1   **BEHAVIOR OF EPOXY BONDED BARS IN CONCRETE AFFECTED BY**

2   **ALKALI-SILICA REACTION**

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cementitious materials.

ABSTRACT

Installation of drilled-in epoxy bonded reinforcing bars is generally an effective strengthening

method to increase the flexural and shear capacities of deficient concrete structures. However,

most of the available studies characterizing the bond behavior of epoxy bonded bars in concrete

have been carried out on sound concrete elements, i.e., without any pathological material damage.

This raises the question of bond capacities in existing damaged elements. This study investigates

the influence of alkali-silica reaction (ASR) on the capacity of post-installed reinforcing bars.

ASR is a deleterious mechanism that causes expansion and cracking in the affected concrete

elements. Pull-out tests on post-installed reinforcing bars having embedded lengths of $2d_b$, $4d_b$ and

$5d_b$ with 15M reinforcing bars (d_b = 15.9 mm [0.626 in]) have demonstrated a drop in bond

strength when concrete is affected by ASR. In addition, the study revealed that the progression of

concrete expansion due to ASR, may lead to some confinement of the post-installed reinforcing

bar and possible increases the bond strength.

Keywords: post-installed reinforcing bar; pullout test; alkali-silica reaction (ASR); bond

INTRODUCTION

Many older thick concrete slabs were designed with the shear carried by the concrete, without any

shear reinforcement. However, shear failures in structures without shear reinforcement are very

brittle. This type of failure occurs with little or no warning signs and a low deformation capacity

that limits the redistribution of the internal shear stresses in the structure. The collapse of the

Concorde overpass on September 30th, 2006 in Laval (Canada), after 36 years in service,

demonstrates the hazardousness and the brittleness of shear failures in aging thick concrete

48   members without shear reinforcement[1]. This event demonstrated that degradation of the concrete

49   material may reduce the concrete shear capacity. Moreover, increased traffic loads compared to

50   the loading considered at the time of the design reduces the safety of such structures. Similar

51   structures built between the 50's and 70's in Canada now show signs of concrete degradation and

52   some of them required shear strengthening.

53   Several shear strengthening techniques for existing thick concrete slabs were investigated in the

54   past few years. Tests indicated that one of the most efficient strengthening methods consists of

55   inserting vertical shear reinforcing bars in pre-drilled holes, anchored with epoxy adhesive [2, 3] (see

56   Fig. 1). This technique also proved to be efficient in slabs to resist punching[4] and for beam-to-

57   column connections[5]. Results indicated that the bond behavior of post-installed bonded shear

58   reinforcement appeared to be the most important parameter governing the shear reinforcement

59   efficiency. The intersection of the critical shear crack with the post-installed bonded bars

60   determines the bar embedded lengths above the crack intersection and below the crack

61   intersection. The bond-slip relationship dictates the ability of the added bars to develop tensile

62   stresses and hence influences the shear contribution of the shear reinforcement, as well as the

63   crack width which, in turn, influences the aggregate interlock and thus, the shear capacity

64   provided by the concrete [6, 7].

65   The bond-slip relationship of an embedded bar is governed by three principal bond force transfer

66   mechanisms: chemical adhesion along surfaces (bar and/or concrete) as well as friction, and

67   bearing on the bar ribs [8, 9]. On the contrary to cast-in-place bars, post-installed bonded bars are

68   composed of two different interfaces: bar to bonding material (grout, epoxy, etc.) and bonding

69   material to concrete. The chemical adhesion and the friction mechanisms are influenced by the

70   installation conditions, i.e., the surface roughness and the presence of dust or any other

71   contaminant on the bars and/or concrete surfaces. For cast-in-place bars, the bearing capacity

72 depends on the concrete mechanical properties and on the bar geometry, i.e., the rib bearing area

73 and the spacing between ribs along the bar [10-17]. The capacity of post-installed reinforcing bars

74 also depends on the bonding material properties. The effect of cracks on the response of bonded

75 anchors and post-installed reinforcing bars have been investigated [5, 16]. It was observed that

76 concrete cracks parallel to the anchors influence bond strength. In his researches, Mahrenholtz[5]

77 observed a large scatter in the bond strength in cracked specimens due to the influence of the bars

78 post-installation conditions. According to Eligehausen *& al.*[16], the bond strength loss due to the

79 propagation of mechanical cracking on post installed reinforcing bars can be considered as 50% of

80 the typical bond strength of post-installed reinforcing bars in sound concrete. The cracking pattern

81 studied was one single crack crossing the bar. Although some bond slip relationships are available

82 in the literature for cast-in-place or post-installed epoxy bonded bars [16, 18, 19] these are based on

83 experimental studies performed on sound concrete elements. In the context of shear strengthening

84 of existing concrete structures, it is expected that concrete may have experienced damage and

85 cracking that may originate from many different phenomena such as loading cycles, freeze-thaw

86 cycles, steel corrosion and alkali-silica reaction (ASR) to name a few.

87 Among these, ASR is a very common deleterious mechanism resulting from the chemical reaction

88 between the alkali hydroxide ions (K^+, $Na^+ - OH^-$) within the pore solution of concrete and some

89 siliceous mineral phases from the fine and/or the coarse aggregates. ASR generates a hygroscopic

90 alkali-silica gel inside the reactive aggregates that swells under high relative humidity conditions,

91 leading to the formation of cracks in the aggregate particles and, eventually, extending into the

92 cementitious matrix. The cement dosage, the use of de-icing salts and the exposure to sea water

93 represent examples of alkali sources which may be significant in existing bridges and influencing

94 the reaction kinetic and longevity. Amongst other parameters, the ASR potential and reaction

95 kinetics of aggregate materials will depend on the nature and proportion of the siliceous phase

96 within the reactive rock types, as well as the aggregate particle size, the ASR potential is

97 influenced by the specific area of the silica particles, i.e., the reaction tends to occur more rapidly

98 as the finesse of particles decreases [20]. Deleterious effects on ASR-affected structures will last as

99 long as the favorable conditions, i.e., pH over 13, high amount of alkali, unstable siliceous

100 material and high relative humidity, are maintained. Research on the effects of ASR in concrete is

101 given by Sims *& al.*[21], Lindgård *& al.*[20], Sanchez *& al.*[22], Thomas *& al.*[23] and Hobbs[24]. Many

102 authors have demonstrated that the material mechanical properties decrease for affected concrete

103 specimens/elements [22, 25-28]. Since ASR is very common in North America, this paper aims to

104 investigate its effect on bond strength of post-installed reinforcement.

105   **RESEARCH SIGNIFICANCE**

106 Since many damaged existing structures are in need of shear strengthening, the effect of a

107 damaged concrete matrix on the bond mechanical properties of post-installed reinforcing bars

108 must be better understood. This research compares the bond mechanical properties of epoxy

109 bonded bars embedded in sound concrete to the ones in concrete affected by alkali-silica reaction

110 (ASR) through pullout tests. Moreover, the effects of ASR progression on the bond strength of an

111 already existing post-installed reinforcing bar were studied.

112   **EXPERIMENTAL INVESTIGATION**

113 The pullout tests were performed on sound and ASR-damaged concrete. The following paragraphs

114 describe the concrete mixes, specimen geometry as well as testing procedure.

115 **Concrete Materials**
116 Concrete mixes were designed in order to reach 35 MPa (5000 psi) of compressive strength after

117 28 days of moist curing. A highly reactive coarse aggregate (high amount of reactive silica

118 particles) from Albuquerque (New Mexico) and a high-alkali Portland cement (Na_2O_{eq} of 1.12%)

119 were selected in order to accelerate the development of ASR. Tests carried-out by Sanchez[29] and

120 Villeneuve[30] confirm the coarse aggregate reactivity. The petrographic facies and potentially

121 reactive phases of the New Mexico (NM) aggregate are presented in Table 1 [30]. NaOH was added

122 to the reactive concrete mix to accelerate the ASR reaction. The final mix design had a Na_2O_{eq}

123 content of 1.25% per mass of cement, which is sufficient to allow ASR with the extremely

124 reactive NM aggregate [24]. Based on the same concrete mix design, a sound concrete mix was

125 produced by incorporating a lithium nitrate ($LiNO_3$ – 30% solid; lithium-to-alkali molar ratio of

126 0.93 calculated in accordance with Thomas & al.[31] admixture to inhibit the ASR. Non-reactive

127 granitic sand from Quebec City was included in the two concrete mixes. Their water-to-cement

128 ratio is 0.47. The two concrete mix designs are presented in Table 2.

