

THESIS FOR THE DEGREE OF LICENTIATE OF ENGINEERING

Reliable assessments of concrete structures with
corroded reinforcement: An engineering approach

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Gothenburg, Sweden 2017

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An engineering approach*

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Thesis for the degree of Licentiate of Engineering 2017:6

ISSN no. 1652-9146

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Cover:

The picture shows a contour plot of the coefficient of variation for bond test results in a large database. Modified from *Paper III*.

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Gothenburg, Sweden, 2017

To Sara

Reliable Assessments of Concrete Structures with Corroded Reinforcement

An engineering approach

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ABSTRACT

When in service, concrete structures are inevitably exposed to detrimental processes which, over time, can affect the structures' ability to fulfil their structural requirements. The most common cause of deterioration of concrete structures is corrosion of the reinforcement; a problem that is expected to become more severe in future, due to climate change. Nevertheless, the demands for load-bearing capacity often increase with time. Advances in engineering methods are needed for reliable structural assessment of existing structures. Such methods should be calibrated with experiments and advanced analyses and complemented with appropriate safety formats. This will allow us to meet future needs in an environmentally friendly and economic way and avoid unnecessary re-constructions.

The influence of load history on reliability level was compared for the safety formats provided in *fib* Model Code 2010. A statically indeterminate structure subject to vertical as well as horizontal loading was analysed. Not all safety formats reached the intended safety levels for all load histories. This illustrates how important it is to consider the load history when making structural assessments.

One stage in developing an engineering model for bond of corroded reinforcement was to investigate the possibility of using thin-section models to represent bond behaviour in beam anchorage zones. Thin-section models were compared to larger, finite element models as well as experimental results. The results showed that the thin-section models may be able to capture the bond capacity, although the results were strongly influenced by the boundary conditions applied to the thin-section model.

An existing one-dimensional model was further developed showing the bond capacity of corroded reinforcement bars in concrete. This new model was based on the local bond stress-slip relationship in *fib* Model Code 2010 and was calibrated against a large database of bond test results. During the verification, the model showed results which were slightly on the safe side. There was also good agreement with an empirical expression for bond capacity found in the literature.

The contributions made by this work provide engineers with a simple, verified model of the bond capacity of corroded reinforcement in concrete. They also facilitate the development of a more holistic and reliable assessment method. The knowledge gained about the influence of load history and boundary conditions will be utilised in further development of the one-dimensional bond model, as it will be expanded to cover more reinforcement layouts and complemented with an appropriate safety format.

Keywords: bond, anchorage, reinforced concrete structures, finite element modelling, reliability, safety formats.

Preface

The work presented in this thesis was mainly conducted between October 2015 and March 2017 at the Division of Structural Engineering at Chalmers University of Technology in Gothenburg and at CBI Swedish Cement and Concrete Research Institute in Borås, Sweden. The research project was funded by the Swedish Road Administration, CBI Swedish Cement and Concrete Research Institute's A-consortium and SBUF.

I would like to thank my supervisor, Assoc. Prof. Kamyab Zandi for his continuous feedback and many interesting discussions. Sincere thanks also to my co-supervisor and examiner, Prof. Karin Lundgren for delivering countless valuable comments and good ideas. I would also like to thank my co-supervisor Oskar Larsson Ivanov who, despite the distance, managed to stay in touch and provide good input for this project. To all three of you, I wish to express my gratitude for your encouragement and support on the way to this licentiate degree. I very much look forward to continued collaboration.

I would also like to thank the other members of the project group: Dániel Honfí and Per Kettil, for all their good ideas and valuable discussions. I would also like to thank my friend and supervisor of my Master's thesis, Morten Engen, for awakening my interest in research.

The friendly and open atmosphere created by all my colleagues at the Division of Structural Engineering and at CBI in Borås, makes it a pleasure to go to work each morning. Special thanks to my colleagues and good friends, Carlos Gil Berrocal and Ignasi Fernandez Perez for all our discussions regarding just about everything.

The ongoing love and support of my family and friends makes me feel truly blessed. Thank you for encouraging me to do what I'm doing, even though you aren't quite sure what it is!

Sara, I can't believe that we have found each other. We are so different but at the same time so very much alike. Thank you for showing me what is truly important in life; without you all this would be pointless.



Mattias Blomfors

Gothenburg, 2017

LIST OF PUBLICATIONS

This thesis is based on the work contained in the following papers:

Paper I

Blomfors M., Engen M., Plos M. (2016): Evaluation of safety formats for non-linear finite element analyses of statically indeterminate concrete structures subjected to different load paths. Published in *Structural Concrete*, **17**(1), pp. 44-51.

Paper II

Blomfors M., Zandi K., Lundgren K. (2016): Development of engineering assessment method for anchorage in corroded reinforced concrete. Published in *Nordic Concrete Research*, No.55 2/2016.

Paper III

Blomfors M., Coronelli D., Lundgren K., Zandi K. (2017): Engineering bond model for corroded reinforcement. Submitted to *Engineering Structures*.

AUTHOR'S CONTRIBUTIONS TO JOINTLY PUBLISHED PAPERS

The contribution of the author of this licentiate thesis to the appended papers is described here.

- I. Partly responsible for planning the study. Conducted the literature review, performed the non-linear finite element analyses and applied the reliability methods. Took lead in writing the paper.
- II. Planned the main part of the paper. Performed the non-linear finite element analyses and wrote the main part of the paper.
- III. Planned the main part of the paper. Developed the main part of the engineering bond model. Took lead in writing the paper.

ADDITIONAL PUBLICATIONS BY THE AUTHOR

Conference Papers

Blomfors M., Zandi K., Lundgren K., Larsson O., Honfi D. (2016): Engineering assessment method for anchorage in corroded reinforced concrete. *Proceedings of the 19th IABSE Congress*, Stockholm, 21-23 September 2016.

Popular Science Papers

Lundgren K., Zandi K., Blomfors M., Fernandez I., Tahershamsi M. (2016): Modellering av förankringskapaciteten i betongkonstruktioner med rostande armering. *Bygg & Teknik (0281-658X)*, Vol.108, 7, pp. 16-20.

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1 Introduction

1.1 Background

Over the last century, society has invested heavily in reinforced concrete infrastructure. Inevitably, concrete structures in service are exposed to detrimental processes which, over time, can affect their ability to fulfil their structural requirements. The most common cause of deterioration is corroded reinforcement (Bell, 2004). In the future, it is anticipated that climate change will make this common problem yet more severe (Stewart *et al.*, 2011) even as demands for load-bearing capacity increase. Advances in development of methods for structurally assessing existing structures are needed, if future needs are to be met in an economic and environmentally friendly way that avoids needless rebuilding.

1.1.1 Uncertainties in assessment of structures

There are many uncertainties connected to the structural design of new structures and structural analysis of existing ones. Regardless of the commonly held view, both the applied loading and the load-bearing capacity of a structure are uncertain (Nowak & Collins, 2000). The randomness of the input variables for loading and structural capacity means that absolute safety for structures is a utopia; the probability of failure will never be zero. Rather, structures must fulfil the requirements for a finite probability of failure. It should be kept in mind that ‘failure’ in this context means that the structure did not perform as intended; it does not necessarily entail a collapse of the structure. Common performance criteria include such things as allowable deflection and crack width in service, plus capacity in the ultimate limit state.

The uncertainties related to the building process comprise the stages of planning, designing, constructing, using and finally demolishing a structure. Broadly, these uncertainties can be grouped into two categories: those emanating from natural causes and those from human causes. Uncertainties due to natural causes include the properties of construction materials and the randomness of environmental loads. Examples of uncertainties springing from human causes include simplified design calculations, lack of communication between parties to the building process and use of the structure in a way that was not intended. There are also uncertainties related to the condition assessment of structures; these may fall into either category depending on their type. For example, the degree of deterioration of a structure is often affected by natural uncertainties, while potential errors in the condition assessment may stem from human causes.

Due to these uncertainties, both the loads applied to a structure and its load-bearing capacities are random variables. A reliability analysis is a structured procedure to account for these uncertainties and achieve a structure with an intended (finite) level of safety.

1.1.2 Consequences of reinforcement corrosion in concrete structures

The reinforcement in structural concrete is initially protected by a passivating layer, formed by the surrounding alkaline concrete. The reinforcement will not corrode until

this passivation is broken. This may occur due to carbonation or the ingress of chloride ions from such things as de-icing salts. Other less common causes include sulphide ion attacks or stray DC currents (Cairns & Millard, 1999). Cracks in the concrete due to, say, service loading may increase the availability of chloride ions at the reinforcement and expedite the corrosion process.

An overview of the mechanisms and consequences of reinforcement corrosion is shown in Figure 1.

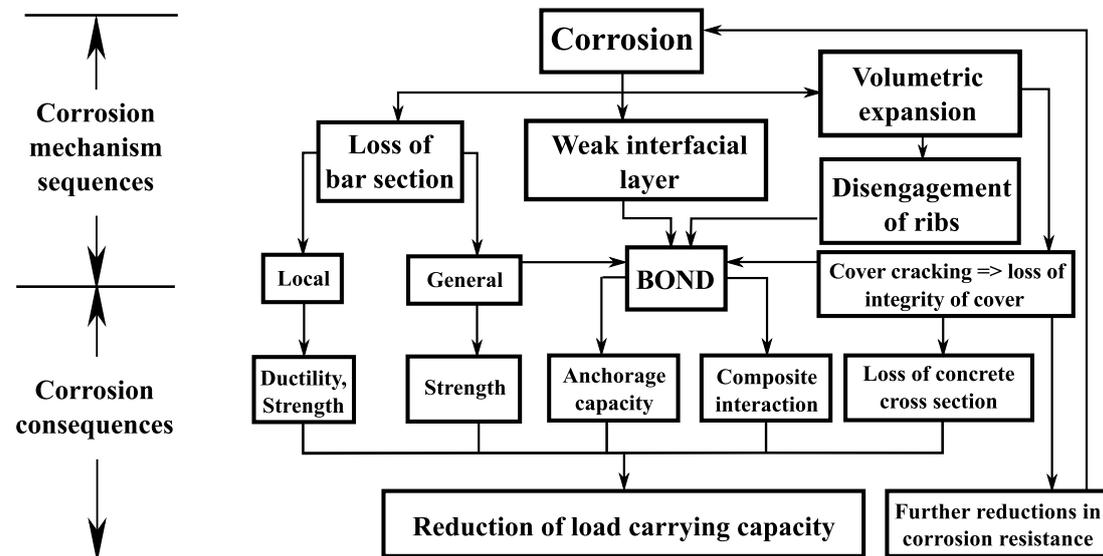


Figure 1: Overview of the mechanisms and consequences of reinforcement corrosion in structural concrete. Adopted from (Tahershamsi, 2016) and originally redrawn from (Cairns & Millard, 1999).

A concrete structure is affected in many ways when the reinforcement corrodes. The most important influences can be summarised as:

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

The corrosion process involves the dissolution of iron from the steel bars. This reduces the remaining steel area and the bars subsequently lose their original ribbed shape. The physical appearance of the corroded bar can be linked to the source of corrosion; uniform corrosion is generally associated with corrosion in carbonated concrete while more localised, pitting corrosion is associated with the presence of chlorides (Chan *et al.*, 2005). However, it is often hard to characterise corrosion damage as either uniform or localised since the two characteristics often appear in combination. Furthermore, when the cross-sectional area of the reinforcing bars is reduced, so is their capacity and ductility (Almusallam, 2001; Du *et al.*, 2005). As many reinforcing bars have higher-strength steel nearer to the surface than the centre of the bar, corrosion may also reduce the tensile strength of the rebar (Fernandez *et al.*, 2015).

As the steel is transformed into rust, a layer of corrosion products forms between the reinforcement bar and its surrounding concrete. This occupies a greater volume than the original steel and internal pressure builds up. If the confinement of the surrounding concrete is sufficient, this may initially increase the bond capacity.

Moreover, a porous concrete has a greater proportion of corrosion products penetrating the pore system. The pressure build-up may therefore occur at a lower rate than in a less porous concrete. As corrosion of the reinforcement bars continues, the surrounding concrete eventually fails to carry the induced tensile stresses and longitudinal splitting cracks develop. Consequently, confinement diminishes and the bond capacity decreases (Almusallam *et al.*, 1996; Auyeung *et al.*, 2000; Cabrera & Ghoddoussi, 1992). The layer of corrosion products may also force the surrounding concrete away from the reinforcement bar, leaving a smaller effective rib area (Cairns & Millard, 1999). After cracking, the bond capacity may either markedly decrease with further corrosion (as with low levels of transverse reinforcement) or slightly increase in case of substantial stirrup content (Al-Sulaimani *et al.*, 1990; Rodriguez *et al.*, 1997; Mangat & Elgarf, 1999; Berra *et al.*, 2003).

On the structural level, the shear and bending capacity is reduced by corrosion. This is due to a) the rebar cross-section being smaller, b) a lower effective depth due to spalling of the concrete cover and c) disturbed bond conditions. The tension stiffening of a member is also affected which influences the deflection and crack widths. Experimental observations show that beams with simultaneous loading and corrosion (as is common for structures in service) prematurely reach their deflection limit in the serviceability limit state (Du *et al.*, 2013). Additionally, the plastic rotation capacity is affected. In indeterminate structures, this influences moment redistribution as well as robustness and seismic resistance (Cairns *et al.*, 2008).

