

THESIS FOR THE DEGREE OF LICENTIATE OF ENGINEERING

Anchorage of naturally-corroded, plain bars in  
Reinforced Concrete structures

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Division of Structural Engineering

CHALMERS UNIVERSITY OF TECHNOLOGY

Göteborg, Sweden 2019

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**Cover**

In the picture: testing of beam 14H.

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## ABSTRACT

Reinforced Concrete (RC) is the most common construction material in existing structures. However, RC structures are susceptible to deterioration over time, with corrosion of the reinforcing steel as most common mechanism. Corrosion reduces strength and ductility provided by the reinforcement bars and affects their interaction with the concrete. Research on the structural effects of corrosion commonly focuses on deformed bars and applies artificial corrosion. Performance is evaluated based on testing the bond between the bars and the concrete. Plain (smooth) reinforcing bars, as typical in older structures, are seldom studied. Plain bars interact differently with concrete, due to the absence of ribs. Additionally, doubts on the relevance of artificial corrosion methods have been raised. Thus, there is a lack of knowledge on the effects of corrosion of plain bars, and naturally corroded specimens are the ideal mean of acquiring it.

This work investigates the bond of naturally corroded, plain reinforcement bars by testing specimens taken from the edge beams of a decommissioned, 80-year-old bridge. Pilot tests were performed to investigate possible test configurations, to which a total of 20 beams were subjected to displacement-controlled 3-point bending. The beams presented different levels of damage, and the corrosion level of each tensile reinforcement bar was afterwards measured using of a 3D scanner. All but three of the tested specimens were able to anchor the yield force of the bars after the opening of one or two major bending cracks in the middle. Bending failure, not bond strength, limited the load-carrying capacity for the majority of test specimens. At large deflections, end-slip of the reinforcement bars was observed; thus, anchorage limited the deformation capacity.

The average bond strength was evaluated separately in the unyielded and in the yielded zones. The average bond strength in the unyielded zone was found to be equal to 7.4 MPa, with a standard deviation of 3.3 MPa. The casting position was identified as an important factor. Bottom-cast bars had higher bond strength when uncorroded, but were more prone to external cracks in the bond region and consequently loss of bond strength for small corrosion levels. Top-cast bars had lower bond strength when uncorroded, but reached higher bond strength with increasing corrosion levels, due to the absence of external cracks. These differences are likely the result of a higher density of the concrete surrounding the bottom-cast bars. In the yielded zones, substantial loss of bond strength was observed, with an average of 1 MPa. This affected the overall structural behaviour, which was observed to change from beam to arch action for larger deflections. To conclude, the results improve our understanding of the behaviour of older structures with plain bars and will enable, in the long run, improved assessment methods.

**Keywords:** plain reinforcement bars, anchorage, bond, corrosion, concrete.



*"Begin at the beginning," the King said gravely,  
and go on till you come to the end: then stop"*  
*Lewis Carroll, Alice in Wonderland*



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## THESIS

This thesis presents a summary and an extension to the following appended papers:

- [I] K. Lundgren, S. Robuschi, and K. Zandi. *Methodology for testing rebar-concrete bond in specimens from decommissioned structures. Submitted.*
- [II] S. Robuschi, K. Lundgren, I. Fernandez, and M. Flansbjerg. *Anchorage of naturally corroded plain reinforcement bars in flexural members. Submitted.*

These publications are always referred to as paper I and II according to the labeling in the list above.

### AUTHOR'S CONTRIBUTION TO JOINTLY WRITTEN PAPERS

The appended papers were prepared in collaboration with co-authors. In the following, the contribution of the author of this licentiate thesis to the appended papers is described.

In **Paper I**, the author participated in the planning of the paper and also in the planning and execution of two of the three presented examples of experimental series. The author contributed to the writing of the sections regarding those experiments, and with comments and discussions on the remaining paper.

In **Paper II**, the author composed the literature study, participated in the planning, and was responsible for the execution of the experimental program. The author performed the analysis of the data, participated in the discussion of the results, and took responsibility for the planning and writing of the paper.

## ADDITIONAL PUBLICATIONS BY THE AUTHOR

In addition to the appended papers, the author has also contributed to the following publications:

S. Robuschi, K. Lundgren, I. Fernandez, K. Zandi, and M. Flansbjerg. “Anchorage capacity of corroded smooth reinforcement bars in existing reinforced structures”. In: *Proceedings of the 12th fib International PhD Symposium in Civil Engineering* (2018), 1039–1046.

S. Robuschi, I. Fernandez, K. Zandi, and K. Lundgren. “Assessment of the load-carrying capacity of existing structures with corroded smooth reinforcement bars.” In: *Proceedings of XXIII Nordic Concrete Research Symposium Vol.B* (Issue 3 2017), pp. 255–258.

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# 1 Introduction

## 1.1 Background

Concrete is one of the oldest and most widely used materials in the history of mankind: concrete floors dated 1400-1200 BC can still be observed in the Royal palace of Tiryns, Greece. The world's largest unreinforced concrete dome, the Pantheon's dome in Rome, dates back to about 126 AD. The first iron-reinforced concrete building dates back to 1853; in 1904, the first skyscraper using reinforced concrete was built in Cincinnati. In a survey dated 2011, about 86% of bridges in Europe were estimated to be made with reinforced or pre-stressed concrete [20]. Nowadays, reinforced concrete structures constitute a large part of the world's infrastructure.

The durability of reinforced concrete (RC) structures started to be questioned years after the first structure was built. Today, corrosion of the steel reinforcements is known to be the most common cause of deterioration in RC structures [3]. Within the last few decades, a number of structural failures have been attributed to corrosion damages [4]. Two substances are commonly responsible for the initiation of the corrosion process: carbon dioxide, unavoidably present in the atmosphere, and chlorides, which are generally a result of exposure to marine environment or de-icing salts. The process of corrosion initiates with the depassivation of the reinforcement embedded in concrete, and eventually leads to the origination of the corrosion products. This can lead to extensive damages, able to compromise the function and structural integrity of the structure.

The degradation process is linked to the nature of the corrosion product. Corrosion products occupy a larger volume than uncorroded steel and induces normal pressure on the surrounding concrete, eventually causing the concrete cover to crack and spall. Simultaneously, corrosion results in a loss of cross-sectional area of the reinforcement bar, and thus a decrease in load-carrying capacity and ductility of the bar. Both effects can compromise the overall structural behaviour of the structure. Hence, in recent years, research has been focusing on acquiring further knowledge on the corrosion process, with the ultimate goal of more precisely assessing existing RC structures.

A correct assessment of the corrosion damage allows for realistic estimations of the service life. This is not only needed to guarantee the safety of the structure but would also possibly avoid needless rebuilding and, consequently, be greatly beneficial from both an economical and environmental point of view. Present-day

society requires RC structures to perform longer and withstand a higher load-carrying capacity than originally designed for. Furthermore, climate change is foreseen to increase the impact of corrosion on infrastructures [29]: higher carbon dioxide concentration, warmer climate, and more frequent freezing cycles will expedite the development of corrosion. Understanding the consequences of such phenomena on structures is therefore of the highest importance.

The oldest operative reinforced concrete structures were built with plain reinforcement bars. Plain bars are no longer utilized in new constructions, since the introduction of deformed bars allowed for higher anchorage capacity through the use of ribs. The use of plain reinforcement bars drastically diminished approximately prior the mid-60s, but with large differences between countries. Sweden introduced deformed bars during the 40s; Italy maintained use of plain bars up to the 70s, to which a large part of the existing infrastructure contains plain reinforcement bars; and, in the UK, plain bars re-emerged for a few years in the mid-1970s as a result of steel shortage due to industrial unrest.

Current research is primarily oriented towards the effects of corrosion on deformed bars; in particular, researchers tend to investigate the impact of corrosion on the bond between the steel and the surrounding concrete; this is most significantly linked to the assessment of anchorage zones [19]. Bond of deformed bars is commonly studied as the result of three contributing mechanisms: chemical adhesion, friction, and mechanical interlocking between ribs and concrete, the latter giving the highest contribute to the total bond strength. The bond of plain reinforcement bars is substantially different from the one of deformed bars. Due to the absence of ribs, mechanical interlock acts only on micro-level, between the concrete and the surface roughness of the reinforcement bar. Thus, friction and chemical adhesion play a more fundamental role. Given that a) structures with plain reinforcement bars are still in use; b) they are most likely affected by corrosion damages; and c) their bond properties cannot be directly compared to existing literature, which is based on the use of deformed bars, an investigation is necessary into the behaviour of corroded, plain reinforcement bars.

