Bond Behaviour of Steel Bars Embedded in Low Strength Concrete

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Abstract. This paper aims at investigating bond behavior of steel bars embedded in low strength concrete compared to those embedded in normal strength concrete. The bond behavior of steel bars was studied using bar pull-out tests on 12 cubic specimens. The influence of bar diameter and concrete compressive strength on bond behaviour of embedded steel bars were investigated in this study. The load-slip behavior of all specimens was recorded during the test. The test result showed that the load-slip behavior of all specimens agreed well with the test report in literature. The splitting failure occurred in low strength concrete specimens when cracks in the surrounding concrete were caused by wedging action of the lugs of the steel bar. Failures occurred suddenly with the formation of the longitudinal cracks. For normal strength concrete specimens, the average peak axial load was significantly higher than those of low strength concrete specimens. The test results revealed that bond strength of the embedded steel bars was significantly dependent on compressive strength of concrete. It was noticed that when the strength of concrete is very low, the bond strength seems not to be proportional to square root of concrete strength (fc') as recommended by design provisions. The test results also showed that the size of steel bar has less influence on bond strength of steel bars compared to concrete compressive strength. In addition, the ACI code (ACI 308-08) was also used to calculate the bond strength of the test specimens. It was found that the experimental-to-design ratios were higher than 1.0 for all cases.

1. Introduction

The low strength concrete is prevalent in many old buildings and common in current detailing practice in regions of lower seismicity [1]. This type of concrete is believed to generally exist in many developing countries where the quality of design and construction material are very poor. The collapse of such low strength concrete under earthquake could be found in much country. For example, in 2008 Wenchuan earthquake, many buildings in Sichuan with poor quality concrete [2] were seriously damage and collapse under the earthquake. For example, in 2008 Wenchuan earthquake, many buildings in Sichuan with poor quality concrete [2] were seriously damage and collapse under the earthquake. After the Kobe Earthquake in 1995, seismic evaluations in Japan have found many existing RC buildings to have very low concrete strength (less than 13.5MPa) [3]. The poor quality concrete due to using very small size of poor quality aggregate was found as shown in figures 1(b). The problem may be aggravated when reinforcement detailing is also poor in beam and column. Under an earthquake these main structures may seriously damage as shown in figures 1(a).
Among many parameters influence reinforced concrete behaviour under an earthquake, bond behaviour of steel bars embedded in low-strength concrete is of most interest in this study. Therefore, the research presented herein is aimed at investigating bond behaviour of low strength concrete using a pull-off test compared to those embedded in normal strength concrete. The better understanding of bond behaviour of steel bar embedded in low strength concrete could be further used to improve the design and construction method to reduce the risk of bond failure in sub-standard concrete structure.

2. Experimental program

2.1. Test Specimen.

The experimental investigation was designed to study the influence of concrete strength and bar diameter on bond strength of embedded steel bars. A total of 12 pull-out specimens were casted using low strength concrete and normal strength concrete. The compressive strength of concrete in this study is 7.5 MPa and 26 MPa for low and normal strength concrete respectively. The deformed bars using in this study have diameter of 12 and 16 mm. Detailing of test specimens are shown in figure 2.

Table 1. Test-Matrix for pull-out specimens and properties of materials used.

| Mix Designation | Deformed bar |
|-----------------|--------------|
|                 | 12 | 16 |
| 7.5 MPa         | 3  | 3  |
| 26 MPa          | 3  | 3  |
| Total           | 6  | 6  |

From figure 2, the cross-section of the cube is 150 mm x 150 mm and the embedment length of the bar ($l_e$) is 140 mm. The steel rebar was embedded into the centre of concrete with 600 mm of the rebar length projecting out at one end and 10 mm of the rebar on the other end. This is to ascertain that the measurements for rebar slip at the other end could be correctly measured and the rebar can be tightly gripped for pull-out tests. Table 1 summarized the test matrix used in this study. All pull-out specimens were tested 28 days after casting.
2.2. Material properties and concrete mix proportions.
The materials used in this investigation include Type I cement, water, sand and aggregate for mix design of low strength concrete. In this study, dry ready mixed concrete was used for casting the normal strength concrete. Two different concrete mixes of 7.5 and 26 MPa compressive strength were designed and used. The concrete mix proportions and the basic properties of these mixes were determined following ASTM standards and presented in table 2.

