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DISTRIBUTED SENSING
FOR SERVICEABILITY ANALYSIS OF
REINFORCED CONCRETE
STRUCTURES

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Abstract

The present doctoral thesis is set on introducing a novel manner of tackling the serviceability analyses of Reinforced Concrete (RC) structures.

Classical approaches have either considered a perfect interaction between steel and concrete or a simplified constant bond relation. Either way, due to the lack of tools that could accurately assess the strains inside RC structures, they adopted theoretical, empirical or numerical solutions to define the analyses' parameters. Modern research, instead, advocates the use of the stress transfer approach which suggests an interaction between the rebars and its surrounding concrete in the form of a force transfer (bond stress). The latter provides a realistic reflection of the behavioral nature of RC, of its bond-slip mechanisms, of the effect of tension stiffening and of the crack development. Still, the issue of accurately investigating the strains inside of a RC structure persists.

With the introduction of Distributed Optical Fiber Sensors (DOFS), accurate, completely-distributed, experimentally measured strain profiles of the RC structure's constitutive materials are finally possible. DOFS finally fills the gap that ever-present instrumental gap holding back the reliability of RC structure serviceability models. The current thesis displays how to marry the potential of DOFS with the one of the stress transfer for the models' substantiation, integration and/or provision of data for the formulation of new ones.

In order to achieve such, the author performed multiple DOFS-powered experimental campaigns on RC tensile members subjected to both double pull-out testing and simple concrete shrinkage. During testing the rebar strain profiles were continuously (250 Hz) and distributedly (every 0.6 mm) sampled. On the ground of the experimental data, the stress-transfer analysis constituting parameters were extracted i.e., concrete-steel bond stress and slip. A new correlation between steel bond stress and reinforcement ratio was discovered and constituted the bod of novel bond stress law. The latter was then integrated in a revisited version of the Strain Compliance crack spacing Model whose predicting capabilities was demonstrated to be extensively superior to the ones of the present day model codes, namely fib Model Code 2010 and Euro Code 2. DOFS was also employed for the assessment of the effect of concrete shrinkage on the residual performance of RC members. In particular, a new tension-stiffening law that accounts for the effect of concrete shrinkage was introduced and substantiated thanks to the DOFS-monitoring of concrete shrinkage-induced strains on embedded steel rebars.

In conclusion, this thesis demonstrates the potential of DOFS to elevate the serviceability analyses of RC structures either substantiating the present day models or presenting valid substitutes to them.

Reziumė

Šiame darbe eksperimentiškai ir teoriškai tiriama gelžbetoninių konstrukcijų tinkamumo ribinio būvio elgsena. Tikslūs eksperimentiniai duomenys yra ypač svarbūs, taikant įtempių perdavimo metodą, kuris neabejotinai yra pranašesnis, nors ir sudėtingesnis, už klasikinius gelžbetoninių konstrukcijų analizės metodus. Įtempių perdavimo metodas yra grindžiamas sąveika tarp armatūros ir ją supančio betono, atsižvelgiant į jėgos perdavimą (sukibimo įtempius). Taikant šį metodą galime pamatyti, kaip „elgiasi“ gelžbetoninės konstrukcijos, kokia yra sukibimo įtempių – slinkties mechanika, tempiamojo sustandėjimo efektas ir ar atsiranda plyšių. Vis dėlto, klausimas, kaip tiksliai nustatyti gelžbetoninių konstrukcijų deformacijas, išlieka.

Norint nustatyti gelžbetoninių konstrukcijų armatūros sukibimo įtempius ir slinktį, būtina atlikti išsamų ir tikslų plieninių armatūros strypų deformacijų tyrimą. Toks tyrimas tik neseniai tapo įmanomu, statybos konstrukcijų bandymuose pradėjus taikyti ištisinio matavimo optinio pluošto jutiklius (DOFS). Darbe keliamas tikslas DOFS jutiklių pagalba tobulinti arba patvirtinti/atmesti esamus gelžbetoninių konstrukcijų tinkamumo modelius.

