1D and 3D analysis of anchorage in naturally corroded specimens

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Abstract

Corrosion of reinforcement causes cracking and spalling of concrete cover which affects the bond; this is a crucial factor in deterioration of concrete structures. Earlier, tests have been carried out on specimens with naturally corroded reinforcements; in this study, the focus is given to the modelling of these specimens. The aim was to evaluate the scope of simpler and more complex bond models to assess the structural behaviour. A comparison of two approaches to model the anchorage behaviour was done: (a) a one-dimensional analysis, where the bond-slip differential equation with a non-linear bond-slip constitutive model is numerically solved, and the mean bond strength as well as the required anchorage length to anchor the yield force are computed. (b) Finite element (FE) analyses were performed using 3D solid elements for concrete, and beam elements for reinforcement, where the interaction was explicitly described using the same bond-slip constitutive model as in approach (a). The results show differences between the two approaches. Each of the modelling alternatives had both drawbacks and advantages; while the more complicated model accounting for more variables led to more realistic results in comparison with observations, the simpler 1D analysis was very fast and efficient.

1 Introduction

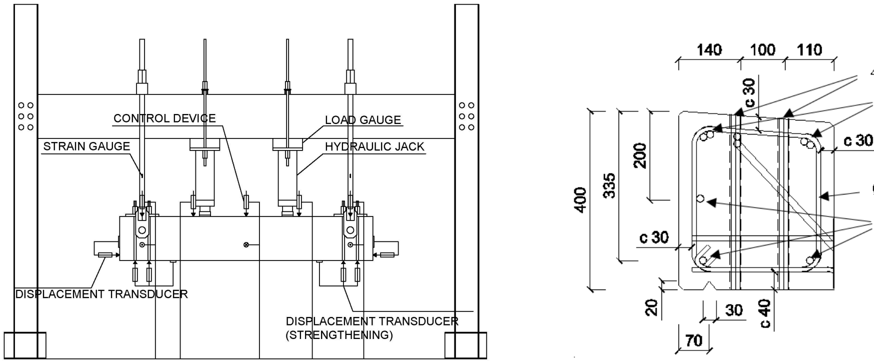
Corrosion of steel reinforcement is one of the main problems in reinforced concrete structures. Study of corrosion effects is crucial for a better understanding of the structural behaviour of existing impaired concrete structures. The most severe effect of reinforcement corrosion is the change in bond properties between the steel and concrete. Volumetric expansion of corrosion products causes splitting stresses along corroded reinforcement which might be harmful to the surrounding material. Generally, the splitting stresses are not tolerated by concrete, and that leads to cracking and eventually spalling of the cover. As the reinforcement becomes more exposed, the corrosion rate may increase and facilitate the deterioration process.

The effect of corrosion process on bond deterioration has been studied extensively by many researchers. Several studies have investigated parameters which may influence bond and anchorage capacity of corroded structures, see [1-4]. Even though tests of artificially corroded specimens with low corrosion rates indicated a closer relation to the natural corrosion conditions, literature shows that accelerated corrosion methods may still result in spurious bond deterioration and change the anchorage behaviour, [5], [6]. Thus, there was a strong need for experiments on naturally corroded specimens in order to facilitate the evaluation of the methods and models which were mostly developed based on accelerated-corrosion tests.

In the present paper, different approaches of modelling the anchorage capacity of naturally corroded specimens were used. A one-dimensional analytical model was used to calculate the local and global bond-slip behaviour along the naturally corroded reinforcements of the tested beams. Furthermore, three-dimensional non-linear finite element analyses were performed to describe the anchorage behaviour of the specimens. The results of the analytical and numerical models with different corrosion levels were compared with the experimental data.

