concrete)
Cover to reinf. (mm)	(15, 30, 40, 50) mm
Area of concrete (A_c)	22,186mm2
Diameter of bar (mm)	4#10
Steel Stress (σ_s)	320MPa
Modulus of Elastici. (E_s)	199.5GPa (Modulus of Steel)
Reinf. Ratio (ρ)	0.014

Figure 10: Tension prismatic specimens: (a) Reinforcement skeletons before placement into the moulds; (b) Specimens brought out of the moulds (Gribniak *et al.*, 2020).

3.3.1.2 Crack Width Subjected to Flexural Loading

To see the parametric effects on crack width, three bars having different diameters [12,16,24] mm would have been considered. Further, concrete cover (15,30,40,50) mm was assumed as per conducted test. Test Specimen, Properties and its Results from (Gilbert and Nejadi, 2008):

Table 3: Specimen description from test (Gilbert and Nejadi, 2008)- Flexural Loading Test

Specimen Type	Reinforced Concrete Flexural Beam
Specimen Length	250*348*3500 (w*d*l) (in mm)
Load Level	109kN
Concrete Strength (f_{cm})	36.6MPa (Mean Compressive Strength)
Tensile Strength (f_{ctm})	3.06Mpa (Mean Tensile Strength)
Modulus of Elasti. (E_{cm})	33GPa (Mean Modulus of Concrete)
Cover to reinf. (mm)	40
Area of concrete (A_c)	86,598mm2
Steel Stress (σ_s)	226MPa
Modulus of Elastici. (E_s)	200GPa (Modulus of Steel)
Reinf. Ratio (ρ)	0.0174

The mean/maximum crack width that obtained from test result is 0.28/0.38mm. Similar crack width calculation procedures have been conducted as that of tension member.

3.4 Analytical Investigation of Crack Spacing in Different Standard Codes

Crack spacing is mostly used in crack width calculation models. Under this particular topic, Ethiopian standard code and Model Code were considered. The expression for its analysis has different forms in the aforementioned codes. The following codes discussed their differences.

a) Ethiopian Standard Code

$$S_{r,max} = k_3 * c + k_1 * k_2 * k_4 * \left(\emptyset / \rho_{p,eff}\right)$$

b) Model Code (MC-2010)

$$S_{r,max} = 2\left\{k * c + \frac{1}{4} * \frac{f_{ctm}}{\tau_{bms}}\left(\emptyset / \rho_{p,eff}\right)\right\}$$

3.4.1 Experimental Description for Crack Spacing

The experimental program consisted of four-point bending test and axial tensile test was selected for this particular paper. The axial tensile members were selected, as it represents the tensile region of a bending member. The relevant mechanical properties of concrete, compressive strength and tensile strength, 31.5MPa and 2.9MPa, respectively, have been obtained from test result conducted by (Naotunna, Samarakoon and Fosså, 2020). Yield strength and Young's modulus of the reinforcement, 500MPa and 200GPa, respectively, have been used by the authors' during the Experiment. Effect of parameters such as concrete covers (15, 35, 40, 50) mm and bars diameter (16, 24, 28, 32) mm on crack spacing were considered under this subtopic.

3.4.1.1 Crack Spacing of RC Element Subjected to Tension Loading

For analysis of such member, 2-m-length specimens which was conducted by (Naotunna, Samarakoon and Fosså, 2020) was selected. The specimen was designed to generate a large number of cracks and therefore to obtain a large number of crack spacing data. As per author intentions, the axial tensile specimen is assumed to represent only the tensile region of a bending member. The specimen with 0.2 m × 0.2 m × 2 m (width × height × length) was casted with four 32-mm-diameter reinforcement bars by the authors. The concrete cover thickness of the specimen was selected as 35 mm. The details of the axial tensile RC specimen is given in figure (11), with all dimension in mm. The test system was assured to resist a load of 900kN, where the stress in reinforcement can reach up to 280 N/mm^2. Experimentally Determined Maximum/Mean Crack Spacing was 203/135mm.

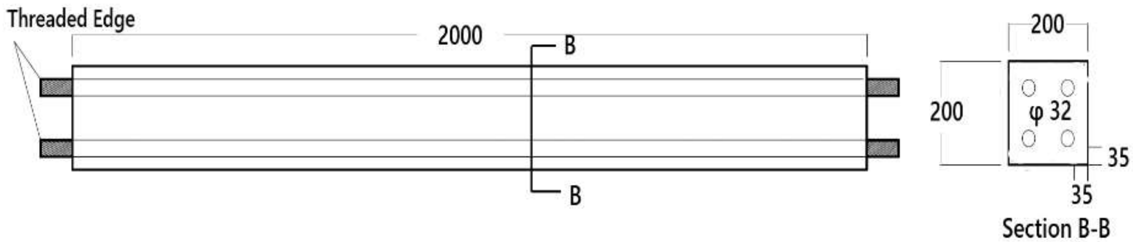


Figure 11: Details tensile specimen for crack spacing (Naotunna, Samarakoon and Fosså, 2020)

❖ Test Result from (Naotunna, Samarakoon and Fosså, 2020) for Crack Spacing Calculation:

Element Cross Section	0.2*0.2*2 (w*h*l) (m)		
Bar diameter (mm)	32ϕ4	Strength of concrete (f_{ctm})	3.2MPa
Concrete cover (mm)	35mm	Yield strength of reinf. (f_y)	500MPa
Maximum Load (kN)	900kN	Modu. of elas. of steel (E_s)	200GPa
Reinforcement stress (σ_s)	280MPa	Modu. of Concrete (E_{cm})	32GPa
Strength of concrete (f_{cm})	31.5MPa		

3.4.1.2 Crack Spacing of RC Element Subjected to Bending

A beam which was casted with 0.25 m × 0.3 m × 2.2 m (width × height × length) dimensions was selected from (Naotunna, Samarakoon and Fosså, 2020) too. The bending specimen was designed with two 32-mm-diameter tensile reinforcements and a cover of 35 mm, as shown in Figure below. The test had the maximum constant moment span of 800 mm. Furthermore, the beam was designed to avoid shear failure by using shear links, apart from in the constant bending zone. No stirrups were used at the constant bending zone. Specimen was loaded, exceeding the stabilized cracking stage, up to 290kN of final load. Experimentally Determined Maximum/Mean Crack Spacing was 160/112mm.

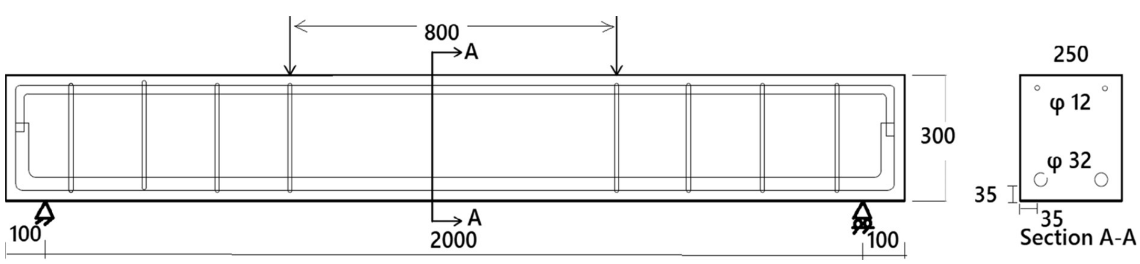


Figure 12: Detailing of bending specimen for crack spacing (Naotunna, Samarakoon and Fosså, 2020)

Test Result from (Naotunna, Samarakoon and Fosså, 2020) for Crack Spacing Calculation:

Element Cross Section Width*Height*Length (m) = 0.25*0.3*2.2

Bar diameter (mm)	32ϕ2	Strength of concrete (f_{ctm})	2.9MPa
Concrete cover (mm)	35mm	Yield strength of reinf. (f_y)	500MPa
Maximum Load (kN)	290kN	Modulus of steel (E_s)	200GPa
Reinforcement stress (σ_s)	180MPa	Modu. of Concrete (E_{cm})	31GPa
Strength of concrete (f_{cm})	31.5MPa		

3.5 Finite Element Modelling

The modelling of crack initiation and propagation is one of the most important aspects in the failure analysis of concrete structures. The Concrete Damage Plasticity (CDP) is the most comprehensive continuum model that was used in the reinforced concrete beam simulation to define concrete behaviour in this analysis. Concrete damaged plasticity model approach has been chosen to represent the behaviours of concrete including damages properties. Contribution of concrete in tension after cracking of reinforced concrete beam have been carry out using this model.

3.5.1 Materials Descriptions

In Abaqus the parameters required defining the yield surface consists of four constitutive parameters. The Poisson's ratio which controls the changes of volume of concrete for stresses below the onset of inelastic behaviour. Once the critical stress value is reached concrete exhibits an increase in plastic volume under pressure, which is defining by a parameter called the angle of dilation. The ratio of initial biaxial compressive yield stress to initial uniaxial compressive yield stress, 'fb0/fc0', with a default value of 1.16. 'Kc' is the ratio of the second stress invariant on the tensile meridian to the compressive meridian at initial yield with a default value of 2/3. To describe plastic properties of concrete, Abaqus default values of such parameters are accepted in this study.

3.5.2 Stress-Strain Curve of Concrete and Steel

The stress-strain data for concrete in compression and tension is important for analysis concrete damaged plasticity model. However, this was not reported as part of the experimental results, only the ultimate compressive strength of the concrete at tested days was recorded in experimental test. In this study, the complete $\sigma_c - \varepsilon_c$ curve proposed by Euro Code 2/Model

Code-2010, was adopted for concrete under compression, which suggests the following expression:

$$\sigma_c = \left[\frac{k\eta - \eta^2}{1 + (k-2) * \eta}\right] * f_{cm};$$

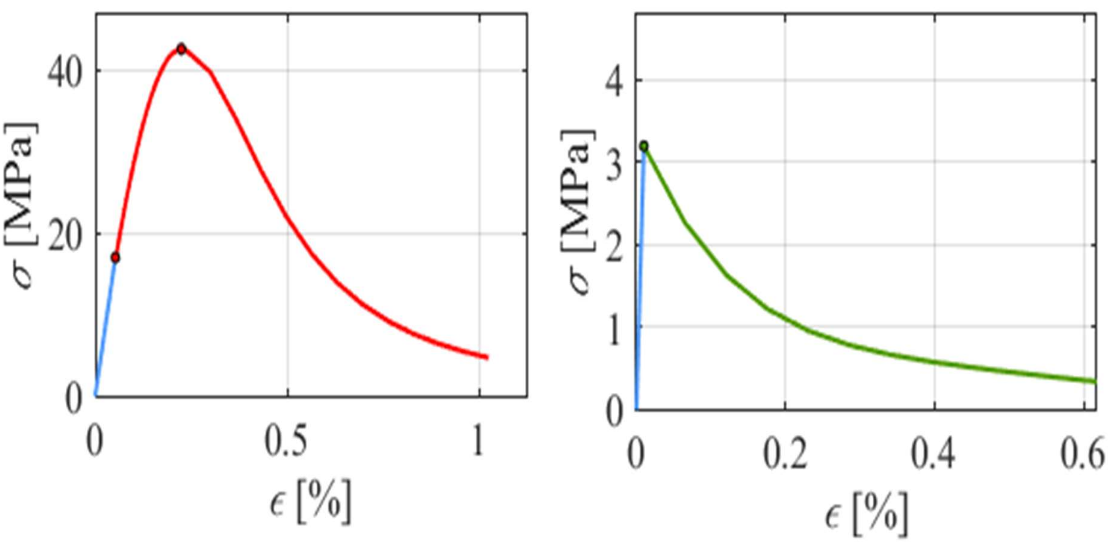
$$\text{where, } \eta = \frac{\varepsilon_c}{\varepsilon_{c1}}; k = 1.05 * E_{cm} * \frac{\varepsilon_{c1}}{f_{cm}};$$

$$\varepsilon_{c1}(\%) = 0.7 * (f_{cm})^{0.31} \leq 2.8;$$

$$E_{cm} = 22(0.1 * f_{cm})^{0.3}$$

The steel for the finite element models has been assumed to be an elastic-plastic material and identical in tension and compression. It is based on a linear elastic response up to yielding and a constant stress from the point of yielding to the ultimate strain. For this particular study, Elastic modules is taken as 199500MPa and poison ratio for steel reinforcement is 0.3.

