Concrete splitting and tip-bearing effect in the bond of anchored bars tested under fatigue loading in the push-in mode: An experimental investigation

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Abstract The load scenario with a compressive force applied to an anchored bar (push-in mode) has not been sufficiently addressed so far with reference to fatigue, since most of the studies available in the literature are focused on bond behavior under tensile forces (pull-out mode). However, special structures like the towers of wind turbines subjected to alternating moments (and tensile-compressive forces) due to the variable wind direction, are fostering the interest for the fatigue behavior of concrete-bar bond under compressive forces, all the more because several millions of load cycles may be applied. An extensive experimental investigation has been carried out in this research project using a modified beam-end test in the push-in mode. Fifty beam-end specimens were tested under monotonic loads, as well as under low-cycle and high-cycle fatigue loads, with different bar diameters and bonded lengths. The failure modes and bond behavior with the end sections of the anchored bars either unloaded (free ends) or pushing against the concrete (compression ends exhibiting the well-known tip-bearing effect) were investigated as well. The results of the cyclic tests in the push-in mode are compared with those available in the literature in the pull-out mode, through the usual Wöhler curves and fatigue-induced creep curves. Based on this comparative analysis, the failure mechanisms of bond under fatigue loading are identified, with specific attention to the interaction between bond behavior and splitting cracks.

Keywords Bond fatigue · Reinforced concrete (RC) · High-cycle fatigue · Bond-slip · Cyclic loading · Push-in loading · Pull-out · Beam-end test · Splitting cracks · Tip-bearing effect · Damage

Table with the symbols

| Symbol | Description |
|--------|-------------|
| $\eta_2$ | factor depending on bond conditions |
| $d_s$ | rebar diameter |
| $H$ | height of the specimen |
| $L_b$ | embedded/bonded length |
| $L$ | length of the specimen |
| $W$ | width of the specimen |
| $\omega$ | damage parameter |
| $\tau_{u,\text{split}}$ | splitting bond strength |
| $\tau_b$ | bond strength |
| $b$ | parameter of the fatigue creep curve by FIB Model Code [1] |
| $C, m$ | parameters of Wöhler curve proposed in [2] |
| $C_{\text{max}}, C_{\text{min}}$ | maximum/minimum concrete cover |
| $E_{b0}$ | initial bond stiffness |
| $E_b$ | unloading bond stiffness |
| $E_c$ | concrete modulus of elasticity |
| $f_y$ | yield strength of steel reinforcement |

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$f_{cm,cyl}$ cylindrical compressive strength of concrete
$F_{\text{crack}}$ force level at the onset of concrete cracking by splitting
$f_{ct,\text{spl}}$ concrete indirect tensile strength by splitting
$F_{\text{max}}$ upper level of the loading range
$F_{\text{min}}$ lower level of the loading range
$F_u$ ultimate load under monotonic loading
$K_m, K_{tr}$ transverse reinforcement factors
$N$ number of cycles
$s$ total slip
$s^p$ plastic slip
$S_{\text{max}}$ ratio of the upper level of the loading range
$S_{\text{min}}$ ratio of the lower level of the loading range
$t$ concrete age in days
$w$ slip at the loaded/unloaded ends
$w_0(N)$ slip at the first loading cycle
$w_{n}(N)$ slip after the $N$ loading cycle

1 Introduction

The characterization of resistance under fatigue loading is a paramount requirement for an economical and reliable structural design of reinforced-concrete (RC) structures subjected to variable loads. As an example, wind turbine towers may be cited, as they must withstand millions of load cycles with reverse loading (tension-compression) because of the variable wind direction. In addition to the tensile stresses, this alternating effect induces compressive stresses in the reinforcement. The investigation of the behavior of the bond between rebars under high-cycle compressive loading and concrete is required for a more economic design in terms of enhanced service life and reduced material consumption. One of the major differences between the rebars loaded in tension and those loaded in compression is that an additional load transfer contribution is provided by rebar end in the case of compressive loading. Therefore, it is also vital to evaluate this contribution and to study its influence on the bond behavior under monotonic loading as well as under high-cycle fatigue loading.

The fatigue of bond between steel reinforcement and concrete has been studied by many researchers in the last decades. The earliest bond tests with cyclic loading were presented by [3–5]. The first more comprehensive test campaign in this field was conducted later by [6] where 308 cylindrical pull-out specimens were tested with up to one million load cycles and for different values of the bonded length, bar diameter and concrete class. Based on the results obtained in that experimental campaign, a simple relation for the estimation of bond slip as a function of the number of load cycles was proposed, which has been later included in the FIB Model Code 2010 [1]. Later research contributions were presented in [7, 8], where bond fatigue was investigated for different values of the bonded length, load sequence and load scenarios. Further recent experimental investigations of the bond behavior between concrete and steel reinforcement under cyclic and fatigue loading were presented [9–12]. The effect of fatigue loading on the residual monotonic bond strength has been experimentally studied in [13]. The effect of corrosion of the steel reinforcement on bond fatigue behavior was also investigated in [14, 15]. RC structural members, e.g. non-prestressed bridge decks, are often subjected to biaxial cyclic loading. The tensile stresses in the longitudinal direction induce cracking along the transverse reinforcement, which affects the bond behavior along the reinforcement. For this reason, the bond fatigue behavior of pre-cracked specimens has been comprehensively investigated in [16–18] with the goal to study the influence of the lateral tension on bond fatigue behavior using the pull-out test setup. Based on these studies Wöhler/ S-N curve approximations have been proposed for the cracked and uncracked bond conditions.

In most of the mentioned experimental studies, the typical RILEM pull-out test setup has been used. Despite its wide usage e.g. [19–23], the bond behavior characterized using this test setup inherently includes the effect of the compressive stresses within the bond zone, as shown by many authors e.g. [24–27]. To avoid this influence, the push-in beam-end test, previously introduced by the authors in [28], is used here because it is able to realistically reflect the stress-state pulsation occurring in structural members. Moreover, the effect of stabilized splitting cracks along the reinforcement rebar can be systematically investigated, as the concrete cover can also be varied.
within a relevant range. In comparison with the previous paper, the range of materials and bond configurations is extended in the current paper and a general classification of fatigue bond behavior in the context of splitting cracks and compression end is provided.

