Anchorage behavior of Fe-SMA rebars Post-Installed into concrete

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ABSTRACT

The unique self-prestressing behavior exhibited by Iron-based shape memory alloys (Fe-SMAs) coupled with the ease of application makes them suitable for numerous structural applications requiring post-installed reinforcement. However, the practical realization of Fe-SMA rebars as post-installed reinforcement systems entails an understanding of their bond behavior with concrete to ensure the use of adequate anchorage length for prestress transfer and prevention of premature failure. The bond behavior of cast-in-place and near-surface mounted Fe-SMA rebars with concrete has been investigated in detail in a few recent studies. However, no study has so far investigated the bond behavior of Fe-SMA rebars post-installed into concrete. To address this important knowledge gap, this paper presents an extensive experimental campaign aimed at evaluating the anchorage of post-installed Fe-SMA rebars in concrete. The experimental campaign comprised 23 specimens, with the variable parameters of the study including the bond length, state of Fe-SMA rebar (non-activated or activated), drill-hole size, strength of the adjacent concrete, type of rebar (plain, threaded, or deformed), and the type of loading history (monotonic or cyclic). The experimental results showed that for bond lengths \( \geq 5d_p \), the ribbed Fe-SMA rebars primarily fail in a pull-out failure mode and exhibit large inelastic rebar strains and load capacity at bond failure. The paper is concluded with the comparison of anchorage length predictions of conventional design models with the results of the current experimental testing campaign. The comparison demonstrates that the anchorage length recommendations of existing design standards are generally conservative for Fe-SMA rebars.

1. Introduction

1.1. Post-installed reinforcement systems

Post-installed reinforcement refers to the rebars that are installed in holes which are mechanically drilled into the existing concrete and bonded using an adhesive. The post-installed reinforcement systems are used for a range of structural applications, including the extension of structural elements to increase the floor space, end-connection of new beam/slab into walls, anchorage of the new column into the foundation, and new concrete overlays for strengthening/retrofitting purposes [1]. Hence, these systems are useful for horizontal, vertical as well as overhead applications, and provide a lot of design flexibility. The post-installed rebars are mainly provided to ensure a safe and reliable connection between the existing and new concrete so that the transfer of forces can be accomplished in an intended manner.

The serviceability and ultimate limit state behavior of the reinforced concrete structures retrofitted with the post-installed rebars strongly depend on the bond behavior of the post-installed reinforcement with concrete. The bond mechanism is particularly important to attain a ductile behavior upon failure of a reinforced concrete structure. According to the current design philosophy, the anchorage length design of post-installed rebars can be done according to the existing reinforced concrete design standards provided that the post-installed rebars experimentally exhibit bond strength and stiffness comparable to the cast-in-place rebars [2]. Therefore, for design purposes, the bond behavior of post-installed rebars is at maximum considered equivalent to the cast-in-place rebars even though they might exhibit greater bond strength and stiffness due to the use of high-strength mortar. The evaluation requirements for assessing and comparing the bond behavior of the post-installed rebars with cast-in-place rebars are described in detail in EAD-330087 [3], EAD-331522 [4] for Europe, and in AC 308 [5] for the US.

In the past, the bond behavior of post-installed steel reinforcement with concrete has been investigated in detail under monotonic loading. Spieth et al. [6] reported that straight post-installed rebars can exhibit similar or greater bond strength than straight cast-in-place rebars.
However, it was shown that the bond strength of straight post-installed rebars can be lower than bent cast-in-place rebars with standard hooks [7]. Ahmed et al. [8] evaluated the bond behavior of post-installed high-strength steel rebars with concrete and reported that epoxy resins are effective bonding agents. Recently, a new beam-end specimen has been proposed in [9] to evaluate the bond behavior of post-installed rebar. Mahadik et al. [10] performed a numerical study to re-evaluate the existing tests on reinforced concrete (RC) connections with post-installed rebars to explore the influence of the assumptions and boundary conditions on the original conclusions. Lee et al. [11] proposed a strut and tie model to design the post-installed rebars in wall-slab moment connections.

A ductile bond behavior under cyclic loading is essential as the sudden loss of bond between the reinforcement and concrete in the anchorage zones can lead to brittle failure under cyclic loading. In addition, the hysteretic behavior of reinforced concrete structures under cyclic loading strongly depends on the bond stress relationship between concrete and reinforcement as bond deterioration under repeated loads can significantly reduce the stiffness of RC structures. Ismail and Jirsa [12] reported that the most important parameter affecting the bond behavior under cyclic loading is the peak stress reached in the previous cycles. Hofacker and Elgiehausen [13] assessed the pull-out behavior of post-installed steel rebars under reversed cyclic loading and concluded that the post-installed rebars may exhibit poor behavior under monotonic and reversed cyclic loading as compared to the cast-in-place rebars if the pullout under monotonic loading is caused by failure of the bond between mortar and concrete instead of bond failure between mortar and rebar. This is an important finding that underscores the importance of evaluating the bond behavior of new post-installed reinforcement systems under cyclic loading before using them in practice.

1.2. Iron-based shape memory alloy reinforcement

Until now, most of the post-installed reinforcement systems comprised steel reinforcement due to their wide range of applications, cost-effectiveness, and availability. However, more recently iron-based shape memory alloys (Fe-SMAs) have emerged as an attractive and effective alternative for many strengthening and retrofitting applications [14]. The main advantage offered by Fe-SMAs is the self-prestressing ability upon heating and hence offering a positive effect to the existing concrete substrate by compressing the concrete as well as partly releasing stresses in the tensile reinforcement. This results from the unique shape memory effect exhibited by Fe-SMA which is the characteristic owing to which a material can recover inelastic deformations upon heating. This property can be used to generate recovery stress in Fe-SMA if the recovery of the inelastic strains during the heating (activation) process is prevented by using clamps, mechanical end-anchors, cementitious anchorages, or similar. The shape memory effect of Fe-SMA can be triggered in two steps to generate the prestress [15–16]. The first step involves prestraining of Fe-SMA and then unloading to zero load as schematically shown in Fig. 1 (a). Upon unloading, Fe-SMA exhibits a deviation from the linear elastic behavior, which is due to the small pseudoelasticity in the material. The pseudoelastic strain i.e. the strain that is recovered nonlinearly upon unloading is generally in the range of 0.15–0.2 % for Fe-SMA rebars prestrained to 4 %. The second step involves activation in which Fe-SMA is heated to 160 °C or above followed by cooling down to ambient temperature. During this process, Fe-SMA partially recovers the remaining permanent strain in the material, as delineated in Fig. 1 (a). This strain is referred to as the recovery strain. If during the activation process, the recovery of strain is prevented by restraining Fe-SMA at both ends (e.g. by anchorage into concrete), then this can result in the generation of recovery stress, as shown in Fig. 1 (b). The figure illustrates that during the early stages of heating, Fe-SMA tends to expand and as a result, the stress decreases. However, beyond the transformation temperature, the shape memory effect of Fe-SMA is triggered and recovery stress is generated owing to the austenite to martensite phase transformation. The recovery stress further increases on cooling due to the thermal contraction of SMA. The readers are referred to [16] for a more detailed explanation of the phase transformation behavior of Fe-SMA associated with the shape memory effect. More recent research has shown that additively manufactured Fe-SMA by laser powder bed fusion method also exhibits pronounced pseudoelasticity in addition to the shape memory effect compared to Fe-SMA manufactured by conventional methods [17].