Pasak klasikinio požiūrio, gelžbetoninių konstrukcijų tinkamumas skaičiuojamas laikantis prielaidos, kad armatūros ir betono sąveika yra ideali arba imant, kad tarp armatūros ir betono veikia pastovūs sukibimo įtempiai. Autoriaus nuomone, nagrinėjant gelžbetoninių konstrukcijų tinkamumą naudoti, šios prielaidos nėra tikslios. Tikslesnių analizės modelių kūrimą riboja nepakankamai toboli eksperimentiniai konstrukcijų elgsenos stebėsenos įrankiai. Dėl to kuriami modeliai, netiksliai atspindintys konstrukcijų elgseną.

Darbe buvo eksperimentiškai tiriami gelžbetoniniai tempiamieji elementai taikant DOFS. Eksperimento metu ištisai visame ilgyje (kas 0,6 mm) optiniais jutikliais buvo matuojamos armatūros deformacijos. Gauti įtempių perdavimo parametrai – betono-armatūros sukibimo įtempiai ir slinktis. Gautas naujas sukibimo įtempių ir armatūros koeficiento dėsnis. Šis dėsnis pritaikytas deformacijų darnos plyšių modeliui, kuris tiksliau prognozuoja gelžbetoninių konstrukcijų deformacijas bei atstumus tarp plyšių nei Model Code 2010 ir Eurocode 2 modeliai. Taikant DOFS taip pat tiriamas gelžbetoninių konstrukcijų betono traukumas. Pateiktas naujas tempiamojo sustandėjimo dėsnis, kuris atspindi betono traukumo poveikį. DOFS veiksmingai pademonstravo, kad Model Code 2010 pakankamai tiksliai (tiksliau nei Eurocode 2) prognozuoja betono traukimosi deformacijas.

Apibendrinant galima teigti, kad gelžbetoninių konstrukcijų tinkamumo nustatymas taikant DOFS atveria naujas galimybes, tikrinant esamų modelių adekvatumą bei kuriant naujus modelius.

Notations

Symbols

ε_c	concrete strain
ε_{ct}	concrete tensile strain
$\varepsilon_{ct,max}$	maximum concrete tensile strain
ε_{cr}	concrete strain corresponding to the stress value f_{ct}
ε_{el}	elastic strain of the uncracked section
ε_s	steel reinforcement strain
$\varepsilon_{s,28}$	steel reinforcement strain embedded in concrete that has dried for 28 days
$\varepsilon_{s,shk}$	concrete shrinkage induced steel reinforcement strain
ε_{shk}	free concrete shrinkage
$\varepsilon_{shk,EC2}$	free concrete shrinkage as calculated by the EC2
$\varepsilon_{shk,MC2010}$	free concrete shrinkage as calculated by the MC2010
ε_{si}	bare rebar strain and/or of rebars in a transversally cracked section
ε_{sm}	average steel reinforcement strain value
$\varepsilon_{sm,EC2}$	average steel reinforcement strain value according to the EUROCODE 2
ε_{s0}	minimum steel reinforcement strain
$\varnothing_s$	steel rebar diameter
σ_s	steel reinforcement stress
σ_c	concrete compressive stress
$\sigma_{c,28}$	concrete stress after 28 days of drying time

σ_{ct}	concrete tensile stress
$\sigma_{ct,max}$	maximal concrete tensile stress
χ	ageing factor
φ	concrete creep factor
ξ	interpolation coefficient
ρ	reinforcement ratio
τ_b	concrete-steel bond stress
τ_{max}	maximal concrete-steel bond stress
A	strain slope coefficient
A_c	cross-sectional area of concrete
A_s	cross-sectional area of steel
c	concrete cover
E_c	modulus of elasticity of concrete
E_{ca}	age adjust modulus of elasticity of concrete
E_s	modulus of elasticity of steel
f_b	relative rib area of the bar
f_{ck}	characteristic concrete compressive strength
f_{cm}	average concrete compressive strength
f_{ct}	concrete tensile strength
f_{ctm}	average concrete tensile strength
f_y	steel yielding stress
f_u	ultimate tensile strength of reinforcement
L	element length
l_c	central zone length
$l_{c,simp}$	simplified central zone length
l_d	debonding zone length
l_{eff}	effective zone length
l_t	bond stress transfer length
n	modular steel/concrete ratio and
N_c	axial force in the concrete
N_s	axial force in the steel reinforcement
P	study case load
P_{cr}	cracking load of the tensile element
P_{shk}	fictitious force used for modelling the concrete shrinkage strain
s	slip
s_{rm}	average crack spacing
$s_{rm,exp}$	experimental average crack spacing
S_1	surface 1
S_2	surface 2
S_3	surface 3
t	time
t_0	loading time