129 **Specimens**
130 Twenty four (24) concrete blocks, 350 x 350 x 350 mm (13.78 x 13.78 x 13.78 in) in size, were

131 cast. These geometrical dimensions were chosen to avoid concrete splitting during testing. Twelve

132 (12) specimens of ASR reactive concrete (specimens A) and twelve (12) other companion

133 specimens of sound concrete (specimens S) were cast. All these specimens were cured and

134 conditioned under the same environmental conditions.

135 After 28 days of moist curing at room temperature (23 ± 2ºC [73.4 ± 3.6ºF]), test specimens type

136 S and type A were stored in hermetic containers, in a room at 38 °C (100.4 °F). Relative humidity

137 inside the containers was kept over 95% during conditioning. The expansion caused by ASR was

138 monitored according to embedded stainless steel stud's relative displacements on three faces of

139 the concrete cubes. Dimensional changes parallel and perpendicular to the casting plane (Fig. 2)

140 were monitored separately.

141 Two series of pullout tests were carried out. For the first series, specimens A1 and S1 were tested

142 when the perpendicular expansions of specimens A1 have reached 0.20% in average (~100 days).

143 For the second series, specimens A2 and S2 were tested when the perpendicular expansions of

144 specimens A2 have reached 0.30% in average (~200 days).

6

145 Following the conditioning period, holes were drilled with a percussion drill into the concrete

146 specimens and cleaned according to the manufacturer specifications. Epoxy adhesive (see Table 3

147 for the mechanical properties of epoxy) was injected into the concrete holes and steel reinforcing

148 bars (cross section area $A_b = 200$ mm^2 [0.310 in^2] and diameter $d_b = 15.9$ mm [0.626 in]) were

149 installed according to three different embedded lengths (h_{eff}) of $2d_b$, $4d_b$ and $5d_b$ (~32 mm [1.26

150 in], 64 mm [2.52 in] and 80 mm [3.15 in]). The rib index F_R of the reinforcing bars, which is

151 defined by the average rib height (1.2 mm, 0.047 in) divided by the average rib spacing (9.7 mm,

152 0.382 in), was 0.125 satisfying ASTM-A996/A996M[32] requirements. The post-installed

153 reinforcing bars were installed so that their longitudinal axis was perpendicular to the concrete

154 casting plane. The selected bar diameter is the same as the one considered for post-installed shear

155 reinforcement in previous concrete thick slab tests [2, 3] .

156 Some of the concrete blocks were used twice as one bar was installed on two opposite sides of the

157 same concrete specimen making sure that these two bars would not disturb one another's behavior

158 under pullout testing. Finally, two specimens type A1 and two specimens type A2 were used in a

159 third series to study the effects of further ASR damage progression on the bond behavior of

160 already installed bars. These specimens, identified as D1 and D2, are based on specimens A1 and

161 specimens A2, respectively, who were returned to conditioning and monitored for an

162 approximatively additional 100 days after the installation of the bonded bars. It is worth

163 mentioning that the exposed steel bars in these specimens were protected against corrosion with a

164 specialized paint.

165 **Testing Procedure**
166 The test setup designed is shown in Fig. 3. The tension load was applied through the reinforcing

167 bar at a rate of 2 mm/min (0.75 in/min) and the tested specimen concrete block upper face offered

168 bearing capacity with the help of a supporting plate and steel rods. A ball joint was used to ensure

169 proper uniaxial tensile loading. This design was chosen to better control the failure mode, i.e., to

170 avoid concrete cover splitting or concrete cone failure, and to determine the full bond-slip

171 relationship of a pulled out bar. Fig. 3 also shows that the steel reinforcing bar is not bonded to

172 concrete along the top 100 mm (3.94 in) of embedment. That design reduces the effect of

173 confining pressure caused by the top supporting plate [33, 34]. The elongation of the steel reinforcing

174 bars was monitored by two extensometers (unbonded length) while the relative displacement

175 between concrete surface and the steel reinforcing bars (slip s) was monitored by four linear

176 variable differential transformers (LVDTs). After each test, the bar was extracted from the block,

177 the embedded length was precisely measured and pictures of the bar and the concrete block were

178 taken.

ASR Damage Assessment
180 ASR concrete damage was assessed through the stiffness damage test (SDT)[35] and the concrete

181 mechanical properties were determined according to the compressive strength test[36], the splitting

182 tensile strength test [37] and the Young's modulus test[38]. These tests were performed on concrete

183 cores (100 X 200 mm [3.94 X 7.87 in]) extracted perpendicularly to the concrete casting plane

184 from the tested specimens. Five (5) compressive loading-unloading cycles were performed on

185 selected concrete cores to assess the degree of damage in ASR affected concrete. Young's

186 modulus (E_c) and stiffness damage index (SDI) were determined based on the SDT results. SDI is

187 a diagnosis parameter defined as the irreversible deformation energy divided by SDT total energy

188 (elastic and irreversible) [39]. As explained by Allard & al.[40], the SDI, provides information

189 regarding the level of internal cracking. The Young's modulus was determined as the average

190 from the second and the third loading cycles of the SDT test. In accordance with the SDT

191 procedure proposed by Sanchez & al.[39], the maximum stress reached during the SDT test

192 corresponded to 40% of the compressive strength of the sound concrete.

MATERIAL RESULTS AND DISCUSSION

194   The following section presents the results of the damage assessment test (SDT) and the material's

195   mechanical properties which are summarized in Table 4.

196   The slump and air content of fresh concrete were 155 mm [6.10 in] and 1.7% for sound concrete

197   mix and 155 mm [6.10 in] and 1.6% for ASR reactive concrete mix (CSA A23.2[41]).

198   The compressive strengths, f'_c, of sound concrete determined on cylinders at 7 and 28 days were

199   respectively 29.1 MPa (4220 psi) and 37.0 MPa (5370 psi), while compressive strengths of 29.8

200   MPa (4320 psi) and 36.3 MPa (5260 psi) were respectively measured for ASR reactive

201   concretes[36]. It must be noted, however, that after 28 days of moist curing, none of the "reactive"

202   specimens showed macroscopic signs of ASR damage.

203   The steel reinforcing bars were tested in uniaxial tension in accordance with ASTM-E8[42] and

204   ASTM-E111[43]. The yield strength (f_y), the ultimate strength (f_u), the strain at rupture (ε_u) and the

205   Young's modulus (E_s) were respectively 456 MPa (66.1 ksi), 567 MPa (82.2 ksi), 0.175 and 190

206   GPa (27600 ksi).

Observation of ASR Damage

207
208   Observation of concrete cores extracted from specimen types A and D and examined with a

209   stereomicroscope (15X magnification) has revealed signs of damage associated with ASR which

210   may be characterized as follows: opened cracks in the coarse aggregate particles with reaction

211   product (OAC+RP), cracking in the cement paste with reaction product (CCP+RP), air voids

212   filled with reaction product (V+RP) and reaction rims around the reactive aggregate particles

213   (RR). These signs are typical of ASR and confirmed its presence in specimens A and D. In

214   addition, the reaction product observed in cracks and voids had the typical texture of alkali-silica

215   gel. Typical damage observed from polished concrete core sections are shown in Fig. 4 from a

216   specimen type A.

Expansion

217
218   Expansion was monitored in all three directions[44] but results are reported according to the highest

219   values measured, i.e. in the direction perpendicular to the casting plane, ε_p. Fig. 5 presents the

220   development of ε_p with time of conditioning for sound and ASR affected concrete blocks. The

221   shaded areas in the graphs mark the maximum and minimum expansion values. As noted in Fig. 5,

222   the conditioning period for the first and second series was about 100 and 200 days, respectively.