Concrete structures are generally designed to fail in a ductile manner if their ultimate capacity is exceeded, thus allowing people to avoid immediate danger. However, reinforcement corrosion may lead to an abrupt collapse of the structure. For example, a bridge in Pennsylvania, US, collapsed onto highway I-70 below while subjected to dead load (Naito *et al.*, 2010). Subsequent forensic investigations found considerable corrosion damage to the reinforcement and also spalling of the concrete cover. Fortunately, the collapse only resulted in minor personal injuries. Nevertheless, it clearly signals that all possible measures should be undertaken to avoid these situations the future.

1.1.3 Modelling of concrete structures with corroded reinforcement

In structural engineering, finite element (FE) analyses are often used for design and analysis of concrete structures. The design process regularly involves linear FE analyses to find one possible set of sectional forces in equilibrium with the applied loads. The amount of reinforcement and its layout in the structure is then designed for these sectional forces using basic sectional resistance calculations and following restrictions to ensure that the requirements for redistribution of moments are met. In reality, reinforced concrete structures show a non-linear behaviour when subjected to increased loading. Cracking of concrete and yielding of reinforcement changes the stiffness properties of the material and thus stress redistributions may occur within the structure (Cervenka, 2013). Non-linear FE analyses (NLFE) are required to capture this behaviour. Such analyses allow geometric and material non-linearities to be accounted for, thus providing a more realistic estimate of the capacity.

To realistically analyse the behaviour of concrete structures with corroded reinforcement, the non-linear behaviour needs to be considered as well as the different consequences of reinforcement corrosion. The reduction in area (due to corrosion) of

the reinforcement and concrete cross-sections can be modified in the geometrical input to the FE analysis. Also the changed ductility of the reinforcement bars can be included in the analyses; empirical expressions for the relationship between ductility and corrosion level are available.

However, more complexity arises from the change of bond between reinforcement and concrete due to the corrosion level and its effect on the structure. Several models have been proposed to represent the complex bond properties of corroded reinforcement. Typically, such methods require the use of detailed three-dimensional (3D) NLFE models, cf. (Berra *et al.*, 2003; Lundgren, 2005b; Zandi Hanjari *et al.*, 2013). Whilst the modelling implementations differ between the models, they are all capable of representing the expansion of corrosion products and the cracking of concrete cover. The latter also accounts for the flow of corrosion products through cracks.

Although existing models can represent bond behaviour between corroded reinforcement and concrete, widespread practical application is limited. This is because 3D NLFE analyses require major resources in terms of time and competence. Simplified models are needed in order to utilise the knowledge and understanding from previous research and advanced modelling in engineering practice. These models should be sufficiently accurate as well as time-effective for use in assessing existing concrete structures.

A simplified model for assessing anchorage in corroded reinforced concrete structures has previously been established, to meet the needs of the engineering community. Its short name is ARC1990 and it was originally formulated based on the analytical bond stress-slip relationship in *fib* Model Code 1990 (CEB, 1993) plus the findings of (Schlune, 2006; San Roman, 2006), combined with a parametric study using 3D NLFE analyses (Lundgren *et al.*, 2009). ARC1990 has been verified using test results from naturally corroded specimens (Tahershamsi *et al.*, 2016) and validated by 3D NLFE analyses and experiments for high-corrosion attacks leading to cover spalling (Zandi, 2015).

1.1.4 Potential benefits of a simplified assessment method

The empirical benefits of the ARC1990 model were demonstrated in a pilot study. This was conducted on two bridges in Stockholm, Sweden, which had systematic corrosion damage in cast joints where a lot of the reinforcement had been spliced (Lundgren *et al.*, 2015). Small spacing between the corroded bars and a relatively large maximum aggregate size in the concrete caused spalling of the concrete on the underside of the main beam at the cast joints.

The bond assessment was conducted by calculating the anchorage length needed to anchor the yield force, based on the bond stress-slip relationship from ARC1990. This direct and simple analysis approach proved easy to use in practice. The bridges were shown to have sufficient capacity and just for these two, use of the assessment model saved some €3 million by avoiding unnecessary strengthening.

However, areas for improvement in the model were also identified during the case study. In the main, these relate to the cross-sectional position of the studied bar, the influence of transverse reinforcement, influence of several layers of reinforcement and closely spaced bars. These aspects are addressed in the objectives of this thesis.

The Swedish Road Administration manages some 20,000 bridges and there are around a million bridges in EU27, many of them built from reinforced concrete and located in corrosive environments. The potential for societal and environmental cost savings is therefore huge, if simple and reliable assessment methods are made available to the engineering sector.

1.2 Aim and objectives

The aim of the work for this licentiate thesis was to develop a reliable assessment model for anchorage of corroded reinforcement in reinforced concrete. Some specific objectives have been targeted to serve the main aim of this work:

- To provide a background on reliability assessment of structures, investigate different safety formats and the influence of load history. The overall aim is served by addressing important aspects, to be included in the further development of assessment model for anchorage.
- To develop a simplified 3D NLFE model suitable for parametric studies, capable of describing the bond strength of corroded reinforcement in concrete. This contributes to the overall aim of the work by allowing for incorporation of more cross sectional geometries in the 1D-bond model.
- To implement *fib* Model Code 2010 in the existing ARC1990 model for assessing anchorage, and also calibrate and verify the new model. The reliable assessment model, which is sought outcome of the work, will be based on this model. The two bullets points above cover important parts for the further development of the model.

1.3 Method

In this section the methods used in the three papers included in this thesis are presented, together with an explanation of how the works fit together and how the knowledge gained from each paper contributes to the overall aim.

Paper I compared safety formats for NLFE analyses given by *fib* Model Code 2010 for two different load paths. The comparison is made for a relatively simple statically indeterminate structure; a concrete frame. Simple reliability methods are applied to study if the different safety formats yielded the intended safety level, and the influence of loading history is investigated. This knowledge is important for further FE analyses, and also when establishing an appropriate safety format for the engineering bond model for corroded reinforcement.

Paper II investigated whether NLFE models of thin sections of a beam-end were capable of showing bond behaviour that was similar to a larger model. This work contributes to further development of the analytical 1D-model, in particular by facilitating numerical studies of reinforcement layouts that are scarce in the literature.

Paper III further developed an existing analytical 1D-model based on Model Code 1990 to incorporate provisions given in *fib* Model Code 2010. International collaboration afforded access to a large database of experimental results to which calibration and validation of the model was performed. To include more reinforcement layouts, the results from **Paper II** are to be used in further development of the 1D-model.

1.4 Limitations

The main limitations of **Papers I to III** are summarized in the following.

In the structural analyses performed in **Paper I**, the ultimate capacity was generally governed by concrete compression failure in one of the frame corners. More complex load paths, such as cyclic loading and structures exhibiting other types of failure (shear failure for example) would be required for an exhaustive comparison of the safety formats. However, these investigations were not included due to the timeframe of the project.

Regarding the work presented in **Paper II**, an important limitation was that the applicability of thin-section models for modelling of splices is not investigated.

In **Paper III**, the calibration of the new model is based on test specimens corroded by means of artificial corrosion. Although there are questions regarding the applicability to natural corrosion, this was the best available data since tests of naturally corroded specimens are very scarce. Furthermore, as the engineering bond model is constructed to give the average bond strength, it must be complemented with an appropriate safety format before application in assessments. However, this was beyond the scope of this paper.

1.5 Original features

Reliability-based assessments of concrete structures with corroded reinforcement are not conducted by practicing engineers today. However, in this author's opinion it would be highly beneficial if their toolkits could be enhanced by these types of analyses. This work constitutes the foundation upon which further developments of the reliability-based assessment method for engineering applications will be built.

1.6 Outline

This thesis is outlined as follows:

Chapter 1 provides a background and also presents the aim, method, limitations and original features for the work.

Chapter 2 describes the procedure for reliability assessment of existing structures and also presents the concept of safety factors.

Chapter 3 elaborates on the bond strength of corroded reinforcement in concrete.

Chapter 4 describes different approaches, with varying complexity, to modelling anchorages in structures with corroding reinforcement.

Chapter 5 draws conclusions based on the conducted work and presents suggestions for further research.

At the end of the thesis the three papers included papers are appended.

2 Reliability assessment of existing structures

The main overall aim for the work included in this thesis is to develop an assessment model for anchorage in corroded reinforced concrete. In order to contextualise such a model, it is important to present a background describing how structural reliability assessments are performed in practice. The procedure comprises several tasks and this section presents a brief overview of the general framework for assessing structures as presented by (Holický *et al.*, 2013), based on (ISO2394, 1998; ISO13822, 2001; ISO12491, 1998).

In many ways, designing new structures and assessing existing ones are similar activities, but they also differ in key areas. Obviously, at the design stage of a construction process, any differences between the original and the as-built design, or possible structural damage sustained during the service life are unknowns which will be determined in the future. However, for an existing structure this information about a structure is known, at least to some extent. The structure may have been subjected to changes in the original design, the as-built design or both. It may also have been misused or experienced deterioration such as reinforcement corrosion. The current structural state and loading conditions of a structure should be used when making a reliability assessment.

The safety of a structure is usually assessed due to one (or more) of the following reasons:

- **Changes in use:** an assessment may be necessary to demonstrate the safety of a structure if the conditions of use (and thereby the associated loads) are changed.
- **Addition of new structural members:** if new elements are added to the load-bearing system, these influence structural behaviour and a reliability assessment of the structure may be necessary.
- **Repair:** structures damaged by accidents or natural phenomena, or by environmental impacts such as reinforcement corrosion may require different repair measures, depending on the outcome of an assessment. Also of interest is the safety level of a repaired structure.
- **Doubts as to safety:** assessments for answering questions on structural safety which are raised for other reasons.
- **Other circumstances:** insurance companies, authorities or owners may have special requirements for safety assessments.

Furthermore, reliability assessments are generally conducted according to two main principles:

- **Use of current codes:** the codes valid at the time of assessment should be used; previous codes, for example those valid at the time when the structure was designed, should serve as guiding documents only.
- **Use of actual structure:** the *in-situ* material properties, actual applied loads and actual geometries should be used. Furthermore, the structural behaviour at the time of assessment should be estimated. Hence design documentation should be used for guidance only.

Figure 2 shows an overview of the processes associated with a structural reliability assessment.

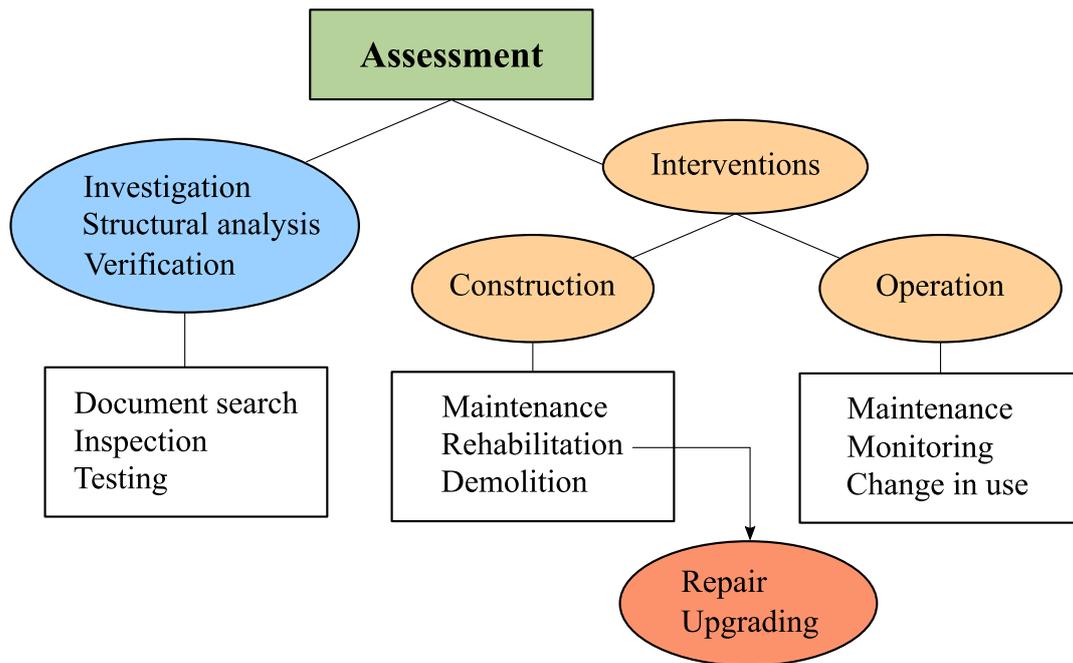


Figure 2: Overview of the parts of an assessment of an existing structure, adopted from (ISO13822, 2001).

It is not necessary to include all parts of the structure in all assessments. If parts of a structure are not affected by changes (due to repair or change of use for example) and are neither damaged nor suspected of having insufficient reliability, those parts can be excluded from the assessment. Figure 3 shows a flow chart of the processes involved in a reliability assessment.

At the initiation stage of the assessment process of a structure, the objectives in terms of its future performance should be established by the client, the assessing engineer and any relevant authorities.

The next step is to specify possible scenarios associated with changes in the structural conditions or actions. This is to help identify any critical situations for the structure. The identified scenarios form the basis for the assessment and any interventions to ensure the structure's safety and reliability.

A preliminary assessment follows, in which available documents and other evidence is studied. This establishes the state of knowledge regarding the structure. In a preliminary on-site inspection, the structural system and any damage is identified by visual inspection. Visible damage typically includes deformations, cracks, spalling and signs of corrosion and is graded in qualitative terms (none, minor, moderate, severe, destructive, unknown). The corrosion level is difficult to quantify using non-destructive measures. However, efforts have been directed towards linking the surface crack widths to the corrosion level, see (Tahershamsi *et al.*, 2016). Based on the information acquired, preliminary checks are made to identify current and future deficiencies critical to the safety and serviceability of the structure. If these preliminary checks clearly indicate that the structure is in a dangerous condition, immediate measures should be prescribed to reduce danger to the public. Based on the preliminary assessment, a determination is made as to whether further investigations are necessary. If so, a detailed assessment may be carried out.