Abbreviations

AAEM	Age Adjusted Modulus of Elasticity
CR	Correlation Ratio
DIC	Digital Image Correlation
DOFS	Distributed Optical Fiber Sensors
EC2	Euro Code 2
FBG	Fiber Braggs Gratings
MC2010	fib Model Code 2010
OBR	Optical Backscatter Reflectometer
PICM	Polynomial Interpolation Comparison Method
RC	Reinforced Concrete
SCM	Strain Compliance Model
SHM	Structural Health Monitoring
SRA	Strain Reading Anomaly
St.Dv.	Standard Deviation

Contents

INTRODUCTION	1
Problem Formulation.....	1
Relevance of the Thesis.....	2
Object of the Research	2
Aim of the Thesis	2
Tasks of the Thesis	2
Research Methodology	3
Scientific Novelty of the Thesis	3
Practical Value of the Research Findings.....	4
The Defendant Statements.....	4
Approval of the Research Findings	5
Structure of the Dissertation.....	6
Acknowledgments	6
1. REVIEW ON SERVICEABILITY ANALYSES OF REINFORCED CONCRETE STRUCTURES AND DISTRIBUTED OPTICAL FIBER SENSORS.....	7
1.1. Serviceability analysis of Reinforced Concrete structures	8
1.1.1. The interaction between concrete and steel rebars.....	9

1.1.2. Cracking in Reinforced Concrete structures	12
1.1.3. Influence of concrete shrinkage on the performance of Reinforced Concrete structures	15
1.2. Distributed Sensing for the assessment of the strain profiles of rebars embedded in Reinforced Concrete structures for the study of bond stress and slip between concrete and steel.....	17
1.3. Conclusions of Chapter 1 and formulation of the tasks of the thesis	24
 2. EXPERIMENTAL CAMPAIGNS FOR REINFORCED CONCRETE STRUCTURES' SERVICEABILITY ANALYSES	27
2.1. Towards a standard technology for Reinforced Concrete structures' Distributed Sensing-monitored tests.....	27
2.2. The experimental campaigns	37
2.3. Conclusions of Chapter 2	44
 3. DISTRIBUTED SENSING FOR THE VALIDATION, INTEGRATION OR DISMISSAL OF REINFORCED CONCRETE STRUCTURES' SERVICEABILITY MODELS	45
3.1. Assessment of the concrete-steel bond stresses and slip through Distributed Sensing	46
3.1.1. Distributed Sensing monitoring for bond stress/slip assessment: <i>modus operandi</i>	48
3.1.2. Distributed Sensing monitoring for bond stress/slip assessment: cracking members	53
3.1.3. Distributed Sensing monitoring for bond stress/slip assessment: non-cracking members.....	58
3.1.4. Single Distributed Sensing bond stress values per load.....	64
3.2. Revisiting the Strain Compliance crack spacing Model enhancing it through Distributed Sensing	67
3.2.1. Strain Compliance Model bases: methodology and assumptions ...	68
3.2.2. Strain Compliance Model: integral version	70
3.2.2.1. Zone lengths definition	75
3.2.2.2. Construction of the central zone length l_c equation	77
3.2.3. Strain Compliance Model: simplified version	81
3.2.4. Contribution of the Distributed Sensing to the Strain Compliance Model	82
3.3. Assessment of the shrinkage-induced performance reduction of Reinforced Concrete structures through Distributed Sensing	87
3.3.1. Shrinkage-free tension stiffening law for various concrete grades.	88
3.3.2. Distributed Sensing contribution to studying the effect of concrete-shrinkage in a Reinforced Concrete member	92

3.3.2.1. Short term shrinkage	93
3.3.2.2. Long term shrinkage	96
3.3.2.3. Comparison with the Eurocode 2 and the Model Code 2010 predictions.....	99
3.4. Conclusions of Chapter 3	102
GENERAL CONCLUSIONS.....	105
REFERENCES	107
LIST OF SCIENTIFIC PUBLICATIONS BY THE AUTHOR ON THE TOPIC OF THE DISSERTATION	115
SUMMARY IN LITHUANIAN	119
ANNEXES ¹	133
Annex A. Database for the construction of parameter lc	134
Annex B. Database of RC ties for the comparison between experimental RC ties' crack spacings and the SCM extracted crack spacing predicted values.....	139
Annex C. Declaration of academic integrity	142
Annex D: The co-authors' agreements to present publications material in the doctoral dissertation.....	143
Annex E. Copies of scientific publications by the author on the topic of the dissertation	144

¹The annexes are supplied in the enclosed compact disc.