A general overview of the applications of prestressed Fe-SMAs for civil engineering structures was provided in [18]. Michels et al. [19] demonstrated the potential of prestressed Fe-SMA strips for flexural
strengthening of concrete. Rojob and El-Hacha [20–21], El-Hacha and Rojob [22] and Hong et al. [23] showed a significant improvement in the serviceability and ultimate limit state load and deformation performance of RC beams retrofitted with near-surface-mounted (NSM) Fe-SMA rebars and strips, respectively. Streider et al. [24] used mechanically end-anchored Fe-SMA strips for the flexural strengthening of RC beams and demonstrated a significant improvement in the cracking and ultimate loads. Montoya-Coronado et al. [25] and Cladera et al. [26] demonstrated the suitability of externally anchored prestressed Fe-SMA strips in shear strengthening of deficient RC beams. More recently, Czaderski et al. [27] have used U-shaped ribbed Fe-SMA stirrups embedded in a shotcrete layer for improving the shear performance of deficient RC beams. The efficacy of prestressed Fe-SMA rebars in strengthening RC bridge decks has been illustrated in [28–29]. A few studies have also performed finite element modelling of RC beams and masonry walls strengthened with NSM Fe-SMA strips [30–31] and RC beams strengthened with Fe-SMA rebars embedded in a shotcrete layer [32].

Few recent studies have characterized the cyclic behavior of non-prestressed Fe-SMA rebars under tension–compression reversals to evaluate their suitability for seismic damping applications. It has been reported that Fe-SMAs exhibit high post-yield stiffness along with excellent ductility and fatigue behavior compared to steel-bar-based dampers [33]. Rosa et al. [34] found that Fe-SMAs show a higher hardening response than conventional steels. More recently, the recovery stress loss behavior of prestressed Fe-SMA rebars under cyclic loading reversals has been characterized extensively in [35] to facilitate their useability for seismic retrofitting applications.

An adequate anchorage length and concrete cover is needed to ensure the proper transfer of recovery stress from Fe-SMA rebars to the concrete substrate. In addition, adequate bond strength of rebars with concrete prevents a premature brittle failure. Prestressed Fe-SMA rebars have a promising potential to be used for strengthening old RC structures that were made with low-strength concrete of poor quality. In such cases, it is important to investigate if the strength of the confining concrete has any significant effect on the pullout behavior of the post-installed rebars. Furthermore, the prestressing force needs to be controlled/determined considering the strength of the concrete, otherwise, it might result in the crushing of the low-strength concrete. The bond behavior of non-prestrained NSM ribbed Fe-SMA rebars with short embedment lengths (5d_b) has been thoroughly investigated in [36]. In this study, the maximum bond strength of 16 mm Fe-SMA rebars, provided with a cover depth of 2d_b and mortar compressive strength of 94 MPa, was reported to be about 18 MPa. Fawaz and Murcia [37] reported a similar maximum bond strength for cast-in-place embedded Fe-SMA rebars of the same diameter and bond length. However, in contrast to [37], these rebars had an initial prestrain of about 4 % and were provided with a concrete cover greater than 4d_b. Furthermore, the concrete compressive strength was relatively lower i.e. 45 MPa in this study. Based on the analytical estimation, the minimum transfer length required for recovery stress of 250 MPa was found to be about 11d_b for embedded Fe-SMA rebars. In a separate study, Schranz et al. [38–39] evaluated the bond behavior of NSM Fe-SMA rebars (activated and non-activated) for long embedment lengths and concluded that the required anchorage length to reach the tensile rupture (800 MPa stress) of the 12 mm diameter Fe-SMA rebars is greater than 33d_b. On the other hand, no study has so far investigated the bond-slip behavior of post-installed Fe-SMA rebars that have the potential to be used in many retrofitting applications. It is important to investigate the bond-slip behavior of post-installed rebars because the average bond strength of cast-in-place Fe-SMA rebars has been reported to be lower than the conventional steel rebars and thermal activation of Fe-SMA further reduces the bond strength [37]. Furthermore, such an investigation is needed to establish an appropriate bond length required for the transfer of prestress and adequate anchorage of rebars. Similarly, for seismic retrofitting applications, the effect of cyclic loading on the bond deterioration of Fe-SMA rebars needs to be understood. It is important to mention here that the bond behavior of Fe-SMA rebars under cyclic loading has not been investigated in the past.

To address the outlined research gaps, this study aims to investigate the bond behavior of post-installed Fe-SMA rebars under monotonic and cyclic loading. The description of specimens, test set up, and loading protocols is provided in the next section, followed by the results and discussion section where the results of the experimental campaign are described in detail. The paper is concluded with the comparison of anchorage length recommendations of design standards with the results of the current experimental testing campaign.

2. Experiments

2.1. Specimen details and preparation

The experimental testing campaign comprised 23 specimens with post-installed Fe-SMA rebars. Concrete block sizes of 250 × 200 × 300
The prestrained activated and prestrained non-activated specimens refer to those prestrained rebars that were thermally activated after the installation. On the other hand, the non-activated specimens refer to the rebars that were received from the manufacturer with an initial prestrain of about 4% and were not thermally activated after the installation. For simplicity, the prestrained activated and prestrained non-activated specimens will be referred to as activated and non-activated specimens throughout the manuscript. Prestraining is performed by a hydraulic jack setup. The bar is clamped on one side with a fixed grip and a mobile one on the other, both elements being anchored against a rigid steel frame. A hydraulic jack with a control device prestrains the bar with a defined stroke based on a required strain level. The chemical composition of Fe-SMA rebars used in this study is Fe–17Mn–5Si–10Cr–4Ni–1(V,C) (numbers in mass%). The modulus of elasticity of non-activated and activated Fe-SMA rebars is about 160–180 GPa and 75–100 GPa, respectively [16,35]. The average yield stress, ultimate stress, and failure strain of the rebars are about 400 MPa, 800 MPa, and 40%, respectively. The recovery stress of the rebars upon thermal activation at 160 °C is about 300 MPa.

The specimen preparation included the drilling of holes into the concrete blocks using a diamond core drilling machine. The drilled holes were cleaned using compressed air and steel wire brushes. The concrete parameters of the study included bond length, state of Fe-SMA rebar (prestrained non-activated or prestrained activated), drill-hole size, the type of loading history (monotonic or cyclic). The plain, ribbed and threaded Fe-SMA rebars used in this study are commercially available and were supplied by the Swiss company re-

Table 1
Matrix for Pull-out Tests.

