Experimental Investigation of Bond Characteristics of Deformed and Plain Bars in Low Strength Concrete

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Abstract

The use of inferior quality materials, inadequate detailing and poor construction practices are responsible for most of the brittle failure modes of non-engineered reinforced concrete structures. Bond failures in non-engineered reinforced concrete elements due to short anchorages or low concrete cover result in large slip deformations prevent the development of plastic deformations and reduce energy dissipation capacity. Until now, little work has been carried out that can lead to the development of bond-slip relationships for low strength non-engineered reinforced concrete structures. To address this, experiments have been carried out on pullout and splitting specimens under monotonic loading to investigate bond characteristics of typically used steel bars in non-engineered reinforced concrete structures. Various deficient parameters are considered in the experiments in order to develop multi-parameter bond strength relations for low strength concrete ≤15MPa. The key parameters examined in the experiments are low strength concrete, bar development length, concrete cover, re-bar types (deformed and plain) and re-bar diameter. This paper presents the experimental details and results which are further processed to develop bond strength equations for different bar types in low strength concrete. These equations can be used to define the bond-slip relation for conducting seismic vulnerability assessment of non-engineered structures.

Keywords**:** Low strength concrete, normal strength concrete, non-engineered reinforced concrete, Reinforced Concrete, bond-slip, deformed bar, cold-formed bar, plain bar

1 Introduction

Non-engineered reinforced concrete (NERC) buildings in developing countries are known to be highly vulnerable to seismic motion, Naseer et al. [1]. Post earthquake damage surveys from developing countries (Naseer et al. [1], Duranni et al. [2], Nisikawa et al. [3], Naseer et al. [1], Peiris et al.[4], Bal et al.[5]) attribute the poor performance of reinforced concrete (RC) structures to the use of poor materials, bad design, detailing, and inappropriate construction practices [1-5]. Most of the collapsed

RC structures in the Kashmir earthquake (2005), Pakistan had an average concrete compressive strength ($f_c^{'}$) of around 15MPa. Bal et al. [5] tested cores taken from 1178 existing RC buildings, located in Istanbul and its surroundings, and reported a mean $f_c^{'}$ of 17MPa. It is widely accepted that Low Strength Concrete (LSC) is one of the main reasons for many brittle failures in NERC. Even in developed countries such as Japan, post-earthquake studies after the Kobe Earthquake in 1995, have reported many existing RC buildings to have very low concrete strength (less than 13.5MPa) which is basis of research for Hong and Araki [6].

Pullout and splitting was commonly observed bond failure modes during the Kashmir earthquake and became one of the causes for brittle failures in RC buildings as shown by Ahmad [7]. The re-bar slip in structural components due to bond failure are schematically shown in Fig. 1a-b. Moreover, reports by Chaudat et al. [8] and Pinho and Elanashai [9] regarding seismic testing of various low strength RC frames, designed according to old codes or construction practices, has also shown that insufficient lap splices and bond degradation are the predominant factor for strength and stiffness degradation of these structures at higher peak ground acceleration levels.

Pullout and splitting failures depend on the shear and tensile strength of concrete, respectively. The tensile and shear strength of concrete have a strong correlation to compressive strength less than 10000psi (69MPa), ACI408[10]. The force is mainly transferred by bearing against the lugs, which either exceeds the concrete tensile or shearing strength causing the failure to occur by tensile splitting or pullout (shearing of concrete), respective1y. Splitting usually occurs due to lower concrete cover ($<2d_b$) and insufficient confinement, ACI408.2[11]. When the bar moves with respect to concrete, splitting failure initiates due to the wedging action of ribs. Splitting is normally the critical bond failure mode for RC buildings and its capacity is lower than for pullout for a given anchorage length. Bar pullout usually occurs in elements with enough confinement.

Bond performance is also related to rib geometry and an increase in rib height, generally increases initial bond stiffness and enhances bond strength [10-11]. Transverse reinforcement resists splitting failure after cracking of a member especially under cyclic loading and provides confinement which results in pullout failure. Casting position and improper compaction also affect the bond strength [11].

The problem of improper compaction is quite significant in poorly constructed RC structures, where the voids and water pockets are formed due to plastic flow of concrete [11].

The bond-slip (τ-s) behaviour of different bar types used typically in LSC has not been studied much and past vulnerability assessment studies by Kyriakides (2008) [12] had to assume that the τ-s behaviour of re-bars in LSC is similar to that of Normal Strength Concrete (NSC). Only a few studies examined this behaviour; for example, Mohamad [13] conducted pullout tests on extremely LSC specimens (200x300x300mm specimens with links and $f_c^{'}$ ~ 5MPa) to evaluate the $\tau_{\max}$ of both top and bottom cast deformed and plain bars with varying cover (c) to bar diameter (d_b) ratios. The typical LSC range for NERC buildings (10-15MPa) was not considered, Feldman [14] carried out pullout tests using 16 and 32mm d_b plain bars in LSC (12 to 14MPa) to evaluate the τ-s characteristics of plain bars with different roughness levels. Experimental results showed an average slip value of 0.01mm at $\tau_{\max}$. The reported average value of $\tau_{\max}$ ranges between 0.98 to 2.2MPa for different roughness levels. The effect of smaller cover, shorter development lengths and other bar types on τ-s behaviour was not studied. More recently, Bedirhanoglu [15] carried out cyclic tests on 9 exterior beam-column joints made of LSC concrete (<10MPa) using plain bars. The mean $\tau_{\max}$ was found to be between $0.33\sqrt{f_c^{'}}$ and $0.5\sqrt{f_c^{'}}$ MPa. Hong and Araki [6] conducted pull-out tests under load reversal to study the bond characteristics of plain round bars having 13 ,19mm diameter and embedment length of $10d_b$ in LSC (11.2MPa). The maximum bond stresses of the specimens were less than the allowable stress in RC Codes of Japan Architectural Institute for the long term load, and that the degradations of bond stress were apparently found to be influenced by the loading cycles. The average bond strength value was 0.33MPa and 0.32MPa for 13 and 19mm d_b, respectively.