The typical studies of the bond fatigue are usually carried out under tensile pull-out loading. Only few studies addressing the bond behavior under monotonic compressive push-in loading can be found in the literature, e.g. [29–31], however, the bond fatigue behavior under compression is still not sufficiently addressed in the literature. The study of bond fatigue behavior under push-in loading will provide the basis for distinguishing the fatigue effects observed under compression and tension in reinforced concrete members. Such an analysis can allow for the characterization of the fatigue behavior of reinforcement lap joints in large structures in compression-loaded zones and pre-stressed components, e.g., in the towers of wind turbines. Additionally, the push-in loading allows to test the contribution and the effect of the concrete compression end, which is not possible in the case of a pull-out loading.

The goal of the present paper is to study the bond behavior under push-in loading considering monotonic, low-cycle and high-cycle fatigue load scenarios. This study aims to fill the gap in the literature regarding the bond fatigue under compression accounting for the effect of the concrete compression end. In addition, the pre-peak and post-peak cyclic behavior is included to allow for the identification of the main dissipative mechanisms involved in the bond fatigue development. Moreover, the investigated range of fatigue response covers the low-, high- and very high-cycle fatigue up to 10 millions of loading cycles.

2 Experimental program

2.1 Material properties

According to the EC2 specification, the beam-end tests were performed with the concrete strength class C40. A W/C ratio of 0.5 was used for the concrete mixture with the cement type (CEM I 42.5 N). The aggregate composition consists of 62% limestone and 38% quartz sand with a maximum grain size of 16 mm.

The beam-end tests were accompanied by material tests to determine the compressive strength $f_{cm,cyl}$, modulus of elasticity $E_c$, and tensile strength $f_{ct,spig}$ of the used concrete. In accordance with the norm [32], these tests were performed on cylinders with 150 mm diameter and 300 mm height and on cubes with an edge length of 150 mm. During the curing phase, all specimens were stored under the same environmental conditions as the beam-end specimens, which were stored at laboratory temperature. The results of the material tests are summarized in Fig. 1. The steel bars with diameters of 16 mm and 25 mm were hot rolled and can be classified as B500B according to [33] with a yield strength of $f_y = 500$ MPa. The bars have two longitudinal ridges and two rows of inclined transverse ridges as shown in lower part of Fig. 2b.

2.2 Test setup for the push-in mode

Following the concept presented in [26], the modified beam-end test setup described in [28, 34, 35] was used in the present test program with the specimen dimensions, concrete cover, and bonded length scaled proportionally to the rebar diameter, as shown in Fig. 2. This scaling of the test setup enables a comparable relationship between the stresses in the steel reinforcement within the critical concrete sections. The dimensions of the specimen were defined as $(L \times W \times H = 20 \text{ ds} \times 8 \text{ ds} \times 14 \text{ ds})$ (see Fig. 2a). In the experimental program, two bonded lengths were considered, namely 2.5 ds and 5 ds. The bond zone was followed by a bond free area of 7.5 ds in case of the specimens with the free end. For the tests with a

![Fig. 1](image_url) Development of the mechanical properties of concrete: a) compressive strength; b) tensile splitting strength; c) modulus of elasticity
The steel rebar was embedded in the concrete up to the defined bonded length without bond free zone as shown in Figs. 2a, 3. To avoid buckling of the bar within the free length at the loaded end, the length of the bar was reduced by introducing a recess within a specimen and applying the load in a short distance from the bond zone as depicted in Fig. 2a. The used test setup as well as the configuration of the

Fig. 2 Planning, fabricating and testing of the beam-end specimens: a modified setup for the beam-end tests in the push-in mode, with the end-section of the bar (opposite to the load) free and pushing against the concrete; b production of the beam-end specimen and the reinforcement bars reinforcement bars; c the test setup
supports and the load application are sketched in Fig. 2c.

An important consideration in characterizing the bond behavior using the beam-end test setup is the level of confinement, which can be controlled through the concrete cover and/or the transverse reinforcement. To achieve a practically relevant test configuration, the concrete cover was set to 2 ds. Furthermore, transverse reinforcement in the form of stirrups was arranged within the bonded length to restrict the width of the splitting cracks. Two stirrups were used for the specimens with a bonded length of 2.5 ds, while four stirrups were used for the specimens with a bonded length of 5 ds. The stirrup diameters of 6 mm and 10 mm were used for the specimens with ds=16 mm and ds=25 mm, respectively.

To avoid the influence of transverse cracks developing from the corners of the recess shown in Fig. 4, longitudinal reinforcement was placed at three levels of specimen height with the diameters of 16 mm and 10 mm for the large specimens with bonded rebar of 25 mm and small specimens with bonded rebar of 16 mm, respectively.

2.3 Preparation of the beam-end specimens

The beam end specimens were cast in several batches. The position and orientation of the reinforcing rebars were chosen with the goal to ensure reproducible bond configurations along the embedded length. The specimens were cast in an upside-down position as shown in upper part of Fig. 2(b) to achieve good bond conditions between concrete and steel rebar in the sense of the FIB Model Code 2010 specification [1].

2.4 Load scenarios

To better understand the bond behavior and to obtain more information about the main dissipative mechanisms involved in the fatigue behavior of the bond in both low and very high cycle regimes, several load scenarios were used in the conducted test program, which are summarized in Table. 1. A systematic set of load scenarios was adapted in the experimental program with the intention to provide a large dataset, to be used in the development, calibration, and validation of numerical models capable of covering a wide range of loading conditions e.g. [36, 37].

**LS1:** The first load scenario introduces a monotonically increasing displacement controlled loading applied with the rate of 1.0 mm/min. This test provides the ultimate push-in force $F_u$ and can directly deliver the estimation of the bond slip law that governs the bond behavior in the case of a short bonded length.

