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Modelling and parametric study of the re-anchorage of ruptured tendons in bonded post-tensioned concrete
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Abstract:	The contribution of ruptured tendons to the residual strength of bonded post-tensioned concrete structures is currently assessed based on pre-tensioned concrete bond models. However, this approach is inaccurate due to the inherent differences between pre-tensioned and post-tensioned concrete. In this paper, a non-linear 3D finite element model is developed for the re-anchoring of a ruptured tendon in post-tensioned concrete. The model is validated using full-field displacement measurement from 33 post-tensioned concrete prisms and previous experimental data on beams from the literature. The influence of different parameters was investigated, including tendon properties (i.e. diameter, roughness), duct properties (i.e. diameter, thickness, material), initial prestress, concrete strength, grout strength, grout voids, stirrups, and strands, on the tendon re-anchorage. The most influential parameters are found to be tendon and duct properties.
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Papers published in ASCE Journals must make a contribution to the core body of knowledge and to the advancement of the field. Authors must consider how their new knowledge and/or innovations add value to the state of the art and/or state of the practice. Please outline the specific contributions of this research in the comments box.	A 3D finite element model to simulate the re-anchorage of a ruptured tendon was developed New test setup using ESPI full-field displacement measurement after tendons rupture was used in the model validation Both the model and experiments confirmed the re-anchorage of ruptured tendons A parametric study revealed that tendon and duct properties are the most parameters influencing tendon re-anchorage
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1 # Modelling and parametric study of the re-anchorage of ruptured tendons

2 # in bonded post-tensioned concrete

3 A. O. Abdelatif[1] , J. S. Owen[2] and M. F. M. Hussein[3]

4 **Abstract**

5 The contribution of ruptured tendons to the residual strength of bonded post-tensioned

6 concrete structures is currently assessed based on pre-tensioned concrete bond models.

7 However, this approach is inaccurate due to the inherent differences between pre-tensioned

8 and post-tensioned concrete. In this paper, a non-linear 3D finite element model is developed

9 for the re-anchoring of a ruptured tendon in post-tensioned concrete. The model is validated

10 using full-field displacement measurement from 33 post-tensioned concrete prisms and

11 previous experimental data on beams from the literature. The influence of different

12 parameters was investigated, including tendon properties (i.e. diameter, roughness), duct

13 properties (i.e. diameter, thickness, material), initial prestress, concrete strength, grout

14 strength, grout voids, stirrups, and strands, on the tendon re-anchorage. The most influential

15 parameters are found to be tendon and duct properties.

16 **Keywords**

17 Post-tensioned concrete; Re-anchorage; Corrosion; Tendon; Bond, Modelling; Finite element;

18 Rupture; ESPI

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1. Introduction

Bonded post-tensioned concrete had been considered as a durable form of construction because of the multi-layered protection for the prestressing steel. However, many post-tensioned concrete bridges have been reported to have suffered ruptured tendons due to corrosion, which led to total structural collapse in some extreme cases (Concrete Society, 2002, Highways Agency et al., 1999, NCHRP, 1998). Previous investigations showed that a ruptured tendon is able to re-anchor into the surrounding grout, which is mainly designed as a corrosion protection, and, as a result, contributes to the residual structural capacity of the structure (Highways Agency, 1995, Buchner and Lindsell, 1987b). While much effort has been devoted to developing corrosion detection techniques, little attention has been given to assessing the structural capacity of bonded post-tensioned concrete structures with ruptured tendons.

Approaches to the structural assessment of damaged post-tensioned bridges often utilise pre-tensioned models or empirical bond slip relations to approximate the re-anchorage length (Highways Agency et al., 1999, Cavell and Waldron, 2001, Coronelli et al., 2009). In some cases, the re- anchorage of tendons is completely ignored (Jeyasehar and Sumangala, 2006, Zeng et al., 2010, Watanabe et al., 2011). However, this is not appropriate. Wrong estimation of the re-anchoring phenomenon for a ruptured tendon can result in inaccurate predictions of the residual structural capacity. Accurate prediction enhances the process of decision making and helps in decreasing the maintenance cost. Equally important, it minimises disruption and avoids unnecessary replacement or strengthening work. Therefore, this paper aims to develop a 3D non-linear finite element (FE) model to simulate the re-anchorage phenomenon of the ruptured tendon in bonded post-tensioned concrete. The model will then be used in a parametric study to investigate the influence of different parameters. This will help improve

43   the understanding of the behaviour of bonded post-tensioned concrete structures with

44   ruptured tendons and facilitate the prediction of their residual structural capacity.

45   This paper presents the background of bond in post-tensioned concrete in Section 2. Section 3

46   presents a 3D non-linear FE model for the re-anchorage of a ruptured tendon. Section 4

47   shows the experimental procedure used in the validation of model, which is used to validate

48   the numerical model in Section 5. Section 6 presents a parametric study, which was

49   performed using the developed FE model. Conclusions are summarised in Section 7.

50   ## 2.   Bond in grouted post-tensioned concrete

51   Bond between steel and grout is attributed to adhesion between steel and grout, friction

52   between steel and grout, and the mechanical resistance. The adhesion always has an

53   insignificant influence on the load-deformation response of the structure, because the

54   adhesion fails after a very small relative slip (Marti et al., 2008, Cairns et al., 2007). The

55   mechanical resistance only contributes to bond when deformed steel bars are used. However,

56   when smooth strands are used, a strand slips through the grout following the pre-shaped

57   groves without shearing off the concrete (fib, 2000, Abdelatif et al., 2015). Therefore, the

58   friction between steel and grout is largely responsible for the transfer of stress into the

59   surrounding material. In the following paragraphs, the literature on bond in grouted post-

60   tensioned concrete is chronologically presented.

61   In 1960s, experiments on 19 beams were conducted to investigate the effect of grout

62   properties on the re-anchorage length, indicated as "transmission length", and the structural

63   behaviour of bonded post-tensioned concrete beams (Geddes and Soroka, 1963, Geddes and

64   Soroka, 1964). The investigations showed that the "transmission length" is independent of

65   time, but depends on the compressive strength of the grout.

66   In controlled demolition tests in the eighties, important results about structures suffering from

67   rupture of tendons were revealed (Buchner and Lindsell, 1987b, Buchner and Lindsell, 1988).

68   These results were: 1) the ruptured tendon is able to re-anchor into surrounding material over

69   a certain length known as the *re-anchorage length*; 2) the re-anchorage depends on the grout

70   condition, friction between individual wires or strand within a tendon, and the level of

71   confinement provided by shear links.

72   The Model code 1990 (MC90) introduced a model for bond in post-tensioned concrete. It is

73   based on bond between concrete and the outer surface of tendon sheathing (CEB-FIP MC90,

74   1990). In reality, however, the bond actually develops between pre-stressing steel and grout

75   rather than between pre-stressing steel and concrete. This model for bond in post-tensioned

76   concrete was removed in the latest version of the model code MC2010, (fib, 2010).

77   The UK Highways Agency, in DMRB BA51/95, has proposed a conservative empirical

78   relationship to estimate the re-anchorage length of a ruptured bonded post-tensing tendon

79   (Highways Agency, 1995). It modified the BS5400 transfer length model (BS 5400, 1990) for

80   *pre-tensioned* concrete to account for multi-strand tendons. The model assumes linear

81   distribution of tendon stresses over the re-anchorage length.

82   In 2001, FE and experimental investigations were conducted to study the dynamic bond

83   characteristics of smooth pre-stressing bars that are embedded in grout (Belhadj and Bahai,

84   2001). The study demonstrated the importance of friction in controlling the slip of pre-

85   stressing steel in grout. In another study in 2008, the influence of emulsfiable oil, which is

86   used as a temporary corrosion protection, in bond behaviour in bonded post-tensioned

87   concrete was investigated. The results showed a reduction on bond shear stress for treated

88   strands compared to untreated ones (Marti et al., 2008, Luthi et al., 2008).

89   An empirical relation was proposed in 2011 to calculate the re-anchorage length, which is

90   referred to as "the size of the stress-decreasing region", based on experiments on nine beams

91   subjected to tendon cutting (Watanabe et al., 2011). The proposed relation assumed constant

92   bond stress over the re-anchorage length, which gives a linear stress distribution. This

93   assumption contradicts with the findings of other experimental observations (Geddes and

94   Soroka, 1964, Coronelli et al., 2009).

95   Most of the reviewed literature that addresses the bond of post-tensioned tendons aims to

96   study the impact of certain parameters on the bond mechanism rather than developing a

97   model for the re-anchorage of the tendon. Therefore, in 2012, the authors developed an

98   analytical model based on the linear thick-wall cylinder theory and the Coulomb friction law

99   to estimate the stress distribution in the tendon after the rupture and hence the re-anchorage

100 length (Abdelatif et al., 2012), Eq. (1).

$$x = \frac{r_s}{2\alpha\phi}\left[\left(\frac{1}{B} + \frac{\mu_s}{E_s}\frac{1}{B^2}\right)\ln\left(1 + \frac{B}{A}f_s\right) - \left(\frac{1-\mu_s}{E_s} + \frac{\mu_s}{BE_s}\right)f_s\right] \tag{1}$$

101 Here f_s is tendon stress at distance x from the rupture point, α is a factor to account for voids in

102 the grout, μ_s and E_s are the Poisson's ratio and the Young's modulus of steel. A and B are

103 coefficients depending on geometry and material properties of steel, concrete, grout and duct

104 that can be calculated as shown in (Abdelatif et al., 2012). Eq. (1) estimates the re-anchorage

105 length when the stress in the pre-stressing steel (f_s) is substituted by the effective pre-stress (f_{se}).

106 The model has been verified using an axisymmetric FE model and validated experimentally

107 (Abdelatif et al., 2013). The results of axisymmetric and analytical model show that the stress

108 on the grout might exceed the tensile strength. Therefore, non-linear material behaviour

109 should be considered.

3.   3D non-linear FE modelling of the re-anchorage of ruptured tendons

In this paper, an FE package ABAQUS (Dassault Systemes Simulia, 2010) is used to model

post-tensioned concrete members with a single wire. The modelling of the post-tensioned

concrete components (i.e. concrete, grout, steel and duct), simulation of post-tensioning,

tendon rupture, and the solution algorithms are discussed in this section. All post-tensioned

concrete components are modelled using 8-node isoparametric elements with reduced

integration points to minimise the cost of computation. The reduced integration method

improves the computational efficiency without losing the accuracy of the results (Koh and

Kikuchi, 1987).

3.1   Modelling concrete, grout, tendon and duct elements

3.1.1 Concrete

The magnitude of the radial stresses (compression) in concrete is always less than the

magnitude of the circumferential stresses (tension) as shown in Eq. (2), based on thick-wall

cylinder theory (Timoshenko et al., 1974).

$$\sigma_r = \sigma_\theta - r \frac{\partial \sigma_r}{\partial r} \tag{2}$$

This suggests that the compressive radial stress (σ_r) will be less than the magnitude of

concrete tensile strength (σ_θ). Therefore, the behaviour of concrete in compression was

modelled as linear-elastic.

In tension, the behaviour of concrete is modelled as linear-elastic up to its tensile strength (f_t)

which is taken here as a tenth of the concrete compressive strength. The post-cracking was

modelled using the Concrete damage plasticity (CDP) model in ABAQUS (Dassault

Systemes Simulia, 2010) with Hillerborg's fracture energy cracking concept (Hillerborg et

6

131   al., 1976) and assuming linear tension softening. More details about the CDP parameters used

132   are discussed elsewhere (Abdelatif et al., 2015).

3.1.2   Grout

134   The grout material was modelled in a similar way as the concrete elements. According to

135   EN447, a water cement ratio w/c of 0.4 is recommended for grout in prestressing tendons.

136   For this w/c, the values of fracture energy, G_f, were found to be in the range of 22-26 N/mm

137   (Padevĕt and Zobal, 2011). In this study the averaged value of 24 N/mm was used.

3.1.3 Prestresing steel, duct, and anchor plates

139   Post-tensioning tendons are usually stressed to a level below the yield stress. Therefore,

140   tendon, steel duct, and anchor plates are modelled using linear elastic material model

141   adopting the idealised stress-strain constitutive model in both tension and compression (CEB-

142   FIP MC90, 1990). In this study, the Young's modulus for steel material was assumed to be

143   200 GPa.

144   As noted in section 2, friction is the principal means of transferring stress from the tendon

145   into the surrounding material for both smooth strands and wires. Therefore only the frictional

146   component was considered in modelling bond in this work. The pressure in the frictional

147   component is generated by the radial expansion of the tendon after rupture as a result of the

148   Poisson effect, Abdelatif et al., 2015. In this work, only tendons made from a single wire are

149   considered to avoid the meshing problems that would occur for a helical strand. For a given

150   change in pre-stress the Poisson effect will give the same change in diameter for both strand

151   and wire. However, for the strand there is an additional effect that resulting from the

152   mechanical interlocking of the wires making up the strand. Therefore, modelling the tendon

153   as a wire will provide a lower bound on the bond and hence a conservative estimate of the re-

154   anchorage length.

3.2   Simulation of post-tensioning process

155

156   To simulate the interface between steel-grout, grout-duct, and duct-concrete surface-to-

157   surface contact elements were used. The tangential behaviour between steel and grout was

158   modelled using Coulomb friction law with zero cohesion and 0.4 coefficient of friction

159   (Abdelatif et al., 2012, Abdelatif et al., 2015, fib, 2000). Note that the type of corrosion

160   usually found in post tensioned structures is the pitting corrosion, which is localised and

161   occurs in grout voids due to the presence of chlorides and moisture. Therefore, in this study,

162   the coefficient of friction between steel and concrete close to the tendon fracture was

163   assumed to be unchanged by corrosion. Other contact interfaces (i.e. grout-duct, duct-

164   concrete and end anchors-concrete) were considered to be fully bonded.

165   The slip between two paired nodes at contacted interfaces is considered to take place when

166   the tangential friction exceed the static friction and the relative tangential displacement

167   exceeds the specified tolerance. In ABAQUS by default, the tolerance set to 0.5% of the

168   average length of all contact elements in the model (Dassault Systemes Simulia, 2010).

169   The normal and tangential contact behaviours were solved using an augmented Lagrange

170   multiplier algorithm and Penalty method, respectively.

171   The model consists of three main solution steps: prestressing, grouting, and rupture.

172   1) The prestressing was modelled by applying an initial stress equivalent to the

173   magnitude of prestress on steel elements. The model was then solved in this first step

174   to allow transfer of prestress from steel to concrete through the end anchorage. In this

175   step, the grout elements, contact between post-tensioning steel and grout, and contact

176   between grout and duct are deactivated (no stress on grout in this step).

177   2) The grouting process was then simulated in the second solution step by activating the

178   grout elements and grout's contact elements with steel and the duct. Note that the

179   grout element remain un-stressed throughout this step. This is because the contact

180   elements can be stablished between any two paired of nodes if they are within the

181   predefined tolerance without displacing the nodes.

182   3) In the third step the tendon rupture was simulated. The rupture of post-tensioning steel

183   was modelled by deactivation steel elements at the location of the rupture, and then

184   the model is solved for equilibrium. This simplification is supported by the fact that

185   the corrosion causing rupture is localised and occurs in the form of pitting. With more

186   localisation and high rate of corrosion, the pitting mechanism may produce points of

187   stress concentration. Eventually this may lead to a sudden brittle failure after a

188   negligible loss of material.

189 The model was solved utilising a full Newton-Raphson solver under the static condition, with

190 un-symmetric matrix storage to handle the contact solution. A small time step size, and

191 therefore, a large number of increments were used to promote the convergence of the

192 complex non-linear material behaviour and the solution of the contact problem.

193 **3.3   Mesh sensitivity**

194 The mesh sensitivity study was conducted for a bonded post-tensioned concrete prism (Fig.

195 1) in order to find an optimum spatial discretization. The study shows that, 32 segments

196 around the tendons and 10 mm element size in the longitudinal direction satisfied the mesh

197 sensitivity investigations, Fig. 2. The influence on the model result for elements smaller than

198 this was found to be insignificant (Fig. 3).

199 **4.   Experimental work**

200 A number of laboratory investigations have previously been carried out to study the bond

201 behaviour between grout and the post-tensioning tendons, as well as the quality performance

202 of the grout (Marti et al., 2008, Minh et al., 2007, Minh et al., 2008). However, only limited

203   experiments were conducted to understand the re-anchoring phenomenon of ruptured

204   tendons.

205   In this section, a brief review of previous tests about measuring techniques, rupture

206   simulation, and the size of test specimen will be given.

207   In previous studies on monitoring strain changes after tendon rupture/release on post-

208   tensioned concrete, two types of strain gauges were used: the demountable

209   mechanical (DEMEC) and the electrical gauges. The DEMEC strain gauges were used to

210   investigate the effect of grout properties on transmission length on nineteen bonded post-

211   tensioned concrete beams (Geddes and Soroka, 1964). The electrical strain gauges were

212   usually attached on the concrete surface to (Schupack and Johnston, 1974, Buchner and

213   Lindsell, 1987a, Buchner and Lindsell, 1988) or attached to the pre-stressing steel (Coronelli

214   et al., 2009, Watanabe et al., 2011). However, the tendon slippage after the rupture might

215   break the attached electrical strain gauges (Coronelli et al., 2009).

216   Because the simulation of the corrosion process is time demanding (Jeyasehar and

217   Sumangala, 2006), the corrosion rupture was simulated by releasing the prestress force at the

218   location of rupture. This can be done either by using nuts to simulate the rupture at the end

219   (Geddes and Soroka, 1964) or by cutting the tendon through a prefabricated hole using a

220   flame torch (Schupack and Johnston, 1974) or saw (Coronelli et al., 2009, Watanabe et al.,

221   2011).

222   Different sizes of specimens were used in previous experimental investigations. Experiments

223   were conducted on real bridge girders (Buchner and Lindsell, 1987a, Buchner and Lindsell,

224   1988), laboratory beams with large cross section (Schupack and Johnston, 1974) and long

225   spans (~10 m) (Tanaka Y. et al., 2001), and on 3-5 metre beams (Coronelli et al., 2009,

226   Geddes and Soroka, 1964, Watanabe et al., 2011).

227   The drawbacks of the previous experiments on the behaviour of the ruptured tendon can be

228   summarized in three points:

229   • The use of electrical resistance strain gauges or DEMEC gauges restricts the number of

230   the measuring points based on the specimen length. This will influence the prediction

231   of strain profile along the tendon

232   • Simulating tendon rupture by saw cutting might disturb and damage the grout.