A detailed assessment includes an in-depth scrutiny of all available documentary information. If there are reasons to doubt the information, it should instead be

collected during the detailed inspection and through material testing. This may include structural dimensions and properties of materials. These quantitative results give updated values on relevant parameters and are used in the structural analysis, alongside the actions determined for the structure. The structural analysis provides information about the load effects from the actions on the structure and the capacity of its structural components. It is of great importance that deterioration of the existing structure should be considered in the analysis. Appropriate methods should be used to carry out a reliability assessment of time-dependent deterioration problems. Testing can be used to estimate the load-bearing capacity and certain properties of the structure. Structural tests can provide additional information if detailed inspections and structural analyses fail to give a clear picture, or if the demonstrated structural reliability is deficient.

Structural performance is verified by ensuring that the structure meets the target reliability level. Past performance may also form the basis of this verification. The results of the assessment should be documented in a report, which provides conclusions and any suggested interventions.

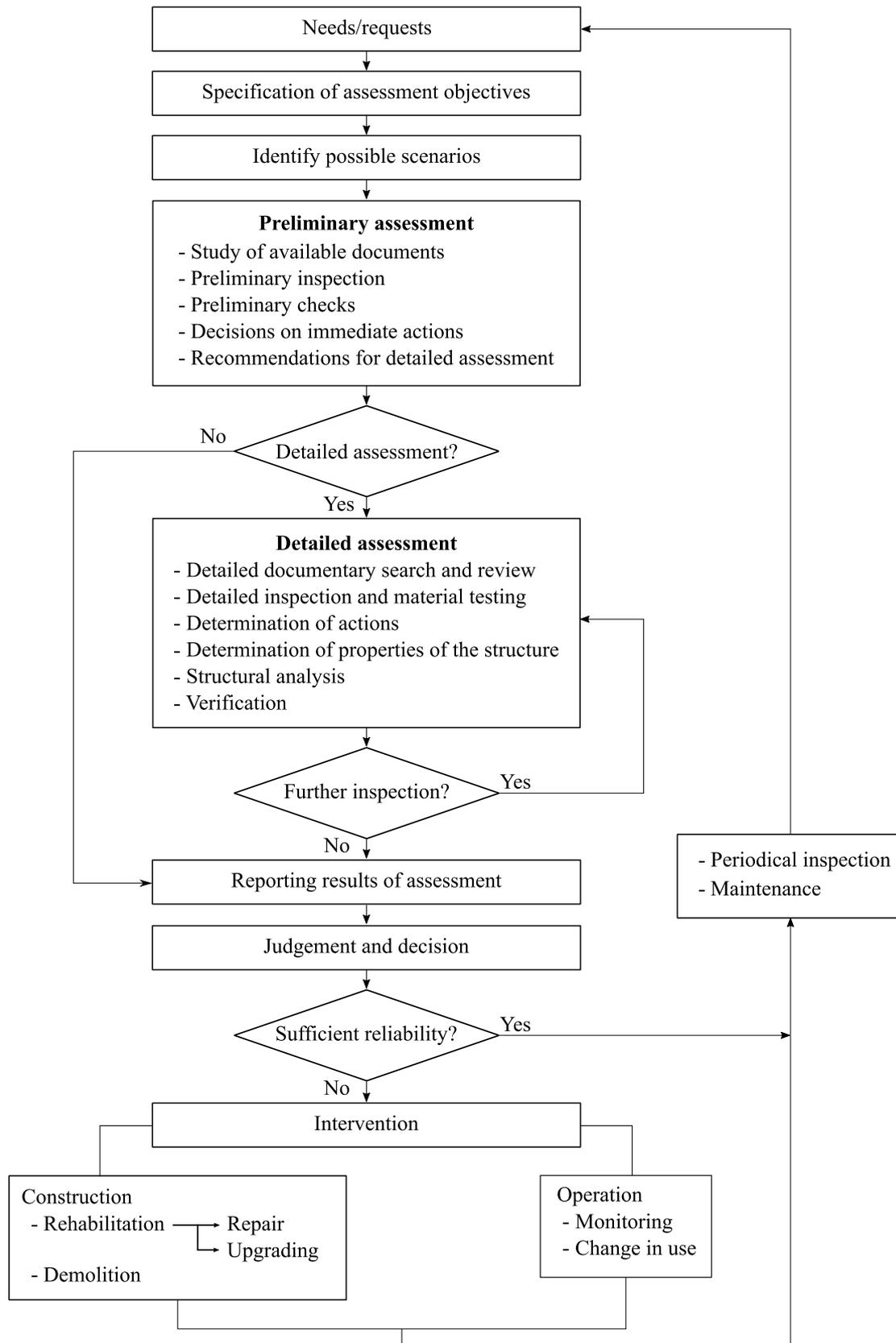


Figure 3: Flow chart for structural safety assessments (ISO13822, 2001).

Paper II and Paper III mainly focus on expanding the knowledge within the assessment process entitled “structural analysis”, while Paper I also comprises the “verification” process.

As the bond capacity is one of the largest concerns in concrete structures with corroding reinforcement, providing assessment engineers with better numerical tools to analyse the anchorage in structures might influence the results of many structural assessments. It can for example lead to another outcome when verifying the reliability level, compared to more conventional calculation models; consequently also the need for, and extent of, interventions.

2.1 Safety formats for structural analysis

If the above procedure for reliability assessments is to be followed, then this may necessitate an advanced structural analysis. NLFE analyses generally aim to represent the real behaviour of structures. The mean values of the model input parameters are thus normally used. This means that appropriate safety formats must be used to ensure the structures fulfil their intended safety level. The general design condition used is:

$$F_d \leq R_d, \quad R_d = \frac{R_m}{\gamma_R^* \gamma_{Rd}} \quad (1,2)$$

where F_d is the design value of actions, R_d is the design resistance, R_m is the mean resistance, γ_R^* is the global safety factor and γ_{Rd} is the modelling uncertainty factor.

There are several safety formats described in *fib* Model Code 2010 (CEB, 2013) which can be used to evaluate R_d . The purpose of a safety format is to account for the uncertainties in the basic variables and yield a design resistance that is consistent with the chosen safety level.

Paper I gives the safety formats in *fib* Model Code 2010 and applies them to an indeterminate structure (a concrete frame). This is to investigate whether the intended safety level was obtained. Two loads were applied to the frame: one vertical point load at mid-span and one horizontal load, in the upper left corner. Two load histories were considered: the first had the vertical load applied first, followed by the horizontal (denoted “main load history”); the second had the horizontal load applied first, followed by the vertical (denoted “inverse load history”).

Figure 4 and Figure 5 show the deformed shape and crack pattern for the frame with both loads applied to their characteristic levels for the main and inverse load history, respectively.

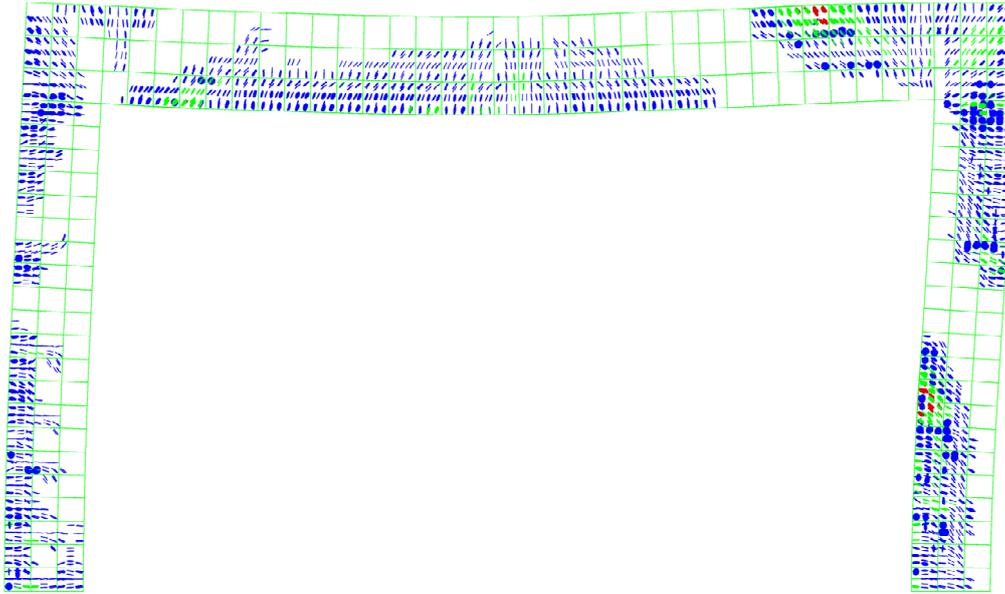


Figure 4: Deformed shape and crack pattern for the frame loaded with both loads to their characteristic level, as per the main load history. Adopted from Paper I.

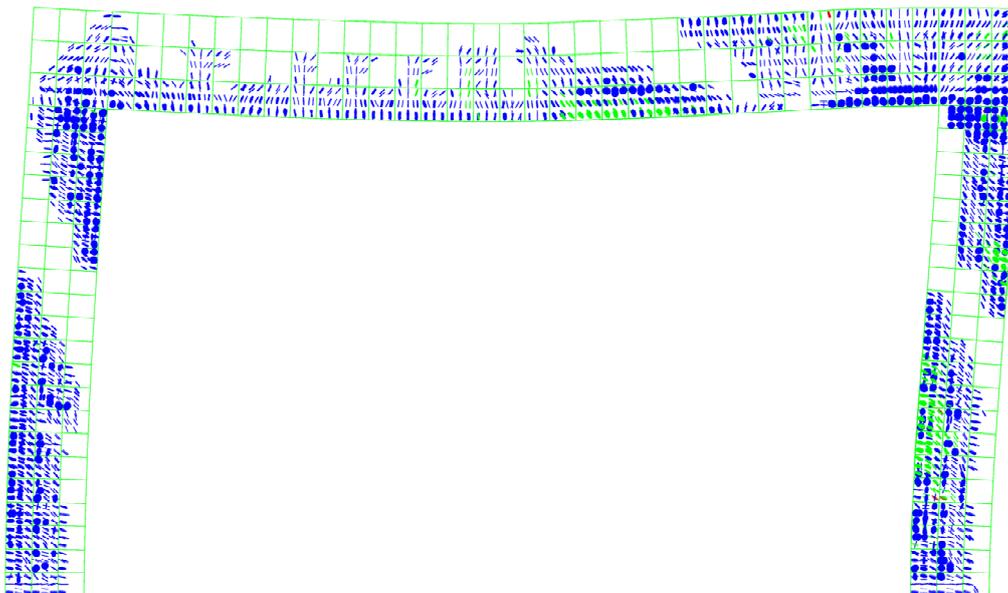


Figure 5: Deformed shape and crack pattern for the frame loaded with both loads to their characteristic level, as per the inverse load history. Adopted from Paper I.

The procedure for determining the safety levels given in the safety formats meant using response surfaces to approximate the limit state functions. The reliability indices were calculated by first and second-order reliability methods. See *Paper I* for a more detailed presentation of this procedure. One of the studied safety formats did not meet the intended safety level for one of its load histories. This highlights the importance of load history in structural analyses and verifications using NLFE analyses.

In conducting this work, it was evident that only limited information is provided in *fib* Model Code 2010 as to how structural resistance should be defined when used in these safety formats. It depends on the current loading situation of the structure, but

resistance is also affected by prior load history. The load situation considered here was simple. Nevertheless, determining the measure of structural resistance to use for the safety formats was no trivial matter. It was determined according to the sum of the applied loads, but other quantifications could be used. If the analyses were to involve time-dependent aspects or cyclic loading, the definition of resistance would become even more complex.

3 Bond strength of corroded reinforcement

The bond between reinforcement and the surrounding concrete is a key feature in reinforced concrete structures. The bond properties can influence both the ultimate limit state (ULS) behaviour of the structure, its ultimate capacity for example, and the performance in serviceability limit state (SLS), for example the deflections under service loading. Bond strength is influenced by corrosion of the steel bars; this will be further examined in this section.

Firstly, there is a brief presentation of different types of bond tests. There then follows a presentation of a database compiled from a large number of bond tests on corroded specimens (found in literature). Finally, there is a description of the variability in test results.

3.1 Types of bond tests

Bond tests can, in principle, be divided into two categories: tests to evaluate local bond capacity and tests of anchorage capacity (Mancini & Tondolo, 2014). The former have a short embedment length l_b (usually limited to about five times the main bar diameter). Reinforcement bars are cast into concrete and then tested by pulling the bars. Depending on the test setup the surrounding concrete is either in compression, as in the RILEM pull-out test (RILEM, 1994b), or in tension as in (Auyeung *et al.*, 2000), shown in Figure 6 and Figure 7 respectively.