Several recent studies highlight the importance of the topic, testing the bond strength of plain uncorroded bars [26] [14], and ultimately applying artificial corrosion to the bars to observe its affect on the steel-concrete bond, both through pull-out tests and flexural tests[6] [7]. However, there are doubts on the soundness of artificial corrosion methods [22][24][31][30].

Testing naturally corroded specimens taken from decommissioned structures offers the opportunity of studying corrosion as the result of exposure to natural environments, without the need of using electrical current to accelerate the corrosion process. In recent years, a few studies on naturally corroded specimens with deformed bars were successfully carried out [16] [25], but to the knowledge of the author there are no published results for the case of plain reinforcement bars. Accordingly, there is a need for experiments on naturally corroded structures with

plain reinforcement bars.

## 1.2 Scope and Objectives

The aim of this work is to increase knowledge that could contribute to the assessment of older RC structures with plain reinforcement bars. The work focuses on the effect of corrosion damages in the anchorage zone, by testing naturally corroded specimens taken from a decommissioned structure. Within the overall vision of the project, the following objectives are identified:

- To find efficient ways to test the anchorage in specimens taken from existing structures by studying the methods discussed in the literature and consider necessary improvements;
- To study the effect of different levels of corrosion on the overall behaviour and on the anchorage of bars in flexural members;
- To identify factors influencing the bond strength of plain reinforcement bars in existing structures;

Although not answered in this thesis, additional objectives were defined to meet the overall aim of the project. The following objectives were taken into consideration while designing the experimental part of the project, though their results will be presented in future work:

- To investigate the effect of corrosion on the anchorage capacity of flexural members with plain reinforcement bars with end-hooks in the anchorage zone;
- To investigate the effect of corrosion on the bond-slip relationship of corroded, plain reinforcement bars;
- To utilize the knowledge acquired through experimental work to develop numerical models that link data from visual inspection to the remaining load-carrying capacity of RC structures with plain reinforcement bars.

## 1.3 Method and Scientific Approach

This work primarily comprised the design and execution of an experimental program aimed at fulfilling the objectives stated in section 1.2. All the results presented in this work are the outcome of the designed structural tests and the corresponding observations. Necessary reference was made to relevant literature for the experimental design and data analysis. All the experiments were conducted on specimens taken from the edge beams of a 80-years old decommissioned bridge

(Gullspång bridge). The visual damages in the beams were carefully documented before testing, and information on the corrosion distribution on each tensile reinforcement bar was collected after testing. Different set-ups were considered to investigate the anchorage of decommissioned structures with plain reinforcement bars affected by corrosion. The design needed to be adapted to the specific geometrical and material properties of the specimens. Three test series were designed: 1) 3-point bending flexural tests of beams with plain, straight reinforcement bars, 2) 3-point bending flexural tests of beams with plain reinforcement bars with end-hooks, and 3) pull-out tests of bars with short embedment length. The first test series was carried out and the results were analysed. The second and third series are currently ongoing, and their results are not yet available. The results of 3-point bending tests of beams with plain, straight reinforcement bars is presented in this work.

## 1.4 Limitations

Some of the limitations in this work are a direct result of working with specimens taken from a decommissioned structure:

- The use of indirect supports would have been advantageous in the 3-point bending tests. By not applying direct support pressure to the tensile reinforcement bars, it would have avoided the risk of locally increase their bond strength. However, the risk of damaging the specimens when drilling the core for the suspension hangers was considered too high. Furthermore, the uncertainty in the positioning of the stirrups introduced the additional risk of drilling through a stirrup;
- It was not possible to monitor the strains in the tensile reinforcement during the 3-point bending tests. This is the result of not having cast the specimen. The outcome is such that bond strength has to be estimated from observable data, being the yield force and yield penetration length of the reinforcement bar;
- The lack of information on the weight and volume of the reinforcement bars before the initiation of the corrosion process made the evaluation of the corrosion level challenging. 3D scanning was eventually preferred to gravity methods;
- The irregular spatial distribution of stirrups in the specimens made it difficult to reach any conclusion on their influence on the bond strength.

Additional limitations were:

- The use of a jack hammer to extract the bars from the beams after the 3-point bending tests resulted in damage to some of the samples. Damaged bars were removed from the database;
- Only the end-displacements of the tensile reinforcement bars were monitored during testing. In two experiments, the reinforcement bars in the (theoretical) compressive zone showed signs of end-slipping, but no data on their displacement was collected;
- The use of a directly supported 3-point bending test leads to the introduction of support pressure in the anchorage zone. Even though narrow supports were used to limit this effect, this can possibly locally enhance the measured bond strength;
- Few studies are present in literature on corroded plain reinforcement bars. This made it difficult to properly compare the results with similar studies.

## 1.5 Original Features

Plain reinforcement bars are the object of very few recent works, and no investigation including both plain reinforcement bars and natural corrosion exist to the knowledge of this author. This study introduces the following results as original findings:

- Naturally corroded edge beams were taken from a 80-year old decommissioned bridge;
- The anchorage of plain reinforcement bars subjected to natural corrosion damages was investigated;
- The anchorage was investigated by designing different test set-ups to be carried out on specimens extracted from the same structure, with the aim of reaching a comprehensive view of the anchorage behaviour;
- The remaining bond strength at yielding is taken into account and evaluated separately from the bond strength in the unyielded zone.

## 1.6 Outline

This thesis consists of an introductory part and two appended papers. The outline of the work is as follows:

Chapter 1 provides a background and presents the aim, method, limitations, and original features of the study;

Chapter 2 introduces the theoretical framework of the study by reviewing state-on-the-art knowledge on the bond capacity of plain reinforcement bars, with and without corrosion damages;

Chapter 3 presents an overview of the experimental program;

Chapter 4 presents the main findings from the experimental program;

Chapter 5 addresses the main conclusions from this study and advances suggestions for future research.

## 2 Theoretical framework

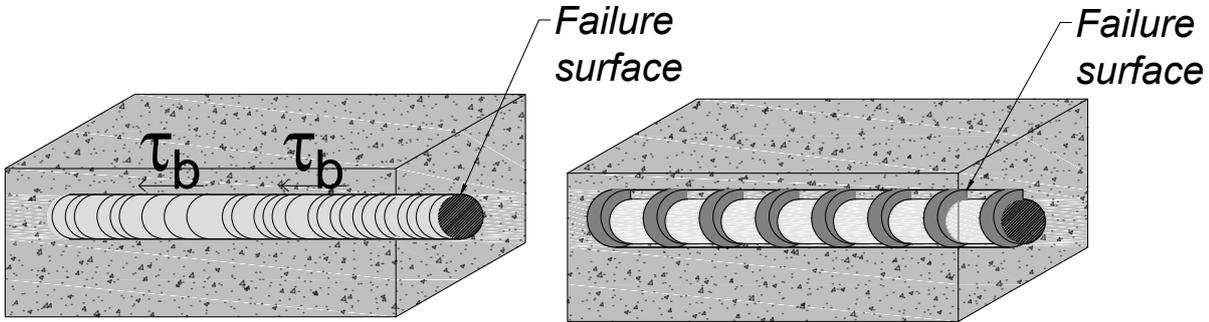
This chapter aims at establishing the theoretical framework to this work by presenting state-of-the-art knowledge on the bond of plain reinforcement bars and on the effect of corrosion on structures reinforced with such bars.