233   • Conducting experiments on large beams or in a real bridge beams restricts the number

234   of parameters to be varied for practical and cost issues.

235   These drawbacks can be resolved by measuring the full-field displacement (instead of strain

236   gauges) after the rupture on small prisms (instead of beams).

237   In this study, the 3D Electronic Speckle Pattern Interferometry (ESPI) system was utilised to

238   measure the full field displacement on 500 mm long post-tensioned concrete prisms. ESPI

239   has proven to be a valuable alternative to conventional displacement measuring techniques

240   (Jones and Wykes, 1989). It provides considerably more information, such as full-field

241   measurement, compared to the conventional method. The rupture of the tendon was simulated

242   by undoing nuts of a special type of anchor (instead of using saw cutting or accelerated

243   corrosion), which was designed for the purpose of the tests (Fig. 1). Undoing nuts also gives

244   a sufficient number of steps/images to be capture by the ESPI system throughout the test.

245   **4.1   Test setup**

246   33 Concrete prisms of 500x100x100 mm with an embedded duct along the centre were cast

247   and then post-tensioned using a single pre-stressing wire (Fig. 1)**.** The wire was tensioned

248   using a manual hydraulic prestressing jack. The prestressing load was controlled by means of

249   a load cell and adjusted with a bespoke extended anchor system before grouting. The prisms

250   were grouted vertically from the anchor side using a manual pointing gun.

251 As illustrated in Fig. 4, the post-tensioned concrete prism was then bolted down to a vibration

252 isolation table and two rounded bars were inserted below the prism to minimize expected

253 downward movement during the test (because of the sensitivity of the ESPI system to rigid

254 body motion). Two different support setups were used in this test to investigate the influence

255 of the support position on the tendon re-anchorage. In the first one, the prism was bolted

256 down from the releasing end (live end), and in the second, the prism was fixed at the other

257 end (dead end). This support boundary condition (bolting-down) was represented in the FE

258 model by fixing the nodes at the bottom of the end-plate.

259 The rupture of the tendon at the end of the prism (i.e. anchor) was simulated by gradually

260 releasing the nuts between the extended anchorage plates (Fig. 1) in many steps. A load cell

261 was used to monitor the level of prestress during releasing steps and to record the tendon

262 force before and after the test to eliminate the prestress losses. The full-field displacement at

263 the concrete surface after prestress release was measured using ESPI, Fig. 4.

264 One of the intrinsic limitations to be avoided in ESPI measurements is the rigid body motion

265 (RBM). The presence of rigid body motion (i.e. movement of the sample) locates the speckle

266 pattern fringes away from the surface of the tested object (Jones and Wykes, 1989).

267 Therefore, fringes can become unrelated to the surface displacement. This is a major

268 disadvantage of the ESPI system, which limits its application. The correction of such errors

269 might be difficult or impossible to account for and it is important to minimise this error by a

270 suitable design of the testing setup. In this study, two LVDTs were used to monitor the

271 prisms' rigid body motion. One third of the prisms used in this study were excluded because

272 of the presence of the rigid body motion.

5. Model validation

The validation was conducted by comparing the results from the FE model with full field displacement at the concrete surface of post-tensioned concrete prisms. Many parameters were varied to examine the 3D FE model, these include: tendon diameter, duct material, prestressing force, concrete strength, grout strength, reinforcement and support setup (Table 1). To increase the range of validation, previous experimental results on beams were also used to validate the FE model. The FE model stress profile was also compared to that from the analytical model proposed by the authors, Equation (1).

5.1 Model validation against ESPI data

The full-field displacement that was obtained from the ESPI test was compared with the 3D non-linear FE model displacement contour map in the longitudinal direction (Fig. 5 and Fig. 6). The comparison shows a reliable simulation of 3D non-linear model for re-anchoring behaviour.

From the contour maps, the displacement profile at the level of the tendon can be extracted for each prism (Fig. 7 and Fig. 8). In order to accommodate the numerical model (Abdelatif et al., 2012) in the comparison, all results are normalised to the maximum displacement. Based on the impact of the rigid body motion on the test results, the results for negligible rigid body motion (c≤$3\mu m$) are given in Fig. 7 while those with small rigid motion (3μm ≤RBM≤10μm), are presented in Fig. 8. All the results for specimens with excessive rigid body motion group ((RBM≥10μm) were excluded for the reasons discussed in Section 4.1. From observations, it was found that the prisms which were fixed from the far end (i.e. dead end) exhibit less rigid body motion compared with those which were fixed from the releasing end (i.e. live end).

296   Both of the analytical and 3D non-linear FE re-anchorage models show satisfactory

297   prediction of the real behaviour (Fig. 5 to Fig. 8). In a few cases, the agreement at the end the

298   prism is poor; this can be due to the fact that the intensity of the laser at the end is quite low.

299   This reason limits the use of ESPI system with large specimens (Jones and Wykes, 1989).

5.2   Model validation using data from previous tests on beams

301   In addition to the experimental work done as a part of this study, data from previous

302   experiments on post-tensioned concrete beams were used to validate the proposed model

303   (Geddes and Soroka, 1964). The beams had dimensions of 5 inch (127 mm) width by 6 inch

304   (152.5 mm) height and were post-tensioned using a single 7/8 inch (22.225 mm) post-

305   tensioning bar which was enclosed by 45mm/49mm steel duct filled with grout. The re-

306   anchorage length was estimated at 95% of the average maximum strain (95% AMS) method

307   (Russell and Burns, 1993, Abdelatif et al., 2012).

308   Fig. 9 shows normalised strain profile of the experimental data, 3D non-linear FE model, linear

309   theoretical model, and Highways Agency model (Highways Agency, 1995). All strain data

310   were normalised to average maximum strain. Good agreement between the proposed models

311   (FE and analytical) with the experimental data are observed. The re-anchorage length is over

312   predicted as 29% and 5% of the experimental value using the analytical and 3D FE model,

313   respectively. The 29% is because of non-linear material behaviour is ignored while the 5% is

314   regarded as acceptable. It is worth mentioning that the FE model runs in about three hours

315   compared the milliseconds for the analytical model. In contrast, the Highways Agency model

316   (DMRB BA51/95) overestimated the re-anchorage length by 125% from the experimental

317   value assuming a linear stress profile which is clearly contradicted with the experimental results.

318   The DMRB The DMRB BA51/95 overestimation of the re-anchorage length (and thereby

319   underestimation of the residual capacity) could lead in practice to increased maintenance cost,

320   traffic disruption and unnecessary replacement or strengthening work (Abdelatif et al., 2016).

321 **5.3   Stress distribution across the concrete section**

322 The von Mises contour plot on the cross section at the end of post-tensioned concrete beam

323 after tendon rupture is shown in Fig. 10. It was found that, the stress contours in concrete tend

324 to take a circular shape around the tendon. This finding supports the assumption of the thick-

325 wall cylinder theory, which was used in development of the analytical model in Equation (1)

326 (Abdelatif et al., 2012) and are in line with previous findings (Abdelatif et al., 2015).

327 # 6.   Parametric study on factors affecting the tendon re-anchorage

328 Using the 3D non-linear FE model, the influence of different parameters on the re-anchorage

329 of the ruptured tendon can be assessed. In this parametric study, the influence of the diameter

330 and surface roughness of post-tensioning steel, diameter, thickness and material of sheathing

331 (i.e. duct), initial prestress, concrete strength, grout strength, voids in the grout, and shear

332 links, on the re-anchorage of a ruptured tendon are investigated. Unless stated otherwise, the

333 beam that was used in this study has the properties shown in Table 2. The cross section of the

334 beam was chosen from previous literature, (Coronelli et al., 2009), while the length was

335 chosen to just occupy the re-anchorage length (to minimize the computational cost).

336 Re-anchorage length normalised to the diameter of the post-tensioning steel (l_r/d) and the

337 effective prestress is normalised to the initial prestress (f_{se}/f_{si}) were considered as reference

338 values in this study.

339 Studying the large number of parameters in this study was not possible without the aid of

340 high performance computing cluster using eight CPUs with 16 GB RAM at the HPC unit in

341 the University of Nottingham. This greatly reduced the computational cost.

6.1   Diameter of post-tensioning steel

Fig. 11 shows the influence of the tendon size on the re-anchoring behaviour in the case of a constant ratio of 2.3 for duct to diameter of post-tensioning steel. The length required for tendons to re-anchor is directly proportional to the diameter (Fig. 11b). It is found that the tendons with a smaller diameter tend to re-anchor in a short length and hold more force in comparison to those with a larger diameter (Fig. 11b and Fig. 11c). Fig. 11a shows that, the slope of the stress profile, which is related to the bond stress, is also affected by the size of the tendon. Small tendons tend to produce higher bond stress than large tendons, Fig. 11a.

6.2   Surface roughness

In this parametric study, the coefficient of friction was varied from 0.3 to 0.7 to investigate the influence of the surface roughness on the re-anchorage phenomenon. It was found that tendon re-anchorage depends significantly on the surface condition of the prestressing steel at the time of rupture (Fig. 12). Fig. 12 shows that rough tendons need a shorter length to re-anchor compare to the smooth tendons. A similar conclusion was drawn in pre-tensioned concrete elements (Abdelatif et al., 2015).

6.3   Initial prestress

The investigation is conducted on a 3000 mm beam using 7 mm wire and 20 mm steel ducting with 1.9 mm thickness. In general, no significant influence of initial prestress on the re-anchorage phenomenon was observed (Fig. 13). The influences on the stress profile (Fig. 13a) and the effective prestress (Fig. 13c) are negligible. The difference of 1000 MPa in the initial prestress, results only in less than 5d difference in the re-anchorage length (Fig. 13b). As the higher initial prestress always generates higher hoop stresses in concrete and grout (due to the higher exerted radial pressure at the interface), this suggests that the non-linear behaviour of the concrete and grout has insignificant influence on the tendon re-anchorage.

6.4 Concrete strength

In most design codes, the concrete compressive strength is recommended to be more than 35 MPa for prestressed concrete structures. However, in this study, concrete with 20-50 MPa compressive strength is used to observe the overall influence of concrete strength on the re-anchoring. Generally, concrete with strength within the practical range does not show a significant influence on the tendon re-anchorage behaviour (Fig. 14). The difference of the re-anchorage length through the tested range of concrete strength is less than 2d while the difference in the effective prestress is about 2%. This finding supports the assumptions made in Section 6.3 that, the non-linear behaviour of concrete has an in significant influence on the re-anchorage of the ruptured tendon.

6.5 Grout strength

In this investigation, neither the stress distribution nor the effective prestress show significant changes due to the change of the grout compressive strength (Fig. 15). However, the re-anchorage length shows less than 10d difference through the tested range (10-40 MPa), Fig. 15b. In other words, each 5 MPa increase in the grout strength results in only 1.5d decrease in the re-anchorage length in this simulation. This finding is in agreement with previous experimental results (Geddes and Soroka, 1964, Watanabe et al., 2011) and observations during demolition of bridge girders (Buchner and Lindsell, 1987b). This could be due to the confinement introduced by the duct to the grout.

6.6 Presence of the grout voids within the re-anchorage length

Presence of voids on the grout is known to affect the re-anchorage behaviour of ruptured tendons in addition to creating a corrosive environment (Cavell and Waldron, 2001). In this investigation, three sizes of voids are considered: 25%, 50% and 100% void across the grout section (Fig. 16). Voids of 200 mm length (15.75d) were introduced in the grout to investigate the impact of grout voids on the re-anchorage mechanism. These voids were

391   located to start at 100 mm far from the end of the prism. The results show that, the presence

392   of grout voids within the re-anchorage zone makes a significant change in the prestress

393   profile and results in much longer re-anchorage length, Fig. 17a and Fig. 17b. If the beam is

394   long enough for the tendon to re-anchor, no significant drop will occur in the effective

395   prestress, Fig. 17c. The re-anchorage length of the voided tendons was approximated in a

396   previous study by adding the void's length to the non-voided re-anchorage length (Cavell and

397   Waldron, 2001). However this study shows, the re-anchorage length is increased by 67%,

398   114%, and 142% of the void's length for 25%, 50%, and 100% of void in the grout cross

399   section, respectively.

400   **6.7   Duct diameter (Grout thickness)**

401   An increase in the diameter of the duct increases the grout thickness. Thicker grout allows the

402   tendon to deform in the longitudinal direction more than the thinner grout. Therefore, tendons

403   with a large duct diameter (thicker grout layer) were found to have a longer re-anchorage

404   zone, a higher prestress and less bond stress in comparison with tendons with smaller duct

405   diameter (thinner grout layer), Fig. 18.

406   **6.8   Duct thickness**

407   The practical range of duct thickness is 0.25 mm for steel ducts, 2.5-3.0 mm for

408   polypropylene (PP) ducts, and 4.0-6.0 mm for high density polyethylene (HDPE) ducts

409   (Hewson, 2003). A steel duct with outer diameter of 30 mm was used to examine the impact

410   of ducting thickness on the re-anchorage behaviour. It is found that, increasing the duct

411   thickness increases the level of confinement around the post-tensioning steel which results in

412   a shorter re-anchorage length (Fig. 19). No significant influence is observed on the effective

413   prestress (Fig. 19c).

6.9 Duct material

The ducts used in post-tensioned concrete are usually manufactured from steel, polypropylene (PP), and high density polyethylene (HDPE). These materials, in addition to polyvinyl chloride (PVC), which is used in this study, were considered to investigate the influence of ducting material on the re-anchoring phenomenon. The investigation considered a 6000 mm beam that was post-tensioned to 750 MPa and injected with grout of 40 MPa compressive strength. The Young's moduli were taken as 200 GPa, 3.0 GPa, 1.75 GPa, and 0.8 GPa for steel, PVC, PP, and HDPE respectively. The parametric study shows that, the tendons with a duct of low Young's modulus show poor re-anchorage (i.e. long re-anchorage length, small effective prestress and low bond stress) as shown in Fig. 20. The tendons with steel ducting show good re-anchorage behaviour compared to those with PP and HDPE ducts and less longitudinal deformation.

6.10 Shear reinforcement

Fig. 21 shows the influence of shear reinforcement (stirrup) on the re-anchorage of the ruptured tendon. In this simulation, no significant influence of stirrups spacing on the re-anchorage of the ruptured tendon was observed. This contradicts the observation during the controlled demolition of grouted post-tensioned concrete structures (Buchner and Lindsell, 1987b). This contradiction can be attributed to:

i) Experimental point of view: The observation of Buchner and Lindsell (1987b) were based on comparisons between beams of two different bridges with different properties, different number of strands in tendon and different level of prestress.

ii) Modelling point of view: Modelling the behaviour of confined concrete correctly needs to consider the influence of confinement on the following factors in the CDP model: a) yield criterion; b) hardening and softening rule c) flow rule. None of the existing CDP type models includes all the three factors (Yu et al., 2010). However, the model shows good

439   agreement compared to experimental results in this study of post-tensioned concrete

440   prisms with shear reinforcement as shown in Fig. 8e.

441   **7.   Conclusions**

442   The paper has presented 3D non-linear finite element model for re-anchorage of a ruptured

443   tendon in bonded post-tensioned concrete structures. The model was compared to previous

444   analytical models and validated using experimental data on prisms, utilising the 3D

445   Electronic Speckle Pattern Interferometry system (ESPI). The proposed model was also

446   validated against previous experiments on beams and compared with the Highways Agency

447   DMRB BA51/95 re-anchorage model. The developed model was found to be able to predict

448   the re-anchorage length compared to the Highways Agency model DMRB BA51/95 which

449   greatly overestimates the re-anchorage length. The findings related to the tendon re-

450   anchorage (modelling and experiments) are:

451   • The experiments confirmed the re-anchorage of the ruptured tendon

452   • The 3D finite element is able to capture the re-anchorage phenomenon

453   • The developed models and experiments support the hypothesis that the re-anchorage

454   phenomenon is influenced by the confining materials

455   The 3D FE non-linear re-anchorage model was then used in a parametric study to investigate

456   the influence of prestressing steel diameter and surface roughness, duct diameter and

457   thickness, duct material, initial stress, concrete strength, grout strength, presence of voids in

458   the grout, longitudinal reinforcement, shear links, and number of strands in the tendon on the

459   re-anchoring behaviour of the ruptured tendon. The results of this parametric study is

460   summarised as follows:

- The parameters that have a significant influence on the re-anchorage are tendon properties (i.e. diameter, roughness) and duct properties (i.e. diameter, thickness, and material)

- Tendons with larger diameter have a long re-anchorage length whereas those with thicker duct and those with smaller diameter have a shorter re-anchorage length

- Tendons with rough surfaces re-anchor better than those with smooth surfaces

- Tendons with steel ducts re-anchor much better compared to those with polypropylene (PP), and high density polyethylene (HDPE) ducts

The presence of voids in the grout around the ruptured tendons results in longer re-anchorage length depending on the size of the voids

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Table 1: Test matrix showing variation of model parameters

Test No.	Prism ID	Stress (MPa)	Bar Dia.	Duct Material	Duct OD/T (mm)*	Concrete E	Concrete f_t	Concrete f_c	Grout E	Grout f_t	Grout f_c	Steel E	Steel f_y	End Cond.	Remarks
1	P1	1020	7	PVC	20/1.9	31.0	3.6	57.1	26.8	2.1	68.9	207	1562	Setup-1	Plain concrete
	P2	1030													
	P3	1021													
2	P4	607.5	7	PVC	20/1.9	37.9	3.9	56.9	24.9	11.1	60.8	207	1562		
	P5	755.7													
	P6	815.7													
3	C1	783	7	PVC	20/1.9	43.2	3.8	50.6	23.7	7.5	47	207	1562		
	C2	880													
	C3	807.3													
4	G1	917.3	7	PVC	20/1.9	43.2	3.9	50.6	18.7	2.5	39	207	1562		
	G2	1032													
	G3															
5	CG1	866.5	7	PVC	20/1.9	43.4	3.6	46.1	19.2	9.5	31.7	207	1562		
	CG2	980.7													
	CG3	922.3													
6	CC1	872	7	PVC	20/1.9	43.9	3.6	46.6	-	-	51.7	207	1562		
	CC2	1090													
	CC3	832.4													
7	Dm1	900.4	7	Steel	19.05/1.59	44.0	4.0	49.0	8.53	12.7	53.7	207	1562		
	Dm2	837.3													
	Dm3	809.6													
8	C10	922	7	PVC	20/1.9	42.3	4.1	51.2	21	14.8	58.1	207	1562	Setup-2	
	C20	1098													
	C30	953													
9	ST1	861.5	7	PVC	20/1.9	44.0	4.5	52.5	25.1	12.4	71.6	207	1562		Reinforced concrete**
	ST2	979.7													
	ST3	953.4													
10	V1	1113	5	PVC	20/1.9	44.3	3.6	53.7	24.5	3.2	53.7	207	1562		Plain concrete
	V2	1062													
	V3	1096													
11	Gr1	801	7	PVC	20/1.9	44.6	2.7	46.8	13.2	3.8	15.2	207	1562		
	Gr2	982													
	Gr3	866													