This main aim of conducting current research work is to investigate the bond characteristics of typically used steel bars in NERC/ existing RC structures by considering various deficient parameters and to develop bond strength relations for low strength concrete ≤15MPa. Previous researches did not account for the considered deficient parameters and provide multivariable bond strength equation for low strength concrete <15MPa. This paper initially presents the results of experiments undertaken on pullout and splitting specimens. The main parameters of the study included low strength concrete, rebar type, diameter, concrete cover and embedment length. The statistical variation of the experimental data is presented and the bond performance of different bar types is discussed. The paper finally presents development of bond strength models for different bar types in LSC by using the current experimental data

2 Experimental Programme

The experimental programme is planned to study pullout and splitting bond failure modes in LSC under monotonic loading. This experimental work is part of research work conducted by Ahmad[16], Ahmad et al.[17] for development of analytical seismic vulnerability assessment framework for reinforced concrete structures in developing countries. All the tested specimens were unconfined and made from plain concrete. The main parameters included LSC (~ 15MPa), bar development length (L_d), concrete cover (c), rebar type and diameter (d_b). Pullout and splitting tests have been conducted in a specially designed rig. The pullout and splitting experiments are designed so as to include the effect of different deficient parameters observed in the post-Kashmir earthquake surveys and are more important for bond-slip of reinforcement behaviour in RC structures of developing countries. Most common steel bars types used in old and new construction of RC structures in Pakistan are used in experiments. Low strength concrete mix design is used to prepare pullout and splitting specimens. These specimens having varying development lengths, cover, bar type and sizes are tested in testing setup. Mechanical properties of different steel bars and the LSC used are described in the following section.

2.1 Steel bars

Two different types of steel bars were used in the experimental programme, with different surface deformations and diameter. These bars were: 1- hot-rolled deformed (*def.*) 2- plain. The mechanical characteristics of the different types of steel bars are presented in Tables 1. Table 2 gives the rib details of the deformed bar used in the tests and the bar pattern is also shown schematically in Fig. 2.

2.2 Concrete

Since the concrete compressive strength for the majority of NERC structures falls between 8 and 15MPa, the LSC mix proportions mentioned in Table 3 were used to cast all the pullout and splitting specimens. The compressive strength was determined according to BS1881-121[18] by casting 100mm x 200mm cylinders from the mix. Indirect splitting tests in accordance with BS:EN12390-6[19] were also carried out to evaluate the tensile strength of concrete specimens. The mean (μ) and (σ) standard deviation values of mean $f_c^{'}$ and f_{ct} of the pullout and splitting specimens are listed in Tables 4.

2.3 Test arrangement

The test set-up including test rig and instrumentation used for conducting pullout and splitting tests is shown schematically in Fig. 3a. A 10mm rubber plate was placed on the specimen's top face to ensure an even pressure and minimum confinement on the concrete surface. The reaction plate of the rig has appropriate holes, so that the LVDT's can pass through. A mounting rig was used for holding two LVDTs (for the splitting tests) or three LVDTs (for the pull-out tests) at the loaded end (L.E) of the specimen and the mounting rig was clamped with screws on the bar (see Fig. 3b). A small metal (aluminum) angle was glued on the unloaded end (U.L.E) of each specimen to mount an LVDT. This transducer was positioned at the centre of the bar and was used to measure unloaded end slip. This arrangement corresponds to RILEM/CEB/FIP [20] in which the bonded length is located at the end of the specimen to avoid conical failures near loaded end. An example of pullout and splitting specimen in testing rig is shown in Fig. 3c and Fig 3d, respectively.

2.4 Specimen details for pullout and splitting tests

2.4.1 Pull out tests

Pullout cube specimens with two bar types and three different development lengths, $5d_b$, $10d_b$, $15d_b$, were tested. The details of the bar size, type, specimen size and embedment lengths used for making pullout specimens are shown in Fig. 4a and Fig. 5a. For L_d more than 150mm, 150mm (diameter) x 300mm (height) cylinders were used. A cubic and cylinder pullout specimens are shown in Fig. 4 b and

5 b, respectively. Bars were debonded with two layers of cling film and PVC tape to achieve the desired embedment length. The bars were cut to 500mm to fit the testing apparatus.

To measure the loaded end slip for pullout specimens, three transducers were placed in a radius of 50mm from the centre of the bar. These transducers were mounted on a small rig at an angle of 120 degree from each other. The schematic arrangement of the LVDTs positioned over the specimen surface is shown in Fig. 6

All the tests were displacement controlled and the displacement rate was set to 0.5mm/min. The LVDTs used had a maximum range of 10mm and the data record was stopped when the slip reached between 8 and 10mm. The bar was then pulled out completely at a faster rate. A typical rebar after pullout is shown in Fig. 7

2.4.2 Splitting tests

In the splitting test specimens, bars were positioned eccentrically using varying concrete covers, (i.e. c = $0d_b$, $1d_b$, $2d_b$) with reference to the concrete edge as shown in Fig. 8 a and b to achieve splitting failure mode. The embedment length in all the splitting specimens was $5d_b$ and the specimens were cast up to the same height. The bars were cut to 500mm to fit the testing apparatus.

The average of the displacements from three transducers was used to eliminate possible bending of the bar. The loaded and unloaded end slip for the splitting specimens having c = $2d_b$, was measured using the same LVDT arrangement as used for pullout tests (Fig. 6) whereas loaded end slips for the specimens having c = $0d_b$, $1d_b$ were measured by making a 2 point arrangement of the LVDT's at the loaded end, as shown in Fig. 9 a and 9b. This arrangement was necessary due to lack of space for third LVDT.

The splitting specimens with varying concrete cover after splitting are shown in Fig. 10a-10c, respectively.

3 Experimental Results

Representative results for the deformed and plain bar pullout specimens, having d_b=12mm, L_d= $5d_b$ are shown in Fig. 11 a and 11b. U.L.E. and L.E. represent the un-loaded and loaded end bond-slip curves, respectively. The L.E. slip values are determined from the average slip measurements, taken by either 2 or 3 LVDTs, minus the calculated extension of the bar outside the embedment length.

The deformed bars in general showed low bond strength for concrete with $f_c^{'}$ < 10MPa. For plain bars, the slip corresponding to bond strength is very low and the load-slip curve decays gradually.

Typical results for the deformed and plain bars splitting specimens having d_b=12mm, L_d= 5d_b and c=0-2d_b are shown in Fig. 12a and 12b, respectively. The value of c=0 represents extremely small concrete cover and practically represents exposed reinforcement condition in beams or columns typically observed in developing countries due to poor construction practices.

Deformed bar split specimens with extremely small cover showed a very low $\tau_{\max}$ at a small slip value. The bond strength increased by almost four times for curves c =1 and 2d_b (Fig. 12a). For plain bar split specimens (d_b=12mm), splitting did not occur in all cases. A few specimens with c=0 and c=1d_b showed brittle behaviour, but most of the specimens especially with cover 1 and 2d_b showed a gradual decay of the load slip curve, as shown in Fig. 12b.

