INVESTIGATION ON TENSILE AND SHEAR CAPACITY OF POST INSTALLED BONDED REBAR IN BRICK-AGGREGATE CONCRETE

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Abstract

Post-installed bonded anchor is often used for retrofitting of structurally unsafe RC buildings. However, due to the lack of data on behavior of full scale post-installed bonded rebar in concrete with broken bricks as aggregates, it is difficult to use them for retrofit purpose. The necessity of retrofitting of RC buildings in countries (e.g. Bangladesh, Nepal etc.) where brick aggregates are more popular than stone aggregates cannot be ignored as some of these countries are at earthquake prone areas. Moreover, the damage and loss of lives experienced by the structural design errors of existing RC buildings in such countries (e.g. Rana Plaza collapse in Bangladesh) show that necessary step must be taken to identify and retrofit existing unsafe RC buildings made of concrete with brick aggregates. Therefore, in this experimental study a series of pullout tests and shear tests have been conducted to understand the behavior of full scale post installed bonded rebar in pure concrete with brick aggregates. The effects of change in concrete strength (varying from10MPa to 33MPa), rebar diameter (10mm, 16mm and 20mm) and effective embedment length ($7d_b$, $10d_b$ and $13d_b$) were also investigated. The test results showed that a minimum embedment length of $10d_b$ is required to avoid brittle failure of concrete of different strengths and rebar sizes. In this study, rebars were designed by following the Japanese guidelines but the study also investigated the applicability of equations provided by ACI 318-14 code, ACI 318R-14 commentary and Japanese guidelines to estimate the tensile capacity of bonded rebar in low strength concrete. The test results showed that the Japanese standard equations evaluate the tensile capacity of the post installed bonded rebars conservatively, but equations provided by ACI 318-14 code and ACI 318R-14 commentary were found to be even more conservative than Japanese standard equations in this case.

Keywords: bonded rebar, pure concrete, brick aggregates, tensile & shear capacity, seismic retrofit

1. INTRODUCTION

Due to the geological position several countries like Bangladesh, Nepal etc. have the potentiality to have earthquakes any time as shown in Fig. 1[1]. It can be seen from the Fig. 1 that several fault lines run through the country (Bangladesh) which are the potential sources of earthquakes. Even though there is no record of big earthquakes in Bangladesh, it was affected by the recent earthquake at the neighboring country Nepal. The Fig. 2 shows tilting of RC building in Dhaka city (the capital of Bangladesh) due to the Nepal earthquake 2015.

The existing RC buildings in Bangladesh are structurally of poor strength. There are records of collapse of RC buildings without any occurrence of major earthquakes in Bangladesh. For example the tragic Rana plaza collapse (2013) occurred in Bangladesh without any earthquake and Fig. 3 shows the glimpse of collapsed Rana plaza.

A vital reason of why such buildings in Bangladesh are structurally not strong enough is that, most of these existing RC buildings are constructed with concrete having broken bricks in it as coarse aggregate. There are a number of reasons to use broken bricks instead of stone aggregate in concrete in Bangladesh.

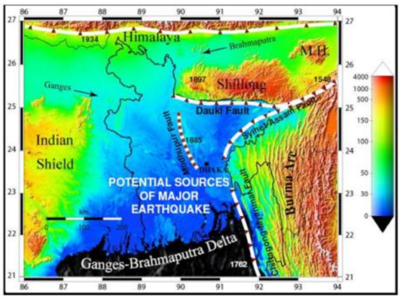


Fig. 1 – Major faults exist in Bangladesh [1]

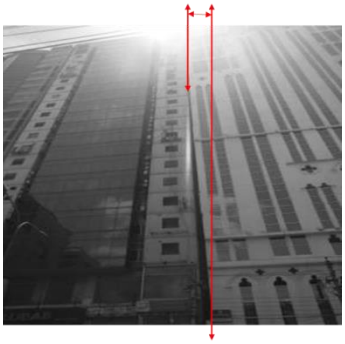


Fig. 2 – Tilting of building at Dhaka due to Nepal earthquake of 2015 [2]

One of the most important reasons of it is that Bangladesh is a riverine country and for this the natural stone sources are limited resulting stone aggregate expensive. However, the bricks are cheaper and produced abundantly in Bangladesh and that is why broken brick aggregate is more popular than stone aggregate for construction works. Some other countries like Nepal, India, Iran also uses bricks for their construction works resulting poor strength of RC buildings.

