Journal of Structural Engineering

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

--Manuscript Draft--

Manuscript Number:	STENG-5593R2
Full Title:	Modelling and parametric study of the re-anchorage of ruptured tendons in bonded post-tensioned concrete
Manuscript Region of Origin:	UNITED KINGDOM
Article Type:	Technical Paper
Section/Category:	Concrete and Masonry Structures
Funding Information:	
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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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]

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.

Keywords

Post-tensioned concrete; Re-anchorage; Corrosion; Tendon; Bond, Modelling; Finite element; 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 the understanding of the behaviour of bonded post-tensioned concrete structures with ruptured tendons and facilitate the prediction of their residual structural capacity.

This paper presents the background of bond in post-tensioned concrete in Section 2. Section 3 presents a 3D non-linear FE model for the re-anchorage of a ruptured tendon. Section 4 shows the experimental procedure used in the validation of model, which is used to validate the numerical model in Section 5. Section 6 presents a parametric study, which was performed using the developed FE model. Conclusions are summarised in Section 7.

2. Bond in grouted post-tensioned concrete

Bond between steel and grout is attributed to adhesion between steel and grout, friction between steel and grout, and the mechanical resistance. The adhesion always has an insignificant influence on the load-deformation response of the structure, because the adhesion fails after a very small relative slip (Marti et al., 2008, Cairns et al., 2007). The mechanical resistance only contributes to bond when deformed steel bars are used. However, when smooth strands are used, a strand slips through the grout following the pre-shaped groves without shearing off the concrete (fib, 2000, Abdelatif et al., 2015). Therefore, the friction between steel and grout is largely responsible for the transfer of stress into the surrounding material. In the following paragraphs, the literature on bond in grouted post-tensioned concrete is chronologically presented.

In 1960s, experiments on 19 beams were conducted to investigate the effect of grout properties on the re-anchorage length, indicated as “transmission length”, and the structural behaviour of bonded post-tensioned concrete beams (Geddes and Soroka, 1963, Geddes and Soroka, 1964). The investigations showed that the “transmission length” is independent of time, but depends on the compressive strength of the grout.

In controlled demolition tests in the eighties, important results about structures suffering from rupture of tendons were revealed (Buchner and Lindsell, 1987b, Buchner and Lindsell, 1988). These results were: 1) the ruptured tendon is able to re-anchor into surrounding material over a certain length known as the *re-anchorage length*; 2) the re-anchorage depends on the grout condition, friction between individual wires or strand within a tendon, and the level of confinement provided by shear links.

The Model code 1990 (MC90) introduced a model for bond in post-tensioned concrete. It is based on bond between concrete and the outer surface of tendon sheathing (CEB-FIP MC90, 1990). In reality, however, the bond actually develops between pre-stressing steel and grout rather than between pre-stressing steel and concrete. This model for bond in post-tensioned concrete was removed in the latest version of the model code MC2010, (fib, 2010).

The UK Highways Agency, in DMRB BA51/95, has proposed a conservative empirical relationship to estimate the re-anchorage length of a ruptured bonded post-tensing tendon (Highways Agency, 1995). It modified the BS5400 transfer length model (BS 5400, 1990) for *pre-tensioned* concrete to account for multi-strand tendons. The model assumes linear distribution of tendon stresses over the re-anchorage length.

In 2001, FE and experimental investigations were conducted to study the dynamic bond characteristics of smooth pre-stressing bars that are embedded in grout (Belhadj and Bahai, 2001). The study demonstrated the importance of friction in controlling the slip of pre-stressing steel in grout. In another study in 2008, the influence of emulsfiable oil, which is used as a temporary corrosion protection, in bond behaviour in bonded post-tensioned concrete was investigated. The results showed a reduction on bond shear stress for treated strands compared to untreated ones (Marti et al., 2008, Luthi et al., 2008).

An empirical relation was proposed in 2011 to calculate the re-anchorage length, which is referred to as "the size of the stress-decreasing region", based on experiments on nine beams subjected to tendon cutting (Watanabe et al., 2011). The proposed relation assumed constant bond stress over the re-anchorage length, which gives a linear stress distribution. This assumption contradicts with the findings of other experimental observations (Geddes and Soroka, 1964, Coronelli et al., 2009).

Most of the reviewed literature that addresses the bond of post-tensioned tendons aims to study the impact of certain parameters on the bond mechanism rather than developing a model for the re-anchorage of the tendon. Therefore, in 2012, the authors developed an analytical model based on the linear thick-wall cylinder theory and the Coulomb friction law to estimate the stress distribution in the tendon after the rupture and hence the re-anchorage length (Abdelatif et al., 2012), Eq. (1).

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

Here f_s is tendon stress at distance x from the rupture point, α is a factor to account for voids in the grout, μ_s and E_s are the Poisson's ratio and the Young's modulus of steel. A and B are coefficients depending on geometry and material properties of steel, concrete, grout and duct that can be calculated as shown in (Abdelatif et al., 2012). Eq. (1) estimates the re-anchorage length when the stress in the pre-stressing steel (f_s) is substituted by the effective pre-stress (f_{se}).

The model has been verified using an axisymmetric FE model and validated experimentally (Abdelatif et al., 2013). The results of axisymmetric and analytical model show that the stress on the grout might exceed the tensile strength. Therefore, non-linear material behaviour 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 al., 1976) and assuming linear tension softening. More details about the CDP parameters used are discussed elsewhere (Abdelatif et al., 2015).

3.1.2 Grout

The grout material was modelled in a similar way as the concrete elements. According to EN447, a water cement ratio w/c of 0.4 is recommended for grout in prestressing tendons. For this w/c, the values of fracture energy, G_f, were found to be in the range of 22-26 N/mm (Padevět and Zobal, 2011). In this study the averaged value of 24 N/mm was used.

3.1.3 Prestresing steel, duct, and anchor plates

Post-tensioning tendons are usually stressed to a level below the yield stress. Therefore, tendon, steel duct, and anchor plates are modelled using linear elastic material model adopting the idealised stress-strain constitutive model in both tension and compression (CEB-FIP MC90, 1990). In this study, the Young's modulus for steel material was assumed to be 200 GPa.

As noted in section 2, friction is the principal means of transferring stress from the tendon into the surrounding material for both smooth strands and wires. Therefore only the frictional component was considered in modelling bond in this work. The pressure in the frictional component is generated by the radial expansion of the tendon after rupture as a result of the Poisson effect, Abdelatif et al., 2015. In this work, only tendons made from a single wire are considered to avoid the meshing problems that would occur for a helical strand. For a given change in pre-stress the Poisson effect will give the same change in diameter for both strand and wire. However, for the strand there is an additional effect that resulting from the mechanical interlocking of the wires making up the strand. Therefore, modelling the tendon as a wire will provide a lower bound on the bond and hence a conservative estimate of the re-anchorage length.

3.2 Simulation of post-tensioning process

To simulate the interface between steel-grout, grout-duct, and duct-concrete surface-to-surface contact elements were used. The tangential behaviour between steel and grout was modelled using Coulomb friction law with zero cohesion and 0.4 coefficient of friction (Abdelatif et al., 2012, Abdelatif et al., 2015, fib, 2000). Note that the type of corrosion usually found in post tensioned structures is the pitting corrosion, which is localised and occurs in grout voids due to the presence of chlorides and moisture. Therefore, in this study, the coefficient of friction between steel and concrete close to the tendon fracture was assumed to be unchanged by corrosion. Other contact interfaces (i.e. grout-duct, duct-concrete and end anchors-concrete) were considered to be fully bonded.

The slip between two paired nodes at contacted interfaces is considered to take place when the tangential friction exceed the static friction and the relative tangential displacement exceeds the specified tolerance. In ABAQUS by default, the tolerance set to 0.5% of the average length of all contact elements in the model (Dassault Systemes Simulia, 2010).

The normal and tangential contact behaviours were solved using an augmented Lagrange multiplier algorithm and Penalty method, respectively.

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

1) The prestressing was modelled by applying an initial stress equivalent to the magnitude of prestress on steel elements. The model was then solved in this first step to allow transfer of prestress from steel to concrete through the end anchorage. In this step, the grout elements, contact between post-tensioning steel and grout, and contact between grout and duct are deactivated (no stress on grout in this step).
2) The grouting process was then simulated in the second solution step by activating the grout elements and grout's contact elements with steel and the duct. Note that the

grout element remain un-stressed throughout this step. This is because the contact elements can be stablished between any two paired of nodes if they are within the predefined tolerance without displacing the nodes.

3) In the third step the tendon rupture was simulated. The rupture of post-tensioning steel was modelled by deactivation steel elements at the location of the rupture, and then the model is solved for equilibrium. This simplification is supported by the fact that the corrosion causing rupture is localised and occurs in the form of pitting. With more localisation and high rate of corrosion, the pitting mechanism may produce points of stress concentration. Eventually this may lead to a sudden brittle failure after a negligible loss of material.

The model was solved utilising a full Newton-Raphson solver under the static condition, with un-symmetric matrix storage to handle the contact solution. A small time step size, and therefore, a large number of increments were used to promote the convergence of the complex non-linear material behaviour and the solution of the contact problem.

3.3 Mesh sensitivity

The mesh sensitivity study was conducted for a bonded post-tensioned concrete prism (Fig. 1) in order to find an optimum spatial discretization. The study shows that, 32 segments around the tendons and 10 mm element size in the longitudinal direction satisfied the mesh sensitivity investigations, Fig. 2. The influence on the model result for elements smaller than this was found to be insignificant (Fig. 3).

4. Experimental work

A number of laboratory investigations have previously been carried out to study the bond behaviour between grout and the post-tensioning tendons, as well as the quality performance of the grout (Marti et al., 2008, Minh et al., 2007, Minh et al., 2008). However, only limited experiments were conducted to understand the re-anchoring phenomenon of ruptured tendons.

In this section, a brief review of previous tests about measuring techniques, rupture simulation, and the size of test specimen will be given.

In previous studies on monitoring strain changes after tendon rupture/release on post-tensioned concrete, two types of strain gauges were used: the demountable mechanical (DEMEC) and the electrical gauges. The DEMEC strain gauges were used to investigate the effect of grout properties on transmission length on nineteen bonded post-tensioned concrete beams (Geddes and Soroka, 1964). The electrical strain gauges were usually attached on the concrete surface to (Schupack and Johnston, 1974, Buchner and Lindsell, 1987a, Buchner and Lindsell, 1988) or attached to the pre-stressing steel (Coronelli et al., 2009, Watanabe et al., 2011). However, the tendon slippage after the rupture might break the attached electrical strain gauges (Coronelli et al., 2009).

Because the simulation of the corrosion process is time demanding (Jeyasehar and Sumangala, 2006), the corrosion rupture was simulated by releasing the prestress force at the location of rupture. This can be done either by using nuts to simulate the rupture at the end (Geddes and Soroka, 1964) or by cutting the tendon through a prefabricated hole using a flame torch (Schupack and Johnston, 1974) or saw (Coronelli et al., 2009, Watanabe et al., 2011).

Different sizes of specimens were used in previous experimental investigations. Experiments were conducted on real bridge girders (Buchner and Lindsell, 1987a, Buchner and Lindsell, 1988), laboratory beams with large cross section (Schupack and Johnston, 1974) and long spans (~10 m) (Tanaka Y. et al., 2001), and on 3-5 metre beams (Coronelli et al., 2009, Geddes and Soroka, 1964, Watanabe et al., 2011).

The drawbacks of the previous experiments on the behaviour of the ruptured tendon can be summarized in three points:

- The use of electrical resistance strain gauges or DEMEC gauges restricts the number of the measuring points based on the specimen length. This will influence the prediction of strain profile along the tendon
- Simulating tendon rupture by saw cutting might disturb and damage the grout.
- Conducting experiments on large beams or in a real bridge beams restricts the number of parameters to be varied for practical and cost issues.

These drawbacks can be resolved by measuring the full-field displacement (instead of strain gauges) after the rupture on small prisms (instead of beams).

In this study, the 3D Electronic Speckle Pattern Interferometry (ESPI) system was utilised to measure the full field displacement on 500 mm long post-tensioned concrete prisms. ESPI has proven to be a valuable alternative to conventional displacement measuring techniques (Jones and Wykes, 1989). It provides considerably more information, such as full-field measurement, compared to the conventional method. The rupture of the tendon was simulated by undoing nuts of a special type of anchor (instead of using saw cutting or accelerated corrosion), which was designed for the purpose of the tests (Fig. 1). Undoing nuts also gives a sufficient number of steps/images to be capture by the ESPI system throughout the test.

4.1 Test setup

33 Concrete prisms of 500x100x100 mm with an embedded duct along the centre were cast and then post-tensioned using a single pre-stressing wire (Fig. 1**).** The wire was tensioned using a manual hydraulic prestressing jack. The prestressing load was controlled by means of a load cell and adjusted with a bespoke extended anchor system before grouting. The prisms were grouted vertically from the anchor side using a manual pointing gun.