Introduction

Problem Formulation

The classic approaches to the serviceability of RC structures tackle the issue by means of theoretical, empirical or numerical solutions. This is mainly due to the lack of tools that can truly assess the inner-workings of RC structures in each of its points. The stress transfer method is commonly considered a modern and more performant alternative. The latter suggests an interaction between the concrete and the embedded rebars in the form of a stress transfer (bond stress τ_b) occurring along their interface. Yet, an accurate knowledge of the rebar strains, of the concrete-rebar bond stresses and slip is crucial for correct stress transfer-based and experimentally backed serviceability models. Until now, the sampling of such data has proven to be extremely challenging, hence the little to no congruence between the numerous current models. The introduction of Distributed Sensing may very well be the solution to this issue as it can easily sample the above mentioned parameters with spacing resolutions smaller than a millimeter. Consequently, the present thesis intends to assess the viability of DOFS for the validation, integration or dismissal of current RC structures' serviceability models.

Relevance of the Thesis

With the introduction of Distributed Optical Fiber Sensors (DOFS), accurate, completely-distributed, experimentally measured strain profiles of the RC structure's constitutive materials under different loading conditions, both before and after concrete cracking are possible. This technology finally fills the gap that prevented the complete use of the stress-transfer potential for the serviceability analyses of RC structures. The present doctoral thesis is aimed at assessing the potential of DOFS to substantiate, integrate or dismiss current RC structures' serviceability models. In particular, the present work takes full advantage of potential of DOFS to introduce two serviceability analysis approaches, both performing substantially better than their modern day code counterpart.

Object of the Research

The object of this research are Distributed Optical Fiber Sensors monitoring tools for the extraction of RC structures' rebar strain data on the grounds of which to revisit modern day RC serviceability models.

Aim of the Thesis

Development and substantiation of novel Distributed Optical Fiber Sensors (DOFS)-based techniques for advancement in bond-slip, cracking, tension stiffening and restrained shrinkage analysis of RC structures through distributed strain monitoring of reinforcement bars embedded inside concrete ties.

Tasks of the Thesis

In order to achieve the objective, the following steps were undertaken:

1. To assess the performance of DOFS as a monitoring tool by means of a comparison against classic ones.
2. To establish techniques for the deployment of DOFS inside RC structures and for the cleansing of potential inaccuracies in the sampled strain profiles (computational algorithms).
3. To design and execution of DOFS-monitored experimental tests for the extraction of steel strain profiles of rebars embedded inside RC ties subjected to tensile testing and shrinkage-induced compression.

4. To process the test data to assess the magnitude of the steel-concrete bond stress/slip, crack spacing and concrete shrinkage-induced tensile strength reduction.

Research Methodology

The methodology employed in the present thesis consists firstly in the definition of which RC structures' serviceability models would require novel investigation. Secondly, in the design and execution of DOFS-monitored experimental campaigns aimed at collecting novel experimental data of the main parameters for the improvement of such models. Thirdly, employing computational softwares for data post-processing, model substantiation, creation and integration. Finally, extracting conclusions on the performance of DOFS for the improvement of serviceability analyses of RC structures.

Scientific Novelty of the Thesis

The aspects of scientific novelty on the theoretical and experimental investigation presented herein are as follow:

1. DOFS is in itself a novelty as it has been only recently applied to structural engineering. Its elevated accuracy, reliability, ease of use and, most importantly, the distributed nature of its measurements (even every 0.63 mm) have led to a rapid increase of popularity. DOFS is presently regarded as the most cutting-edge tool in the entire Structural Health Monitoring and experimental strain monitoring panorama.
2. Very few experimental structural engineering campaigns made monitored strains with DOFS and even less did so inside a RC structure itself. Indeed, most experimental campaigns saw the bonding of the fiber on a RC structure's surface rather than embedding it. The experimental efforts presented in this thesis, instead, make use of this last deployment technique.
3. DOFS' reduced cross-sectional size allows it to be bonded in an un-intrusive manner to any rebar. This was important during the author's experimental investigation, in particular when using small rebar diameters (such as Ø8), previously almost impossible to monitor with older strain monitoring tools.