*OD/T: Outer diameter/wall thickness

** Four 6 mm steel bars were used as main reinforcement with 6 mm stirrups every 50 mm

*** f_c, f_t, f_y: compression, tensile, yield strength in MPa. E: Young's modulus in GPa

580 **Table 2: Properties of the beam used in the parametric study**

Beam			Prestressing steel			Grout	Concrete	Steel Duct	
width (mm)	height (mm)	Length (mm)	Initial prestress (MPa)	Diameter (mm)	E_s (GPa)	f_{cg} (MPa)	f_c (MPa)	Outer dia. (mm)	Thickness (mm)
150	200	1500	1250	12.7	2.07	20	50	25	2.5

581

Figures captions

604   **Fig. 15: Influence of the grout compression strength on: (a) Stress distribution; (b) Re-anchorage length;**

605   **(c) Effective prestress**

606   **Fig. 16: Size of the voids introduced in the grout**

607   **Fig. 17: Influence of grout voids on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective**

608   **prestress**

609   **Fig. 18: Influence of the duct diameter and grout thickness on: (a) Stress distribution; (b) Re-anchorage**

610   **length; (c) Effective prestress**

611   **Fig. 19: Influence of the duct thickness on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective**

612   **prestress**

613   **Fig. 20: Influence of the duct material (a) Stress distribution; (b) Re-anchorage length; (c) Effective**

614   **prestress**

615   **Fig. 21: Influence of shear links on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective**

616   **prestress**

617

PL 100x100x12

Wedge

PSC bar

duct

Grout

PL 100x100x12

4 M12 bolts

100

Section A

500

Releasing nuts (8 M12)

A

(a)

(b)

(a)

(b)

S, Mises
(Avg: 75%)
+2.078e+01
+1.905e+01
+1.733e+01
+1.561e+01
+1.389e+01
+1.217e+01
+1.044e+01
+8.723e+00
+7.001e+00
+5.279e+00
+3.557e+00
+1.835e+00
+1.127e-01

(a)

(b)

(c)