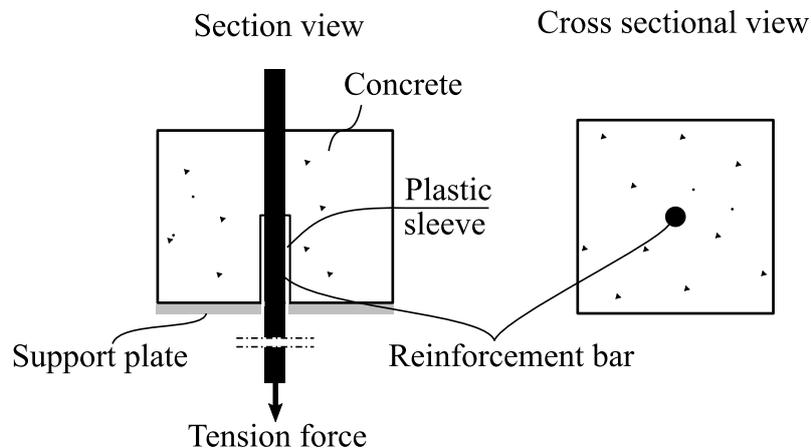


Figure 6: Pull-out test with surrounding concrete in compression, adopted from (RILEM, 1994b).

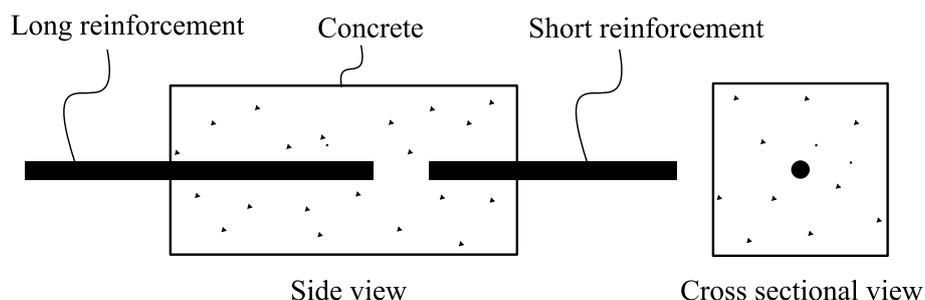


Figure 7: Pull-out test with surrounding concrete in tension, adopted from (Auyeung *et al.*, 2000).

Moreover, the stress state in the surrounding concrete can be important, for example when studying crack widths in the serviceability limit state (Mancini & Tondolo, 2014). Longer embedment lengths are used for testing the anchorage capacity: l_b is ten times the bar diameter for RILEM beam tests (RILEM, 1994a). The stress level in the steel bars is often close to, but should not reach, the point of yielding. Bond strength is determined as an average over the embedment length. This type of test is sometimes also referred to as an eccentric pull-out test or beam-end test and can be carried out using modified test setups. See Figure 8 for an example of a beam-end test by (Zandi *et al.*, 2011).

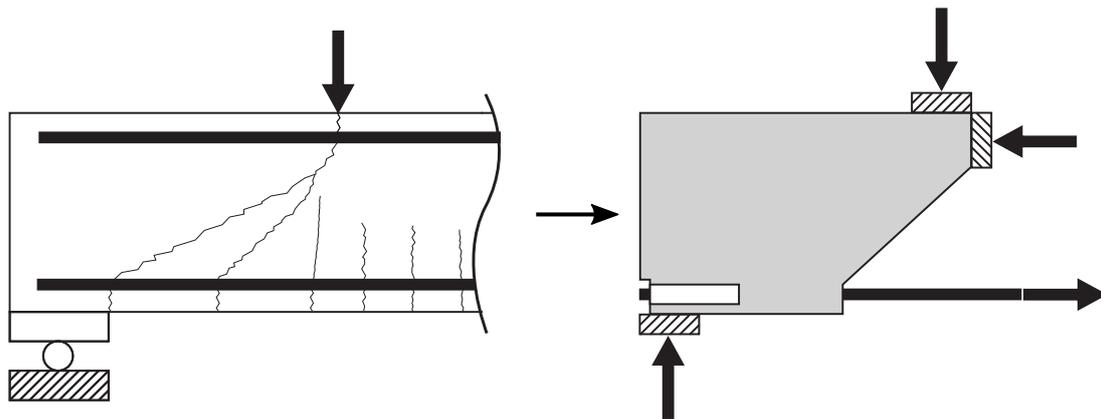


Figure 8: Structural simplification of a beam-end test, from (Zandi *et al.*, 2011).

There are also bond tests of full-length beams with the reinforcement bars bonded only at the embedment lengths in the support regions, see (Al-Sulaimani *et al.*, 1990) for example. Vertical loading is applied to the beam and the slip of the reinforcement bars is measured as free-end slip, i.e. the slip of the bar at the ends of the beam.

Efforts have been made to find a harmonised European bond test (Cairns & Plizzari, 2003). However, it was demonstrated that a single test for bond would not be adequate. A system of bond testing is required to measure each aspect of structural performance.

3.2 Database of bond tests of corroded specimens

The present work used a compilation of 500 bond tests of specimens with corroded reinforcement to calibrate a calculation model for bond, as described in Section 4.2.2. The database includes results from several types of test setups, including pull-out and beam tests reported in 21 research works (Almusallam *et al.*, 1996; Al-Sulaimani *et al.*, 1990; Auyeung *et al.*, 2000; Cabrera & Ghoddoussi, 1992; Zandi & Coronelli, 2010; Coronelli, 1998; Fang *et al.*, 2004; Fischer & Ozbolt, 2012; Lee *et al.*, 2002; Lin & Zhao, 2016; Mangat & Elgarf, 1999; Coccia *et al.*, 2016; Rodriguez *et al.*, 1994; Rodriguez *et al.*, 1994; Rodriguez *et al.*, 1996; Shima, 2001; Stanish *et al.*, 1999; Horrigmoe *et al.*, 2007; Law *et al.*, 2011; Ng *et al.*, 2002; Ghandehari *et al.*, 2000).

The database includes information on the test setup including bar diameter ϕ_m , concrete cover c , embedment length l_b , stirrup content $A_{sr}/(s_t \cdot \phi_m)$, yield strength of stirrups f_{yr} , concrete compressive strength f_{cm} , current density used in accelerated corrosion process ν and corrosion level W_c . The results of the bond tests are listed as

the absolute bond strength $\tau_{DB,abs}$. Typically, this is calculated as the anchored force divided by the surface area of the bar over the embedment length. The relative bond strength $\tau_{DB,rel}$ is also included. This is defined as the ratio between absolute bond strength of the corroded and uncorroded (reference) test.

The current densities used for the electrochemical corrosion vary between different researchers' test setups. This property may influence bond capacity. Furthermore, the embedment lengths also vary between the tests. For shorter embedment lengths, the absolute bond strength appears as the local bond strength. For longer lengths, this no longer holds true and the results consist instead of the average bond stress at maximum load. This should be kept in mind when using the test results.

3.3 Variation in bond test results

Many parameters influence the bond between concrete and reinforcement. No fewer than ten parameters are used in the calculation model for bond strength of uncorroded specimens provided in *fib* Model Code 2010 (CEB, 2013). Bond test results are typically subjected to considerable scatter, due to the natural variability of the influencing parameters and the difference in test setups.

The results presented by (Berrocal *et al.*, 2017) are used as an example to quantify the variations among test results. Pull-out tests conducted on identical uncorroded specimens resulted in splitting failures, with a variation in of 55% in maximum bond strength for cases where $c/\phi_m = 2.5$ and 19% for cases where $c/\phi_m = 4.0$. Given that the controllable parameters for these tests were kept the same, this variation is quite high.

The scatter among the bond test results of corroded specimens was quantified by sorting the test specimens into groups based on the confinement and corrosion level. For each group the coefficient of variation of the relative bond strength was calculated, with respect to the uncorroded case.

The confinement was quantified using a well-acknowledged empirical bond model (Orangun *et al.*, 1977) and called "bond index" here. It includes the main parameters of confinement and can be written as:

$$I_A = 0.1 + 0.25 \cdot \frac{c}{\phi_m} + 4.15 \cdot \frac{\phi_m}{l_b} + 0.024 \cdot \frac{A_{st}}{s_t \cdot \phi_m} \quad (3)$$

where c is concrete cover, ϕ_m bar diameter, l_b embedment length and $A_{st}/(s_t \cdot \phi_m)$ the amount of transverse reinforcement.

The bond index and corrosion intervals for the groups were selected so as to have a similar number of tests in each group. This resulted in a division of the bond indices into four groups and the corrosion levels into five. The chosen bond index groups were 0-1.0, 1.0-1.5, 1.5-2.0 and 2.0-3.0, and the corrosion level groups were 0-1.5%, 1.5-3.0%, 3.0-4.5%, 4.5-10% and 10-%. Uncorroded cases were filtered out, leaving a total of 460 bond test results distributed among 20 (4x5) groups. The number of bond tests in each group is shown in Figure 9. It can be seen that the number of bond tests is more or less equal between most groups, although some groups have relatively few.

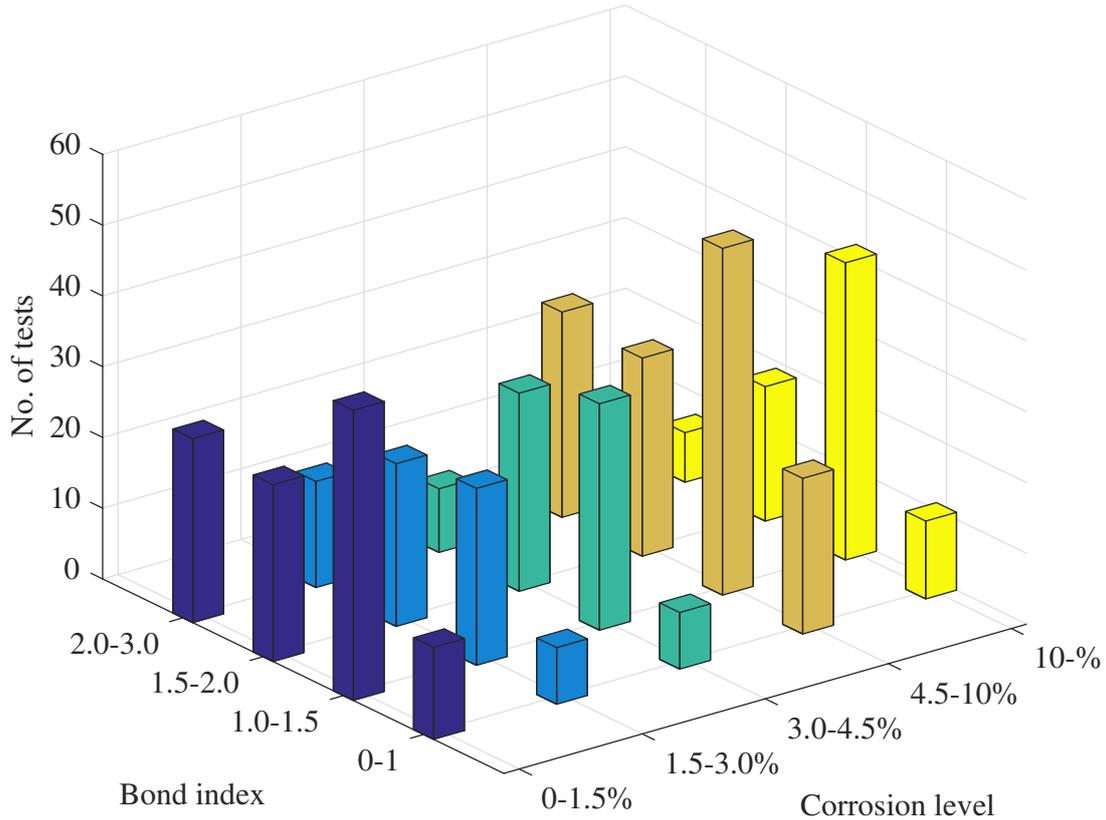


Figure 9: The number of test results in each group is shown as vertical bars, with bond index and corrosion level on the horizontal axes. Adopted from paper III.

The maximum corrosion levels in tests are often related to the experimental time needed, since a suitably slow corrosion speed must also be used for high corrosion levels. It is noteworthy that for low corrosion levels, the inherent variation in weight of a steel bar from production can be important. In other words, if reinforcement bars are not weighed before corrosion is applied (a common procedure in experiments) and nominal values are used instead.

For each of the 20 groups, the coefficient of variation (CoV) of the relative bond strengths was used to measure scatter. It was calculated as the standard deviation in the groups divided by the mean value of the relative bond strengths, see Equations 4-6:

$$c_v = \frac{\sigma}{\mu} \quad (4)$$

$$\sigma = \sqrt{\frac{1}{N-1} \sum_{i=1}^N |\tau_{DB,rel,i} - \mu|^2} \quad (5)$$

$$\mu = \frac{1}{N} \sum_{i=1}^N \tau_{DB,rel,i} \quad (6)$$

where N is the total number of tests in the considered group and $\tau_{DB,rel,i}$ is the relative bond strength in test i .

Figure 10 shows a contour plot of the CoV for the groups, with intermediate values interpolated. The highest variation can be seen in the group with a bond index of 2.0-3.0 and a corrosion level of 4.5-10%. The variation in this group is reasonable, since confinement from concrete and stirrups both contribute to the bond index. If a thick concrete cover makes up the main part of the confinement, with only minor

contribution from stirrups, much of the bond strength can be lost for corrosion levels exceeding the level required to crack the concrete cover. On the other hand, in cases where stirrups are the main source of confinement, a larger capacity can be expected after corrosion-induced cracking. Furthermore, for cases with either a low bond index (0-1.0) or a low corrosion level (0-1.5%), the coefficient of variation is lower. This can be explained by a lesser effect from cracking on the bond capacity, as opposed to the highly confined cases mentioned previously.

