A MODEL FOR THE ANCHORAGE OF CORRODED REINFORCEMENT: VALIDATION AND APPLICATION

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Abstract

When reinforcement in concrete corrodes, splitting stresses around corroded bars may lead to cover cracking and even cover spalling, affecting the anchorage. The aim of this study was to validate an existing one-dimensional (1D) analysis for anchorage capacity, and to show how it can be applied in assessment of existing bridges. The 1D analysis was validated through comparisons to experiments and detailed 3D finite element analyses. The methodology is exemplified in assessment of two bridges built in the 1960s. The bridges exhibit systematic damage in the form of spalled concrete on the bottom side of the main beams at cast joints where large amounts of reinforcement are spliced. The anchorage length needed to anchor the yield force was calculated from the bond-slip response, using the one-dimensional bond-slip differential equation. The model proved to be easy to use in practical design work. Furthermore, the bridges could be shown to have sufficient capacity, and costly strengthening could be avoided. This work clearly demonstrates the potential to certify sufficient load-carrying capacity of corroded reinforced concrete structures through improved models.

Keywords: Anchorage, Assessment, Corrosion, Bond-slip, Concrete, Reinforcement

1 Introduction

Infrastructures represent a large capital investment in all developed countries. To establish sustainable development, it is of great importance that the investments result in safe structures with predictable performance. Despite significant advances in construction design and practice, corrosion in reinforced concrete (RC) structures is still a leading cause of deterioration worldwide (Sustainable Bridges 2007). This situation has led to a growing demand for better assessment of existing concrete structures and has revealed a need for an improved understanding of the structural effects of corrosion.

Corrosion of the steel reinforcement has two major effects: 1) Reduction of the effective rebar area, and 2) change of bond properties between the reinforcement and the concrete, which is the topic of this paper. This has been studied by many researchers; for a state-of-the-art report see fib (2000). Some researchers, e.g. Lee et al. (2002), have proposed functions for bond capacity versus corrosion level based on experiments. Others, e.g. Coronelli (2002) and Wang & Liu (2006), have proposed analytical models for calculating the bond strength. Berra et al. (2003) and Lundgren (2005) have used detailed finite element modelling to investigate the bond mechanism for corroded bars in concrete, in particular the effect of splitting stresses induced in the concrete by the volume increase of the corrosion products. However, this type of detailed three-dimensional (3D) modelling of the region around reinforcement bars is impractical for analysis of complete structures. Thus, there is a need for a general simple model predicting the bond-slip behaviour for corroded bars, as the one presented in Lundgren et al. (2012). The aim of the present study was to validate this one-dimensional (1D) analysis for anchorage capacity, and to show how it has been applied in assessment of existing structures.

2 Bond-slip model for corroded reinforcement

In the following, the bond-slip model is shortly presented, for details see Lundgren (2012). It is an extension of the bond-slip model given in the CEB-FIP Model Code 1990, CEB (1993). In Lundgren (2012), this is reformulated into a plasticity model. Here, a less formal way of presenting the model is chosen; however, for implementation purposes, the description in Lundgren (2012) is to be preferred.

In CEB(1993), parameters for the extreme cases "confined" (i.e. ductile pull-out failure) and "unconfined" (i.e. brittle failure due to cover cracking induced by the radial tensile stress) are given. Lundgren (2012) describes a method to interpolate for intermediate cases, which often occur in practice. An interpolation factor k_{uncor} is determined by

$$k_{uncor} = \max\begin{cases} k_{c/d} \\ k_{Asw} \end{cases} \tag{1}$$

where $k_{c/d}$ is a factor that depends on the cover to bar diameter ratio, and k_{Asw} is a factor that depends on the amount of transverse reinforcement A_{sw}/s according to Fig. 1.

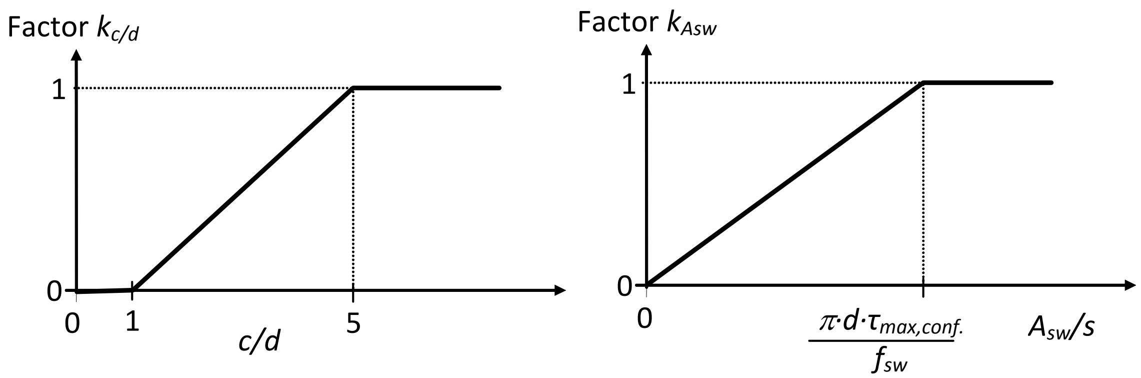

Fig. 1 Factors for interpolation between "confined" and "unconfined" case.