223   According to the rate of expansion, the ASR reaction seems to be constant during the first 200

224   days of conditioning. The average expansion reached for specimens A1 and A2 was respectively

225   0.20% and 0.33%, that corresponding to a severe and a very severe level of expansion and

226   concrete damage according to Sanchez & al.[45]. For a severe level of expansion, cracks are

227   expected within and around the aggregates, and they are typically connected to other cracks for a

228   very severe level of expansion [45]. It can be seen that specimens type S did not experience

229   significant concrete expansion at 100 and 200 days. These results confirm the efficiency of the

230   added lithium nitrate solution into specimen S concrete mix for the duration of the project. After

231   the first and second series, specimens D1 and D2 were returned to conditioning for an additional

232   100 days. The average expansions before conditioning for the two specimens D1 and D2 were

233   0.21% and 0.32%. After the additional 100 days, the average expansion of specimens D1 and D2

234   was respectively 0.40% and 0.41%, that corresponding to a very severe level of expansion and

235   concrete damage[45].

236   The anisotropy of the reaction can be observed by comparing the expansions in parallel (ε_a) and

237   perpendicular (ε_p) directions to the casting plane (A-B and C-D axis in Fig. 2). As mentioned by

238   Smaoui & al.[46], the concrete expansion is highly related to the casting direction and the vibration

239   during the concrete placement. Fig. 6 compares ε_p and ε_a measured at the day of the test. It can be

240   seen that the expansion in the parallel direction is about 0.6 times the expansion in the

241   perpendicular direction (Eq. (1)). This figure also shows a good correlation between both the

242   expansions ε_p and ε_a (coefficient of determination $R^2 = 0.98$, coefficient of variation CoV = 16%)

243   for ASR reactive specimens and a similar ratio was observed by Smaoui & al.[46].

$$\varepsilon_a = 0.601\varepsilon_p - 0.008 \qquad\qquad (1)$$

245   where ε_p and ε_a are expressed in percentage.

Material Properties

247   The relation between ε_p and the mechanical properties of concrete cores extracted from tested

248   specimens are presented in Fig. 7. One can observe in Fig. 7a a significant decrease of the

249   compressive strength with increasing concrete expansion. By comparing the specimens S1 to A1

250   (approximatively 100 days of conditioning), the compressive strength experienced a decrease of

251   19% (40.5 MPa [5870 psi] to 32.7 MPa [4740 psi] in average) with an expansion (ε_p) increase

252   from 0 to 0.20%. A similar behavior can be observed for the specimens S2 to A2

253   (approximatively 200 days of conditioning) that experienced a decrease of their compressive

254   strength by about 31% (42.7 to 29.3 MPa in average) with an expansion (e_p) increases from 0 to

255   0.33%. Interestingly, for a similar 35 MPa (5000 psi) concrete mix incorporating the same NM

256   aggregate, Sanchez & al.[22] reported a 23% decrease in compressive strength for concrete cylinders

257   having reached an expansion of about 0.20%.

258   Compared to the concrete compressive strength, a less significant decrease of the splitting tensile

259   strength (f_{sp}) is observed in Fig. 7b and Table 4. While the tensile strength decreased between an

260   expansion of 0 and 0.20% (approximatively 100 days of conditioning), no reduction of tensile

261   strength was measured afterward. Comparing the tensile strength of specimens S1 and A1, a

262   reduction of approximatively 21% is found while a loss of 28% was observed looking at

263   specimens S2 and A2 (2.9 to 2.3 MPa [420 psi to 330 psi] and 3.1 to 2.2 MPa [450 psi to 320 psi]

264   respectively). As mentioned, the decrease of the tensile strength seems to have reached a plateau

265   from 0.20% to 0.41% of expansion. The reduction in tensile strength is actually very much related

266   to the type of testing method used [47]. For instance, for a similar 35 MPa (5000 psi) concrete mix

267   incorporating the same NM aggregate, Sanchez & al.[22] reported a 60% decrease in tensile strength

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268 measured on concrete cylinders having reached an expansion of about 0.20% when using a gas

269 pressure tension test proposed by Komar & al.[48].

Stiffness Damage Test

271 As expected, no significant variation of Young's modulus of specimens S (around $\varepsilon_p = 0$) is

272 reported in Fig. 7c and Table 5. However, a significant decrease, of about 43% of Young's

273 modulus, was observed with an expansion ranging from 0.00 to 0.20% (35.2 to 19.2 GPa [5100 to

274 2780 ksi] on average). The decrease was less significant with an expansion progressing from

275 0.20% to 0.33%, and seemed to reach a plateau of about 15 GPa (2180 ksi) afterward. A similar

276 reduction of the Young's modulus (from 30 to 20 GPa [4350 to 2900 ksi]) was observed from 0 to

277 0.20% of expansion for the same concrete mix design tested by Sanchez & al.[22]. By comparing

278 the different parameters in Fig. 7, it can be seen that the Young's modulus results exhibit a smaller

279 scatter than results relative to compressive and tensile strengths for each series. This tends to

280 indicate that Young's modulus seems to be a better indicator of concrete expansion and therefore

281 of concrete damage than the other investigated mechanical properties (f'_c and f_{sp}).

282 Sanchez & al.[22] carried-out SDT tests on twenty (20) different reactive aggregates and obtained

283 SDI values between 0.20 and 0.35 for a ASR expansion of 0.23% to 0.37% and between 0.23 to

284 0.37 for a ASR expansion of 0.30%, which agree well with the results presented in 8d. Indeed, the

285 average SDI of 0.12 and 0.09 was determined for specimens S1 and S2 (about no expansion)

286 respectively, while it was 0.27 and 0.33 for specimens A1 and A2 (average expansion of 0.20%

287 and 0.33%), respectively. Among the tested aggregates, NM coarse aggregates tested by Sanchez

288 & al.[22] in cast concrete cylinder resulted in slightly lower SDI than the one presented in this

289 research. However, they agree well with the ones presented by Allard & al.[40] and obtained from

290 concrete cores extracted into affected concrete thick slab strips containing NM coarse aggregates.

291 Specimens D1 and D2 also presented in Table 4 experienced an average concrete expansion of

292 0.40% and 0.41%, respectively, and an average SDI of 0.31 and 0.37. For comparison, the average

12

293 SDI of specimens A2 (expansion of 0.33%) was 0.33. It therefore appears that increasing the

294 concrete expansion over 0.33% does not significantly affect the SDI of the NM aggregates. That

295 plateau was also observed by Sanchez[29] and Allard & al.[40] and was explained by the formation of

296 alkali-silica gel inside cracks, which mitigates the increase of SDI values with the increase of

297 ASR expansion.

PULLOUT TEST RESULTS AND DISCUSSION

299 The pullout tests results are presented in the following text and figures. A summary of the results

300 as well as measured embedded lengths are presented in Tables 4 and 5.

Failure Mode
302 Typical failure modes obtained for embedded lengths of $2d_b$, $4d_b$ and $5d_b$ are shown in Fig. 8 and

303 are summarized in Table 5. All specimens with a $2d_b$ embedded length failed after bar debonding

304 (failure mode D). Most of the specimens with a $4d_b$ embedded length failed by debonding after the

305 yielding of the bar (failure mode YD) while a few debonded just before yielding of the bar (mode

306 D). Specimens with a $5d_b$ embedded length having ASR reactive concrete also failed according to

307 the YD failure mode while they failed by bar rupture (mode U) for specimens with sound

308 concrete. Fig. 8 shows that some epoxy adhesive remained fixed between the bar ribs and that no

309 significant concrete material remained bonded to the epoxy adhesive (failure modes D and YD).

310 Consequently, the observe failure surface for pullout failure is located between the epoxy and the

311 concrete interface or through the adhesive, mainly along the bar ribs top surface.

Bond-Slip Relationship
313 Fig. 9 shows the bond stress, τ_b, according to the loaded end slip, s, of the post-installed

314 reinforcing bars. Fig. 9a, b and c respectively show the pullout results of specimens with

315 embedded length $2d_b$, $4d_b$ and $5d_b$ from first series and Fig. 9d, e and f, results from the second

316 series. The bond stress was determined by the force induced to the reinforcing bar for a specific

317 embedded length divided by the nominal surface area associated to that embedded length. The

318   embedded lengths presented in Table 5 were used to calculate bond stresses. Pullout energies, E,

319   shown in Table 5 were calculated considering the area under the pullout curves from Fig. 9 for

320   slips ranging from 0 to 9.7 mm (0.382 in), i.e., the spacing between ribs for the reinforcing bars

321   used in this project (pitch).