0% voids   25% voids   50% voids   100% voids
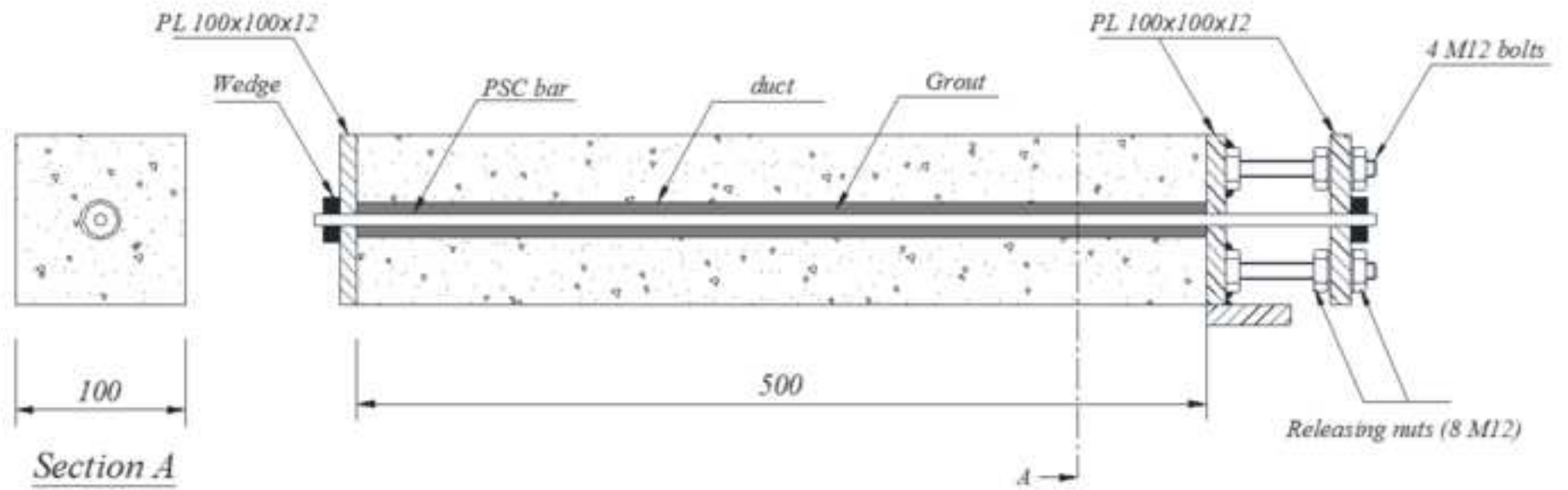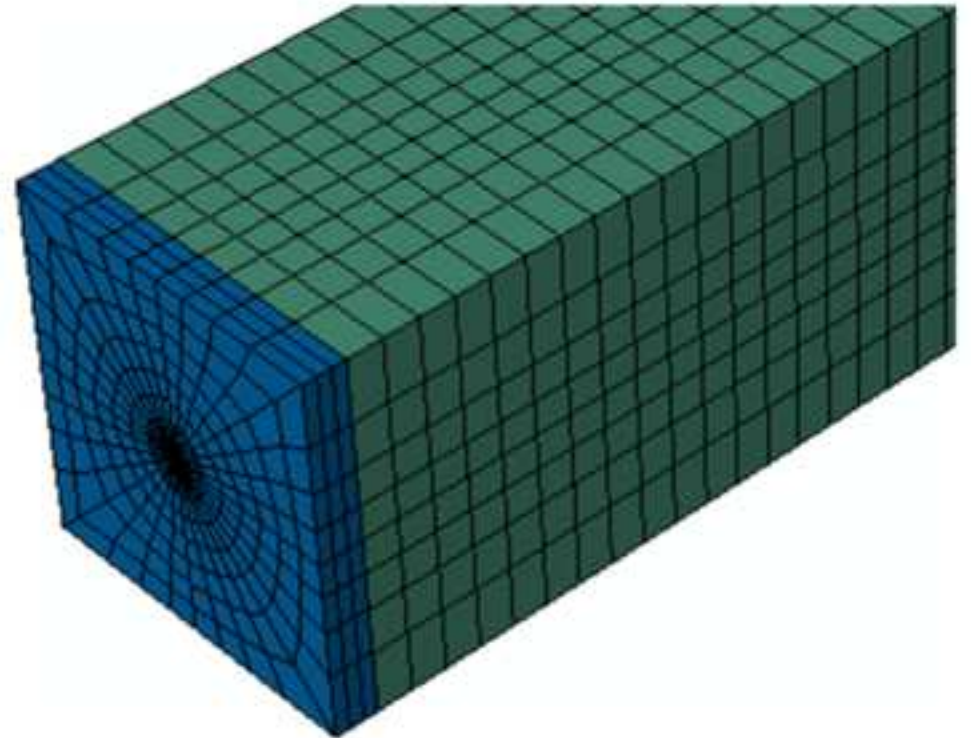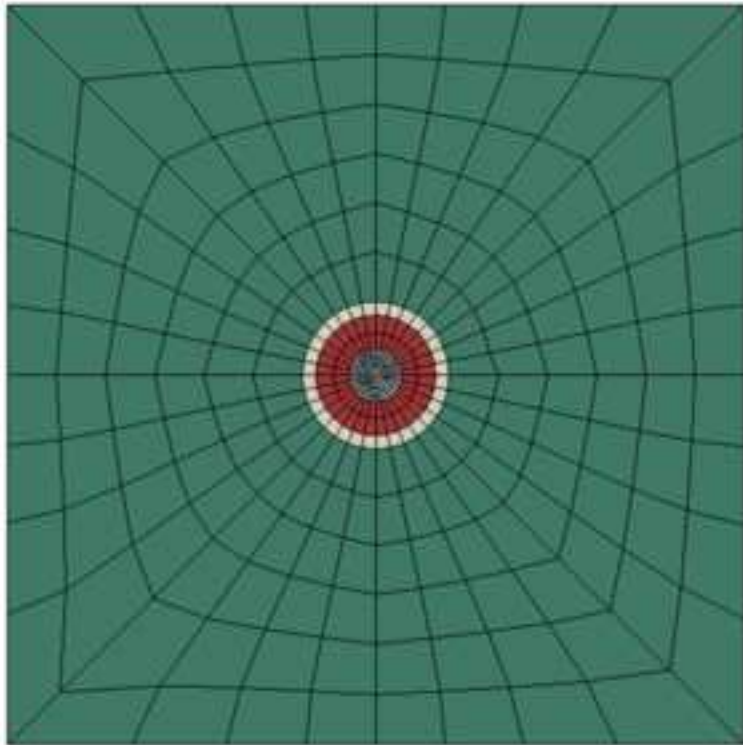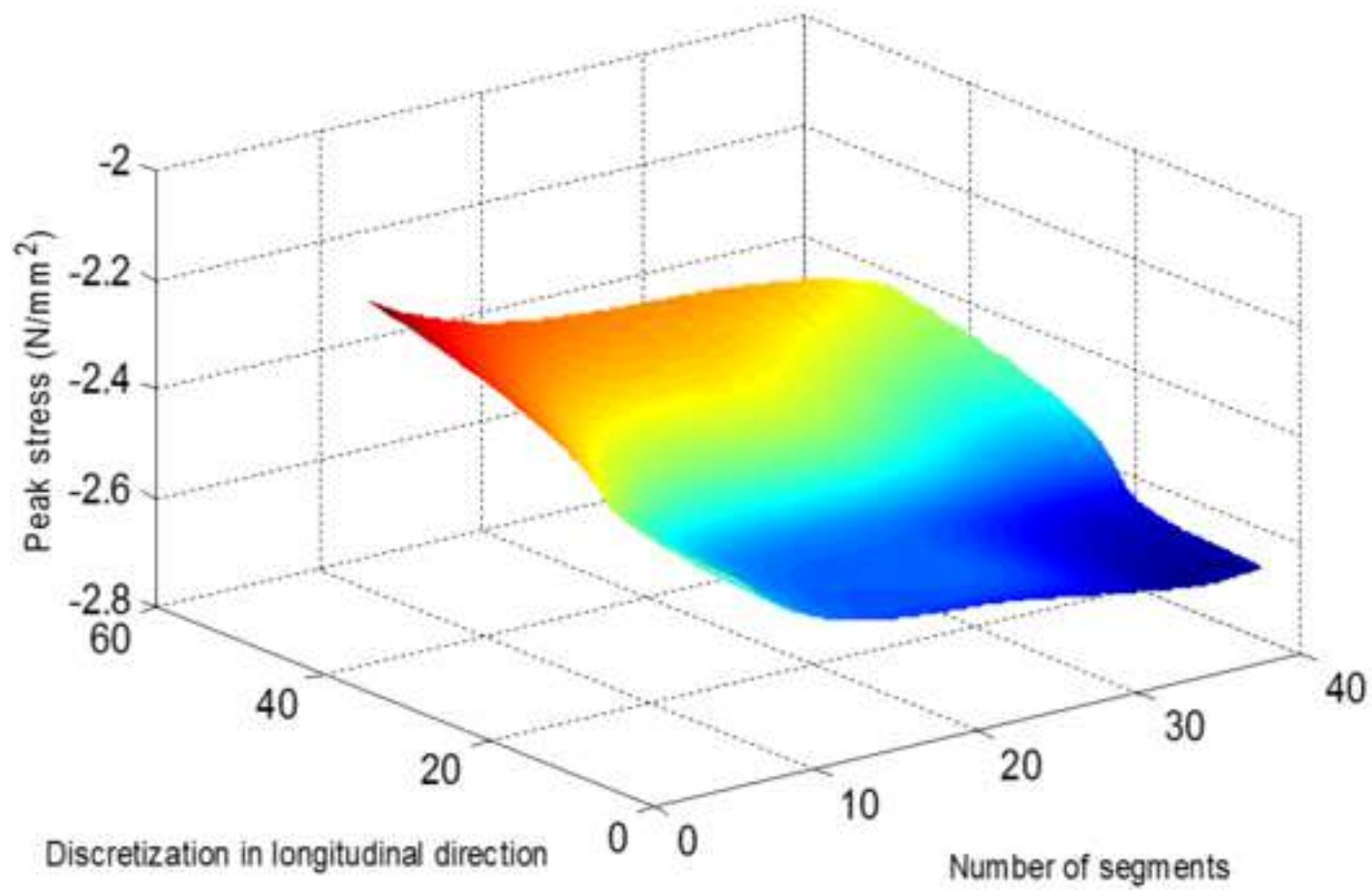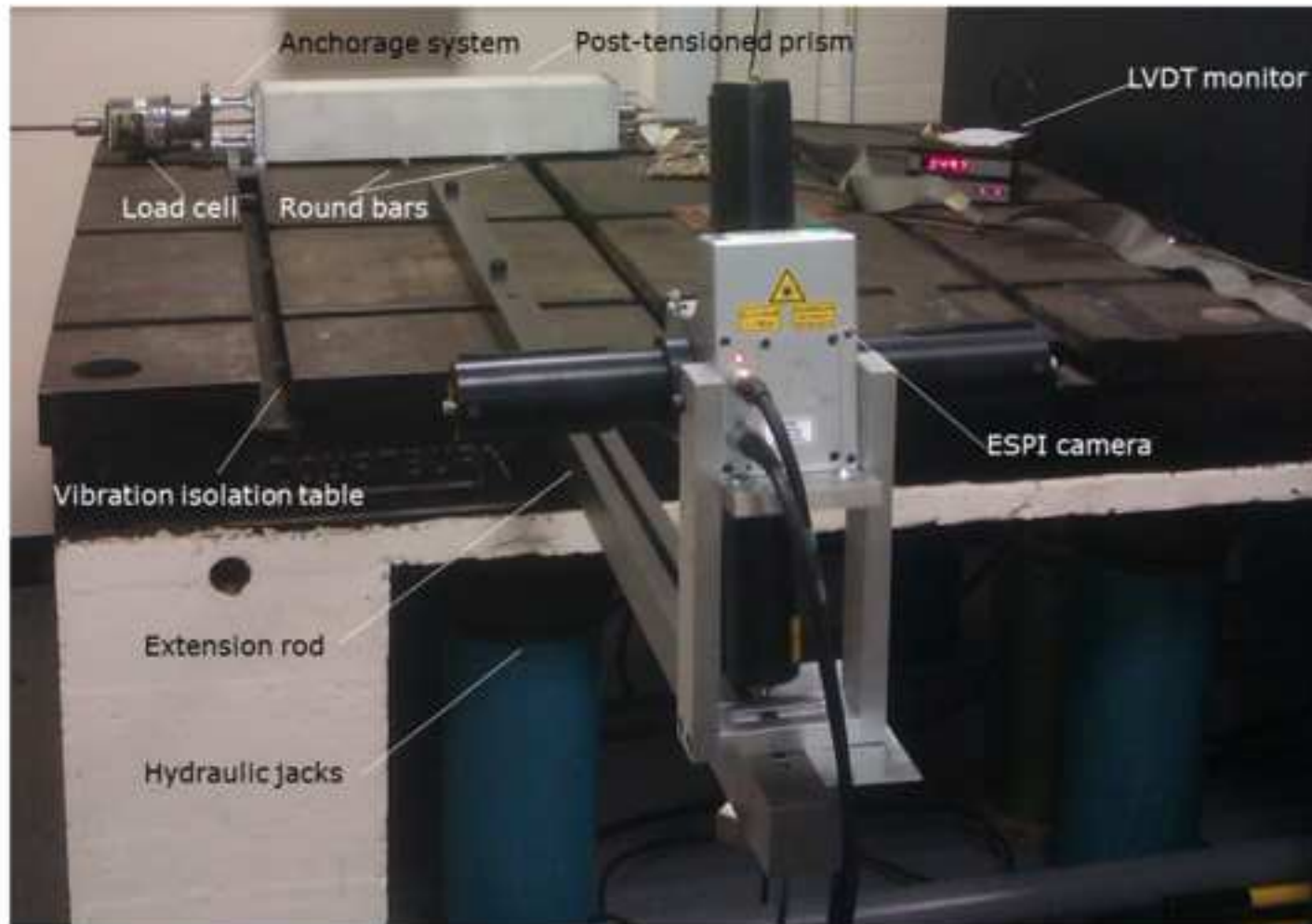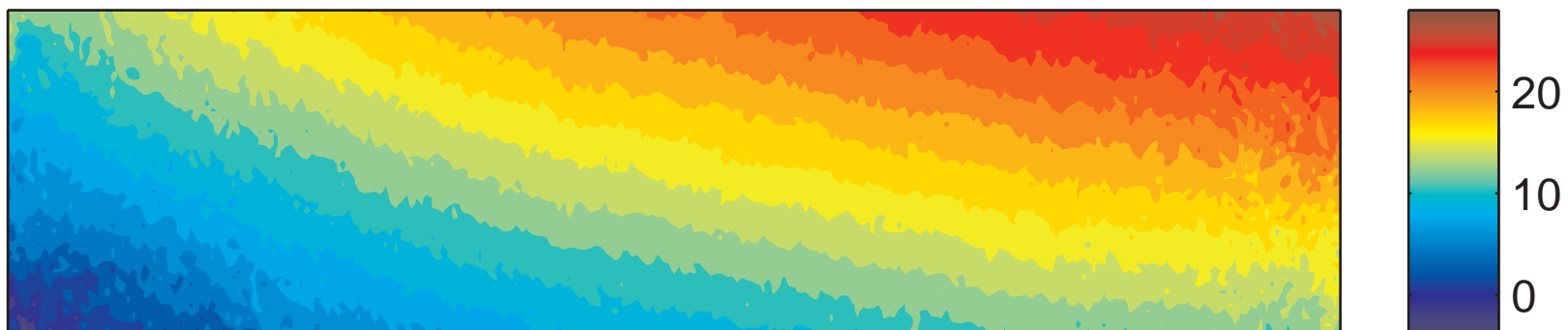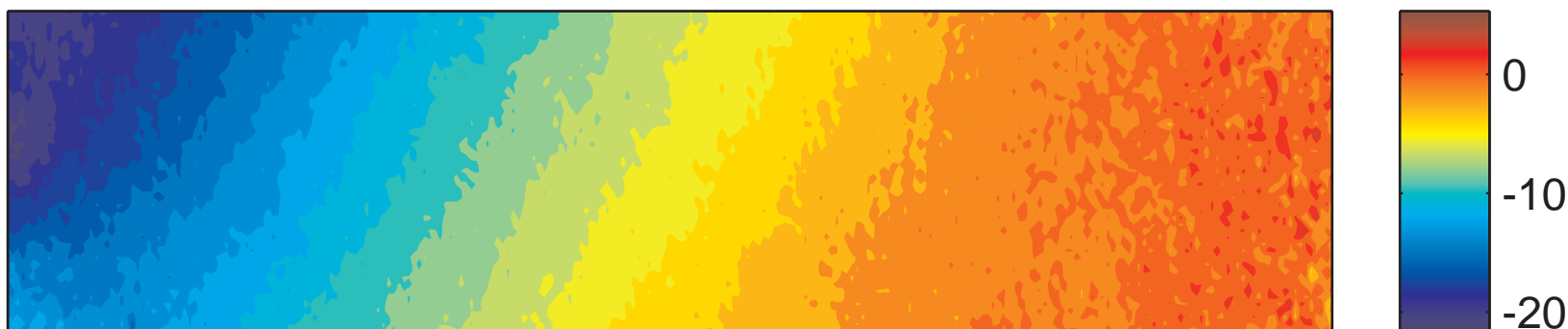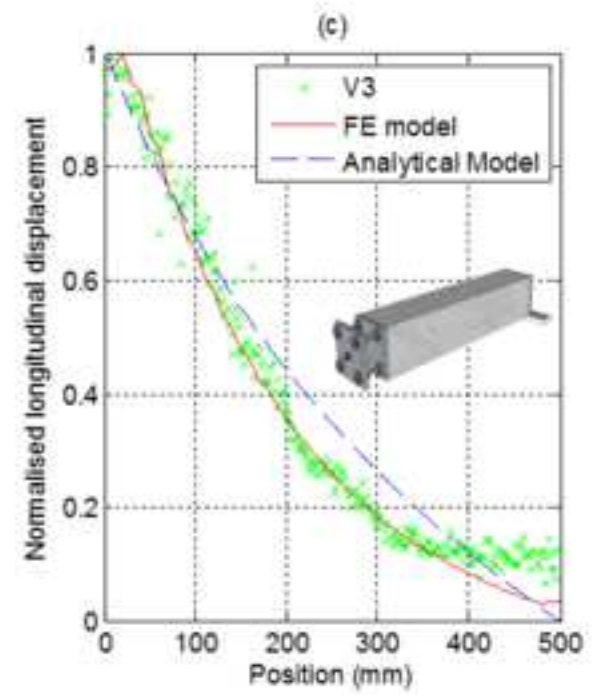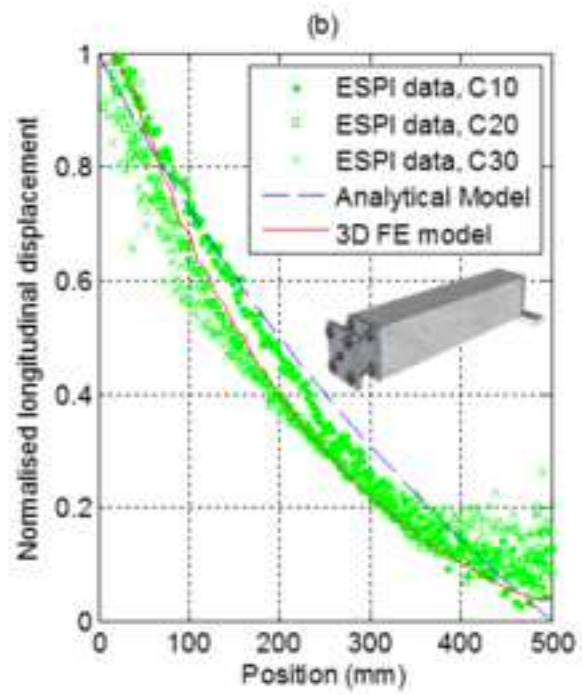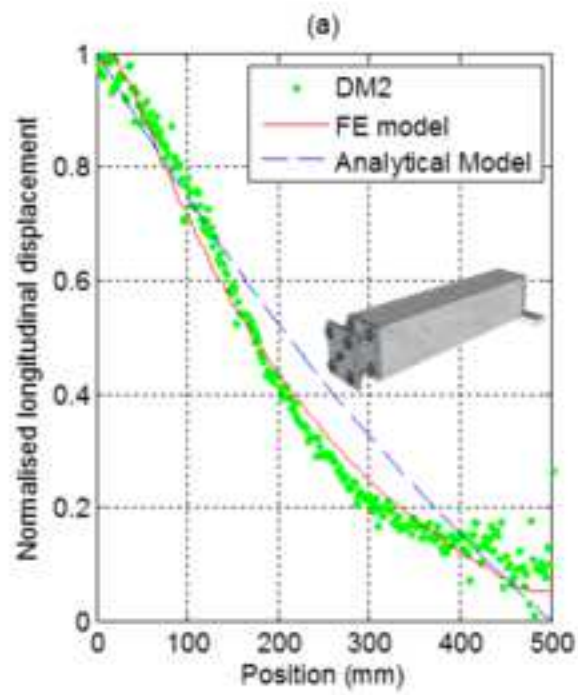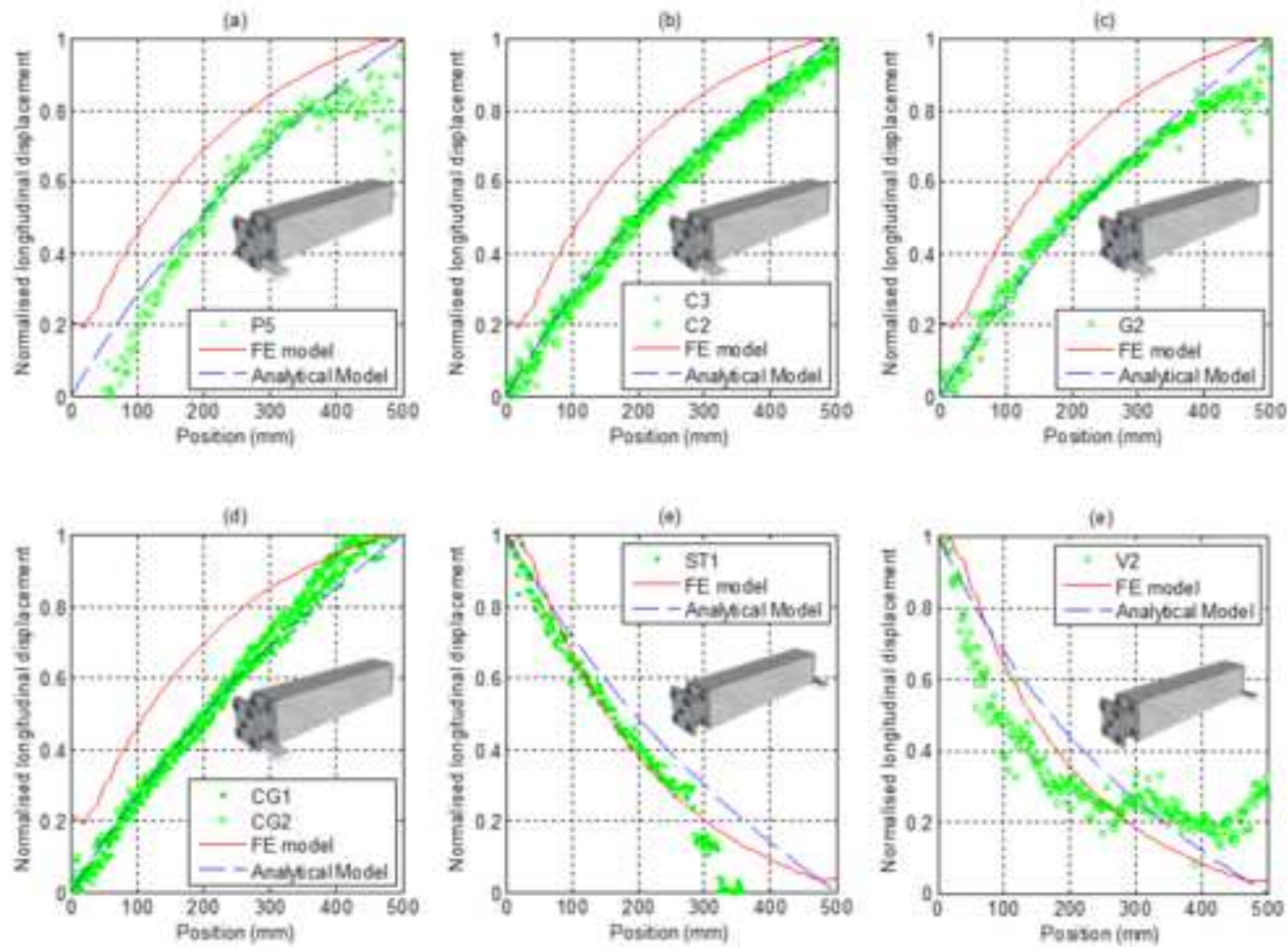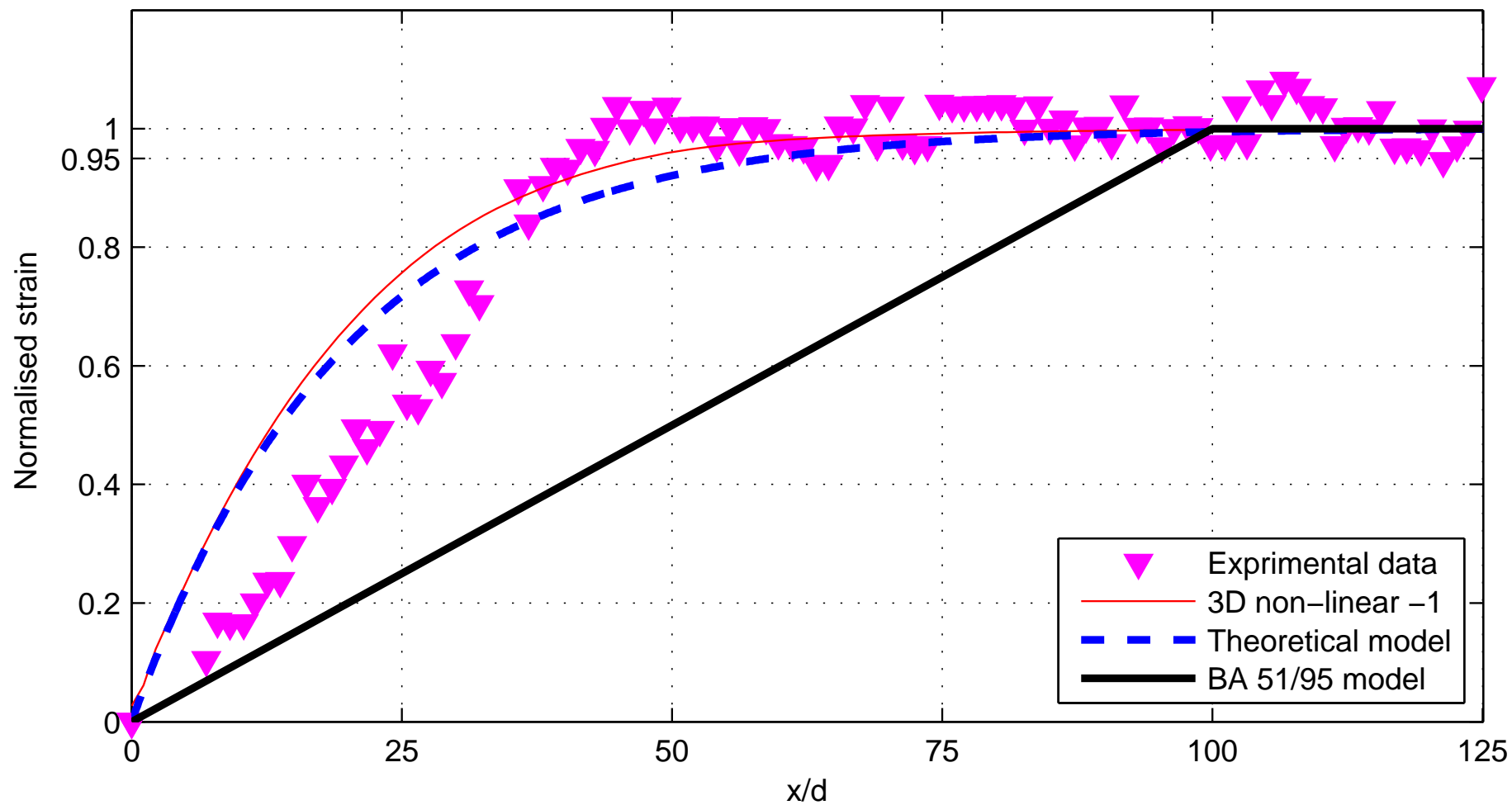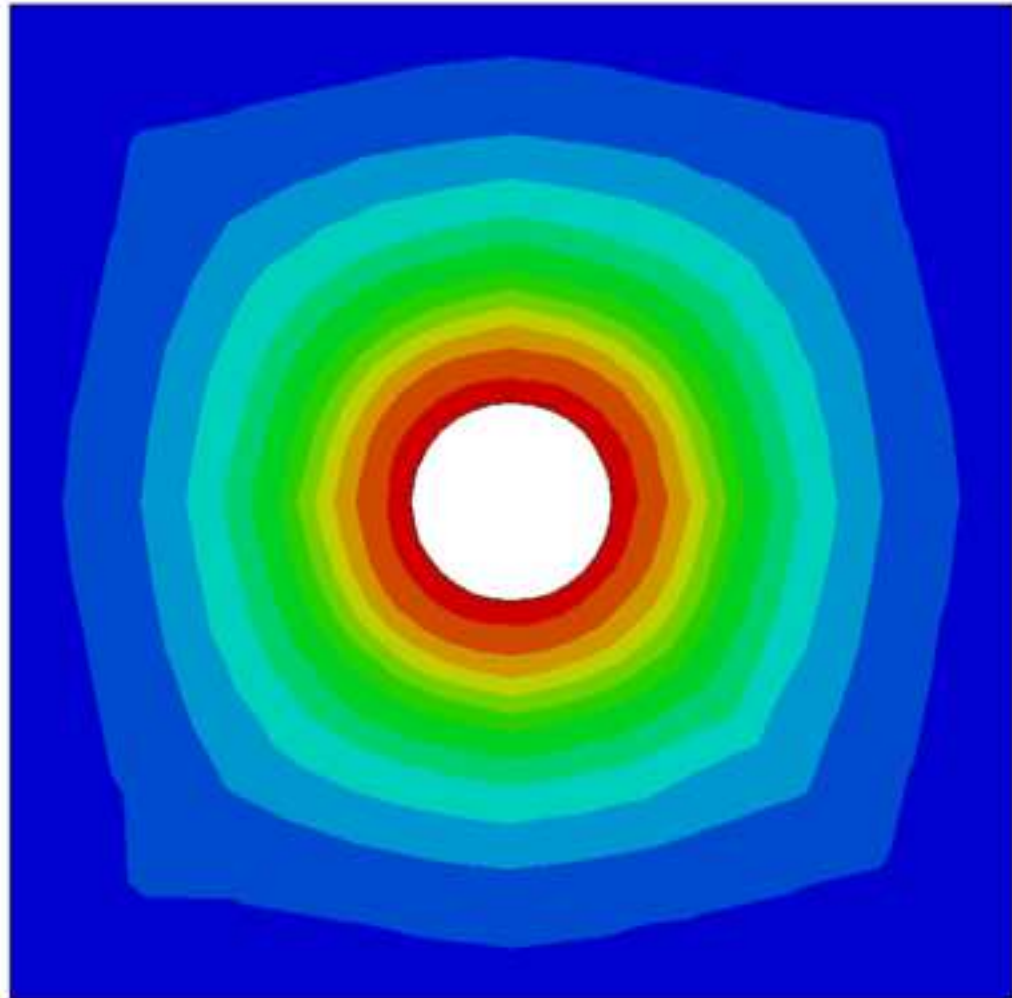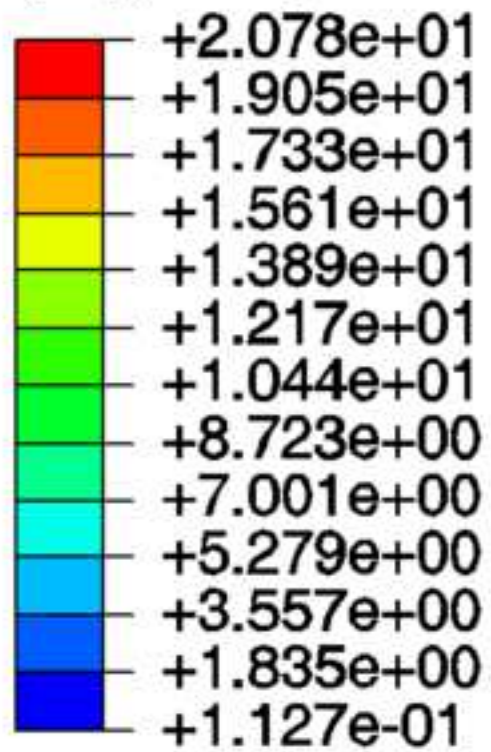