For corroding reinforcement, the failure mode can change from pull-out to splitting failure, unless sufficient confinement is provided by efficient transverse reinforcement. This change of failure mode is here accounted for by an interpolation factor k_{cor} that depends on the ratio x/x_{cr}, where x is the actual corrosion penetration and x_{cr} is the corrosion penetration that causes cover cracking. Thus, when the cover is cracked, the remaining bond capacity depends only on the transverse reinforcement. Before the cover is cracked, the cover also contributes to the capacity. To get a numerically stable modelling, a linear decrease from the capacity including the effect of the cover to the capacity only due to the transverse reinforcement is assumed to take place from a corrosion penetration of 85% of cover cracking, see Fig. 2. The bond-slip is assumed to be the weighted sum of the bond-slip curves for the "confined" and "unconfined" cases according to

$$\tau_b = k_{cor} \cdot \tau_{b,conf} + \left(1 - k_{cor}\right) \cdot \tau_{b,unconf} \ . \tag{2}$$

An example is shown in Fig. 3. The corrosion penetration that causes cover cracking was found by curve fitting to analysis results from the detailed 3D model in Lundgren (2005), as given by the following formula:

$$x_{cr} = 11 \cdot \left(\frac{f_{cc}}{40}\right)^{0.8} \cdot \left(\frac{c}{d}\right)^{1.5} \cdot \left(\frac{d}{16}\right)^{0.5} \tag{3}$$

where x_{cr} is the corrosion level that cracks the cover in µm, f_{cc} is the concrete cylinder compressive strength in MPa, c is the concrete cover in mm, and d is the reinforcement bar diameter in mm.

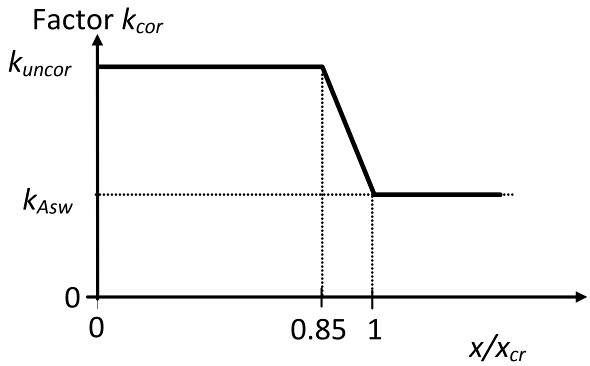

Fig. 2. Factor to take a change of failure mode into account for corroding reinforcement.

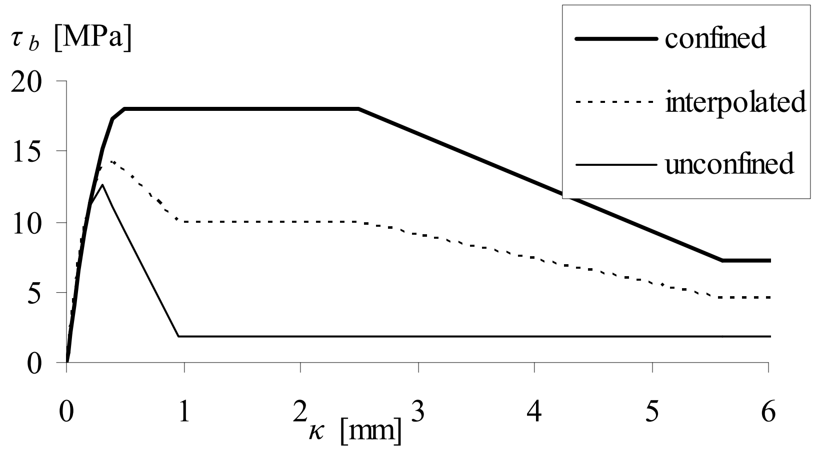

Fig. 3. Bond strength plotted versus hardening parameter for good bond, an assumed clear rib spacing of 5.8 mm and a compressive strength of 40 MPa. An example of an interpolated curve is also shown; in this case the interpolation factor k_{cor} is 0.5.

From the results presented by Soto San Roman (2006), it was noted that it was possible to obtain the bond-slip response of corroded reinforcement by shifting the bond-slip curve of uncorroded reinforcement along the slip axis. The explanation is the exhaustion of the confinement resistance by splitting stresses and cracking induced by the corrosion pressure, which is an effect similar to that produced by bar slipping. In Lundgren (2012) this observation is introduced in the plasticity formulation of the CEB-FIP bond-slip model by making the hardening parameter dependent on both the plastic slip and the corrosion penetration. With the less formal way to present the model chosen in this paper, the bond-slip curve for corroded reinforcement is simply obtained by shifting the interpolated bond-slip curve by ax along the slip axis, as shown in Fig. 4. The parameter a is assumed to be a constant, around 8.1, according to Schlune (2006).