4. Until now, DOFS has never been employed for the validation, integration or dismissal of RC structures' serviceability models. The proposal of novel such models is also exclusive to the present thesis.
5. DOFS is usually employed for Structural Health Monitoring purposes. On rarer occasions it was used for structural engineering purposes and, at the time when the DOFS-powered tests were performed, it had never been used for the assessment of the concrete-steel bond stress and slip. Nowadays only a couple more such tests have been performed by other researchers.
6. The present research revisits a novel crack spacing prediction model based on the hypothesis of Strain Compliance and uses DOFS to substantiate and elevate it.
7. The monitoring of concrete shrinkage-induced strains on embedded steel rebars was performed only once before by means of DOFS. The present research, though, expands the calculations to the one of the stress inside the concrete surrounding the rebar.
8. The present research uses DOFS to substantiate a novel shrinkage-free tension stiffening model.

Practical Value of the Research Findings

1. By means of DOFS-powered experimental research, the stress transfer approach is finally entirely viable for the definition of RC structures' serviceability models. On the grounds of such, modern day models can be improved or substituted. New RC structures designed on the foundation of such new DOFS-powered models would reach the minimum serviceability requirements with more consistency. Furthermore, with more accurate modelling of the serviceability issues, structural redundancy would be pushed down thus saving on costs and material consumption.
2. The experimental guidelines provided in the present work represent the groundings for future structural engineering oriented DOFS monitored experimental research.

The Defendent Statements

Based on the results of the present investigation and the defendant's cumulative experience the following statements are provided:

1. DOFS holds a large potential pool for the future studies of serviceability issues for RC structures. The present work provides several examples of the improvement and/or validation/disproving that its integration can bring to serviceability models.
2. DOFS non-intrusiveness was instrumental for the successful and accurate monitoring of RC structures, notably including those with small rebar diameters.
3. DOFS allowed to study with unprecedented precision the strains of a rebar embedded inside a RC structure. This led to an equally accurate definition of the interaction between concrete and steel, namely bond stress, slip and the transfer lengths in which these occur.
4. The extraction of a novel DOFS-based bond stress model allowed for the modification and validation of the Strain Compliance Model (SCM) allowing to obtain crack spacing predictions superior to the ones obtained from the EC2 and MC2010.

Approval of the Research Findings

Over the course of the PhD studies (2016 - 2021) the dissertation results were published in 9 scientific journals and in 6 international conference proceedings:

- G. Kaklauskas, M.F. Bado. A novel concept of crack analysis: prediction of mean crack spacing in FRP reinforced concrete beams. *COST Action TU1207 Next Generation Design Guidelines for Composites in Construction*. Budapest, Hungary. April 2017.
- G. Kaklauskas, M.F. Bado, R. Ramanauskas. Crack analysis of reinforced concrete members based on predicted mean strain of the reinforcement. *EAC2 2nd International RILEM/COST Conference on Early Age Cracking and Serviceability in Cement-based Materials and Structures*. Brussels, Belgium. September 2017.
- M. F. Bado, J. R. Casas, A. Barrias. Performance of Distributed Optical Fiber Sensors bonded to reinforcement bars in bending. *APWSHM Asian Pacific Workshop of Structural Health Monitoring 2018*. Hong Kong, Hong Kong. November 2018.
- M. F. Bado, J. R. Casas, G. Kaklauskas. Performance of Distributed Optical Fiber Sensors (DOFS) and Digital Image Correlation (DIC) in the monitoring of RC structures. *EACEF 7th International Conference of Euro Asia Civil Engineering Forum*. Stuttgart, Germany. September 2019.

- M. F. Bado, J. R. Casas, G. Kaklauskas. Performance of embedded Distributed Optical Fiber Sensors (DOFS) for Structural Health Monitoring (SHM) of concrete bridges. *IABMAS 10th International Conference on Bridge Maintenance, Safety and Management*. Sapporo, Japan. April 2021.
- M. F. Bado, J. R. Casas. Monitoring of reinforced concrete structures by Distributed Optical Fiber Sensors. *EUROSTRUCT 2021 – 1st Conference of the European Association on Quality Control of Bridges and Structures*. Padova, Italy. August 2021.