However, to easily produce concrete stress-strain curve for both compression and tension behaviour, "ABAQUS CDP Generator" software was used in this paper. Such curves can be shown below. Fig.13 & 14 indicates, plastic stages for compression and tension, respectively.



a) b)

Figure 13: Stress- Strain Curve; a) under compression, b) under tensio

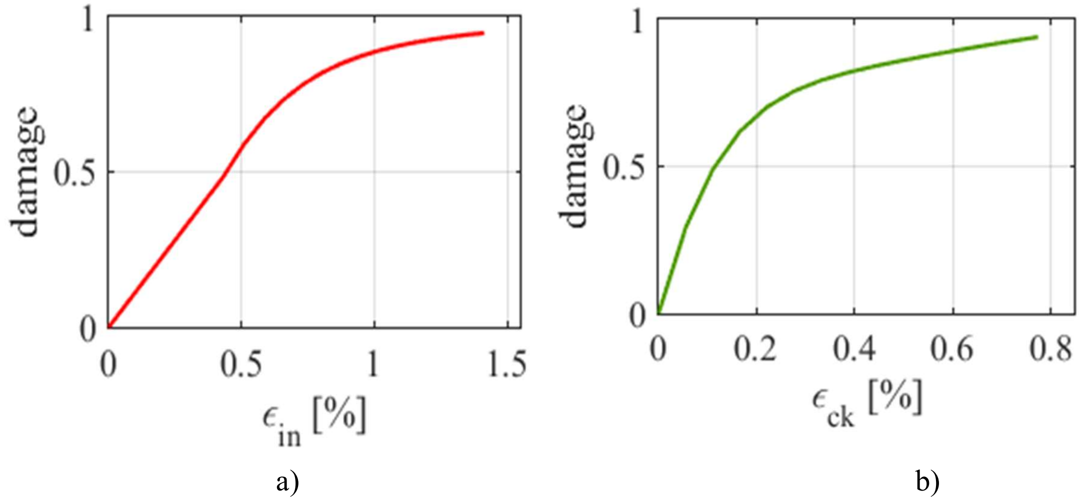


Figure 14: Damage behaviours a) under compression b) under tension

Data supporting Figure-13 and 14, can be seen from Appendix-C .

To see the cracking behaviours of reinforced concrete, abeam of length 1200mm, with width and depth of 150mm by 150mm, respectively, subjected to tensile loading was taken from (Gribniak et al., 2020) which was discussed in Table-2. So that, cracking behaviours that analysis through Abaqus is compared with Experimental cracking behaviours.

To get accuracy in results, all the elements of the FE model were assigned the same mesh size so that every two different materials can share the same node among them. After assembling and assigning the properties, an input file is created which is then imported to create a mesh. The elements used for the study are C3D8R (8-node linear brick) and T3D2 (2- node linear 3-D truss. The concrete beam is modelled in 3-D assigned with C3C8R elements and reinforcement in the longitudinal is modelled in 2-D was assigned with the T3D2 element. Meshing was adopted for all of the elements used in the models.

CHAPTER FOUR
RESULTS AND DISCUSSIONS

4.1 Crack Width

In this chapter, the effects of different parameters on crack width and crack spacing would have been discussed. Parameters such as concrete cover, bar diameter and reinforcement ratio were considered. While analyzing crack width and spacing using different codes, type of loading (tension and flexure) was considered as a major factor for comparing the codes. For each loading condition, conducted test results from literatures was selected and compared with analytical results of different codes. For comparison purposes; ES, ACI and BS Codes were considered for crack width analysis. while, ES and MC-2010 were considered for crack spacing analysis for each loading condition.

4.1.1 Crack Width Subjected to Tension Loading

Conducted test results subjected to tension loading was considered here. Axial tensile tests with multiple bars have been considered, as it represents the tensile region of actual cracking behavior in practical RC members. Thus, the conducted test of (Gribniak *et al.*, 2020) was selected from literature. The measured maximum crack widths and the estimated crack width according to Ethiopian Standard (EN ES), ACI and BS code have been discussed in (Table-4). Thus, parameters such as concrete covers (15,30,40,50) mm and bar diameter (10,12,16) mm was considered for analysis.

a) Effect of Concrete Cover on Crack Width

The effect of concrete cover on crack width in different codes of standard have been shown in chart (1). For comparison purpose, Ethiopian Standard (ES) code, ACI code and BS Code with one of the latest conducted test results have been considered. From this chart (1), it can be seen that concrete cover affects crack width in different codes differently.

Consequently, as concrete cover increase, the trend of increases in crack width were observed in all codes and conducted test. Further, Ethiopian Code estimates higher crack width, while BS Code estimates lower crack width in comparison with test results. However, in comparisons with other codes, ACI Code estimates crack width nearly in similar trend with that of experimental results. Besides, crack width estimated by BS code was related to mean crack width of experimental results (Table-4), and Percent of variation in each code with experimental results of crack width have been discussed in (Table-5).

Table- 4: Concrete cover effects on crack width in different standard codes

Concrete cover (mm)	Experimental Result (mm)	EN ES-2 Result (mm)	ACI Result (mm)	BS8110 Result (mm)
15	$w_{max/mean}$ = 0.13/0.09	0.27	W_{cr} = 0.14	0.08
30	$w_{max/mean}$ = 0.21/0.15	0.38	W_{cr} = 0.21	0.14
40	$w_{max/mean}$ = 0.23/0.15	0.42	W_{cr} = 0.25	0.18
50	$w_{max/mean}$ = 0.36/0.24	0.46	W_{cr} = 0.28	0.22

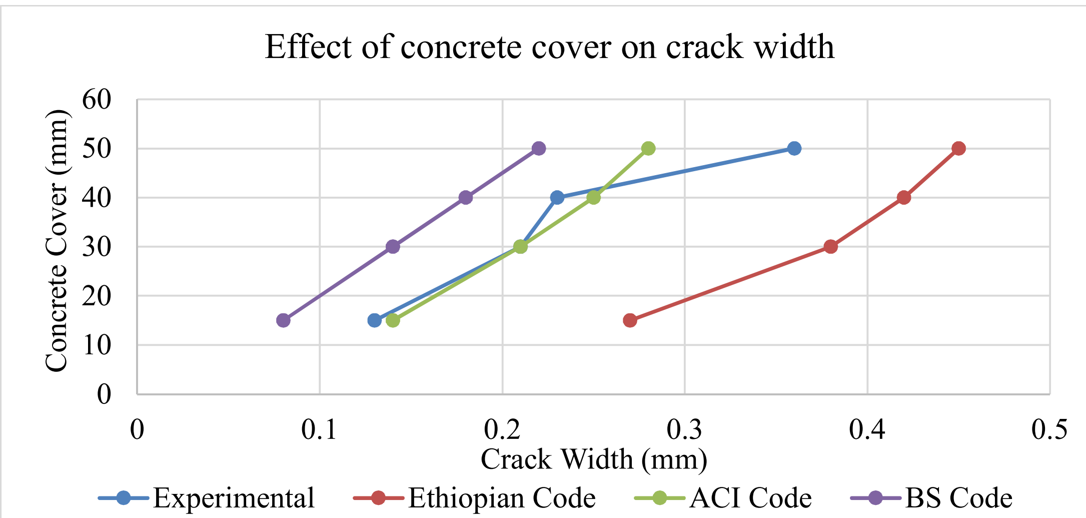


Chart 1: Effect of concrete cover on crack width in different codes with respect to test results

Table 5: Percent of variations in each code with respect to experimental crack widths

Concrete cover (mm)	Exp. crack width (mm)	Predicted crack width per standard codes (mm)			Percent of variation in each codes (%) **		
		ES	**ACI**	**BS**	ES	ACI	BS
15	**0.13**	0.27	0.14	0.08	***-108***	***-8***	***38***
30	**0.21**	0.36	0.21	0.14	***-81***	***0***	***33***
40	**0.23**	0.42	0.25	0.18	***-83***	***-9***	***22***
50	**0.36**	0.45	0.28	0.22	***-25***	***22***	***39***

$$** \text{ Percent of variations} = \left(\frac{\text{Exp.crack width} - \text{Predict crack width}}{\text{Exp.crack width}}\right) * 100$$

From Table-5, the predicted crack width according to ES Code, for the same bar diameter as that of experiment (10mm), large percent of variation was observed between the code and experimental crack widths. However, according to ACI Code, small percent of variations was observed between the code and experimental crack widths and similar trend was also observed. Furthermore, BS Code estimate less percent of variation of crack width than ES Code to the experimental crack width for considered bar diameter (10mm). Thus, ACI code fitted better than other two code to the experimental crack width for considered bar diameter (10mm).

b) Effect of bar diameter on crack width

Similar to concrete cover, bar diameter has an influence on crack width. Table-6 discussed the effect of bars diameters in different standard code

Table 6: Effect of bar diameter and concrete cover on crack width per different codes

Cover(mm)	15			30			40			50		
Bar diam. (mm)	Predicted crack width per each code (mm)											
	ES	**ACI**	**BS**	**ES**	**ACI**	**BS**	**ES**	**ACI**	**BS**	**ES**	**ACI**	**BS**
10	0.27	0.14	0.08	0.38	0.21	0.14	0.42	0.25	0.18	0.45	0.28	0.22
12	0.16	0.1 0	0.06	0.23	0.15	0.10	0.26	0.17	0.13	0.28	0.20	0.16
16	0.07	0.06	0.04	0.10	0.09	0.06	0.12	0.10	0.08	0.13	0.12	0.09