<table>
<thead>
<tr>
<th>No.</th>
<th>Block Size (mm)</th>
<th>Concrete Strength $f_{con}(MPa)$</th>
<th>Grout Strength $f_{grout}(MPa)$</th>
<th>Bond Length $d_{b}$ (mm)</th>
<th>$d_{hole}$ (mm)</th>
<th>State</th>
<th>Loading Type</th>
<th>Results $F_{max}$ (kN)</th>
<th>$\varepsilon_{max}$ rebar (%)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>250 × 200 × 300</td>
<td>63</td>
<td>89</td>
<td>5d$_b$</td>
<td>30</td>
<td>16.5</td>
<td>Activated (Ribbed)</td>
<td>Monotonic</td>
<td>110</td>
<td>3.9</td>
</tr>
<tr>
<td>2.</td>
<td>250 × 200 × 300</td>
<td>63</td>
<td>80</td>
<td>8d$_b$</td>
<td>30</td>
<td>16.5</td>
<td>Activated (Ribbed)</td>
<td>Monotonic</td>
<td>124</td>
<td>8.6</td>
</tr>
<tr>
<td>3.</td>
<td>250 × 200 × 300</td>
<td>63</td>
<td>84</td>
<td>12d$_b$</td>
<td>30</td>
<td>16.5</td>
<td>Activated (Ribbed)</td>
<td>Monotonic</td>
<td>126</td>
<td>9.2</td>
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<tr>
<td>4.</td>
<td>500 × 400 × 300</td>
<td>61</td>
<td>86</td>
<td>16d$_b$</td>
<td>30</td>
<td>16.5</td>
<td>Activated × 1 + Non-Activated × 1</td>
<td>Monotonic</td>
<td>128</td>
<td>9.7</td>
</tr>
<tr>
<td>5.</td>
<td>500 × 400 × 300</td>
<td>61</td>
<td>80</td>
<td>12d$_b$</td>
<td>30</td>
<td>16.5</td>
<td>Activated × 1 + Non-Activated × 1</td>
<td>Monotonic</td>
<td>124</td>
<td>8.3</td>
</tr>
<tr>
<td>6.</td>
<td>250 × 200 × 300</td>
<td>58</td>
<td>70</td>
<td>8d$_b$</td>
<td>30</td>
<td>16.5</td>
<td>Non-Activated (Ribbed)</td>
<td>Monotonic</td>
<td>126</td>
<td>9</td>
</tr>
<tr>
<td>7.</td>
<td>500 × 400 × 300</td>
<td>53</td>
<td>74</td>
<td>16d$_b$</td>
<td>30</td>
<td>16.5</td>
<td>Non-Activated × 2 (Ribbed)</td>
<td>Monotonic</td>
<td>129</td>
<td>10.5</td>
</tr>
<tr>
<td>8.</td>
<td>250 × 200 × 300</td>
<td>55</td>
<td>76</td>
<td>12d$_b$</td>
<td>24</td>
<td>16.5</td>
<td>Non-Activated (Ribbed)</td>
<td>Monotonic</td>
<td>125</td>
<td>8</td>
</tr>
<tr>
<td>9.</td>
<td>250 × 200 × 300</td>
<td>55</td>
<td>76</td>
<td>12d$_b$</td>
<td>30</td>
<td>16.5</td>
<td>Non-Activated (Ribbed)</td>
<td>Monotonic</td>
<td>128</td>
<td>9</td>
</tr>
<tr>
<td>10.</td>
<td>250 × 200 × 300</td>
<td>55</td>
<td>76</td>
<td>12d$_b$</td>
<td>36</td>
<td>16.5</td>
<td>Non-Activated (Ribbed)</td>
<td>Monotonic</td>
<td>128</td>
<td>10</td>
</tr>
<tr>
<td>11.</td>
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<td>16d$_b$</td>
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<td>16.5</td>
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<td>Monotonic</td>
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<tr>
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<td>8d$_b$</td>
<td>24</td>
<td>10.5</td>
<td>Non-Activated (Ribbed)</td>
<td>Monotonic</td>
<td>51</td>
<td>5</td>
</tr>
<tr>
<td>13.</td>
<td>250 × 200 × 300</td>
<td>13</td>
<td>77</td>
<td>8d$_b$</td>
<td>24</td>
<td>10.5</td>
<td>Non-Activated (Ribbed)</td>
<td>Monotonic</td>
<td>51</td>
<td>5.5</td>
</tr>
<tr>
<td>14.</td>
<td>250 × 200 × 300</td>
<td>23</td>
<td>70</td>
<td>8d$_b$</td>
<td>24</td>
<td>10.5</td>
<td>Non-Activated (Ribbed)</td>
<td>Monotonic</td>
<td>51</td>
<td>5</td>
</tr>
<tr>
<td>15.</td>
<td>250 × 200 × 300</td>
<td>23</td>
<td>77</td>
<td>8d$_b$</td>
<td>24</td>
<td>10.5</td>
<td>Non-Activated (Ribbed)</td>
<td>Monotonic</td>
<td>53</td>
<td>6</td>
</tr>
<tr>
<td>16.</td>
<td>250 × 200 × 300</td>
<td>13</td>
<td>70</td>
<td>12d$_b$</td>
<td>24</td>
<td>10.5</td>
<td>Non-Activated (Ribbed)</td>
<td>Monotonic</td>
<td>57</td>
<td>11</td>
</tr>
<tr>
<td>17.</td>
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<td>78</td>
<td>15d$_b$</td>
<td>30</td>
<td>18</td>
<td>Non-Activated × 2 (Plain)</td>
<td>Monotonic</td>
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<tr>
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<td>53</td>
<td>78</td>
<td>14d$_b$</td>
<td>30</td>
<td>19.3</td>
<td>Non-Activated × 2 (Threaded)</td>
<td>Monotonic</td>
<td>181</td>
<td>6.5</td>
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<tr>
<td>19.</td>
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<td>80</td>
<td>16d$_b$</td>
<td>30</td>
<td>16.5</td>
<td>Non-Activated × 2 (Ribbed)</td>
<td>Monotonic</td>
<td>130</td>
<td>7</td>
</tr>
<tr>
<td>20.</td>
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<td>78</td>
<td>16d$_b$</td>
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<td>16.5</td>
<td>Non-Activated × 2 (Ribbed)</td>
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<td>130</td>
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<td>Non-Activated × 2 (Ribbed)</td>
<td>Monotonic</td>
<td>180</td>
<td>6</td>
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<td>16d$_b$</td>
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<td>129</td>
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<td>23.</td>
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<td>16d$_b$</td>
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<td>Non-Activated × 2 (Ribbed)</td>
<td>Monotonic</td>
<td>132</td>
<td>7</td>
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Fig. 2. Details of pullout specimens: a) 200 × 250 × 300 mm concrete block; b) 400 × 500 × 300 mm concrete block; and c) set up for resistive heating.
blocks were submerged into water for at least two hours before grouting the rebar to ensure a good bond at the interface of the concrete and grout. It should be noted that for large-scale practical applications, it is not possible to submerge the whole element into water. In such cases, the drilled holes should be thoroughly wetted by injecting water. A shrinkage-compensated high strength mortar of type Sikagrout 311 [40], with a maximum aggregate size of 1 mm, was used for bonding the post-installed rebars. The grout was mixed according to the datasheet. The grout strength (refer Table 1) was investigated by testing of 40 × 40 × 160 mm mortar prisms on the testing day. Similarly, the compressive strength of concrete blocks was determined by testing cubes of 150 × 150 × 150 mm on the test day. The concrete blocks with 500 × 400 × 300 mm size were drilled with two holes spaced at a clear distance of 80 mm, as shown in Fig. 2(b). The Fe-SMA rebars were installed in both holes, whereas only one rebar was pulled out during the test. This was done to study the effect of the presence of another rebar in the vicinity of the rebar being pulled out, as in retrofitting operations usually a number of rebars are installed close to each other. Three types of rebar surfaces i.e. plain, ribbed, and threaded surfaces were used to investigate the effect of rebar surface type on the bond behavior. Most of the pull-out experiments were performed on 16.5 mm ribbed Fe-SMA rebars. 10.5 mm ribbed Fe-SMA rebars were used for pull-out experiments on lower quality concrete to reduce the bond stresses and possible splitting of the block.