### 2.1 Plain Reinforcement Bars

The first RC structures were constructed with plain bars. Plain bars were eventually replaced with the introduction of deformed bars. The use of deformed bars in RC construction became common during the mid-1900s, though implementation differed between countries: Sweden adopted them in the 1940s, Canada, USA, and Australia in the mid-1950s, and Italy maintained use of plain reinforcement bars for about 20% of RC structures through the 1980s [28]. The bond behavior of plain bars is fundamentally different than that of deformed bars. Deformed bars increased bond capacity through the introduction of ribs to interlock with the surrounding concrete. Their bond is the result of three different mechanisms: chemical adhesion, friction, and mechanical interlocking between the ribs of the reinforcement bars and concrete. The latter provides the largest contribution to the bond strength; the bearing action of the ribs transfers inclined forces to the concrete. These forces are generally divided into a longitudinal component (bond), and a normal component (splitting stress). The splitting stress resulting from bond action can result in splitting failure, while pull-out failure originates from the failure of the concrete between the ribs (Fig.2.1). On the other hand, plain reinforcement bars, having no ribs, rely mostly on adhesion and friction for transferring forces; mechanical interlock takes place only on a micro level, between the concrete and the surface roughness of the steel bar. Friction, being a function of the normal stresses on the bar, increases with increasing confinement, and, as a result, the bond capacity of plain reinforcement bars depends on the level of confinement of the bar itself [21] (Fig.2.1). Sliding friction [1], as to indicate the wedging action of small particles of concrete detached by the initiation of the slip, contributes to the residual bond of plain bars.

Factors such as the density of the concrete surrounding the bar and the casting position are expected to have higher impact on the bond of plain reinforcement bars. Top-cast bars are more likely to be surrounded by less dense concrete. This is the result of settlement of the aggregate below the bar and the accumulation of bleed water at the bar [2]. Lower density results in a lower bond strength than for

bottom-cast bars [6], since the decreased confinement reduces the normal stresses on the bar.



**Figure 2.1:** *Failure surface: plain bars (left) and deformed bars (right) at pull-out.*

In Model Code 2010 [15], the bond of hot-rolled plain reinforcement bars is calculated as a function of the concrete compressive strength, expressed in MPa ( $0.3\sqrt{f_{cm}}$ ). This peak value is assumed to correspond to 0.1mm of slip. Good and poor bond conditions (depending on the confinement of the bar) are distinguished, and a reduction factor of 0.5 is suggested for the case of poor conditions ( $0.15\sqrt{f_{cm}}$ ). Alternatively, both empirical and simplified expressions to estimate the bond strength of plain bars embedded in concrete can be sourced from the literature. These equations are most commonly based on results of simple pull-out tests [26] [27] [23] [14] [8]. In an attempt to appear consistent with deformed bars, many of these expressions are a function of the compressive strength. E.g., Verderame et al.[27] estimated the maximum bond strength to 31% of the square-root of the concrete cylindrical compressive strength, in MPa, while Melo et al.[23] and Feldman and Bartlett[14] took additionally into consideration, respectively, the yield stress of steel (Melo et al.), the surface roughness, and the development length (Feldman and Barlett).

When investigating the influence of surface roughness on the bond of plain reinforcement bars, Feldman and Barlett[14] observed a relationship between bond capacity and surface roughness. Since the bars used in the past are expected to present a rougher surface than those produced today, in order to simulate the surface roughness typical to older rebars, the study opted to sandblast new bars prior to lab testing. Gustavson [17] studied the influence of concrete density and surface roughness on the bond behaviour of three-wire strands and found that an increase in the micro-roughness of the strand surface strongly increased adhesion in the initial bond response. The study on three-wire stands also determined a relationship between the bond strength of non-corroded bars with an increase of the concrete density. Support pressure (a supplemental source of compressive confinement of the reinforcing bars), has also been linked to increased bond strength [21].

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## 2.2 Effect of corrosion on the bond of plain reinforcement bars

Corrosion affects RC structures in different ways:

- Corrosion changes the properties of the bar itself; by reducing the cross-section of the bar it decreases both strength and ductility;
- Corrosion products occupy a larger volume than the uncorroded steel. This introduces pressure between the bar and the surrounding concrete. As the corrosion product continues to develop, this pressure may induce cracking and/or splitting of the concrete cover;
- Corrosion introduces a layer of corrosion product that is substantially weaker than the original steel-concrete interface.

The two last points, by affecting the concrete-steel interface, influence the bond between the steel and the surrounding concrete. Cairns et al.[5] performed a study on the changes in friction characteristics of the interface between corroded reinforcement and concrete using artificial corrosion methods; the results indicated that the corrosion product does not impair friction characteristics for specimens showing less than 1.0 mm surface cracks. It was concluded that volumetric expansion is the effect of corrosion that has the highest impact on the bond capacity. Volumetric expansion affects the bond between plain bars and the surrounding concrete in different ways: if no cracks are present in the anchorage region, the increase in mechanical pressure provided by the corrosion product would noticeably increase friction, as friction is a function of the normal stresses on the concrete-steel interface [7]. There is a limit, however, to this phenomena, in that excessive pressure is likely to split the concrete cover and consequently lead to a loss of confinement.

An additional factor that is likely to have an impact on the way corrosion affects the bond capacity is the difference in concrete density between top-cast and bottom-cast bars. This has been shown in tests by Cairns et al. [6]. The tests investigated the bond behaviour of artificially corroded plain bars using RILEM pull-out tests and "beam-end" tests. The results show that the reported bond capacity of top-cast bars substantially increased with the introduction of small levels of corrosion. The bond capacity of corroded top-cast rebars increased to match the measured capacity of uncorroded bottom-cast bars. Bottom-cast bars, on the other hand, performed better in an uncorroded state but were more prone to crack and therefore to lose bond capacity with increasing corrosion level. Stirrups were found to be of high importance for maintaining bond capacity with increasing corrosion level.

## 2.3 Effect of corrosion on flexural members

Cairns et al.[7] investigated the effect of corrosion on the behaviour of flexural members with plain bars. A consistent increase in strength was observed in the artificially corroded beams when compared to non-corroded specimens. The additional strength was attributed primarily to the increase in bond due to the volumetric expansion of the corrosion product inducing normal pressure. Most of the tested beams were subject to flexural failure, preceded by yielding of the tensile reinforcements. Although the tests did not include anchorage failure (no measurement of the peak bond capacity), the bond was observed to influence the load-carrying capacity of the beams. The decrease in bond capacity due to yielding of the tensile reinforcement led to a change in the load-carrying mechanism, from purely flexural behaviour to a hybrid arching/flexural action.

A similar mechanism was observed in studies by Dong et al.[9] and Feldman et al.[13]. Both studies investigated the influence of bond capacity on the failure mechanism of flexural beams. Dong et al.[9] performed 4-point bending tests on twenty, artificially corroded, RC beams with deformed bars. A transition from beam to arch action was observed in the later loading stages in connection to a degraded bond capacity in the bars due to corrosion. Feldman et al.[13] investigated the transition from beam action to arch action in flexural members with plain bars, observing the bond strength distribution along the length of the tensile reinforcement. High bond stresses were observed adjacent to the supports for beams where shear was carried principally by arch action. Arch action was associated with a marked reduction of flexural stiffness. The change from flexural to arch action can be explained by analysing the relationship between the shear force,  $V$ , and the bending moment,  $M$ , in a section  $x$  [9]:

$$V = \frac{dM(x)}{dx} \quad (2.1)$$

Considering  $M(x)$  is equal to the force,  $F(x)$ , in the bar, multiplied by the lever arm,  $z(x)$ , equation 2.1 can be rewritten as:

$$V = \frac{dF(X)}{dx}z(x) + F(x)\frac{dz(x)}{dx} \quad (2.2)$$

The first term in equation 2.2 represents beam/flexural action: the force in the bar decreases outside the high moment region, while the lever arm is constant. Shear forces need to transfer from the reinforcement to the surrounding concrete through bond for beam action to occur. Low bond capacity limits the amount of beam/flexural action. The second term represents arch action: the decrease of the moment outside the high moment region is linked to the decrease of the lever arm, but the force in the bars is constant. Pure arch action does not involve the transfer of forces between the tensile reinforcement and the surrounding concrete.

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A decrease of bond capacity, due to, e.g., yielding of the tensile reinforcements, corresponds thus to a transition from beam action to arch action.

## 2.4 Artificial and Natural Corrosion

All available literature pertaining to the assessment of RC structures with corroded, plain reinforcement included the application of artificial corrosion techniques to simulate corroded conditions. Artificial corrosion techniques are popular due to a substantially reduced waiting time in the exposure of specimens. It is possible to simulate 30-years of corrosion in a matter of a few months. The most commonly used method for artificially inducing corrosion is the impressed current technique (or galvanostatic method). This technique applies a constant current from a DC source to the rebar embedded in concrete to induce corrosion. Chlorides are often used as depassivating substance, either by adding salt to the concrete during the mixing process or by exposing a pre-cracked specimen to a highly concentrated salt solution.