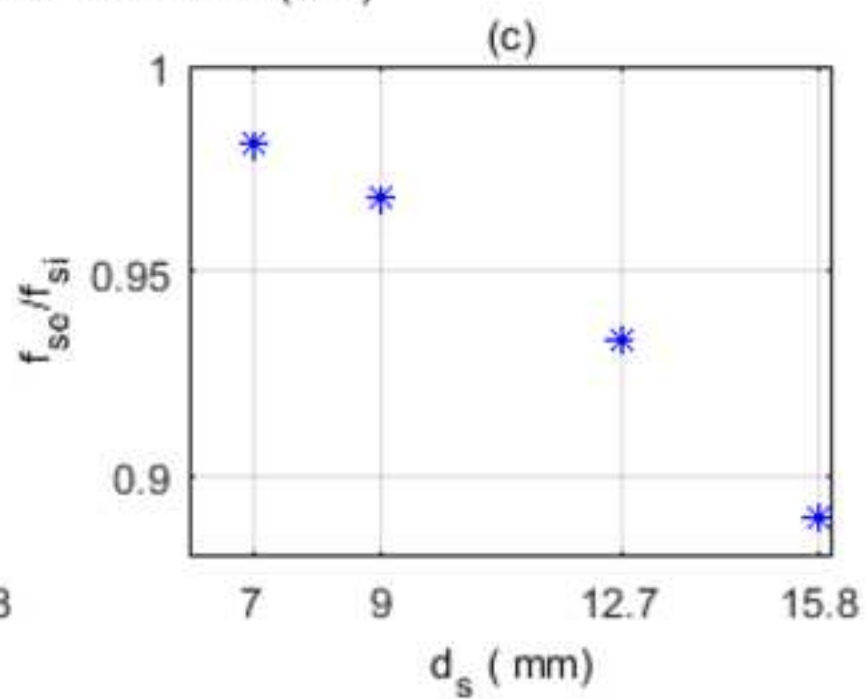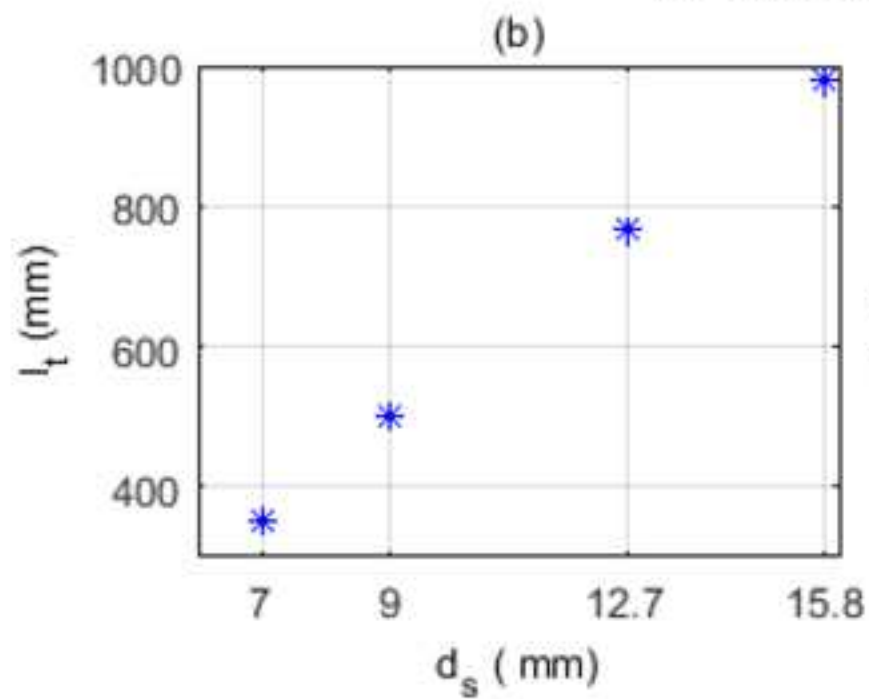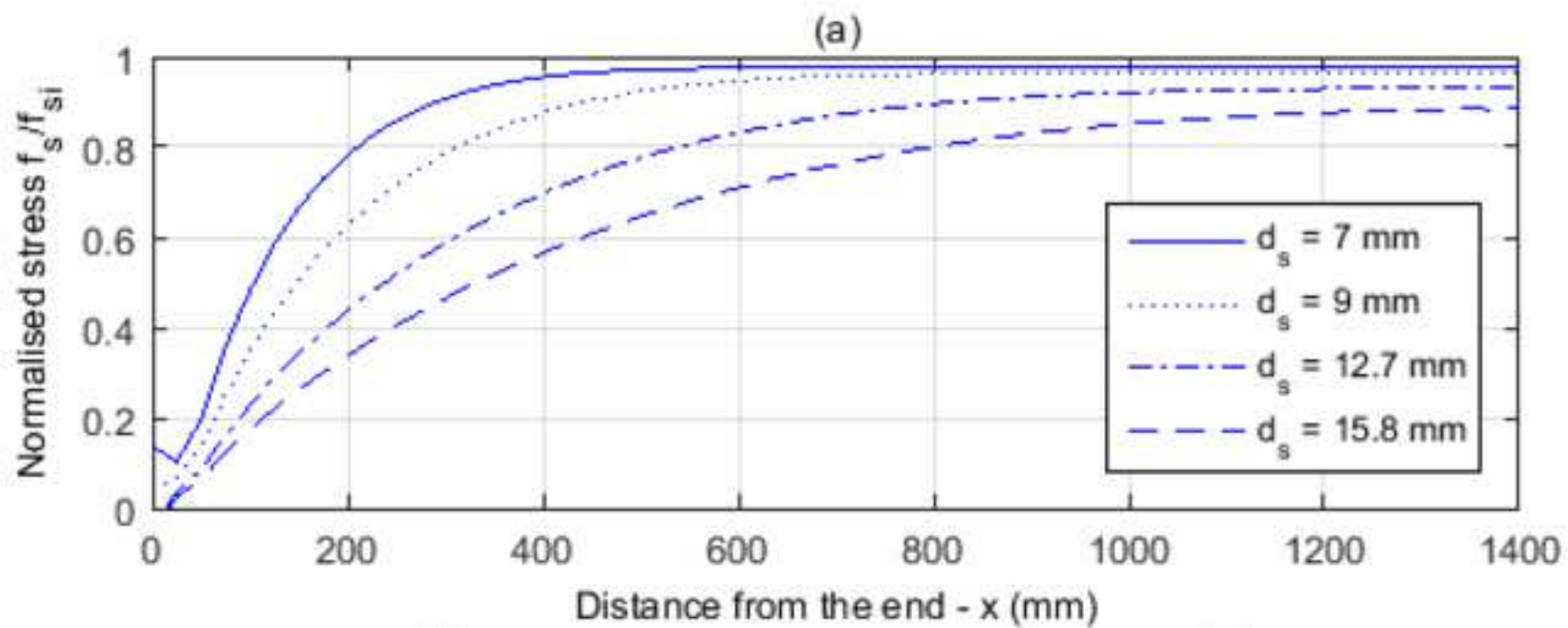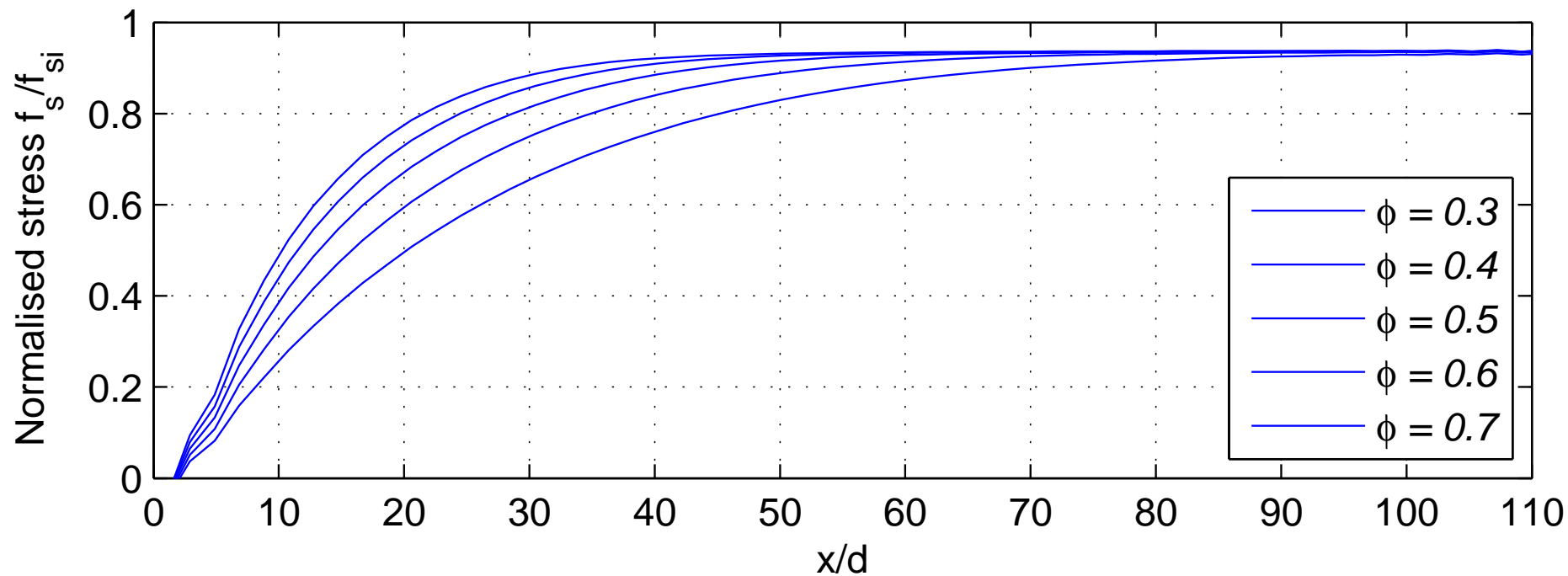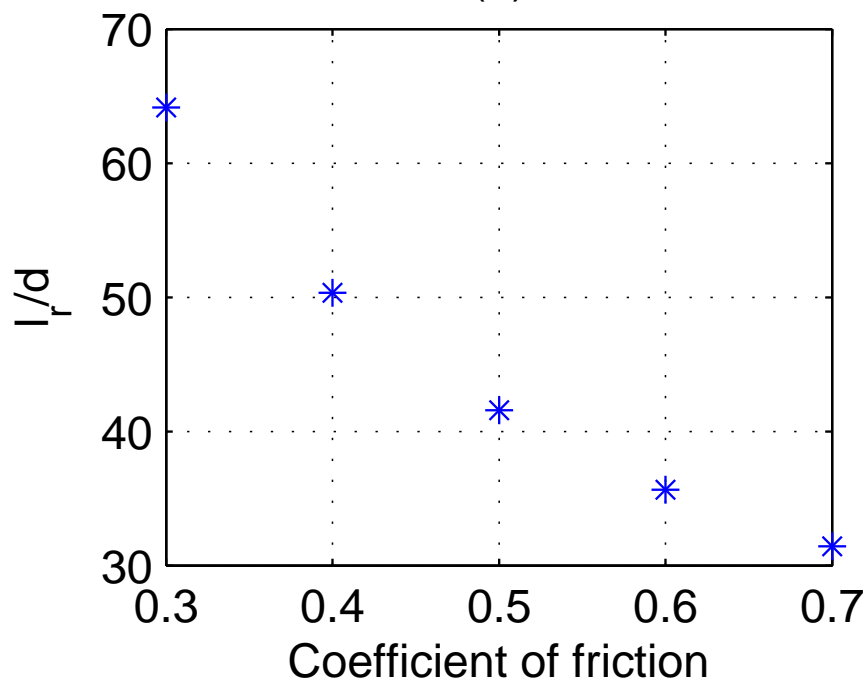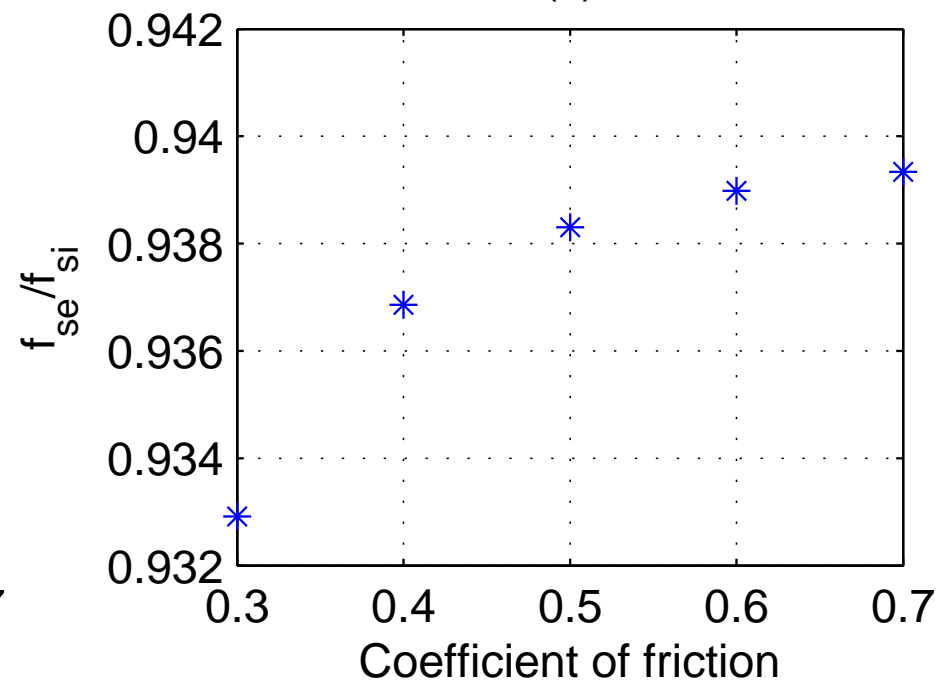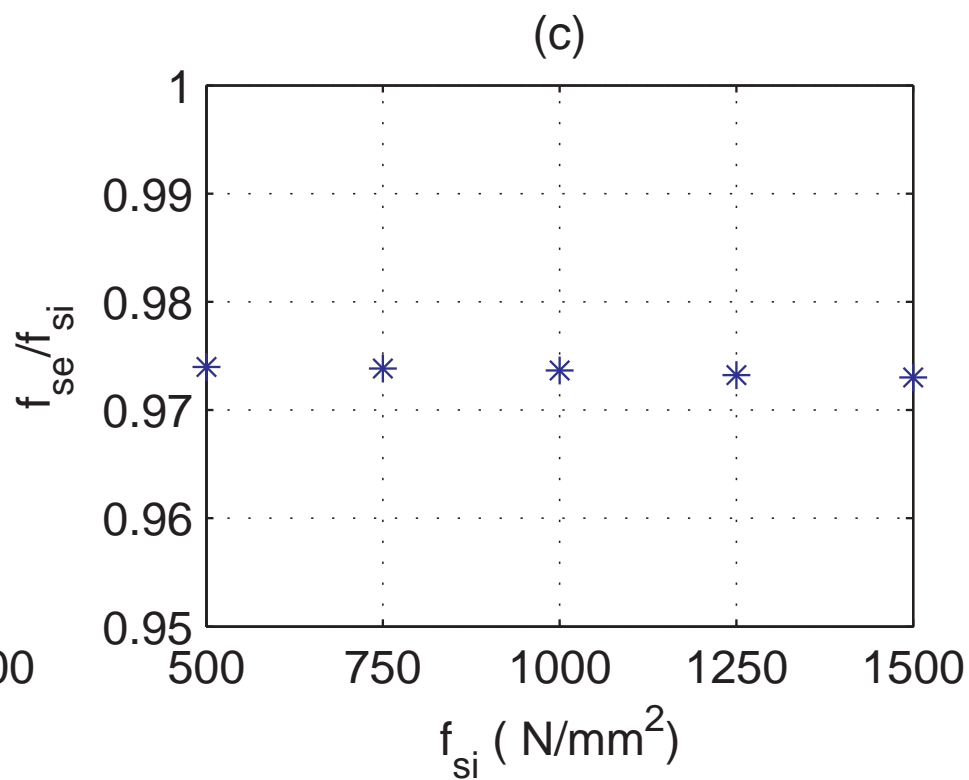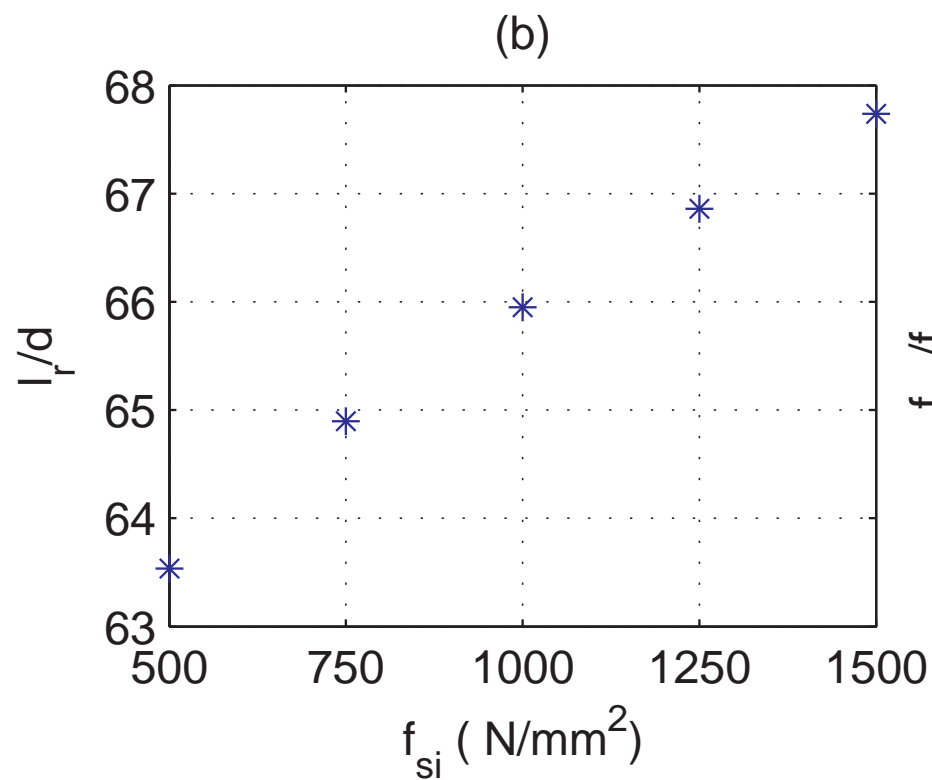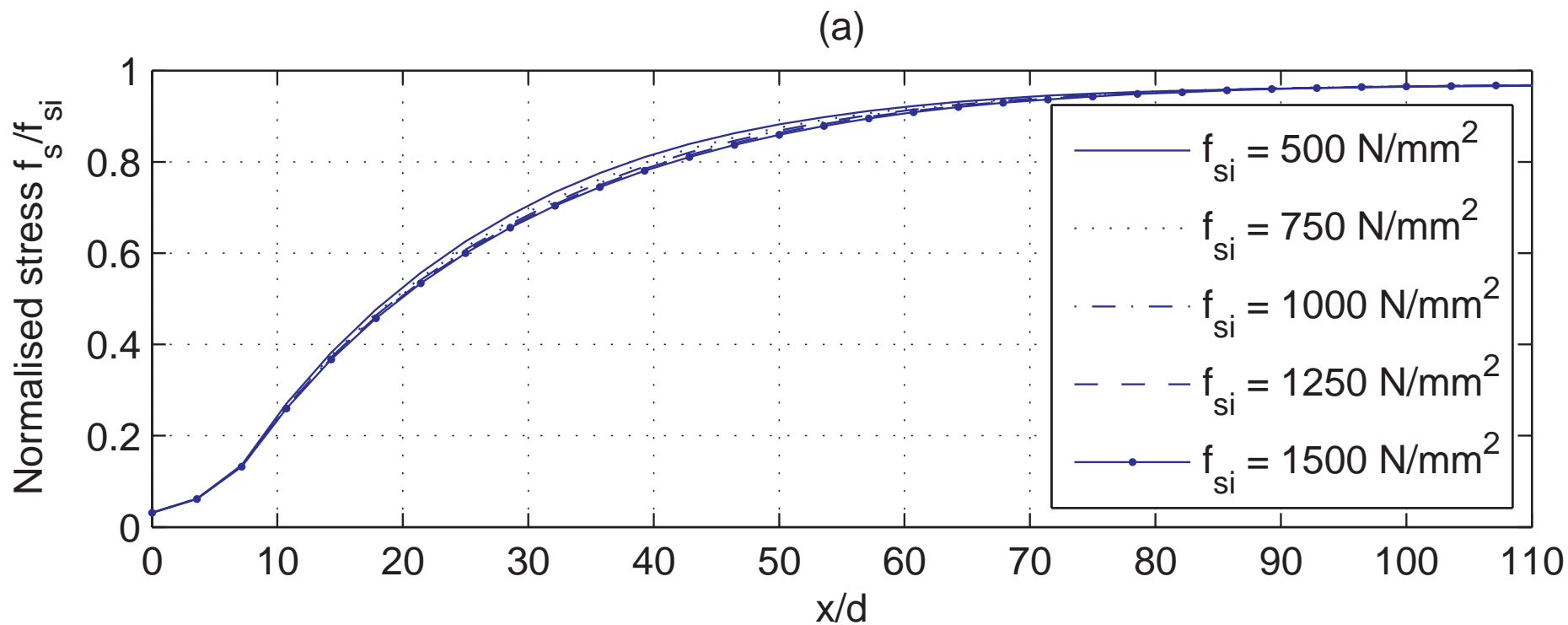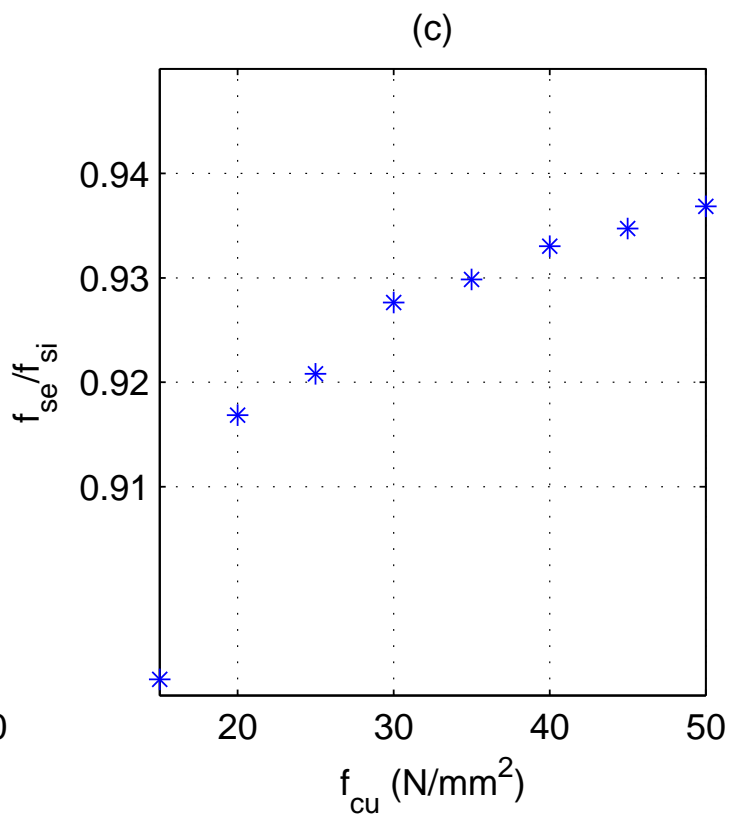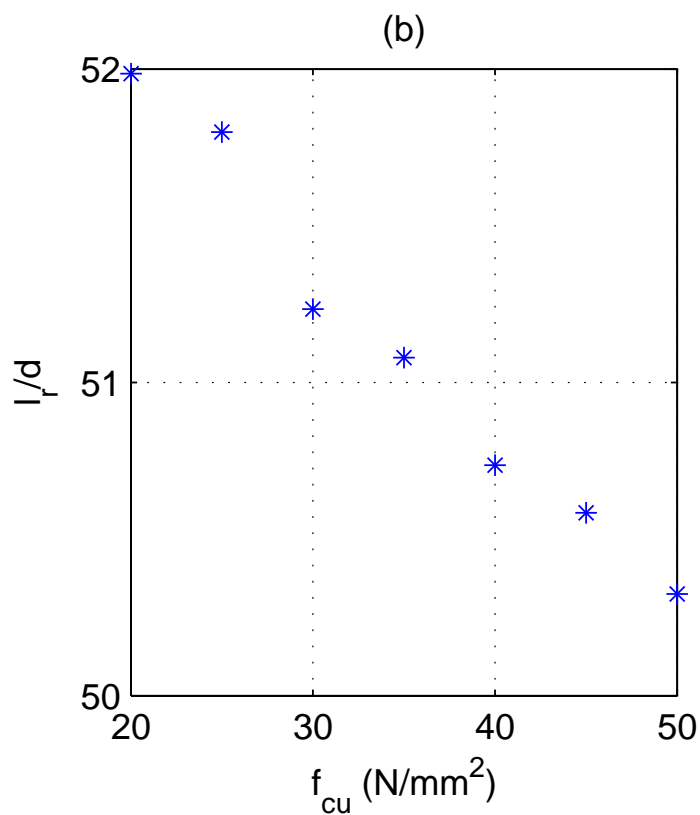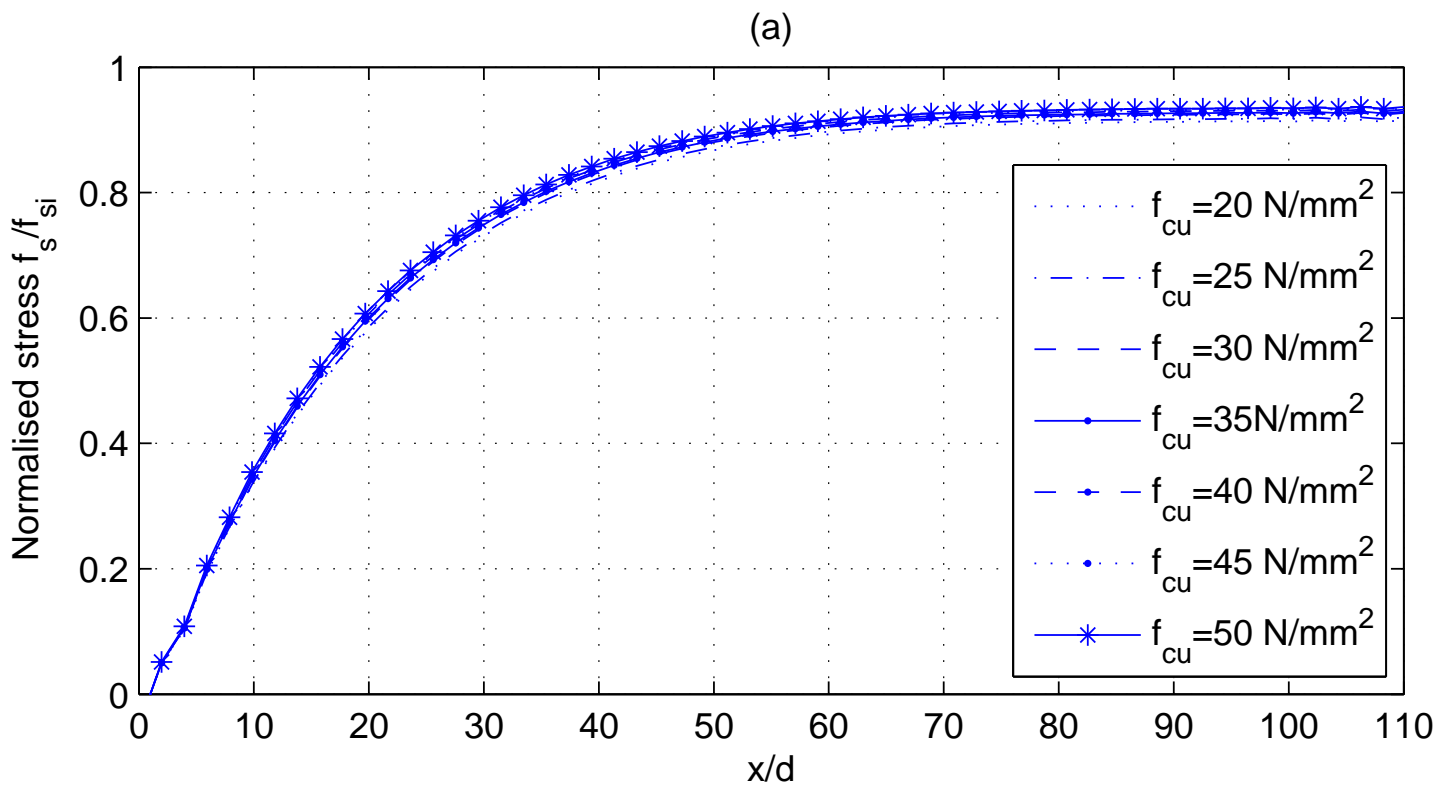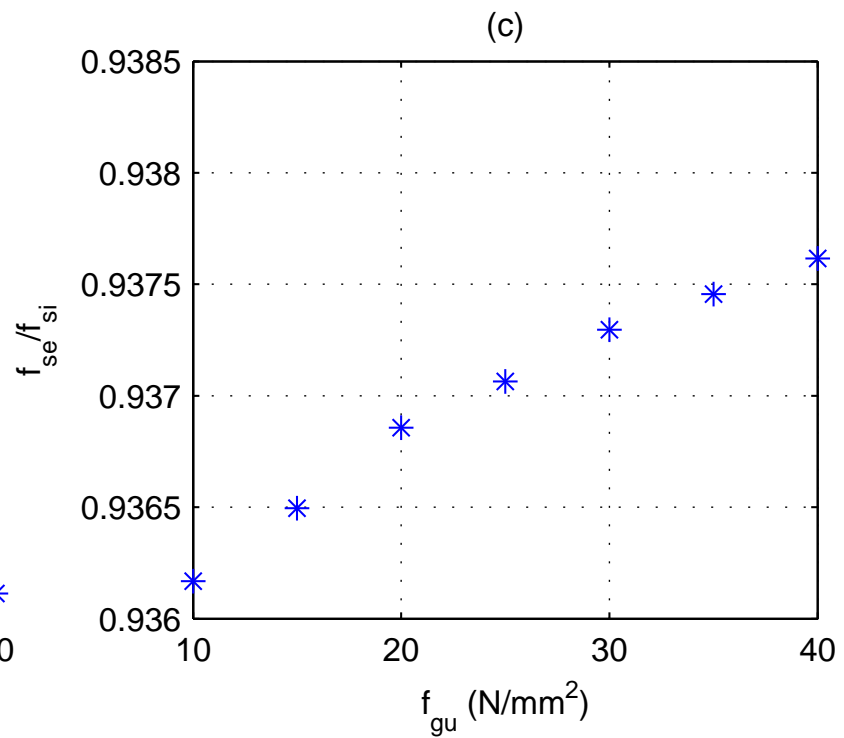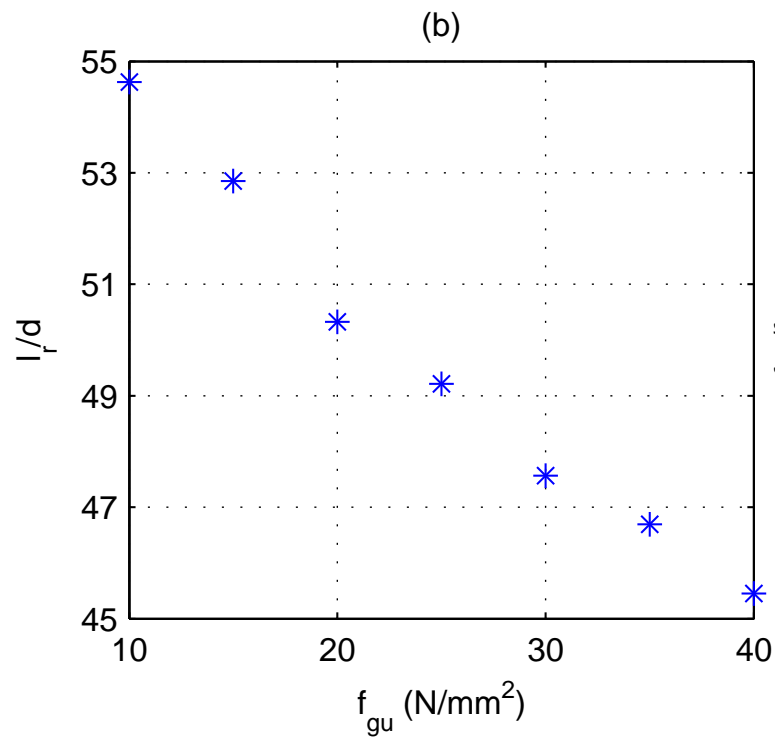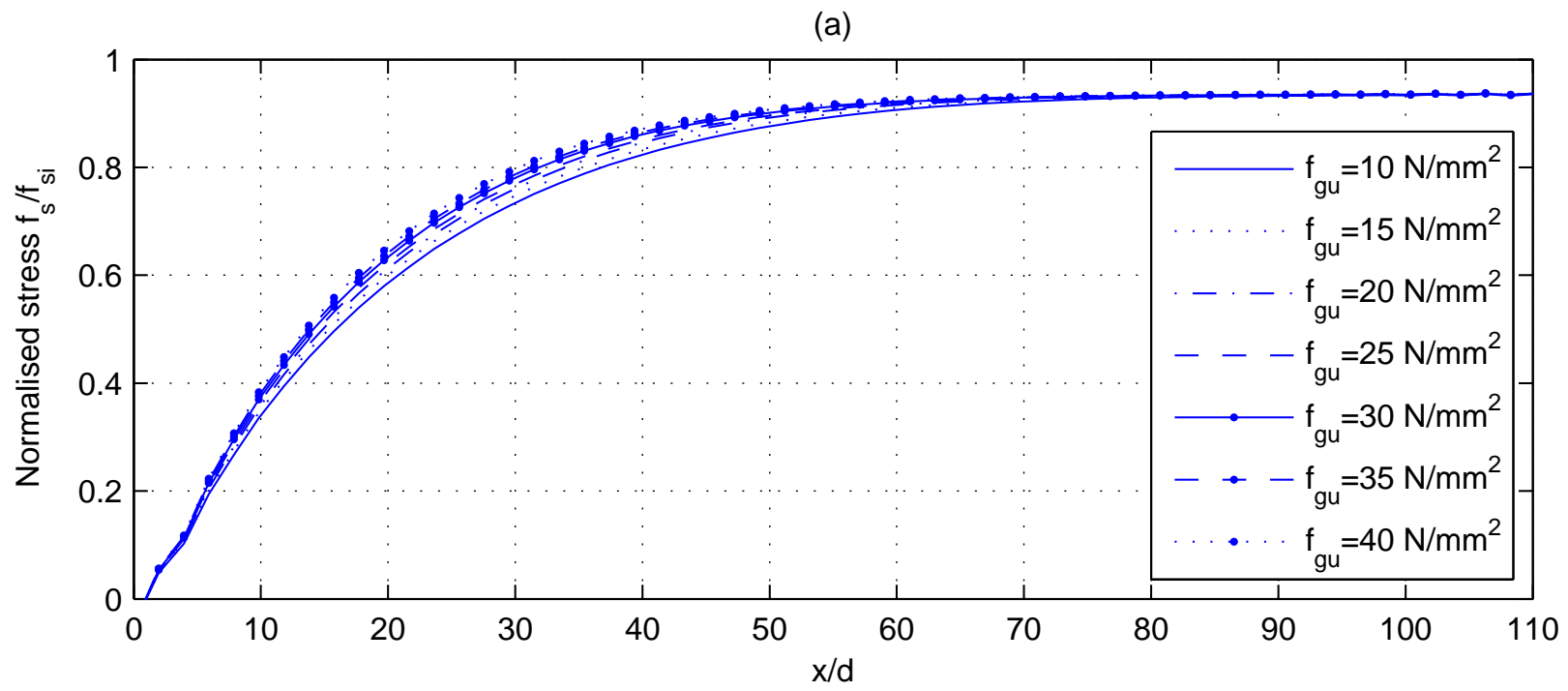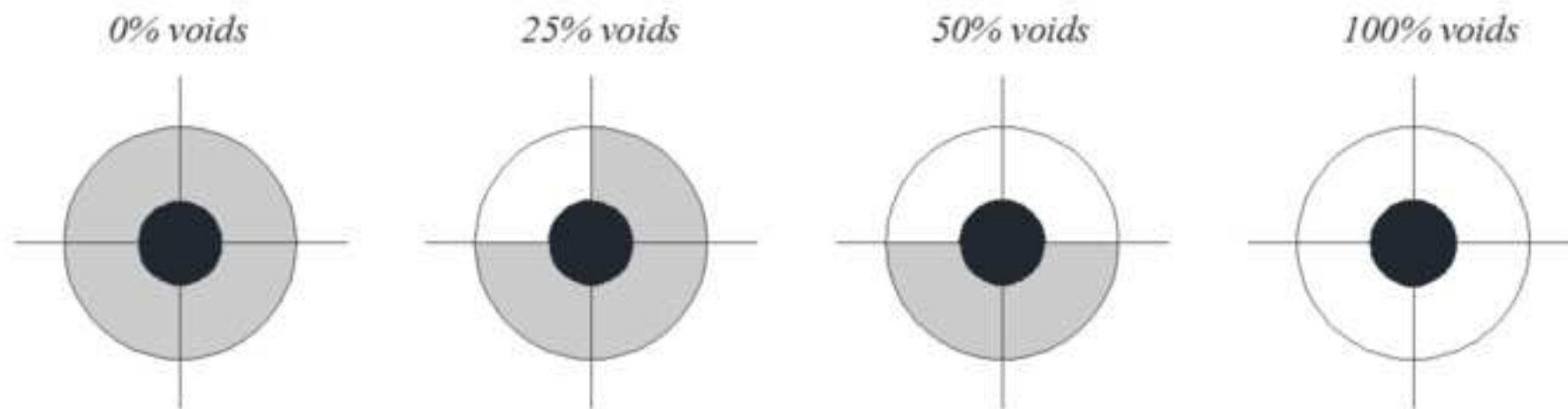