As illustrated in Fig. 4, the post-tensioned concrete prism was then bolted down to a vibration isolation table and two rounded bars were inserted below the prism to minimize expected downward movement during the test (because of the sensitivity of the ESPI system to rigid body motion). Two different support setups were used in this test to investigate the influence of the support position on the tendon re-anchorage. In the first one, the prism was bolted down from the releasing end (live end), and in the second, the prism was fixed at the other end (dead end). This support boundary condition (bolting-down) was represented in the FE model by fixing the nodes at the bottom of the end-plate.

The rupture of the tendon at the end of the prism (i.e. anchor) was simulated by gradually releasing the nuts between the extended anchorage plates (Fig. 1**)** in many steps. A load cell was used to monitor the level of prestress during releasing steps and to record the tendon force before and after the test to eliminate the prestress losses. The full-field displacement at the concrete surface after prestress release was measured using ESPI, Fig. 4.

One of the intrinsic limitations to be avoided in ESPI measurements is the rigid body motion (RBM). The presence of rigid body motion (i.e. movement of the sample) locates the speckle pattern fringes away from the surface of the tested object (Jones and Wykes, 1989). Therefore, fringes can become unrelated to the surface displacement. This is a major disadvantage of the ESPI system, which limits its application. The correction of such errors might be difficult or impossible to account for and it is important to minimise this error by a suitable design of the testing setup. In this study, two LVDTs were used to monitor the prisms' rigid body motion. One third of the prisms used in this study were excluded because 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 ($c \leq 3\mu m$) 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).

Both of the analytical and 3D non-linear FE re-anchorage models show satisfactory prediction of the real behaviour (Fig. 5 to Fig. 8). In a few cases, the agreement at the end the prism is poor; this can be due to the fact that the intensity of the laser at the end is quite low. 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

In addition to the experimental work done as a part of this study, data from previous experiments on post-tensioned concrete beams were used to validate the proposed model (Geddes and Soroka, 1964). The beams had dimensions of 5 inch (127 mm) width by 6 inch (152.5 mm) height and were post-tensioned using a single 7/8 inch (22.225 mm) post-tensioning bar which was enclosed by 45mm/49mm steel duct filled with grout. The re-anchorage length was estimated at 95% of the average maximum strain (95% AMS) method (Russell and Burns, 1993, Abdelatif et al., 2012).

Fig. 9 shows normalised strain profile of the experimental data, 3D non-linear FE model, linear theoretical model, and Highways Agency model (Highways Agency, 1995). All strain data were normalised to average maximum strain. Good agreement between the proposed models (FE and analytical) with the experimental data are observed. The re-anchorage length is over predicted as 29% and 5% of the experimental value using the analytical and 3D FE model, respectively. The 29% is because of non-linear material behaviour is ignored while the 5% is regarded as acceptable. It is worth mentioning that the FE model runs in about three hours compared the milliseconds for the analytical model. In contrast, the Highways Agency model (DMRB BA51/95) overestimated the re-anchorage length by 125% from the experimental value assuming a linear stress profile which is clearly contradicted with the experimental results. The DMRB The DMRB BA51/95 overestimation of the re-anchorage length (and thereby underestimation of the residual capacity) could lead in practice to increased maintenance cost, traffic disruption and unnecessary replacement or strengthening work (Abdelatif et al., 2016).

5.3 Stress distribution across the concrete section

The von Mises contour plot on the cross section at the end of post-tensioned concrete beam after tendon rupture is shown in Fig. 10. It was found that, the stress contours in concrete tend to take a circular shape around the tendon. This finding supports the assumption of the thick-wall cylinder theory, which was used in development of the analytical model in Equation (1) (Abdelatif et al., 2012) and are in line with previous findings (Abdelatif et al., 2015).

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

Using the 3D non-linear FE model, the influence of different parameters on the re-anchorage of the ruptured tendon can be assessed. In this parametric study, the influence of the diameter and surface roughness of post-tensioning steel, diameter, thickness and material of sheathing (i.e. duct), initial prestress, concrete strength, grout strength, voids in the grout, and shear links, on the re-anchorage of a ruptured tendon are investigated. Unless stated otherwise, the beam that was used in this study has the properties shown in Table 2. The cross section of the beam was chosen from previous literature, (Coronelli et al., 2009), while the length was chosen to just occupy the re-anchorage length (to minimize the computational cost).

Re-anchorage length normalised to the diameter of the post-tensioning steel (l_r/d) and the effective prestress is normalised to the initial prestress (f_{se}/f_{si}) were considered as reference values in this study.

Studying the large number of parameters in this study was not possible without the aid of high performance computing cluster using eight CPUs with 16 GB RAM at the HPC unit in 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 located to start at 100 mm far from the end of the prism. The results show that, the presence of grout voids within the re-anchorage zone makes a significant change in the prestress profile and results in much longer re-anchorage length, Fig. 17a and Fig. 17b. If the beam is long enough for the tendon to re-anchor, no significant drop will occur in the effective prestress, Fig. 17c. The re-anchorage length of the voided tendons was approximated in a previous study by adding the void's length to the non-voided re-anchorage length (Cavell and Waldron, 2001). However this study shows, the re-anchorage length is increased by 67%, 114%, and 142% of the void's length for 25%, 50%, and 100% of void in the grout cross section, respectively.

6.7 Duct diameter (Grout thickness)

An increase in the diameter of the duct increases the grout thickness. Thicker grout allows the tendon to deform in the longitudinal direction more than the thinner grout. Therefore, tendons with a large duct diameter (thicker grout layer) were found to have a longer re-anchorage zone, a higher prestress and less bond stress in comparison with tendons with smaller duct diameter (thinner grout layer), Fig. 18.

6.8 Duct thickness

The practical range of duct thickness is 0.25 mm for steel ducts, 2.5-3.0 mm for polypropylene (PP) ducts, and 4.0-6.0 mm for high density polyethylene (HDPE) ducts (Hewson, 2003). A steel duct with outer diameter of 30 mm was used to examine the impact of ducting thickness on the re-anchorage behaviour. It is found that, increasing the duct thickness increases the level of confinement around the post-tensioning steel which results in a shorter re-anchorage length (Fig. 19). No significant influence is observed on the effective 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 agreement compared to experimental results in this study of post-tensioned concrete prisms with shear reinforcement as shown in Fig. 8e.

7. Conclusions

The paper has presented 3D non-linear finite element model for re-anchorage of a ruptured tendon in bonded post-tensioned concrete structures. The model was compared to previous analytical models and validated using experimental data on prisms, utilising the 3D Electronic Speckle Pattern Interferometry system (ESPI). The proposed model was also validated against previous experiments on beams and compared with the Highways Agency DMRB BA51/95 re-anchorage model. The developed model was found to be able to predict the re-anchorage length compared to the Highways Agency model DMRB BA51/95 which greatly overestimates the re-anchorage length. The findings related to the tendon re-anchorage (modelling and experiments) are:

- The experiments confirmed the re-anchorage of the ruptured tendon
- The 3D finite element is able to capture the re-anchorage phenomenon
- The developed models and experiments support the hypothesis that the re-anchorage phenomenon is influenced by the confining materials

The 3D FE non-linear re-anchorage model was then used in a parametric study to investigate the influence of prestressing steel diameter and surface roughness, duct diameter and thickness, duct material, initial stress, concrete strength, grout strength, presence of voids in the grout, longitudinal reinforcement, shear links, and number of strands in the tendon on the re-anchoring behaviour of the ruptured tendon. The results of this parametric study is 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	Setup-2	
	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		
	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	983													
	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

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

Figures captions

Fig. 1: Geometry of the test specimens (dimensions are in mm)

Fig. 2: Typical mesh arrangement for the prism models

Fig. 3: results of mesh sensitivity study

Fig. 4 Test setup: post-tensioned concrete prism with the anchorage system and ESPI camera on vibration isolated table

Fig. 5: Prism G2 full field displacement: a) ESPI data; b) FE results (μm)

Fig. 6: Prism C10 full field displacement: a) ESPI data; b) FE results (μm)

Fig. 7: Deformation at concrete surface after release: a) Prism DM2; b) Prisms C10, C20 and C30; c) Prism V3

Fig. 8: Deformation at concrete surface after release: a) Prism P5; b) Prisms C2 and C2; c) Prism G2; d) Prism CG1 and CG2; e) Prism ST1; f) Prism V2

Fig. 9: Strain changes at concrete surface after the rupture at the level of the tendon

Fig. 10: von Mises contours on a concrete cross section within the re-anchorage zone (MPa)

Fig. 11: Influence of tendon's diameter on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective prestress

Fig. 12: Influence of the tendon's surface roughness on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective prestress

Fig. 13: Influence of the initial stress on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective prestress

Fig. 14: Influence of the concrete compression strength on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective prestress

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

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

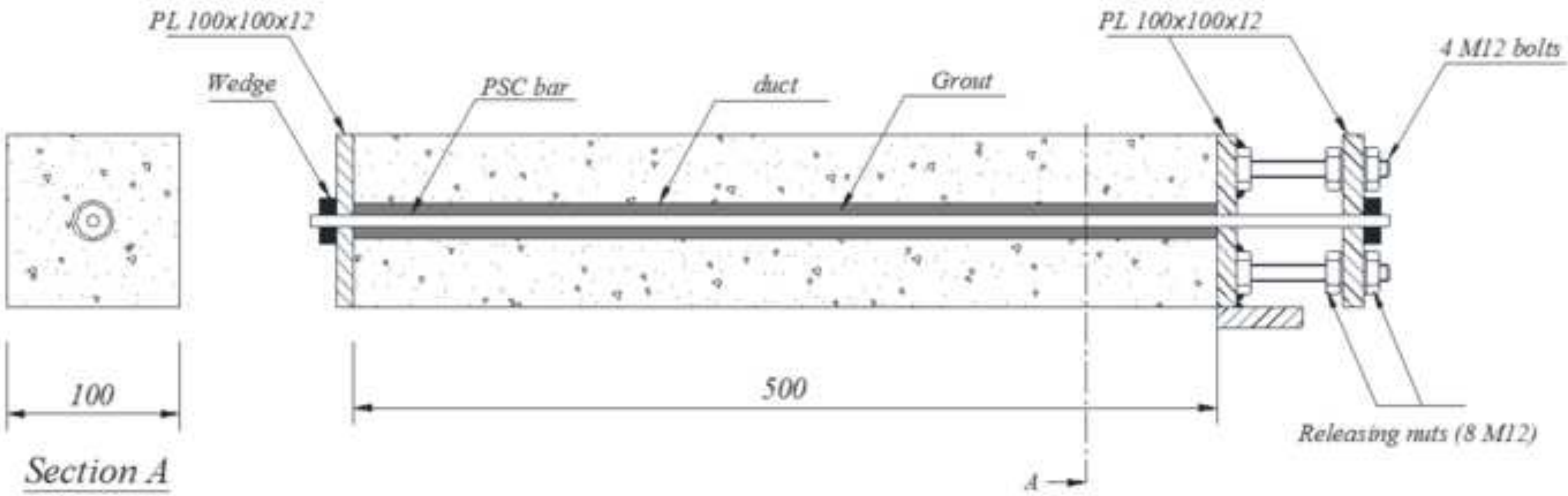
Fig. 17: Influence of grout voids on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective prestress