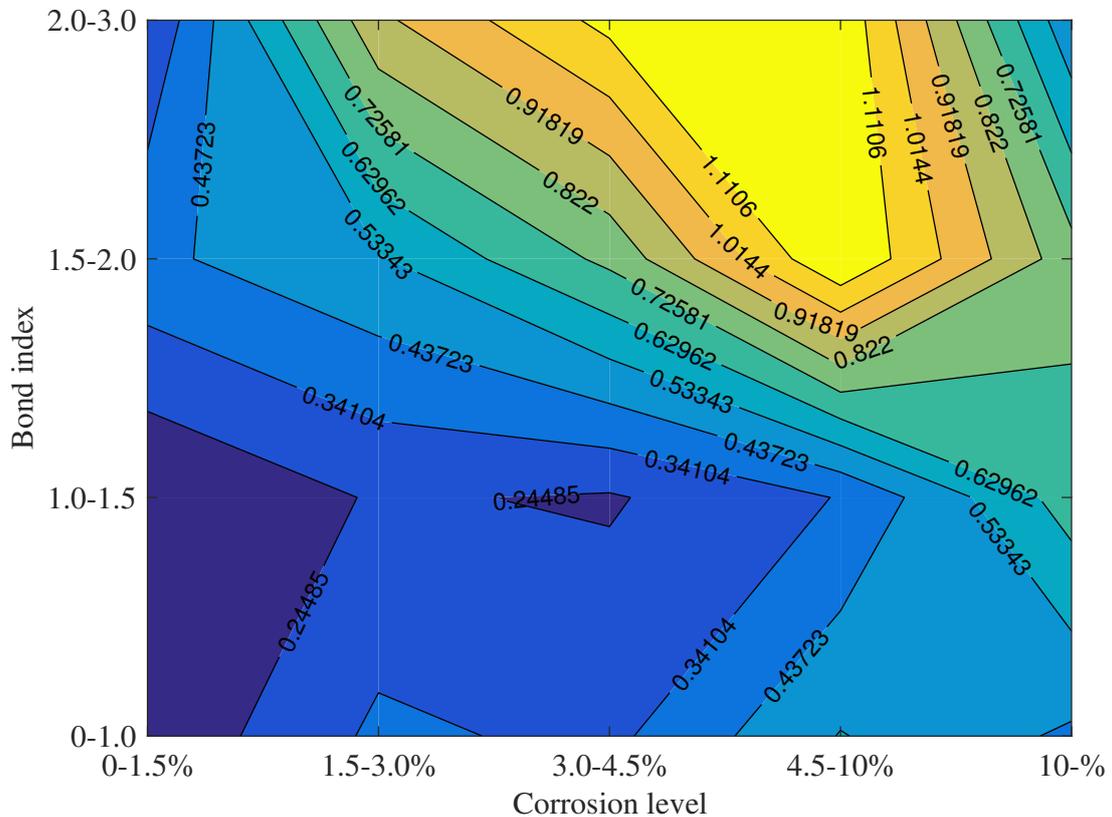


Figure 10: Contour plot of the coefficient of variation for the relative bond strength in the database for varying bond index and corrosion level. Adopted from Paper III.

Bond tests of corroded reinforcement in concrete typically show quite a large degree of scatter. This can also be seen here, with a CoV of up to 110%. This should be born in mind when validating the ARC2010 model in Section 4.2.3.

4 Modelling of anchorage in concrete structures with corroding reinforcement

The previous chapter explained that many parameters influence bond strength, all of them subject to some level of uncertainty. This also causes the results of bond strength tests to be rather scattered. Nevertheless, several suggestions have been made on how to represent the influence of corrosion on bond in numerical models of structural capacity. Cf. (Coronelli *et al.*, 2013; Lundgren, 2005b; Berra *et al.*, 2003) who use 3D NLFE analyses and (Prieto *et al.*, 2016; Castel *et al.*, 2016; Lundgren *et al.*, 2009) who use more simple approaches. This chapter presents different levels of detail for modelling anchorage as well as suitable bond models. An example showing application of the most detailed modelling level is given in *Paper II*. A proposed bond model is also given and verified against bond test data, based in work detailed in *Paper III*.

4.1 Four levels of detail in anchorage assessment

If the real physical behaviour of a structure is to be represented, this calls for complex analysis procedures. Costs are high in terms of both knowledge and the time needed to set up and run the analysis. However, this level of accuracy is not always preferable. In some cases, particularly in engineering applications, it is better to set the level of detail so that the desired aspect of the behaviour is captured sufficiently accurately but at lower resulting cost.

An approach based on the principle of successively improved evaluation at various structural assessment levels has been proposed by (Tahershamsi *et al.*, 2016). These assessments are divided into four categories based on their levels of detail, with level I the least detailed and level IV the most detailed. Figure 11 depicts the assessment levels.

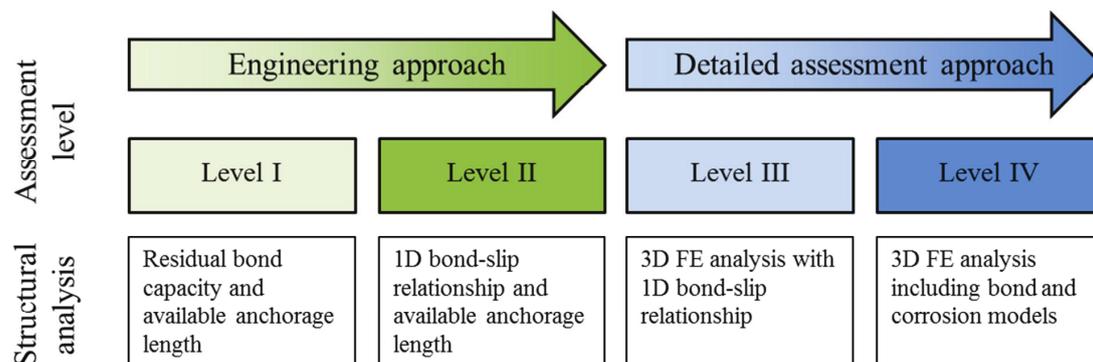


Figure 11: Description of assessment levels I to IV for assessing anchorage capacity in reinforced concrete with corroded reinforcement. Proposed by, and adopted from (Tahershamsi *et al.*, 2016).

Assessment levels I and II are considered reasonable for engineering applications, since they are one-dimensional (1D) approaches and do not require NLFE analyses. Level I assessments consider only the residual capacity given by the local bond stress-slip relationship over an assumed anchorage length. In the level II approach, the anchorage capacity is calculated using the entire local bond stress-slip relationship, and by solving the 1D differential equation over the available anchorage length.

Assessment levels III and IV are considered detailed assessment approaches, applicable to research and in enhanced structural assessments where detailed analyses are deemed necessary. As both these levels require the use of NLFE analyses, the main difference concerns how the interaction between reinforcement bars and concrete is modelled. At level III, the reinforcement is modelled by 1D elements and assigned an appropriate local bond stress-slip relationship. However, at level IV the reinforcement is modelled by 3D solid elements and the interaction between reinforcement and concrete is explicitly described by bond and corrosion models in the interface layer.

The bond models for levels I to IV are presented in the following. This thesis includes work on assessment levels II and IV; these are shown in more detail in Sections 4.1.3 and 4.1.5 respectively.

4.1.1 1D local bond stress-slip relationship for corroded reinforcement

As mentioned above, in assessment levels I to III the bond between concrete and reinforcement is described by a 1D local bond stress-slip relationship. The analysis procedures are the same independent of the level of corrosion, this since the local bond stress-slip relationships used should represent the corrosion level. That is, the corrosion is given implicitly as input for the analyses on assessment levels I-III. In this section the model for representing corrosion in a 1D local bond stress-slip relationship, which formed the basis for recent developments in the present work, is presented.

The procedure for representing corrosion using modified bond stress-slip curves was established by (Lundgren *et al.*, 2009), based on the findings of (Schlune, 2006; San Roman, 2006). This will be discussed only briefly here; for the full description, please refer to the referenced paper.

The model, denoted ARC1990, is based on Model Code 1990 (CEB, 1993), with interpolation between the “unconfined” and “confined” cases. Corrosion is accounted for by:

- Excluding any confining effects from the concrete cover for corrosion levels estimated to cause cracking.
- Applying an equivalent slip of the curve, and choosing the lesser of the uncorroded curve and shifted curve.

This is based on the argument that corrosion exhausts the bond strength in similar fashion to mechanical slip. See Figure 12 for an illustration of the shifting procedure.

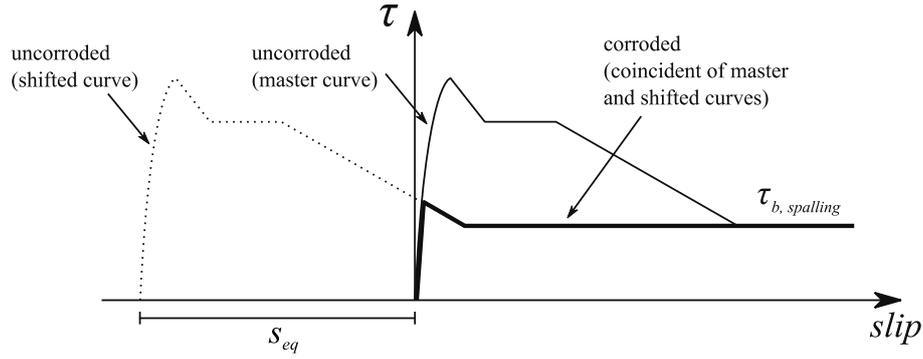


Figure 12: Illustration of procedure of shifting local bond stress-slip curve to account for corrosion. The curve is modified by (Tahershamsi, 2016) based on work by (Zandi, 2015).

The resulting 1D bond stress-slip relationship can be used when assessing anchorage on levels I to III.

4.1.2 Level I assessment of anchorage

At level I, the most simplified procedure for analysing anchorage, only the residual part of the bond stress-slip curve is considered. Figure 12 shows the residual bond stress, denoted $\tau_{b,spalling}$. The anchorage force is calculated by simply multiplying the residual bond stress by the bar circumference and anchorage length:

$$F_a = \tau_{b,spalling} \cdot \pi \cdot \phi_m \cdot l_b \quad (7)$$

where ϕ_m is the reinforcement diameter, and l_b the anchorage length. The main advantage of this assessment method is that it can be easily understood and quickly performed by hand calculation. It also yields results which lie on the safe side, making it suitable as an initial check of anchorage capacity (Tahershamsi *et al.*, 2016). If the capacity is proved insufficient using this approach, it may be meaningful to conduct a higher-level assessment.

4.1.3 Level II assessment of anchorage

Level II uses full bond stress-slip relationship. The anchorage force is solved by studying the equilibrium conditions along the reinforcement bar. This is described by the differential equation (Lundgren *et al.*, 2009):

$$\frac{\pi \cdot \phi_m^2}{4} \cdot \frac{d\sigma_s}{dx} - \pi \cdot \phi_m \cdot \tau_b = 0 \quad (8)$$

where ϕ_m is the reinforcement diameter, σ_s is the stress in the reinforcement and τ_b is the bond stress. Since the reinforcement bar along the anchorage length is assumed to be in the elastic range, its stress and strain can be expressed as:

$$\sigma_s = E_s \varepsilon_s, \quad \varepsilon_s = \frac{du}{dx} \quad (9,10)$$

where E_s is the elastic modulus, ε_s is the strain and u denotes the displacement of the reinforcement bar. The bond stress τ_b is defined by the local bond stress-slip relationship; for uncorroded cases this can be obtained in e. g. *fib* Model Code 2010 (CEB, 2013). For a corroded bar, modified bond stress-slip curves can be used, see for

example Section 4.1.1. If the deformation of the surrounding concrete is neglected, the slip, s , equals the displacement of the rebar:

$$u = s \quad (11)$$

The boundary conditions are for pull-out of a reinforcement bar with embedment length l_b and prescribed displacement u_{l_b} :

$$\sigma_s(0) = 0, \quad u(l_b) = u_{l_b} \quad (12,13)$$

The differential equation can be solved numerically to obtain the anchorage force and end-slip behaviour. Analysis under level II is deemed suitable for practical application and a logical step if an assessment under level I fails to yield the desired outcome

4.1.4 Level III assessment of anchorage

In assessments on level III NLFE models must be established. The models can be in either 1D, 2D or 3D and the corroded reinforcement bars are modelled using 1D embedded elements, with the associated bond-slip behaviour described via a 1D bond stress-slip curve for corroded reinforcement, see e.g. Section 4.1.1.

This assessment level can be fairly similar to level IV but also rather different, depending on the FE model used for the level III analyses. The main differences between assessment levels III and IV, with respect to the model, included aspects and analysis outcomes, can be summarised as:

- **Model:** Level III is flexible in regard to model choice; 1D, 2D and 3D analyses are admissible, while level IV requires 3D analyses with solid elements.
- **Included aspects:** in level III analyses, it is not possible to factor in the influence on bond capacity of transverse pressure or flow of corrosion products through cracks.
- **Outcome:** in level III analyses, it is not possible to obtain the crack pattern induced by corrosion. Depending on the chosen model, it may or may not be possible to obtain other outcomes from the analyses, such as the crack pattern from bond action.

4.1.5 Level IV assessment of anchorage

Level IV assessments of anchorage require the use of 3D NLFE analyses where solid elements are used for reinforcement and concrete. The interaction between the two materials is explicitly described by bond and corrosion models in the interface layer.

The model for bond and corrosion used in the 3D NLFE analyses in *Paper II*, was originally formulated in (Lundgren & Gylltoft, 2000) and further developed in (Lundgren, 2005a; Lundgren, 2005b). It describes the volumetric expansion of a rebar with the associated normal stresses when steel turns into rust, plus the normal and bond stresses arising when a corroded bar is pulled. In the NLFE model, this is implemented via 2D interface elements between the reinforcement and concrete. The graphics in Figure 13 illustrate important parameters in the model.