Although the use of artificial corrosion has evident benefits, several uncertainties have been raised on how well it captures real-time corrosion formation that would occur naturally in RC structures. Austin et al.[24] observed differences in the electrochemistry behind natural and artificial, chloride-induced corrosion. The primary electrochemical difference from naturally corroded system was the gradual reduction of the local pH due to electrolysis of the pore water. This may lower the critical chloride concentration required to induce corrosion and accelerate the degradation process. Saifullah and Clark [22] showed that differing current density has an effect on the bond strength. Yuan et al.[31] found that different corrosion-induction methods led to different surface characteristics in corroded steel bars. The galvanostatic method produces a homogeneously corroded surface, whereas naturally corroded specimen commonly display surface heterogeneity with randomly distributed pitting. Williamson and Clark [30] tested artificially corroded plain bars with different levels of corrosion (0–20%) and current density (0.25–2mA/cm<sup>2</sup>). The current density was found to be responsible for changes in the morphology of the corrosion product and, consequently, in the bond strength.

Testing naturally corroded specimens taken from decommissioned structures offers an alternative to the use of artificial corrosion methods. This allows for the study of damages due to many factors influencing the aging processes of RC structures, such as corrosion, creep, and freezing-thawing cycles. Examples of successful studies on naturally corroded specimens can be found in [16] [25] [32]. To the knowledge of the author there are no published studies on specimens with naturally corroded, plain, reinforcement bars.



# 3 Overview of the experimental program

The experimental program was designed to test and obtain data on the anchorage of plain reinforcement bars from the edge beams of a naturally corroded 80-year old bridge. Fig.3.1 gives an overview of the experimental program, including future plans.

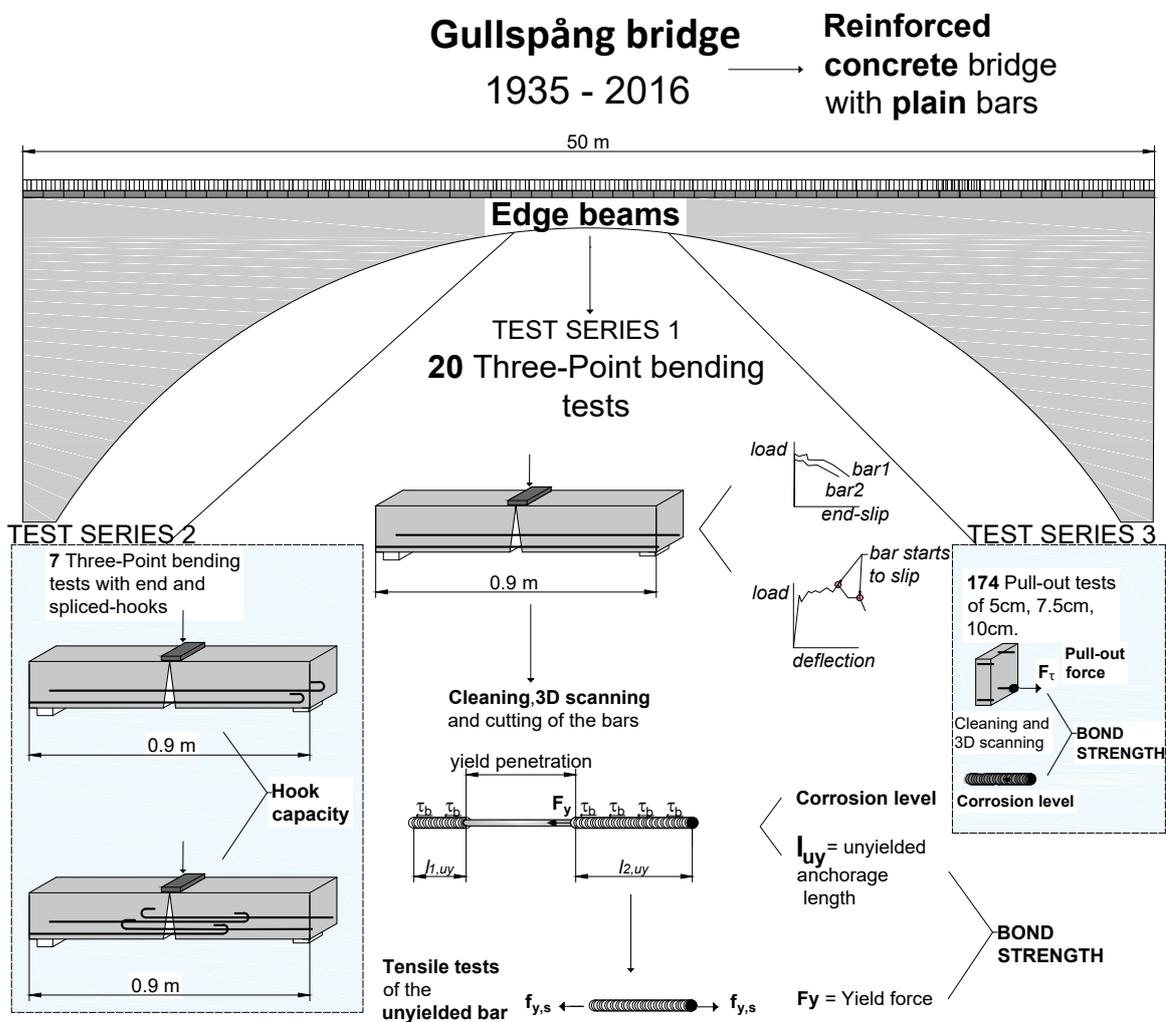


Figure 3.1: Overview of the tests planned from the edge beams of Gullspång bridge.

Three different test series were planned:

1. 20 3-point bending tests of beams with plain reinforcement bars;
2. 7 3-point bending tests of beams with plain reinforcement bars featuring end-hooks and positioned in two different configurations (4 with one pair of hooks and 3 with spliced hooks);
3. 174 pull-out tests of reinforcement bars with various embedment length (5, 7.5 and 10 cm).

This thesis focuses on the first test series, as the other two are still ongoing. A brief overview of the remaining two experimental series is provided.

### 3.1 Gullspång bridge

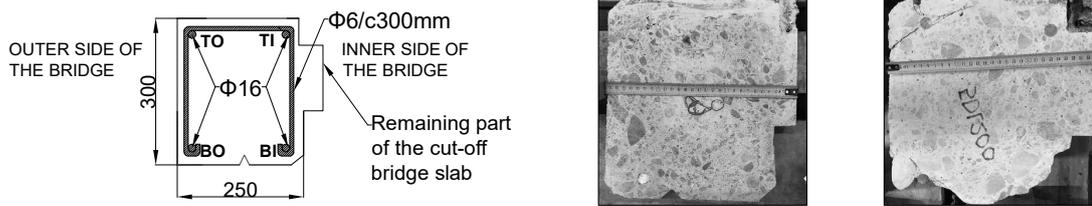
Gullspång bridge (Fig.3.2) was built in 1935 and torn down in 2016 due to heavy corrosion damages. The edge beams were carefully taken out, cut in segments and designated to be used for research. Having been exposed for 81 years to weather conditions that include snow, freezing-thawing cycles and wind, as well as to de-icing salts and traffic loads, the beams presented different cracks on their surface, and spalling strongly affected the geometry in some locations (Fig.3.2). Signs of corrosion were clearly visible, but not uniformly distributed.



**Figure 3.2:** *Gullspång bridge: view of the bridge before demolition (left) and exposed tensile reinforcement bars and spalling in an edge beam (right).*

*Photos courtesy of Ignasi Fernandez*

The edge beams were characterized by  $\phi 6$  stirrups, open on the bottom side with respect to the original position on the bridge and  $2\phi 16$  plain reinforcement bars, top and bottom (Fig.3.3). The concrete cover varied approximately between 20 and 55 mm, but was reported equal to 34 mm in the original drawings.