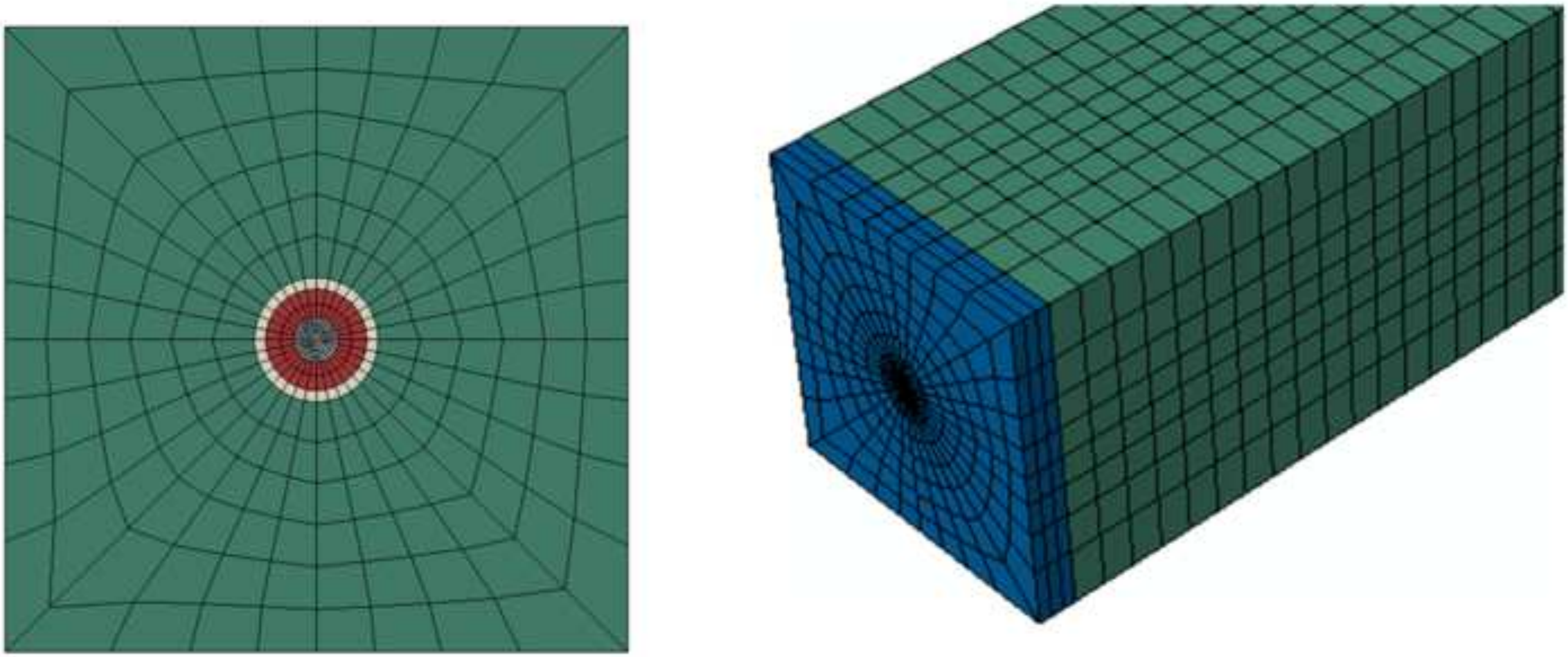
Fig. 18: Influence of the duct diameter and grout thickness on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective prestress

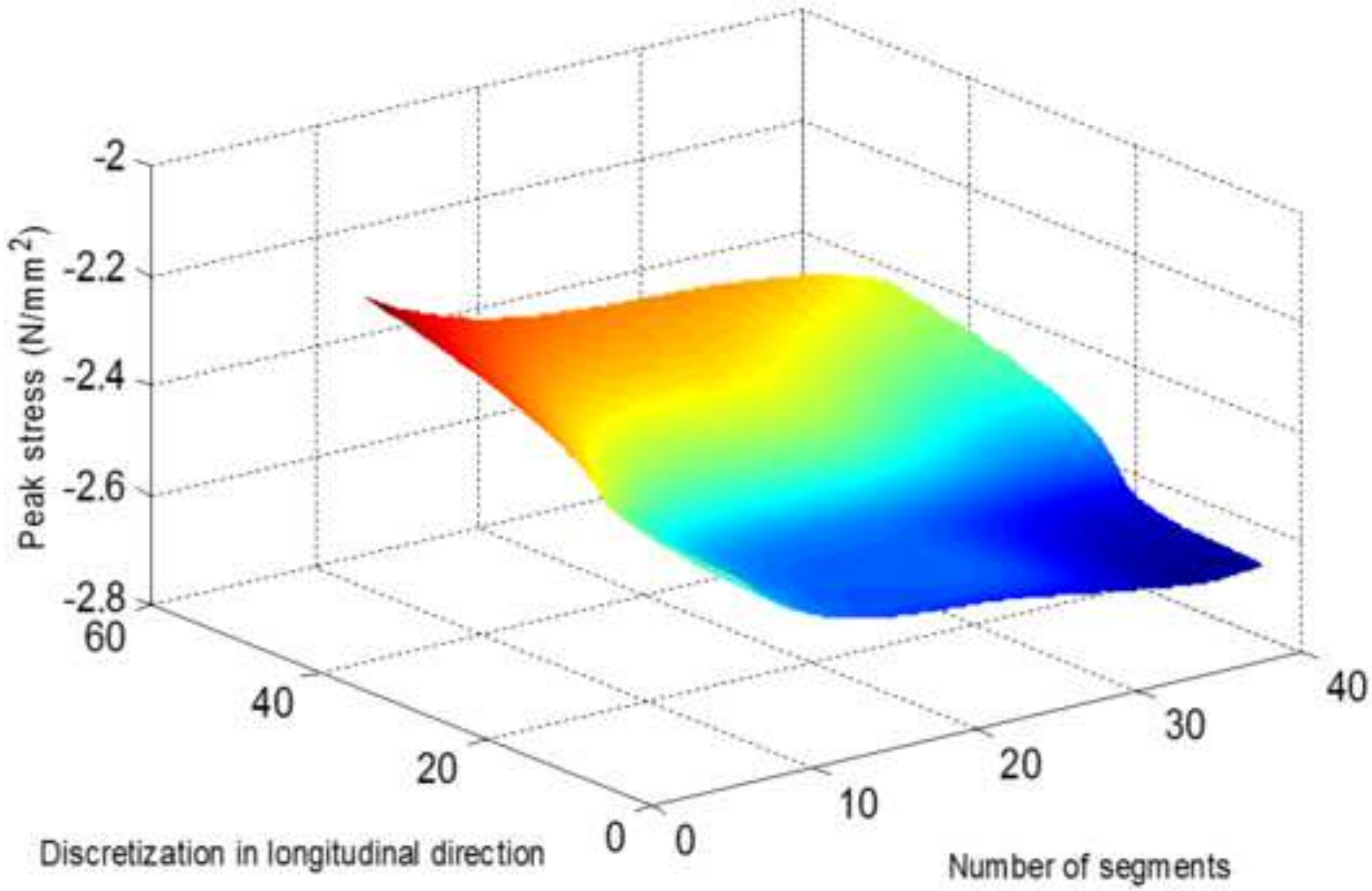
Fig. 19: Influence of the duct thickness on: (a) Stress distribution; (b) Re-anchorage length; (c) Effective prestress

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

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


PL 100x100x12
Wedge
PSC bar
duct
Grout
PL 100x100x12
4 M12 bolts
100
500
Releasing nuts (8 M12)
Section A
A





Peak stress (N/mm^2)
-2
-2.2
-2.4
-2.6
-2.8
60
40
20
0
Discretization in longitudinal direction
0
10
20
30
40
Number of segments

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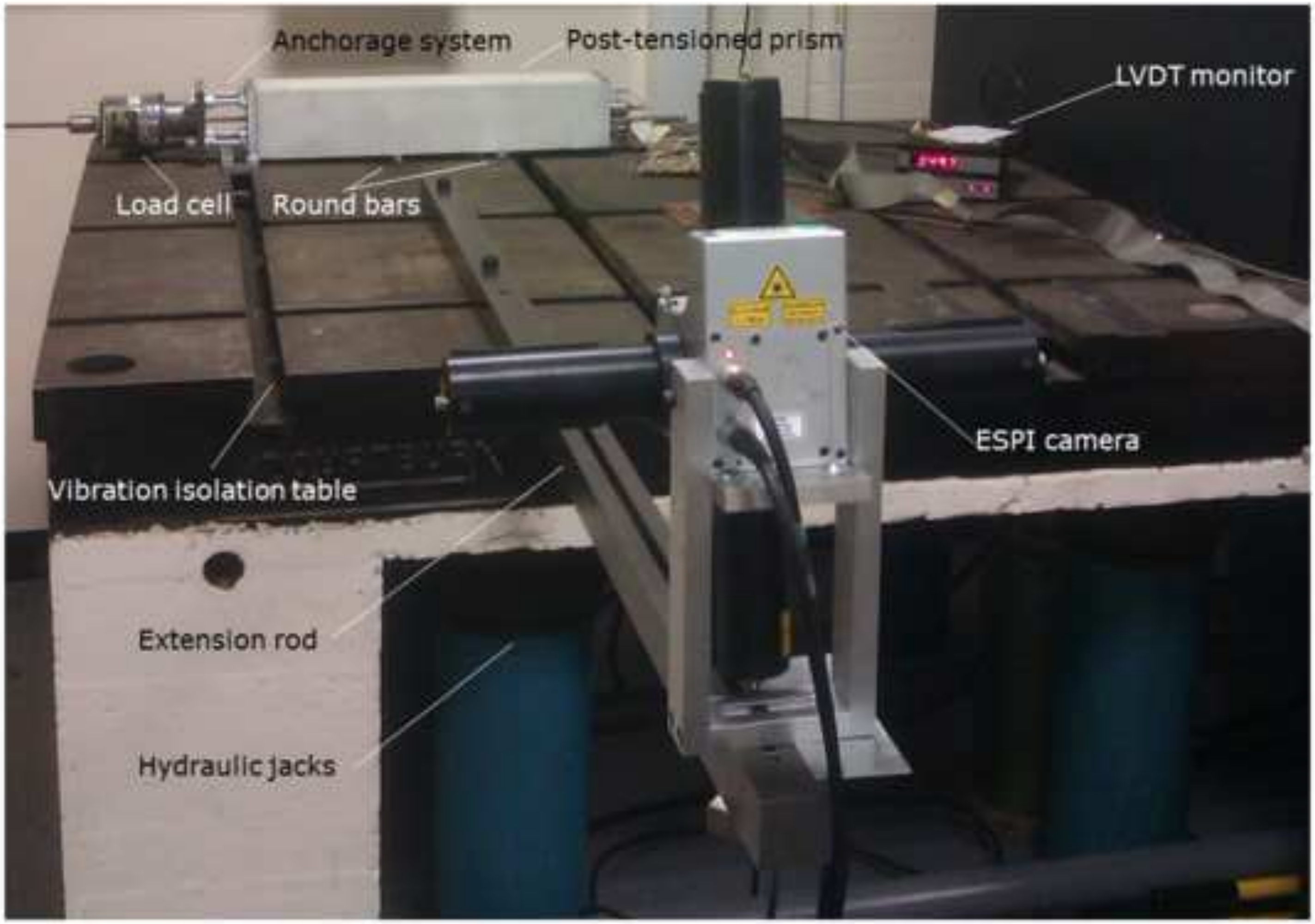