The thermal activation of Fe-SMA rebars was performed after 3 days of grouting. The activation was performed by resistive heating wherein 535A current was passed through the two ends of the rebar using a custom-made electric power supply, as shown in Fig. 2(c). This process would essentially activate the bonded part of the SMA rebar. Three thermocouples of type K were attached to the unbonded region of the rebar outside the concrete block for monitoring the temperature. The target temperature for activation of the bonded region of the rebar was 160 °C. The power supply was stopped when the temperature in thermocouples attached to the unbonded region of the rebar reached 220 °C. This was done based on the findings of previous studies [41], where it was shown that the temperature of the rebar in the bonded and unbonded regions differs by 60 °C–100 °C approximately.

2.2. Experimental test setup

The pullout tests were conducted on a servo-hydraulic testing machine of make Amsler with a load and stroke capacity of 200 kN and 200 mm, respectively. The concrete blocks were mounted on the traverse of the machine, as shown in Fig. 3. The bottom end of the rebar was clamped to the machine from where the pullout load was applied. A linear variable differential transducer (LVDT) with a stroke of 10 mm was attached to the top end of the rebar for measuring the end slip. The strain in the rebar was calculated using the displacements measured by an LVDT (stroke 200 mm) attached to the free length of the rebar. The LVDT measured the rebar displacement over a gauge length of 275 mm. The calculated strains are the average strains in the free length of the rebar.

2.3. Loading protocols

The bond behavior of post-installed Fe-SMA rebars was evaluated under monotonic loading by subjecting the specimens to a displacement-controlled loading with a machine speed of 1.2 mm/min until rebar pullout, fracture, or splitting of the concrete block. The cyclic bond behavior of post-installed Fe-SMA rebars was investigated using two different cyclic tension loading histories. The incrementally increasing displacement-controlled loading protocol was designed to account for the loading and unloading cycles experienced by the reinforcement in structural components under seismic actions. In this loading protocol, the machine-controlled cyclic displacements were applied in increments of 10 mm at a rate of 2.4 mm/min until failure. The speed of the cyclic tests was double of the corresponding monotonic tests to reduce the total time required for the cyclic tests. To avoid compression, the specimens were unloaded to zero force instead of zero displacement at the end of each displacement increment.

On the other hand, the constant loading protocol comprised 100 cycles of 8–80 kN load applied at a rate of 1 kN/s, followed by displacement-controlled monotonic loading (1.2 mm/min) until failure. The 8–80 kN load range was selected to maintain an R ratio of 10. The maximum load limit was chosen to be 80 kN to evaluate the bond behavior at load amplitudes close to the yielding of the rebar. This protocol aims to represent the repeated loadings experienced by the RC beams and girders in bridge structures under the serviceability limit state. The overall purpose of using the cyclic loading protocols was to evaluate the extent of deterioration in bond under cyclic loading compared to the monotonic loading.

3. Results and discussion

3.1. Load-slip behavior under monotonic loading

3.1.1. Effect of bond length

The load-slip behavior of specimens 1–4 with bond lengths of 5db, 8db, 12db, and 16db of activated Fe-SMA rebars is shown in Fig. 4. The lowest pullout load capacity was obtained for the bond length of 5db, whereas the other three bond lengths i.e. 8db, 12db, and 16db exhibited similar maximum load capacities. This implies that among the bond lengths considered, the active bond length is 8db, beyond which only an insignificant increase in the load capacity could be obtained. The zoomed-in load-slip response up to a slip of 0.1 mm is shown in Fig. 4(b), which indicates that an increase in the bond length increased the initial stiffness of the load-slip curve. The secant stiffness of the load-slip response was calculated based on the reduced stiffness equivalent elasto-plastic yield method [42] and was found to be 1.25 kN/m, 5.1
kN/m, 10.2 kN/m, and 15 kN/m, respectively, for the bond lengths of 5d_b, 8d_b, 12d_b, and 16d_b. This shows that an increase of 50 % and 100 % in the bond length i.e. from 8d_b to 12d_b and 8d_b to 16d_b increased the secant stiffness of the load-slip response by 100 % and 200 %, respectively. Thus the increase in stiffness is two times the increase in bond length for these cases. On the other hand, a 33 % increase in the bond length i.e. from 12d_b to 16d_b increased the secant stiffness by 50 % only. These results indicate that the rate of increase in the secant stiffness of the load-slip curve reduces at larger bond lengths. It is also interesting to note that due to increased stiffness, the specimens with longer bond lengths attained the ultimate load at a relatively lower slip compared to the specimens with smaller bond lengths.

The mode of failure observed in all cases was pullout wherein a localized failure at the interface of rebar and grout occurred due to the shearing of the grout, as shown in Fig. 5. This was expected because all the specimens were well-confined with a concrete cover of more than 5.5d_b. It is noted that the confinement is considered high when the concrete cover exceeds 3–4 times the rebar diameter [43]. The Fe-SMA rebar yielded in all cases. As such, the maximum rebar strain attained at failure was 3.9 %, 8.6 %, 9.2 %, and 9.7 % for the bond lengths of 5d_b, 8d_b, 12d_b, and 16d_b, respectively, as shown in Fig. 4 (c). This indicates that the post-installed SMA rebars can successfully develop and transfer yield stress even at a short anchorage length of 5d_b.

The effect of the presence of another rebar in the vicinity of the rebar being pulled out was studied by comparing the load-slip behavior of the rebar installed in the 250 × 200 × 300 mm concrete block (specimen 3) with that of the one installed in 500 × 400 × 300 mm block (specimen 5), which had another rebar installed in the vicinity of the rebar being pulled out (refer to Fig. 2(a) and (b)). The results of the comparison of the load-slip behavior of the two specimens for the bond length of 12d_b are shown in Fig. 6. The overall behavior of the two specimens was found to be similar in terms of the maximum load capacity and the softening response, as shown in Fig. 6(b). However, Fig. 6(b) shows that the specimen without the second rebar adjacent to the rebar being pulled out exhibited a higher initial stiffness as compared to the specimen with the second rebar installed in its vicinity. This can be attributed to the greater confinement of the former, where the minimum concrete cover was 7 times the rebar diameter in contrast with the latter where the minimum concrete cover was 5.6 times the rebar diameter.

For the sake of comparison between steel and Fe-SMA rebars, results from a few studies on the bond behavior of post-installed steel rebars are presented herein. Hamad et al. [7] evaluated the bond behavior of ø12 post-installed steel rebars with bond lengths of 12.5d_b, 21d_b, and 24d_b using a hybrid adhesive consisting of organic and inorganic binding agents. The results showed that the ultimate load capacity for 12.5d_b bond length was about half of the 21d_b bond length, whereas the load capacities for 21d_b and 24d_b were nearly identical. This indicates that the bond length might have attained a plateau after 20d_b bond length.

In another study [8], the steel rebars were post-installed into concrete cylinders with epoxy resin-based grouted adhesive to evaluate their bond behavior. The results showed that the pull-out load capacity for ø12 steel rebars kept on increasing as the bond length increased from 6d_b to 8d_b to 10.5d_b to 12.5d_b. A similar observation was reported earlier in [44], where the bond strength of post-installed steel rebars increased linearly with the increase in bond length. The bond lengths considered in the cited study were 6d_b, 9d_b, 14d_b, and 19d_b. The study used different types of mortars for installing the rebars and concluded that the differences in the mortar can lead to measurable differences in the bond strength.