2 Experiments

The tests have been carried out as a part of an experimental campaign at Chalmers University of Technology. The test setup and the test results are described in detail in [7] and [8]. The specimens were extracted from the edge beams of an existing girder bridge with a concrete slab; the edge beams showed different levels of corrosion-induced damage. Based on the damage patterns, the specimens were categorized in three different groups: Reference (R) beams with minor or no-visible damage, Medium (M) damaged specimens with only spalling cracks, and Highly (H) damaged specimens with spalling of the cover. A total of 21 beams were tested in two test series. This work is focused on the second test series consisting of 13 tests described in [8], three of them are presented in this paper. The designed test set-up and specimens geometrical specifications are shown in Figure 1(a) and (b), respectively. The edge beams were tested upside down compared to their placement on the bridge.



a) b)

Fig. 1 (a) Test-setup, and (b) cross-section of tested beams

An indirectly supported four point bending test configuration was used for the experiments. The load was applied by means of two hydraulic jacks defining a central constant moment zone and two shear spans. The beams were suspended by means of a frame which at the same time was used to fix the jacks. The support settlements as well as the mid-span deflection were measured by means of displacement transducers. The end-slip behaviour of the reinforcement bundles was recorded at both ends. The support zones were strengthened to avoid undesirable failure at these locations.

3 1D Bond-Slip Model

3.1 Bond model

An analytical 1D bond-slip model, developed in [9], was used to analyse the bond-slip behaviour of corroded and uncorroded ribbed steel reinforcement. Accordingly, the differential equation expressing equilibrium conditions along the reinforcement in tension can be defined as in Eq.1:

$$\frac{\pi \cdot d^2}{4} \cdot \frac{d\sigma}{dx} - \pi \cdot d \cdot \tau = 0 \tag{1}$$

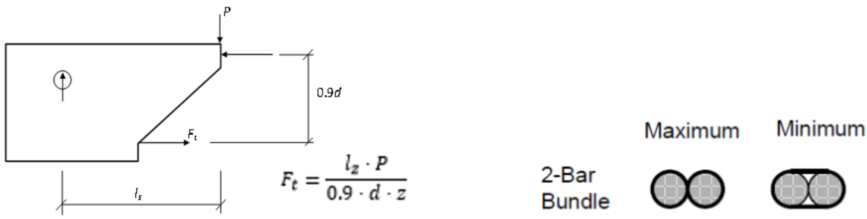
where d is the reinforcement diameter, σ is the stress in the reinforcement and τ is the bond stress.

The local bond-slip behaviour is computed based on the CEB-FIB Model Code 10. Corrosion effect of the reinforcement is taken into account by shifting the uncorroded local bond-slip relationship along the slip axis, as suggested in [9]. The model includes a modification of the first branch of the bond-slip curves compared to Model Code 10. To provide enough stiffness in the beginning, the parameter a was in this case modified to a value of 18, which is significantly higher than the original value of 2. For further information on how the bond-slip is obtained and the description of the equations used refer to [9]. The equivalent perimeter of the reinforcement bundles was taken following [10], considering the average value between the values given in Figure 2b

The model was developed to analyse the bond-slip behaviour of steel reinforcement within the anchorage length, and the stress in the reinforcement is assumed to be in the elastic range. The deformation of the surrounded concrete is neglected, thus all the slip is assumed to be caused due to the reinforcement deformation. By solving the differential equation, the load versus end-slip curves for a given embedded length as well as the distribution of slip, bond and steel stresses along the bar are obtained.

3.2 Structural model

The tensile load, F_t, at the end of the remaining available anchorage length is obtained by integrating the local bond-slip along the bar according to equation (1). To obtain the equivalent load, P, applied on the tested beam, the same structural model presented in [7] was used, see Figure 2a.



a) b)

Fig. 2 (a) Structural scheme to obtain the applied load from the tensile load reinforcement, and (b) Equivalent perimeter for bundled reinforcement bars proposed by [10]