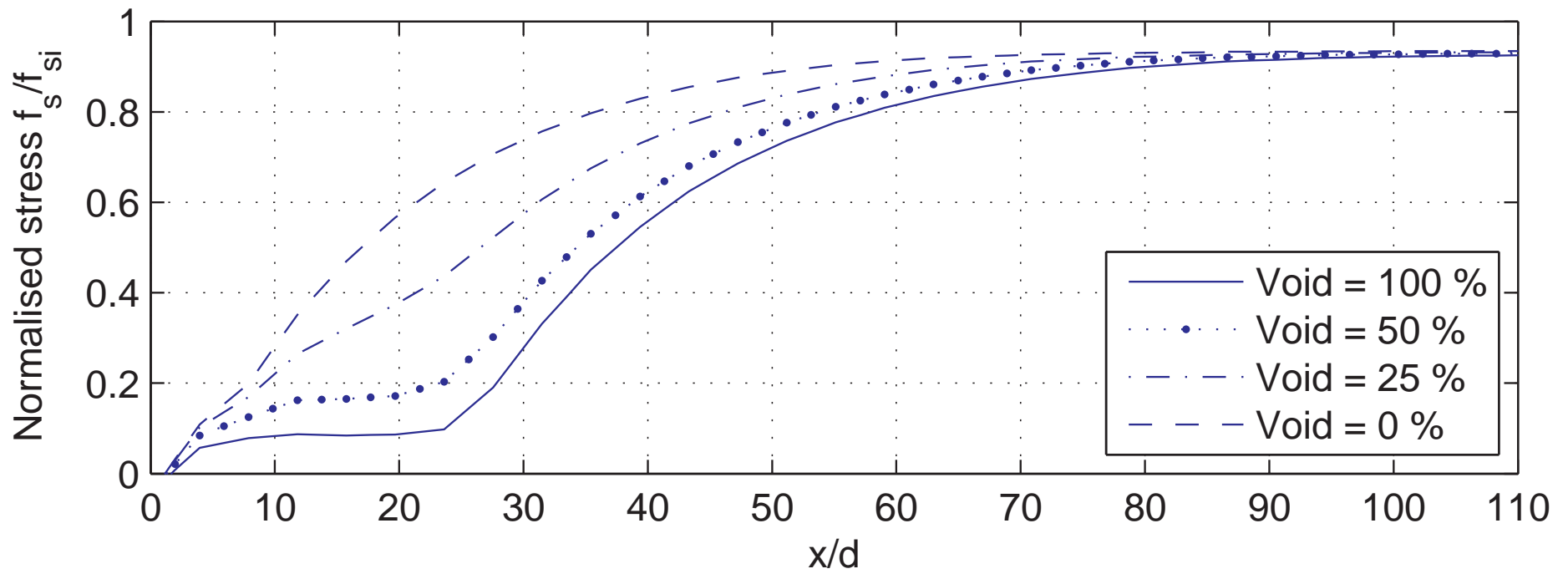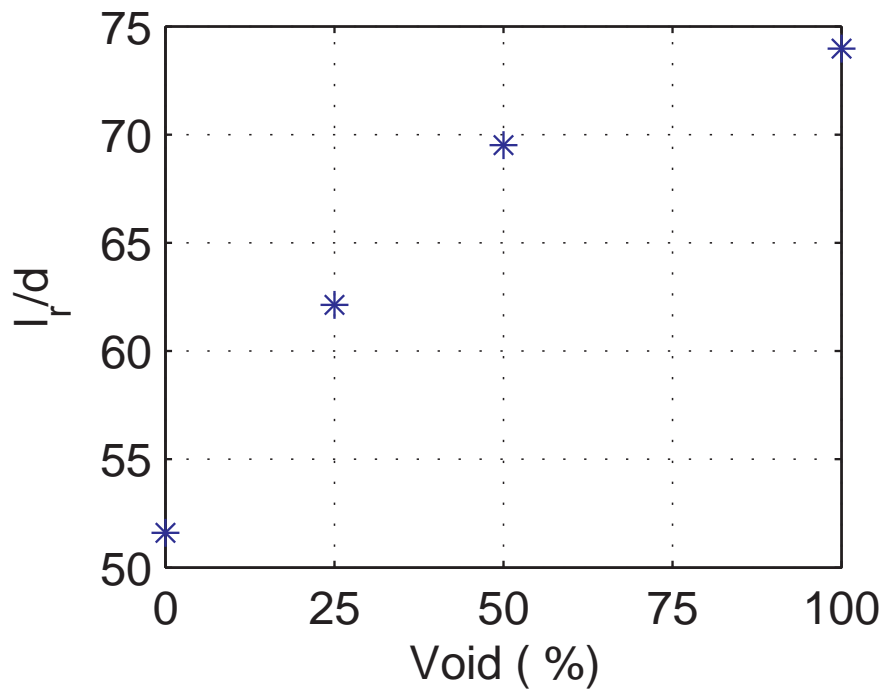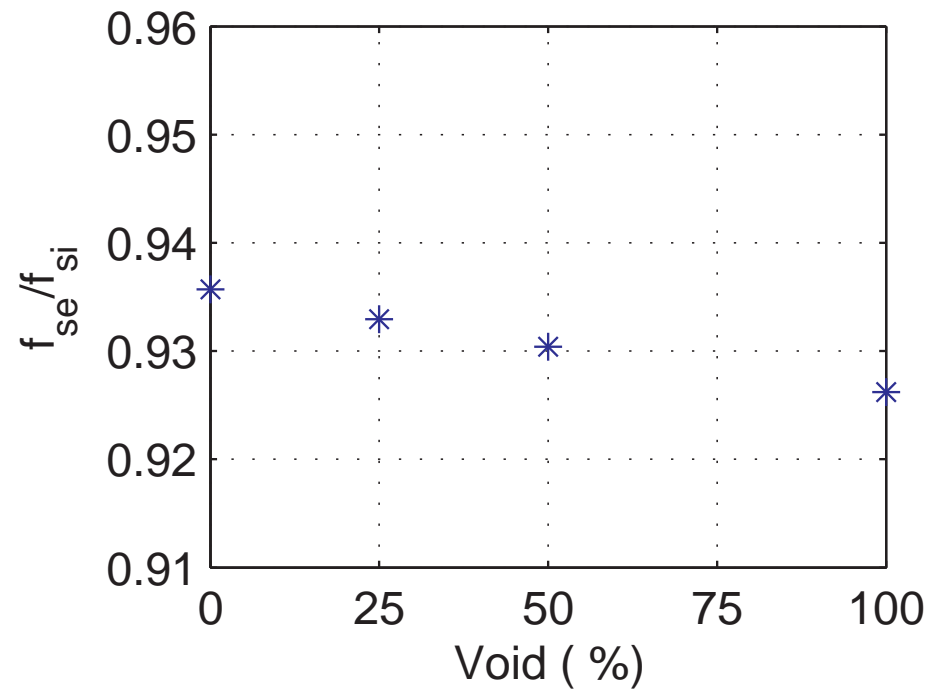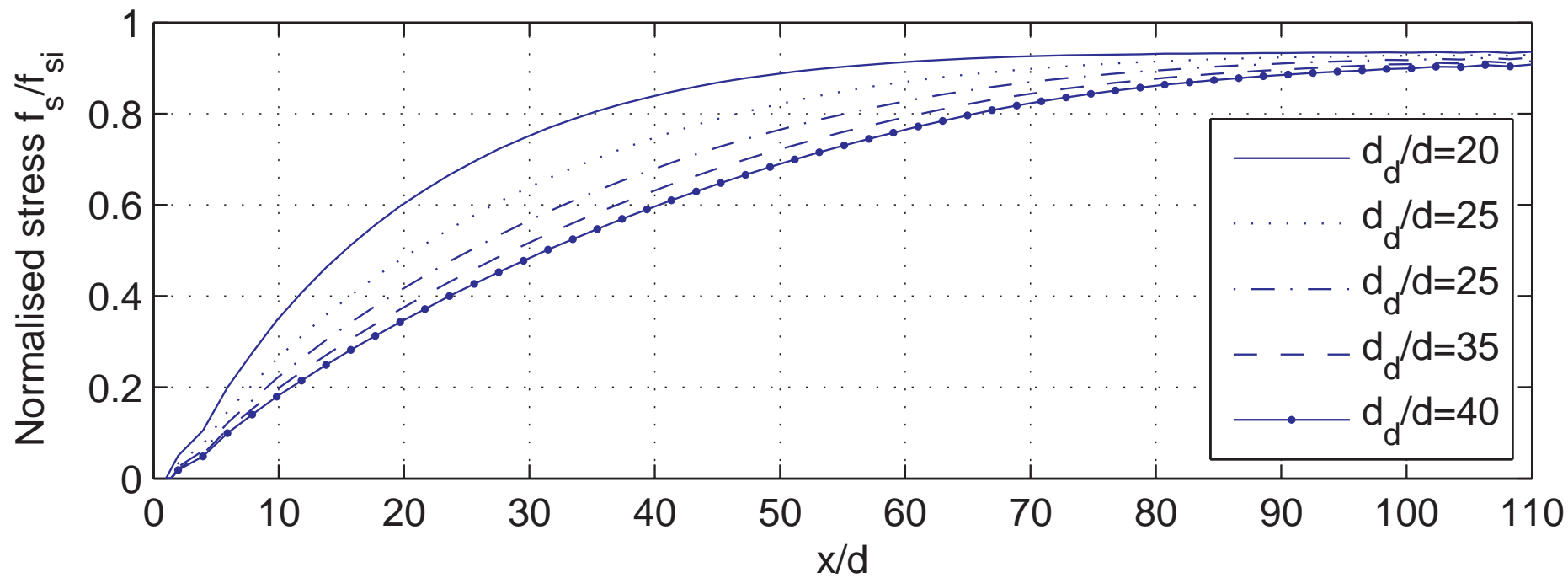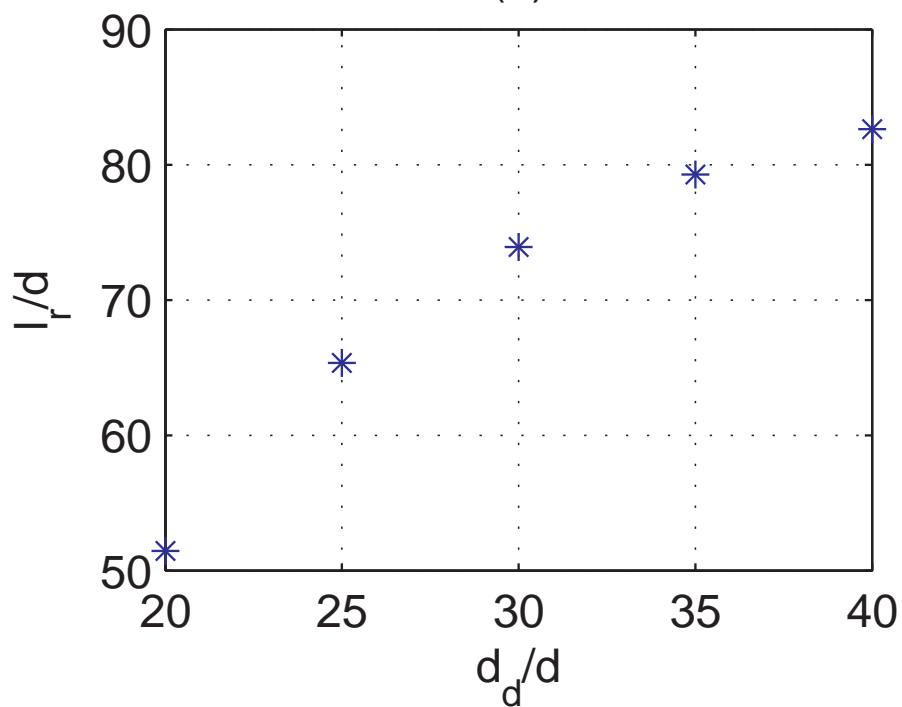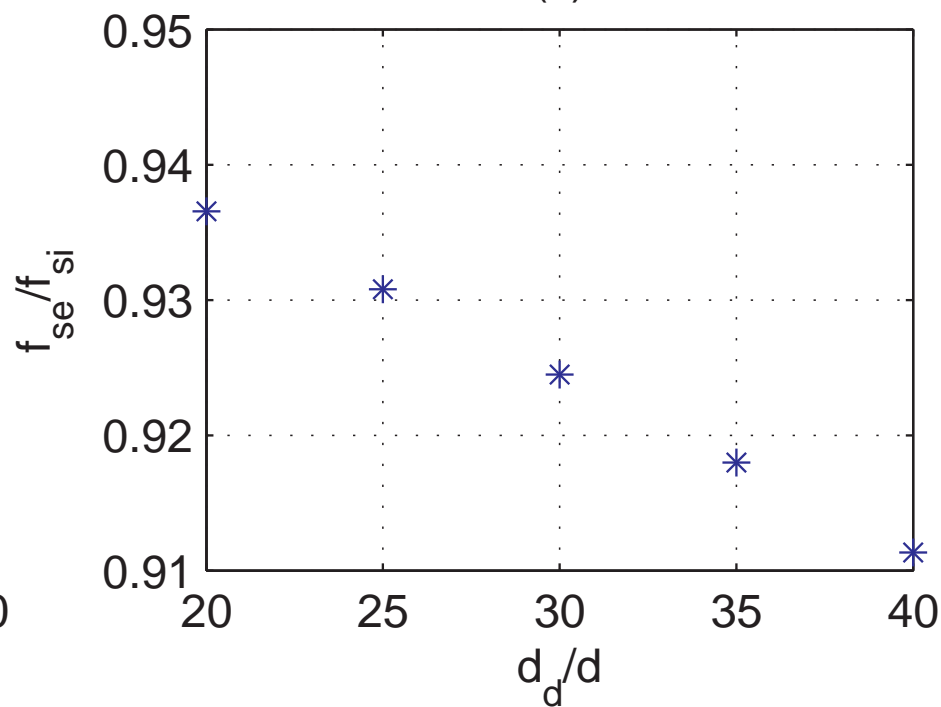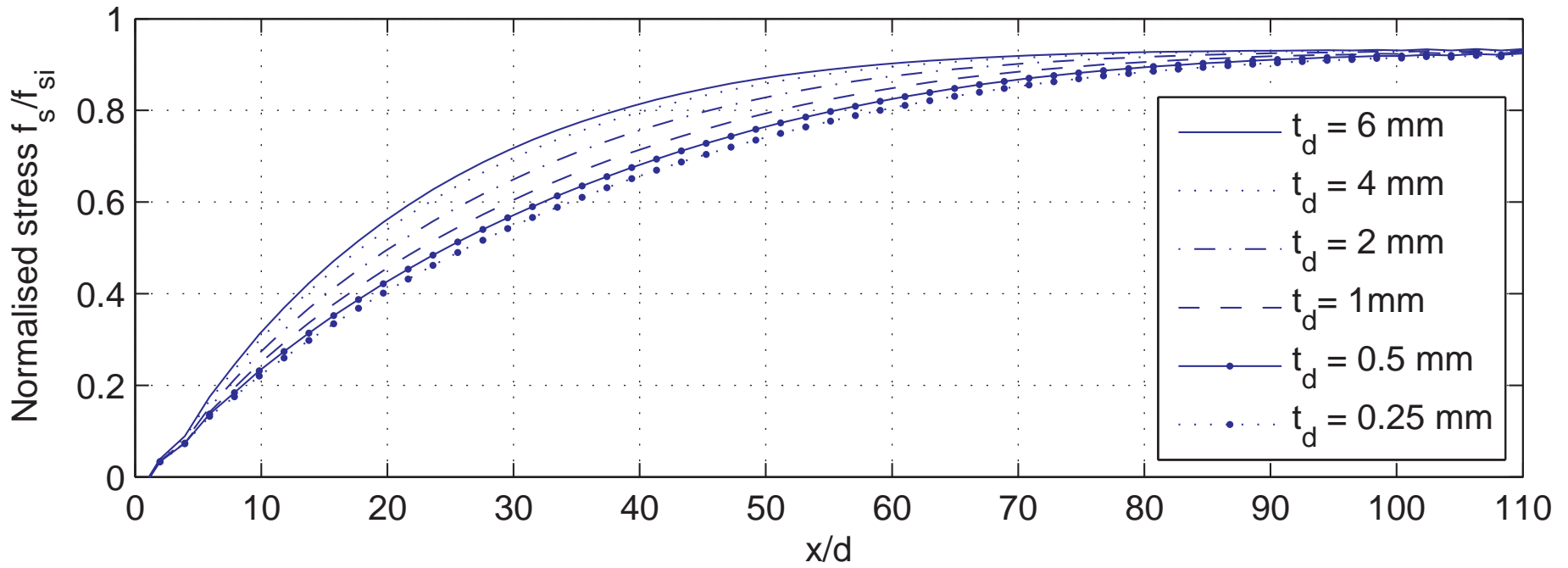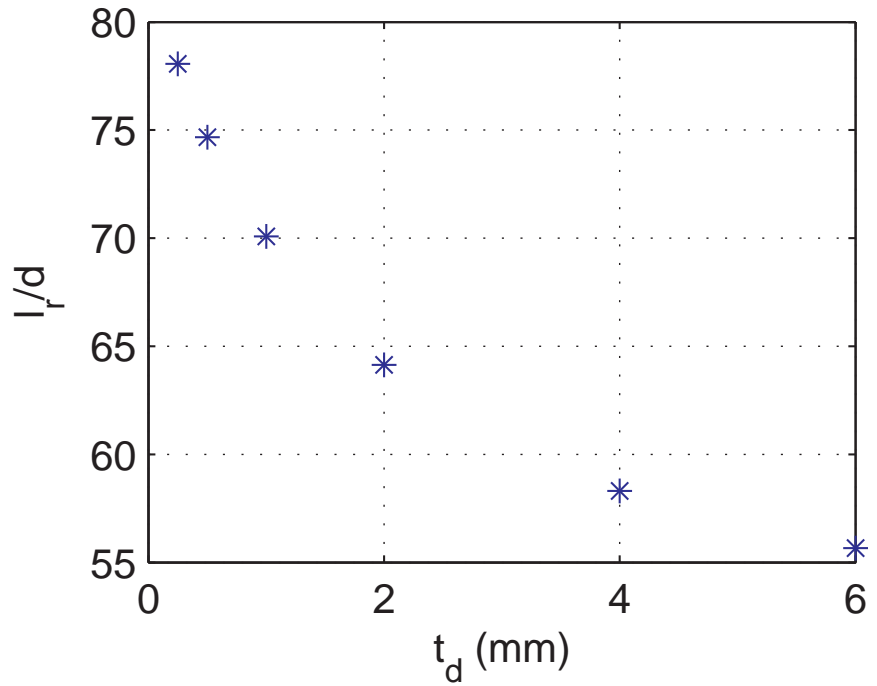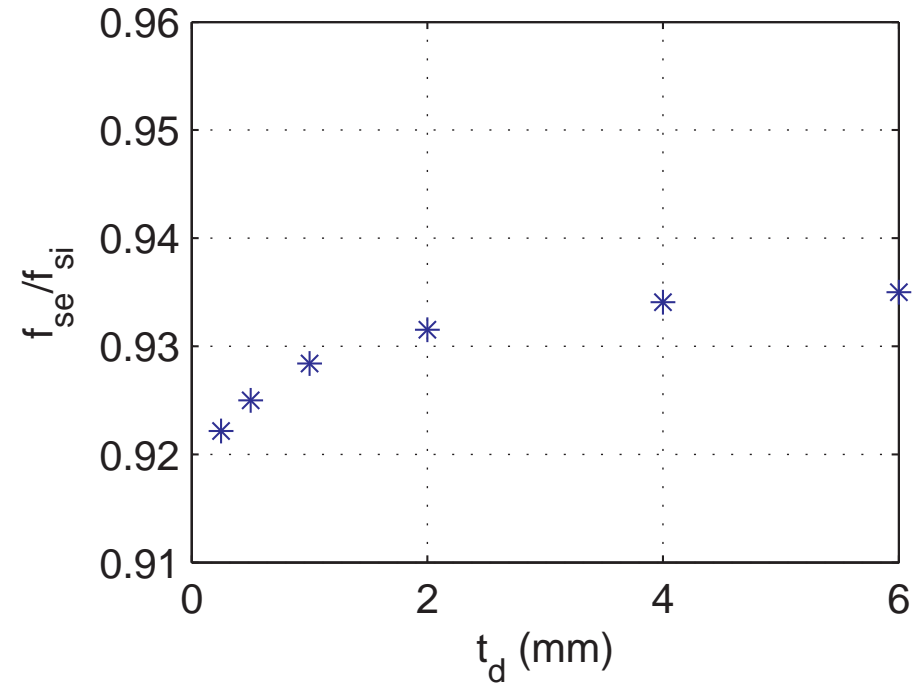
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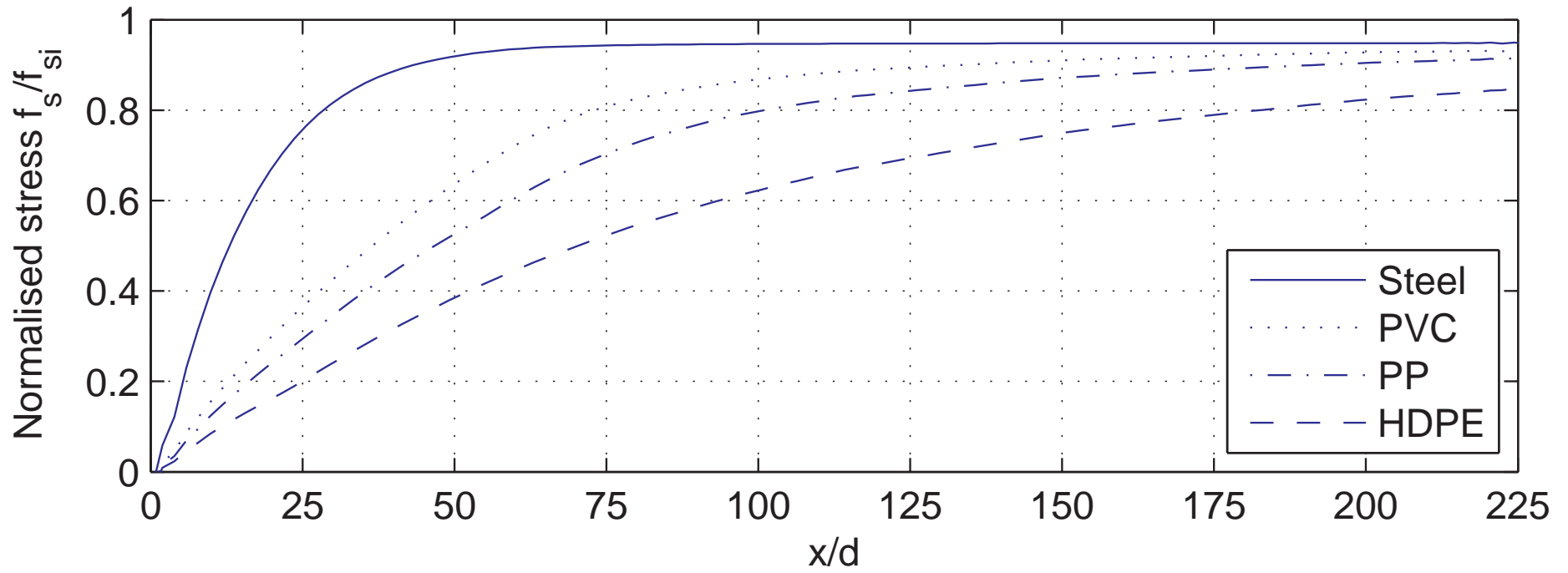
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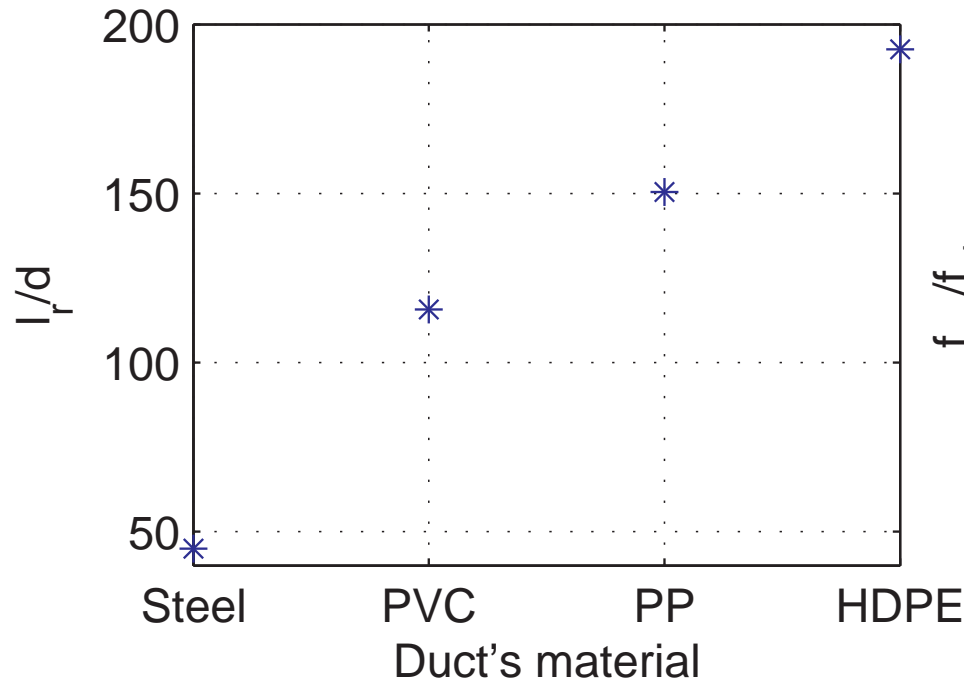
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(a)