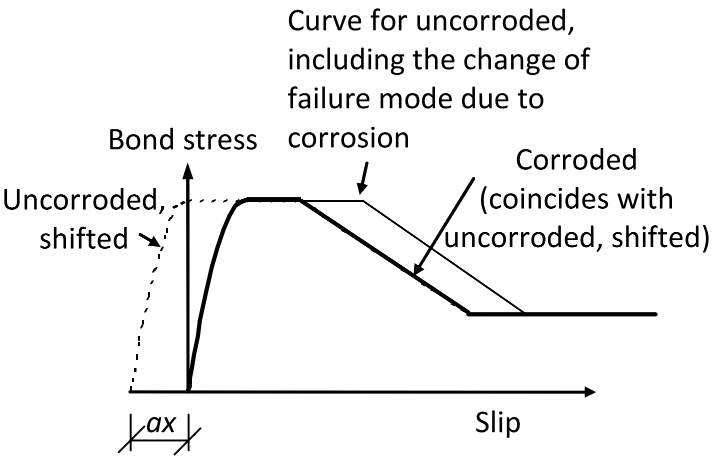

Fig. 4. The proposed plasticity model is equivalent to using a "master curve" and adjusting the slip level to the amount of corrosion.

3 Validation of model

In Lundgren (2012), the model was shown to give a qualitatively reasonable response compared to experiments; i.e. the results are consistent with the physical behaviour. It was also shown to give results that are on the safe side for most cases; however, concerns were raised for cases with large corrosion penetrations and no or small amount of transverse reinforcement. I.e., concerns whether it will be safe to use the model to estimate required anchorage lengths if the concrete cover has spalled off totally, were raised. Therefore, the anchorage capacity in reinforced concrete structures with corrosion-induced cover spalling was specially studied in this work. The model was validated through three series of experiments and an empirical model available in the literature for the anchorage capacity of corroded RC structures with cover spalling. It was also compared with detailed 3D FE analyses, described in Zandi & Lundgren (2015).

The test series used were: the eccentric pull-out tests and beams with lap splices in regions of constant moment by Regan & Kennedy Reid (2009), and the eccentric pull-out tests by Coronelli et al. (2013) and Zandi Hanjari et al. (2011). The empirical model for anchorage capacity after cover spalling devised by Regan & Kennedy Reid (2009) is also evaluated with respect to the analyses and experiments.

The eccentric pull-out tests by Regan and Kennedy Reid (2009) were cast with three concrete covers of $c = 1.5d$ as 'reference' specimens, $c = 0.5d$ in which the cover was 'flush' with the concrete surface, and $c = 0$ where the cover was exposed to 'mid-barrel'. The results in terms of bond strength for varying transverse reinforcement contents are presented in Fig. 5 and compared to the 1D and 3D FE analyses. The results of the empirical models of Regan & Kennedy Reid are also included in the graph; as their experiments were part of the database from which the empirical models were calibrated; good agreement between those can be expected. The 1D analysis under 'good' bond conditions is on the safe side compared to experimental data for 'flush' and its corresponding empirical model; whereas the 1D analysis under 'all other' bond conditions seems to be on the safe side only for transverse reinforcement content higher than 0.01 compared to experimental data for 'mid-barrel' and its corresponding empirical model. The two spalling patterns in the experimental data and empirical model, 'flush' and 'mid-barrel', as well as the two bond conditions in 1D analysis, 'good' and 'all other', identify the extreme cases, whereas, an intermediate condition is most likely in reality based on field observations.

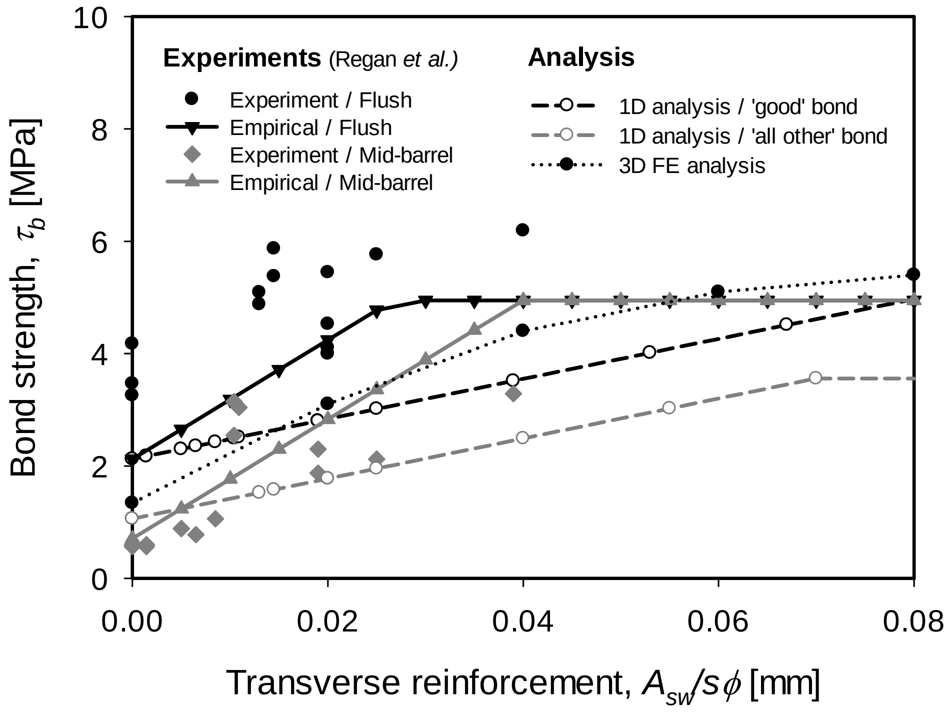

Fig. 5 Analysis of eccentric pull-out tests by Regan and Kennedy Reid (2009) for varying transverse reinforcement contents.