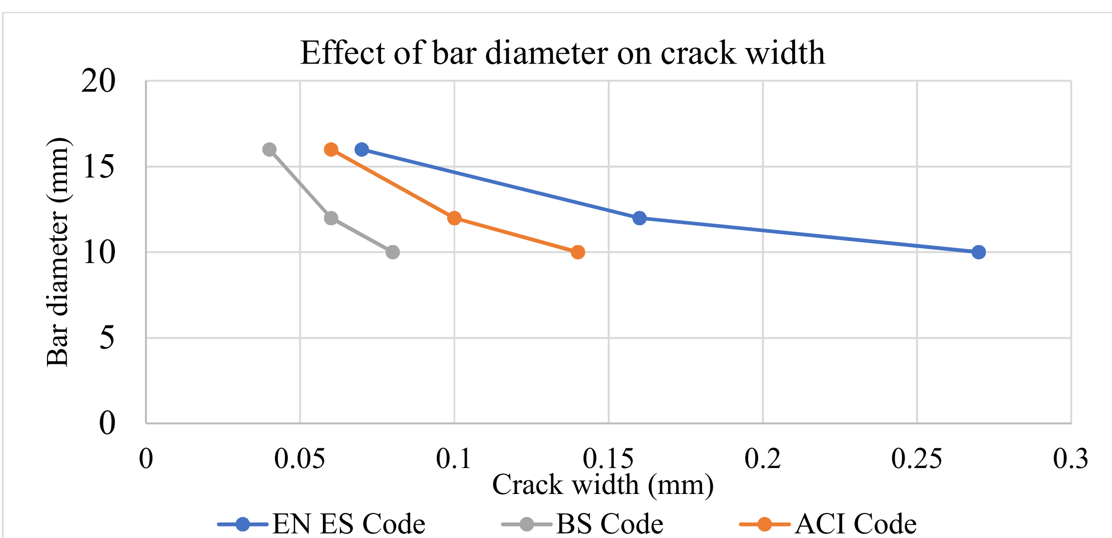


Chart 2: Effect of bar diameter in d/t standard codes for concrete cover-15mm

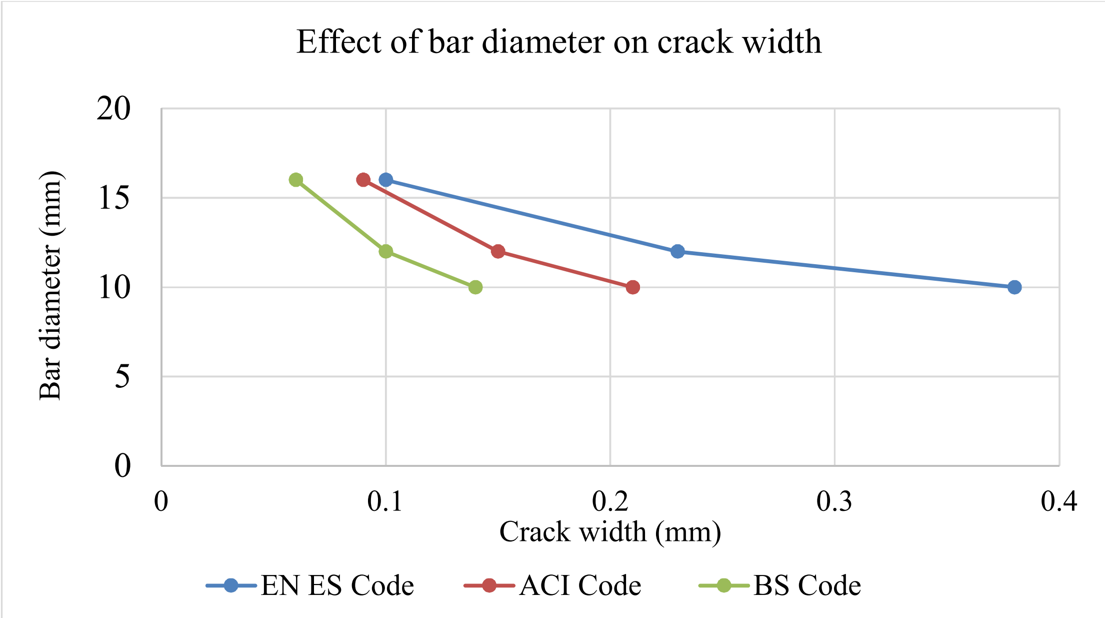


Chart 3: Effect of bar diameter in d/t standard codes for concrete cover-30mm

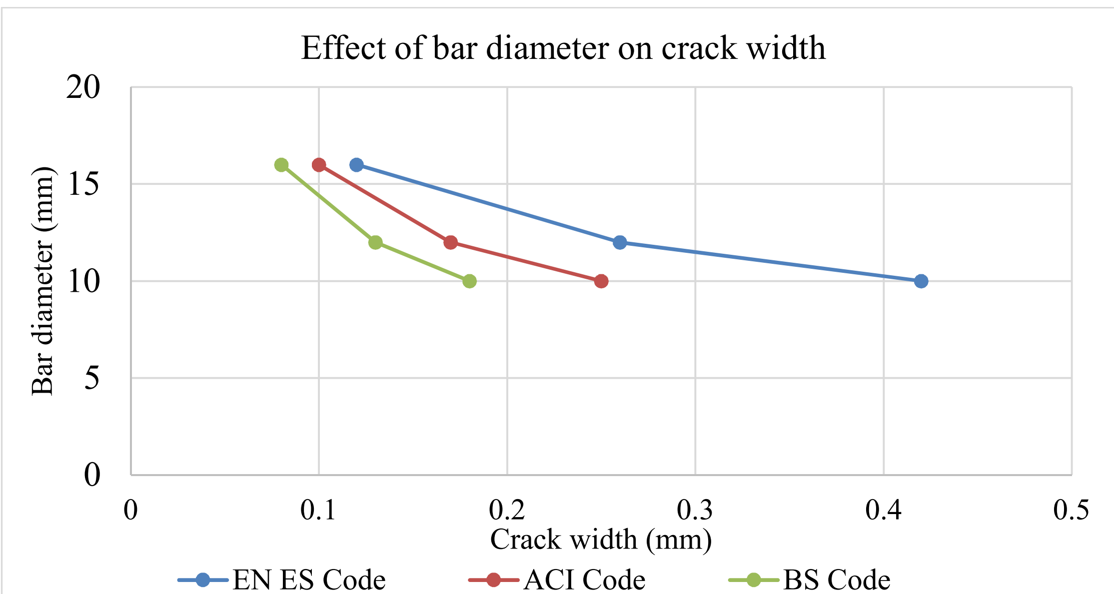


Chart 4: Effect of bar diameter in d/t standard codes for concrete cover-40mm

From charts (2, 3, 4), it was observed that, as bar diameter increases, crack width is decreasing and vise versa. From these charts, we have deduced that large value of crack width has been computed from EN ES Code.

c) Effect of reinforcement ratio on crack width

Reinforcement ratio is a parameter that affect crack width while cracking limit state is considered. As concrete cover increases, reinforcement ratio is decreased which in turn increases crack with. Thus, charts (5 & 6) discussed such behaviours.

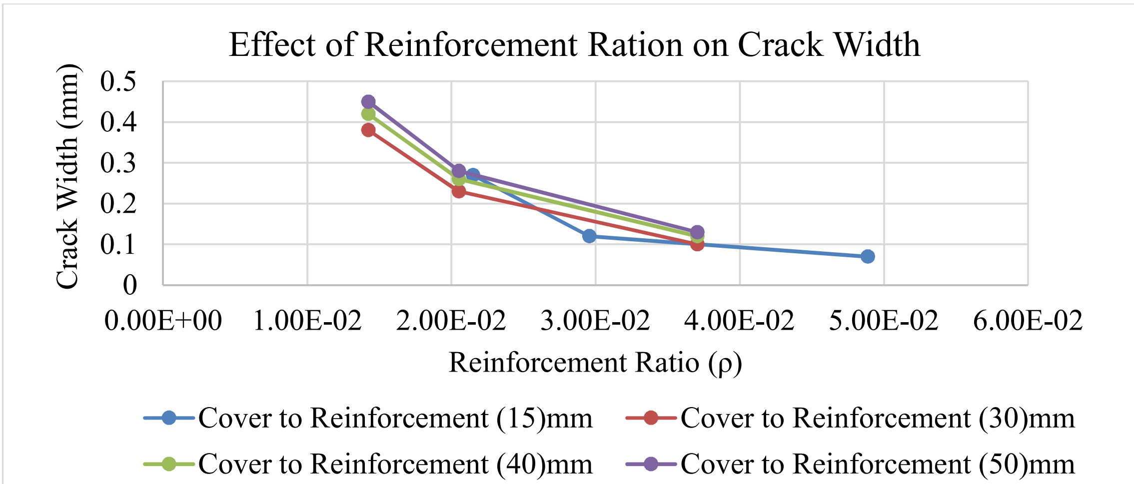


Chart 5: Effect of Reinforcement Ratio on Crack Width per EN ES Code

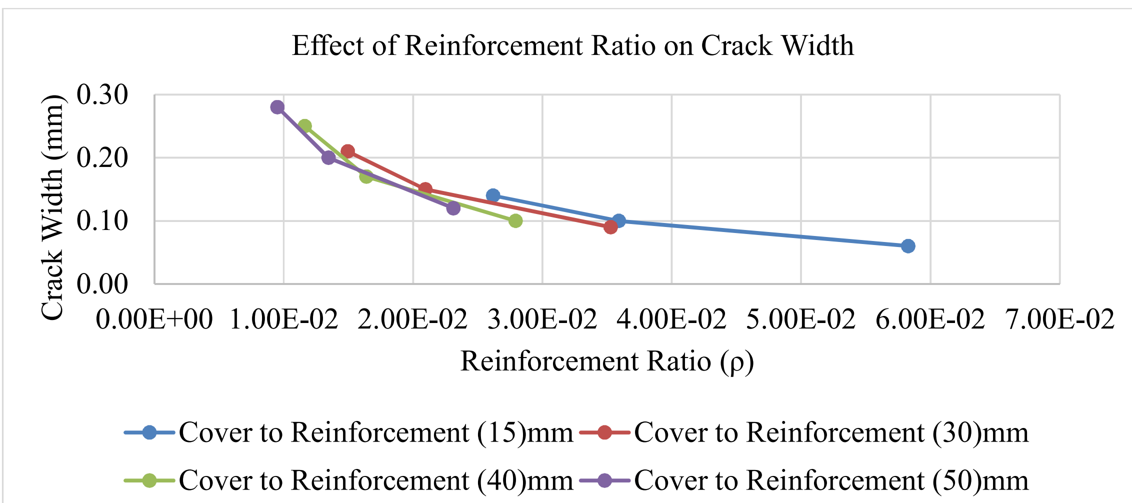


Chart 6: Effect of Reinforcement Ratio on Crack Width per ACI Code

Generally, while considering the estimation of crack width in different codes, it could be observed that the empirically based ACI and BS codes give relatively good fit for the experimental crack widths than ES code. However, when considering the cases of all codes, and based on the percentages of variations (Table-5), it seems like that the existing codes need to improve the expressions for crack width calculation.

4.1.2 Crack Width Subjected to Flexural Load

When a member is subjected to bending actions, the effect of curvature on crack width is considered. Thus, ES Code consider this effect when crack spacings is analyses. It consists the 'k_2' parameter, which cause to predict lower crack spacings for the specimens subjected to bending. The empirically based codes, ACI and BS, considered the data from bending tests to develop their models, where they use the 'β' parameter and $(d-x)/(d_1-x)$ factor, respectively, to include the effect of curvature.

For this particular topic, conducted test results subjected to bending action was considered. Thus, the conducted test of (Gilbert and Nejadi, 2008) was selected from literature. Thus, parameters such as concrete covers (15,30,40,50) and bar diameter (12,16,24) was considered for analysis.

a) Effect of Concrete Cover

The effect of concrete cover on crack width when the reinforced concrete elements were subjected to flexural load have been discussed in Table-8. Detail discussion have been also given in subsequent charts. Table-7, discuss the analytical crack width of different standard codes with experimental crack width in their respective percent of variations.

Table 7: Percent of variation in each code with respect to experimental crack widths

Cover (mm)	Bar dia. (mm)	Pred. crack width per codes (mm)			Exp. mean/max. crack width (mm)	Percent of variation in each codes wrt Experiment (%) **		
		ES	**ACI**	**BS**		ES	ACI	BS
40	16	0.22	0.31	0.21	**0.28/0.38**	***21.4/42.1***	***-11/18.4***	***25/44.7***

$$** \text{ Percent of variation} = \left(\frac{\text{Exp. crack width} - \text{Predicted crack width}}{\text{Exp. crack width}}\right) * 100$$

From Table-7, the estimated crack width using ES code with bar diameter (16mm) and concrete cover (40mm) is 0.22mm. However, the experimental max/mean crack width was determined as 0.38/0.28mm considering long term effect. The predicted crack width, 0.22mm, have a percent of variations, 42.1% and 21.4%, with experimental maximum and mean crack width, respectively. While, the estimated crack width using ACI Code with bar diameter (16mm) and concrete cover (40mm) is 0.31mm. The predicted crack width has a variation of, 18.4% and -11%, with experimental maximum and mean crack width, respectively. Further, the estimated crack width using BS Code with bar diameter (16mm) and concrete cover (40mm) is 0.21mm. The predicted crack width has a variation of, 44.7% and 25%, with experimental maximum and mean crack width, respectively.