**Figure 3.3:** *The beam cross-section geometry (according to the original drawings) is shown on the left. Photographs of two beam cross-sections are shown on the right. Bar naming convention is as follows: TO= Top-Outer, TI= Top-Inner, BO= Bottom-Outer and BI= Bottom-Inner. The remaining part of the slab-deck is clearly visible to the right. All dimensions are in mm.*

Average cross-section dimensions were 300x250 mm. All specimens contained approximately a 50 mm part of the slab deck. Spacing of the stirrups ranged between 100 and 450mm, in contrast to the typical 300mm spacing referenced on the drawings.

All specimens underwent visual inspection prior to testing, to record exact measurements of the cross-section geometry and locations (and width) of existing cracks. The average opening of the cracks was measured with an optical microscope and their location and length were documented and photographed. Data on cracks was used to group the anchorage zones into three different categories: anchorage zones with cracks smaller than 0.5 mm (C1), anchorage zones with cracks between 0.5 and 1 mm (C2), and anchorage zones with cracks with more than 1 mm of average opening (C3). Additional categories, 'reference' (R) and 'severely damaged' (S) were added to provide low and upper bound groupings of damage state.

## 3.2 Material properties

The original drawings cited an average concrete compressive strength of 30MPa and a nominal yield strength of reinforcement of 300MPa. This information was not consistent with test measurements from a field survey dated 1988. The steel yield point was recorded at approximately 250MPa, and the concrete compressive strength approximately 45MPa. The increased compressive capacity is consistent with the age of the bridge. Further material tests were therefore performed to reach a more comprehensive knowledge on the material properties of the specimens.

### 3.2.1 Concrete

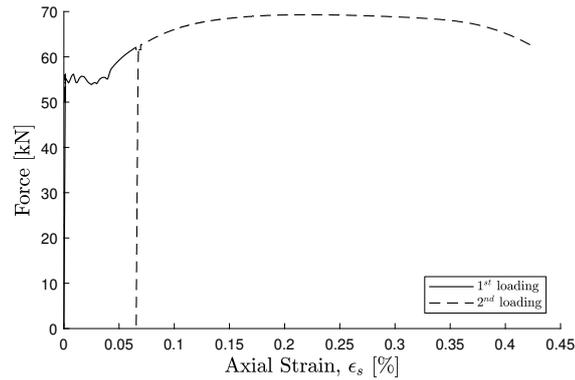
Eight concrete cores (100x200mm) were drilled according to EN 12504-1:2009 [12] and tested for cylindrical compressive strength,  $f_c$ , according to EN 12390-3:2009 [11]. The result was an average compressive strength of 45.6 MPa, with a standard deviation of 4.6 MPa.

Three additional cores (100X200mm) were tested to characterize the concrete compressive curve. The result was an average compressive strength of 47.6 MPa, with a standard deviation of 0.4 MPa, and an average elastic modulus of 27.4 GPa, with a standard deviation of 3.3 MPa. Wedge splitting tests are planned to characterize the behaviour of the concrete in tension. Further information on the concrete material properties can be found in *Paper I* (Section ??) and in the *Appendix* (Section A).

### 3.2.2 Reinforcement bars

After testing, the tensile reinforcement bars of the beams tested in 3-point bending were extracted, cleaned, 3D scanned, and tested in tension. The results were used to evaluate the bond strength of the bars in the beam tests. Additionally, uncorroded bars were extracted from untested segments of the bridge. The results from uncorroded bars were used to characterize the mechanical properties of the steel. From 27 tests, the tensile behaviour of the bars could be estimated to an average yield stress of  $259.6 \pm 10.1$  MPa. Further information can be found in *Paper II* (Section ??).

Two bars were subjected to additional tests so as to characterize the material properties of the steel. The bars were uncorroded and extracted from an untested segment of the bridge. Each bar was loaded first in tension until indication of strain hardening and then unloaded. The bars were then reloaded until rupture. Fig. 3.4 shows good agreement with the hypothesis of isotropic hardening, meaning that, by testing the already hardened bar in tension, it was possible to estimate, with good confidence, the force reached in the bar when the first test was interrupted.



**Figure 3.4:** Example of tensile test of an uncorroded reinforcement bar from Gullspång bridge: the same bar was loaded in two different stages to better capture the mechanical properties of the steel.

The rebar type and quality reflect common practice at time of construction. Bars were generally of lower capacity compared to the modern counterparts and anchorage was assured through use of end-hooks, when it was common to have low yield strength and anchorage was achieved by means of bending the bars in hooks.

### 3.3 Designing of the test setups

Testing specimens taken from decommissioned structures present some challenges. Test set-ups must be adapted to the given material and geometrical properties. Modifications often risk damaging the specimens and must be done carefully. Another complication is that the geometrical properties, such as concrete cover and stirrup spacing, often vary from specimen to specimen. Appropriate, non-destructive methods to investigate the characteristics of each specimen are needed. Design of the test set-up was divided in three main phases:

- Phase 1: a study was conducted into available literature and finite element (FE) modeling was employed to assess possible set-up configurations;
- Phase 2: three prospective test configurations were tested in-lab as part of a pilot investigation. The results of this study would inform the test procedure;
- Phase 3: The test set-ups for the experimental campaign was chosen by analysing and comparing the results of the pilot study.

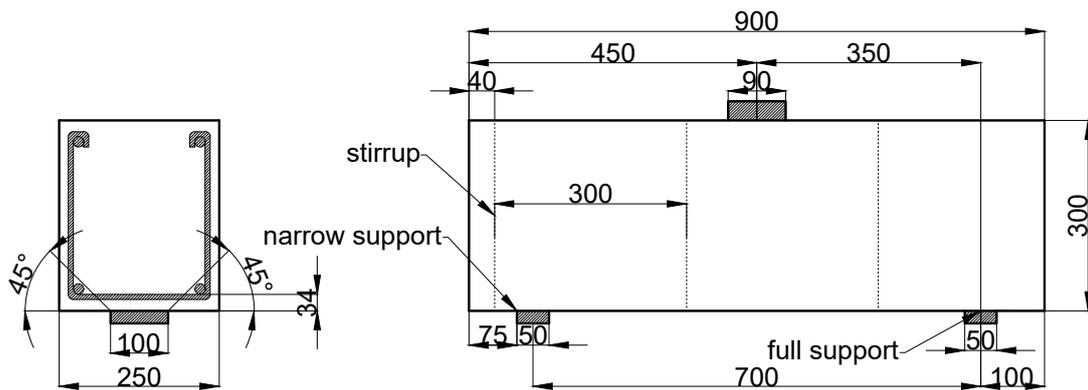
In the preliminary phase, the following general design requirements were established:

- The test set-up should have a high chance to reach anchorage failure;

- The test set-up must be simple and easy to carry out without need for strengthening of the specimen;
- The tensile reinforcements must be disturbed as little as possible;
- Clear boundary conditions must be established for comparison with a FE model;
- The test set-up must allow for testing both bottom-cast and top-cast bars.

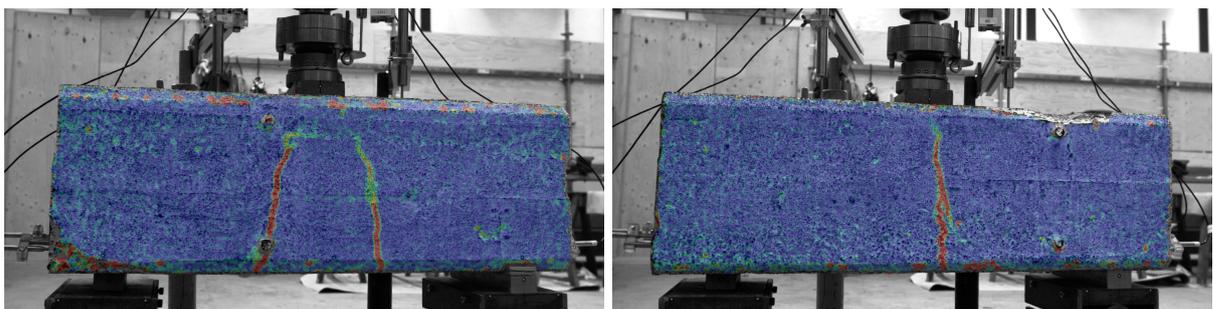
Two 4-point bending tests and a 3-point bending test were tested in a pilot study. Descriptions and details of the pilot testing is given in *Paper I* (Section ??). A directly supported 3-point bending test was selected to provide the best collection of results, owing to the simplicity of the test set-up and the well defined crack pattern, characterized by a single bending crack underneath the load plate. The 4-point bending test was not selected due to higher uncertainties in the crack pattern. Three bending cracks were observed in the high moment region before end-slip of the reinforcements, leading to higher scatter in the anchorage length of the bars.