The same three concrete covers as in the pull-out tests were included in beam tests with splices in the region of constant moment by Regan and Kennedy Reid (2009). The compressive strength of concrete varied slightly in the tests which seems partly the reason for the scatter seen in Fig. 6. The corresponding 1D analysis was therefore carried out with the highest and lowest compressive strength resulting in upper and lower limits, respectively. As before, the two spalling patterns of ‘flush’ and ‘mid-barrel’ seem to be the two extremes and the 1D analysis seems to make safe predictions of bond strength at cover spalling.

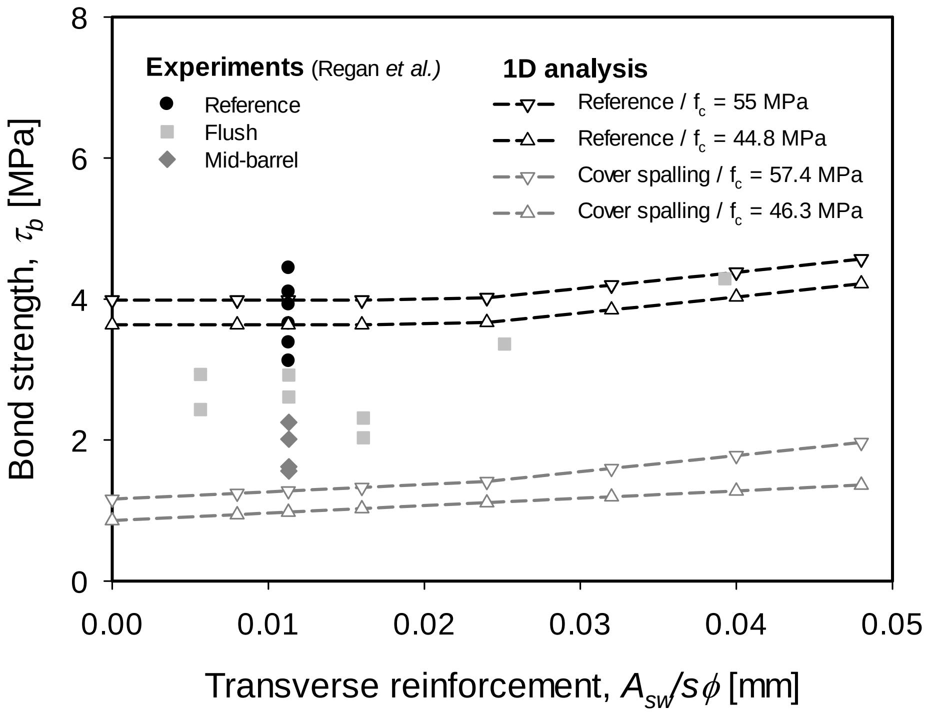

Fig. 6 Analysis of beams with lap splices in region of constant moment tested by Regan & Kennedy Reid (2009) for varying transverse reinforcement contents.

The last series of experiments used here for comparison purposes were the pull-out tests by Coronelli et al. (2013) and Zandi Hanjari et al. (2011). These were studied using 1D and 3D FE analyses over a wider range of corrosion, 0-20% weight loss, and were compared to the experiments, the empirical model of Regan & Kennedy Reid, as well as the provisions given in fib Model Code 2010 (2013), see Fig. 7. The results normalized with respect to the bond strength experimentally measured for uncorroded specimens are shown separately for Type I and II specimens and for middle and corner bars in Fig. 7. Overall, the estimations made with all methods and models show a similar trend and fall on the safe side when compared with the experiments. The bond strength calculated using 1D and 3D FE analyses in most cases show a margin of safety for both low corrosion when compared with Model Code 2010 and for high corrosion when compared with the empirical model of Regan & Kennedy Reid. A major shortcoming inherent in the provisions of Model Code 2010 is the lack of indicative values to predict the bond strength beyond 5% corrosion, whereas the analyses presented here illuminate the importance in particular of specimens with transverse reinforcement, see Figures 7c-d. Moreover, it seems appropriate to further differentiate between the bond strength for bars located in the middle or corner positions and to more precisely quantify the influence of transverse reinforcement.