Fig. 3 – Rana plaza collapse [3]

Bangladesh is ill prepared to tackle the aftermath of any strong earthquake and if a massive earthquake with 7 or greater magnitude occurres in this country will lead a major human tragedy due to the faulty structures of many buildings and inproper awareness [1]. Moreover, the damage and loss of lives experienced by the structural design errors of existing RC buildings in countries (e.g. Rana Plaza collapse in Bangladesh) show that necessary steps must be taken to identify and retrofit such existing unsafe RC buildings made of concrete with broken brick aggregates.

For the seismic potentiality and existing unsafe RC buildings the government of Bangladesh is now planning of retrofitting those buildings. There are a number of retrofitting techniques around the world. Some retrofitting techniques use post-installed bonded anchor to connect the new structural members with the existing buildings. It’s an unfortunate that due to the lack of data on behavior of full scale post-installed bonded rebar in concrete with brick aggregates, it is difficult to use them for retrofit purpose of those buildings. However, the necessity of retrofitting of RC buildings in countries like Bangladesh or Nepal cannot be ignored as ignorance might lead a massive human tragedy.

Therefore, in this experimental study a series of pullout tests and shear tests have been conducted to understand the behavior of full scale post installed bonded rebar in pure concrete with brick aggregate. The effects of change in concrete strength, rebar diameter and effective embedment length were also investigated. One of the main objectives of this test was to find out the required embedment length of the bonded rebar to avoid the brittle type failure of concrete with different strengths and rebar sizes. In this study, rebars were designed by following the Japanese guidelines. However, the study also investigated the applicability of equations provided by ACI 318-14 code, ACI 318R-14 commentary and Japanese guidelines to estimate the tensile capacity of bonded rebar in low strength concrete. The experimental results have been compared with the design calculations provided by the Japanese guidelines and ACI code. This comparison will be helpful for the selection of the most convenient design method while designing the bonded anchors for retrofitting work in Bangladesh in future.

2. DESIGN OF POST-INSTALLED BONDED REBAR FOR TENSION AND SHEAR TEST

The post-installed bonded specimens were designed for the tensile test (pull out test) and the shear test by using the Japanese guidelines. The tensile capacity of the specimens were also determined by the equations provided by ACI 318-14 code and ACI 318-14 commentary. These design results have been compared with the experimental results to find out the applicability as well as appropriateness of these guidelines for concrete with brick aggregates and low strength as well.

2.1 Design of tensile capacity of post installed single bonded rebar

2.1.1 Design of tensile capacity of post installed single bonded rebar by Japanese guidelines

The anchor specimens were designed based on Japanese guidelines [4]. The tensile capacity of single anchor, T_a is determined by three basic failure modes, as shown in Fig. 4. T_a shall be the smallest value of T_{a1} which is determined by steel strength, T_{a2} which is determined by concrete cone failure, and T_{a3} which is determined by bond strength. T_{a1}, T_{a2} and T_{a3} can be evaluated by the following equations:

$$T_a = \min(T_{a1}, T_{a2}, T_{a3}) \quad \text{(Eq. 1)}$$

$$T_{a1} = \sigma_y \, . \, a_o \quad \text{(Eq. 2)}$$

$$T_{a2} = 0.23\sqrt{\sigma_B} \, . \, A_c \quad \text{(Eq. 3)}$$

$$T_{a3} = \tau_a \, . \, \pi \, . \, d_a \, . \, l_e \quad \text{(Eq. 4)}$$

$$\tau_a = 10\sqrt{\frac{\sigma_B}{21}} \quad \text{(Eq. 5)}$$

where,

σ_B = Compressive strength of concrete (N/mm²).
σ_B = Yield stress of steel (N/mm²).
a_o = Nominal cross-sectional area of anchorage bar (mm²).
A_c = Projected area of concrete cone failure (mm²).
= $\pi \, . \, l_e \, . \, (l_e + d_a)$ assuming 45° cone failure surface to the horizontal/vertical (shown in Fig. 5).
d_a = Anchor diameter (mm).
l_e = Effective embedment length of anchor.
τ_a = Bond strength of bonded anchor against pullout force.