Twenty beam specimens were tested. Figure 3.5 gives details on the beam geometry and the 3-point bending test configuration. The tested beams were 900 mm long, with a theoretical span of 700 mm. They were supported on one side by a narrow support (50x100 mm) and a full support (50x250mm) on the opposite side. Both the load plate and the two supports consisted of a steel block and a thin wood-fibre layer inserted between steel and concrete. The aim of the narrow support was to minimize the effect of support pressure. To meet this objective, a smaller support plate was positioned in the middle of the beam (between the bars) and specifically located on the side expected to have anchorage failure. The specimen was specifically cut to have a stirrup external to the narrow support to help redistribute the higher concentrated stresses generated by the support. On the opposite end, at the location of the full support, the ends of the reinforcing bars were restrained from anchorage failure by means of a bolt/washer configuration. However, out of the first four tests, two beams showed bending of the washers and unwanted slipping of the restrained tensile reinforcements. It was opted not to restrained the reinforcement bars, but to monitor the end-slip on both sides for subsequent beams.



**Figure 3.5:** *Experimental set-up and view of the narrow support for a beam tested upside-down compared to its position on the bridge.*

Due to the high cracking and low bending moment in the cross section of the beams, it became necessary to include the effect of bar yielding on the anchorage of the tensile reinforcements. Yielding of the reinforcement bars took place in all but one of the 20 beams tested<sup>1</sup>, the low yield strength of the tensile reinforcement resulting in the bars reaching yield stress near the time of opening of the first bending crack. Yielding, by reducing the diameter of the bar, results in a local loss of bond strength. Slipping of the tensile reinforcement took place after the opening of one or two major bending cracks and yielding of the tensile reinforcements. In Fig.3.6, the typical crack patterns exhibited by the specimens are shown. With the given design of the beams and the condition of the specimen, it was not possible to initiate shear failure before anchorage failure.



**Figure 3.6:** *Beam 17H (Left) and Beam 9B (Right): example of typical crack patterns, as captured by Digital Image Correlation (DIC) cameras.*

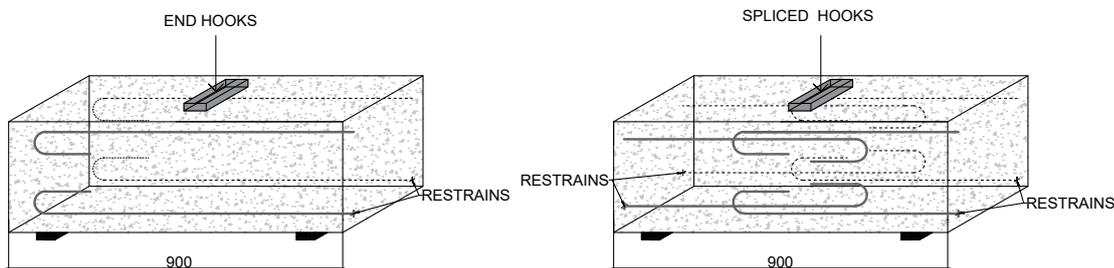
The second series of tests designed for the specimen from Gullspång bridge consisted of seven 3-point bending tests where the bars were bent into hooks.

<sup>1</sup> A total of 4 rebars slipped without yielding in the anchorage zone. However, only one beam (10I) had both the tensile reinforcement bars failing in anchorage without yielding

This is typical of anchorage regions in structures with plain bars. The hooks appeared in one of two configurations (See Fig.3.7):

- The tensile reinforcement bars have a single pair of hooks at one end. This is typical for specimens cut from the ends of the edge beams;
- The tensile reinforcement bars are spliced, and two pairs of hooks are present. This was a typical feature every 6-8 m along the length of the edge beams, given the maximum length limitations of rebars at the time of construction.

The test configuration is similar to that used for the beams without hooks. One exception is that all rebars were restrained at the ends through use of a nut/washer fixture. The objective was to assess the anchorage capacity of the hooks and not the end-slip of the straight reinforcement. At the time of printing of this thesis, the beams were already tested, but data on the corrosion level of the bars are still to be obtained. The tensile reinforcement bars are to be cleaned and 3D scanned, and the results are to be processed. The two categories of beams exhibited different failure modes in the structural tests. The 3 beams with spliced hooks failed in shear. The 4 beams with end-hooks failed in bending. Failure of the hooks was not observed in any of the tests.



**Figure 3.7:** Test set-up for beams with end hooks (left) and spliced hooks (right). All measurements in mm.

The third series of tests was designed to avoid the situation of reinforcement reaching the yield point. Direct pull-out tests were performed using cut sections (50, 75 and 100 mm in thickness ) from untested segments of the edge beams from Gullspång bridge. The test set-up required drilling, threading, and inserting a threaded rod in the individual rebars. Thereafter, each individual rebar was pulled out using a hydraulic load cell. A special rig was designed and produced (See Fig.3.8). This consisted of three legs, two of which could be length-adjusted to account for any skew angle between bar and cut concrete surface. The need of avoiding yielding of bars gave an upper limit of 100 mm embedment length. The lower limit was set to 50 mm, for feasible cutting of concrete slices and drilling (20 mm deep) not to affect the major part of the length. Based on these arguments,

embedment lengths of 50, 75, and 100 mm were chosen. There was a total of 174 bars, of which 104 were in concrete without visible damage, 35 had cracks, and 35 had some spalling of the concrete cover. At the time of printing of this thesis, all the reinforcement bars were tested, extracted from the cross-sections, and are in the process of being cleaned and 3D scanned. A rough first approximation of the bond stress at peak shows value comparable to the flexural tests in the first test series and a large scatter.



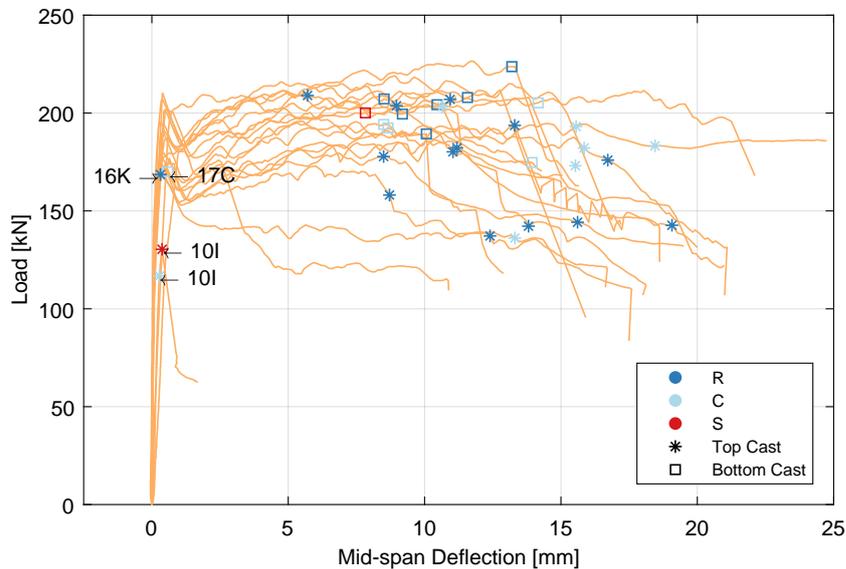
**Figure 3.8:** *Pull-out test set-up*



## 4 Results

In this section, an overview of the 3-point bending test series is provided and discussed. These results are presented and discussed further in *Paper II* (Section ??). A total of 20 beams were tested in 3-point bending. Slip of the tensile reinforcement bars was observed in 18 of them, after the opening of one or two bending cracks. The other two beams experienced rupture of the bolt/washer system used in an attempt to prevent anchorage failure on one end of the beam. Section 3.3 details the rationale behind this restraint system. 9 beams were tested as positioned on the bridge, and 11 were tested upside-down.