4 Direct comparison and statistical analysis (Part-1 Pullout specimens)

A summary of results of tests along-with the average results for each set of variables is presented in Table 5.

The concrete strength effect on bond is traditionally taken into account by normalizing $\tau_{\max}$ with respect to $\sqrt{f_c^{'}}$. This use of $\sqrt{f_c^{'}}$ for normalization has been proved to be effective up to concrete strengths of 55MPa (ACI408 (2003)).

The bar chart in Fig. 13 shows the mean values of $\tau_{\max}/\sqrt{f_c^{'}}$ for deformed and plain bar specimens with different development lengths.

4.1 Bond strength scatter for pullout specimens

Fig. 14a shows the results of pullout tests for the specimens with 12 and 16mm diameter deformed bars and L_d =5d_b. All these specimens had a low concrete strength (~10 to 15MPa). Bond strength of specimens having concrete strength of around 10MPa is found to be almost half the bond strength of specimens with relatively higher concrete strength (~15-20MPa).The failure of most specimens with L_d=10 and 15d_b was either due to bar yielding or concrete splitting.

For plain bars, a lower variation can be seen (Fig. 14b) in the results as compared to the deformed bars (Fig. 14a). As expected an overall reduction in normalized τ_{max} is evident with larger L_d.

5 Direct comparisons and statistical analysis (Part-2 Splitting specimens)

Table 6 presents a summary of results of splitting tests and the bar chart in Fig. 15 shows the mean value of $\tau_{max}/\sqrt{f_c^{'}}$ for specimens tested for the splitting failure mode. This includes 13 and 17mm *def.* and 12 and 16mm plain bars having $L_d = 5d_b$ and varying concrete cover.

5.1 Bond strength scatter for split specimens

Fig. 16a shows that for $c/d_b = 0$, the normalized τ_{max} value for both diameters of deformed bar is significantly lower than for c/d_b = 1 and 2, which highlights the severity of the problem when proper cover is not maintained. In general, the splitting strength increases from c/d_b = 1 to 2 for both bar sizes. Nonetheless, the 16mm bar shows lower results than the 12mm bar which indicates that bar diameter also affects the splitting strength.

No splitting was observed in the plain bar specimens which means that bar roughness is the dominant factor that mobilizes the friction between the bar and the concrete to give bond strength. As a result a lower variability is obtained in the results of plain bars (Fig. 16b). However, the value of normalized τ_{max} for the plain bars clearly increases with the increase of c/d_b ratio for both 12 and 16mm diameter as shown in Fig. 16b. This means the cover thickness leads to increased confinement and as a result increased frictional resistance.

6 Development of bond strength relation

Multivariable nonlinear regression analysis is used to develop $\tau_{\max}$ models by using suitable summation function for both pullout and splitting failure modes. Variables such as concrete compressive strength ($f_c^{'}$), development length (L_d), cover (c), diameter (d_b) and bar type are included in the $\tau_{\max}$ models. Orangun et al. [21] summation function offers the best choice for use as an input function in the nonlinear regression analysis, since it includes all the important variables that were considered in the experimental work carried out by the authors. This function can be calibrated to give a $\tau_{\max}$ equation for both pullout and splitting bond failure modes.

The general form of the selected input functions for Orangun et al. [21] is given in Eq. (1).

$$Y = (A + Bw + Cx)z^{D} \qquad \text{Eq. (1)}$$

Where;

A, B, C and D are the parameter values determined through calibration, Y is the dependant variable and *w*, *x*, and *z* are the independent variables.

6.1 Regression analysis of $\tau_{\max}$ experimental data

6.1.1 $\tau_{\max}$ equations for deformed bar pullout/splitting failure mode in LSC

For evaluating a general $\tau_{\max}$ equation for the pullout and splitting failure mode of deformed bars, Eq. (1) was calibrated by conducting nonlinear regression analysis using the current experimental data and the derived new parameter values are given in Eq. (2).

An additional dataset which were selected from a report by Darwin et al [22] in which an extended database from different studies was used to develop an expression for evaluating the splice strength and development length without considering the effect of transverse reinforcement. The specimens with concrete strength ranging between 15 and 21MPa were extracted from each dataset. The predominant failure mode in all these test data was splitting and the beams were tested to evaluate the bond strength considering different parameters. It was observed from the data that large bar diameter and embedment lengths were used almost in all the specimens except for the Tepfers[23] dataset which includes specimens with bar diameters 12, 16 and 19mm. Moreover, c/d_b in the majority of the tests varied between 1 and 2 with very few specimens having c/d_b ratio > 2. Due to the variability in the experimental data from different sources, the error between predicted and experimental values is assumed to be normally distributed and an uncertainty factor in accordance with ±1σ was evaluated. The resulting equation is given in Eq. (2) with an uncertainty factor of ±2.1 MPa.

$$\frac{\tau_{\max}}{f_c^{'0.68}} = -0.048 + 0.22\frac{c}{d_b} + 3.22\frac{d_b}{L_d} \pm 2.1 \qquad R^2=0.72 \qquad \text{(MPa)} \qquad \text{Eq. (2)}$$

The bond strength predictions ($\tau_{\max,pred.}$) from Eq. (2) are compared with the experimental bond strength values ($\tau_{\max,\exp}$) in Fig. 17a. The upper and lower bound has been set to ± 1σ MPa to assess the percentage of data above and below this range. It was found that 15% of the data were out of this range . The frequency of ratio $\tau_{\max,pred.}/\tau_{\max,\exp}$ as a percentage of the total data is shown in Fig. 17b. For Eq. (2), a larger percentage ratio of ratios $\tau_{\max,pred.}/\tau_{\max,\exp}$ was found to be close to one as shown in Fig. 17b.

6.1.2 $\tau_{\max}$ equation for plain bar pullout/splitting failure mode

Eq. (1) was further calibrated for plain bars and the resulting equation is given as Eq. (3). The uncertainty factor for this equation is calculated to be ±0.96.

$$\frac{\tau_{\max}}{\sqrt{f_c^{'}}} = 0.253 + 0.1902\frac{c}{d_b} + 2.385\frac{d_b}{L_d} \pm 0.96 \qquad R^2=0.69 \qquad \text{(MPa)} \qquad \text{Eq. (3)}$$

The predictions from Eq. (3) are compared with the experimental data in Fig. 17c. The frequency of the ratio $\tau_{\max,pred}/\tau_{\max,\exp}$ as a percentage of total data is shown in Fig. 17d. 19% data was found to be out of the set bound for the plain bar bond strength.