322   As observed in Fig. 9 for specimens failing by debonding (Table 5), the bars experienced almost

323   no slip and have a large stiffness up to about 20 MPa (2900 psi), and then the bonded bar stiffness

324   decreases until the maximum bond stress is reached. Thereafter, except for specimens S1-5d_b and

325   S2-5d_b experiencing failure mode U (Fig. 9c and f), the bond stress progressively decreases until

326   complete pullout of the bar. Very similar bond-slip response and bond strength were observed for

327   epoxy-bonded bars embedded in sound concrete [5, 34, 49].

328   The average bond strengths of specimens S1, S2, A1 and A2 with an embedded length of 2d_b were

329   respectively 32.1, 33.2, 31.6 and 31.3 MPa (4660, 4790, 4580, 4540 psi) (Table 5). The results

330   were respectively 31.1, 32.2, 29.1 and 30.6 MPa (4510, 4670, 4220 and 4440 psi) for an

331   embedded length of 4d_b and 26.6, 28.0, 25.5 and 25.9 MPa (3860, 4060, 3700 and 3760 psi) for

332   embedded length 5d_b. One can observe that the results between 2d_b and 4d_b specimens were

333   similar and that the ones with embedded length 5d_b experienced lower average bond strengths.

334   These results may be due to a non-uniform stress distribution along the bar for embedded length

335   longer than 4d_b [10]. Reinforcing bars embedded in sound concrete (A1 and A2) experienced

336   slightly higher bond strength and generally higher pullout energy than the ones embedded in

337   concrete affected by ASR (S1 and S2). Based on the pullout energy presented in Table 5, failures

338   of specimens S were generally less brittle (higher pullout energy) than the one of specimens A.

339   Moreover, specimen A1-4d_b-2 exhibited a more brittle (lower pullout energy as shown on Table 5,

340   Fig 8 b) compared to the other tested A1-4d_b specimens. Based on these results, the pullout

341   behavior of specimens with an embedded length of 4d_b embedded in ASR affected concrete seems

342   to be more variable than the one in sound concrete, considering that the specimen might

343   experience a more brittle failure.

344   Fig. 10 shows the relation between the bond strength and the concrete expansion for specimens

345   experiencing Y or YD failure modes from the first and second series (A1, A2, S1 and S2). As

346   previously mentioned, it can be seen that the bond strength slightly decreases with an increase of

347   the concrete expansion (approximatively a 2 MPa decrease in average over the 0 to 0.33%

348   expansion range). It can also be observed that the bond strength of the specimens A1 and A2

349   spreads over a larger range than specimens S1 and S2 with sound concrete. The linear regression

350   between the bond strength and the concrete expansion for specimens experiencing Y or YD

351   failure modes is define by Eq. (2) (Predicted/Test = 1.01 and CoV = 10%).

352 $$\tau_b = -4.83\varepsilon_p + 30.9 \quad [\text{MPa}] \tag{2}$$

353   where ε_p is expressed in percentage and, for customary units (psi), -4.83 and 30.9 have to be

354   replaced by 701 and 4482, respectively. From this equation, the development length of the tested

355   bars (f_y = 456 MPa, 66.1 ksi) is 59 mm (2.32 in) in sound concrete. In ASR affected concrete with

356   ε_p = 0.33%, this development length increased to 62 mm (2.44 in).

357   Fig. 11 shows the bond stress due to the loaded end slip for specimens D1 and D2. As illustrated,

358   the average bond strengths of specimens D1-4d_b and D2-4d_b were respectively 30.3 and 32.0 MPa

359   (4390 and 4640 psi) compared to 29.1 and 30.6 MPa (4220 and 4440 psi) for specimens A1-4d_b

360   and A2-4d_b (Table 5). Despite the progression of ASR following the bar installation on specimens

361   D, their average bond strength was slightly higher than the one of associated specimens A (no

362   progression of ASR after the installation of anchorage in specimens A). Progression of ASR

363   following the bar installation (specimens D) therefore increases the bond strength and requires

364   more pullout energy. According to the expansion results presented in Table 4, this increase could

365   be due to the expansion of the concrete caused by ASR which, in turn, produces a confinement

366   pressure around the bar. It has been reported that confinement pressure applied perpendicularly to

367   the bar axis has a significant effect on the bond strength [19, 49, 50]. More energy is therefore required

368   to perform the bar pullout. However, similarly to the specimen A1-4d_b-2, it can be observed that

369   the specimen D1-4d_b-1 from the third series experienced a more brittle failure (lower pullout

370   energy as shown on Table 4) at bond strength of 27.9 MPa (4050 psi). It is suggested that the

371   scatter of these results may be due to the likelihood of encountering a crack in the embedment

372   length of the bar. The potential presence of cracks in the periphery of the bar reduces the area

373   available for the transfer of forces between the epoxy resin and the concrete matrix, causing a

374   drop in the mechanical capacity of the post-installed reinforcing bar.

375   Fig. 12 shows the relationship between the bond strength and the bar embedded length (h_{eff}) for

376   specimens from the first and the second series failing in to D and YD modes. For specimens of

377   sound concrete, no significant decrease of the bond strength is observed between embedded length

378   2d_b and 4d_b, and a lower bond strength for an embedded length 5d_b. For ASR affected concrete

379   specimens, Fig. 12a shows that the effect of ASR on bond strength becomes significant for

380   embedded lengths greater than 4d_b. One can observe that specimens A1-5d_b and A2-5d_b failed by

381   debonding (mode YD) while specimens S1-5d_b and S2-5d_b experienced bar rupture (mode U).

382   ASR affected concrete to reduce the bond strength of post-installed bonded bars of embedded

383   length 5d_b. These results can be attributed to the higher probability to encounter a crack along a

384   longer post-installed reinforcing bar embedment interface.

385   Fig. 13 presents the relationship between the bond strength of specimens A and S with embedded

386   lengths 2d_b and 4d_b and parameters used to assess ASR damage, i.e., the concrete compressive

387   strength, the Young's modulus and the SDI. As shown in Fig. 13 specimens A experienced a

388   lower compressive strength, a lower Young's modulus and a higher SDI than comparison

389   specimens S. These results are associated with slightly lower bond strength. According to the

16

390　　experimental results, relations between the bond strength and material characterization parameters

391　　can be expressed as: Eq. (3), (4) and (5).

392
$$\tau_b = 0.071 f_c' + 29.3 \quad [MPa] \tag{3}$$

393
$$\tau_b = \frac{E_c}{19500} + 30.5 \quad [MPa] \tag{4}$$

394
$$\tau_b = -5.32 SDI + 33.0 \quad [MPa] \tag{5}$$

395　　For customary units (psi), 29.3, 30.5, 5.32 and 33.0 have to be replaced by 4250, 4420, 770 and

396　　4790, respectively. Fig. 13 shows that the data dispersion expressed by the 95% confidence

397　　interval and the coefficient of variation is narrower for the SDI results. Fig. 13c shows that the

398　　SDI (CoV = 2.9%) seems to be a slightly more accurate parameter than the compressive strength

399　　and the Young's modulus (Fig. 13a and b, both CoV = 3.1%) to assess the bond strength losses

400　　associated with ASR damage.

401　　The relation between the bond strength τ_b and the axial bar stress $f_{s,\ max}$ is presented in Fig. 14 for

402　　specimens from the first and the second series failing according to modes D and YD. For

403　　specimens A and S, the coefficients of determination, R^2, between τ_b and $f_{s,\ max}$ are respectively

404　　0.005 and 0.013. These coefficients reveal no relation between these two parameters. According

405　　to these results, the effect of the transverse bar contraction due to the longitudinal bar tensile strain

406　　has no influence on the bond strength of epoxy bonded bars experiencing a YD failure mode (Fig.

407　　14a). For comparison purposes, this effect plays a major role for cast-in-place embedded bar in

408　　concrete where the bond strength may be reduced up to about 75% after the yielding of the bar [19,

409　　51]. For cast-in-place bars, a large part of the bond strength can be attributed to the bearing action

410　　of the bars ribs on the concrete. Contraction of the bar due to axial tension stress therefore reduces

411　　the bearing area and the bond strength [5, 52]. For post-installed bonded bars, it may be suggested

412　　that the larger flexibility of the epoxy compared to the concrete ($E_b/E_c \sim 20$) may enable the

413   adhesive to follow the steel deformations and insure adequate bearing conditions for the bar ribs.