In Fig.4.1, mid-span deflection is plotted as a function of the applied load. The applied load was sourced from the hydraulic load cell and the mid-span deflection was extrapolated from data obtained with Digital Image Correlation. The initiation of slip of the reinforcement bars, for each test specimen, is marked. Colors are used to differentiate the observed external damage respective to the concrete surrounding the individual rebars and the marker symbol indicates top-cast and bottom-cast positioning.



**Figure 4.1:** Load-Mid-span deflection curves for the 3-point bending tests. The markers show where each bar started to slip; colours indicate the level of damage in the anchorage zone. The influence of the cast position is shown by asterisk for top-cast and squares for bottom-cast bars. The names of the beams where end-slip of one or more bars took place without yielding are marked.

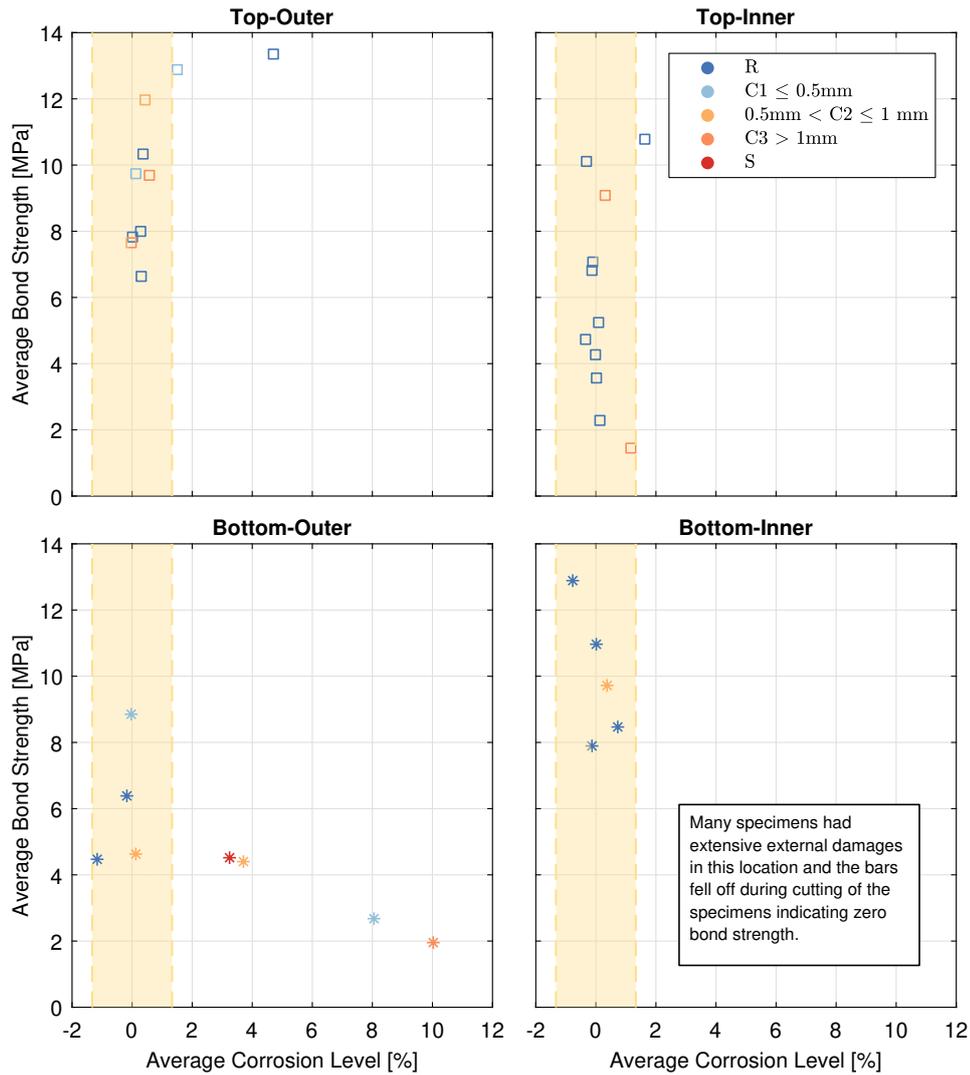
All the beams were characterized by the opening of one, or maximum two, major bending cracks localized in the mid span. In most cases, yielding of tensile reinforcement was noted shortly after cracking; though, anchorage failure did occur for 4 bars in three beams preceding tensile yielding. In all other bars, the force was increased beyond strain-hardening condition of the reinforcement before the eventual slip of rebars and loss of load. Thus, the bars started to slip at a point when other failure mechanisms had already initiated. The anchorage capacity did not limit the load-carrying capacity of the specimen. Thus, the anchorage was enough to carry the yield force. End-slip was observed at a later stage, when yielding of the reinforcements locally decreased the bond strength. The deformation capacity was therefore affected by the anchorage failure.

After the 3-point bending tests, the tensile reinforcement bars were extracted from the beams, cleaned and 3D scanned. Measurements from the 3D scan data were used to estimate the corrosion level as the loss of cross sectional area. 3D scanning allowed as well for the identification of the yielded length of the bar. To account for the effect of yielding on the bond strength, a 'yielded' zone was distinguished from an 'unyielded' zone. The bond strength was evaluated separately for each zone, using data acquired from tensile testing of the tensile reinforcement bars. *Paper II* (Section ??) contains detailed information on the evaluation of the bond strength.

In Fig.4.2, the average corrosion level of the bars is presented and plotted against the calculated bond strength in the unyielded zone. Different colours denote the deterioration classification introduced in Section 3.1, based on visual inspection of the beams prior to testing. Separating the results by bar position highlights the influence of casting position on the bond strength. This showed to be a critical factor relative to maximum average bond strength.

Considering first the bars with no relevant corrosion amounts, top-cast bars were found to have a lower bond strength than bottom-cast bars. This is believed to be due to different densities of the surrounding concrete, where lower density concrete provides less confining pressure and thus reduced friction. Considering top-cast bars with small amounts of corrosion product, the bond strength is found to increase up to values comparable with the bond strength of bottom-cast bars without corrosion. Top-cast bars present as well less external cracks than bottom-cast bars for the same corrosion level. Bottom-cast bars are shown to lose bond strength with increasing corrosion levels.

It is interesting to note that the average bond strength calculated from the results of the experiment was significantly higher than data available in literature. This conclusion even acknowledges the high scatter in the results. The average bond strength of the bars was calculated as 7.39 MPa, with a standard deviation of 3.33 MPa; in contrast, Cairns et al. [6] presents measured bond strengths between 1 and 3 MPa. One explanation for the different outcomes between this study and the literature is that no previous study made use of naturally

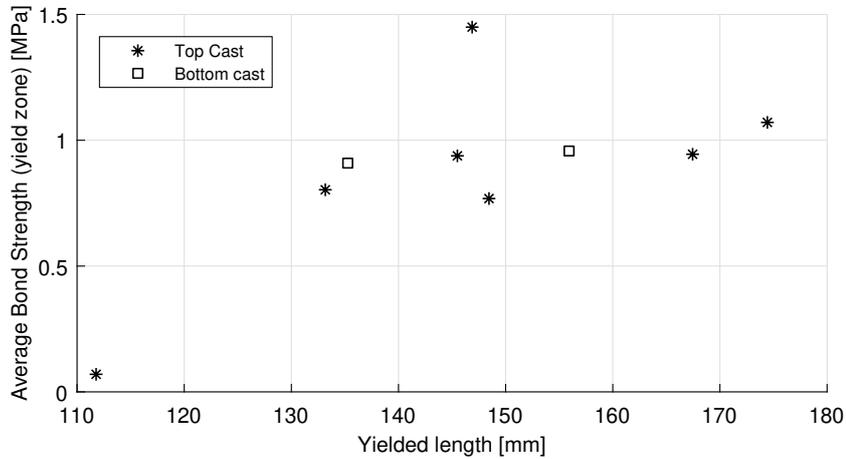


**Figure 4.2:** The average bond strength in the unyielded zone is plotted against the average corrosion level. Different colours are used to indicate the level of damage in the anchorage zone visible before the test. Results from different positions in the cross section are displayed separately: top-outer (left, top), top-inner (right, top), bottom-outer (bottom, left), bottom-inner (bottom, right). The shadowed area represents the accuracy in the evaluation of the corrosion level at zero, due to the uncertainties in the reference area of the uncorroded bars

corroded specimens. Factors to be expected to contribute to this difference are as well: the difference in casting techniques (concrete in 1930s was usually stamped instead of being vibrated), the surface roughness of the bars, the presence of active support pressure, and the evaluation method.