**LS2:** The second load scenario introduces the load using a cyclic increasing control displacement for the upper and lower levels including seven unloading cycles applied with the rate of 1.0 mm/min. This load scenario provides detailed data on the post-peak loading and unloading behavior. This information is useful for distinguishing the main dissipative mechanisms, e.g. such as damage and irreversible slip, governing the cyclic bond behavior in the softening regime.
The third scenario is a load-controlled cyclic loading with up to 100 cycles applied with a frequency of 0.02 Hz. The upper load level starts with $S_{\text{max}} = 0.50$ and is gradually increasing by $\Delta S = 0.05$, with 10 cycles performed at each loading level. The lower loading level remains constant at $S_{\text{min}} = 0.10$, as shown in Table 1. This load scenario provides detailed data on the pre-peak loading and unloading behavior which can be used for the analysis of the main dissipative mechanisms at subcritical load levels.

The last scenario represents the typical uniform fatigue loading with a constant amplitude. This cyclic loading has been applied with a frequency of 5 Hz. The upper load levels have been varied between $S_{\max} = 0.80$ to $S_{\max} = 0.99$, and lower load level were set to $S_{\min} = 0.40$.

2.5 Parameters investigated in the test program

The parameters studied in the test program are the rebar diameter ($d_s$), the bonded length ($L_b$), and the effect of the concrete compression end in combination with the described four load scenarios (LS). For the section opposite the loaded end, two cases (free end and compression end) can be distinguished. In the first case, the rebar end is unloaded (free end, typical for pull-out and push-through tests), and it pushes against the concrete in the second case (compression end, characterized by the tip bearing effect). The four configurations of the beam-end specimens investigated in the current experimental program are depicted in Fig. 3. The test matrix with the number of the

Table 1  Description of the load scenarios used in the experimental program

| Load scenario | Description | Purpose | Load rate/ frequency | Figure |
|---------------|-------------|---------|----------------------|--------|
| LS1 | Monotonic load | Studying the monotonic behavior and identifying the ultimate load | 1.0 mm/ min | ![Graph](image1.png) |
| LS2 | Cyclic load | Providing detailed description of unloading and reloading of the post-peak behavior | 1.0 mm/ min | ![Graph](image2.png) |
| LS3 | Cyclic step-wise increased load with 10 cycles each level | Providing detailed description of unloading and reloading behavior of the pre-peak regime | 0.02 Hz | ![Graph](image3.png) |
| LS4 | Fatigue load with constant amplitude | Characterizing the bond fatigue under constant amplitudes | 5 Hz | ![Graph](image4.png) |
replications for each parameter combination is summarized in Table 2.

2.6 Recorded data

For the specimens with free end, the slips at the loaded and unloaded ends have been measured using linear variable differential transformers (LVDT). Whereas, only the slip at the loaded end was measured for the specimens with compression end. For better evaluation of the recorded slip at the loaded end and to exclude any bending effect at the compressive loaded end, two LVDTs were used as shown in Fig. 4. Furthermore, the crack width at the concrete surfaces along the bonded rebar and in the transverse direction has been measured with two LVDTs (see Fig. 4). The positions of these LVDTs were selected based on the longitudinal and transverse crack development observed in preliminary tests.

3 Monotonic behavior and bond strength

3.1 Push-in curve

The effect of the parameters on the bond behavior under monotonic loading is shown in terms of the push-in curves as well as the load versus crack width curves in Fig. 5. The effect of the rebar diameter on the push-in behavior is shown in Fig. 5a, d for the tests with bonded length of 2.5 ds. The effect of the bonded length is illustrated in Fig. 5b, e for the bonded lengths 2.5 ds and 5 ds and the rebar diameter ds = 16 mm. Furthermore, the effect of the compression end is shown in Fig. 5c, f. The comparison between the behavior of a specimen with a compression end and with a free end is shown in Fig. 5c. The additional contribution of the compression end and the contribution of the bond are shown in dark and light gray colors, respectively. The average of the ultimate push-in force measured in the tests with the compression end was 140.36 kN, while an average of 92.40 kN was obtained for the tests with the free end, as summarized in Table 3. The push-in curves plotted in Fig. 5c show that the tests with the a compression end result in a more brittle response after the peak compared to the tests with a free end. This can be attributed to the development of the push-out cone [38] below the compression end as indicated in Fig. 6b.

A comparison between the push-in curves measured at the loaded and unloaded ends for two selected tests from each parameter combination are presented in the first row of Fig. 5. In the second row of Fig. 5, all push-in curves measured at the unloaded end are plotted. The shape of the curves in Fig. 5e directly indicates the higher scatter obtained for the 2.5 ds bonded length in comparison to 5 ds bonded length. Similar observation can be seen in Fig. 5d which show a higher scatter for the results with rebar diameter of ds = 16 mm in comparison with the tests with the rebar diameter of ds = 25 mm. A comparable level of scatter has been observed in the tests with compression end and with free end as documented in Fig. 5f.

The average values of the ultimate push-in forces measured for each parameter combination are summarized in Table 3 including the statistical evaluation. The comparisons depicted in Figs. 5a–c show no difference between the loaded and unloaded push-in curves for both studied bonded lengths. This behavior is in agreement with the assumption of constant bond stress for steel rebar with bonded length up to $L_b \leq 5$ ds, e.g. [39, 40].

### Table 2 Test matrix

| Case  | ds [mm] | $L_b$ | Rebar end | Load scenario |
|-------|---------|-------|-----------|---------------|
| 1     | 16      | 2.5 ds| Free end  | 6 LS1, 1 LS2, 1 LS3, 12 LS4 |
| 2     | 16      | 5 ds  | Free end  | 3 LS1, – LS2, – LS3, 6 LS4 |
| 3     | 25      | 2.5 ds| Free end  | 3 LS1, – LS2, – LS3, 9 LS4 |
| 4     | 25      | 2.5 ds| Compression end  | 3 LS1, – LS2, – LS3, 6 LS4 |
| Sum:  |         | 50    |           |               |

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Fig. 5 Bond behavior under monotonic loading (LS1): (a–c) comparison of push-in curves for loaded and unloaded ends for all tested cases; (d–f) comparison of push-in curves for unloaded end for all tested cases; (g, h, i) combined push-in curves and crack opening evolution showing mutual interactions; (j–l) comparison of the crack opening development for longitudinal and transverse cracks.
3.2 Visible cracks

Two main types of cracks were observed in the monotonic tests as shown in Fig. 6a. In the majority of tests, a single longitudinal splitting crack developed along the bond bar. This longitudinal crack branched in some experiments. (see Fig. 6a). In addition, transverse cracks developed from the corners of the recess in both directions. The specimens with the compression end failed due to a complete splitting of the concrete cover into three parts, where more brittle failure was observed in comparison with the specimens with free end. An example of a crack pattern observed in these specimens is shown in Fig. 6b.