Seven types of anchor specimens were designed using Eq. 1- Eq. 5 provided by the Japanese guidelines for tensile test of single bonded anchor. The parameters of the specimens were anchor diameter, concrete strength and embedment length, as shown in Table 1. Concrete of 10MPa, 20MPa and 30MPa strength were selected for this experiment. However, in real the values varied a bit as shown Table 1.

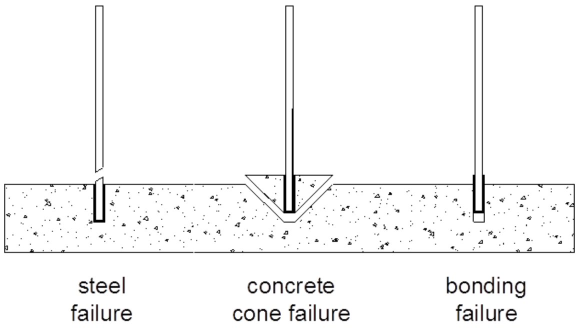


Fig. 4 – Basic failure modes of post-installed bonded single anchors

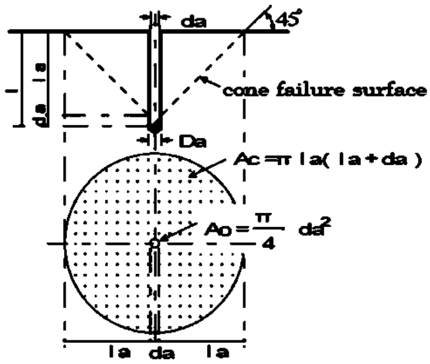


Fig. 5 – Effective projected failure area of single bonded anchor[5].

Deformed rebar of diameter 10mm, 16mm and 20mm have been used for this experiment. The embedment length also satisfied the Japanese guidelines and the minimum effective embedment length was 7 times the rebar diameter. Additionally just to understand the effect of the embedment length 10 times the rebar diameter and 13 times the rebar diameter were also chosen for the selection of effective embedment length.

Table 1– Test parameters of rebar specimens

<table>
<tr><th>Specimen Type</th><th>Concrete Strength [Design] (MPa)</th><th>Concrete Strength [Actual] (MPa)</th><th>Diameter d_a (mm)</th><th>Effective Embedment Length (mm)</th><th>Specimen Size (mm^3)</th></tr>
<tr><td>P1</td><td>10</td><td>12.2</td><td rowspan="2">10</td><td rowspan="5">$7d_a$</td><td rowspan="3">400×400×200</td></tr>
<tr><td>P2</td><td>20</td><td>24</td></tr>
<tr><td>P3</td><td>30</td><td>33.23</td><td>10</td></tr>
<tr><td>P4</td><td rowspan="4">10</td><td>13</td><td>16</td><td rowspan="4">500×500×200</td></tr>
<tr><td>P5</td><td>10.65</td><td>20</td></tr>
<tr><td>P6</td><td>11.56</td><td rowspan="2">10</td><td>$10d_a$</td></tr>
<tr><td>P7</td><td>11.07</td><td>$13d_a$</td></tr>
</table>

The selection of the size of the concrete block was determined in a way so that the concrete cone failure can occur smoothly. The depth was considered larger than 1.5 times the effective embedment length as per 6.4.1 section of ASTM E-488. In that section of ASTM guidelines it is mentioned that the depth of the structural member shall be equal to the minimum member depth specified by the manufacturer [6]. The structural member shall be at least 1.5 l_e in thickness so long as the depth is suitable for normal installation of the anchor and does not result in premature failure of either the structural member or anchor, unless the specific test application requires a lesser thickness [6]. A structural member with a thickness of at least 1.5 l_e will minimize bending during the application of the tensile load to the test anchor [6]. For a more accurate understanding of the effect of concrete strength the actual strength of the concrete has been used in calculating the design tensile strength of the bonded rebar. Table 2 shows the design tensile capacity and the expected failure modes of the specimen by Japanese guidelines.