The findings mentioned above about the effect of bond length on the pull-out load capacity of post-installed steel rebars seem to be in contrast with the behavior of Fe-SMA rebars investigated in the current study, as the pull-out load capacity of Fe-SMA rebars attained a plateau at a relatively short bond length of 8d_b. Besides other factors, one possible reason for this difference may be that the specimens in the past studies mainly failed in a splitting mode, whereas, in the present work, the specimens showed a pull-out failure. It should also be noted that the type of binding agent, which plays a significant role in determining the pull-out behavior, is different among these studies. It is recommended that...
future studies should compare the bond behavior of post-installed Fe-SMA and steel rebars for a given binding material to provide a better perspective in this regard.

3.1.2. Effect of activation

The effect of activation on the load-slip behavior of Fe-SMA rebar was studied by comparing the behavior of activated and non-activated rebars provided with the same bond length. The effect was studied for the bond lengths of $8d_b$ (using specimens 2 and 6) and $16d_b$ (using specimens 4 and 7). The load slip behavior of the specimens is shown in Fig. 7. The results indicate that the load-slip response of the activated and non-activated rebar is in general similar in terms of maximum load capacity up to the failure point; however, the plots show that non-activated specimens exhibit relatively more stiff load-slip response. As such, the maximum load capacity for non-activated specimens was attained at a relatively smaller slip compared to the activated specimens. This aspect can be more clearly seen for the bond length of $16d_b$. This essentially implies that the activation of rebar tends to reduce the initial stiffness of the load-slip curve at larger bond lengths as opposed to smaller bond lengths. It should be noted that this observation is based on the results of a limited number of specimens. More experiments should be performed in future to verify this observation. The reduction in the initial stiffness might result from the relatively lower elastic modulus of activated rebar (about 75 GPa) compared to the non-activated rebar (about 180 GPa) due to austenite to martensite transformation during the activation process [45–46]. Not much difference was observed in the maximum rebar strains and maximum load capacity developed by the activated and non-activated specimens. All the specimens developed rebar strains greater than 8 % at bond failure. Regarding activation, it should be noted that the elevated temperatures (greater than 270 °C) in the event of a fire during the service life can accelerate the prestress loss of concrete embedded activated Fe-SMA [47].

3.1.3. Effect of the drill-hole size

The effect of the drill-hole size and the associated grout cover on the pullout behavior of the post-installed Fe-SMA rebars was investigated by experimentally testing non-activated Fe-SMA rebars ($D = 16.5$ mm) grouted into drilled holes of 24 mm (specimen 8), 30 mm (specimen 9), and 36 mm (specimen 10) diameters, respectively. The bond length used was $12d_b$ in all cases. The load-slip behavior of the specimens shown in Fig. 8(a) indicates a negligible increase in the pullout load capacity with the increase in the hole size. It can be observed, however, that the post-peak response of the specimens with bigger holes ($D = 30$ mm and $36$ mm) is relatively less steep than the specimen with the smaller hole diameter i.e. $24$ mm. The maximum rebar strain at pullout failure was 10 % for specimen with $36$ mm hole diameter and 8 % for specimens with $24$ mm and $30$ mm hole diameter. Overall, it can be concluded that the size of the drill-hole does not seem to influence the pull-out behavior of the post-installed Fe-SMA rebars significantly.

3.1.4. Effect of the strength of confining concrete

To investigate the effect of concrete block, first a pullout test was conducted on specimen 11 that comprised 16.5 mm non-activated Fe-SMA rebar post-installed into a very low strength concrete block (i.e. compressive strength = 13 MPa). The rebar was grouted with a bond length of $16d_b$ into a 30 mm diameter hole size. The strength of the grout was 64 MPa on the test day. During the test, a premature splitting failure of the concrete block occurred and the specimen exhibited low pullout load capacity and significantly lower maximum rebar strain compared to specimen 7 which comprised a rebar post-installed into a concrete block with 53 MPa compressive strength, as shown in Fig. 9. This implies that a concrete cover of 5.5$d_b$ is not enough when a Fe-SMA rebar with 16.5 mm diameter is post-installed into a concrete block of very low strength.

The splitting failure of the concrete block occurred because the tensile stresses generated in the block owing to the bond shear stresses were higher than the tensile strength of the concrete block. It was assumed that the level of bond stresses were caused by the large diameter of the reinforcement bar, i.e. 16.5 mm. Subsequently, to reduce the bond stresses, pullout tests were conducted on small diameter (i.e. 10.5 mm) non-activated Fe-SMA rebars, post-installed in three concrete blocks with a compressive strength of 13 MPa (specimens 12, 13 and 16) and two blocks with a compressive strength of 23 MPa (specimen 14, 15). The rebars were grouted in 24 mm diameter drill-holes. Among these specimens, four were provided with a bond length of $8d_b$ where concrete and grout strength were the variables, and the last specimen was provided with a bond length of $12d_b$. The load-slip and rebar stress-strain behavior of the specimens are shown in Fig. 10. For these rebars, the failure mode observed in all the cases was a localized pullout at the interface of the rebar and grout. The results also indicate that for a given bond length (i.e. $8d_b$), the specimens with higher compressive strength of the concrete block (i.e. 23 MPa) show a relatively less steep
post-peak behavior, which can be attributed to the improved confinement. Similarly, higher grout strength (i.e. 77 MPa) for a given bonded length (i.e. 8d₀) resulted in slightly higher ultimate load capacity and a comparatively less steep post-peak behavior. The increase in bond length to 12d₀ increased both the ultimate load capacity and the maximum strain of the rebar before pullout failure. As such, rebars with 8d₀ bond length developed a maximum strain of about 5–6 %, as opposed to the maximum strain of 11 % in specimens with 12d₀ bond length.

It can be concluded from the results that for low strength confining concrete, the 10.5 mm diameter Fe-SMA rebars can effectively transfer their maximum prestress (above 300–350 MPa) for a short anchorage length of 8d₀. This is primarily because with the reduction in rebar diameter from 16.5 mm to 10.5 mm, the size of the minimum concrete cover provided to the rebars increased from 5.5d₀ to 11d₀. This highlights the important role of rebar diameter and concrete cover on the bond behavior and failure modes. Previous research [48] on bond strength has concluded that for a given bond length, large diameter rebars develop higher total bond forces compared to smaller bars for the same degree of confinement. This implies that to avoid splitting of low-strength concrete block for 16.5 mm rebars, a degree of confinement greater than that provided for 10.5 mm rebars may be required.

3.1.5. Load-slip behavior of plain vs ribbed vs threaded rebar

To study the effect of the surface properties of the reinforcement bars on the bond behavior, the pullout tests were conducted on plain (specimen 17), ribbed (specimen 7), and threaded (specimen 18) Fe-SMA rebars for a bond length of 264 mm. It is noted that ribbed and threaded rebars were made from the plain rebar of 18 mm diameter. The three rebar surfaces are shown in Fig. 11. The diameter of the ribbed rebar was 16.5 mm and the threaded rebar had a maximum and minimum diameter of 19.3 mm and 16.7 mm, respectively, in between the threads and along the threaded region. Since rebars with these surfaces were not
Fig. 8. Effect of the drill-hole size on the: a) load-slip behavior (specimens: 8–10); and b) rebar stress–strain behavior (specimens: 8–10).