Structure of the Dissertation

The dissertation consists of an introduction, 3 chapters, general conclusions, reference list of cited sources (105), list of the author's publications on the dissertation research (14 publications), summary in Lithuanian and 5 annexes. The volume of the dissertation is 145 pages, with 62 figures and 12 tables.

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Review on Serviceability Analyses of Reinforced Concrete Structures and Distributed Optical Fiber Sensors

The goal at hand is the one of developing novel Distributed Optical Fiber Sensors (DOFS)-based techniques for advancement in the serviceability analysis of RC structures. As such, the present chapter aims at providing the reader with a multifaceted background literature review on RC serviceability analysis, of the effects of concrete shrinkage on the former and on Distributed Optical Fiber Sensors. This being a revolutionary strain monitoring tool that has only recently been applied to civil engineering and been innovatively deployed inside a RC structure amongst the first time in the present research. This chapter includes the material presented in journal publications (Bado & Casas 2021; Berrocal et al. 2021).

1.1. Serviceability analysis of Reinforced Concrete structures

Among the two most essential criteria of the structural design, safety (translating in ultimate limit states) and serviceability (translating in serviceability limit states), the latter deals with the structure's usefulness (intended as its complete use as originally designed and intended), working conditions and overall performance along its service life. According to the Model Code 2010 (Fédération Internationale du Béton – International Federation for Structural Concrete (fib) 2013), the serviceability limit states address the fitness-for-use of a structure and are translated into operational limit states (satisfied when the facility has suffered practically no damage and can continue serving its original intention with little disruption of use for repairs) and immediate use limit states (satisfied when the structure itself is very lightly damaged with a negligible risk to life). The corresponding serviceability limit states criteria are related to functionality of the structure related to its normal use and comfort of using the structure. For example, they both refer to unacceptable deformations or deflections which impair the functionality of the structures or their contents, cause damage to non-structural components and that cause discomfort to people (vibrations and appearance of damaged structural or non-structural elements).

Structural safety and serviceability are not mutually exclusive. Indeed, a structurally strong building can be considered unserviceable if it fails to meet the entailed reliability. For example, a beam with no danger of collapsing may develop a notable deflection or its surface concrete may spall outwards exposing its reinforcement thus triggering its unserviceability.

Focusing on the serviceability of RC structures, the formulation of its design and analysis guidelines is complicated by the concrete's stress-strain relationship non-linearity. In compression, concrete develops elastic-plastic properties under monotonic loading whilst in tension, the non-linearity is exacerbated by the formation of cracks which radically change a RC structure's behavior. The steel reinforcement also displays elastic-plastic behavior along with strain hardening under cyclic loading (Noh et al. 2003). Yet, the biggest source of RC structure non-linearity and performance unpredictability comes from its intrinsic composite material nature as the combination of concrete and reinforcement steel. Moreover, this interaction varies along time with the evolution of the structure's service life. Indeed, time dependent phenomena such as restrained concrete shrinkage and concrete creep may lead to deformations which, if left unaccounted for, may lead to fierce serviceability issues (sometimes chipping the edge of structural safety issues). As a consequence of all the previously mentioned non-linearities, it is generally accepted that the probabilistic calculations, basis for the serviceability design, are arduous and complex (Stewart 1996).

1.1.1. The interaction between concrete and steel rebars

Granted the serviceability performance of RC structure hinges on the interaction between its two constitutive materials (dictating both its behavior and performance), the latter has been and still presently is the topic of a large research effort from research groups worldwide. Aside from empirical techniques, the theoretical models that study this interaction can be categorized into two major groups:

- Models with averaged (smeared) representation of deformations and cracking. This methodology, which includes analytical, semi-empirical models and numerical methods based on the smeared crack approach, boast advantages of simplicity and numerical effectiveness. Their main shortcoming is their lack of versatility as they cannot produce explicit results of crack spacing and crack width.
- Models with discrete representation of cracking. These are able to extract deformation results and perform cracking analysis based on the principle of stress transfer (also called partial interaction or bond-slip). The major drawback of the stress transfer approach is the lack of an adequate bond-slip law, principally governing the results of analysis. The present thesis' work deals with the serviceability issues according to the present methodology tackling directly the above mentioned drawback though the use of the DOFS technology.