Table 8: Effect of bar diameter and concrete cover on crack width per different codes

Cover(mm)	**15**			**30**			**40**			**50**		
Bar diam. (mm)	Predicted crack width per each code (mm)											
	ES	**ACI**	**BS**	**ES**	**ACI**	**BS**	**ES**	**ACI**	**BS**	**ES**	**ACI**	**BS**
12	0.32	0.31	0.30	0.38	0.43	0.33	0.43	0.45	0.35	0.46	0.58	0.38
16	0.16	0.19	0.17	0.19	0.26	0.19	0.22	0.31	0.21	0.25	0.35	0.22
24	0.04	0.09	0.08	0.06	0.12	0.09	0.07	0.14	0.10	0.13	0.17	0.11

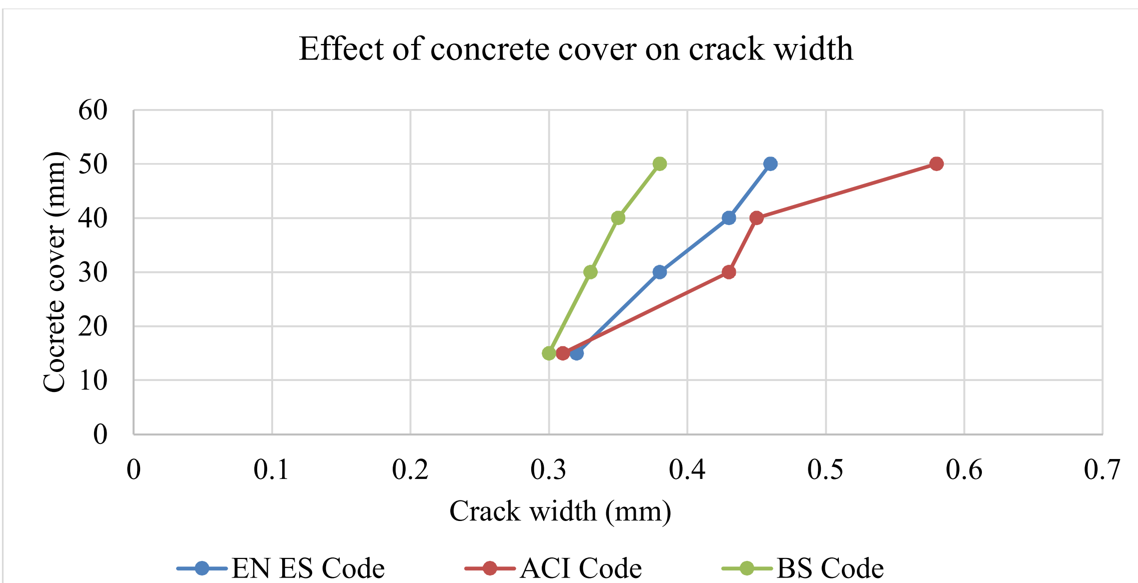


Chart 7: Effect of concrete cover on crack width in d/t standard codes for bar dia. of 12mm

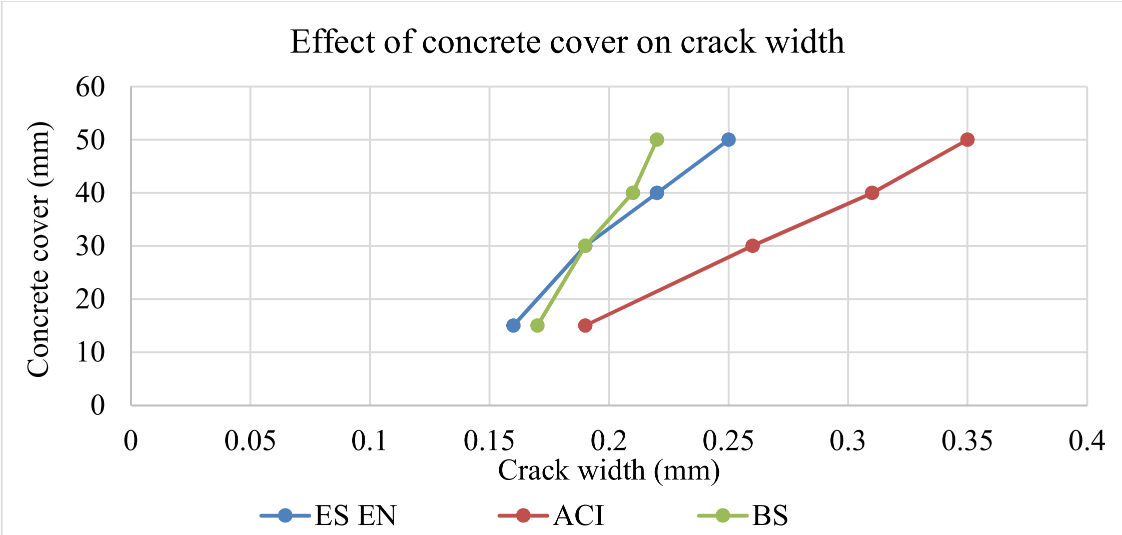


Chart 8: Effect of concrete cover on crack width in d/t standard codes for bar dia. of 16mm

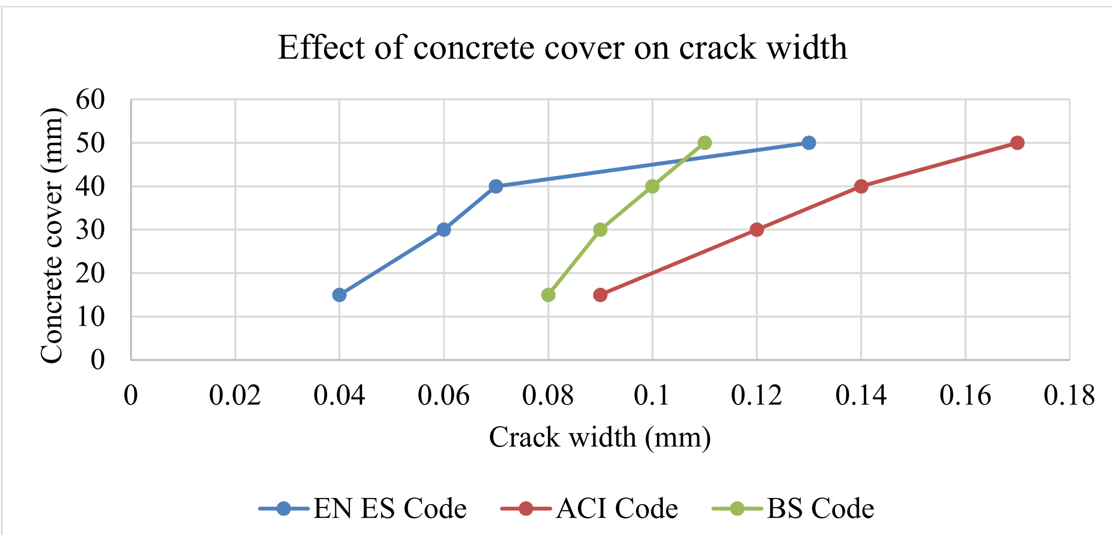


Chart 9: Effect of concrete cover on crack width in d/t standard codes for bar dia. of 24mm

The effects of concrete cover on crack width in flexural loading beam would have been discussed in charts (7-9). From the charts, it can be seen that concrete cover affects crack width in different codes differently. The standard codes considered for discussion includes; Ethiopian, American and British Standard code. In each code, crack width is increasing as concrete cover increases. From discussions of above charts (7,8,9) and Table-7, ACI code seems like better than other two codes.

b) Effect of bar diameter

Bars diameter are parameters that mostly carries tensile load in reinforced concrete structures. Thus, the effect of these bars on crack width was discussed from charts (10, 11) in different codes.

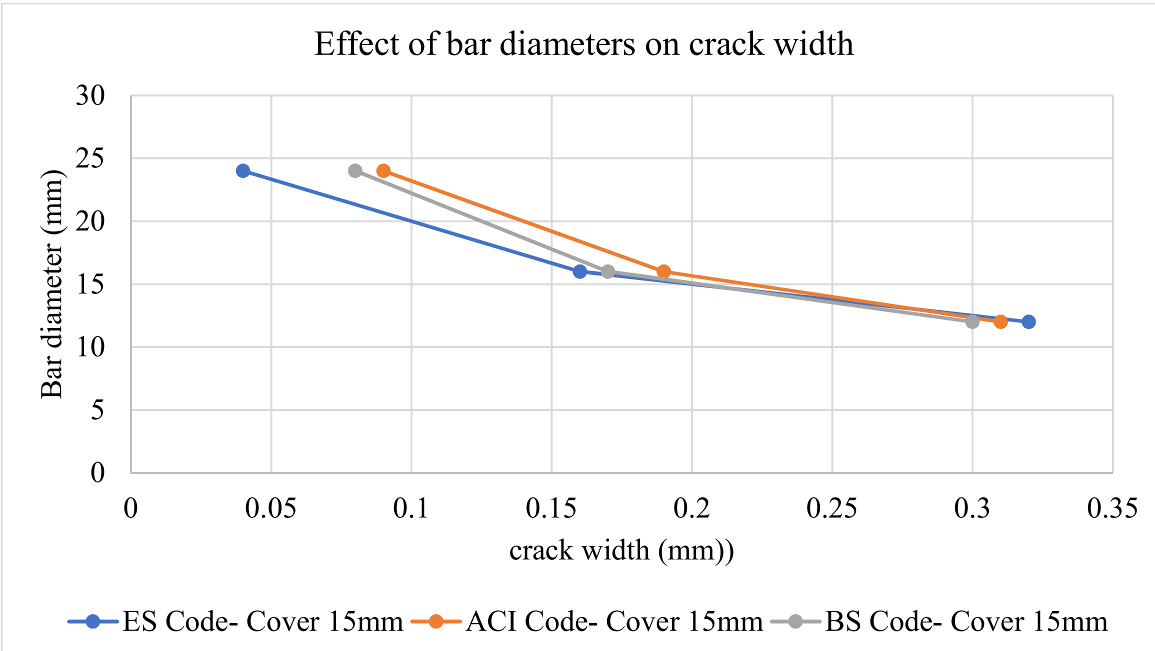


Chart 10: Effect of bar diameter on crack width in different standard codes

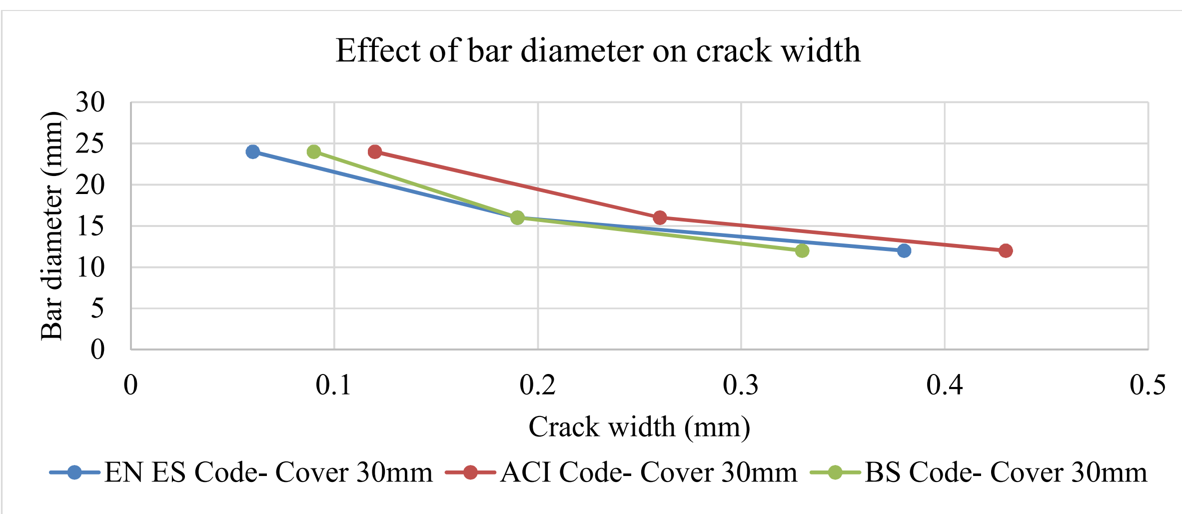


Chart 11: Effect of bar diameter on crack width in different standard codes

From charts (10 and 11), crack width is increases as bar diameter decreased and vise versa. Moreover, maximum crack width is also estimated by ACI Code than ES and BS Code.