2.1.2 Design of tensile capacity of post installed single rebar (anchor) by American Concrete Institute (ACI) guidelines ACI 318-14 code and ACI 318R-14 commentary

The specimens were also designed using the equations provided in ACI 318R-14[7]. According to the ACI

318R-14[7] the nominal strength of an anchor in tension, N_{sa} shall not exceed

$$N_{sa} = A_{se,N} f_{uta} \quad \text{(Eq. 6)}$$

where,

N_{sa} = The nominal strength of an anchor in tension in lb.

$A_{se,N}$ = The effective cross-sectional area of an anchor in tension, in^2.

f_{uta} = Ultimate tensile capacity of anchor rebar in psi, and f_{uta} shall not be taken greater than the smaller of 1.9 f_{ya} and 125000psi.

The nominal concrete breakout strength in tension, N_{cb} of a single anchor shall not exceed,

$$N_{cb} = (A_{Nc}/A_{Nco})\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b \quad \text{(Eq. 7)}$$

where,

N_{cb} = The nominal concrete breakout strength in tension of a single anchor in lb

A_{Nc} = The projected concrete failure area of a single anchor that shall be approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward 1.5h_{ef} from the centerline of the anchor, in inch^2. A_{Nc} shall not exceed nA_{Nco} where n is the number of anchors in the group that resist tension.

$A_{Nco} = 9h_{ef}^2$ where, h_{ef} = Effective embedment length in inch.

$\psi_{ed,N}$ = The modification factor for edge effects for single anchor loaded in tension.

$\psi_{c,N}$ = 1.4 for post-installed anchors

$\psi_{cp,N}$ = The modification factor for post installed anchors designed for uncracked concrete without supplementary reinforcement to control splitting.

N_b = The basic concrete breakout strength of a single anchor in tension.

The nominal bond strength of single adhesive anchor is,

$$N_a = (A_{Na}/A_{Nao})\, \psi_{ed,Na}\, \psi_{cp,Na}\, N_{ba} \quad \text{(Eq. 8)}$$

where,

A_{Na} = The projected concrete failure area of a single anchor, in inch^2.

$A_{Nao} = (2c_{Na})^2$ $\left[\, c_{Na} = 10d_a\sqrt{\frac{\mathcal{T}_{uncr}}{1100}} \,\right]$

$\mathcal{T}_{uncr}$ = Bond stress.

$\psi_{ed,Na}$ = The modification factor for edge effects for single adhesive anchor loaded in tension.

$\psi_{cp,Na}$ = The modification factor for adhesive anchors designed for uncracked concrete without supplementary reinforcement to control splitting.

N_{ba} = The basic bond strength of a single adhesive anchor in tension in cracked concrete.

The tensile capacity of the anchor rebar has been calculated following the Eq. 6 – Eq.8 which are provided in ACI 318-14 code and ACI318-14 commentary.

The summary of the calculation is shown below in Table 2.

Table 2 – Expected tensile capacity and failure modes by Japanese guidelines and ACI 318R-14 code

Specimen type	Japanese guidelines		ACI 318R-14 code	
	Expected tensile capacity(kN)	Expected failure mode	Expected tensile capacity(kN)	Expected failure mode
P1	14.13	Concrete cone	15.16	Bond
P2	19.82	Concrete cone	15.16	Bond
P3	23.33	Concrete cone	15.16	Bond
P4	37.34	Concrete cone	38.82	Bond
P5	52.82	Concrete cone	48.07	Concrete cone
P6	23.31	Bond	21.66	Bond
P7	29.81	Bond	28.16	Bond

2.2 Design of shear capacity of post installed single bonded rebar by Japanese guidelines

The shear capacity Q_a is defined as the capacity resisted by a single anchor at the concrete interface. Shear capacity shall be the smaller value of Q_{a1} and Q_{a2}, which are determined by steel strength and bearing strength of concrete, respectively [4]. According to the Japanese standards [4], for bonded anchor in case of $l_e \geq 7d_a$,

$$Qa_1 = 0.7.\sigma_y.a_e \qquad \text{(Eq. 9)}$$

$$Q_{a2} = 0.4\sqrt{E_c\sigma_B}.\ {}_sa_e \qquad \text{(Eq. 10)}$$

However, bond stress, $\mathcal{T} = \frac{Q_a}{{}_sa_e} \leq 294$ MPa

where, σ_y= Compressive strength of existing concrete. σ_B shall be determined according to the standard.