In Fig. 4.3, the average corrosion level of the bars is presented and plotted against the calculated bond strength in the yielded area. The results presented a rather small scatter, with an average of about 1 MPa. They also show a noticeable

loss of bond strength after yielding (about 90% loss compared to the bond strength in the unyielded zone of the same rebar). However, less data were available due to damages in the extraction process of the bars.



**Figure 4.3:** *The average bond strength in the yielded zone is plotted against the length of the yielded zone in the tested bar. The influence of the cast position is shown by asterisk for top-cast and squares for bottom-cast bars.*

# 5 Conclusions and Outlook

## 5.1 General conclusions

The scope of this work was to investigate the bond of plain reinforcement bars subjected to natural corrosion damage in concrete. The major outcomes of this investigation are as follows:

- Testing of specimens sourced from decommissioned structures allows for a comprehensive overview of material and geometrical characteristics typical of elder constructions. Some of these aspects, such as surface roughness of the reinforcement bars, different casting techniques, and aggregate sizes may lead to differing results as compared to tests on newly produced specimens;
- Testing of specimens sourced from existing structures requires adaptations of conventional test configurations to meet the particular conditions of the sample specimen. This introduces additional challenges. More than one series of tests may be needed to fully characterize the intended phenomena;

From the outcome of the first test series, where 3-point bending tests were conducted on twenty, naturally corroded beams, the following conclusions were drawn:

- The loss of bond strength due to yielding of the bar was the main cause of anchorage loss in the tensile reinforcements. The bond strength in the yielded area was equal to about 10% of the bond strength in the unyielded zone of the same rebar;
- The casting position was recognized as an important factor for the bond of plain bars, attributed to concrete density. The different casting positions interact differently with the corrosion;
- Bottom-cast bars have a higher bond strength than top-cast bars when negligible amounts of corrosion are present;
- Bottom-cast bars lose bond strength with increasing corrosion level. The concrete surrounding the bar was found prone to spalling cracks for small amounts of corrosion;
- Top-cast bars gain bond strength with small amounts of corrosion. The bond strength of corroded top-cast bars was in the range of the bond strength of uncorroded bottom-cast bars;

## 5.2 Outlook (and future research)

The following is suggested for future research:

- To analyze the results from the second and third test series, briefly introduced in this thesis (beams with hooks and pull-out tests). This will provide further knowledge on the anchorage and the bond of plain reinforcement bars, and on how it is affected by corrosion.
- The impact of additional confinement caused by the supports in the 3-point bending test can be evaluated numerically using non-linear, FE modeling;
- Results from the experimental study can be used to calibrate a bond-slip model to be used in FE modelling for the assessment of structures with plain reinforcement bars, as a function of the corrosion level;
- Visible, external damage can be linked to the anchorage capacity of naturally corroded plain reinforcement bars;
- Advanced, non-destructive tomography techniques, such as neutron imaging, could be used to observe the distribution and the composition of the corrosion product in naturally corroded specimens. The results can be compared to artificially corroded specimens, possibly with similar properties;
- Additional tests of specimens with naturally corroded plain bars can help validate the results of this work. The specimens can be taken from a different structure, possibly exposed to a different environment;
- Tests on artificially corroded specimens would make an interesting comparison with the results of this work. The specimens can have similar characteristics to the specimens used in this work. Influencing factors such as surface roughness of the bars and different casting techniques can be investigated in a controlled environment.

# Appendices



# Appendix A

## Material properties of the concrete

In this appendix the results of the tests characterising the compressive behaviour of the concrete are given. Two different tests results are presented: the cylindrical compressive strength and the stress-strain relationship in compression.

### A.1 Cylindrical compressive strength

Eight concrete cores (100x200mm) were drilled according to EN 12504-1:2009 [12] and tested for cylindrical compressive strength,  $f_c$ , according to EN 12390-3:2009 [11]. The result was an average compressive strength of 45.6 MPa, with a standard deviation of 4.6 MPa (See Table A.1).

**Table A.1:** *Cylindrical compressive strength,  $f_c$ , results.*

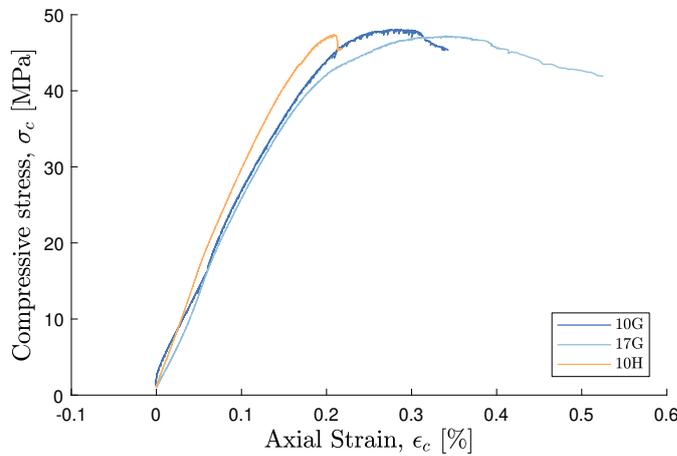
Core ID	Height [mm]	Diameter [mm]	Density [ $kg/m^3$ ]	Compressive force [kN]	$f_c$ [MPa]
17D	198	100	2400	402	51.0
10G-A	200	100	2380	378	48.0
14D	198	100	2380	346	43.9
18A	197	100	2360	388	48.9
10C	199	100	2380	358	45.4
16B	196	100	2390	293	37.1
14H	198	100	2370	382	48.5
10G-B	200	100	2380	328	41.6
Average	198	100	2380	359	45.6
Std	1.4	0	12	36	4.6

## A.2 Stress-strain relationship in compression

Three cores (100X200mm) were tested to characterize the concrete compressive curve. The stabilized secant modulus of elasticity  $E_{c,s}$  was measured according to EN12390-13:2013 (method B). After determination of the modulus of elasticity, the stress-strain diagram in compression were obtained in accordance with EN12390-3:2009 [11] and EN12390-13:2013 [10], with the exception that the load was applied in displacement-control with a displacement rate of 0.2 mm/min. The compressive strength,  $f_c$ , was defined as the peak stress shown on the compressive curve. The results of these tests are given in Fig. A.1 and Table A.2.

**Table A.2:** *Cylindrical compressive curves results.*

Core ID	Height [mm]	Diameter [mm]	Density [ $kg/m^3$ ]	$E_{c,s}$ [GPa]	$f_c$ [MPa]
17G	194.2	100.1	2350	25.1	47.3
10H	198.3	100.1	2381	31.1	47.4
10G	195.2	100.1	2365	26	48.1
Average	195.9	100.1	2365	27.4	47.6
Std	2.1	0	16	3.3	0.4



**Figure A.1:** *Stress-strain curves of tested cores 17G, 10H, and 10G.*

Two of the three tested cores showed a change in the elastic slope at low stresses. This is not usual, however, there are two possible explanations to this behaviour. A change of stiffness in the ascending branch of the stress-strain curve often indicates frost-damage in the concrete sample [18]. This is likely linked to the presence of randomly oriented micro-cracks prior testing, resulting from freeze-thaw cycles. Large strains at peak stresses and a more ductile behavior are generally as well

associated with frost-damage concrete. However, Hanjari et al. [18] observed a significant decrease of the compressive strength at peak stress in presence of frost damages. This did not apply to the three tested specimens. There was no significant difference in the compressive strength at peak between the three tested samples, even if only two of the three showed changes in the ascending branch of the elastic slope. A second reason for such behavior could be found in the unevenness of the external surface of the cores. At the surface, cut-off parts of large aggregates were observed. This may have introduced uneven deformations in the core. Wedge-splitting tests are planned for characterizing the behaviour of the concrete in tension. This will allow to further investigate this behaviour.



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