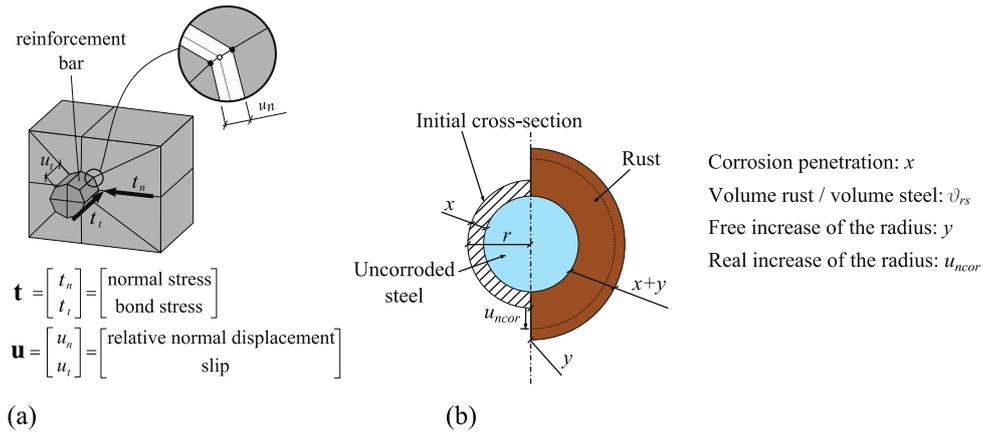


Figure 13: (a) shows stress and displacement components in the bond model from (Lundgren, 2005a), while (b) shows the variables in the corrosion model from (Lundgren, 2005b).

The bond behaviour is described by a frictional model and elastic-plastic theory represents the relationships between stresses and deformations. In the elastic range, the relationship between the relative displacements (u_n and u_t) and the tractions (t_n and t_t) is given as:

$$\begin{bmatrix} t_n \\ t_t \end{bmatrix} = \begin{bmatrix} D_{11} & 0 \\ 0 & D_{22} \end{bmatrix} \begin{bmatrix} u_n \\ u_t \end{bmatrix} \quad (14)$$

where D_{11} and D_{22} are the stiffness in normal and slip directions respectively.

The limits of the elastic domain are defined by two yield functions, one describing friction F_1 and the other pull-out failure F_2 . The first is described by a frictional coefficient, while the latter is obtained from the stress in the inclined compressive struts resulting from the bond action. For the case with adhesion, the yield functions can be written:

$$F_1 = |t_t| + \mu(t_n - f_a) = 0 \quad (15)$$

$$F_2 = t_t^2 + (t_n + c_c)(t_n - f_a) = 0 \quad (16)$$

The graphics in Figure 14 illustrate the yield surface.

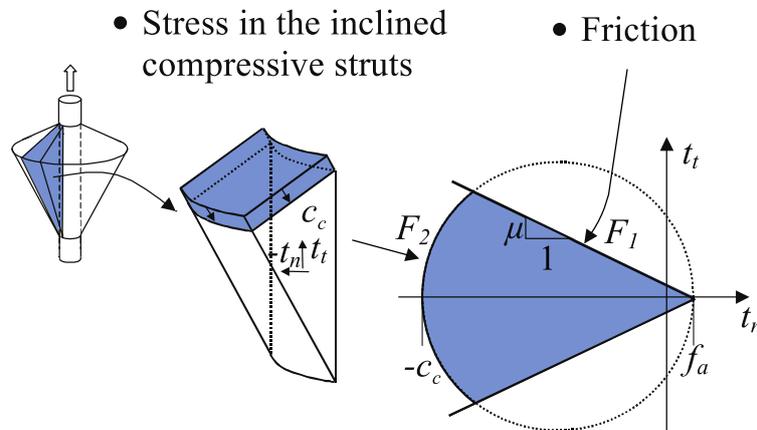


Figure 14: Yield surface for the model, composed of yield line F_1 for friction and yield line F_2 for pull-out. Modified from (Zandi, 2015; Jansson et al., 2012).

An associated flow rule is assumed for plastic loading along yield line F_2 . A non-associated flow rule is assumed for the frictional yield line F_1 , where the plastic part of the deformation is:

$$d\mathbf{u}^p = d\lambda \frac{\partial G}{\partial \mathbf{t}}, \quad G = \frac{|u_t|}{u_t} t_t + \eta t_n = 0 \quad (17)$$

To describe the hardening of the model, a hardening parameter κ is defined as:

$$d\kappa = \sqrt{du_n^{p2} + du_t^{p2}} \quad (18)$$

For the analyses conducted in *Paper II*, the input parameters for the model were chosen according to (Jansson *et al.*, 2012). These parameters include the frictional parameter μ and the stress compressive strut c_c ; both functions of hardening parameter κ . Also included are parameters describing the mechanical behaviour of rust and the ratio between rust and virgin steel.

Furthermore, material properties were chosen for the concrete, e.g. compressive and tensile behaviour and crack model, and the steel, e.g. yield hardening behaviour, see *Paper II* for details. The non-corroding stirrups were modelled as embedded 1D elements. Corrosion was modelled as a function of time and was thus applied in time steps. Spalling of the concrete cover can be included in level IV analyses if the FE mesh is updated based on evaluation of the spalling pattern; i.e. removing spalled concrete elements from the mesh (Zandi, 2015).

This assessment level constitutes the most detailed analyses of anchorage considered here and requires major resources, both in terms of analyst knowledge and time to set up and run the analysis. However, through this procedure it is possible not only to determine the anchorage capacity, but also study the crack patterns and spalling of concrete cover.

4.2 Improved bond model for assessment levels I to III

As stated above, the analytical 1D model for anchorage in concrete structures with corroded reinforcement shown in Section 4.1.1 has been verified against test results with natural corrosion and proved useful in practice. However, the case study (Lundgren *et al.*, 2015) also helped identify areas for improving the model. The main points for this improvement were found to be: i) incorporation of cross-sectional position of the studied bar, ii) influence of transverse reinforcement, iii) several layers of reinforcement and iv) closely spaced bars.

Points i) and ii) are addressed by implementing the *fib* Model Code 2010 (CEB, 2013) in the model. This enables specification of two-dimensional covers and direct calculation of a splitting strength based on the amount of transverse reinforcement. The implementation is briefly discussed in Section 4.2.1. More details can be found in *Paper III*.

The basis for addressing points iii) and iv) is discussed in Section 4.3 and elaborated in *Paper II*, in which a modelling strategy is given for analysing anchorage zones using 3D NLFE analyses of thin sections. In future works, a parametric study using the thin-section model will be used to study the effect of several reinforcement layers and closely-spaced bars.

4.2.1 Proposal for improved 1D bond model for corroded reinforcement

The proposed model for assessing Anchorage in corroded Reinforced Concrete structures, denoted ARC2010, is based on the local bond stress-slip relationship in *fib* Model Code 2010 (CEB, 2013). The following modifications and additions were made to the original model to account for the effect of corrosion:

- *Introduction of equivalent slip to account for bond degradation due to corrosion:* A function for the equivalent slip was established, depending on the corrosion level and the presence of stirrups.
- *Change of failure mode due to corrosion-induced cracking of the concrete cover:* At the point of corrosion cracking the concrete cover, the local bond stress-slip relationship is changed; the confinement originating from stirrups remains while the contribution from concrete cover is reduced.
- *Modification of residual bond stress in case of low stirrup content:* The lower boundary, i.e. for cases without stirrups, was set at 16% of the reduced splitting strength (reduced contribution from concrete cover). The upper boundary of the residual bond stress is reached when stirrup content, K_{tr} , is greater than 0.02, and was set at 40% of the reduced splitting strength. These modifications were made based on comparison with test results in the literature.

For a more detailed presentation of the proposed model the reader is referred to *Paper III*.

4.2.2 Calibration of the proposed model

In this section an overview of the calibration of the equivalent slip is provided, a detailed presentation of the calibration is found in *Paper III*.

The database of bond test results, shown in Section 3.2 was used to calibrate the proposed bond model. The calibration was conducted in two steps:

- Finding equivalent slips for the proposed model to yield relative average bond strength similar to the database (an inverse procedure was adopted). The relative average bond strength was calculated as the average bond strength for a corroded case divided by the average bond strength in the corresponding uncorroded case.
- Performing non-linear regression to fit a function to the equivalent slips.

The two factors that clearly influenced the equivalent slip were the presence of stirrups and the corrosion level; the equivalent slip is generally larger in cases with stirrups and increases with corrosion level. Therefore two different functions for the equivalent slip were calibrated, one for cases without stirrups and one for those with them.

These functions are plotted in Figure 15 with the data points used for the regression. The function for equivalent slip in the ARC1990 model, shown in Section 4.1.1, is 8.1 times the corrosion penetration in mm. The linear function was calibrated using NLFE analyses (Schlune, 2006; San Roman, 2006) and is also included in Figure 15 for

$\phi_m = 10 \text{ mm}$ and $\phi_m = 25 \text{ mm}$ reinforcement bars. Evidently, it would be overly conservative to use this expression for the ARC2010 model, especially for cases with stirrups.

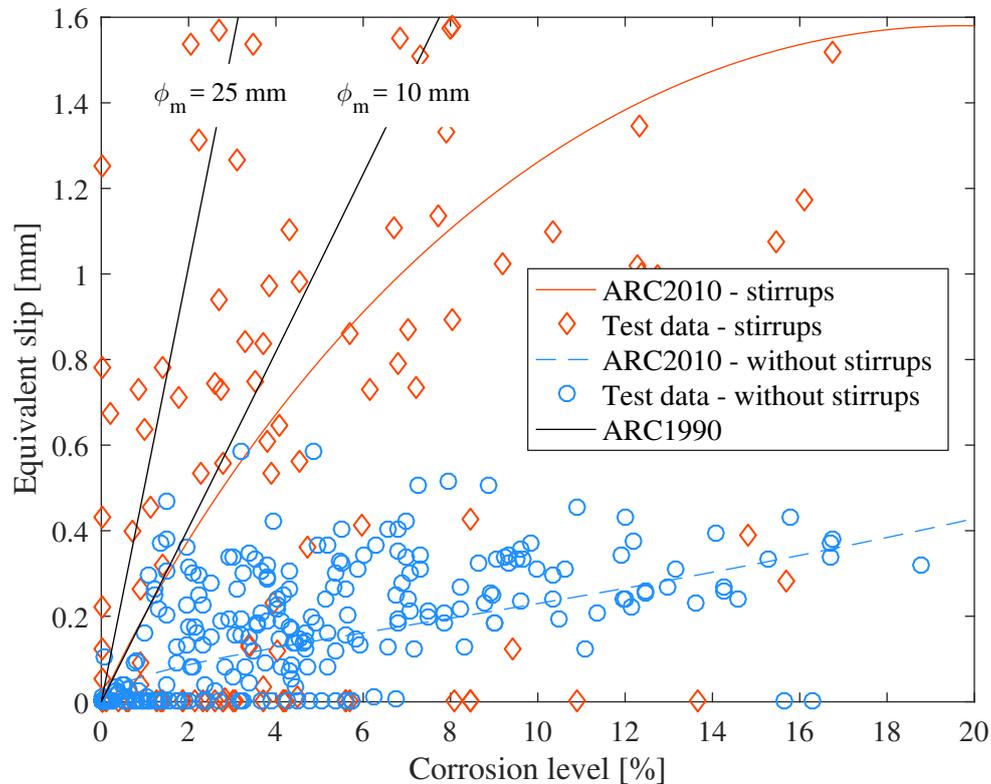


Figure 15: Presentations of the expressions of equivalent slips for ARC2010 and ARC1990, plus test data. Adopted from Paper III.

4.2.3 Validation of the proposed model

This section presents an overview of the verification of the proposed ARC2010 bond model. It includes a comparison between ARC2010 with the calibrated functions for the equivalent slips and the bond tests values in the database, and also a comparison to an empirical model by (Castel *et al.*, 2016). The verification of ARC2010 is presented in detail in Paper III.

The relative average bond strengths from the database are plotted against those obtained from the ARC2010 for cases with stirrups in Figure 16. It is noteworthy that although there is considerable scatter, the points are almost equally distributed around the diagonal line. The diagonal line represents full agreement between ARC2010 and the database.