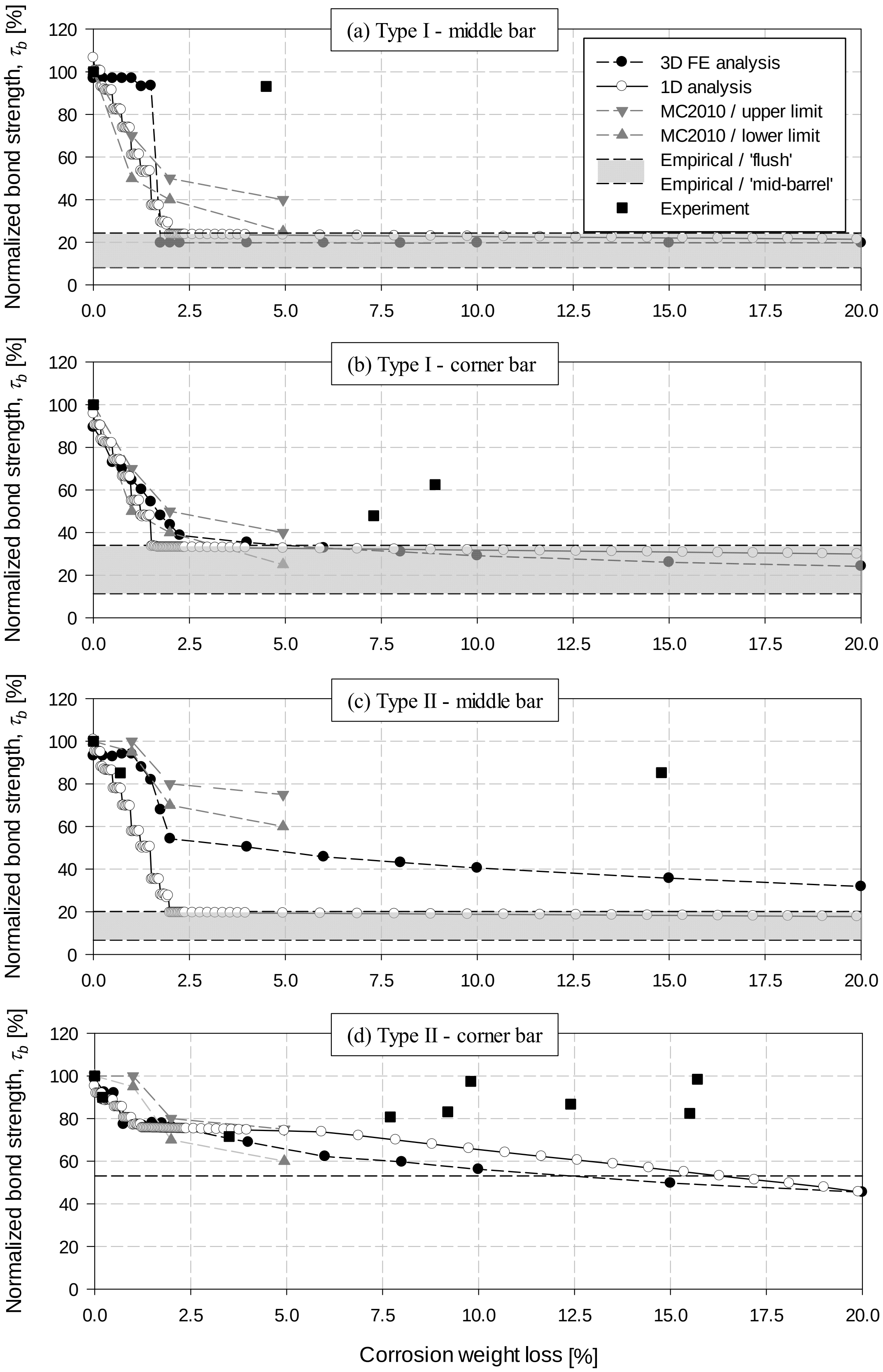

Fig. 7 Analysis of eccentric pull-out tests by Coronelli et al. (2013) and Zandi Hanjari et al. (2011) for varying corrosion levels. Type I specimens had no stirrups, whereas Type II specimens had four stirrups along the embedment length.

4 Application of model in assessment of two bridges

The methodology is exemplified in assessment of two bridges, Blommenbergs- and "Gröndalsviadukten", bridges in Stockholm built in the 1960s. For details about the assessment see Nilsson & Lundgren (2014a, b). The bridges were built in phases; the first phase consisted of columns, cross beam, and a part of the superstructure, see Fig. 8. Thereby, a cast joint was placed at each main beam on either side of each row of columns. As can be seen in Fig. 8, this led to that large amounts of reinforcement were spliced at each cast joint. Today, the bridges exhibit systematic damage in the form of spalled concrete on the bottom side of the main beams at these cast joints; an example is shown in Fig. 8.