E_c = Young's modulus calculated based on σ_B.

The design parameter for the shear test was the varying concrete strength. The specimens for shear test were designed by using Eq. 9-Eq. 10 and the summary of the shear capacity and the expected failure mode predicted by Japanese standards is shown in Table 3.

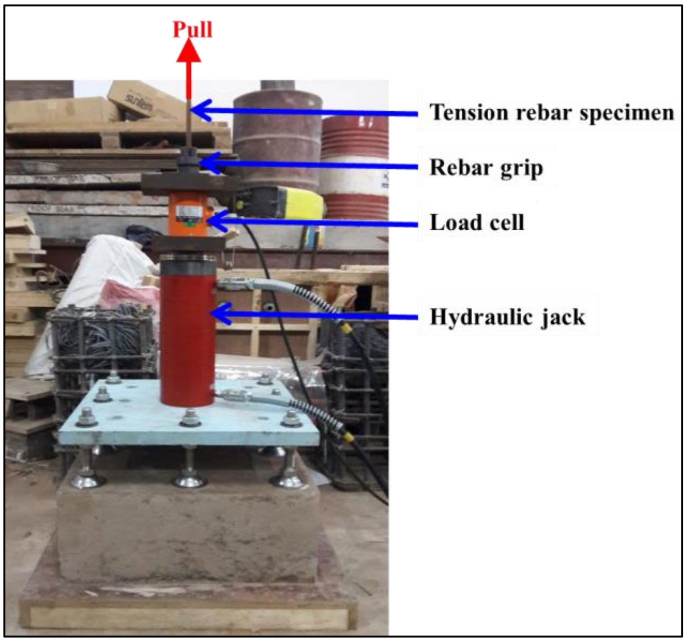
Table 3 – Expected shear capacity and failure modes of shear specimen

Specimen type	Diameter of rebar (mm)	Concrete strength (MPa)	Qa_1 Steel strength based (kN)	Qa_2 Concrete strength based (kN)	Expected shear capacity (kN)	Expected Failure mode
S1	10	12.96	30.79	16.03	16.03	Concrete bearing
S2	10	25.25	30.79	25.01	23.10	Concrete bearing
S3	10	33.9	30.79	30.44	23.10	Concrete bearing
S4	10	10.83	30.79	14.23	14.23	Concrete bearing

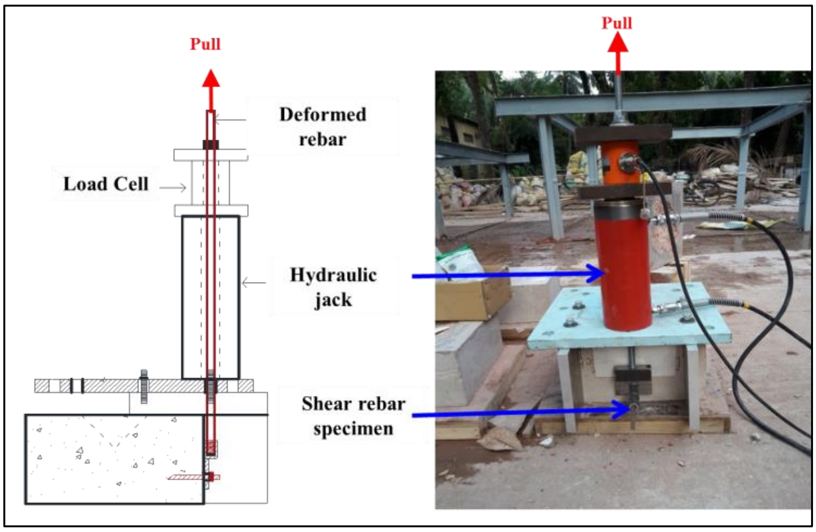
3. EXPERIMENTAL SET- UP, LOADING AND MATERIAL PROPERTIES

The rebars were embedded in blocks made with pure concrete using brick chips as coarse aggregate. The concrete mix for each specimen was 1:2:4 and for 10MPa concrete the W/(C+vita sand) ratio was 0.6. For 20MPa and 30MPa w/c ratio was 0.8 and 0.5. The yield stress of the rebar used in this experiment varied from 328 MPa to 409 MPa whereas the ultimate stress varied from 475MPa to 560MPa. Materials used for pull out test and shear test were the same.