Fig. 9. Effect of low-strength confining concrete on a) load-slip behavior (specimen 7 and 11); b) rebar stress–strain behavior of 16.5 mm Fe–SMA rebar (specimen 7 and 11); c) splitting of low strength concrete block (specimen 11).

Fig. 10. Effect of low-strength confining concrete on a) load-slip behavior (specimens: 12–16); b) rebar stress–strain behavior of 10.5 mm Fe–SMA rebar (specimen: 12–16).
available in the same diameter, therefore the available diameters of these rebars were considered mainly to study the difference in failure modes. It should be noted that the effect of rebar size on the bond behavior is twofold i.e longer bond lengths are required with the increase in rebar size and higher total bond forces are developed for larger rebars for the same confinement [49].

The test results showed that the plain and ribbed rebars failed in pull-out mode, whereas the threaded rebar experienced a fracture. Fig. 12(a) shows that the maximum load capacity by the three types of rebars was developed in the following order i.e. threaded rebar > ribbed rebar > plain rebar. The threaded rebar experienced a sudden fracture at a slip of 0.05 mm only. The fracture of the threaded rebar occurred due to the localized concentration of stresses within the unbonded section of the threaded reinforcement bar. The diameter of the threaded rebar was 16.7 mm in this region, therefore when the global stress in the rebar was about 615 MPa (as shown in Fig. 12(b)), the maximum localized stress developed in this region exceeded 800 MPa, which led to the fracture of the rebar. It should be noted that the ribbed rebar showed a smaller load capacity than the threaded rebar due to its relatively smaller diameter (refer Table 1). The plain rebar didn’t yield and developed only 13 % of the maximum load capacity of the threaded rebar, thus indicating a very low bond strength. This shows that more than 80 % of the bond strength is provided by the mechanical interlock between the ribs/threads and the grout, which is absent in the case of plain rebars. The maximum global strains developed in the plain, threaded and ribbed rebars were 0.05 %, 6.5 %, and 10 %, respectively.

The results of this investigation indicate that ribbed Fe-SMA rebars seem to be a more suitable choice for applications where high bond strength is required and large strains need to be sustained by the reinforcement. On the other hand, for cyclic loading applications (e.g. self-centering) where low bond strength is required in the plastic hinge region to delay the yielding of Fe-SMA rebar for preventing early loss of initial recovery stress, plain Fe-SMA rebars with threaded mechanical end anchorages may be used. In such a case, the threaded portion in the anchorage region will enable adequate anchorage to the base member while the plain region will allow for partial debonding from the cementitious material in the plastic hinge, thereby delaying the loss of the recovery stress. However, the length of the plain segment of the rebar needs to be carefully determined to avoid unwanted failure modes. It should be noted that for partially bonded rebars, a rocking mode of deflection with a major base crack at the footing-column interface is expected. The major base crack and the associated rocking mode of deflection occur owing to the slippage of the plain rebars at the footing-column interface. Such columns may also exhibit vertical splitting cracks owing to the low bond strength between rebars and concrete. It is recommended that the maximum stresses and strains in the threaded portion of the rebar in the anchorage region should remain less than 600 MPa and 5 %, respectively. Otherwise, a localized fracture of the rebar may occur in the anchorage region.

It is important to note here that the rib height and center-to-center
spacing between the ribs in the ribbed rebar were about 0.8 mm and 6 mm, respectively, whereas the center-to-center spacing of the threads in the threaded rebar was about 3 mm. In previous studies [49–50], it has been shown that the bond strength is a function of relative rib area (ratio of projected rib area normal to bar axis to the product of nominal bar perimeter and center-to-center rib spacing) and is independent of any specific combination of rib height and spacing. It has been reported that the initial stiffness of the load-slip curve and the bond strength increase with the increase in the relative rib area. As such, doubling the relative rib area can decrease the required anchorage length by 20 % [50]. The relative rib area of the Fe-SMA rebars used in this study is about 0.13, which is above the minimum requirement for steel rebars as specified by the BS 6744: 2001 [51].

3.2. Load-slip behavior under cyclic loading

3.2.1. Incremental cyclic loading tests

The specimens 19, 20, and 21 were subjected to displacement loading at increments of 10 mm and subsequent unloading to zero load after each displacement increment. The specimens 19 and 20 comprised non-activated ribbed Fe-SMA rebars with an embedment length of 16d, and specimen 21 comprised non-activated threaded rebar. Fig. 13 shows the cyclic load-slip behavior of specimen 19 along with the corresponding monotonic behavior of specimen 7 with the same bond length. The specimen experienced a pull-out failure and exhibited maximum load capacity similar to that of the monotonic test, as shown in Fig. 13(a). However, the maximum rebar strain at bond failure was 35 % less than the corresponding monotonic test i.e. bond failure occurred at a strain of 7.0 % as opposed to 10.5 % under monotonic loading. The results showed the accumulation of a permanent slip in the specimen with each loading and unloading cycle. A very small portion of the slip recovered during the few initial loading cycles. Before the final loading cycle that triggered the bond failure, a total permanent slip of about 1.2 mm had accumulated in the specimen. Reduced strength and stiffness in the initial stages of cyclic loading can be noticed in Fig. 13(b). This means that the bond slip under cyclic loading commences at relatively smaller loads compared to monotonic loading. Furthermore, the ultimate load is attained at a higher slip and the degradation in pull-out load also commences at a higher slip and is more steep compared to monotonic loading. While the degradation in load capacity under monotonic loading stabilized around 5 mm slip, the pull-out load capacity under cyclic loading degraded steadily. A repeat test was done on specimen 20, which exhibited similar load-slip behavior and a rebar strain of 7.8 % at bond failure which was about 25 % less than the rebar strain at bond failure under monotonic tests. This implies that for a given anchorage length, a less ductile bond behavior will be exhibited by Fe-SMA rebars under cyclic loading as compared to monotonic loading. It should be noted here that the ribbed Fe-SMA rebars used in cyclic tests were accidentally prestrained twice by the manufacturer and thus had an initial prestrain of about 8 %, whereas the Fe-SMA rebar tested under monotonic loading had an initial prestrain of 4 % only. Therefore, these results should be evaluated considering that the elastic modulus of an 8 % prestrained rebar is about 20–25 GPa higher than that of a 4 % prestrained rebar. In view of this, it would be reasonable to expect slightly higher strains at bond failure for a 4 % prestrained rebar under cyclic loading as it would reach the same load capacity as that of 8 % prestrained rebar at a relatively higher value of strain.

The load-slip behavior of threaded Fe-SMA rebar (specimen 21) with a bond length of about 14d, and an initial prestrain of 4 % is shown in Fig. 14(a) under incremental cyclic loading. The results show that the threaded Fe-SMA rebar experienced a sudden fracture after developing a maximum load capacity of 180 kN at a slip of about 0.05 mm. The fracture occurred (outside the bonded length) due to localized concentration of stresses in between the consecutive threads where the diameter of the rebar was relatively smaller i.e. 16.7 mm as opposed to the diameter of 19.3 mm along the threads. As a consequence, when the global maximum stress in the rebar was about 615 MPa (at 180 kN load), the localized stress in between the threads exceeded 800 MPa and led to the fracture of the rebar. The maximum global strain developed in the rebar was about 6 % before experiencing the fracture, as shown in Fig. 14(b).