4.2 Crack Spacing

4.2.1 Crack Spacing Subjected to Tensile Load

The experimental results consisted axial tensile test was selected from (Naotunna, Samarakoon and Fosså, 2020) for analyzing crack spacing calculation using Ethiopian Code and Model Code-2010. In both Codes, the parameters that mostly affects crack spacing are: Concrete cover, Bar diameter and Reinforcement ratio. Thus, the effect of these parameters, concrete cover (15,35,40,50) mm and bar diameter (16,24,28,32) mm with its content of reinforcement ratio, on crack spacing were discussed from following charts. According to the test result (for 32mm diameter of bar and 35mm concrete cover), the maximum crack spacing was 203mm.

- Assuming Constant Concrete Cover (35mm) and varying bar size/reinforcement ratio

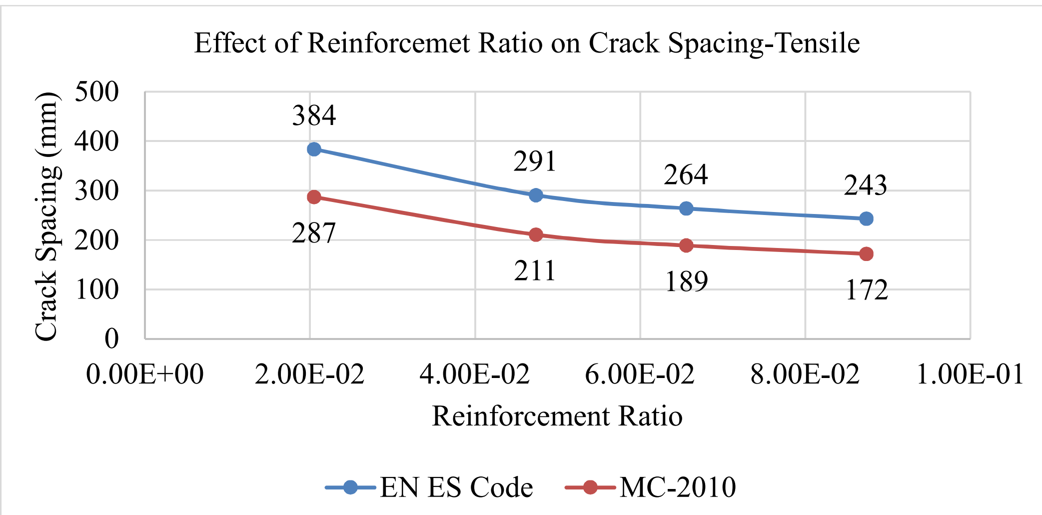


Chart 12: Effect of Reinforcement Ratio on Crack Spacing per EN ES Code

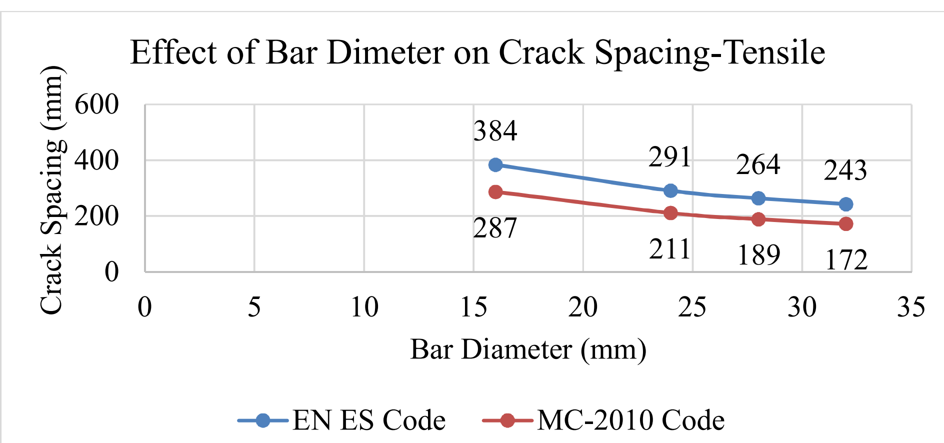


Chart 13: Effect of Bar Diameter on Crack Spacing per EN ES Code

- Assuming Constant Bar Diameter (32mm) and Varying Concrete Cover

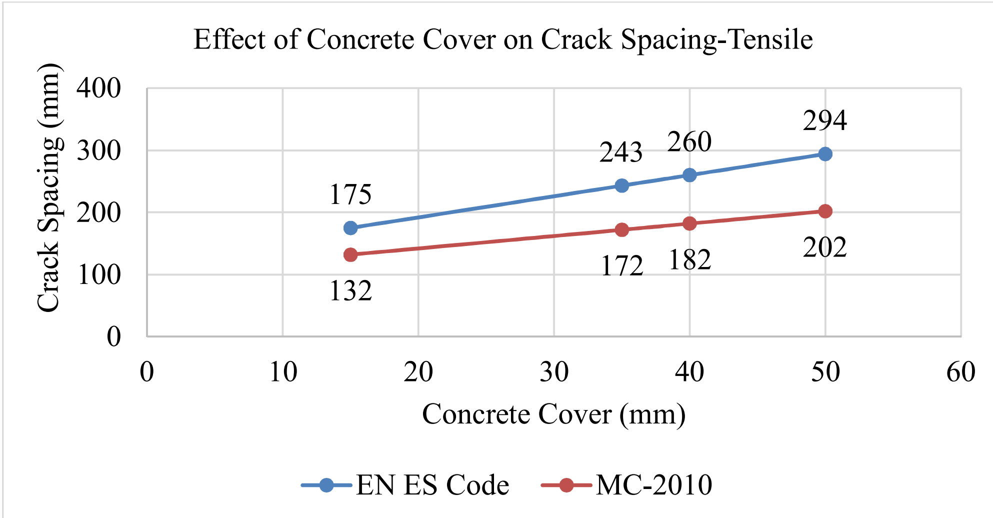


Chart 14: Effect of concrete cover on crack Spacing-Tension

For considered bar diameter (32mm) and concrete cover (35mm), the maximum crack spacing that obtained through test was 203mm. However, the maximum crack spacing that obtained through analytic using Ethiopian code (ES) and Model Code 2010 was 243mm and172mm, with variations of, -19.7% and 15.3%, respectively. Moreover, from charts (12 and 13), as bar diameter and reinforcement ratio decrease, the crack spacing is increases, and from chart (14), as concrete cover increases, the crack spacing is increases. Besides, from charts (12,13,14), Ethiopian Code (ES) estimates higher crack spacing values than MC-2010 Code.

4.2.2 Crack Spacing Subjected to Flexural Load

A beam which was casted with 0.25 m × 0.3 m × 2.2 m (width × height × length) dimensions was selected from (Naotunna, Samarakoon and Fosså, 2020) too. The bending specimen was designed with two 32-mm-diameter tensile reinforcements and a cover of 35 mm. The test had the maximum constant moment span of 800 mm. Specimen was loaded, exceeding the stabilized cracking stage, up to 290kN of final load. For conducted test, maximum/mean Crack Spacing was 160/112mm.

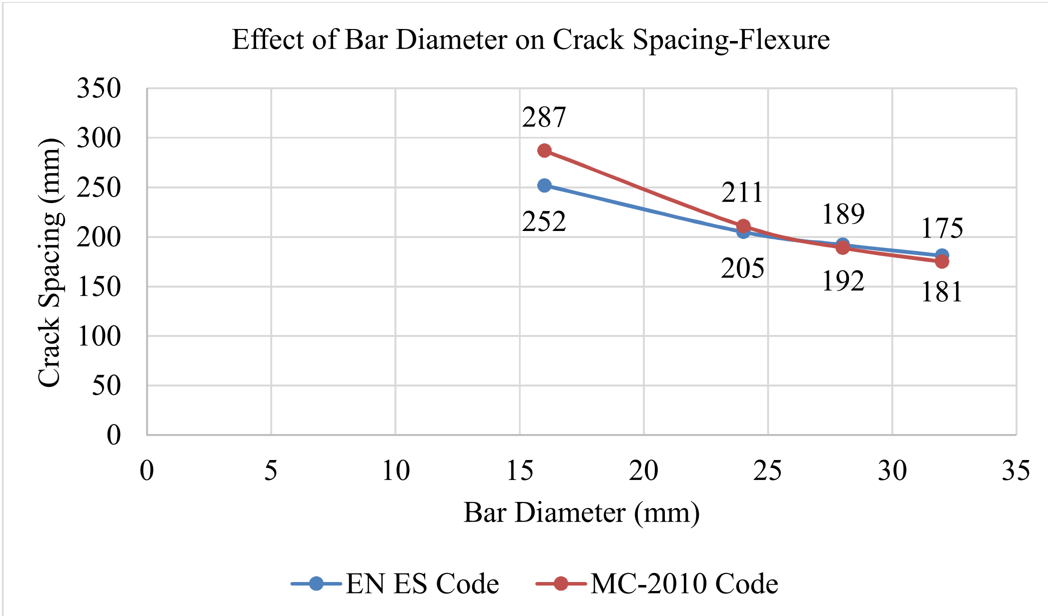


Chart 15: Effect of bar diameter on crack spacing-Flexure

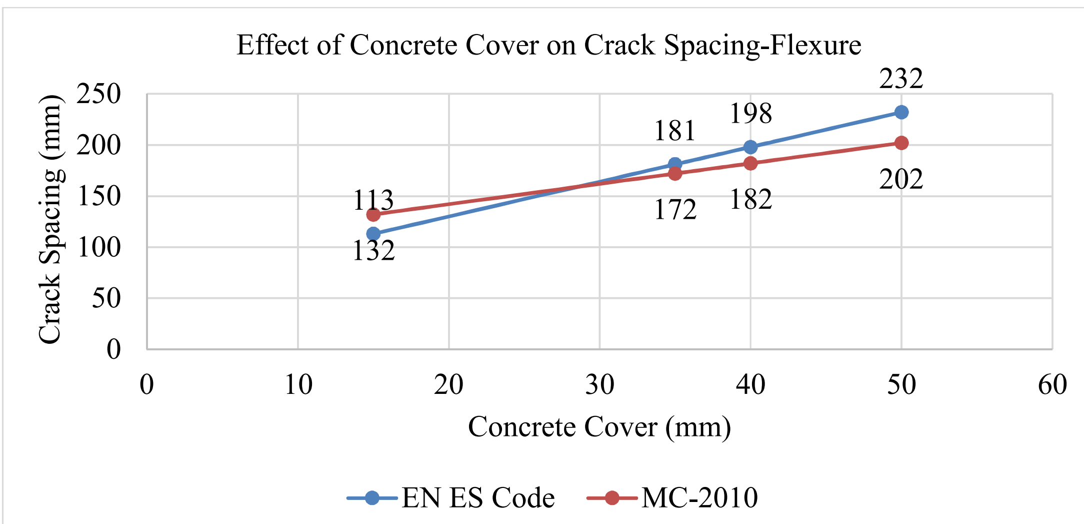


Chart 16: Effect of concrete cover on crack spacing-Flexure

For bar diameter (32mm) and concrete cover (35mm), the maximum crack spacing that obtained through test was 160mm. However, the maximum crack spacing that obtained through analytic using Ethiopian Code (ES) and Model Code 2010 was 181mm and 172mm, with variations of, -13.1% and -7.5%, respectively. Moreover, from chart (15), as bar diameter decrease, the crack spacing is increases, and from chart (16) as concrete cover increases, the crack spacing is increases.

4.3 Finite Element Results

Crack pattern observed from FE result is in good fit with the experimental result. In relation to experimental result, the parameters that captioned from FE result is discussed below.

a) Displacement

At ultimate load, the maximum displacement obtained from experimental result is 1.4mm. While, the magnitude of displacement obtained from FE result is around 1.2mm.