The growth of longitudinal and transverse cracks vs. push-in force is plotted for selected tests in the fourth row of Fig. 5. It can be observed from Fig. 5j–l that the transverse cracks started to develop already at low load levels, while the bond behavior was still in the elastic range. Due to the three arranged longitudinal reinforcement bars, the maximum observed width of the transverse cracks was limited to 0.15 mm. Therefore, no significant influence of these cracks on the bond behavior could be detected and they could be disregarded in the further analysis of the bond behavior. On the other hand, a significant influence of the longitudinal splitting cracks on the bond behavior was observed. The curves plotted in the third row of Fig. 5 for representative tests show the correspondence between the push-in force-slip curve and the growth of the splitting cracks during the test. As an example, in the test shown in Fig. 5h with green color, the occurrence of the splitting crack is marked with the unfilled circle. The corresponding load is denoted as $F_{crack}$. After the initiation of the splitting crack, a significant reduction of the bond stiffness is observed. The force continues to increase until the maximum push-in force $F_u$.

The average values of the maximum force $F_u$ and cracking force $F_{crack}$ for all monotonic tests are summarized in Table 3. The ratios of the cracking force $F_{crack}$ to the maximum force $F_u$ for all the performed tests with the concrete C40 are summarized in Fig. 7d. To set the results into a broader context, the values of maximum force measured for high-strength concrete grades C80 and C120 published in [28] are included as well. As expected, the results show that the average value of the ratio $F_{crack}/F_u$ increases with the increase of concrete strength.

It is worth mentioning that the observed longitudinal cracks did not always follow the same pattern in all tests, so that the values of the crack width measured at the predefined locations of the LVDTs exhibit some dispersion. However, the main aim of the current investigation is to study the fatigue mechanisms involved in the bond behavior under well-defined

| Tests    | $d_s$ [mm] | $L_d$ | Rebar end | Num. tests | $F_u$ [kN] | $\tau_u$ [MPa] | SD [MPa] | CoV [%] | $F_{crack}$ [kN] | $w(F_{crack})$ [mm] | $w(F_u)$ [mm] |
|----------|------------|-------|-----------|------------|------------|----------------|----------|---------|----------------|----------------------|----------------|
| T01 - T06 | 16         | 2.5   | Free end  | 6          | 35.82      | 17.81         | 2.19     | 12.3    | 29.30         | 0.79                  | 0.169          |
| T07 - T09 | 16         | 5     | Free end  | 3          | 61.69      | 15.34         | 0.43     | 2.8     | 48.70         | 0.79                  | 0.206          |
| T10 - T12 | 25         | 2.5   | Free end  | 3          | 92.40      | 18.82         | 0.72     | 3.8      | 77.97         | 0.82                  | 0.201          |
| T13 - T15 | 25         | 2.5   | Compression end | 3      | 140.36     | –             | –        | –       | 106.69        | 0.76                  | 0.465          |

Fig. 6 An overview of all observed crack patterns: a free end tests; b compression end tests
Therefore, it is sufficient to use the LVDTs to detect the initiation of the splitting cracks and their width growth during further loading. In this way, it is possible to qualitatively characterize the effect that splitting cracks have on bond behavior.

3.3 Failure modes and crack pattern

Based on the FIB Model Code 2010 [1], the three types of failure modes are distinguished. The first type is the debonding failure either through pull-out or push-through without splitting cracks. If the bond failure is accompanied with moderate splitting cracks that are stabilized by the transverse reinforcement, the failure mode can be considered as a combined pull-out / push-through × splitting failure. The third type of failure is the splitting failure, in which there is not sufficient transverse reinforcement, resulting in a sudden failure of the connection. Due to the presence of the stirrups and the relatively low concrete cover of 2 ds in the test program, the monotonic push-in curves exhibit a combined push-through × splitting failure with stabilized splitting cracks. With this type of failure, the push-in force usually continues to increase after the splitting crack occurred until the maximum push-in force is reached. A similar mode of bond failure has been observed by many authors, e.g. [41–44]. Even though the cracking behavior observed in the tests with compression end was relatively brittle, nevertheless, this can still be considered as a stabilized cracking mode since no sudden failure was observed.

3.4 Bond strength

Since the stress distribution can be considered constant within the limits of the bonded length, it is possible to evaluate the bond strength directly by dividing the push-in force obtained in the monotonic test by the contact area, i.e.

$$\tau(w) = \frac{F(w)}{\pi \cdot L_b \cdot d_s},$$  \hspace{1cm} (1)

where \(w\) represents the slip at the loaded/unloaded end, and \(F\) is the push-in force. The bond strength is then obtained as maximum bond stress as

$$\tau_u = \max_w \tau(w).$$  \hspace{1cm} (2)

To compare the bond strength \(\tau_u\) obtained under push-in loading with the bond strength under pull-out loading, the following relationships can be derived:

$$\tau_u = \frac{F_u}{\pi \cdot L_b \cdot d_s},$$

$$\tau_u = \max_w \tau(w).$$
loading, three empirical formulas presented in the literature derived have been used, which approximate the bond strength under consideration of the pull-out and of the combined pull-out × splitting failure modes. In case of pull-out failure the bond strength can be estimated based on the FIB Model Code 2010 [1] as a square root function of the concrete compressive strength \( f_{cm,cyl} \) as follows

\[
\tau_u = 2.5 \cdot \sqrt{f_{cm,cyl}}
\]  

(3)