The results indicate that ribbed and threaded rebars exhibit a similar failure mode under monotonic and cyclic loading. As such, the ribbed rebars experienced a pull-out failure under both loading histories, whereas threaded rebars fractured in both cases. However, the maximum strain attained in the ribbed rebar at bond failure reduced by 25–35 % under incremental cyclic loading in comparison with monotonic loading.

3.2.2. Constant cyclic loading tests

The specimens 22 and 23 consisting of ribbed Fe-SMA rebars with a bond length of 16d, were subjected to 100 load cycles of 8–80 kN, followed by displacement-controlled monotonic loading until bond failure. The specimens exhibited a pull-out failure mode. Fig. 15(a) shows the load-slip behavior of specimen 22. The results indicate that under constant load cycles, a higher slip of the rebar occurred during the first cycle, which subsequently became smaller with the increase in the number of cycles. As a result, the hysteretic area (energy dissipation)
between the load-slip curve decreased with the increase in the number of cycles. This behavior has also been reported for the basalt fibre-reinforced polymer rebars in [52] and was attributed to the presence of micro-voids between the rebar and concrete during the initial cycles of loading, which gradually close as the cyclic loading progresses. It can be noticed in Fig. 15(a) that a permanent slip of about 0.23 mm accumulated in the specimen at the end of 100 cycles of loading. The overall load-slip behavior of the specimen under constant load cycles and subsequent monotonic loading until failure is shown in Fig. 15(b). The results indicate that despite 100 constant load cycles, the specimen was able to develop a similar maximum pull-out load capacity as the specimen subjected to monotonic loading. This is because repetitions of load cycles with constant peak stress result in a gradual bond deterioration [12]. It should be noted, however, that bond failure occurred at 38 % smaller rebar strain (i.e. 6.5 % as opposed to 10.5 %) compared to the corresponding monotonic specimen. The test was repeated on specimen 23 which showed similar behavior and exhibited a maximum rebar strain of 7 % at bond failure, which implies a reduction of about 33 % in the maximum rebar strain compared to the monotonic test. These specimens like the previous ribbed rebar specimens subjected to incremental cyclic loading attained the maximum pull-out load capacity at a higher slip and exhibited a more steep degradation in pull-out strength compared to monotonic loading.

According to the ACI 408.2R-12 report [53] on the bond of steel reinforcing rebars under cyclic loading, cyclic stress ranges $\geq 0.4f_y$ can reduce the bond strength at failure by up to 50 % of the corresponding monotonic bond strength. The results of this study for Fe-SMA rebars are in contrast with these findings, as the maximum pull-out load capacity under monotonic and cyclic loading was identical. However, the reduction in the maximum rebar strain for Fe-SMA rebars was observed in the range of 25–40 % at bond failure. The report also indicated that reversed cyclic loading can accelerate the bond deterioration and cause failure even at fewer cycles. Hence, it is recommended to investigate the bond behavior of Fe-SMA rebars under reversed cyclic loading in future studies.

4. Comparison with bond Stress-Slip and anchorage length models of design standards

The design standards usually provide anchorage length recommendations for the cast-in-place rebars, which can be extended to the post-installed reinforcements. However, it has to be demonstrated experimentally that the used anchorage system can develop a comparable bond strength to the cast-in-place rebars. In this regard, the evaluation
requirements for the post-installed reinforcement system in Europe and the US are established by EOTA (European Assessment Document (EAD) 330,087 for static and fire loading conditions) [3–4] and AC 308 [5], respectively. If the anchorage system satisfies the evaluation requirements of these standards, then the anchorage length expressions of design codes can also be used for the post-installed rebars.

Fig. 16 compares the analytical bond stress-slip relationship for cast-in-place steel rebars according to the fib model code [54] with the relationship observed for the post-installed Fe-SMA rebars. The standard bond stress-slip curve shown in Fig. 16(a) comprises an ascending branch, a stabilized branch, and a descending branch, which is quite similar to the relationship observed for the post-installed Fe-SMA rebar with the 5\(d_b\) bond length. The bond stress-slip relationship for this specimen is plotted according to the recommendations of the fib Model code for the pre-yielding conditions and compared with experimental results in Fig. 16(b). The global comparison demonstrates that the post-installed Fe-SMA rebar exhibits a relatively stiffer ascending branch and slightly higher maximum bond strength compared to the fib model.

Table 2 provides a summary of the main features of the fib model and observed experimental results for this specimen. The comparison shows that the maximum bond shear stress and slips obtained in the experiments correlate to a greater extent with the fib model. However, a difference of about 40 % can be observed for bond shear stress at failure. The difference between the experimental results and the fib model may be because of the assumption that 5\(d_b\) represents an incremental bond element with a constant bond stress distribution. This simplification may have led to the overestimation of the experimental bond stress. It is expected that the experimental results may show a better correlation with the fib model curve if a comparison is made for a smaller bond length.

For the comparison of anchorage length, the design recommendations of Eurocode 2 [55] and ACI 318 [56] for the cast-in-place straight rebars (without hooks) under gravity loading and seismic loading are compared with the experimental results. The design anchorage length recommended by Eurocode 2 and ACI 318 for the gravity loading can be computed from expressions (1) and (2), respectively.

\[
l_b(\text{Eurocode}) = \frac{\alpha_1 \alpha_2 \alpha_3 \tau_{\text{fib}} \tau_{\text{fib}}}{4 \tau_{\text{fib}}{\tau_{\text{fib}}}^2} = \frac{\sigma_{\text{fib}}}{4 \tau_{\text{fib}}{\tau_{\text{fib}}}^2} \quad (1)
\]

\[
l_b(\text{ACI}) = \frac{(3.5 \psi)}{f_i} \frac{1}{1.3 \sqrt{f_i}} \frac{\gamma}{(d_1 + d_2)} \quad (2)
\]

The design anchorage lengths for 16.5 mm and 10.5 mm ribbed rebars determined according to the above-mentioned expressions are summarized in Table 3. For these calculations, the bonding characteristics such as grout compressive strength, concrete cover, and rebar yield strength were kept similar to the specimens of the experimental campaign. Table 3 illustrates that an anchorage length of \(\geq 15 d_b\) is recommended by the design standards to develop yield strength in the cast-in-place rebars with these characteristics. These recommendations seem to be quite conservative for the post-installed Fe-SMA rebars considered in this study as they developed yield strength even at a small anchorage length of 5\(d_b\). Thus, ACI 318 and Eurocode-2 design expressions for the cast-in-place rebars require more than 3 times the anchorage length than that observed in the current experimental campaign to develop the yield strength in the rebars. It should be noted that since the above-mentioned design expressions are mainly intended for cast-in-place rebars, therefore they do not take into account the difference in the strengths of grout and adjacent concrete, and hence for

![Fig. 16. Bond stress-slip relationships: a) fib analytical bond stress-slip relationship; b) Comparison of fib model predictions with experimental results for post-installed Fe-SMA rebar with 5\(d_b\) bond length (specimen 1).](image-url)
the calculation, only the strength of grout was considered while ignoring the strength of adjacent confining concrete.