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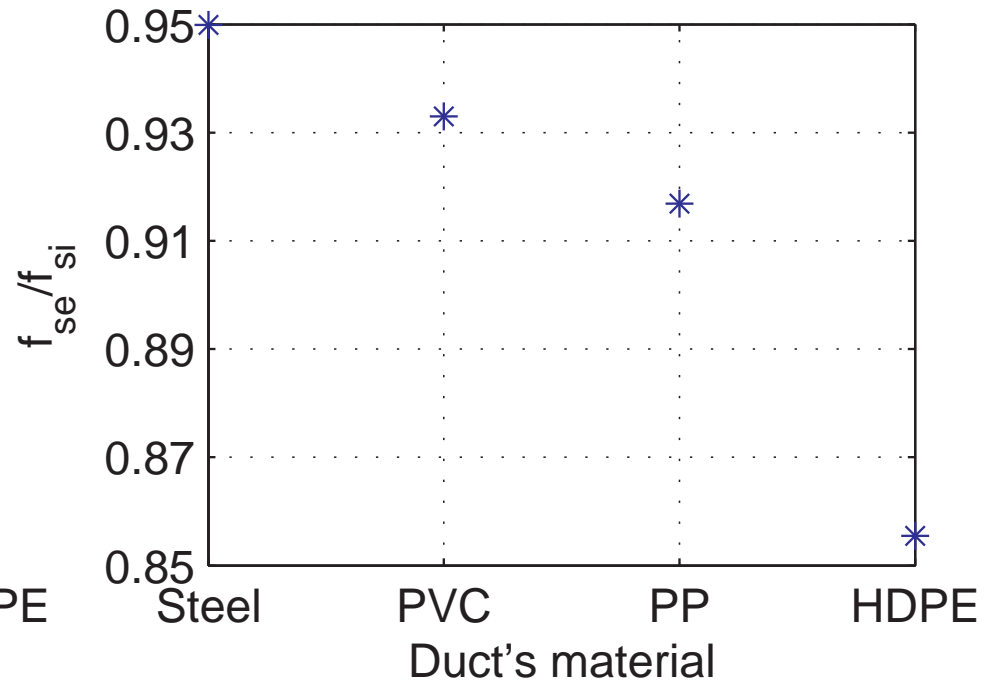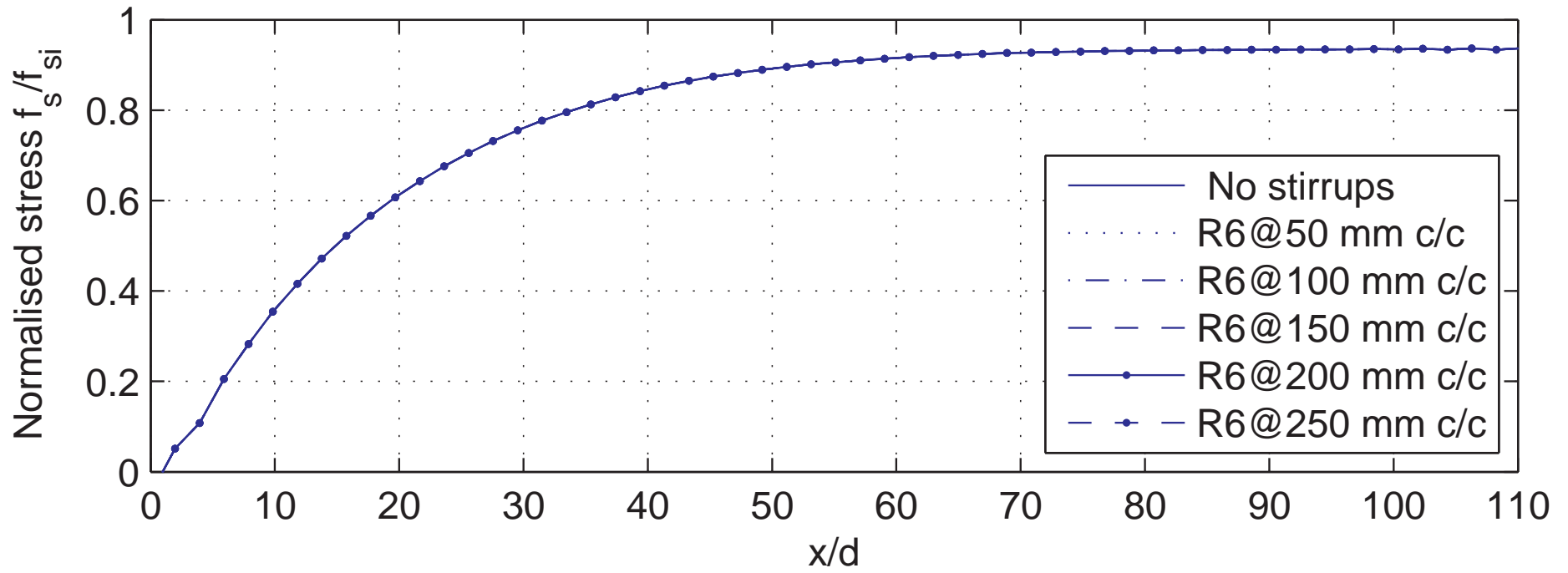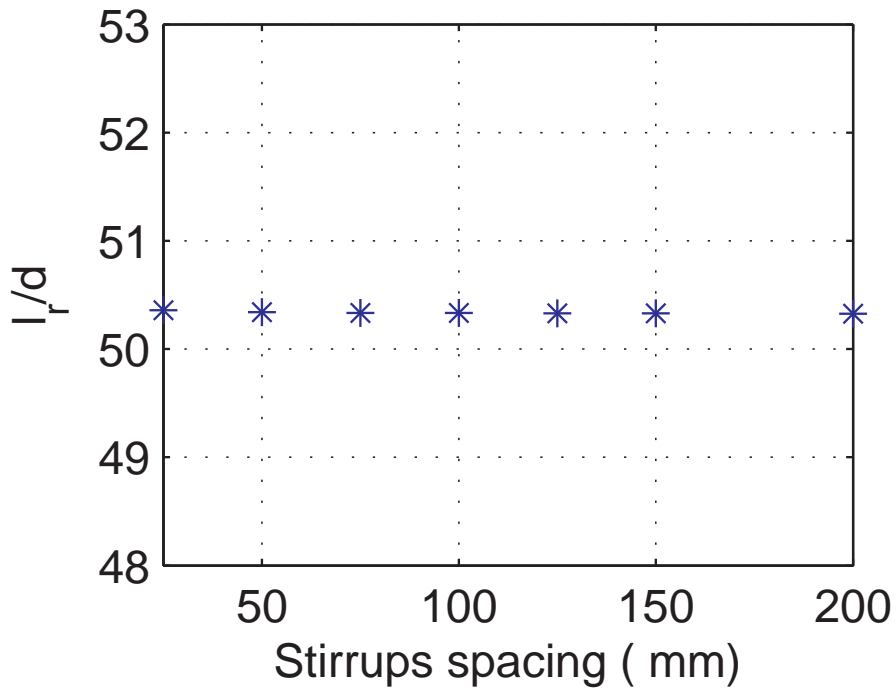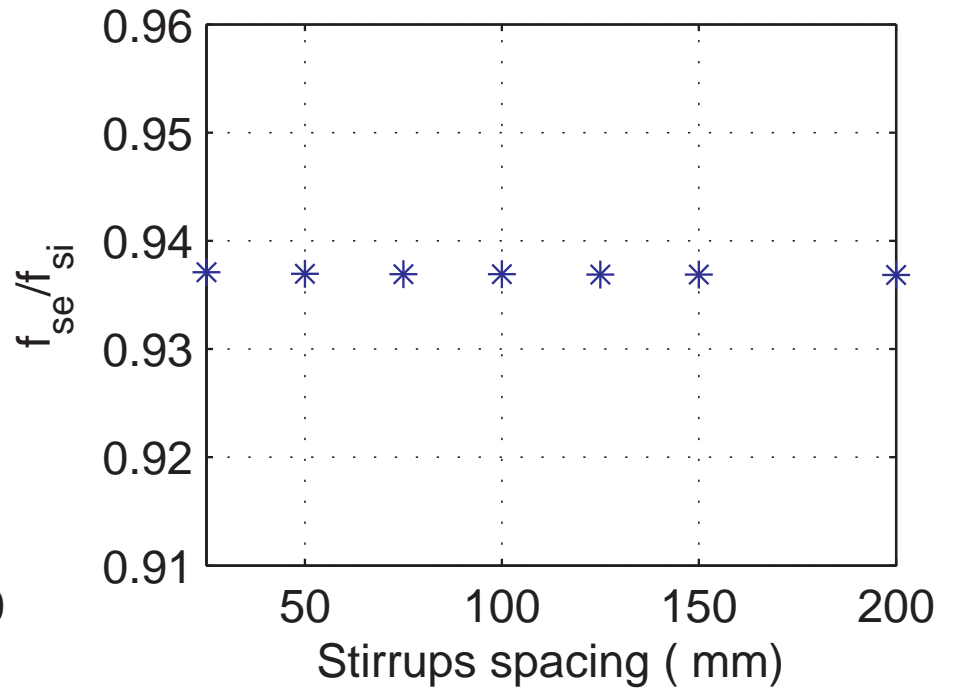
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Modelling and parametric study of the re-anchorage of ruptured tendons in bonded post-tensioned concrete