Another simplified approximation was proposed in [45], assuming a linear relationship between bond strength and concrete compressive strength

\[
\tau_u = 0.45 \cdot f_{cm,cyl}
\]  

(4)

In the case of the combined pullout and splitting failure, the bond strength depends on several factors such as concrete strength, concrete cover, transverse reinforcement and rebar diameter [46]. According to the FIB Model Code 2010, its estimation can be performed as follows

\[
\tau_{u,split} = 6.5 \cdot \eta_2 \cdot \left( \frac{f_{cm,cyl}}{25} \right)^{0.25} \cdot \left( \frac{25}{ds} \right)^{0.2} \cdot \left[ \left( \frac{C_{\text{min}}}{ds} \right)^{0.33} \left( \frac{C_{\text{max}}}{ds} \right)^{0.1} + K_m K_{tr} \right]
\]  

(5)

where \( \eta_2 \) is a factor related to bond conditions, \( C_{\text{max}}, C_{\text{min}} \) are the maximum and minimum concrete cover, respectively, and \( K_m, K_{tr} \) reflect the effect of transverse reinforcement.

The bond strength values obtained from the monotonic tests of the used C40 concrete are summarized in Fig. 7a together with the previously published results of C80 and C120 concretes described in [28], and compared with the values provided by Eqs. (3, 4, 5) based on the previously-mentioned approximations. The theoretical values proposed by the 2010 FIB model code for the pull-out failure (Eq. 3) and for the combined pull-out splitting failure (Eq. 5) are plotted in Fig. 7a as blue and red lines, respectively, whereas the approximation in (Eq. 4) is plotted as a black line. The comparison shows that the FIB Model Code 2010 approximation underestimates the bond strength especially for high-strength concrete.

The effect of the two reinforcement diameters adopted in this experimental program in combination with the current concrete C40 and the previously investigated concretes C80 and C120 [28] on the bond strength is summarized in Fig. 7b.

For high-strength concrete (i.e., C80 and C120), a slight decrease in average bond strength is observed for larger diameter, while for normal-strength concrete (i.e., C40), a slight increase in average bond strength is observed for larger diameter.

The evaluated results show a large scatter of the bond strength for the tests with a bonded length of 2.5 \( ds \) having Coefficient of Variation (CoV) of 12.3, while this scatter is significantly reduced for the tests with the longer bonded length of 2.5 \( ds \), where a Coefficient of Variation (CoV) of 2.8 is obtained, as indicated in the Table. 3.

3.5 Characteristic values of slip and crack width

In addition to bond strength and first-cracking load, it is important to evaluate the magnitudes of slip and splitting crack width. Three characteristic values were considered in each test, namely the slip values at the cracking force \( w(F_{\text{crack}}) \), the ultimate push-in force \( w(F_u) \), and the splitting crack width at the ultimate push-in force as shown in Fig. 7c. The average values for each case are shown in Fig. 7f. For the tests with free end a relatively similar values of slip have been observed at the cracking force, as well as at the ultimate push-in force as plotted with the red and green colors, respectively. The average crack width at \( F_u \) was larger for the experiments with longer bonded length, i.e., 5 \( ds \), than for the experiments with shorter bonded length, i.e., 2.5 \( ds \), as plotted in blue color in Fig. 7f. The response of the tests with the compression end shows significantly larger values for the slip at \( F_u \), the slip at \( F_{\text{crack}} \), and for the crack width at \( F_u \) compared to the tests with the free end as shown in Fig. 7f. The larger slip values can be seen directly from the push-in response shown in Fig. 5c, where additional displacements are still required to overcome the peak load because of the compressive concrete cover acting beyond the rebar end. In addition, the measured average crack width at \( F_u \) was about three times larger in the tests with compression end than in the tests with free end.
3.6 Bond strength under pull-out and push-in loading

The results recently presented in [47], using the same test setup, i.e. beam end tests under pull-out tensile loading, and the same concrete grade, can be used for comparison with the present push-in test program. The average values of the bond strength under pull-out loading for four bonded lengths ranging from 1 ds to 4 ds are plotted with orange color in Fig. 7e. At the same time, the average values of bond strength under push-in loading for the bonded lengths, i.e. 2.5 ds and 5 ds, are plotted in black and green colors, respectively. This comparison shows that for the concrete grade C40, higher bond strength is obtained under pull-out than under push-in loading. This trend can be attributed to the opposite Poisson effect, which leads to lateral expansion of the steel reinforcement in the case of push-in loading, while lateral contraction can be considered in the case of pull-out loading. Therefore, a larger width of the splitting crack can be expected for push-in loading, resulting in a lower achieved bond strength. This effect increases with increase of the bonded length, where a relatively larger crack width is observed, as shown in Fig. 5h, resulting in a decreased bond strength, as shown in Fig. 7e. It should be noted that although similar grade of concrete was used for the tests under pull-out loading [47] and the tests under push-in loading, larger compressive strength was reported from the test program in [47] compared to the current test program. This could be another reason for the larger bond strength in the tests under pull-out loading.

4 Bond cyclic behavior

4.1 Cyclic post-peak bond behavior

*Cyclic push-in curve and hysteretic loops:* To trigger a possibly broad range of dissipative mechanisms within the bond behavior, the beam-end specimen was exposed to several displacement controlled loading cycles covering also the post-peak regime. Particular attention of the test evaluation was on the changing shape of the hysteretic loops as shown in Fig. 8(a) with almost invisible opening of the hysteretic loops.

*Evaluation of the dissipative mechanisms:* Testing the bond behavior under cyclic loading (LS2) is essential for the macroscopic distinction of the dissipative mechanisms leading to the degradation of the bond in the post-peak regime of a displacement controlled, monotonic test. The primary dissipative mechanisms are the evolution of the plastic slip and the degradation of the unloading stiffness which defines the level of the damage [36, 48–50].

The plastic slip $s^p$ can be obtained for each point of the push-in curve as follow [51, 52]

$$s^p = s - \frac{F}{E_b}, \quad (6)$$

where $E_b$ is the unloading bond stiffness at each unloading/reloading cycle. On the other hand, the damage parameter representing the fraction of the deactivated material can be obtained as

$$\omega = 1 - \frac{E_b}{E_{b0}}, \quad (7)$$

where $E_{b0}$ defines the initial bond stiffness.