414   More studies are required to better understand this phenomenon.

415   **CONCLUSIONS**

416   The objectives of this research project were to investigate the effects of ASR deleterious

417   mechanisms on the bond strength of post-installed reinforcing bars and the effects of the

418   damaging ASR progression on bond strength after the embedment of the epoxy bonded bars.

419   According to the material investigation, the main conclusions are:

420   •   Concrete expansion was measured both perpendicularly and parallel to the casting plane

421   during conditioning. A fairly strong anisotropy of the expansion was observed for

422   specimens affected by alkali-silica reaction (ASR) where the expansion in the parallel

423   direction was about 0.6 times the expansion in the perpendicular direction. The measured

424   concrete expansions corresponded to a severe or a very severe level of concrete damage.

425   •   Compressive strength, tensile strength and Young's modulus have decreased with the

426   increasing of the ASR expansion. Both the tensile strength and the Young's modulus reach

427   a plateau for expansions exceeding 0.20%. The Young's modulus was the most affected

428   concrete mechanical property due to ASR.

429   •   The stiffness damage index (SDI) values determined from the SDT were similar to the

430   results presented by Allard & al.[40] for NM coarse aggregate and have demonstrated that

431   the monitored concrete expansions are correlated to the damage state of ASR.

432   According to the investigation of ASR effects on the bonding behavior, the main conclusions are:

433   •   Debonding failures were observed for specimens with $2d_b$ (bar diameter) embedded

434   length. Most of the specimens with $4d_b$ embedded lengths and ASR affected specimens

435   with $5d_b$ embedded lengths experienced debonding failures after yielding of the steel bar.

436   For these specimens, no significant concrete material remained bonded to the pulled out

437   bar. Finally, sound specimens with $5d_b$ embedded lengths experienced bar rupture.

438   - Pullout tests on epoxy bonded bars embedded in sound concrete and ASR affected

439   concrete have shown that ASR has not a large influence on bond strength although a

440   reduction of bond strength was observed for specimens affected by ASR.

441   - It was suggested that the decrease of the bond strength was caused by the presence of

442   longitudinal cracks (parallel to the bar axis) in the periphery of the bar. No significant

443   decrease of the bond strength was observed after the yielding of the epoxy bonded bars.

444   - SDI appeared to be the most accurate parameter to predict the bond strength of ASR

445   affected concrete and reduction of the bond strength prediction according to the SDI may

446   be a valuable avenue.

447   - The study revealed that the ASR progression after the embedment of reinforcing bars leads

448   to a confinement effect on the epoxy bonded bar which increased bond strength.

449   The proposed equations should be validated with more data and tests carried out with different

450   concrete strengths and ASR damage.

451

456

NOTATION

457   **NOTATION**

458   A_b = reinforcing bar cross section area of

459   d_b = reinforcing bar diameter

460   F_R = rib index of reinforcing bar

461   E = pullout energy

462   E_b = Young's modulus of epoxy adhesive

463   E_c = Young's modulus of concrete

464   E_s = Young's modulus of reinforcing bar

465   f_c' = compressive strength of concrete

466   f_{sp} = tensile strength of concrete

467   f_s = axial reinforcing bar stress

468   f_y = yield strength of reinforcing bar

469   f_u = ultimate strength of reinforcing bar

470   h_{eff} = embedded length

471   s = relative displacement between concrete and reinforcing bar at the unloaded end (slip)

472   ε_p = expansion perpendicular to the concrete casting plane

473   ε_a = expansion parallel to the concrete casting plane

474   ε_u = strain at reinforcing bar rupture

475   τ = bond stress

476   τ_b = bond strength

477

REFERENCES

1. Mitchell D., Marchand J., Croteau P., Cook W. D., "Concorde Overpass Collapse: Structural Aspects", *Journal of Performance of Constructed Facilities*, V. 25, No. 6, 2011, pp. 545-553.

2. Cusson B., "Renforcement des dalles épaisses en cisaillement", Québec, Canada: Université Laval; 2012, 143 pp.

3. Provencher P., "Renforcement des dalles épaisses en cisaillement", Québec, Canada: Université Laval; 2010, 130 pp.

4. Fernández Ruiz M., Muttoni A., Kunz J., "Strengthening of Flat Slabs Against Punching Shear Using Post-Installed Shear Reinforcement", *ACI Structural Journal*, V. 107, No. 4, 2010, pp. 434-442.

5. Mahrenholtz C., "Seismic bond model for concrete reinforcement and the application to column-to-foundation connections", Germany: University of Stuttgart; 2012, 417 pp.

6. Walraven J. C., "Aggregate interlock: a theoretical and experimental analysis", Netherlands: Delft University; 1980, 210 pp.

7. Vecchio F. J., Collins M. P., "The modified compression-field theory for reinforced concrete elements subjected to shear", *ACI Journal*, V. 83, No. 2, 1986, pp. 219-231.

8. Lutz L. A., Gergely P., "Mechanics of Bond and Slip of Deformed Bars in Concrete", *ACI Journal Proceedings*, V. 64, No. 11, 1967, pp. 711-722.

9. ACI-408, "Bond and Development of Straight Reinforcing Bars in Tension", USA: American Concrete Institute; 2003, 49 pp.

10. Azizinamini A., Chisala M., Ghosh S., "Tension development length of reinforcing bars embedded in high-strength concrete", *Engineering Structures*, V. 17, No. 7, 1995, pp. 512-522.

11. Darwin D., Graham E. K., "Effect of Deformation Height and Spacing on Bond Strength of Reinforcing bars", Lawrence, Kansas, USA: University of Kansas Center for Research; 1993, 71 pp.

12. Darwin D., Tholen M. L., Idun E. K., Zuo J., "Splice strength of high relative rib area reinforcing bars", University of Kansas Center for Research, Inc.; 1995, 58 pp.

13. Darwin D., Zuo J., Tholen M. L., Idun E. K., "Development length criteria for conventional and high relative rib area reinforcing bars", University of Kansas Center for Research, Inc.; 1995, 70 pp.

14. Eligehausen R., Popov E. P., Bertero V. V., "Local bond stress-slip relationships of deformed bars under

21

506   generalized excitations", United States: University of California at Berkeley; 1983, 169 pp.

507   15.   Tepfers R., "A theory of bond applied to overlapped tensile reinforcement splices for deformed bars",

508   Sweden: Chalmers University of Technology; 1973, 328 pp.

509   16.   Eligehausen R., Mallée R., Silva J. F., "Anchorage in concrete construction", John Wiley & Sons; 2006, pp.

510   17.   Zuo J., Darwin D., "Splice Strength of Conventional and High Relative Rib Area Bars in Normal and High-

511   Strength Concrete", *ACI Structural Journal*, V. 97, No. 4, July-August 2000, pp. 630-641.

512   18.   Mazzarolo E., Scotta R., Berto L., Saetta A., "Long anchorage bond–slip formulation for modeling of rc

513   elements and joints", *Engineering Structures*, V. 34, 2012, pp. 330-341.

514   19.   fib, "fib Model Code for Concrete Structures 2010", Lausanne, Switzerland: Ernst and Sohn; 2013, 653 pp.

515   20.   Lindgård J., Andiç-Çakır Ö., Fernandes I., Rønning T. F., Thomas M. D., "Alkali–silica reactions (ASR):

516   literature review on parameters influencing laboratory performance testing", *Cement and Concrete Research*, V. 42,

517   No. 2, 2012, pp. 223-243.

518   21.   Sims I., Poole A. B., "Alkali-Aggregate Reaction in Concrete: A World Review", CRC Press; 2017, 768 pp.

519   22.   Sanchez L., Fournier B., Jolin M., Mitchell D., Bastien J., "Overall assessment of Alkali-Aggregate Reaction

520   (AAR) in concretes presenting different strengths and incorporating a wide range of reactive aggregate types and

521   natures", *Cement and Concrete Research*, V. 93, 2017, pp. 17-31.

522   23.   Thomas M. D., Fournier B., Folliard K. J., "Alkali-aggregate reactivity (AAR) facts book", FHWA-HIF-13-

523   019, United States: Federal Highway Administration (FHWA); 2013, 212 pp.