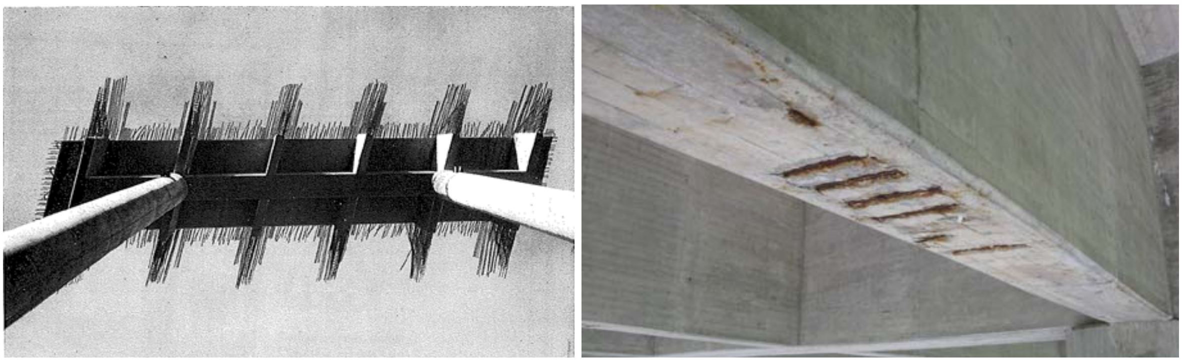

Fig. 8 Left: Photo from the time of construction, showing the large amount of reinforcement spliced at the cast joints. Right: Example of a damage with spalled cover at a cast joint at a bridge today.

At the assessment of the bridges, sufficient capacity could be shown if the structure was assumed to be undamaged. Considering the visible damages, this was however considered to be an unrealistic assumption. As the documented damages are located close to points where the bending moment is zero, it was first examined whether a simplified assumption that the bond strength was zero in the damaged areas would be enough; however, this conservative assumption resulted in insufficient capacity. Accordingly, a more detailed investigation on the anchorage in the damaged sections was needed.

It should be noted that for other damaged structures, reinforcement corrosion affects not only the anchorage capacity, but also the load-carrying capacity in other ways. If corrosion causes cover spalling on the compressed side, the internal lever arm should be accordingly reduced when calculating the bending moment capacity; however in this case, the damage was on the tensile side and didn't affect the inner lever arm. Furthermore, corroding tensile reinforcement obviously decreases the bending moment capacity due to the reduction of reinforcement area; however, in these bridges, it was judged from the inspection that the main reinforcement have not yet begun to corrode, only the stirrups. For these reasons, the anchorage of the main bars was in focus in the assessment.

The model in Lundgren et al. (2012) was applied. The bond versus slip was obtained as described in section 2, and the anchorage length was calculated by numerical solving the basic 1D bond-slip differential equation along a bar in a Matlab routine. The following basic assumptions were used:

- Design values of strength were used for both reinforcement and concrete.
- As the model is based on the bond-slip model in CEB-FIP Model Code 1990, CEB (1993), an assumption regarding bond conditions need to be done. It was assumed to be "all other bond conditions"; thus roughly half the capacity of "good" bond conditions.
- The transverse reinforcement was assumed to be corroded, decreasing its diameter from 10 to 9 mm
- Anchorage was assumed to be affected by spalling in the bottom one-two layers of reinforcement; it was in these layers the reinforcement is very dense.
- As the corrosion level was unknown, the maximum anchorage length for varying corrosion penetration levels was chosen; in the example in Fig. 9 this results in 1472 mm.

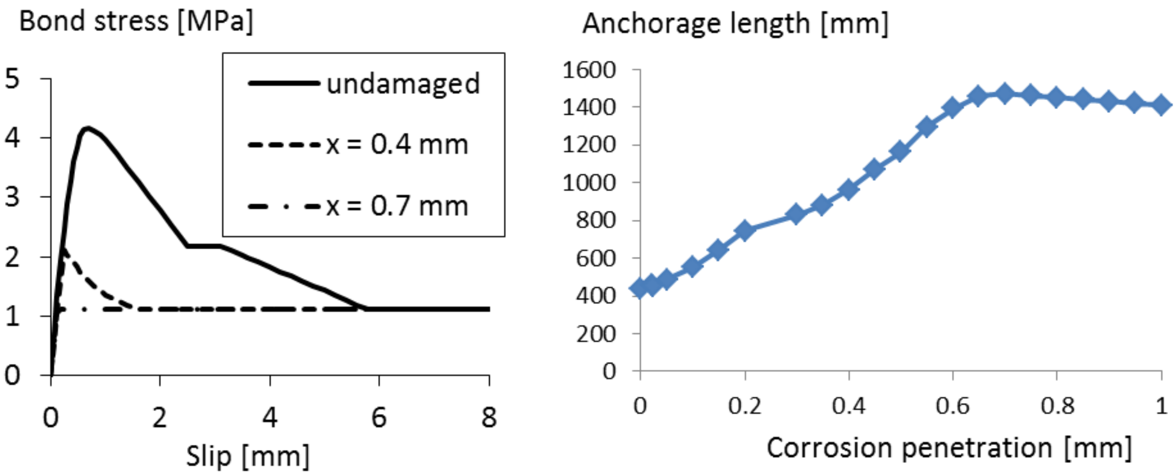

Fig. 9 Left: Bond versus slip for some varying corrosion levels, x is corrosion penetration from each side of a reinforcement bar. Right: Anchorage length depending on corrosion penetration resulting from numerical solving of the basic 1D bond-slip differential equation along a bar, using the bond-slip relation shown in the left graph as input.