In the current study the concrete strength power factor of 0.68 and 0.5 are evaluated for deformed, cold formed and plain bar. This suggests large dependency of concrete strength on bond strength for deformed and cold formed bars as compare to plain bars. In Orungun et al. Bond strength equation,

$\tau_{\max}$ is normalized with respect to $\sqrt{f_c^{'}}$ to represent the effect of $f_c^{'}$ (or the tensile strength) on the bond strength. However, Zuo and Darwin [24] suggested that normalization using $\sqrt{f_c^{'}}$ overestimates bond strength for the HSC and underestimates bond strength for NSC. $f_c^{'1/4}$ was found to have better correlation with bond strength for all ranges of concrete strength.

7 Conclusions

Due to large number of NERC structures in the building stock of developing countries, it is important to investigate effect of different deficient parameters on bond characteristics for more reliable seismic vulnerability assessment.

a. Among the investigated parameters, low strength concrete and concrete cover have more influence on the bond strength. For the deformed bar specimens with concrete strength of around 10MPa, the pullout bond strength (1.98MPa) is almost half the bond strength of specimens with relatively higher concrete strength (15-20MPa). Hence, NERC structures with concrete strength of around 10MPa are expected to have larger slip deformations and brittle failures due to lower bond strength.

b. The specimens with very small cover (i.e. c=0) which is an exposed bar condition, have very low splitting bond strength (0.46Mpa) and indicates high vulnerability of inferior quality structures with exposed bars.

c. The plain bars did not show evidence of splitting in most of the specimens. However, the cover still appears to have effect due to the additional confinement. Nonetheless, plain bars fail at a lower strength and have inferior post peak characteristics.

d. The data from this experimental study are used to develop the bond strength models for different bar types in LSC. The bond strength models from the summation equations are therefore proposed, for prediction both the splitting and pull-out behaviour of low strength structures. The summation function accounts for all the studied parameters and the bond strength for pullout and splitting bond failure modes is predicted reasonably well using the developed equation. These equations can also be used in defining the τ-s behaviour.

e. The higher power factor of 0.68 for concrete strength are evaluated for deformed bars as compare traditional value of 0.5. This indicates larger influence of low strength concrete on bond strength of deformed as compare to normal and high strength concrete.

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List of Figures

Fig. 1 Bond related damages and detailing deficiencies a) Bar pullout b) Short lap splice

Fig. 2 Schematic diagrams of different bars with rib patterns

Fig. 3 Test setup adopted for experiments: a) cross-sectional view of pull-out rig with specimen and instrumentation, b) smaller mounting rig with LVDTs at loaded end, c) rig for splitting tests, d) side splitting of plain bar specimen

Fig. 4 Pullout cube specimens with varying bar sizes, types and embedment lengths

Fig. 5 Pullout cylinder specimens with varying bar sizes, types and embedment lengths

Fig. 6 LVDTs arrangement in rig at the loaded end (pullout test)

Fig 7 Steel reinforcing bar after pullout test

Fig. 8 Splitting specimens with varying covers (c= $0d_b$, $1d_b$, $2d_b$)

Fig. 9 a and b Loaded and unloaded end slip measurement (splitting tests)

Fig 10 a-c Splitting specimen with different failure modes

Fig. 11 Typical bond-slip curves from pullout tests a) 12mm deformed b) 12mm plain

Fig. 12 Typical bond-slip curves from splitting tests a) Deformed (c=0, 1, 2d_b) b) Plain (c=0, 1, 2d_b)

Fig. 13 Mean normalized τ_{max} of different bar types and sizes (Pullout specimens)

Fig. 14 Normalized τ_{max} at different development lengths for different bar types and sizes a) Deformed b) Plain (Pullout specimens)

Fig. 15 Mean normalized τ_{max} of different bars types and sizes with varying cover (Splitting specimens)

Fig. 16 Normalized τ_{max} of different bar types with varying cover a) Deformed b) Plain (Splitting specimens)

Fig. 17 Assessment of different bars bond strength model a) Scatter between deformed bar τ_{max} experimental and predicted results b) % Frequency of deformed bar τ_{max} normalized data c) Scatter between plain bar τ_{max} experimental and predicted results d) % Frequency of plain bar τ_{max} normalized data.

List of Tables

Table 1: Mechanical properties of bars

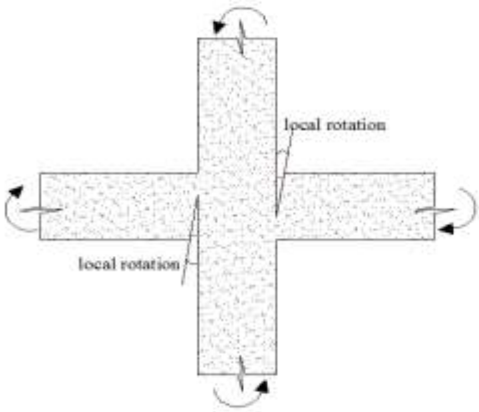
Table 2: Rib details of deformed bars

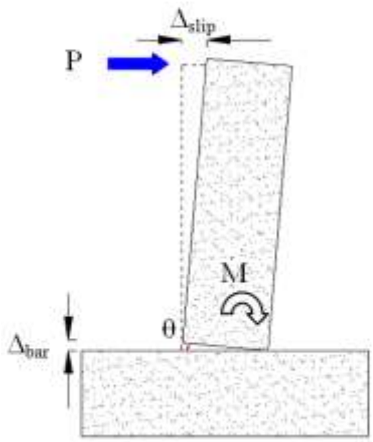
Table 3: Details of the various mixes used for making LSC

Table 4: Mean compressive and tensile concrete strength for pullout and splitting specimens

Table 5: Summary of the pullout tests using averages for each set of variables

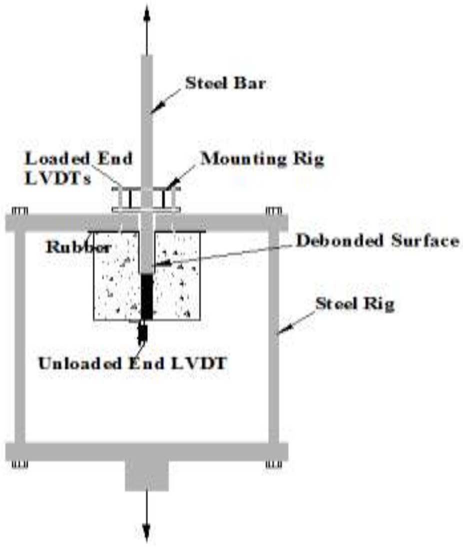
Table 6: Summary of the splitting tests using averages for each set of variables