The development of plastic slip has been evaluated for each loading cycle as shown in Fig. 8b. In the cyclic push-in curve note that the irreversible slip is almost equal to the total applied slip, which is indicating that the bond process between concrete and ribbed reinforcement is governed primarily by plasticity. Similar observations have been reported in the literature e.g. [53–55]. In addition to the development of plastic slip, a slight reduction of the unloading bond stiffness can be observed. The damage parameter has been evaluated according to Eq. 7 and depicted in Fig. 8c.

*Bond degradation within each load cycle:* Besides the degradation of stiffness and the development of irreversible slip in the post-peak regime, another degradation effect on the bond strength can be observed within each loading cycle. This strength degradation $\Delta F$, shown in Fig. 8a, represents the difference of measured force between the begin and the end of the loading cycle. This effect highlights the dissipative mechanism activated during the loading cycle due to the internal friction. The values of $\Delta F$ normalized with respect to the ultimate load $F_u$ are depicted for each loading cycle in Fig. 8d. This evaluation introduces an essential criterion for the validation of the numerical models aiming to capture the cyclic and fatigue behavior of bond [56]. It reveals the cumulative nature of the damage evolution that requires the capturing of the dissipative processes
within load cycles in the formulation of the material models as presented e.g. in [37, 57–60].

4.2 Cyclic pre-peak bond behavior

**Cyclic push-in curve and hysteretic loops:** The cyclic push-in curve shown in Fig. 8c was obtained for the rebar diameter of 16 mm, and the bonded length of 2.5 ds. A cyclic failure was observed after 47 loading cycle. Similar to the post-peak response presented in Sec. 4.1, the pre-peak response indicates that the main inelastic dissipative mechanism governing the cyclic bond behavior is the development of the plastic slip. The response shows a small reduction of the bond stiffness as apparent by comparing the stiffness of the last and first loading cycles. To visualize the shape of the hysteretic loops, Fig. 8f provides a zoomed view of six cycles i.e. cycle 37 to cycle 42. The zoomed view shows that no observable difference between the shape and the area of the hysteretic loops can be distinguished. However, a remarkable difference of the amount of irreversible slip can be observed which is growing with the increased upper loading level. This observation implies that the area of the hysteretic loop is not the only part of energy dissipated during each load cycle. The studies presented in [61] analyzing energy dissipation during fatigue life show that an additional fraction of energy is dissipated outside the hysteretic loops.

**Fatigue creep curve and crack development:** The growth of the push-in slip at the upper and lower levels of loading is plotted in Fig. 8g. These fatigue creep curves show an increase of the slip rate at the last two load levels. The notion of fatigue creep curves has been chosen in this paper to underline the analogy to the static creep curves that display the deformation along the time axis at constant stress load. Similarly, in case of repeated loading the fatigue creep curves show the deformation for constant cyclic loading along the pseudo-time represented by the number of loading cycles. The development of the longitudinal and transverse cracks is depicted in Fig. 8h. It can be clearly observed that the slip growth during cyclic loading depends on the growth of the longitudinal splitting crack.

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Fig. 8 Post- and pre-peak cyclic bond behavior using the load scenarios LS2 and LS3: a push-in cyclic response; b the corresponding development of the plastic slip; c the evolution of damage; d bond degradation during cyclic loading; e push-in cyclic response; f zoomed view of selected loading cycles; g corresponding fatigue creep curves; h development of the surface cracks over the loading cycles.
5 Bond fatigue behavior

The fatigue life as well as the failure mode are summarized in Table 4 for all fatigue tests with constant amplitude (LS4). The results of the fatigue tests, particularly the high cycle fatigue tests, were extracted using a software tool with a number of filter functions and a graphical user interface called High-Cycle Fatigue Tool (HCFT), which is provided in [62]. This tool was developed using the ecosystem of Python packages for scientific computing [63, 64].

5.1 Fatigue creep curves and splitting cracks

The fatigue creep curves for a selected representative three tests from each parameter combination are plotted in Fig. 9a–c, e–g, i–k, m–o in black color together with the corresponding splitting crack width evolution during fatigue life depicted in red color. The representative tests have been selected to show all observed types of bond fatigue behavior, which can be distinguished based on the recorded level of splitting crack width. While the first column in Fig. 9 shows the tests with the largest observed width of the splitting crack, the second and third columns present tests with smaller width of the splitting crack, or for some cases even without a splitting crack, e.g. Fig. 9c. To provide a reference to the monotonic tests, the levels of average slip evaluated from the monotonic tests corresponding to the $F_{\text{crack}}$ and $F_u$ are included in all figures of the fatigue creep curves as dashed lines. The obtained fatigue creep curves show the typical shape of the deformation profile over the loading cycles with rapid increase in the first and the last stages and linear increase in the middle stage as in [7, 65, 66].

For the specimens with the larger bonded length i.e. $L_b = 5d_s$, a larger width of the splitting crack has been observed in comparison to the tests with the shorter