The anchorage length was used to calculate the tensile capacity of the reinforcement in the splice; the stress increase was assumed to be linear. The effect on structural level is exemplified in Fig. 10, where the bending moment in one beam in one span in one of the bridges is shown. As can be seen, the capacity could with the more detailed investigation in this example be shown to be sufficient. The same result was true also for all other beams in all spans except one. There, the difference between the required bending moment and capacity was small, and could be considered to be negligible provided no damage is visible on this particular place, which accordingly should be checked.

The conclusion of the detailed assessment was, therefore, that both bridges were judged to have sufficient capacity; however, a new, more detailed inspection of damages at all cast joints should be performed. In this detailed inspection, the damage location relative to the reinforcement splices, both longitudinally and in height will be determined. Thus, most probable, expensive strengthening can be avoided. Instead of strengthening the bridges at an estimated cost of 46 million SEK, the Swedish Transport Administration estimates that maintenance will cost 19 million SEK. Thus, this investigation, at a cost of 0.2 million SEK, has resulted in an estimated saving of 27 million SEK.

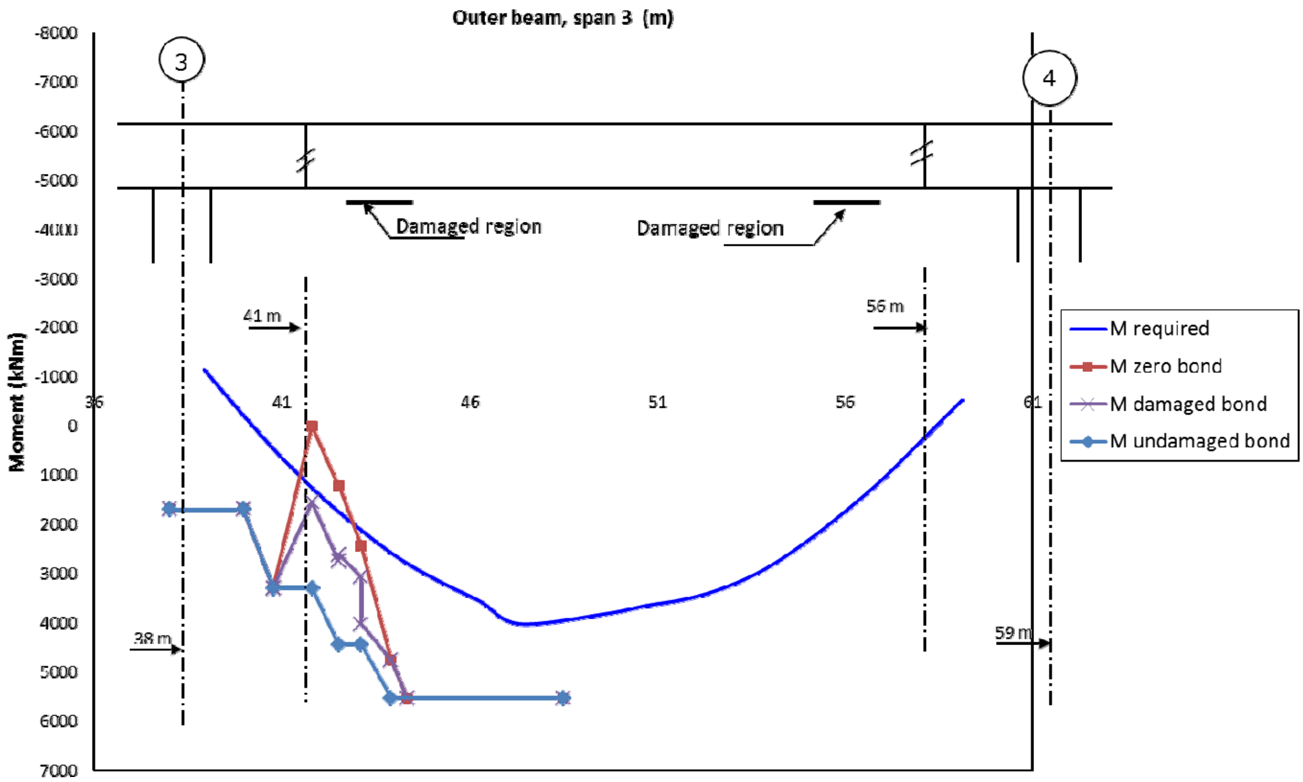

Fig. 10 Bending moment in one span in one of the beams in one of the bridges.

5 Conclusions and outlook

The bond-slip model from Lundgren (2012) was validated for anchorage in RC structures with cover spalling. The validation was conducted through a comparison to 3D FE analysis and experiments. Thereby, sufficient knowledge and models exist to calculate the anchorage capacity in concrete structures damaged by reinforcement corrosion. The model proved to be very useful in practical engineering work. Application to two bridges showed its potential to demonstrate sufficient load-carrying capacity, and thereby avoiding costly strengthening. The economical saving is around 27 million SEK for the two studied bridges only. For the bridges in question, a more detailed damage mapping will be performed, focusing on details that showed to be critical in this evaluation.