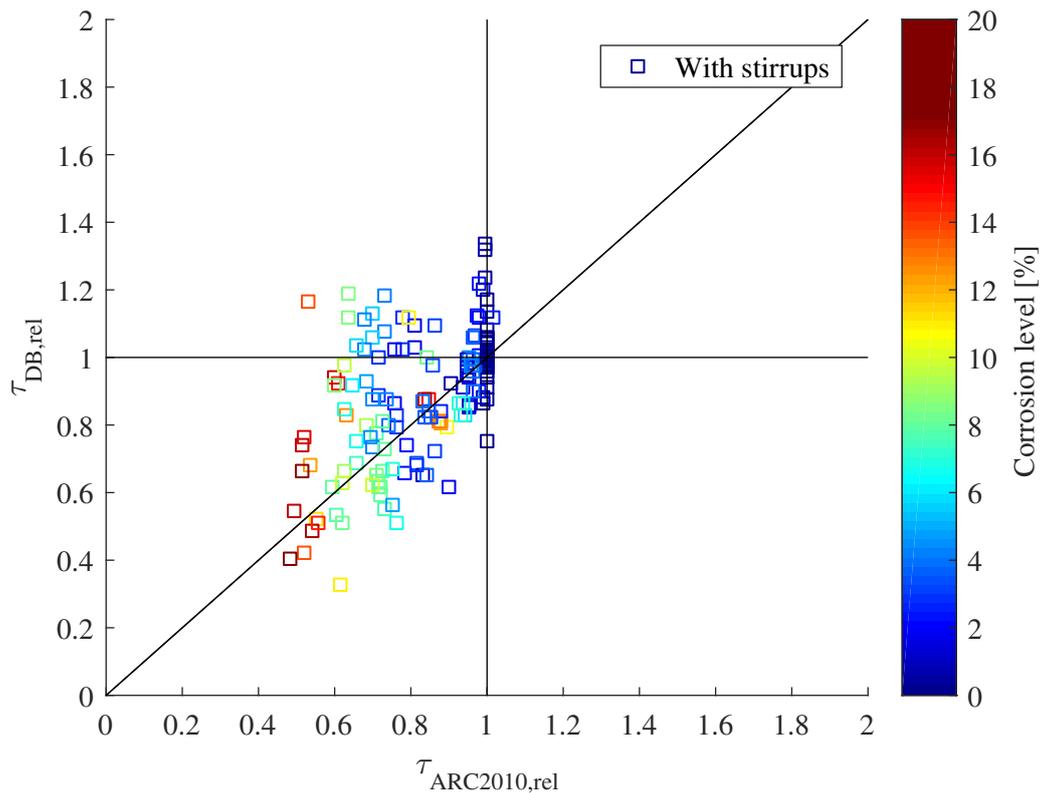


Figure 16: Relative average bond strength from database versus ARC2010 model for specimens with stirrups. The diagonal line corresponds to full agreement. Adopted from Paper III.

An empirical model for the bond strength of corroded reinforcement has been proposed by (Castel *et al.*, 2016). The ARC2010 model was compared to this model, in terms of relative bond strength, for several corrosion levels and stirrup contents. Figure 17 presents a comparison for a case with $f_{ck} = 50 \text{ MPa}$, $\phi_m = 16 \text{ mm}$, without stirrups and for three different concrete covers ($c = 40, 60$ and 80 mm). Figure 18 shows a comparison for a similar case, but the stirrup content is $\phi_t = 8 \text{ mm}$ with a spacing of $s_t = 150 \text{ mm}$.

The relative bond strengths obtained from the two models appear to agree well, and ARC2010 represents the physical behaviour well. Cracking of the concrete cover leads to a marked decrease in relative bond strength. With increased corrosion, the reduction springs from an increased equivalent slip. When the applied equivalent slip is sufficiently large, only the residual bond strength remains.

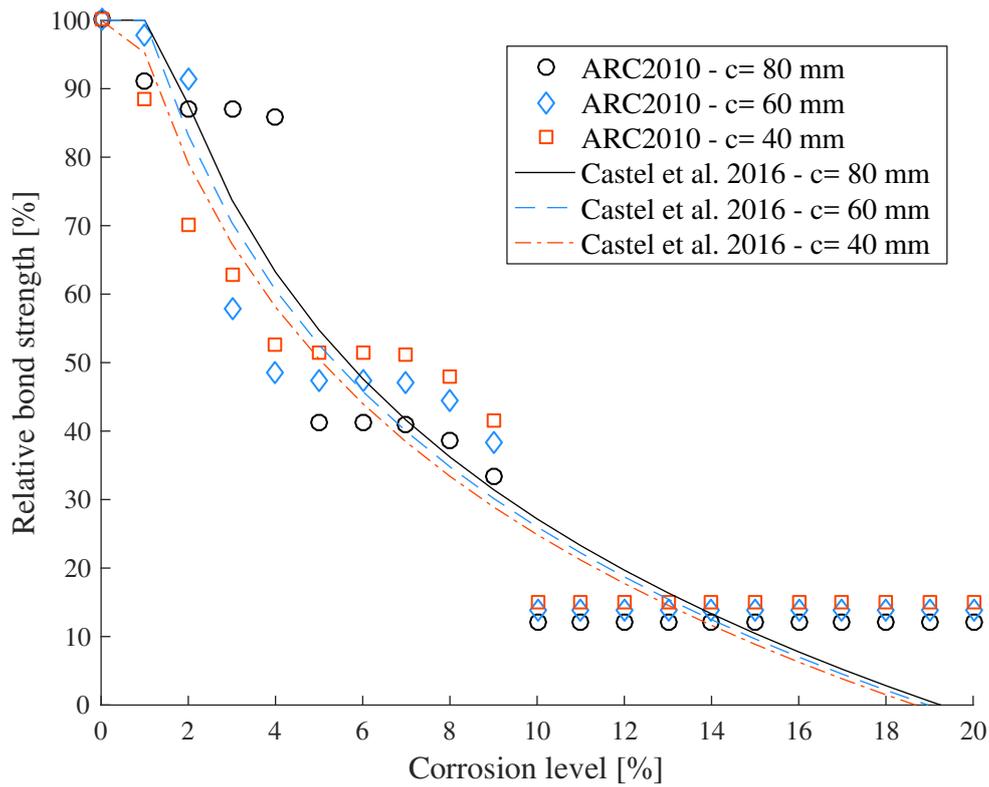


Figure 17: Comparison of Castel et al. 2016 and ARC2010 for $f_{ck} = 50 \text{ MPa}$, $\phi_m = 16 \text{ mm}$, and without stirrups. Adopted from Paper III.

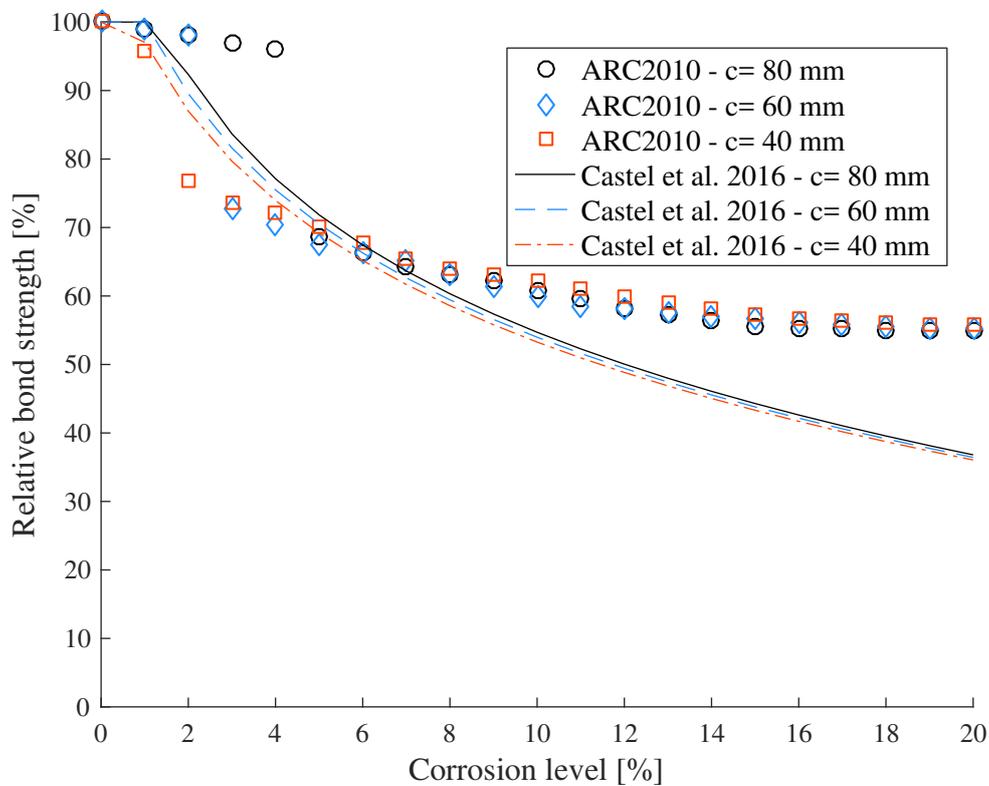


Figure 18: Comparison of Castel et al. 2016 and ARC2010 for $f_{ck} = 50 \text{ MPa}$, $\phi_m = 16 \text{ mm}$, $\phi_t = 8 \text{ mm}$ and $s_t = 150 \text{ mm}$. Adopted from Paper III.

4.3 Modelling strategy for section analyses

A parametric study using FE analyses on assessment level IV will be conducted. It will study the influence of reinforcement in several layers and the effect of spacing between bars on bond capacity. Since this study means analysing many different geometries (and thus FE models), there is need for a practical yet reasonable simplification of the geometry. *Paper II* investigates the possibility of constructing simple 3D NLFE models which can still represent the anchorage region of a beam.

The idea of modelling a thin section is a simplification based on extracting a section within an activated anchorage region along its embedment length. For a beam, this is a section at the position of the last shear crack. See Figure 19 for an illustration.

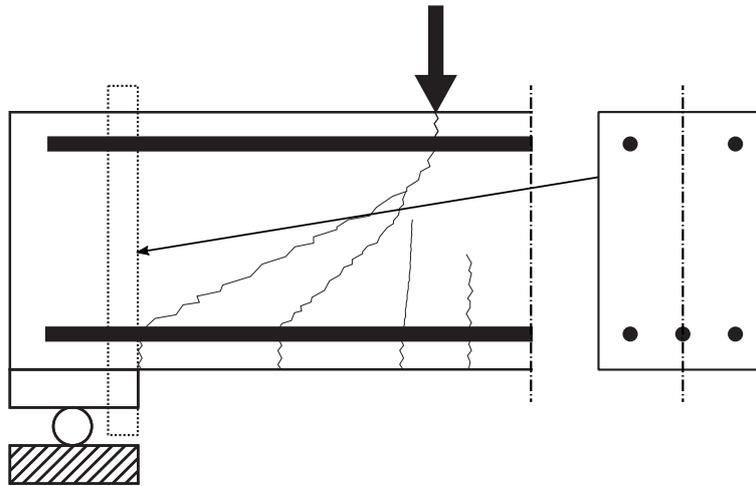


Figure 19: Illustration of thin-section model, as extracted from the anchorage region in a beam, modified from (Zandi *et al.*, 2011).

4.3.1 Description of test setup

Previously conducted physical tests and detailed 3D NLFE analyses of beam-end specimens by (Zandi *et al.*, 2011) were used for comparison with the analysis results from thin sections. Figure 20 illustrates the geometry. The anchorage capacity of three reinforcement bars was examined in the test set-up (shown in Figure 8). Load was applied by pulling either the two main bars at the corners, or the main bar in the middle of the cross-section, and the relative displacement was measured. Cf. (Zandi *et al.*, 2011) for information regarding the properties of the specimens and the test procedure.

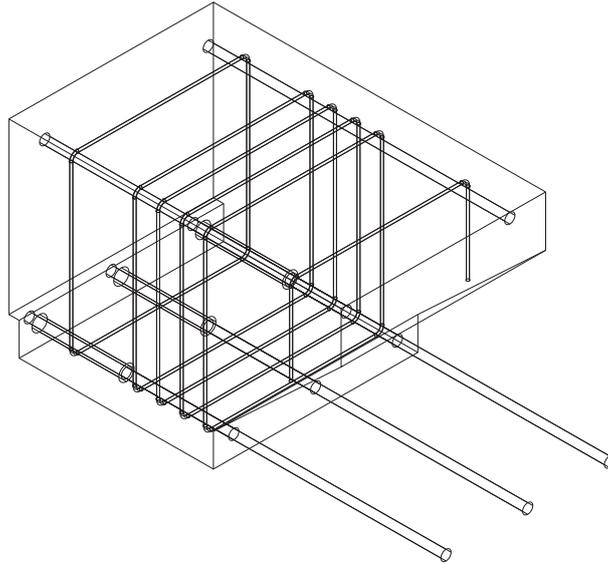


Figure 20: Geometry of Type B specimen. Type A is similar but without stirrups in the bonded zone. Adopted from (Zandi *et al.*, 2011).

4.3.2 NLFE analyses using thin sections

This section presents the NLFE analyses using thin sections. Cf. (Zandi *et al.*, 2011) for an exhaustive presentation of the NLFE analyses of the full beam-end.

The analyses were carried out according to assessment level IV, as described in Section 4.1.5 and using the commercial software DIANA 9.6 (TNO, 2015). *Paper II* gives a detailed presentation of the thin-section FE models and the choices made for the analyses.

Since a small part of a structure is modelled, the boundaries in the FE model inevitably come closer to the region of interest for extracting results. This may make a smaller model more sensitive to the boundary conditions (BCs) that were applied, as compared with one representing a larger proportion of the geometry. Thus, four different FE models were investigated, each with different thicknesses and BCs.

The influence of the section thickness was investigated by modelling sections of three different thicknesses:

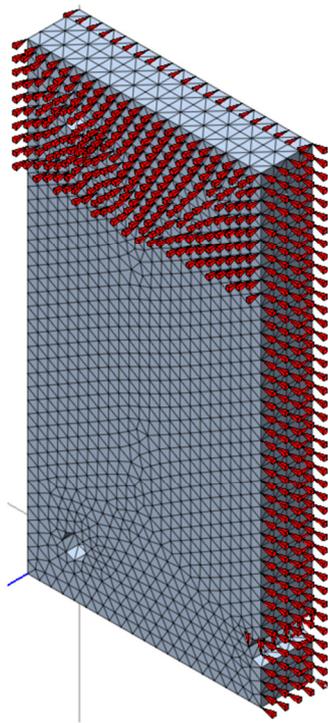
- 44 mm (one stirrup spacing), with the stirrup placed in the centre of the section.
- 22 mm (half stirrup spacing), i.e. the model above split at the centreline. Thus, half a stirrup was placed at the back of the section.
- 10 mm, with the stirrup (equivalent amount) placed in the centre.