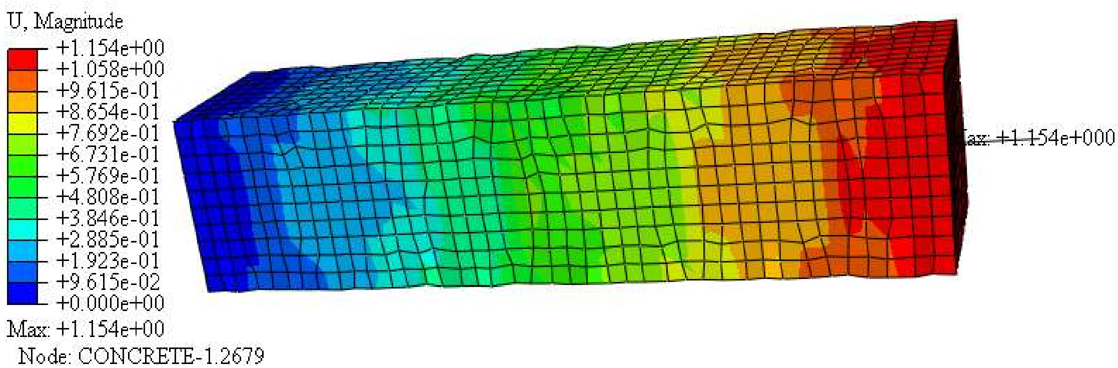


Figure 15: Maximum displacement obtained from FE analysis

b) Tensile Strain in Concrete (PEEQT)

PEEQT refers to tensile equivalent plastic strain also known as cracking strain tells about the intensity of crack opening concerning another crack at a particular loading level.

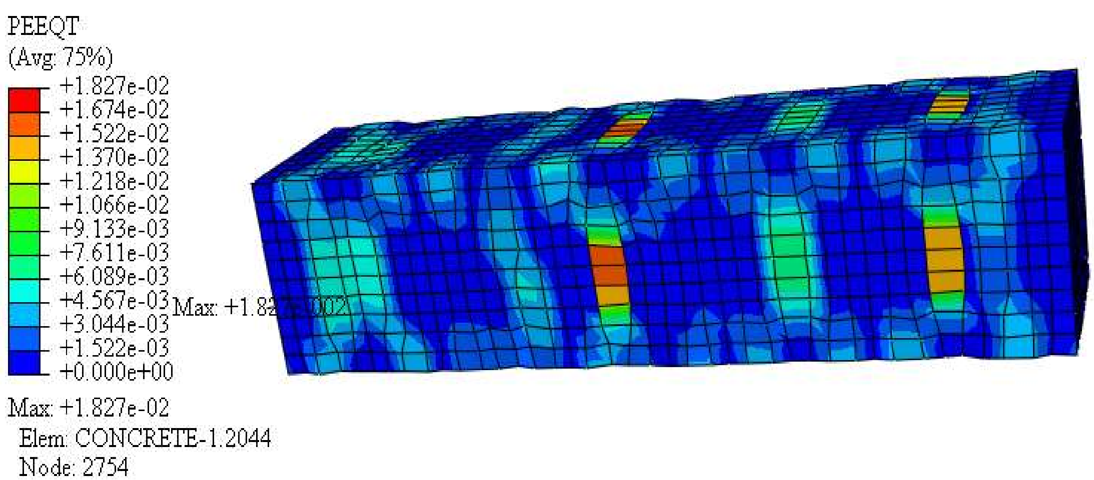


Figure 16: Tensile strain pattern distribution in concrete at loading level

c) Cracking Pattern

It is observed that, from figure-17 and 18, cracks were initiated and propagated in the vertical direction on the surface of tensile loading beam. When Figure-17 of experimental beam and Figure-18 of simulated model of the beam are compared, the location of the cracking pattern is nearly similar. But, experimental crack path seems like different from the analyzed one. This may be due to, in practical, fracture energy depends on the size and crushing strength of aggregate that comes into account for crack path and propagation which lacks in the simulation

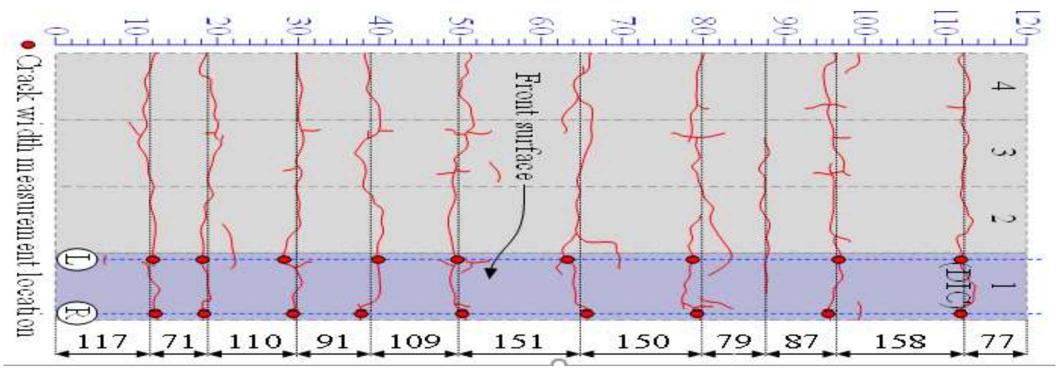


Figure 17:Cracking patterns observed from experimental result (Gribniak *et al.*, 2020)

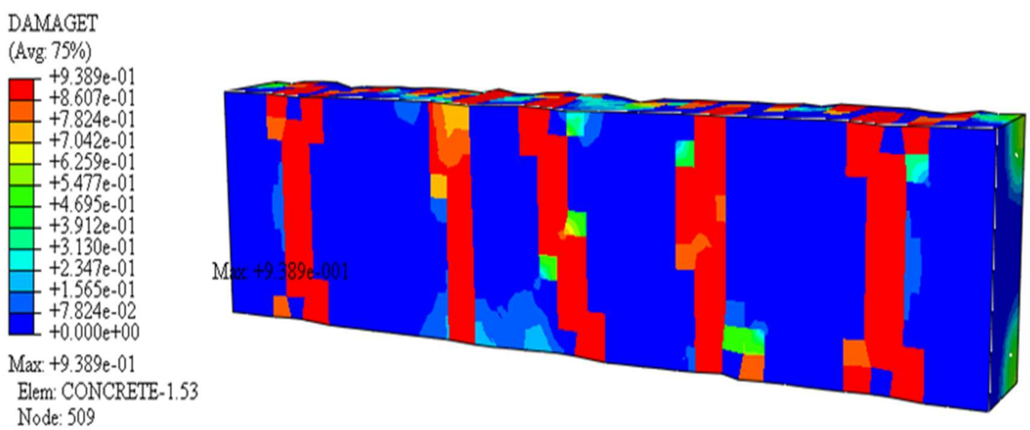


Figure 18: Cracking patterns observed from simulation result

CHAPTER FIVE
CONCLUSION AND RECOMMENDATIONS

5.1 Conclusions

The main objective of this research is to investigate the cracking limit states of different standard codes and numerical analysis of cracking pattern using finite element software, Abaqus. Accordingly, commonly used crack width calculation models such as EN ES, ACI and BS codes have been examined. Further, crack spacing of EN ES and MC-2010 codes were also considered. To compare the aforementioned codes, the main parameters that considered in each code are: concrete cover, bar diameter, and type of loading. The purpose was to study the suitable expression for crack width and crack spacing calculation by comparing with that of experimental results and conducting analytical study. As a result, it has been identified that ACI code gave a good agreement with the experimental crack widths than EN ES and BS codes. Accordingly:

1) Crack width of tensile loading members,

Compared to experimental results, assuming concrete cover (15) mm and bar diameter (10) mm, crack width obtained through analyse using ES, ACI and BS codes have a percentage error of, 108%, -8%, 38%, respectively. From the results, crack width estimated by ACI code is more related to experimental crack width than ES and BS codes. ES code estimate higher crack width than ACI and BS codes. Further, the crack width of ACI code is related to maximum value than the mean values of the experimental result.

2) Crack width of flexural loading member,

Compared to experimental results, assuming concrete cover (40) mm and bar diameter (16) mm, crack width obtained through analyse using ES, ACI and BS codes have a percentage error of, 21.4%, -11%, 25%, respectively. From the results, crack width estimated by ACI code is more related to experimental crack width than ES and BS codes. Further, the crack width of each codes is more related to mean value than the maximum values of the experimental result.

Generally, while considering the estimation of crack width in different codes, it could be observed that ACI code give relatively good fit to the experimental crack widths than ES-2 and BS codes. However, when considering the cases of all codes, it seems like that the existing codes to be needed to improve the expressions for crack width and crack spacing examinations.

3) Crack spacing of tensile loading member,

Ethiopian Standard code (ES) estimates higher crack spacing values than MC-2010 Code. Thus, the maximum crack spacing that obtained through analyse using Ethiopian code and Model Code 2010 was 243mm and172mm, with errors, -19.7% and 15.3%, respectively.

4) Crack spacing of flexural member,

ES code estimates crack spacing nearly in same trend as that of MC-2010 code. The maximum crack spacing that obtained through analyse using Ethiopian code and Model Code 2010 was 181mm and 172mm, with errors, -13.1% and -7.5%, respectively. Further, comparing to an experimental result, for both type of loading, MC2010 estimates less errors than ES code.

Generally, while considering the estimation of crack spacing in both codes, crack spacing estimated by MC-2010 gives relatively good fit to the experimental crack spacing than ES-2 code for both type of loading.

5) A finite element modelling, Abaqus, adopted in this work is suitable to predict the cracking behaviour of the reinforced concrete beams. Thus, the numerical results are in good agreement with the selected experimental cracking pattern results.

5.2 Recommendation

1. As large errors were observed, particularly for tensile loading members, in ES code, it seems like that reassessment of stress distribution parameter (k_2) is important.

2. While evaluation of cracking limit state is considered in different standard codes, compared to experimental results, it seems like that the existing codes need to improve the expressions for crack width calculation.

3. It is suggested that further studies on cracking limit state provision, particularly for crack width analysis that subjected to tensile loading members important.

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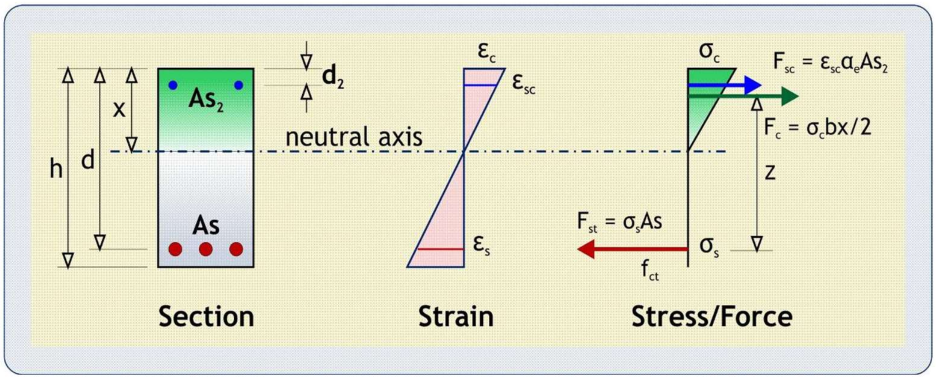
APPENDIX A

Crack Width Excel Templet According to Different Standard Code

LEGEND



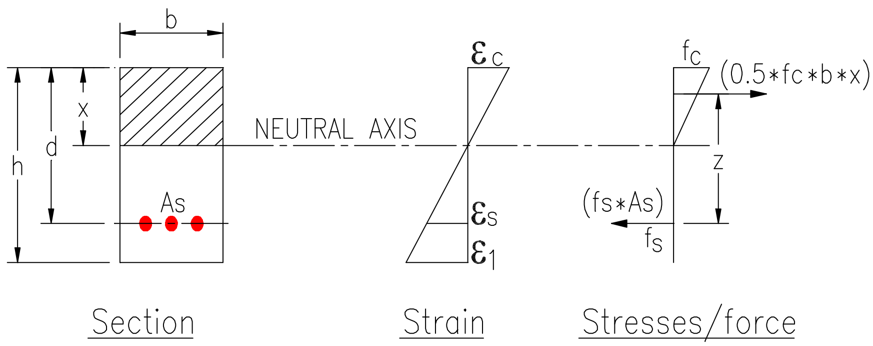
INPUT

f_{ck} =	29	MPa	Area of tension steel, A_s =	402	mm²
f_{yk} =	500	MPa	**d** =	300	mm
b =	250	mm	Area of compression steel, A_{s2} =	226	mm²
h =	348	mm	d_2 =	48	mm
QP moment, **M** =	24.9	KNm	Maxmum tension bar spacing, **S** =	154	mm
Age at cracking =	28	days	Max tension bar dia, $Ø_{eq}$ =	16	mm
Cement type =	N	(S, N, or R)	Short term or long term ?	L	(S or L)
Creep factor, **φ** =	0.0		Cover to A_s, **c** =	40	mm