(a) (b)

Fig.1 Bond related damages and detailing deficiencies a) Bar pullout b) Short lap splice



(a) (b)

(c) (d)

Fig.3 Test setup adopted for experiments: a) cross-sectional view of pull-out rig with specimen and instrumentation, b) smaller mounting rig with LVDTs at loaded end, c) pullout specimen testing, d) splitting specimen testing

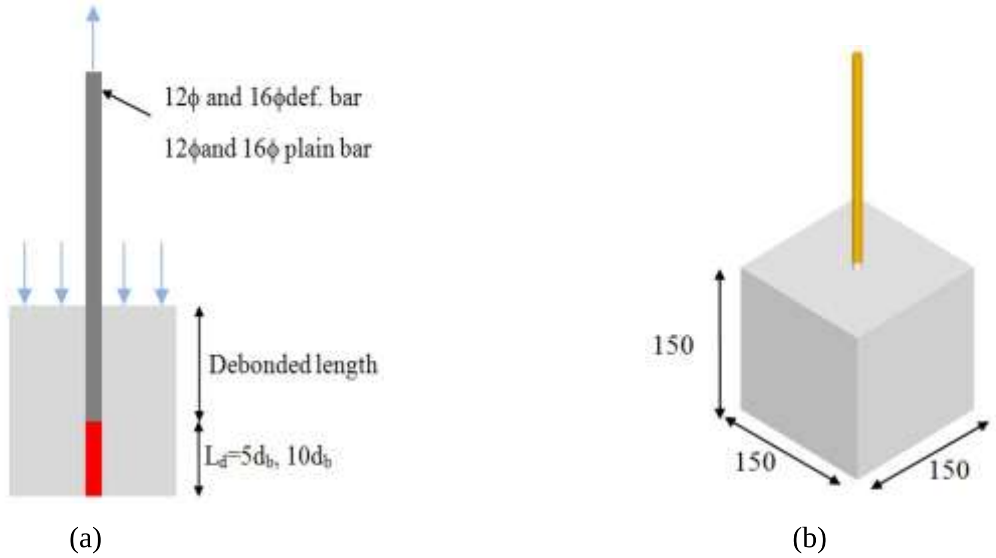


Fig. 4 Pullout cube specimens with varying bar sizes, types and embedment lengths

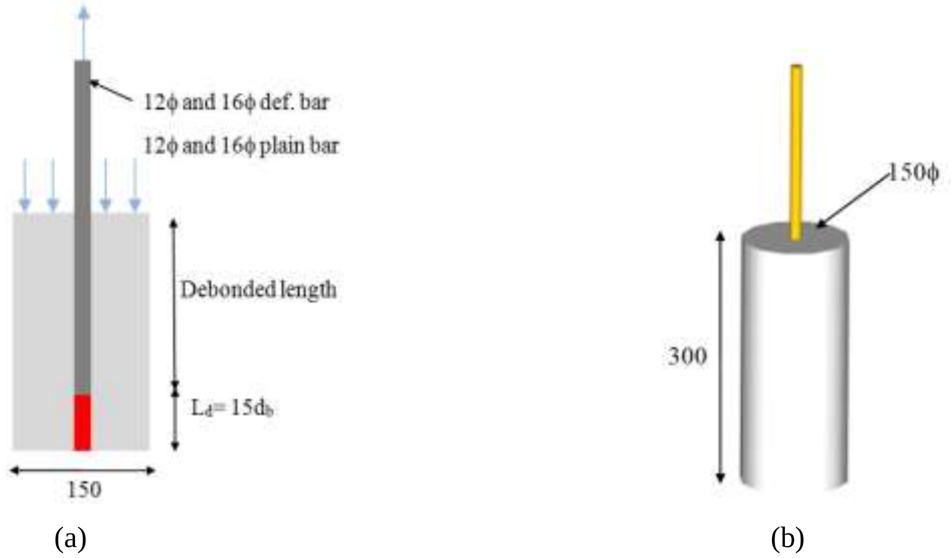


Fig. 5 Pullout cylinder specimens with varying bar sizes, types and embedment lengths

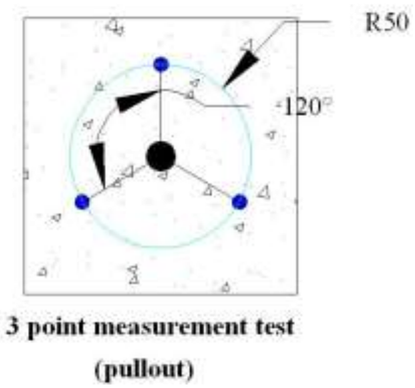
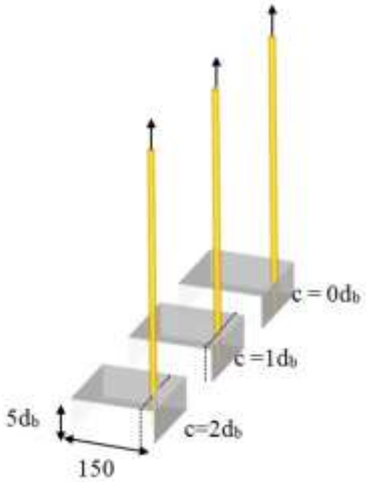


Fig. 6 LVDTs arrangement in rig at the loaded end (pullout test)

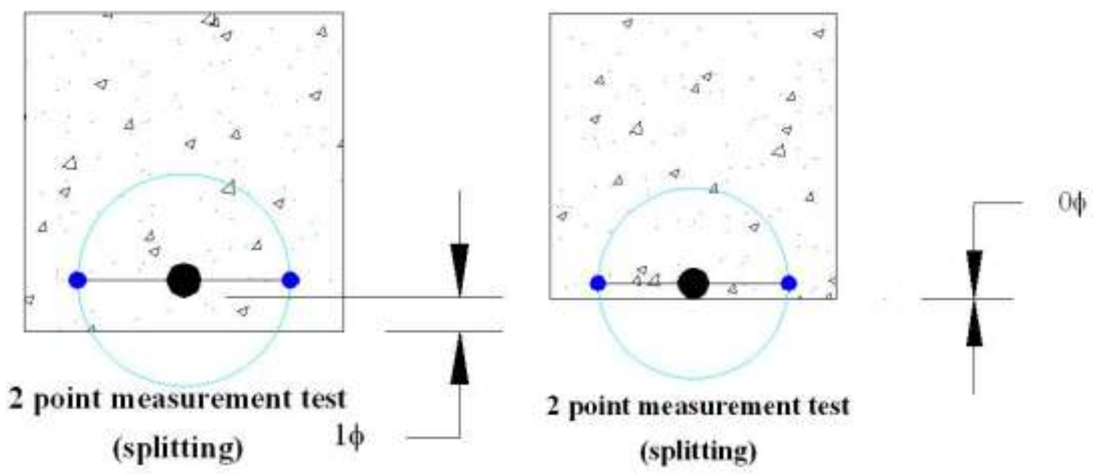
Fig.7 Steel reinforcing bar after pullout test