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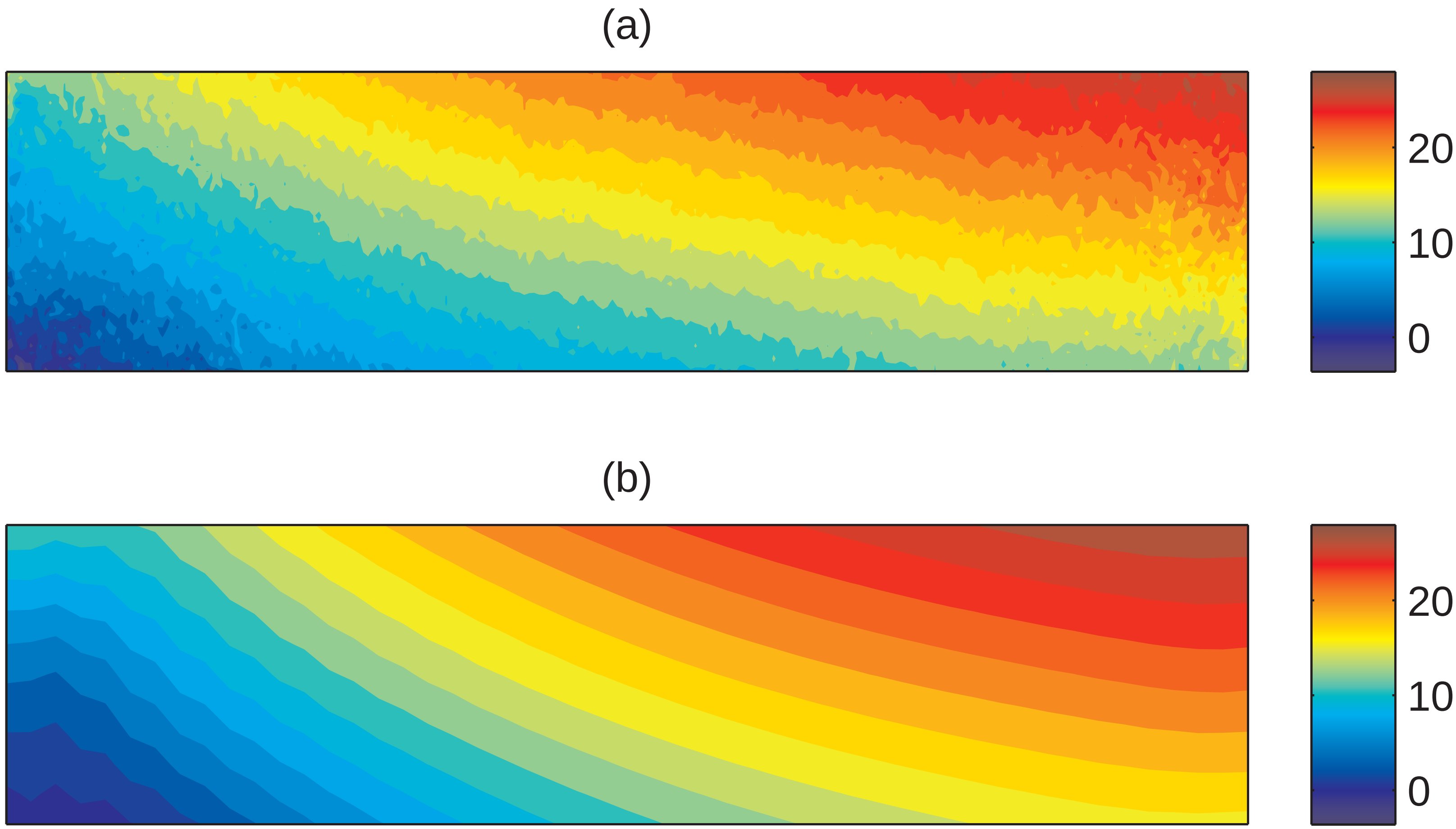
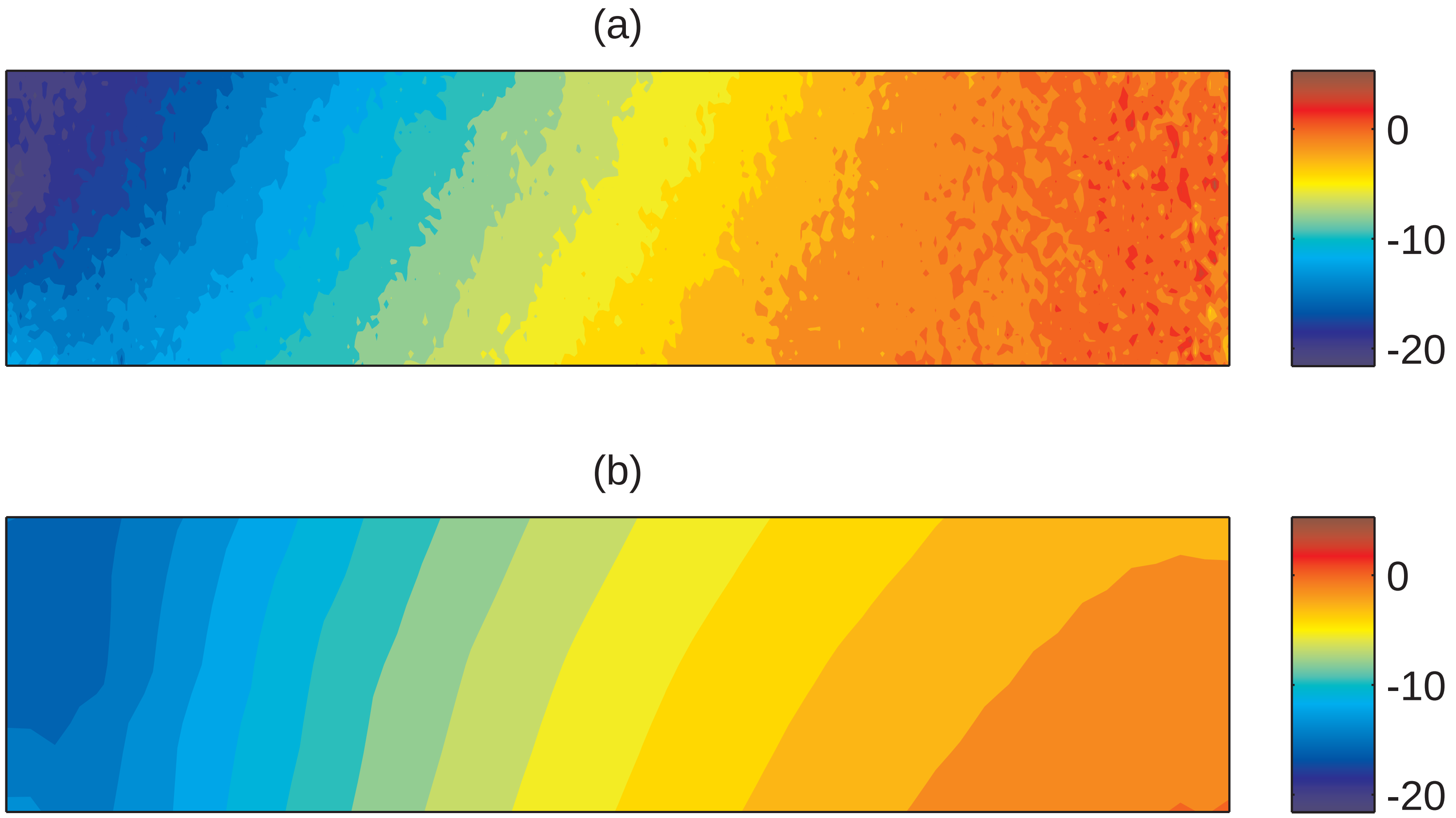
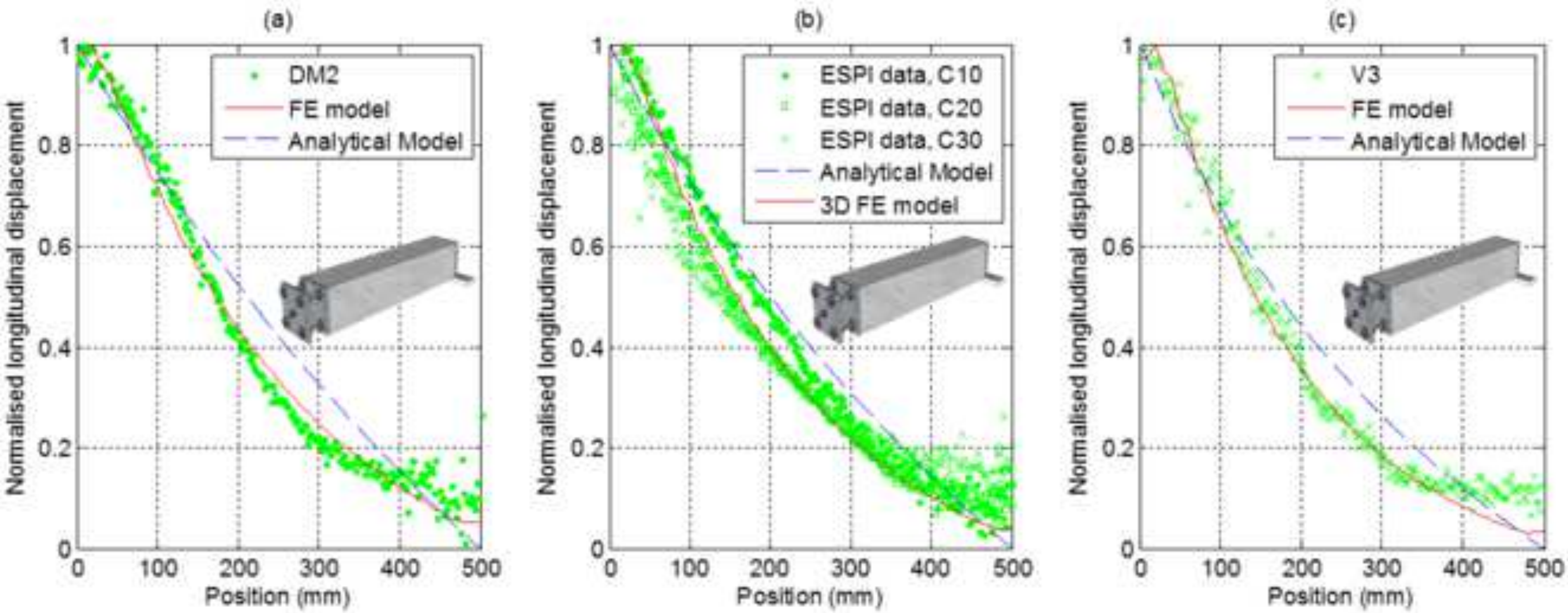
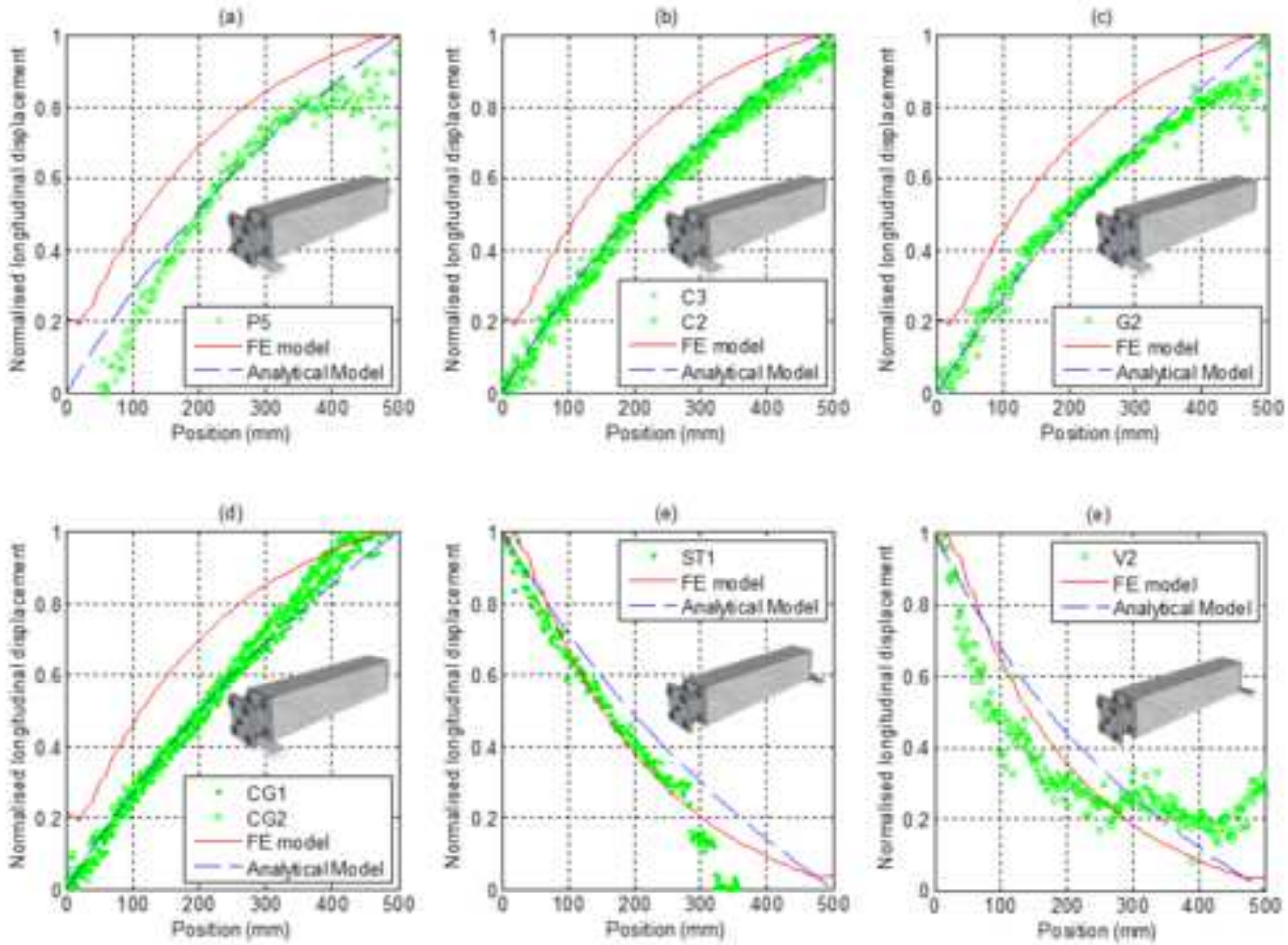
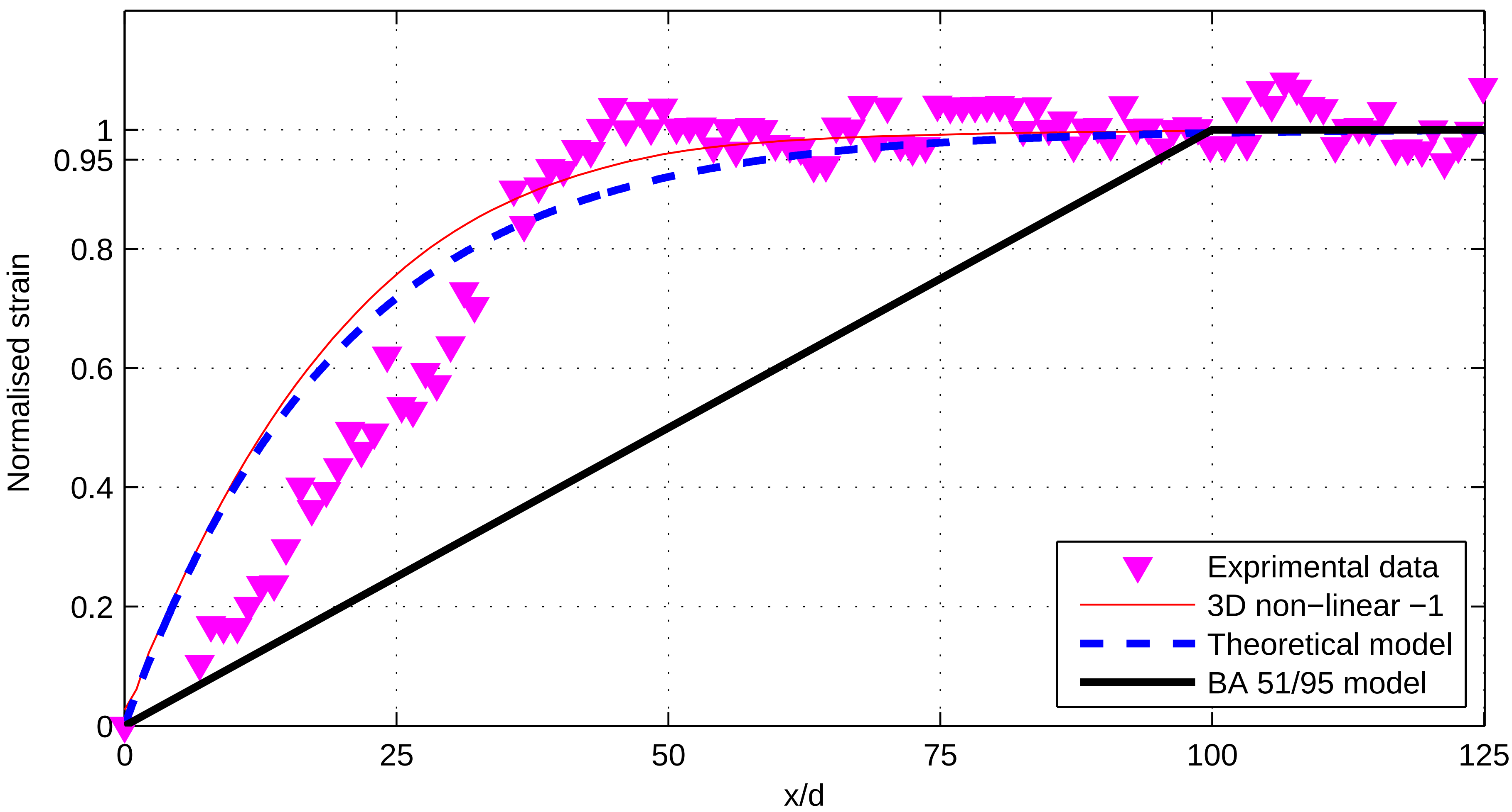