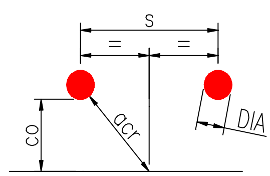
CALCULATIONS

modulus of elasticity of concrete = $22[(f_{ck}+8)/10]^{0.3}$	E_{cm} =	32.5	GPa
moduli of elasticity of steel	E_s =	200.0	GPa
Modular ratio	α_e =	6.16	
mean concrete strength at cracking	$f_{cm,t}$ =	36.60	MPa
mean concrete tensile strength	$f_{ct,eff}$ =	2.81	MPa
uncracked neutral axis depth $[bh^2/2+(\alpha_e-1)(A_s d+A_{s2}d_2)]/[bh+(\alpha_e-1)(A_s+A_{s2})]$	x_u =	175.27	mm
uncracked 2nd moment of area $bh^3/12+bh(h/2-x)^2+(\alpha_e-1)[A_s(d-x)^2+A_{s2}(x-d_2)^2]$	I_u =	929	mm⁴ 10⁶
cracking moment = $f_{ct}I/(h-x)$	M_{cr} =	15.09	kNm
< 24.9 kNm	→ section is CRACKED		
fully cracked neutral axis depth $(-A_s\alpha_e-A_{s2}(\alpha_e-1)+[\{A_s\alpha_e+A_{s2}(\alpha_e-1)\}^2-2b\{-A_s\alpha_e d-A_{s2}d_2(\alpha_e-1)\}]^{1/2})/b$	x_c =	66.69	mm
concrete stress = $M/[bx(d-x/3)/2+(\alpha_e-1)A_{s2}(d-d_2)(x-d_2)/x]$	σ_c =	10.384	MPa
stress in tension steel = $\sigma_c \cdot \alpha_e(d-x)/x$	σ_s =	223.8	MPa
effective tension area = $\min[2.5(h-d), (h-x)/3, h/2]b - A_s$	$A_{c,eff}$ =	23041	mm²
$A_s/A_{c,eff}$	$\rho_{p,eff}$ =	0.0174	
max final crack spacing = $\min[1.3/(h-x), 3.4c+0.17Ø/\rho_{p,eff})]$	$s_{r,max}$ =	291.9	mm
average strain for crack width calculation	$\varepsilon_{sm}-\varepsilon_{cm}$ =	762.7	μstrain
CALCULATED CRACK WIDTH	W_k =	**0.223**	mm

CRACK WIDTH CALCULATIONS - FLEXURE -



INPUT



fcu=	29	N/mm^2
fy=	500	N/mm^2
Area of reinforcement " **As** " =	402	mm^2
b =	250	mm
h =	348	mm
d =	300	mm
Minimum cover to tension reinforcement " **CO** " =	40	mm
Maxmum bar spacing " **S** " =	154	mm
Bar dia " **DIA** " =	16	mm
" $\mathbf{a_{cr}}$ " =(((S/2)^2+(CO+DIA/2)^2)^(1/2)-DIA/2) as default or enter other value =	**82.74**	mm

"acr " is distance from the point considered to the surface of the nearest longitudinal bar

Applied service moment " **Ms** "=	24.9	KNm

CALCULATIONS

moduli of elasticity of concrete " **Ec**" = (1/2)*(20+0.2*fcu) =	12.9	KN/mm^2
moduli of elasticity of steel " **Es** " =	200.0	KN/mm^2
Modular ratio " α " = (Es/Ec) =	15.52	
" ρ " = As/bd =	0.005	
depth to neutral axis, "**x**" = $(-\alpha.\rho + ((\alpha.\rho)^2 + 2.\alpha.\rho)^{0.5}.d$ =	100	mm
" **Z** " = d-(x/3) =	267	
Reinforcement stress " **fs** " = Ms/(As*Z) =	232	N/mm^2
Concrete stress " **fc** " = (fs*As)/(0.5*b*x) =	7.48	N/mm^2
Strain at soffit of concrete beam/slab " $\varepsilon 1$ " = (fs/Es)*(h-x)/(d-x) =	0.001440	
Strain due to stiffening effect of concrete between cracks " ε**2** " =		
$\varepsilon_2 = b.(h-x)^2/(3.Es.As.(d-x))$ for crack widths of 0.2 mm	Used	
$\varepsilon_2 = 1.5.b.(h-x)^2/(3.Es.As.(d-x))$ for crack widths of 0.1 mm	n/a	
ε_2 =	0.000319	
Average strain for calculation of crack width " ε_m "= $\varepsilon_1 - \varepsilon_2$ =	0.001121	

Calculated crack width, " w " = $3.a_{cr}.\varepsilon m/(1+2.(a_{cr}-c)/(h-x))$

CALCULATED CRACK WIDTH, 'w' =	**0.21**	**mm**

APPENDIX B

Crack width sample calculation for different standard codes

a) British standard code

Tension crack width:

$$w_t = 3 * \alpha_{cr} * \varepsilon_m \text{ ; where, } a_{cr} = \sqrt{\left(\frac{s}{2}\right)^2 + d_c{}^2} - \frac{\emptyset}{2}; \ d_c = c + \frac{\emptyset}{2}$$

Tension mean strain (ε_m): $\varepsilon_m = \varepsilon_1 - \varepsilon_2$; $\varepsilon_m = \varepsilon_1 - \frac{2b*h}{3*E_s*A_s}$

Spacing of bars (s): $s = b - (2c + dia.) = 150 - (2 * 15 + 10) = 110mm$

Tension stiffening (ε_2): $\varepsilon_2 = \frac{2*b*h}{3*E_s*A_s} = \frac{2*150*150}{3*199500*314} = 0.000239$

Strain of bar: $\varepsilon_1 = \frac{\sigma_s}{E_s} = \frac{320}{199500} = 0.0016$

Mean strain: $\varepsilon_m = \varepsilon_1 - \varepsilon_2 = 0.0016 - 0.000239 = 0.00136$

$*$ Taking a critical section nearest to longitudinal bar (α_{cr}) as :

$$\alpha_{cr} = c_{min} + {}^{\emptyset}/_{2} = 15 + 5 = 20mm$$

$$w_{cr} = 3 * \alpha_{cr} * \varepsilon_m = 3 * 20 * 0.00136 = 0.08$$

b) ACI Code

$$w_{max} = 0.0145 * f_s * \sqrt[3]{d_c * A_{0*}} 10^{-3} mm$$

Where; $d_c = c + {}^{\emptyset}/_{2} = 15 + \frac{10}{2} = 20mm$; $A_o = \frac{2*d_c*b}{n} = \frac{2*20*150}{4} = 1500mm^2$

$$w_{max} = 0.0145 * 320 * (20 * 1500)^{\left(\frac{1}{3}\right)} * 10^{-3} = 0.14mm$$

c) Ethiopian Code

Effective Tension Area, $(A_{eff}) = 2.5 * (150 - 130) * 150 - 158 = 7342mm^2$

Reinforcement Ration $(\rho_{eff}) = \frac{A_s}{A_{eff}} = \frac{158}{7342} = 0.0215$

Crack Spacing $(S_{rmax}) = 3.4 * 15 + 0.34 * \frac{10}{0.0215} = 210mm$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{320 - \frac{0.4 * 3.18}{0.0215} * (1 + 5.88 * 0.0215)}{200000} = 0.001267$$

$$w_{cr} = S_{rmax} * (\varepsilon_{sm} - \varepsilon_{cm}) = 210 * 0.001267 = 0.27$$

a) British Code

❖ Assuming bar diameters (φ) = 12mm; $\sigma_s = \frac{P}{A_s} = \frac{100.4kN}{3.14*12^2} = 222MPa$

Tension crack width:

$w_{cr} = 3 * \alpha_{cr} * \varepsilon_m$; where, $a_{cr} = \sqrt{\left(\frac{s}{2}\right)^2 + {d_c}^2} - \frac{\emptyset}{2}$; $d_c = c + \frac{\emptyset}{2}$

Tension stiffening (ε_2): $\varepsilon_2 = \frac{2*b*h}{3*E_s*A_s} = \frac{2*150*150}{3*199500*452.4} = 0.000166$

Strain of bar (ε_1): $\varepsilon_1 = \frac{\sigma_s}{E_s} = \frac{222}{199500} = 0.001113$

Mean Strain: $\varepsilon_m = \varepsilon_1 - \varepsilon_2 = 0.001113 - 0.000166 = 0.000947$

∗ Taking a critical section nearest to longitudinal bar (α_{cr}) as :

$$\alpha_{cr} = c_{min} + \emptyset/2 = 15 + 6 = 21mm$$

$$w_{cr} = 3 * \alpha_{cr} * \varepsilon_m = 3 * 21 * 0.000947 = 0.06$$

❖ Assuming bar diameters (φ) = 16mm; $\sigma_s = \frac{P}{A_s} = \frac{100.4kN}{3.14*16^2} = 125MPa$

Tension stiffening (ε_2): $\varepsilon_2 = \frac{2*b*h}{3*E_s*A_s} = \frac{2*150*150}{3*199500*804.25} = 0.0000935$

Strain of bar: $\varepsilon_1 = \frac{\sigma_s}{E_s} = \frac{222}{199500} = 0.0006266$

Mean strain: $\varepsilon_m = \varepsilon_1 - \varepsilon_2 = 0.0006266 - 0.0000935 = 0.000533$

∗ Taking a critical section nearest to longitudinal bar (α_{cr}) as :

$$\alpha_{cr} = c_{min} + \emptyset/2 = 15 + 8 = 23mm$$

Tension crack width: $w_{cr} = 3 * \alpha_{cr} * \varepsilon_m = 3 * 23 * 0.000533 = 0.04$

b) ACI Code

Assume bar diameter (ϕ): 12mm

$$w_{max} = 0.0145 * f_s * \sqrt[3]{d_c * A_{0*}} 10^{-3} mm$$

Where; $d_c = c + \emptyset/2 = 15 + \frac{12}{2} = 21mm$; $A_o = \frac{2*d_c*b}{n} = \frac{2*21*150}{4} = 1575mm^2$

➢ $w_{max} = 0.0145 * 222 * (21 * 1575)^{\left(\frac{1}{3}\right)} * 10^{-3} = 0.10mm$

c) Ethiopian standard code

❖ Bar diameter (ϕ): 12mm

$$\text{Effective Tension Area, } (A_{eff}) = 2.5 * (150 - 129) * 150 - 226.2 = 7648.8\text{mm}^2$$

$$\text{Reinforcement Ration } (\rho_{eff}) = \frac{A_s}{A_{eff}} = \frac{226.2}{7648.8} = 0.02957$$

$$\text{Crack Spacing } (S_{rmax}) = 3.4 * 15 + 0.34 * \frac{12}{0.02957} = 190\text{mm}$$

$$\text{Mean Strain: } \varepsilon_{sm} - \varepsilon_{cm} = \frac{222 - \frac{0.4*3.18}{0.02957} * (1 + 5.88 * 0.02957)}{199500} = 0.0008597$$

$$\text{Crack Width: } w_{cr} = S_{rmax} * (\varepsilon_{sm} - \varepsilon_{cm}) = 190 * 0.0008597 = 0.16$$