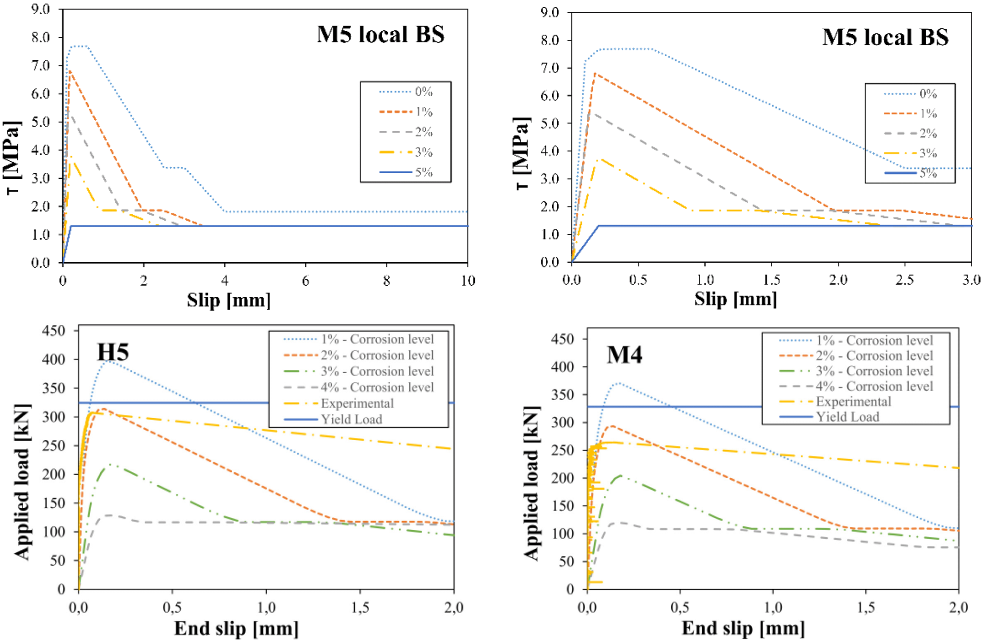


Fig. 3 Top, Bond-slip curves accounting for four different corrosion levels, obtained by means of the 1D bond-slip model for beam M5 (Right figure is enlargement of left). Bottom, Applied load end-slip curves for H5 and M4 specimens accounting for 1 to 4 % corrosion compared experimental data

3.3 Results

A comparison between the analytical model and the test data for some of the specimens is shown in Figure 3. The corrosion level of the damaged specimens was estimated to be around 2-3% [7]. As can be seen in Figure 3, the agreement between the experimental and 1D analysis results is reasonably good; this is true for all specimens. Thus, a reasonable good estimation for the remaining anchorage capacity of corroded specimens can be obtained using the model provided the available anchorage length is known. It should be noted that “all other bond conditions” according to Model Code 10 was assumed to get the local bond-slip curves, taking into account that the specimens were damaged and taken from a real bridge; the assumption of "good bond conditions" would have overestimated the capacity. There is not experimental data for the local bond-slip behaviour from the tests.

The 1D model generally reproduces the pre-peak behaviour and the capacity accurately. The post peak behaviour however does not show the same agreement, whereas the remaining capacity in most of the specimens is estimated reasonably well.

4 3D model with 1D bond-slip relationship

4.1 FE model

3D non-linear finite element analyses were performed to describe the behaviour and capacity of the anchorage zone. The commercial software DIANA with pre- and post-processor FX+ was used for the numerical simulations.

The beams that were modelled had the same dimensions as the tested specimens accounting for the different geometrical specifications of each specimen. The exact positions of the reinforcements as well as the stirrups were taken into account in the development of each model. The symmetry of the test-setup allows for half of the span and loading to be considered in the model as shown in Figure 4. Boundary conditions were applied on the top node of the suspension drill, supporting the displacements in the vertical axis and out of plane one. The load was applied by means of displacement control.