524   24.   Hobbs D. W., "Alkali-silica reaction in concrete", Thomas Telford Publishing; 1988, 183 pp.

525   25.   Nixon P., Bollinghaus R., "The effect of alkali aggregate reaction on tensile and compressive strength of

526   concrete", *Durability of building materials*, V. 2, No. 3, 1985, pp. 243-248.

527   26.   Kubo Y., Nakata M., "Effect of reactive aggregate on mechanical properties of concrete affected by alkali-

528   silica reaction", *Proceedings 14th ICAAR - International Conference on Alkali-Aggregate Reaction in Concrete*, May

529   20-25th 2012, Austin (Texas), United States,

530   27.   Giaccio G., Zerbino R., Ponce J., Batic O. R., "Mechanical behavior of concretes damaged by alkali-silica

531   reaction", *Cement and Concrete Research*, V. 38, No. 7, 2008, pp. 993-1004.

532   28.   ISE, "Structural effects of alkali-silica reaction", United Kingdom: The Institution of Structural Engineers;

533   1992, 48 pp.

534   29.   Sanchez L., "Contribution to the assessment of damage in aging concrete infrastructures affected by alkali-

535 aggregate reaction", Québec, Canada: Université Laval; 2014, 377 pp.

536 30.   Villeneuve V., "Détermination de l'endommagement du béton par méthode pétrographique quantitative ",

537 Québec, Canada: Université Laval; 2011, 366 pp.

538 31.   Thomas M. D., Fournier B., Folliard K. J., Ideker J. H., Resendez Y., "The Use of Lithium to Prevent or

539 Mitigate Alkali-Silica Reactions in Concrete Pavements and Structures", FHWA-HRT-06-133, United States: Federal

540 Highway Administration (FHWA); 2006, 54 pp.

541 32.   ASTM-A996/A996M, "Standard Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete

542 Reinforcement", A996/A996M-16, West Conshohocken, PA, USA: ASTM International; 2016, 5 pp.

543 33.   Cook R. A., "Behavior of Chemically Bonded Anchors", *Journal of Structural Engineering*, V. 119, No. 9,

544 1993, pp. 2744-2762.

545 34.   Appl J.-J., "Tragverhalten von Verbunddübeln unter Zugbelastung (Load bearing behaviour of bonded

546 anchors under tension loading)", Stuttgart, Germany: Universität Stuttgart; 2009, 276 pp.

547 35.   Sanchez L. F. M., Fournier B., Jolin M., Bastien J., "Evaluation of the stiffness damage test (SDT) as a tool

548 for assessing damage in concrete due to ASR: Test loading and output responses for concretes incorporating fine or

549 coarse reactive aggregates", *Cement and Concrete Research*, V. 56, 2014, pp. 213-229.

550 36.   ASTM-C1231, "Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of

551 Hardened Cylindrical Concrete Specimens", C1231-14, USA: ASTM International; 2014, 10 pp.

552 37.   ASTM-C496, "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens",

553 ASTM-C496-17, USA: ASTM International; 2017, 10 pp.

554 38.   ASTM-C469, "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in

555 Compression", ASTM-C469-14, USA: ASTM International; 2014, 10 pp.

556 39.   Sanchez L., Fournier B., Jolin M., Bastien J., Mitchell D., "Practical use of the Stiffness Damage Test (SDT)

557 for assessing damage in concrete infrastructure affected by alkali-silica reaction", *Construction and Building*

558 *Materials*, V. 125, 2016, pp. 1178-1188.

559 40.   Allard A., Bilodeau S., Pissot F., Fournier B., Bastien J., Bissonnette B., "Performance evaluation of thick

560 concrete slabs affected by alkali-silica reaction (ASR) : Part 1 : Material aspects", *Proceedings 15th International*

561 *Conference on Alkali-Aggregate Reaction*, 2016, São Paulo, Brazil, pp. 10.

562 41.   A23.1-14/A23.2-14 C., "Concrete materials and methods of concrete construction / Test methods and

563 standard practices for concrete", CSA Group; 2014, 690 pp.

564 42. ASTM-E8, "Standard Test Methods for Tension Testing of Metallic Materials", ASTM-E8/E8M-13, USA:

565 ASTM International; 2013, 28 pp.

566 43. ASTM-E111, "Standard Test Method for Young's Modulus, Tangent Modulus, and Chord Modulus",

567 ASTM-E111-17, USA: ASTM International; 2017, 14 pp.

568 44. Villemure F.-A., "Effet de la réaction alcalis-silice (RAS) sur l'adhérence des ancrages époxydiques de

569 barres d'armature", Québec, Canada: Université Laval; 2018, 202 pp.

570 45. Sanchez L. F. M., Multon S., Sellier A., Cyr M., Fournier B., Jolin M., "Comparative study of a chemo-

571 mechanical modeling for alkali silica reaction (ASR) with experimental evidences", *Construction and Building*

572 *Materials*, V. 72, Dec 2014, pp. 301-315.

573 46. Smaoui N., Bérubé M.-A., Fournier B., Bissonnette B., "Influence of specimen geometry, orientation of

574 casting plane, and mode of concrete consolidation on expansion due to ASR", *Cement, Concrete & Aggregates*, V.

575 26, 2004, pp. 58-70.

576 47. Clayton N., Currie R., Moss R., "Effects of alkali-silica reaction on the strength of prestressed concrete

577 beams", *The Structural Engineer*, V. 68, No. 15, 1990, pp. 287-292.

578 48. Komar A., Hartell J., Boyd A., "Pressure tension test: reliability for assessing concrete deterioration",

579 *Proceedings Seventh International Conference on Concrete under Severe Conditions - Environment and Loading*,

580 2013, Nanjing, China,

581 49. Lowes L. N., Moehle J. P., Govindjee S., "Concrete-Steel Bond Model for Use in Finite Element Modeling

582 of Reinforced Concrete Structures", *ACI Structural Journal*, V. 101, No. 4, July-Aug 2004, pp. 501-511.

583 50. ACI Committee 408, "Bond and Development of Straight Reinforcing Bars in Tension", United States:

584 American Concrete Institute; 2003, pp.

585 51. Shima H., Chou L.-L., Okamura H., "Bond Characteristics in Post-Yield Range of Deformed Bars", *Doboku*

586 *Gakkai Ronbunshu*, V. 1987, No. 378, 1987, pp. 213-220.

587 52. Brantschen F., Faria D. M., Fernández Ruiz M., Muttoni A., "Bond behaviour of straight, hooked, U-shaped

588 and headed bars in cracked concrete", *Structural Concrete*, V. 17, No. 5, 2016, pp. 799-810.

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618

TABLES

620 *Table 1 – Petrographic facies and potentially reactive phases of New Mexico aggregate from*
621 *Villeneuve (2011)*

Facies	Type	Proportion [%] (-14 + 10 mm [-0.55 + 0.39 in])	Proportion [%] (-20 + 14 mm [-0.78 + 0.55 in])	Potentially reactive phases
A	Andesite-basalt	19.0	36.1	Volcanic glass
B	Quartzite	56.7	35.4	Microcrystalline quartz (10%)
C	Granite	18.5	20.1	Microcrystalline quartz (5%) and quartz with undulatory extinction
D	Granitic gneiss	2.8	3.3	Quartz with undulatory extinction
E	Rhyolite	2.6	2.0	Microcrystalline quartz (~20%)
F	Pelite	0.4	3.1	-

622

623 *Table 2 – Concrete mixes design (kg/m³)*

Components	ASR reactive concrete	Sound concrete
Cement	370.0	370.0
Fine aggregate	742.6	742.6
Coarse aggregate	1091.8	1091.8
NaOH	0.7	-
Lithium nitrate ($LiNO_3$)	-	28.3
Water	174.0	174.0
Density	2368	2376

624 Note: 1 kg/m³ = 1.686 lb/yd³

625

626 *Table 3 – Properties of the epoxy resin according the manufacturer*

Bond strength (ASTM C882-91)	12.4 MPa (1800 psi)
Compressive strength (ASTM D-695-96)	82.7 MPa (12000 psi)
Compressive modulus (ASTM D-695-96)	1493 MPa (220 ksi)
Tensile strength (ASTM D-638-97)	43.5 MPa (6310 psi)
Elongation at break (ASTM D-638-97)	2.0%