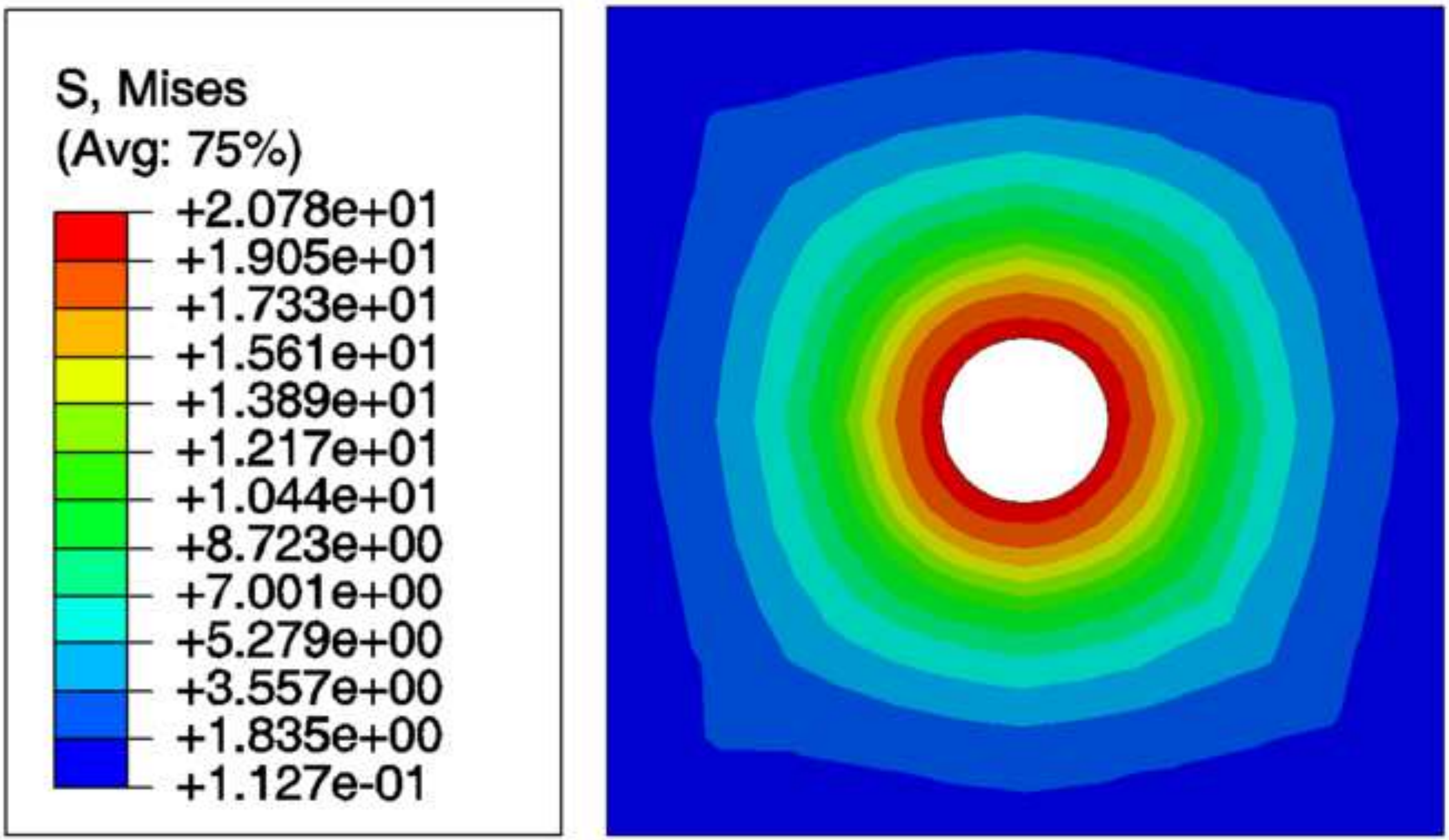

(a)
0
-10
-20
(b)
0
-10
-20

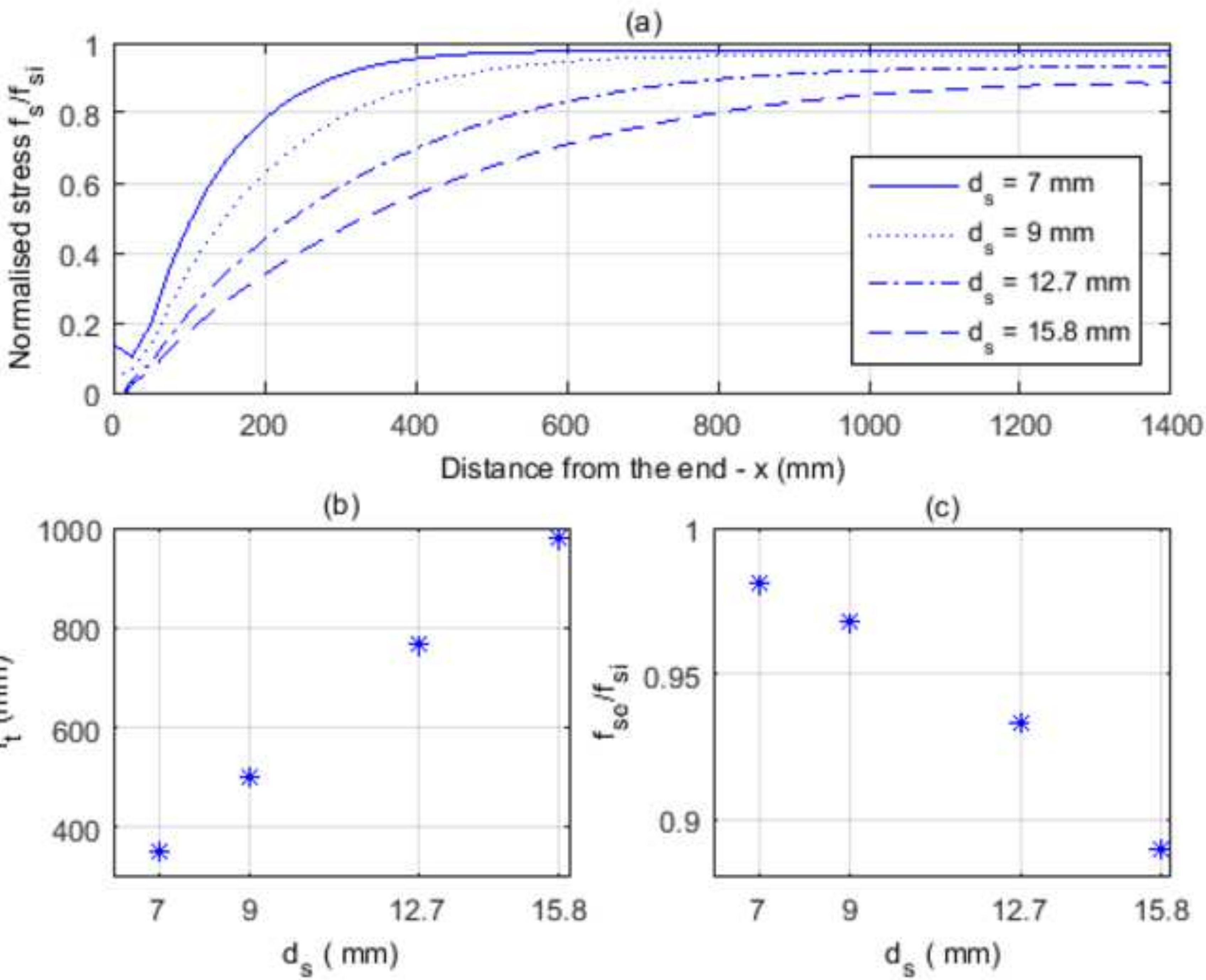

(a)
DM2
FE model
Analytical Model
Normalised longitudinal displacement
Position (mm)
(b)
ESPI data, C10
ESPI data, C20
ESPI data, C30
Analytical Model
3D FE model
Normalised longitudinal displacement
Position (mm)
(c)
V3
FE model
Analytical Model
Normalised longitudinal displacement
Position (mm)

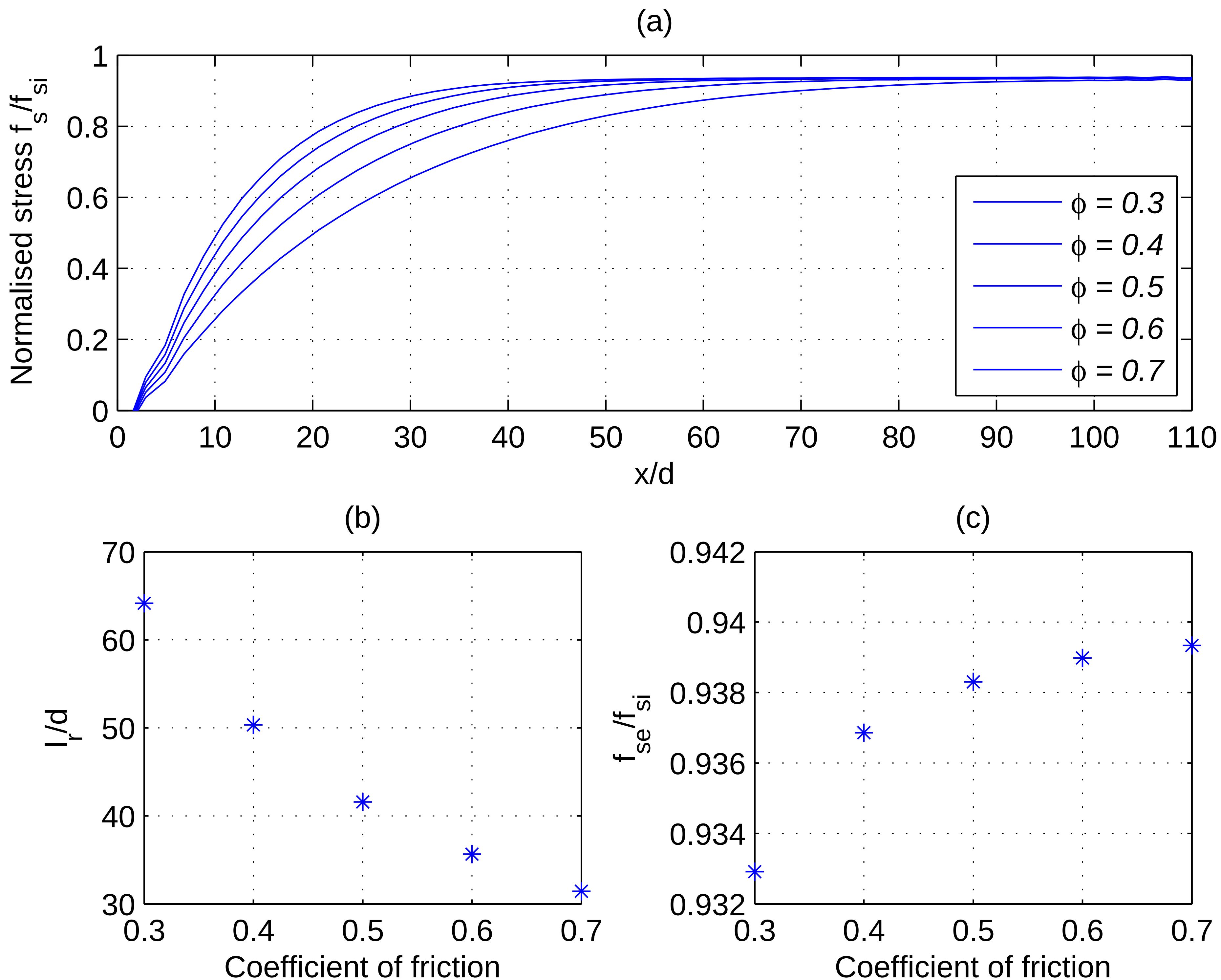

(a)
(b)
(c)
(d)
(e)
(e)
Normalised longitudinal displacement
Position (mm)
P5
C3
C2
G2
CG1
CG2
ST1
V2
FE model
Analytical Model