3D tetrahedral elements (TE12L) were used for the concrete and the reinforcement bars were embedded into concrete element; this allowed to describe the concrete-rebar interaction using a bond-slip relation. An analytical local bond-slip relationship for each specimen was defined according to the outputs of the 1D bond-slip; i.e. the same local bond-slip as used in the 1D analyses, Figure 2. Five different bond-slip relations were used for each specimen taking 5 different corrosion levels into account. The loading zone was modelled by means of a wood board and a steel plate using triangular-prism elements (TP18L). An overview of the model and the boundary conditions are shown in Figure 4. Tying elements with eccent properties were disposed in fixed nodes of the support to avoid undesairble local failure. This tying fixed the upper node with two slave nodes imposing the same rotation respect the master node.

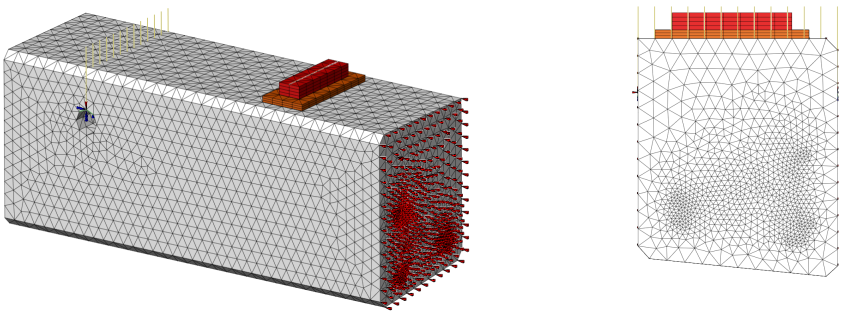


Fig. 4 Overview of the FE model

The concrete was modelled with a constitutive model based on non-linear fracture mechanics using a total strain based smeared-crack model with rotating crack approach. Thorenfeldt compression curve was used in order to more realistically describe the behaviour of concrete in compression. The softening behaviour of this curve was adapted to the element size as described in [11]. The tensile behaviour was modelled using a stress-strain relation proposed by Hordijk for tension softening. The reinforcing steel was modelled with an isotropic plasticity model with Von Mises yielding criterion including hardening. The material properties for steel and concrete used in the analysis can be found in [7] and [8]. Compressive concrete strength, f_c, was obtained in the laboratory from every specimen by means of cylindrical specimens. The E modulus tensile concrete strength was the average from 3 of the first round specimens. The energy fracture was calculated by means of the expression proposed in the *fib* MC 10.

4.2 Results

The results from 3D FE analysis, see Figure 5, show a pull-out failure in most of the cases for a corrosion level of 2% and above. For values below 2%, the reinforcement yields. In general, the maximum load capacity is reasonably well described by these analyses. However, the overall stiffness is overestimated in several cases, most likely because of the pre-existing internal damage of the specimens was not reproduced in the described models. For some of the tests, the initial stiffness of the local bond-slip curve used as input in the model was not enough, yielding high slip values at the first load steps in the applied load versus end slip curves; however, in general the local bond-slip curves yields in a good overall behaviour as it is shown in Figure 5.