627

628   Table 4 – Expansion and mechanical properties summary

		Specimens	ε_p [%]	ε_a [%]	f_c' [MPa]	f_{sp} [MPa]	E_c [GPa]	*SDI*
First series		S1	-0.013	-0.017	40.5	2.9	35.3	0.12
		A1	0.198	0.111	32.7	2.3	19.1	0.27
Second series		S2	0.015	-0.004	42.7	3.1	35.1	0.10
		A2	0.327	0.185	29.3	2.2	16.0	0.33
Third series		D1	0.402	0.225	28.6	2.3	16.0	0.31
		D2	0.406	0.255	24.7	2.6	14.4	0.37

629   Note: 1 MPa = 0.001 GPa = 145.038 psi

630

631 *Table 5 – Pullout test results summary*

Specimen	Failure Mode	h_{eff} [mm]	$f_{s,max}$ [MPa]	τ_b [MPa]		E [J]	
S1-2d_b-1	D	32.7	236	28.6	32.1	348	394
S1-2d_b-2	D	32.4	261	32.1		379	
S1-2d_b-3	D	34.0	287	33.6		437	
S1-2d_b-4	D	30.9	263	34.0		410	
S1-4d_b-1	YD	62.3	473	30.2	31.1	480	496
S1-4d_b-2	YD	61.0	485	31.7		465	
S1-4d_b-3	YD	67.0	540	32.1		544	
S1-4d_b-4	YD	67.7	519	30.5		-	
S1-5d_b-1	U	84.0	553	26.9	26.6	-	-
S1-5d_b-2	U	86.2	582	26.3		-	
A1-2d_b-1	D	32.6	260	31.7	31.6	230	344
A1-2d_b-2	D	32.4	254	31.3		287	
A1-2d_b-3	D	34.8	276	31.6		426	
A1-2d_b-4	D	33.4	268	31.9		434	
A1-4d_b-1	YD	68.9	527	30.6	29.1	485	427
A1-4d_b-2	D	62.0	362	23.2		296	
A1-4d_b-3	D	53.2	449	33.6		501	
A1-5d_b-1	YD	87.4	564	25.7	25.5	441	432
A1-5d_b-2	YD	86.2	559	25.8		434	
A1-5d_b-3	YD	82.0	512	25.0		421	
S2-2d_b-1	D	38.1	338	35.4	33.2	399	417
S2-2d_b-2	D	35.1	286	32.4		395	
S2-2d_b-3	D	32.6	266	32.5		420	
S2-2d_b-4	D	32.9	268	32.4		455	
S2-4d_b-1	YD	67.3	545	32.2	32.2	577	539
S2-4d_b-2	YD	63.9	541	33.7		539	
S2-4d_b-3	YD	63.5	519	32.5		540	
S2-4d_b-4	YD	61.4	468	30.3		499	
S2-5d_b-1	U	80.1	565	28.1	28.0	-	-
S2-5d_b-2	U	80.8	565	27.8		-	
A2-2d_b-1	D	31.8	252	31.8	31.3	360	366
A2-2d_b-2	D	31.5	253	32.1		291	
A2-2d_b-3	D	34.1	274	32.1		402	
A2-2d_b-4	D	32.6	284	29.3		412	
A2-4d_b-1	YD	62.4	476	30.4	30.6	423	435
A2-4d_b-2	D	52.6	401	30.3		421	
A2-4d_b-3	D	50.9	396	31.1		461	
A2-5d_b-1	YD	81.4	533	26.0	25.9	441	462
A2-5d_b-2	YD	82.4	535	25.8		482	
D1-4d_b-1	D	65.3	457	27.9	30.3	363	460
D1-4d_b-2	YD	65.3	537	32.7		557	
D2-4d_b-1	YD	65.6	524	32.0	32.0	573	555
D2-4d_b-2	YD	66.1	526	31.9		537	

632 Note: *anchor in a block tested twice, 1 mm = 0.03937 in, 1 MPa = 145.038 psi

Fig. 1

(a)

(b)

Reinforcing bar
Epoxy adhesive

(c)

Shear crack

Fig. 2

Pour

Concrete casting plane

A-B: Perpendicular to the casting plane (ε_p)
C-D: Parallel to the casting plane (ε_a)
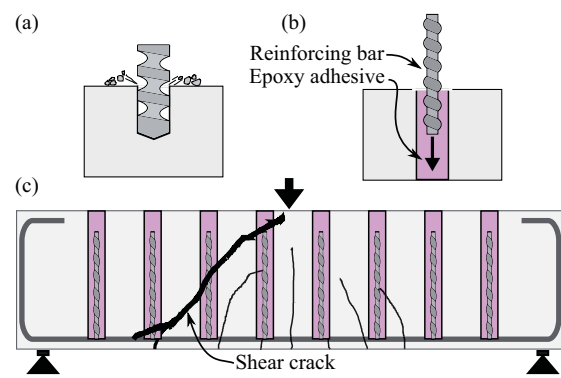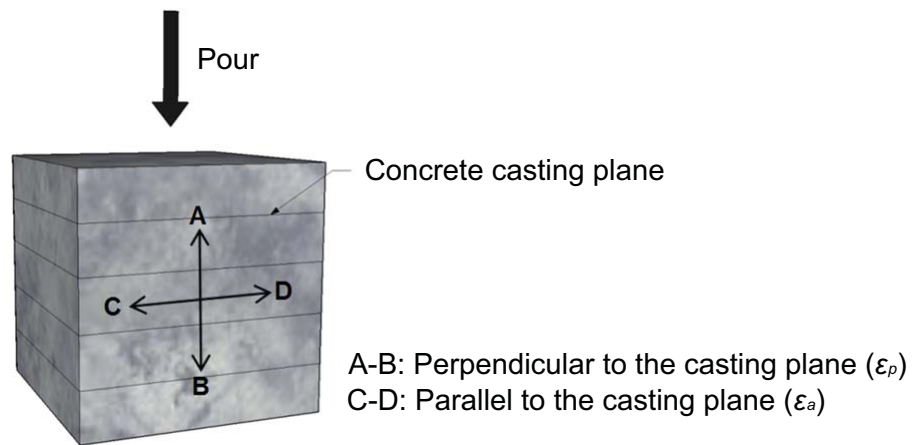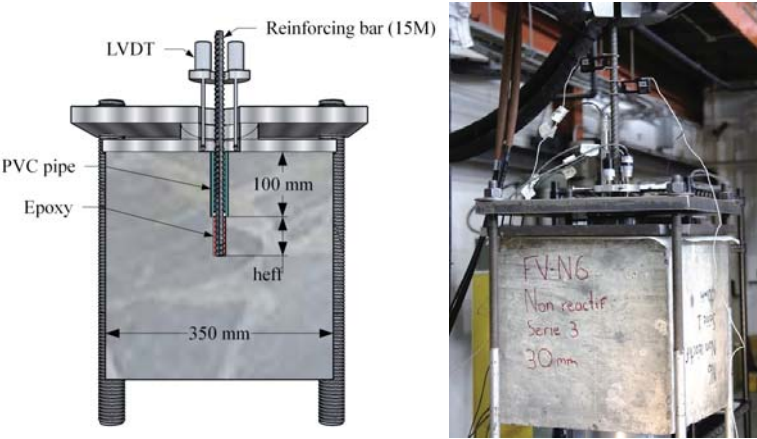
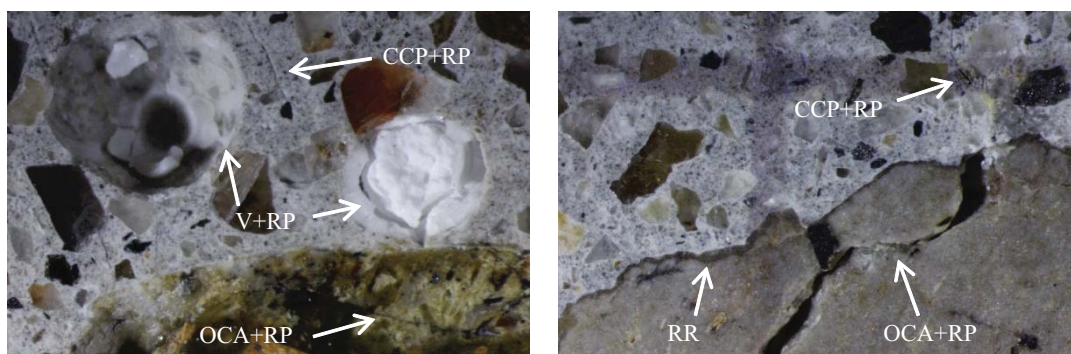Fig. 3