Normalised strain
x/d
0
0.2
0.4
0.6
0.8
0.95
1
0
25
50
75
100
125
Exprimental data
3D non−linear −1
Theoretical model
BA 51/95 model

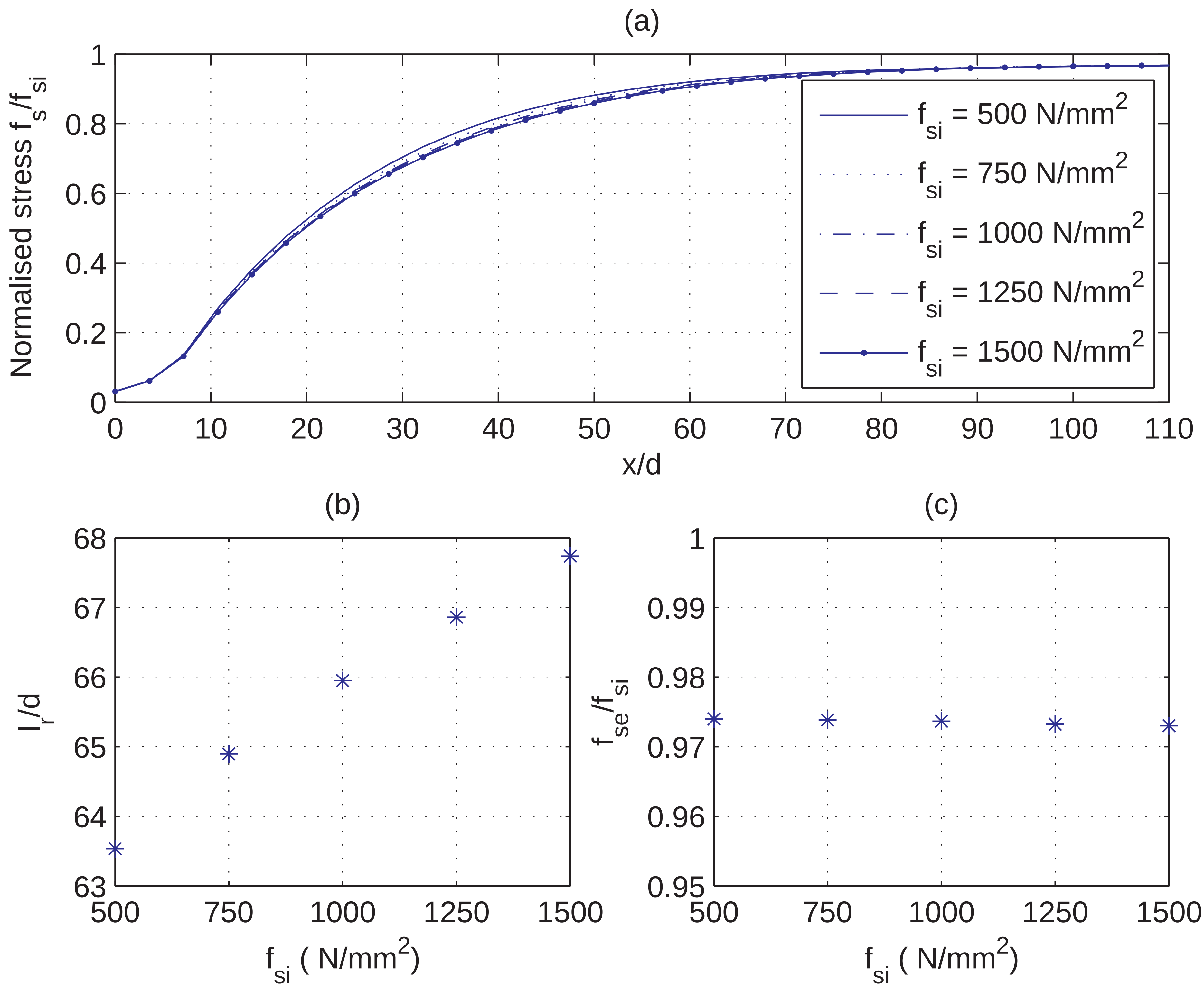
Click here to download Figure Fig 10.tif

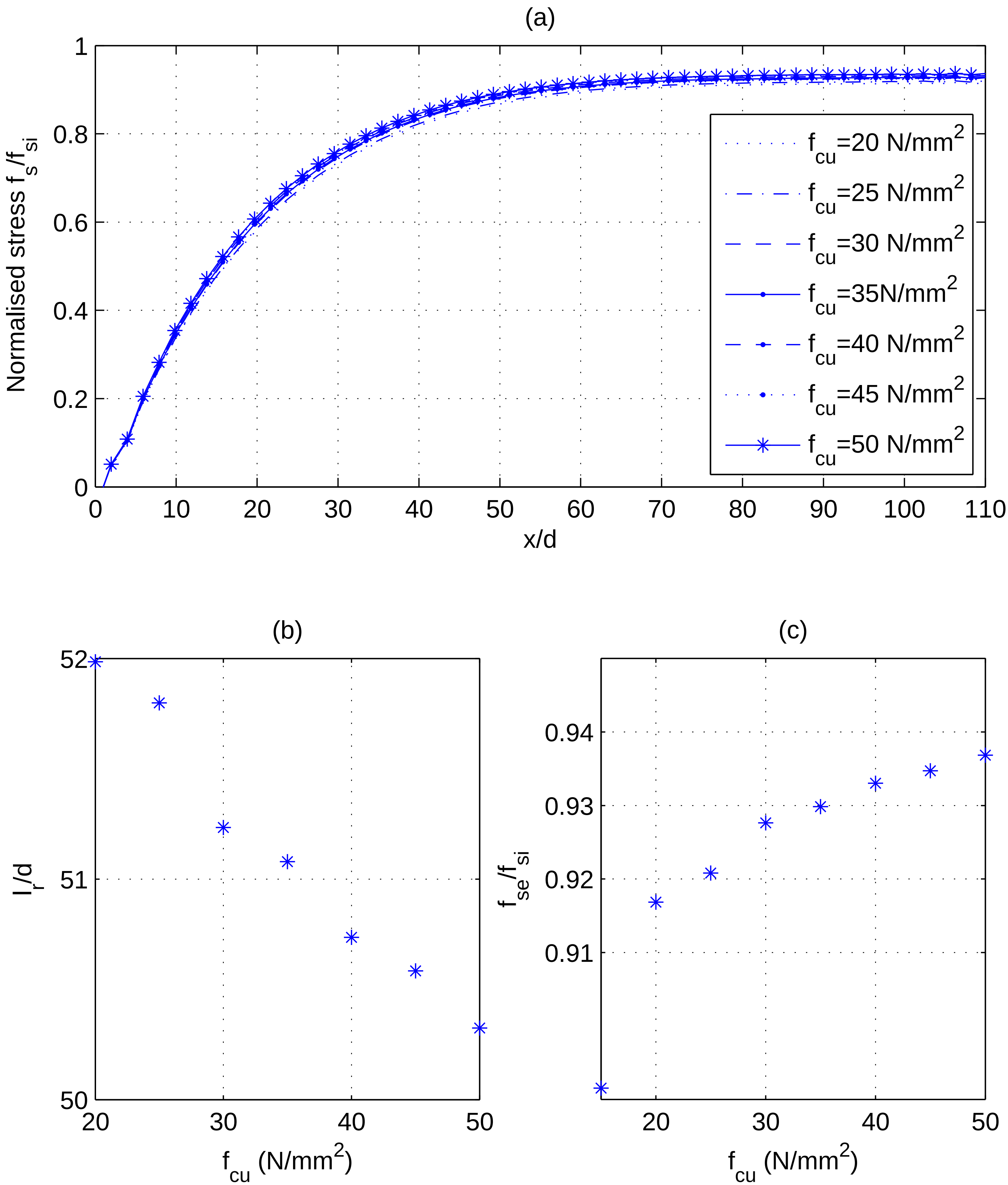


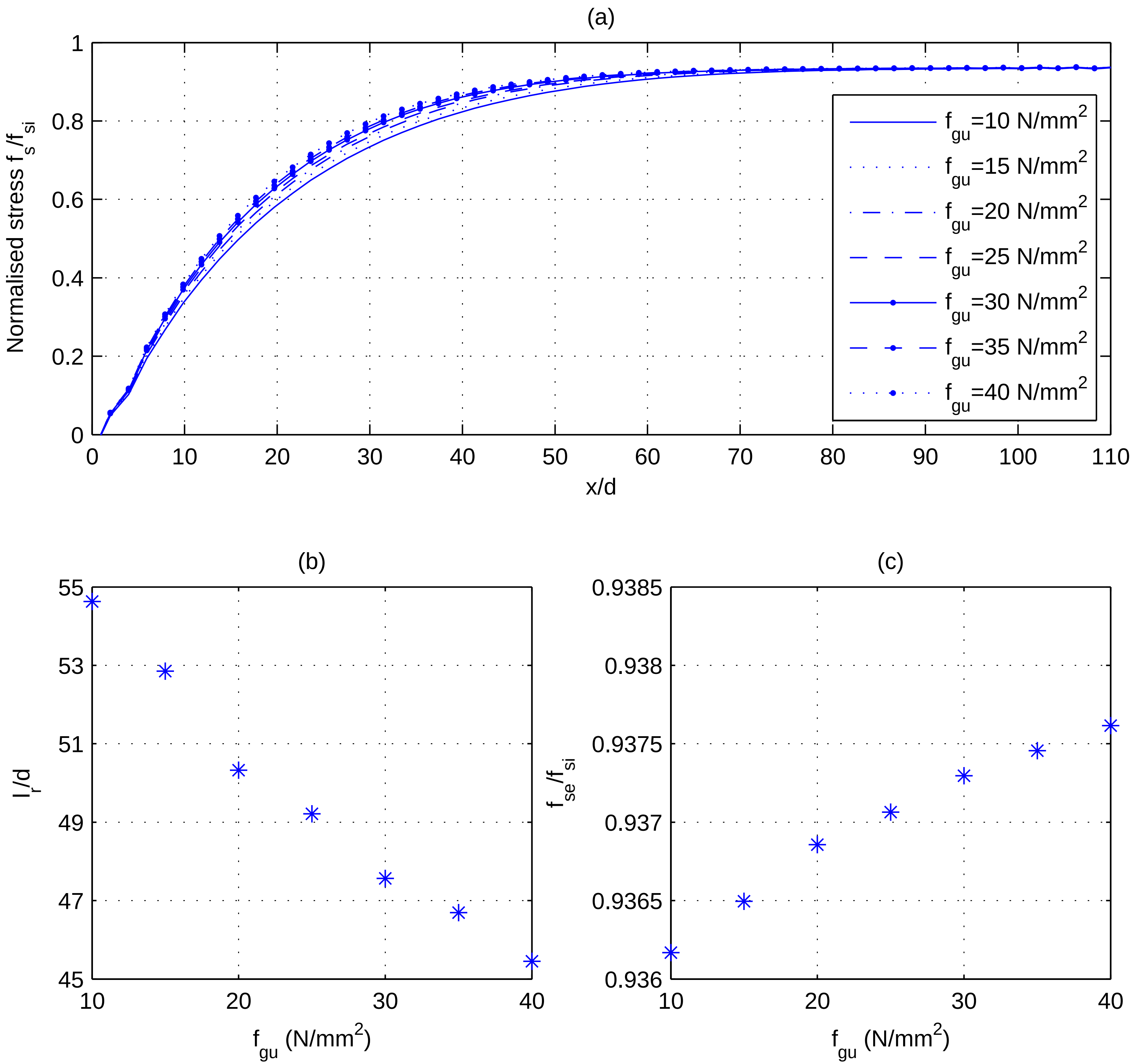

(a)
Normalised stress f_s/f_{si}
Distance from the end - x (mm)
d_s = 7 mm
d_s = 9 mm
d_s = 12.7 mm
d_s = 15.8 mm
(b)
l_t (mm)
d_s (mm)
(c)
f_{se}/f_{si}
d_s (mm)