(a)

(b)

Fig. 8 a and b Splitting specimens with varying covers (c= $0d_b$, $1d_b$, $2d_b$)

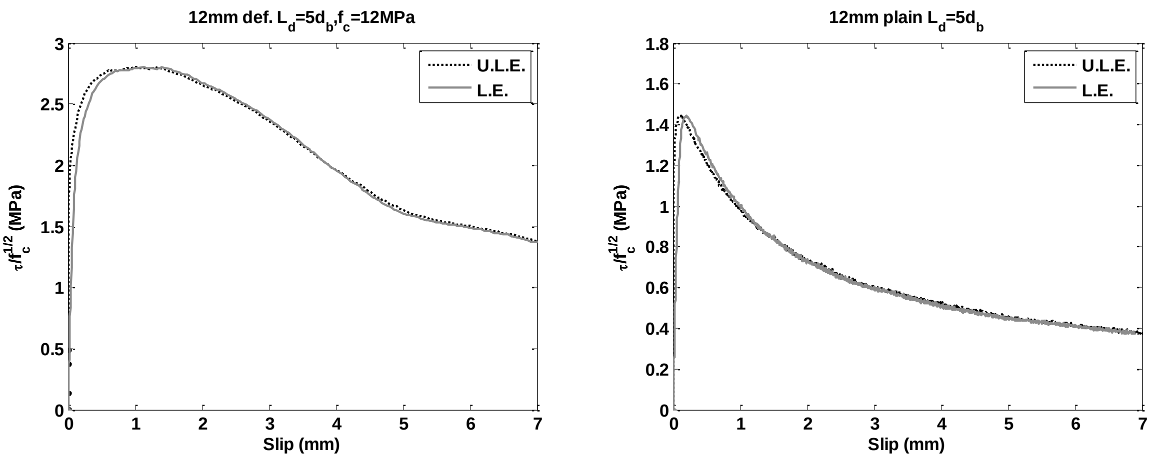


(a) (b)

Fig. 9 a and b Loaded and unloaded end slip measurement (splitting tests)

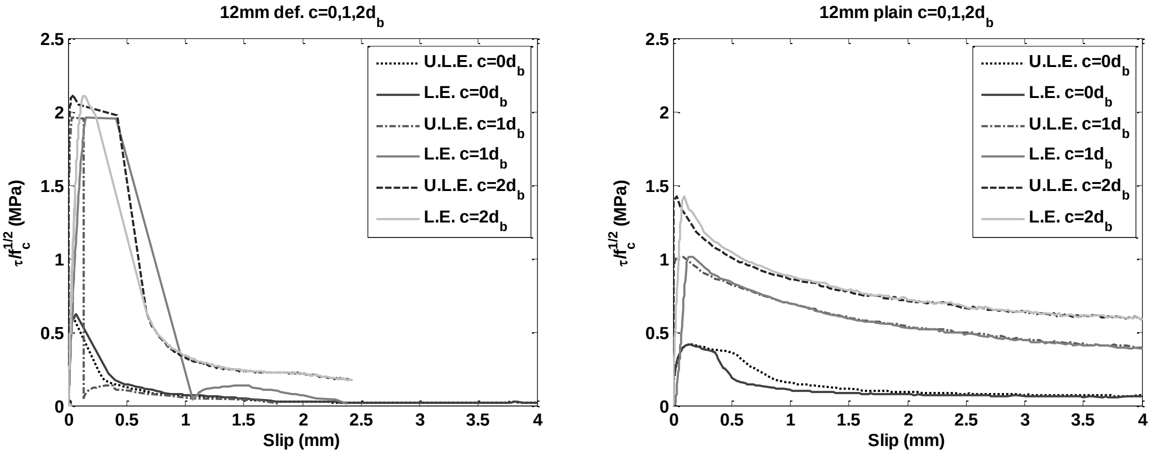
(a) (b) (c)

Fig. 10 a-c Splitting specimen with different failure modes a) V-notch splitting $c=1d_b$ b) Exposed bar $c=0d_b$ c) Side splitting $c=1d_b$



(a) (b)

Fig.11 Typical bond-slip curves from pullout tests a) 12mm deformed b) 12mm plain



(a) (b)

Fig.12 Typical bond-slip curves from splitting tests a) Deformed (c=0, 1, $2d_b$) b) Plain (c=0, 1, $2d_b$)

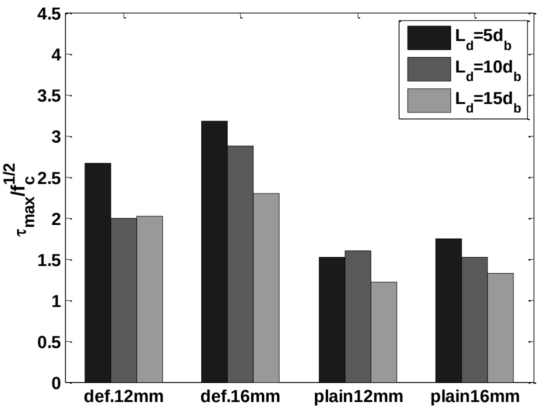
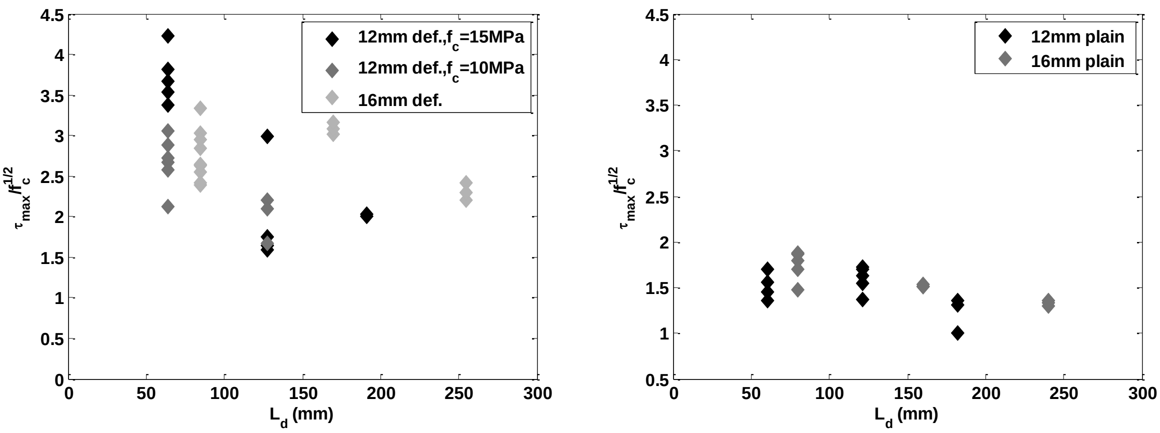