| Test | Case | ds [mm] | $L_b$ | Rebar end | $S_{\text{max}}$ | $S_{\text{min}}$ | Number of cycles | Failure mode |
|------|------|--------|------|----------|----------------|----------------|-----------------|-------------|
| T18  | 16   | 2.5 ds | Free end | 0.80 | 0.40 | 43,163 | CPTS |
| T19/T20 | 16 | 2.5 ds | Free end | 0.80 | 0.40 | 6,400,000*/5,000,000* | NNF |
| T21/T22 | 16 | 2.5 ds | Free end | 0.80 | 0.40 | 6,800,000*/3,000,000* | NNF |
| T23/T24/T25 | 16 | 2.5 ds | Free end | 0.85 | 0.40 | 146/54/949,071 | CPTS |
| T26  | 16   | 2.5 ds | Free end | 0.85 | 0.40 | 5,000,000* | NNF |
| T27  | 16   | 2.5 ds | Free end | 0.90 | 0.40 | 4,000,000* | NNF |
| T28  | 16   | 2.5 ds | Free end | 0.97 | 0.40 | 22 | CPTS |
| T29  | 16   | 2.5 ds | Free end | 0.99 | 0.40 | 105 | CPTS |
| T30/T31/T32 | 16 | 5.0 ds | Free end | 0.80 | 0.40 | 193/3,215,116/1,279,494 | CPTS |
| T33  | 16   | 5.0 ds | Free end | 0.85 | 0.40 | 5,100,000* | NNF |
| T34/T35 | 16 | 5.0 ds | Free end | 0.85 | 0.40 | 200/83 | CPTS |
| T36  | 25   | 2.5 ds | Free end | 0.80 | 0.40 | 5,400,000* | NNF |
| T37/T38 | 25 | 2.5 ds | Free end | 0.825 | 0.40 | 6,400,000*/5,000,000* | NNF |
| T39/T40 | 25 | 2.5 ds | Free end | 0.8375 | 0.40 | 5,000,000*/5,000,000* | NNF |
| T41/T42/T43 | 25 | 2.5 ds | Free end | 0.85 | 0.40 | 439,403/139,472/91,685 | CPTS |
| T44  | 25   | 2.5 ds | Free end | 0.875 | 0.40 | 1,754 | CPTS |
| T45  | 25   | 2.5 ds | Comp. end | 0.80 | 0.40 | 5,400,000* | NNF |
| T46  | 25   | 2.5 ds | Comp. end | 0.85 | 0.40 | 7,200,000* | NNF |
| T47  | 25   | 2.5 ds | Comp. end | 0.85 | 0.40 | 5,188,548 | CPTS |
| T48  | 25   | 2.5 ds | Comp. end | 0.875 | 0.40 | 432,103 | CPTS |
| T49  | 25   | 2.5 ds | Comp. end | 0.875 | 0.40 | 10,000,000* | NNF |
| T50  | 25   | 2.5 ds | Comp. end | 0.90 | 0.40 | 3,359,342 | CPTS |

CPTS: combined push-through × splitting failure mode; NNF: No fatigue failure; (*) denotes a run out test i.e. no fatigue failure
Bond fatigue behavior under constant amplitudes (LS4): (a–c, e–g, i–k, m–o) fatigue creep curves with corresponding crack development for all studied cases; (d, h, l, p) comparison of the results with the existing Wöhler curves in the literature obtained from Pull-out tests.

fatigue tests with the compression end reached a fatigue failure at larger values of slip. The comparison of the width of the splitting crack measured in the tests with a free end and compression end is plotted in Fig. 9m–o.
The third column in Fig. 9 presents selected very high-cycle fatigue tests with no fatigue failure after 5 millions of loading cycles. It can be observed, that the level of slip after 5 millions of cycles is less than the average slip corresponding to the monotonically obtained maximum push-in force \( w(F_{\text{u}}) \). The development of the splitting crack width measured in these tests shows an almost constant level during the fatigue life, whereas an increased width of the splitting crack width was observed in the tests exhibiting fatigue failure. In summary, all results presented in Fig. 9 indicate a direct interaction between the push-in fatigue response, i.e. the fatigue creep curve, and the growth of the splitting crack width.

5.2 Effect of splitting cracks on the bond fatigue life

The recorded number of cycles to fatigue failure for varied upper load levels \( s_{\text{max}} \) for all studied cases are summarized in Table. 4, and shown in Fig. 9d–p. The results show a very large scatter in terms of the number of cycles to fatigue failure for the same \( s_{\text{max}} \) so that no clear trend of a Wöhler curve can be proposed.

If we compare the tests (T24) and (T25) with the same rebar diameter, bonded length and the same upper load \( S_{\text{max}} = 0.85 \) shown in Fig. 9a, b, we find out that the former (T24) failed after 54 cycles, whereas the latter (T25) failed after 949071 cycles, indicating that the huge scatter prevents the identification of a Wöhler curve. However, if we regard the width of the splitting crack developing during the test, we notice that the achieved number of cycles at fatigue failure strongly correlates with the splitting crack evolution. Similar trends have been observed in all cases as highlighted in Fig. 9d–p with the gray band indicating the dependency of the fatigue life on the level of splitting crack width. The scatter of the recorded results of the splitting crack width evolution explains the huge scatter of the fatigue life i.e. the number of cycles to fatigue failure.

To compare the results with the available Wöhler curves for bond fatigue, the approximation proposed in [2] for pull-out configuration has been selected. This approximation is denoted by a straight line in the semi-logarithmic plot between the upper load level \( S_{\text{max}} \) and the number of cycles \( \text{Log}(N) \) given as

\[
S_{\text{max}}(N) = C \cdot N^m,
\]

where \( C \) and \( m \) are parameters controlling the initial position and the slope of the Wöhler curve that have been determined based on the pull-out fatigue tests performed with a normal strength concrete [17]. In this paper, also the effect of splitting cracks, has been investigated on pre-cracked pull-out specimens. The parameters \( C \) and \( m \) have been identified for the tests with a fully confined bond without splitting cracks \( (C = -0.019, m = 0.880) \), as well as for the tests with pre-cracked specimens with the crack width of 0.3 mm \( (C = -0.059, m = 1.016) \) for the lower load level \( S_{\text{min}} = 0.40 \).

Another approximation of the Wöhler curves was proposed by Rehm and Eligehausen 1979 [6] based on the pull-out tests with normal-strength concrete and steel rebars in the following form

\[
S_{\text{max}}(N) = \begin{cases} 
0.89 - 0.037 \cdot \text{Log}(N), & \text{for } S_{\text{min}} = 0.1 \\
0.89 - 0.022 \cdot \text{Log}(N), & \text{for } S_{\text{min}} = 0.3 
\end{cases}
\]

(9)

The comparison of pull-out Wöhler curve approximations obtained for the uncracked and cracked bond with the results obtained for push-in loading are plotted in the last column of Fig. 9 in black and red colors, respectively, as well as the Wöhler curves in Eq. (9) plotted in green color. This comparison has been performed, since other results for push-in loading are not available in the literature. It can be seen that the beam-end tests subjected to push-in loading exhibit longer fatigue life than the pull-out tests presented in [6, 17] as documented using the approximation of the Wöhler curves in Fig. 9d–p. The difference is particularly evident in the tests with small developed splitting crack width. Furthermore, the tests with the compression end exhibit a longer fatigue life with a lower scatter than the tests with free end as shown in Fig. 9i, p. The reason for the longer fatigue life can be ascribed to the contribution of the concrete loaded in compression. However, the longer fatigue life is accompanied with a significantly larger opening of the splitting cracks.