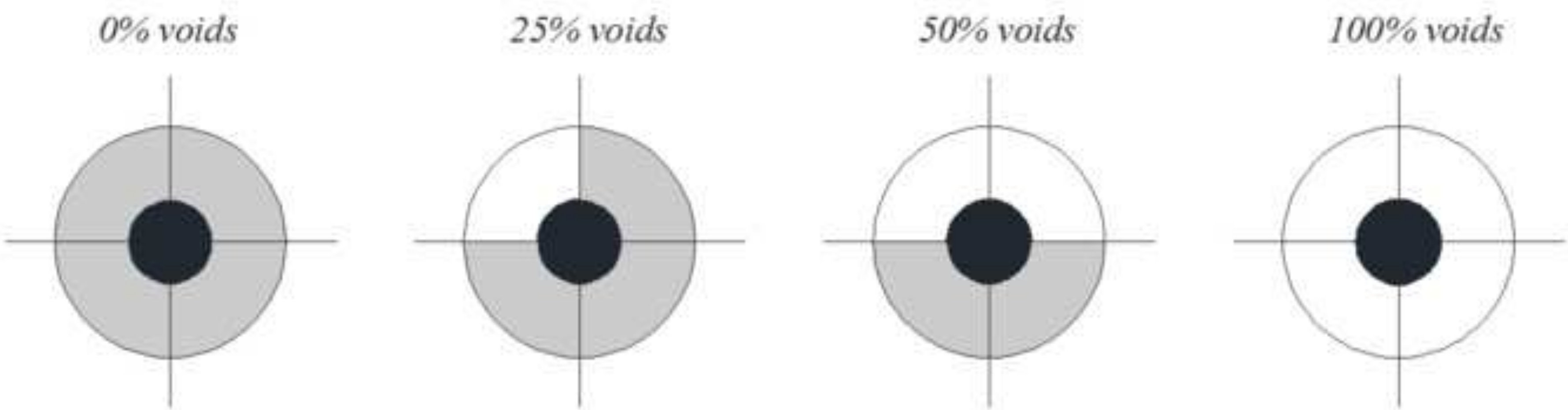
(a)
Normalised stress f_s/f_{si}
x/d
0
0.2
0.4
0.6
0.8
1
0
10
20
30
40
50
60
70
80
90
100
110
ϕ = 0.3
ϕ = 0.4
ϕ = 0.5
ϕ = 0.6
ϕ = 0.7
(b)
l_r/d
30
40
50
60
70
0.3
0.4
0.5
0.6
0.7
Coefficient of friction
(c)
f_{se}/f_{si}
0.932
0.934
0.936
0.938
0.94
0.942
0.3
0.4
0.5
0.6
0.7
Coefficient of friction

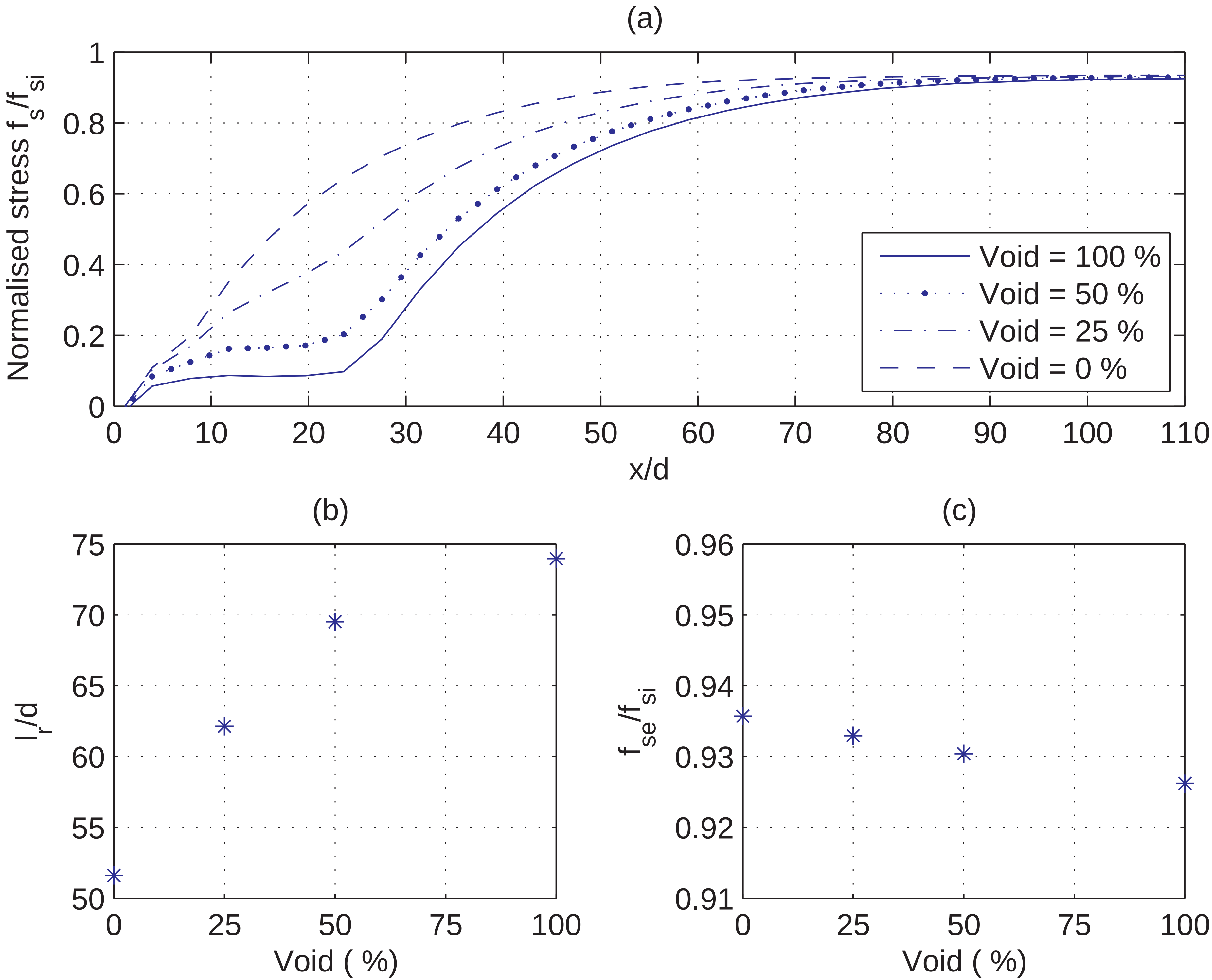

(a)
Normalised stress f_s/f_si
x/d
f_si = 500 N/mm^2
f_si = 750 N/mm^2
f_si = 1000 N/mm^2
f_si = 1250 N/mm^2
f_si = 1500 N/mm^2
0
0.2
0.4
0.6
0.8
1
0
10
20
30
40
50
60
70
80
90
100
110
(b)
l_r/d
f_si (N/mm^2)
63
64
65
66
67
68
500
750
1000
1250
1500
(c)
f_se/f_si
f_si (N/mm^2)
0.95
0.96
0.97
0.98
0.99
1
500
750
1000
1250
1500


(a)
Normalised stress f_s/f_{si}
x/d
f_{cu}=20 N/mm^2
f_{cu}=25 N/mm^2
f_{cu}=30 N/mm^2
f_{cu}=35N/mm^2
f_{cu}=40 N/mm^2
f_{cu}=45 N/mm^2
f_{cu}=50 N/mm^2
(b)
l_r/d
f_{cu} (N/mm^2)
(c)
f_{se}/f_{si}
f_{cu} (N/mm^2)