A. O. Abdelatif[1] , J. S. Owen[2] and M. F. M. Hussein[3]

Jennifer Chapman
Editorial Coordinator
ASCE's Journal of Structural Engineering
26 May 2017

Dear MS Chapman,

Re: Decision on Manuscript MS STENG-5593R1 - [EMID:4b1d9ef1e0025fec]

The authors appreciate the final acceptance of the manuscript by reputable ASCE's Journal of Structural Engineering. The comments address by the reviewers have been responded as follows:

Reviewer #1: The authors addressed the reviewer's comments properly. I recommend its publication in this journal.

The authors thankfully appreciate the Reviewer #1 decision.

Reviewer #3: The authors have addressed the reviewers' comments and I recommend the paper be accepted. Some minor editorial changes are recommended.

The authors thankfully appreciate the Reviewer #3 decision. The minors comments response are given below:

- *Same sentence is repeated on lines 159-160 and 164-165:* The sentence in lines 159-160 is deleted.
- *Provide citation and/or brief explanation of Highway Agency Model on line 310:* Citation of the Highway Agency model is provided as recommended.
- *Sentence that starts "These voids" on line 391-392 is unclear and should be rewritten:* The sentence is re-written as follows: Voids of 200 mm length (15.75d) were introduced in the grout to investigate the impact of grout voids on the re-anchorage mechanism.

Moreover, as required, all text in the manuscript is turned in black font. Also, the space of the reference list is doubled.

Yours sincerely,
Amged O. Abdelatif, Dr.
Civil Engineering Department
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Khartoum, Sudan
On behalf of Dr. John Owen and Dr. Mohammed Hussein

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