The experimental setup for the tension test was as shown in Fig. 6 (a) and the test set up for shear test is shown in Fig. 6 (b). The tensile load was applied by the use of hydraulic jack of 300kN capacity and the reading was recorded from load cell. For tension test the rebar was embedded on the top of the concrete block and for the shear test the rebar was embedded on the side of the concrete block. Pull force was applied on vertical direction resulting tensile force for the pull out test specimen and shear force for the shear test specimen.



(a)



(b)

Fig. 6 – (a) Test set-up of the tension (pull out) test, (b) Side and front view of the shear test set-up

4. EXPERIMENTAL RESULTS AND DISCUSSIONS

4.1 Experimental results from tension (pull out) test

For the tensile test three specimens were prepared for each type of specimen. All specimens were loaded as shown in Fig. 6 (a). From the test it was observed that along with the concrete cone and bond failure modes, splitting of specimen also occurred (Fig. 7). Moreover, some combined modes (i.e. concrete cone + splitting) also happened (Fig. 8). However, most of the specimen failed in a way that was expected by the Japanese guidelines.

(a) (b) (c) (d)

Fig. 7 – Experimental failure modes observed from tension test (a) concrete cone, (b) bond, (c) splitting, and (d) concrete cone + splitting

The experimental test results along with calculated design data by Japanese guidelines are shown in Table 4.

Table 4 – Experimental results and calculated design data on tension test of post-installed bonded anchors

Specimen type		Tensile capacity (kN)	Average tensile capacity (kN)	Failure mode	T_{a2_cal} (kN)	T_{a2_exp} (kN)	T_{a2_exp} / T_{a2_cal}	T_{a3_cal} (kN)	T_{a3_exp} (kN)	T_{a3_exp} / T_{a3_cal}
P1	1	20.46	21.32	Cone	14.13	20.46	1.44			
	2	22.50		Cone		22.50	1.59			
	3	21.00		Cone		21.00	1.49			
P2	1	30.90	28.77	Splitting	19.82	30.90	1.55			
	2	28.40		Cone		28.40	1.43			
	3	27.00		Cone+ Splitting		27.00	1.36			
P3	1	32.80	30.95	Cone+ Splitting	23.33	32.80	1.41			
	2	29.10		Cone+ Splitting		29.10	1.25			
P4	1	40.00	43.3	Splitting	37.34	40.00	1.10			
	2	42.70		Splitting		42.70	1.13			
	3	47.20		Cone+ Splitting		47.20	1.26			
P5	1	68.80	68.80	Splitting	52.82	68.80	1.30			
P6	1	27.60	30.33	Bond				23.31	27.60	1.18
	2	32.60		Bond					32.60	1.39
	3	30.80		Bond					30.80	1.32
P7	1	35.40	33.43	Bond				29.65	35.40	1.19
	2	29.50		Bond					29.50	0.99
	3	35.40		Bond					35.40	1.19

4.1.1 Behavior of concrete cone failure

From the Table 4 it can be observed that the specimens with concrete cone failure modes which were from specimen type P1 and one specimen from specimen type P2 showed that the tensile capacity determined by concrete cone failure was much higher than the design calculations. Table 4 compares the calculated tensile capacity determined by concrete cone failure (T_{a2_cal} by Japanese guidelines) with the experimental results (T_{a2_exp}). The experimental tensile capacity was more than 43% higher than the design calculations. Those with the combined type failure of split and cone failure showed more than 25% higher tensile capacity than the deign calculations. For this reason, the concrete cone failure areas observed in the experiment also exceeded the design assumption in Japanese guidelines, as shown in Fig. 8.

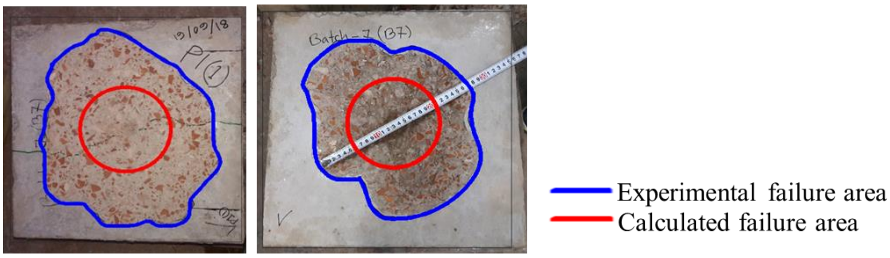


Fig. 8 – Sample photos of concrete cone failure area

One very important point can be observed that when the embedment length was above or equal to 10 times the rebar diameter (Specimen type P6 and P7) no concrete cone or splitting failure occurred. This means no brittle failure was observed when the embedment length was equal or above 10times the bar diameter.