Fig.13 Mean normalized τ_{max} of different bar types and sizes (Pullout specimens)



(a) (b)

Fig.14 Normalized τ_{max} at different development lengths for different bar types and sizes a) Deformed b) Plain (Pullout specimens)

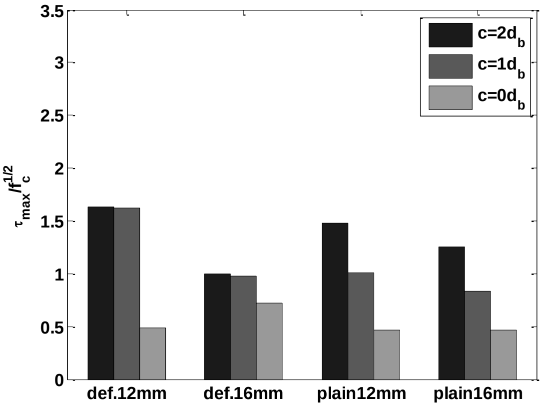
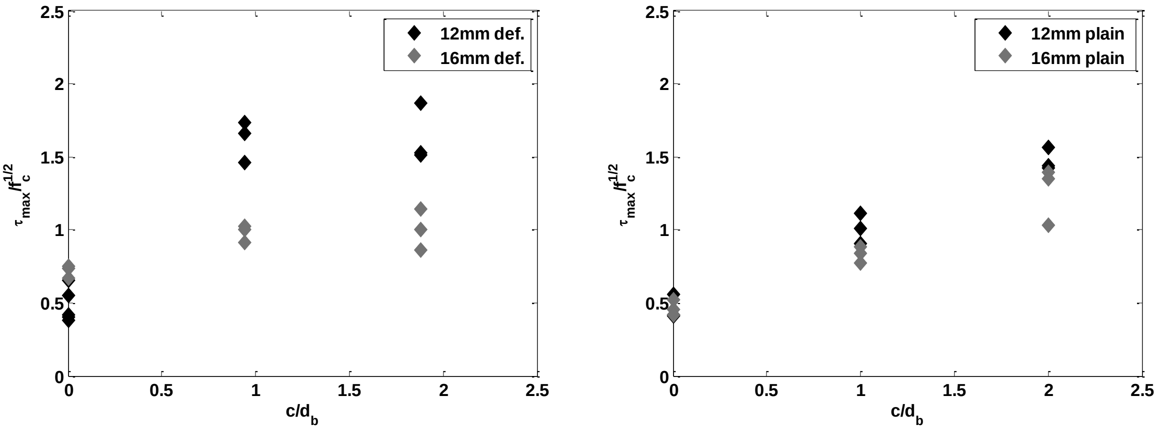
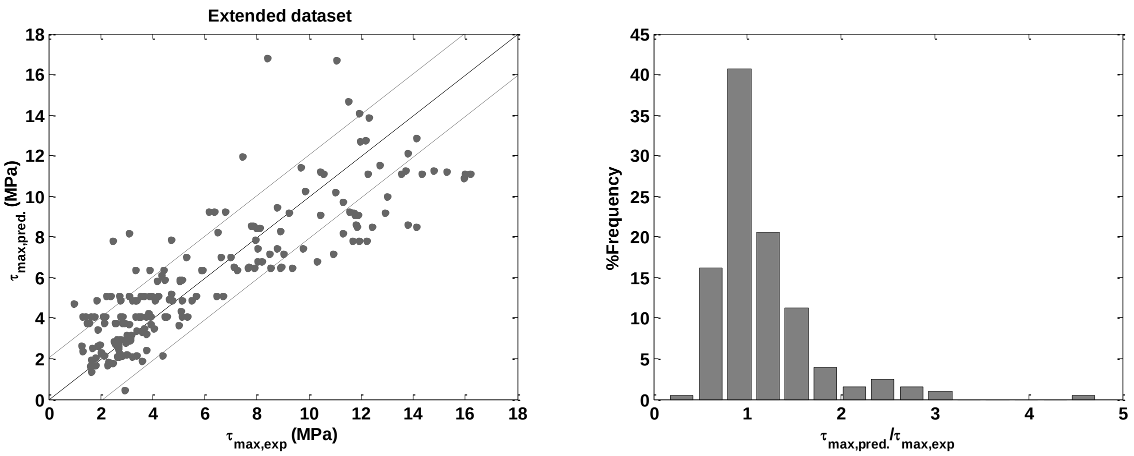


Fig.15 Mean normalized τ_{max} of different bars types and sizes with varying cover (Splitting Specimens)

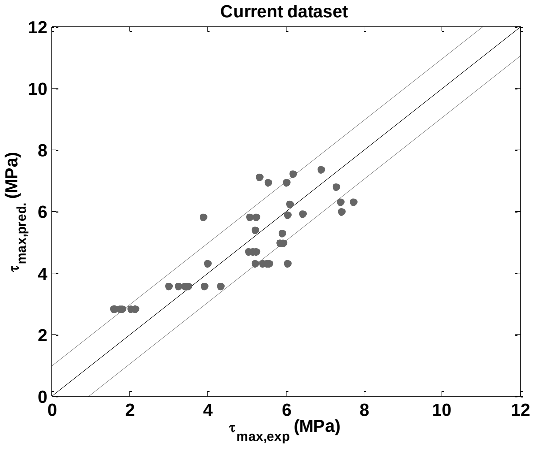


(a) (b)