5.3 Fatigue creep evolution compared to FIB model code

The slip evolution during the fatigue life \( w_{\text{u}} \) proposed in the FIB model code 2010 [1] has the following form
This approximation is based on the results of the large test program presented in [6], where \( w_0 \) is the slip at the first loading cycle corresponding to the upper load level \( F_{\text{max}} \), \( N \) is the number of cycles, and \( b \) is a constant controlling the rate of slip increase over the loading cycles, which can be identified from the experimental data. For the pull-out fatigue test program performed by Rehm and Eligehausen [6], the mean value of all tests was \( b = 0.107 \), while the mean value of the test program by Koch and Balazs [67] was \( b = 0.148 \).

To check the validity and applicability of the formula (Eq. 10) the range of values for the exponent \( b \) has been approximately identified by considering the fastest and the slowest measured growth of the fatigue creep curves. For this purpose, the fatigue tests were divided into two groups, distinguishing between the tests exhibiting fatigue failure and the run-out tests for all four studied cases. The fatigue creep curves for all performed 33 tests are plotted in Fig. 10a, c in semi-logarithmic diagrams with the corresponding fitted range using the analytical formula described in Eq. 10. These results show that the approximation (Eq. 10) can realistically describe the fatigue behavior in the case that no fatigue failure occurs, as shown in Fig. 10c. However, in the case of fatigue failure, the approximation (Eq. 10) is valid until the end of the second phase of the fatigue creep curve as depicted in Fig. 10a. These results reveals that this approximation (Eq. 10) has a limited range of validity and can not describe the last accelerated phase of the fatigue creep development and therefore cannot predict fatigue failure.

The range of the exponent \( b \) for both groups is summarized in Fig. 10b, d with mean values of \( b = 0.217 \) and \( b = 0.098 \) for the tests with fatigue failure and the run-out tests, respectively, depicted as red lines. The comparison of the \( b \)-exponent values identified for the conducted push-in tests with the average values of \( b \) identified for the pull-out tests presented in [6, 67] is provided in Fig. 10b, d.

5.4 Observed mechanisms of bond fatigue and failure modes

In the current study, the region close to the bonded length can be regarded as a partially confined through the transverse reinforcement with relatively small concrete cover of 2 ds. This configuration makes it possible to study the effect of a stabilized splitting crack on the bond fatigue behavior, which represents a highly relevant case in the design of structural members. The fatigue failure failure mode observed in the tests was the combined push-through \( \times \) splitting as summarized in Table. 4 It should be noted that a considerable scatter in the magnitude of the developed splitting crack width was observed, leading to a scatter in the fatigue life, as already discussed in Sec. 5.2. Moreover, in most of the run-out high-cycle fatigue tests with no observable fatigue failure, a very small width of the splitting crack was recorded. To emphasize the behavior of partially-confined bond under fatigue loading in the push-in mode, qualitative fatigue-creep curves are sketched in Fig. 11a. Let us also note, that the patterns of surface cracks in the fatigue tests were similar to those in the monotonic tests, as shown in Fig. 6.

5.5 Features of the bond fatigue behavior

As illustrated in Fig. 11b, the current study, as well as the previously published results of the bond fatigue in high strength concrete members [28], show that according to the level of confinement, three qualitatively different types of push-in fatigue response can be distinguished:

- (I) Fully confined bond: If no splitting crack can develop due to a large concrete cover, the slip increases in a stable manner during the whole fatigue life.
- (II) Partially confined bond: If stabilized splitting cracks can develop due to sufficient transverse reinforcement, even with relatively low concrete cover, a more rapid slip development during the fatigue life can be expected, resulting in a significantly shorter fatigue life.
- (III) Unconfined bond: If there is no transverse reinforcement and the concrete cover is small, an unstable splitting crack can be expected, leading to sudden fatigue failure.

It should be noted that for C40 grade, the splitting crack appeared during the first loading cycle, while the for C80 and C120 grades tests was distributed randomly along the whole fatigue life. Therefore,
their occurrence was induced by the bond fatigue. This scenario is qualitatively depicted in Fig. 11b.

### 6 Conclusions

Based on the test results of this research project on concrete-to-steel reinforcement bond under monotonic, cyclic and fatigue loading in the push-in mode, the following conclusions can be drawn:

- The proposed modification to the usual beam-end test in the push-in mode makes it possible to study the influence of the concrete compression end on the bond behavior.
- For monotonic loading, short bonded length of 2.5 ds, rebar diameter of 25 mm and the thickness of concrete behind the compression end of 7.5 ds a roughly 50% higher ultimate push-in force was obtained in the tests with compression end compared to the tests with free end.
A more sudden failure with a three times larger width of the splitting cracks was observed in the tests with compression end in comparison to the tests with free end.

The pre- and post-peak cyclic behavior shows that the main dissipative mechanism of the cyclic bond behavior is the development of irreversible plastic slip.

The shape and area of the hysteretic loops occurring under the pre-peak cyclic response indicates that this area is not the only part of the energy dissipation during cyclic and fatigue loading.

Significantly longer fatigue life of the bond was observed in the tests with a compression end compared to the tests with a free end.

The results show that the fatigue creep curve approximation of the bond proposed by the FIB Model Code 2010 has a limited validity and cannot correctly reproduce the accelerated last stage during the fatigue life.

Based on the results, the bond fatigue behavior can be categorized into fully confined, partially confined, and unconfined type of bond depending on the development of splitting cracks during fatigue life as described graphically in Fig. 11b.

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**Declarations**

**Conflict of Interest** The authors declare that they have no conflict of interest.

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