A symmetrical BC around the vertical axis was applied to all models, thus reducing the model size by half. Furthermore, the nodes on the back of the section (at the opposite side from where the bars are pulled) were restrained from translation in the pulling direction. Three types of BCs on the front surface of the section model were considered:

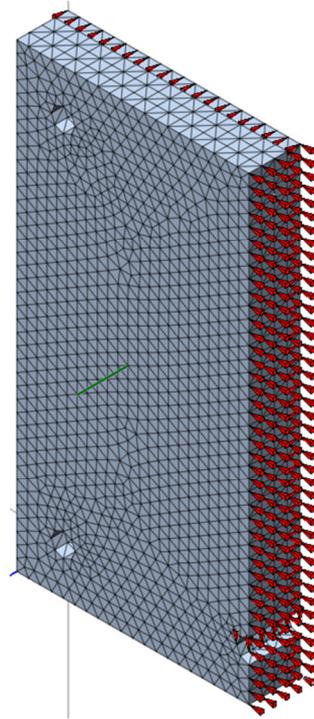
- Top 90 mm of the front surface fixed in the pulling direction, resembling compression block in beam analogy (denoted EL44 & EL10).
- Front surface forced to remain in the same plane, resembling Bernoulli region in a beam (denoted EL44-P).

- Front surface allowed to move unconditionally (denoted EL22).

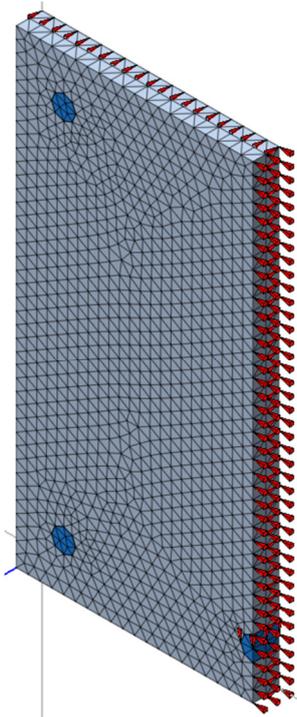
The section models EL44, EL44-P, EL22 and EL10 are shown in Figure 21, and the numbers indicate the section thickness.



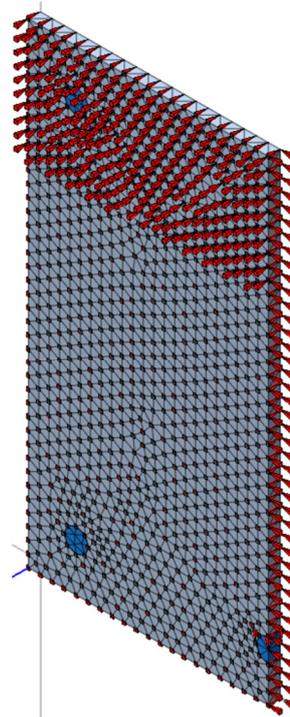
(a) EL44



(b) EL44-P



(c) EL22



(d) EL10

Figure 21: Section models: (a) EL44, stirrup in centre; (b) EL44-P, stirrup in centre, front nodes constrained to remain in the same plane; (c) EL22, half a stirrup at the back and (d) EL10, equivalent stirrup in centre. In (a) and (b) the rebar elements are hidden.

The results of the NLFE thin-section and beam-end models were compared with the experimental results in terms of average bond stress versus imposed bar slip. The average bond stress was calculated by dividing the pulling force on the reinforcement bar with the surface area of the bar in the bonded zone.

Figure 22 and Figure 23 show the results obtained when pulling the middle bar of specimens with stirrups (Type B), for the uncorroded and corroded cases respectively. All analysis results are found in *Paper II*.

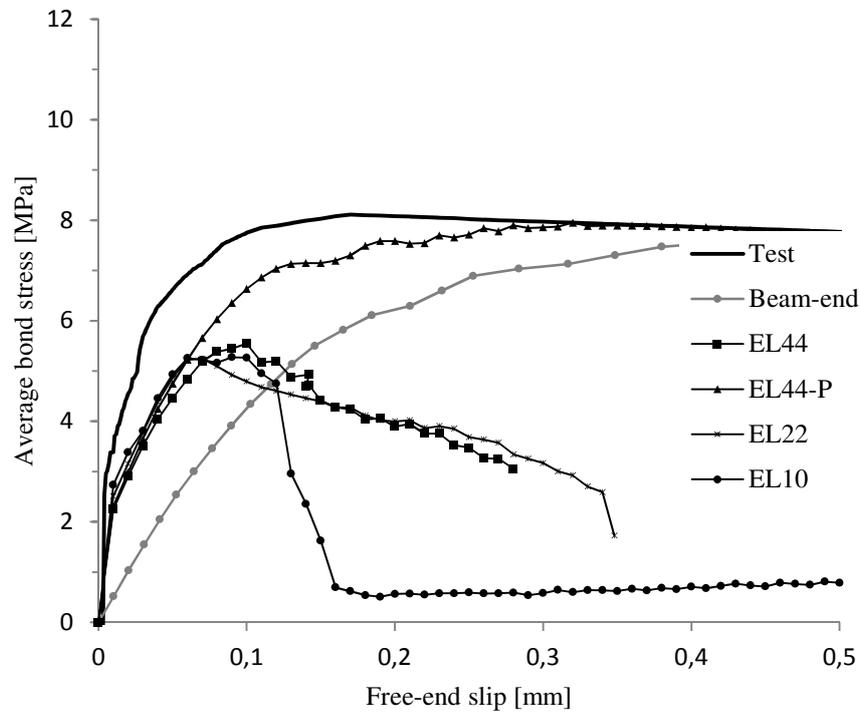


Figure 22: Type B - Reference: middle bar.

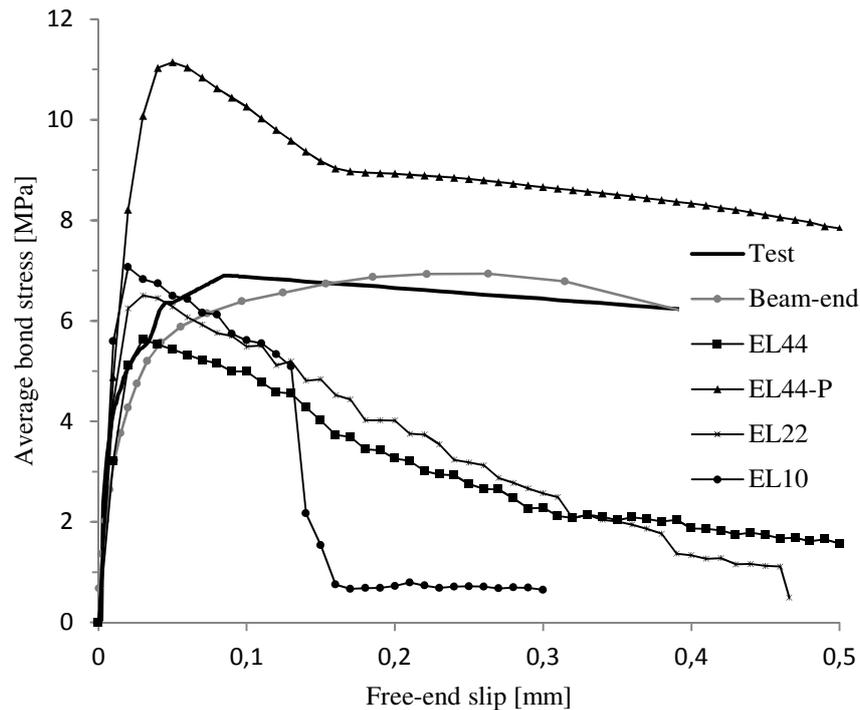


Figure 23: Type B - 0.7% corrosion: middle bar.

Based on the study, it can be said that in most cases the average bond stress between the section model, beam-end model and test agreed reasonably well. However, in several cases the EL44-P section model (with its front nodes forced to remain in the same plane) showed higher capacity than the beam-end and test values. See Figure 23 for example. By studying the stresses around the pulled bar, it was observed that this BC gave rise to compressive stresses around the bar. This might explain the difference in bond capacity, as the section models with other BCs exhibited tensile stresses around the pulled bar.

Furthermore, there was no observable difference in bond capacity between the section models with concrete nodes restrained in the compression zones (EL44 and EL10) and the one with a completely unrestrained concrete front surface (EL22). By studying the stresses in the model, it was seen that the restraint at the top 90 mm did not influence the stresses at the bar level. These results are therefore reasonable.

The knowledge gained from this investigation can be used to set up the thin-section FE models for the parametric study mentioned at the beginning of this section. Close attention should be paid to the stresses in the concrete surrounding the reinforcement bars, as this demonstrably has a heavy influence on bond strength.

5 Conclusions and outlook

The main aim of this work was to develop a reliable assessment method for anchorage of corroded reinforcement in structural concrete. A few main objectives of the present thesis were established to serve this main aim. These included providing a background on reliability assessment, developing a simplified level IV (i.e. NLFE) model to facilitate a parametric study and implementing the most recent Model Code in the existing simple bond model.

5.1 Conclusions

The main conclusions of the individual papers will be presented first, followed by conclusions drawn from the work that was compiled.

Paper I dealt with reliability assessments of structures and compared several safety formats for NLFE analyses, as provided in *fib* Model Code 2010. Based on this work, the following conclusions may be drawn:

- One studied safety format did not meet the intended safety level for one of the load histories being considered. This highlights the importance of load history in NLFE analyses.
- Little guidance is given in *fib* Model Code 2010 regarding how to quantify the structural resistance to apply to the safety formats. It is important to define a representative measure that accounts for relevant aspects. This becomes complicated when such things as load history, time-dependent effects or cyclic loading are to be incorporated.

Paper II developed a simplified model for bond capacity assessments on level IV. This was to facilitate a parametric study addressing some of the aspects important for the development of the simplified model ARC2010, such as reinforcement bars in several layers and closely spaced bars. Small, thin-section NLFE models were compared to larger beam-end ones as well as the results of physical tests. The following conclusions may be drawn:

- For section analyses of uncorroded and corroded specimens, the boundary conditions applied to the model have a major influence on the results.
- If the imposed boundary conditions induce compressive stresses around the pulled reinforcement bars, this may lead to an overly large capacity.
- It was possible to set up a section model which yielded similar or conservative estimates of the bond strength compared to beam-end models and test values for uncorroded and corroded cases.

Paper III presented the development of a 1D engineering assessment method for anchorage of corroded steel reinforcement in concrete. The work includes formulation of the model, calibration against a large database of bond tests and verification against an empirical expression found in the literature. Based on the work presented in the paper, the following conclusions may be drawn:

- Both the concrete cover and the transverse reinforcement contribute to the confinement in the assessment model that was developed. The confinement also changes when corrosion cracks the concrete cover; this is also accounted for in the assessment model.

- The physical behavior is well represented by the model, with a marked decrease in bond strength at cover cracking for low amounts of transverse reinforcement and a less pronounced reduction for larger amounts.
- The position of the reinforcement bar in the cross section is accounted for through the incorporation of *fib* Model Code 2010 in the assessment model.
- The new model is shown to be in better agreement with the results in the bond test database, as compared to the previous version of the model. It yields results that are slightly on the safe side, with or without stirrups.
- The proposed model not only gives the reduction of anchorage capacity and the maximum peak stress, it also yields the full local bond stress-slip relationship.

General conclusions

Based on the work conducted within this, thesis the following general conclusions may be drawn.

- Advanced NLFE analyses are not widely used among design and assessment engineers today. The high cost in terms of skilled personnel and computational time is not always justified by the additional knowledge gained about the structure. There is no intrinsic value in conducting advanced analyses. If a simpler approach can yield the sought results then this is preferable. Therefore there is a need for both advanced assessment methods and more simple approaches.
- In cases where advanced structural analyses are necessary, computational methods are available (although further development and optimisation is always recognised). But when it comes to verifying the structures against the intended safety levels, there is a lack of guidance in the codes. This is particularly true in complex loading situations and treatment of the load history.
- The approach of conducting FE analyses using thin sections, as investigated in *Paper II*, can be used in a parametric study to further develop the ARC2010 model of bond assessment presented in *Paper III*.
- Close attention should be paid to the boundary conditions for a thin-section FE model as well as the load application. For example, the application of corrosion and the pulling sequence of the reinforcement bars can have a major influence on the bond strength results.

5.2 Future research

The following points summarise suggested future research. These are based on the aims of the present work and in the light of the above conclusions. They also embody the author's view of valued contributions to the field:

- Investigation of the influence of multiple reinforcement layers and closely spaced bars; for example by a parametric study using thin-section NLFE analyses.
- Development of guidelines and provisions for dealing with the load history when determining the present safety level of a structure.
- Incorporation of uncertainties into the ARC2010 model and development a probabilistic model.

- Calibration of deterministic safety factors for use in semi-probabilistic approaches. This can be based on the probabilistic version of the ARC2010 model.
- Expansion of the engineering assessment procedure to include the influence of corrosion on the capacity for shear and bending moments.
- Establishment of a framework for conducting reliability evaluations of different structural elements and inclusion of time-dependent effects to enable probabilistic service-life prediction. Moreover, quantification of modification factors in deterministic service-life prediction.
- Demonstration of the applicability of the developed assessment model in a case study of an actual concrete structure damaged by corrosion.

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