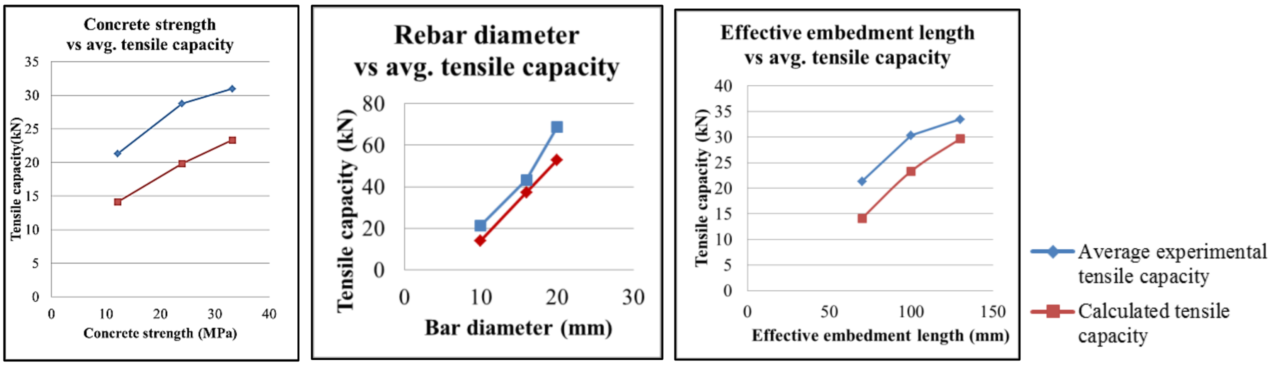
4.1.2 Behavior of bond failure

Table 4 compares the calculated tensile capacity determined by bond failure (T_{a3_cal}) with the experimental results (T_{a3_exp}). The experimental tensile capacity was more than 18% higher than the design calculations. However, for just one specimen of type P7 the experimental tensile capacity was 0.15kN lower than the design calculation, however, which was negligible. In this experiment two categories of bond failure occurred as shown in Fig 9. These categories are bond failure of adhesive/concrete interface and bond failure of steel/adhesive interface. In the case of bond failure of adhesive/concrete interface the experimental tensile capacity was more than 18% higher and for the specimens having bond failure of steel/adhesive interface the tensile capacity was 19% higher than that of the design tensile capacity for most cases.

Fig. 9 – Bond failure of steel/adhesive interface and adhesive/concrete interface (from left)

4.1.3 Effects of concrete strength, rebar diameter and embedment length on tensile capacity

From the experiment the effects of the change in concrete strength (specimen type P1, P2 and P3) are shown in Fig. 10 (a). The effects of change in rebar diameter (specimen type P1, P4 and P5) and embedment length (specimen type P1, P6 and P7) are shown in Fig. 10 (b) and in Fig. 10 (c) respectively. From the graphs of Fig. 10 it can be concluded that the tensile capacity increases with the increase in concrete strength, rebar diameter and embedment length.



(a) (b) (c)

Fig. 10 – Effects of concrete strength, rebar diameter and mbedment length on tensile capacity of post installed single bonded anchors

4.1.4 Comparison of calculation by Japanese guidelines and ACI code with experimental results

From the experimental results it can be seen that the tensile capacity calculated by the Japanese guidelines is much lower than the experimental results. This implies the Japanese guidelines evaluated the tensile capacity of post installed single bonded anchor conservatively.