Table 2. Summaries of mix properties for concrete.

| Mix type | Design strength(MPa) | Cement(kg/m$^3$) | Water(kg/m$^3$) | Sand(kg/m$^3$) | Aggregate(kg/m$^3$) |
|----------|----------------------|------------------|----------------|----------------|-------------------|
| 7.5 MPa  | 7.5                  | 160              | 180            | 1000           | 960               |
| 26MPa    | 26                   | 220              | 175            | 834            | 1152              |

Table 3. Properties of Steel rebars.

| Nominal Diameter (mm) | Elongation (%) | Yield Strength (MPa) | Ultimate Strength (MPa) |
|-----------------------|----------------|----------------------|-------------------------|
| DB12                  | 30             | 538                  | 683                     |
| DB16                  | 32             | 563                  | 654                     |

The reinforcement properties of the test rebars were shown in table 3. The yield and ultimate strength of rebar were obtained from typical tensile strength test. At least 5 rebars from the same batch of those used in pull-out test were selected in this tensile strength test. Only average values were presented here in the table.

In this pull-out test, the universal testing machine as shown in figure 3 and 4 was used under displacement-controlled procedure. The concrete cubes were rested against steel plate and allowing the rebar penetrating through the hold as shown in figure 4. The bottom grip was used to hold the tip of the steel bar tight and gradually pull it down. The pulling force and bar slip at the end of the cube were monitored throughout the test.

3. Results and discussion
The test results of all specimens were presented in table 4. The peak load together with calculated bond strength and slip at peak load including mode of failure were also shown in table 4. The equation below was used to calculate bond strength.

$$\tau = \frac{F}{\pi d_s l_e}$$  (1)
where:

\[ \tau = \text{experimental bond strength (MPa)} \]
\[ F = \text{ultimate axial tension force (kN)} \]
\[ d_b = \text{nominal rebar diameter (mm)} \]
\[ l_e = \text{embedment length (mm)} \]

According to various code provisions including ACI 318-08, the bond strength and the corresponding development length are related to the square root of the concrete compressive strength. This implies that the relationship between bond strength and the square root of compressive strength is linear. In this study the normalized bond strength was introduced in this study. The normalized bond strength was defined as the calculated experimental bond strength divided by \( \sqrt{F_c} \). The normalized bond strength \( \left( \tau_{nz} \right) \) can be expressed as given in equation (2):

\[ \tau_{nz} = \frac{\tau}{\sqrt{F_c}} \quad \text{(2)} \]

![Figure 4. Schematic for details of Test set-up.](image)

From the experimental results shown in Table 4, it can be observed that for the same embedment length, the bond strength of 7.5 MPa concrete ranged from 3.68 MPa to 3.87 MPa, whereas the bond strength for 26 MPa concrete ranged from 7.95 MPa to 9.88 MPa.

**Table 4.** Results of Pull-out tests for embedment length of 140 mm.

| Specimen & Mix Designation | Compressive Strength \( f'_c \) (MPa) | Nominal Bar Diameter (mm) | Peak Axial Load (kN) | Max. Bond Strength \( \tau \) (MPa) | Normalized Bond Strength \( \tau_{nz} \) (MPa) | Slip at Peak Load (mm) | Mode of Failure |
|---------------------------|--------------------------------------|--------------------------|---------------------|---------------------|---------------------|---------------------|-----------------|
| LS7.5-DB12                | 7.5                                  | 12                       | 20.43               | 3.87                | 1.41                | 1.07                | Splitting       |
| LS7.5-DB16                | 7.5                                  | 16                       | 25.85               | 3.68                | 1.34                | 0.66                | Splitting       |
| NS26-DB12                 | 26                                   | 12                       | 52.10               | 9.88                | 1.94                | 1.23                | Slipping        |
| NS26-DB16                 | 26                                   | 16                       | 55.89               | 7.95                | 1.56                | 0.48                | Splitting       |
From the crack pattern and damage observation, the bond strength was governed by a pull-out failure where the concrete between the steel ribs has been sheared off and the rebar slips in a frictional mode of failure. When the bar is embedded in low strength concrete, splitting through the entire concrete cover occurs. From failure mode in figure 5, it was observed that the specimens LS7.5-DB12, LS7.5-DB16 and NS26-DB16 were failed by splitting crack. Such cracks in the surrounding concrete are caused by the wedging action of the lugs of the steel bar. Failures occurred suddenly with the formation of longitudinal cracks. When cracking appears, the bond forces are directed outward from the bar surface and this causes anchorage failure resulting in splitting off of the confining concrete. However, only for the specimen NS26-DB12, the slipping failure of rebar occurs.