For seismic loading, ACI 318 [56] recommends the following expression for the determination of anchorage length of straight rebars (with the depth of the concrete cast in one lift beneath the bar ≤ 300 mm) in the joints of special moment frames:

\[ l_{a(ACI)} = \frac{2.5f'c}{4f_p \sqrt{f_y}} \] (3)

Table 3 shows that this expression yields a recommended anchorage length about 1.6 times the length required for gravity loading. Similarly, Eurocode 8 [57] recommends increasing the anchorage length required for column reinforcement by 1.5 times if, under seismic actions, the axial force in the column is expected to be tensile. The experimental results of this study show that these requirements of increasing the anchorage length under cyclic loading are also applicable to Fe-SMA rebars as it was observed that the maximum strains developed in the ribbed Fe-SMA rebars at bond failure were 25–40 % smaller under cyclic loading compared to the corresponding strains developed under monotonic loading. It should be noted; however, that this reduction was observed for the unidirectional cyclic loading (i.e. loading with tension cycles only). A further reduction in the maximum rebar strains should be expected for the tension–compression reversals that are typically expected under earthquake actions.

As mentioned previously, the presented expressions of design standards are intended for the anchorage length of cast-in-place rebars in concrete. Hence, these expressions do not take into account the properties of the bonding agent and rely only on the compressive/tensile strength of concrete and confinement for estimating the anchorage length. Past studies have shown that the properties of the bonding materials can have a significant effect on the bond strength, stiffness, and anchorage length of post-installed rebars. In a study on the bond behavior of post-installed steel rebars by Speith et al. [2], it was shown that the epoxy system resulted in significantly higher strength and stiffness than the polyester system. On the other hand, the hybrid system (combination of vinylster and cementitious compounds), had a comparable bond strength to the epoxy system but a significantly lower bond stiffness. The study concluded that the bond stiffness is mainly influenced by the composition of mortar (mainly the amount of aggregates), whereas bond strength mainly depends on the shear strength of the mortar and its gluing ability with the wall of the drilled hole. Hofacker and Eligehausen [13] also reported that the hybrid bonding agent (based on styrene-free vinylster and cement) resulted in a higher bond strength than the unsaturated polyester-based bonding material. In view of this, it is recommended to study the effect of different bonding materials on the bond behavior of post-installed Fe-SMA rebars in future studies, as the results of the present study are only specific to the cement-based mortar i.e. SikaGrout-311 used in this work. Such a characterization will facilitate the use of post-installed prestressed Fe-SMA for incorporating a self-centering behavior in existing RC walls, columns and beam-column joints [58].

5. Conclusions

This study presented the results of an investigation into the anchorage behavior of post-installed Fe-SMA rebars for concrete retrofitting purposes. Based on the investigations, the following conclusions can be drawn:

1. The ribbed Fe-SMA rebars with 16.5 mm diameter successfully developed yield stress at a short bond length of 5d₀ when the compressive strengths of grout and confining concrete were about 70 MPa and 50 MPa, respectively. The increase in bond length generally resulted in an increase of the pull-out load capacity and rebar strain at bond failure; however, this increase reached a plateau at bond lengths beyond 8d₀. As a simplification, 8d₀ could therefore be regarded as the active bond length for the post-installed Fe-SMA rebars when concrete and grout compressive strengths are greater than 50 MPa and sufficient cover (greater than 5d₀) is provided.

2. The ribbed Fe-SMA rebars with 16.5 mm diameter exhibited a pull-out failure at bond lengths greater than 5d₀ when the strength of the confining concrete block was adequate (i.e. ≥ 50 MPa). However, for the similar concrete cover, a premature splitting failure of the confining concrete occurred when the strength of the confining concrete block was very low (i.e. ≤ 15 MPa). On the other hand, Fe-SMA rebars with 10.5 mm diameter exhibited a pull-out failure instead of the splitting of concrete block even when the strength of the confining concrete was very low. This is primarily because smaller tensile stresses were generated owing to the reduction in bond forces with the decrease in diameter of the rebar. This implies that prestressing with 10.5 mm Fe-SMA rebars is more suitable for retrofitting old concrete structures with poor-quality concrete, if available concrete cover is small.

3. Among the three Fe-SMA rebar types i.e. plain, ribbed, and threaded, the threaded rebar experienced a premature fracture at 6 % strain owing to the localized concentration of stresses because of the smaller diameter in-between the threads as compared to the diameter along the threaded region. On the other hand, the ribbed rebar experienced a pull-out failure and attained a maximum rebar strain of more than 10 % at bond failure. In contrast, the plain rebar didn’t yield and exhibited pull-out failure at a very small strain i.e. 0.05 %.

4. The maximum pull-out load capacity was found to be identical for monotonic and cyclic loading histories. However, the maximum pull-out load capacity developed at a higher slip under cyclic loads. The bond failure mode observed was also the same for monotonic and cyclic testing. In contrast, while the degradation in load capacity under monotonic loading stabilized around 5 mm slip, the pull-out load capacity under cyclic loading degraded steadily.

5. The maximum strain of ribbed rebars under incremental and constant cyclic loading was found to be about 25–35 % and 33–38 % lower at bond failure, respectively, compared to the corresponding strain under monotonic loading. This implies that bond deterioration is accelerated under cyclic loading, and thus for a given anchorage length, the Fe-SMA rebars may exhibit a less ductile bond behavior under cyclic loading compared to monotonic loading.

6. The bond-slip and anchorage length predictions of existing design standards for ribbed steel rebars were found to be generally conservative for ribbed Fe-SMA rebars.

This study demonstrates that for a concrete cover ≥ 5d₀, the ribbed post-installed Fe-SMA rebars bonded using SikaGrout-311 need relatively short bond lengths to develop the yield strength and exhibit a ductile bond behavior. This indicates the suitability of post-installed Fe-SMA rebars for many retrofitting applications. For instance, post-installed Fe-SMA rebars can be used to prestress existing bridge piers to incorporate a self-centering behavior under seismic actions. For this application, holes can be drilled into the footing and the pier cap beam for the anchorage of rebars using grout. Afterwards, activation of Fe-SMA can be triggered by heating the middle portion of the rebar (between footing and pier cap beam) using a gas flame. This exposed portion of the rebar can then later be embedded in a layer of shotcrete. The authors of the present work are currently evaluating the seismic performance of such retrofitted columns experimentally. A similar concept can be used for adding self-centering behavior to existing RC shear walls. Other potential applications of post-installed Fe-SMA rebars include the development of i) a prestressed end connection between newly cast beam slabs and existing walls ii) prestressed lap splice connection of the new slab to the existing slab, and iii) anchorage of the new column to the existing foundation. The prestressing of Fe-SMA in these applications can be achieved using electrical resistive heating. It is anticipated that for all these applications, a more rigid connection can
be established using Fe-SMA compared to conventional steel rebars owing to the prestressing characteristics of Fe-SMA. It should be noted that the present study was the first experimental study that, with a limited number of specimens, attempted to understand the bond behavior of Fe-SMA rebars post-installed into concrete. The results of this study can help in establishing the anchorage length required for the cement-based bonding agent considered in this study. More experiments should be conducted in the future to study the effect of various parameters not investigated in the present work. Some of the important aspects that should be considered in future studies include the effect of different bonding materials, the effect of dynamic loading on the bond behavior, and the bond behavior of activated Fe-SMA rebars under reversed cyclic loading for various bond lengths.

CRediT authorship contribution statement

Saim Raza: Conceptualization, Methodology, Investigation, Formal analysis, Writing – original draft. Julien Michels: Methodology, Resources, Writing – review & editing. Bernhard Schranz: Formal analysis, Writing – review & editing. Moslem Shahrvedi: Conceptualization, Methodology, Writing – review & editing, Supervision, Funding acquisition.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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References