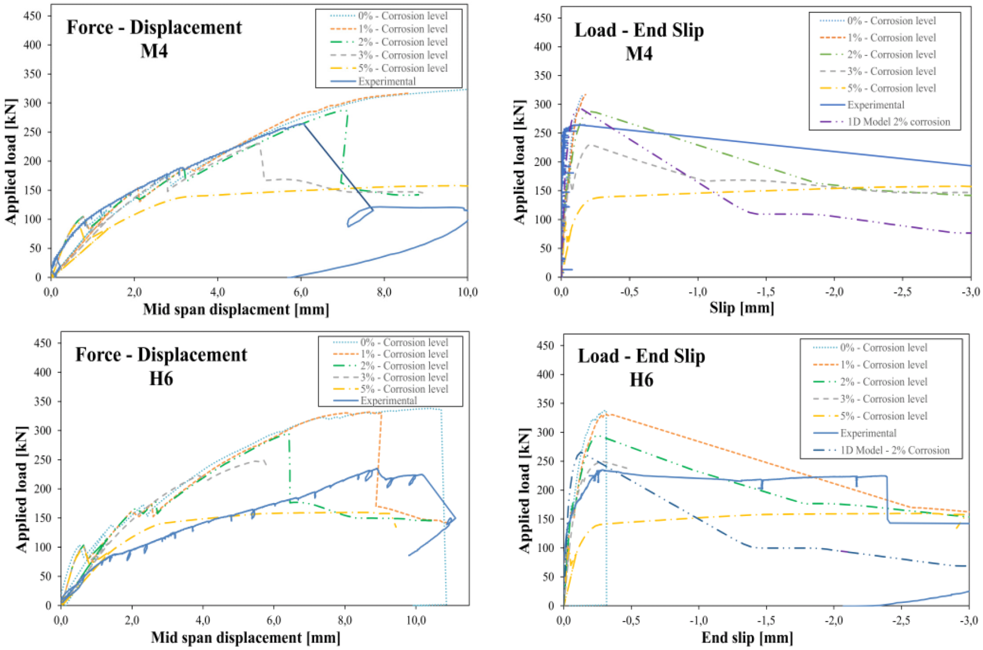


Fig. 5 Results of 3D FE analysis in comparison with experiments and 1D Bond-slip Model

Figure 6 shows the crack pattern and the remaining anchorage length for specimen M4. As can be seen, there were two shear cracks between the loading plate and the support in the experiment (marked with 2 and 3 in the figure), while there was only one shear crack in the corresponding region in the analysis. Thus, in the 3D model, all the damage tends to concentrate to the first shear crack. This results in larger remaining anchorage lengths in the analyses than in the experiments for several specimens. Accordingly, the remaining load capacity was slightly higher in comparison to the tested beams as it is directly related with the anchorage length. A probable reason for this discrepancy is that the modelling does not describe the splitting stresses and cracks in a correct way; in the tested beams the splitting cracks connected with the second shear crack. To describe this interaction, simple bond-slip input is too simplified; more sophisticated modelling including the splitting stresses of both the corrosion and the slip would be needed.

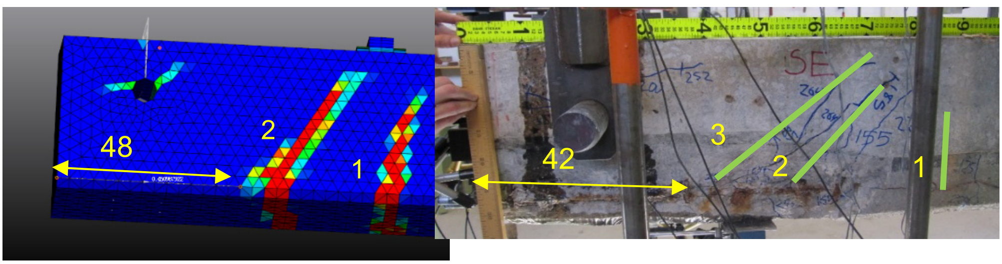


Fig. 6 Crack pattern comparison of M4 specimen; the anchorage length is given in mm

5 Conclusions and Outlook

It was shown that simplified 1D model can be used to obtain rough estimations of the ultimate anchorage capacity. From that point of view these models are very useful, but they are strongly dependent of the available data such as the available anchorage length; data that is not always easy to obtain. More complex models are necessary to describe the overall structural behaviour. Using simple bond-slip relationships between concrete and steel in complex models is useful and relatively fast to obtain a good approximation to the real behaviour. However, it was shown that also this modelling technique has shortcomings, mainly because the splitting action is not included. In future work, a frictional model where the effect of corrosion is taken into account by introducing the swelling action and the flow of rust through cracks will be used, to account for both the internal concrete damage due to steel corrosion and splitting stresses generated by the slip .

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