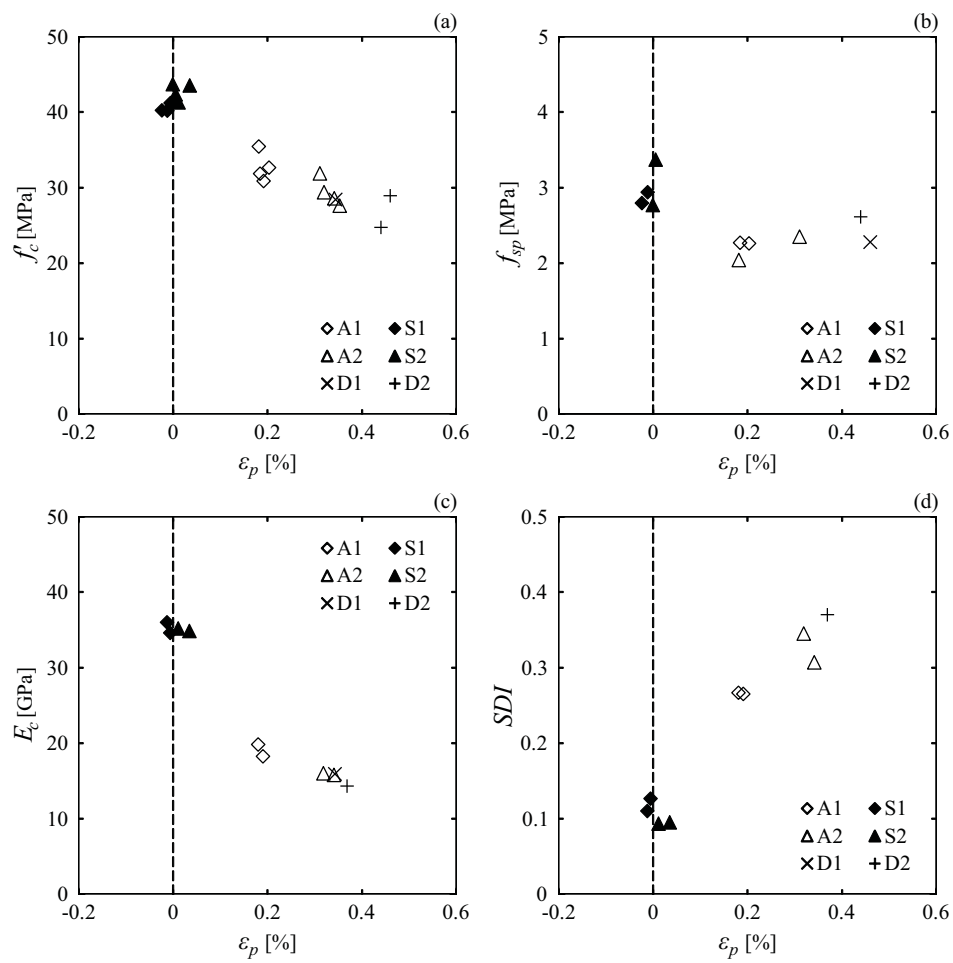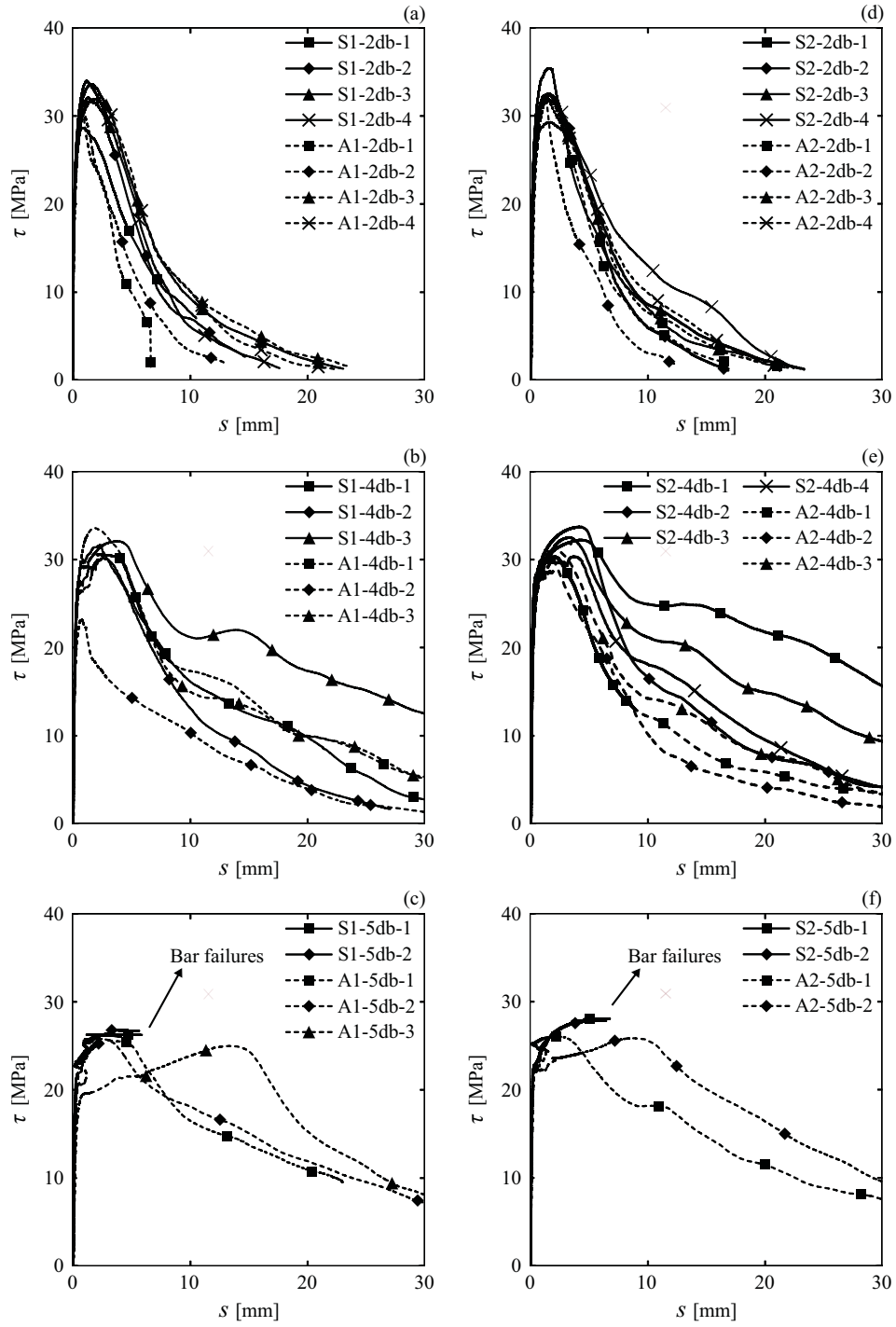Labels in figure: Reinforcing bar (15M), LVDT, PVC pipe, Epoxy, 100 mm, heff, 350 mm
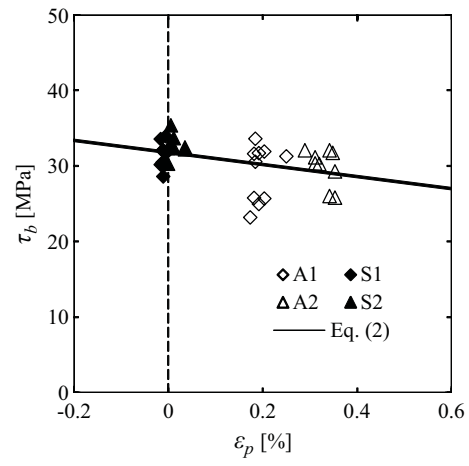
Fig. 4

Fig. 5

Fig. 6

Fig. 7

Fig. 8

D

YD

U

Fig. 9

Fig. 10

Fig. 11

Fig. 12

Fig. 13

Fig. 14