A problem in the assessment of existing structures is to evaluate the current corrosion penetration. In the assessment of the bridges presented here, the very conservative assumption that the anchorage length is the longest expected during the life time was used; for increased corrosion levels the failure mode would instead be rupture of the reinforcement bars. For the investigated bridges, sufficient capacity could be demonstrated despite this conservative assumption. For other cases, and especially for other failure modes such as bending and shear with the reinforcement amount limiting, the corrosion penetration must be estimated. Measurement methods for the corrosion rate exist; however as the corrosion rate typically varies over time, and as the measurements must be combined with assumptions about how long time the corrosion has progressed, the resulting corrosion penetrations become very uncertain. In an ongoing research project, the real corrosion penetration will be measured in a relatively large number of specimens, and the results will be correlated with the visible damages in the form of crack pattern and crack widths. In this way, we hope to develop methods to link visible damages to the effect on load-carrying capacity.

References

Berra, M., Castellani, A., Coronelli, D., Zanni, S., & Zhang, G. (2003), Steel-concrete bond deterioration due to corrosion: Finite-element analysis for different confinement levels. Magazine of Concrete Research, 2003. Vol. 55, No. 3, pp. 237.

CEB Bulletin d'Information 213/214 (1993), CEB-FIP Model Code 1990. 1993, Lausanne, Switzerland.

Coronelli, D. (2002), Corrosion cracking and bond strength modeling for corroded bars in reinforced concrete. Aci Structural Journal, 2002. Vol. 99, No. 3, pp. 267-276.

Coronelli, D., Zandi Hanjari, K., & Lundgren, K. (2013). Severely corroded RC with cover cracking. Journal of Structural Engineering–ASCE, Vol. 139, No. 2, pp. 221-232.

fib Bulletin 10 (2000), Bond of reinforcement in concrete, State-of-art report. Fédération internationale du béton (*fib*), prepared by Task Group Bond Models, Lausanne, Switzerland.

fib Model Code for Concrete Structures 2010 (2013), Lausanne, Switzerland.

Lee, H.-S., Noguchi, T. & Tomosawa, F. (2002), Evaluation of the bond properties between concrete and reinforcement as a function of the degree of reinforcement corrosion. Cement and Concrete Research, 2002. Vol. 32, No 8, pp. 1313-1318.

Lundgren, K. (2005), Bond between ribbed bars and concrete. Part 2: The effect of corrosion. Magazine of Concrete Research, 2005. Vol. 57, No. 7, pp. 383-395.

Lundgren, K., Kettil, P., Zandi Hanjari, K., Schlune, H., & Soto San Roman, A. (2012), Analytical model for the bond-slip behaviour of corroded ribbed reinforcement. Structure and Infrastructure Engineering, 2012. Vol. 8, No. 2, pp. 157-169.

Nilsson, U. & Lundgren, K. (2014), Gröndalsviadukten, 2-2034-1. Kompletterande bärighetsutredning med avseende på kontroll av kapacitet i gjutfogar baserat på beräkningsmodell utvecklad av Chalmers tekniska högskola. In Swedish. Ramböll, Stockholm, Sweden.

Nilsson, U. & Lundgren, K. (2014), Blommenbergsviadukten, 2-2033-1. Kompletterande bärighetsutredning med avseende på kontroll av kapacitet i gjutfogar baserat på beräkningsmodell utvecklad av Chalmers tekniska högskola. In Swedish. Ramböll, Stockholm, Sweden.

Regan, P.E., & Kennedy Reid, I. (2009). Assessment of concrete structures affected by delamination: 1 - effect of bond loss. Studies and Research - Annual Review of Structural Concrete, 29, pp. 245-275.

Schlune, H. (2006), Bond of corroded reinforcement: analytical description of the bond-slip response. Chalmers University of Technology, Department of Civil and Environmental Engineering, Gothenburg, Sweden.

Soto San Roman, A. (2006), Bond behaviour of corroded reinforcement: FE-modelling and parameter study. Chalmers University of Technology, Department of Applied Mechanics, Gothenburg, Sweden.

Sustainable Bridges. (2007). D4.2 Guideline for load and resistance assessment of existing European railway bridges - advices on the use of advanced methods: Prepared by WP4, No. TIP3-CT-2003-001653.

Wang, X. & Liu, X. (2006), Bond strength modeling for corroded reinforcements. Construction and Building Materials, 2006. Vol. 20, No. 3, pp. 177-186.

Zandi Hanjari, K., Coronelli, D., & Lundgren, K. (2011), Bond capacity of severely corroded bars with corroded stirrups. Magazine of Concrete Research, Vol. 63, No 12, pp. 953-968.

Zandi, K. & Lundgren, K. (2015), Numerical 3D modelling of anchorage, corrosion and spalling. Fib Symposium 2015, Copenhagen, Denmark.