*Taking a lesser effective height of (h_{eff}) = h/2 = 75; for cover of {30,40,50}:

$$\text{Reinforcement Ration } (\rho_{eff}) = \frac{A_s}{A_{eff}} = \frac{226.2}{11{,}023.8} = 0.02052$$

$$\text{Crack Spacing } (S_{rmax}) = 3.4 * 30 + 0.34 * \frac{12}{0.02052} = 300\text{mm}$$

$$\text{Mean Strain: } \varepsilon_{sm} - \varepsilon_{cm} = \frac{222 - \frac{0.4*3.18}{0.02052} * (1 + 5.88 * 0.02052)}{199500} = 0.0007646$$

$$w_{cr} = S_{rmax} * (\varepsilon_{sm} - \varepsilon_{cm}) = 250 * 0.0007646 = 0.23$$

❖ Bar diameter (ϕ): 16mm

$$\text{Effective Tension Area, } (A_{eff}) = 2.5 * (150 - 127) * 150 - 402 = 8223\text{mm}^2$$

$$\text{Reinforcement Ration } (\rho_{eff}) = \frac{A_s}{A_{eff}} = \frac{402}{8223} = 0.048887$$

$$\text{Crack Spacing } (S_{rmax}) = 3.4 * 15 + 0.34 * \frac{16}{0.048887} = 162\text{mm}$$

$$\text{Mean Strain: } \varepsilon_{sm} - \varepsilon_{cm} = \frac{125 - \frac{0.4*3.18}{0.048887} * (1 + 5.88 * 0.048887)}{199500} = 0.0004587$$

$$w_{cr} = S_{rmax} * (\varepsilon_{sm} - \varepsilon_{cm}) = 162 * 0.0004587 = 0.07$$

*Taking a lesser effective height of (h_{eff}) = h/2 = 75; for cover of {30,40,50}:

$$\text{Reinforcement Ration } (\rho_{eff}) = \frac{A_s}{A_{eff}} = \frac{402}{10{,}848} = 0.03706$$

$$\text{Crack Spacing } (S_{rmax}) = 3.4 * 30 + 0.34 * \frac{16}{0.03706} = 248.8\text{mm}$$

$$\text{Mean Strain: } \varepsilon_{sm} - \varepsilon_{cm} = \frac{125 - \frac{0.4*3.18}{0.03706} * (1 + 5.88 * 0.03706)}{199500} = 0.000417$$

$$w_{cr} = S_{rmax} * (\varepsilon_{sm} - \varepsilon_{cm}) = 248.8 * 0.000417 = 0.10$$

APPENDIX C

Crack spacing sample calculations

a) Ethiopian Code Crack Spacing Calculation:

$$S_{r,max} = k_3 * c + k_1 * k_2 * k_4 * \left({}^{\emptyset}/{}_{\rho_{p,eff}}\right)$$

$$S_{r,max} = 3.4 * c + 0.8 * 1 * 0.425 * \left({}^{\emptyset}/{}_{\rho_{p,eff}}\right) = 3.4 * c + 0.34 * \left({}^{\emptyset}/{}_{\rho_{p,eff}}\right)$$

*For diameter of bar and concrete cover, 32 and 35, respectively;

$$S_{r,max} = 3.4 * c + 0.34 * \left({}^{\emptyset}/{}_{\rho_{p,eff}}\right) = 3.5 * 35 + 0.34 * \left(\frac{32}{0.087}\right) = 244mm$$

b) Model Code-2010 Crack Spacing Calculation:

$$S_{r,max} = 2\left\{k * c + \frac{1}{4} * \frac{f_{ctm}}{\tau_{bms}}\left({}^{\emptyset}/{}_{\rho_{p,eff}}\right)\right\}$$

*For diameter of bar and concrete cover, 32 and 35, respectively;

$$S_{r,max} = 2\left\{1 * 35 + \frac{1}{4} * \frac{3.2}{1.8 * 3.2}\left({}^{32}/{}_{0.0874}\right)\right\} = 172mm$$

a) Ethiopian Code Crack Spacing Calculation:

$$S_{r,max} = k_3 * c + k_1 * k_2 * k_4 * \left({}^{\emptyset}/{}_{\rho_{p,eff}}\right)$$

$$S_{r,max} = 3.4 * c + 0.8 * 0.5 * 0.425 * \left({}^{\emptyset}/{}_{\rho_{p,eff}}\right) = 3.4 * c + 0.17 * \left({}^{\emptyset}/{}_{\rho_{p,eff}}\right)$$

*For diameter of bar and concrete cover, 32 and 35, respectively;

$$S_{r,max} = 3.4 * c + 0.17 * \left({}^{\emptyset}/{}_{\rho_{p,eff}}\right) = 3.4 * 35 + 0.17 * \left(\frac{32}{0.087}\right) = 182mm$$

b) Model Code-2010 Crack Spacing Calculation:

$$S_{r,max} = 2\left\{k * c + \frac{1}{4} * \frac{f_{ctm}}{\tau_{bms}}\left({}^{\emptyset}/{}_{\rho_{p,eff}}\right)\right\}$$

*For diameter of bar and concrete cover, 32 and 35, respectively;

$$S_{r,max} = 2\left\{1 * 35 + \frac{1}{4} * \frac{3.2}{1.8 * 3.2}\left({}^{32}/{}_{0.0874}\right)\right\} = 172mm$$

APPENDIX D

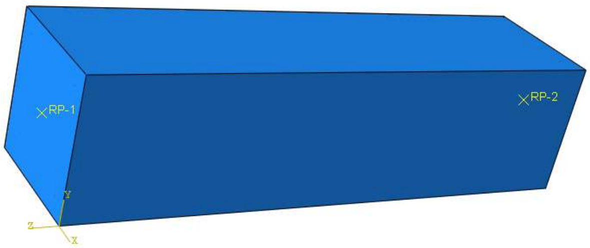
Plastic behaviour's data for concrete used in CDP model

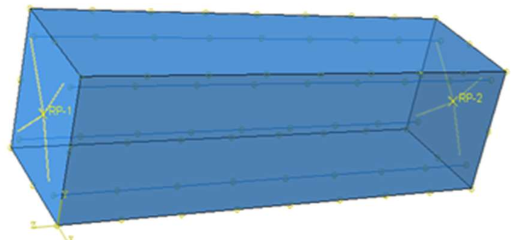
Stress-Strain Data				Damage Behaviors			
Compression		Tension		Compression		Tension	
17.00	***0.000%***	**3.200**	***0.00%***	0	*0.00%*	0.000	*0.00%*
17.60	*0.002%*	2.266	*0.06%*	0.483	*0.43%*	0.292	*0.06%*
20.63	*0.003%*	1.635	*0.11%*	0.589	*0.51%*	0.489	*0.11%*
23.48	*0.006%*	1.222	*0.17%*	0.672	*0.59%*	0.618	*0.17%*
26.16	*0.009%*	0.956	*0.22%*	0.735	*0.66%*	0.701	*0.22%*
28.65	*0.012%*	0.783	*0.28%*	0.782	*0.73%*	0.755	*0.28%*
30.95	*0.016%*	0.666	*0.33%*	0.819	*0.80%*	0.792	*0.33%*
33.07	*0.021%*	0.580	*0.39%*	0.848	*0.87%*	0.819	*0.39%*
34.98	*0.026%*	0.511	*0.44%*	0.871	*0.94%*	0.840	*0.44%*
36.70	*0.032%*	0.450	*0.50%*	0.889	*1.01%*	0.859	*0.50%*
38.20	*0.039%*	0.394	*0.55%*	0.903	*1.08%*	0.877	*0.55%*
39.49	*0.046%*	0.340	*0.61%*	0.915	*1.15%*	0.894	*0.61%*
40.56	*0.054%*			0.925	*1.21%*	0.910	*0.66%*
41.40	*0.062%*			0.934	*1.28%*	0.925	*0.72%*
42.01	*0.072%*			0.941	*1.35%*	0.939	*0.77%*
42.38	*0.082%*			0.947	*1.41%*		
42.51	***0.093%***						
42.46	*0.103%*						
39.66	*0.177%*						
34.00	*0.261%*						
27.65	*0.346%*						
22.00	*0.430%*						
17.45	*0.510%*						
13.95	*0.586%*						
11.29	*0.660%*						
9.25	*0.733%*						
7.69	*0.803%*						
6.47	*0.873%*						
5.50	*0.942%*						
4.73	*1.010%*						

APPENDIX E

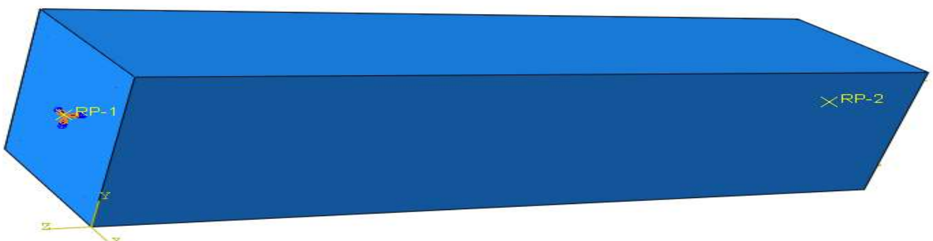
Procedures for Modelling in Abaqus and its Output

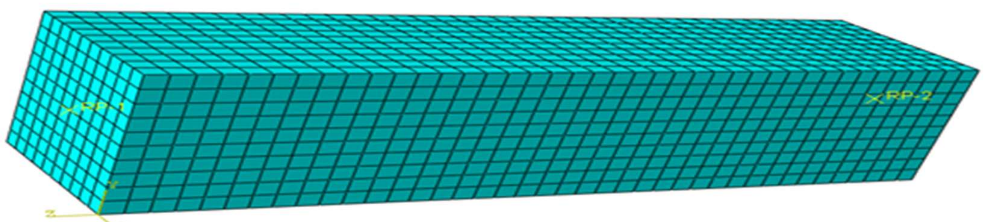
❖ Assembly and interactions of the materials- Beam and Bar



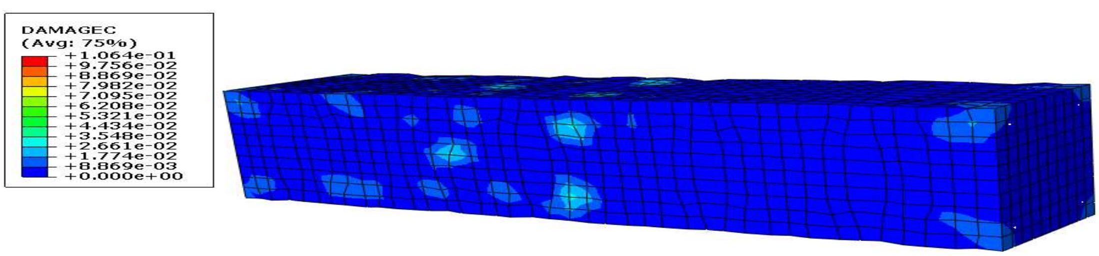


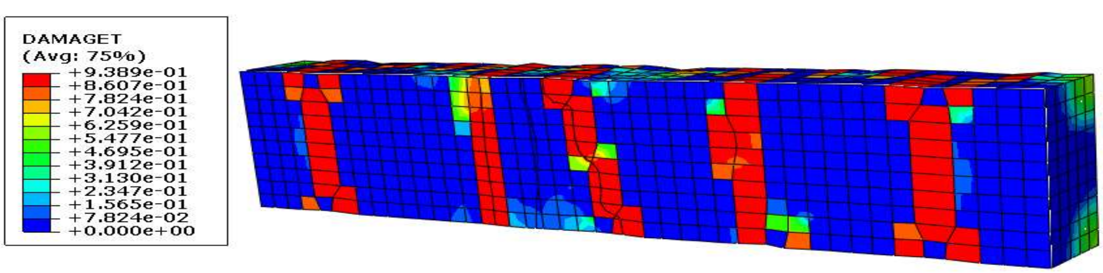
❖ Load, Boundary conditions and meshing





❖ Concrete damage output





❖ Crack opening intensity for cracking, displacement captions and stresses