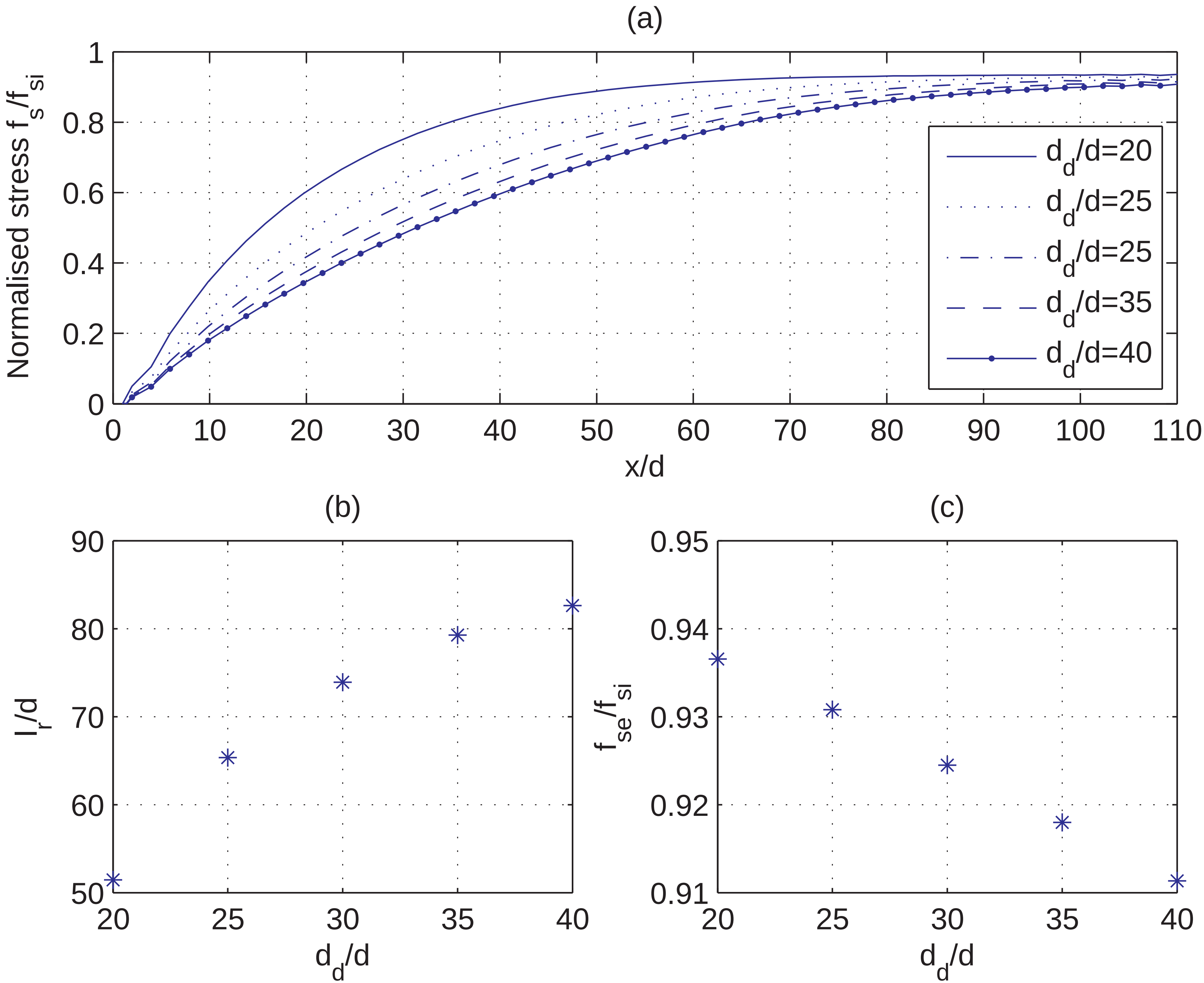


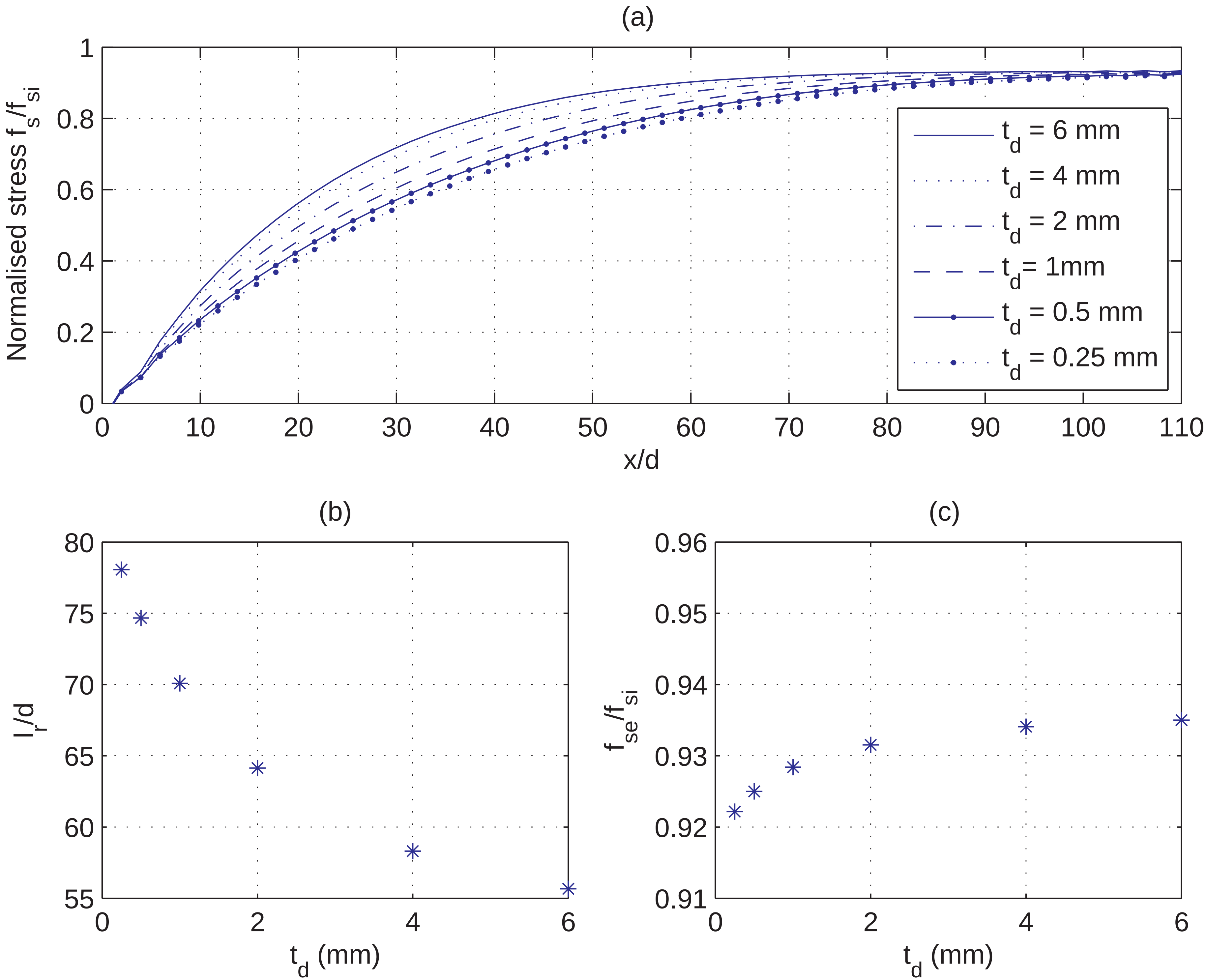
(a)
Normalised stress f_s/f_{si}
x/d
f_{gu}=10 N/mm^2
f_{gu}=15 N/mm^2
f_{gu}=20 N/mm^2
f_{gu}=25 N/mm^2
f_{gu}=30 N/mm^2
f_{gu}=35 N/mm^2
f_{gu}=40 N/mm^2
(b)
l_r/d
f_{gu} (N/mm^2)
(c)
f_{se}/f_{si}
f_{gu} (N/mm^2)

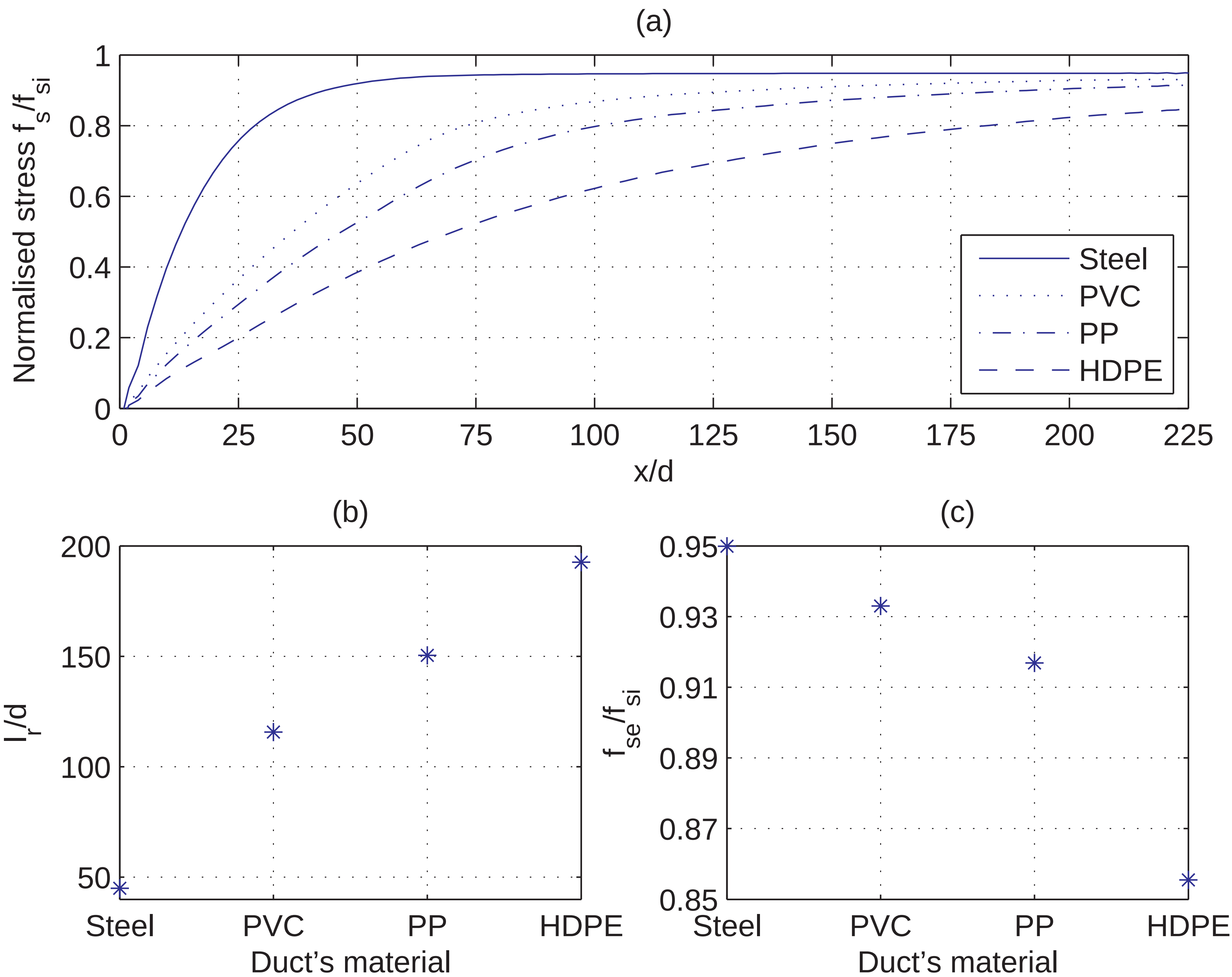

0% voids
25% voids
50% voids
100% voids



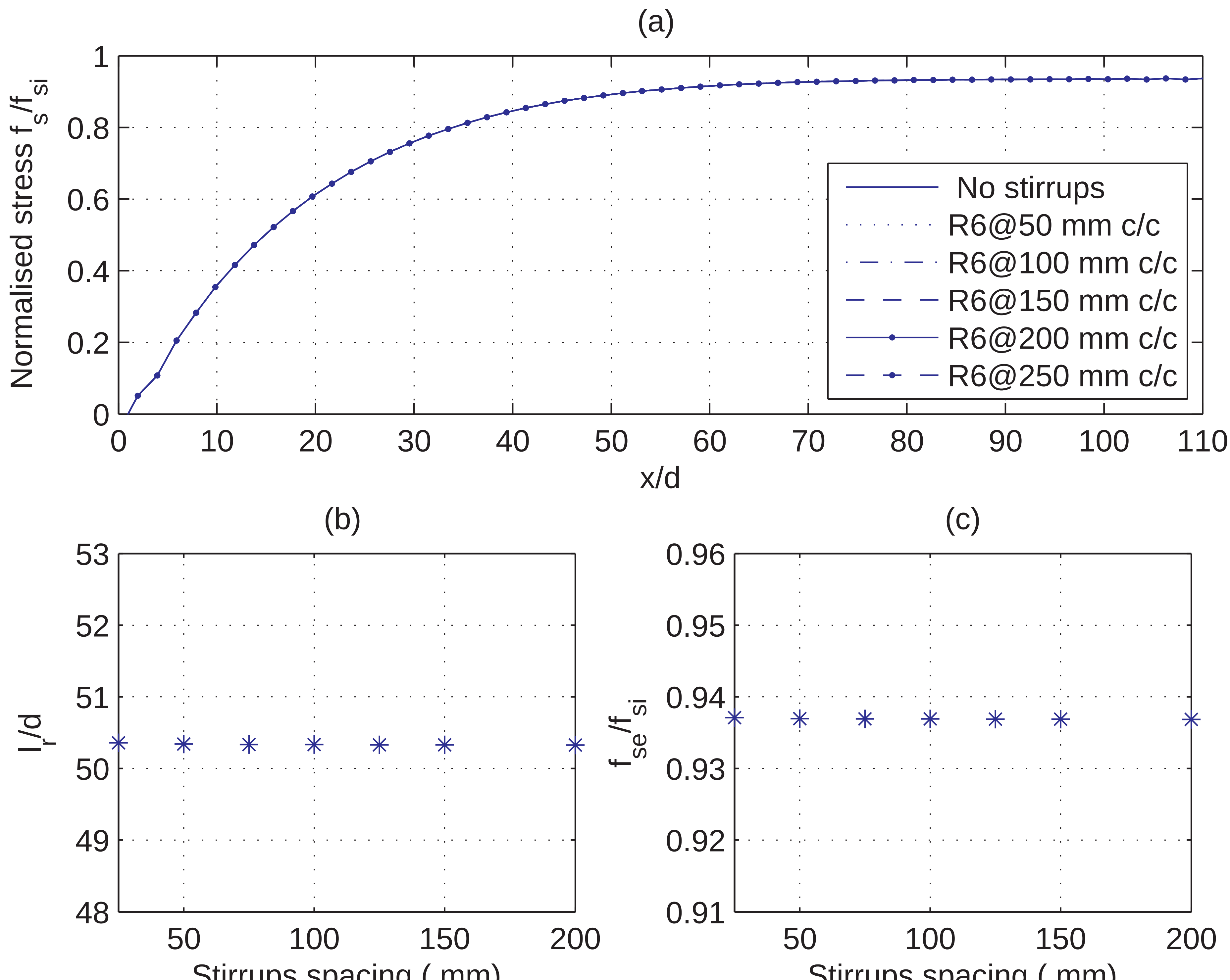
(a)
Normalised stress f_s/f_{si}
x/d
Void = 100 %
Void = 50 %
Void = 25 %
Void = 0 %
0
0.2
0.4
0.6
0.8
1
0
10
20
30
40
50
60
70
80
90
100
110
(b)
l_r/d
Void (%)
50
55
60
65
70
75
0
25
50
75
100
(c)
f_{se}/f_{si}
Void (%)
0.91
0.92
0.93
0.94
0.95
0.96
0
25
50
75
100


(a)
Normalised stress f_s/f_si
x/d
d_d/d=20
d_d/d=25
d_d/d=25
d_d/d=35
d_d/d=40
(b)
l_r/d
d_d/d
(c)
f_se/f_si
d_d/d


(a)
Normalised stress f_s/f_{si}
x/d
t_d = 6 mm
t_d = 4 mm
t_d = 2 mm
t_d= 1mm
t_d = 0.5 mm
t_d = 0.25 mm
(b)
l_r/d
t_d (mm)
(c)
f_{se}/f_{si}
t_d (mm)


(a)
Normalised stress f_s/f_{si}
1
0.8
0.6
0.4
0.2
0
0
25
50
75
100
125
150
175
200
225
x/d
Steel
PVC
PP
HDPE
(b)
200
150
100
50
l_r/d
Steel
PVC
PP
HDPE
Duct's material
(c)
0.95
0.93
0.91
0.89
0.87
0.85
f_{se}/f_{si}
Steel
PVC
PP
HDPE
Duct's material

Click here to download Figure Fig 21.eps



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

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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
University of Khartoum
Khartoum, Sudan
On behalf of Dr. John Owen and Dr. Mohammed Hussein

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