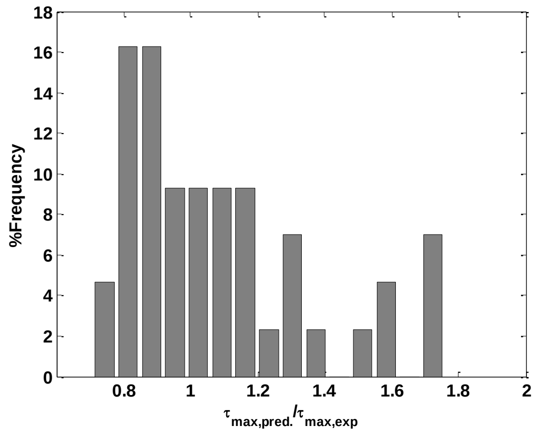
Fig.16 Normalized τ_{max} of different bar types with varying cover a) Deformed b) Plain (Splitting Specimens)



(a) (b)



(c)



(d)

Fig. 17 Assessment of different bars bond strength model a) Scatter between deformed bar τ_{max} experimental and predicted results b) % Frequency of deformed bar τ_{max} normalized c) Scatter between plain bar τ_{max} experimental and predicted results d) % Frequency of plain bar τ_{max} normalized data.

Table 1 Mechanical properties of bars

Bar type	Bar diameter	Young's modulus	Yield strength	Ultimate strength	Strain at yielding	Ultimate strain
	(mm)	(GPa)	(MPa)	(MPa)	%	%
Def.	12	206	472	609	0.23	1.9
Def.	16	200	479	579	0.24	1.7
Plain	12	197	315	359	0.16	0.73
Plain	16	201	323	387	0.16	0.86

Table 2 Rib details of deformed bars

Bar diameter	Rib spacing(s)	Rib height(h)
(mm)	(mm)	(mm)
12	13	2
16	17	2

Table 3 Details of the various mixes used for making LSC

C	S	A	w/c	C:S:A	Curing
kg/m^3	kg/m^3	kg/m^3			days
313	619	1188	0.75	1:2:3.8	5

Table 4 Mean compressive and tensile concrete strength for pullout and splitting specimens

University	$f_c^{'}, \mu$	σ	f_{ct}, μ	σ
	(MPa)	(MPa)	(MPa)	(MPa)
UoS (Pull and splitting)	14.5	1.89	2.66	0.31

Table 5 Summary of the pullout tests using averages for each set of variables

d_b	Bar type	L_d	*n	$f_c^{'}$	τ_{max}	$\tau_{max}/\sqrt{f_c^{'}}$	$\tau_{max}/\sqrt{f_c^{'}}$	$\tau_{max}/\sqrt{f_c^{'}}$
					μ	μ	σ	COV
mm		mm		MPa	MPa	MPa	MPa	
12.75	*Def.*	64	5	14.7	14.3	3.72	0.324	0.087
12.75	*Def.*	64	3	9.2	7.8	2.55	0.397	0.155
12.75	*Def.*	64	3	12.5	9.9	2.79	0.247	0.088
12.75	*Def.*	128	4	15.0	7.7	1.99	0.667	0.335
12.75	*Def.*	128	3	10.0	6.3	1.98	0.283	0.142
12.75	*Def.*	191	3	15.0	7.8	2.01	0.009	0.004
17	*Def.*	85	3	15.0	11.3	2.91	0.480	0.160
17	*Def.*	170	3	15.0	11.9	3.08	0.072	0.023
17	*Def.*	255	3	15.0	8.9	2.30	0.106	0.046
12	*plain*	60	4	15.5	6.0	1.52	0.147	0.097
12	*plain*	120	5	16.2	6.4	1.59	0.143	0.090
12	*plain*	180	3	15.0	4.7	1.22	0.192	0.157
16	*plain*	80	5	14.8	6.7	1.74	0.166	0.095
16	*plain*	160	3	15.0	5.9	1.52	0.013	0.009
16	*plain*	240	3	15.0	5.1	1.33	0.027	0.020

*n is the number of tested specimens

Table 6 Summary of the splitting tests using averages for each set of variables

d_b	Bar type	cover	c/d_b	n	$f_c^{'}$	τ_{max}	$\tau_{max}/\sqrt{f_c^{'}}$	$\tau_{max}/\sqrt{f_c^{'}}$	$\tau_{max}/\sqrt{f_c^{'}}$
		c					μ	σ	COV
mm		mm			MPa	MPa	$MPa^{1/2}$	$MPa^{1/2}$	
12.75	*Def.*	26	2.0	3	15.0	6.3	1.64	0.201	0.123
12.75	*Def.*	13	1.0	3	15.0	6.3	1.62	0.139	0.086
12.75	*Def.*	0	0.0	5	15.0	1.9	0.48	0.118	0.244
17	*Def.*	34	2.0	3	15.0	3.9	1.00	0.138	0.138
17	*Def.*	17	1.0	3	15.0	3.8	0.98	0.058	0.059
17	*Def.*	0	0.0	3	15.0	2.8	0.72	0.044	0.062
12.75	*plain*	26	2.0	3	15.0	5.7	1.47	0.077	0.052
12.75	*plain*	13	1.0	3	15.0	3.9	1.01	0.105	0.104
12.75	*plain*	0	0.0	3	15.0	1.8	0.46	0.081	0.175
17	*plain*	34	2.0	3	15.0	4.9	1.26	0.199	0.158
17	*plain*	17	1.0	3	15.0	3.2	0.83	0.054	0.066
17	*plain*	0	0.0	3	15.0	1.8	0.46	0.055	0.119

Nomenclature and Abbreviations

E_s modulus of elasticity of steel

$f_c^{'}$ concrete compressive strength

f_{ct} concrete tensile strength

$\tau_{\max}$ bond strength

L_d development length

c concrete cover

d_b bar diameter

h rib height of re-bar

s rib spacing of re-bar

μ mean

σ standard deviation

LSC Low Strength Concrete

NERC Non-Engineered Reinforced Concrete

RC Reinforced Concrete

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Dr. Ahmad's Ph. D is related to development of probabilistic analytical framework for seismic vulnerability assessment of low strength reinforced concrete structures of developing countries. He has several publications in reputed international journals and has presented research papers, posters in various conferences. He has done a fellowship in seismic safety assessment of structures, systems and components (SSC's). He has supervised / co-supervised several research thesis at Masters and Ph.D level in different engineering universities. He is member of Research Management Committee for PhD students at UET Taxila, Pakistan for evaluation of research work. He is principal investigator for different projects. His research interests include concrete material, seismic fragility and risk analysis of buildings and bridges, seismic retrofitting using FRP, structural health monitoring, aging of structures. He also has won various awards

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