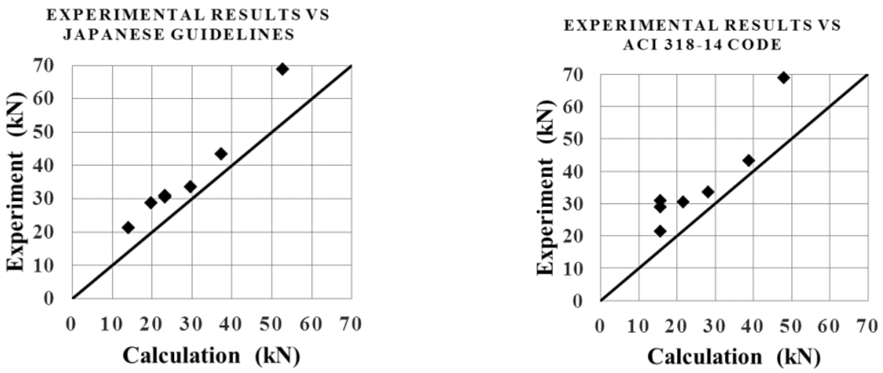


Fig. 11 – Experimental results versus design data by Japanese guidelines and ACI 318-14 code and ACI 318R-14 commentary.

From the left graph of Fig. 11 it can be found that, the mean (Ta_{exp} / Ta_{cal})=1.31 and the coefficient of variation (CoV) =0.10. However, from the right graph the mean (Ta_{exp} / Ta_{cal})=1.48 and the coefficient of variation (CoV) =0.21. Therefore, the graphs shown in Fig. 11 indicate that the ACI code evaluated the tensile capacity of post installed bonded single anchor more conservatively than the Japanese standards.

4.2 Experimental results from shear test

The shear capacity and the failure modes of the shear test specimen are shown in Table 5. From the table it can be seen that the experimental shear capacity was 10% more than the design shear capacity provided by the Japanese guidelines for most cases. Only for S2 type specimen the experimental value was 6% lower than the calculated value by Japanese guidelines. The conservative evaluations for the specimen types of S1 and S4 mean that the Japanese guidelines for shear test can be adopted to design members with low strength concrete.

Table 4 – Experimental results and design data of shear test of post-installed single bonded anchors

Specimen type		Expected shear capacity $Qa_{\_cal}$ (kN)	Experimental Shear capacity $Qa_{\_exp}$ (kN)	$Qa_{\_exp}$ / $Qa_{\_cal}$ (kN)	Experimental Failure mode
S1	1	16.03	21.70	1.35	Concrete bearing
	2		20.20	1.26	Concrete bearing
	3		17.80	1.11	Concrete bearing
S2	1	23.10	21.60	0.94	Concrete bearing
	2		21.60	0.94	Concrete bearing
S3	1	23.10	28.20	1.22	Specimen split
S4	1	14.23	22.50	1.58	Concrete bearing
	2		24.60	1.73	Concrete bearing

The shear specimen failure modes matched exactly with the one expected by the Japanese guidelines and showed concrete bearing failure (Fig. 12). However, only one specimen (S3) showed splitting and the possible reasons behind this were that the concrete block might be damaged during the pull out test performed prior to the shear test and that the block was made of pure concrete with no reinforcement in it. The shear strength increased with the increase of concrete strength for most cases.

Fig. 12 – Failure modes from shear test of post-installed bonded anchors (S1, S2, S4 from left)

5. CONCLUSIONS

The main purpose of this experimental study was to understand the behavior of post installed bonded anchors in concrete with brick aggregates so that the output from this study can be effective to retrofit buildings made with brick aggregate concrete. The following conclusions were achieved from this experimental study of tensile and shear capacity test of single anchors.

I. The existing Japanese standard equations mostly evaluated the tensile capacity and the shear capacity of post installed bonded anchor rebar conservatively while the ACI 318-14 code and commentary ACI 318R-14 calculated the design tensile capacity more conservatively. The Japanese guidelines also showed promising calculation for predicting the possible failure patterns for low strength concrete specimens.

II. The embedment length of the rebar should be taken equal or greater than 10 times the diameter of the rebar to avoid brittle failure or concrete cone failure. This recommendation can be applied for retrofitting design.

III. The tensile capacity of the post installed bonded single anchors increased with the increase in concrete strength, rebar diameter and embedment length.

IV. The experimental shear capacity was higher than the calculated shear capacity for most cases. The shear capacity of the post installed anchor increases with the increase in concrete strength for most cases.

6. ACKNOWLEDGEMENTS

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7. COPYRIGHTS

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