![Figure 5. Pull-out specimens after testing.](image)

The load-slip relationships for all the specimens tested in this investigation are shown in figure 6 (a-d). It was found that, for all low strength concrete specimens, the average peak pull-off loads were 20.83 kN and 25.85 kN for specimen LS7.5-DB12 and LS7.5-DB16 respectively. For normal strength concrete specimens, the average peak pull-off loads were 52.10 kN and 55.89 kN for specimens NS26-DB12 and NS26-DB16 respectively. The average peak pull-off loads of normal strength concrete specimens were significantly higher than those of low strength concrete specimens.
4. Comparison with code recommendations and predicting equation

The bond strengths measured in this test were compared with the design bond strengths recommended by ACI 318-08. The expressions used in calculating the development length $L_d$ are presented in Sections 12.2.2 in ACI 318-08 (equation 3) [6] for 19 bars and smaller:

$$L_d = \left( \frac{f_y \psi_y \psi_e}{1.4\lambda \sqrt{f_c}} \right) d_b$$  \hspace{1cm} (3)

Here, the bond strength $F_b$ has been derived from these expressions for the ultimate state conditions from White and MacGregor in 2008[7]

$$F_b = \frac{1}{4} \frac{f_y d_b}{l_d}$$  \hspace{1cm} (4)

where

- $f_y$ = the steel yield strength
- $\psi_y$ = reinforcement location factor (1.00)
- $\psi_e$ = coating factor (1.00)
- $\lambda$ = aggregate factor (1.00)
Equation 3 is intended for many practical construction cases which allows the designer to account for the actual values of the variables that control the development length. These expressions and their factors were determined in accordance with the conditions that pertain to the current tests.

The experimental-to-design bond strength ratios (EDR) was shown in figure 7 where the design strengths were calculated according to equation 4. The Figures show that the experimental-to-design ratios were higher than 1.0 for all cases when the bond strength was calculated using the ACI 12.2.2 equation of the ACI 308-08. It is also evident that the EDR ratios of low strength specimens is significantly lower than the EDR ratios of the normal strength specimens.

![Figure 7. Experiment - to - Design bond strength ratio](image)

5. Conclusions
This paper presents an experimental and numerical study on pull-out tests of steel bars embedded in low strength concrete (LSC) compared to those embedded in normal strength concrete. The research results are summarized as follows,

1. The load-slip behavior of all specimens was recorded and it was found that the load-slip behavior of all specimens agreed well with the test report in literature.

2. The splitting failure occurred in low strength concrete specimens when cracks in the surrounding concrete were caused by wedging action of the lugs of the steel bar. Failures occurred suddenly with the formation of the longitudinal cracks. For normal strength concrete specimens, the average peak axial load was significantly higher than those of low strength concrete specimens.

3. The test results revealed that bond strength of the embedded steel bars was significantly dependent on compressive strength of concrete. When the concrete strength is very low, the bond strength is also reduced. It was noticed that when the strength of concrete is very low, the bond strength seems not to be proportional to square root of concrete strength (fc') as recommended by design provisions.

4. The test results also showed that the size of steel bar has less influence on bond strength of steel bars compared to concrete compressive strength. In addition, the ACI code (ACI 308-08) was also used to calculate the bond strength of the test specimens. It was found that the experimental-to-design ratios were higher than 1.0 for all cases.

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