In general practice, a full interaction between the interfaces of concrete and steel reinforcement bars with no physical slip is assumed but, while this simplification is reasonable in load bearing capacity analysis, it however becomes unacceptable when serviceability of RC structures is considered (Kaklauskas et al. 2012). For the latter, instead, a stress transfer approach is more suitable as it realistically reflects the nature of RC, its bond-slip mechanics, the effect of tension stiffening and the development of cracks (Kaklauskas et al. 2012). Hence, a deep understanding of such interaction is crucial to the studying of any RC structure's stiffness and mechanical response to stresses (Kaklauskas et al. 2012) in addition to providing key elements for a correct development of finite element models on the grounds of which accurate structural evaluations can be developed . As mentioned above, while classical approaches either consider perfect interaction between steel and concrete (Kaklauskas et al. 2019) or simplified constant bond relations (Kaklauskas et al. 2019), modern research has brought forward a stress-transfer approach which alternatively suggests an interaction between the two in the form of a force transfer (defined as bond stress τ_b). This is fundamentally the shear stress that develops at the surface of the reinforcing bar which prevents the relative movement between the latter and its surrounding concrete. The bond between concrete and reinforcement can be boiled down to y the three basic

effects; frictional resistance, chemical adhesion and mechanical interlock. Frictional resistance is created by the combination of concrete shrinkage strain and the surface roughness of the steel reinforcement bars. The chemical adhesion is created by the hydration process in cementitious materials. Finally, a mechanical interlock is present between the reinforcement ribs and surrounding the solidified concrete (Siempu & Pancharathi 2018) - this does not occur for plain rebars due to the absence of ribs. Keeping in mind all discussed above, several factors influencing the bond behavior are here listed:

- Concrete strength; the bond mechanism is actually based on the stress transfer from the steel to the concrete by compression and shear interfacial forces. The concrete uniaxial strength f_c and concrete tensile strength f_{ct} are therefore essential parameters to ensure the performance of the bond stress and to prevent bond failure by splitting.
- Bar diameter; an increase in the diameter of the bar reduces the maximum bond stress.
- Type and arrangement of the ribs in the reinforcement bars (ACI Committee 408 2003; Rao et al. 2007; Hadi 2008; Moallemi Pour & Alam 2016; Siempu & Pancharathi 2017); Rehm (1969) used pull-out tests to discover a linear relationship between the bond strength and the relative rib area of the bar f_R being:

$$f_R = f(d, a_m, c, e). \quad (1.1)$$

In which the bar diameter is d , maximum height of the transverse ribs a_m , the spacing between the transverse ribs c and the width of longitudinal ribs e influence the value. The higher f_R values, the bigger are bond strength. Other properties such as inclination of the ribs might also affect the bond mechanism but to a smaller extent.

- Confinement; this one has a double influence on bond behavior; it delays the onset of splitting failure and it increases the frictional force in the steel-concrete failure surface therefore resulting in higher bond stresses and in reduced values of slip at a particular load.

Indirectly, another phenomenon that largely influences the bond is concrete/rebar slip. The latter represents a dissimilarity in the absolute displacements of concrete and steel. The occurrence of slip usually results in localized concrete damages (especially in proximity to a member's edges or cracked sections) which significantly reduces the bond action (Ruiz et al. 2007).

Even though the bond mechanics are understood, its intrinsic multiplex nature still needs to be quantified properly through further in-depth investigations (Balázs 2007). Furthermore whilst its interaction with slip is crucial to the study of RC structures and despite having been extensively investigated both

analytically and experimentally over the years (Diab et al. 2014; Wu & Zhao 2012; Kankam 1997) an overall satisfactory prediction bond-slip law has not yet been achieved and there are large discrepancies among its existing models (American Concrete Institute (ACI) 1995; Fédération Internationale du Béton – International Federation for Structural Concrete (fib) 2013).

Various models which effectively predict the maximum or average bond stress in concrete among which the Model Code 2010. Figure 1.1 combined with Table 1.1 reports the latter's bond/slip prediction law on the grounds of the parameters listed in Table 1.2.

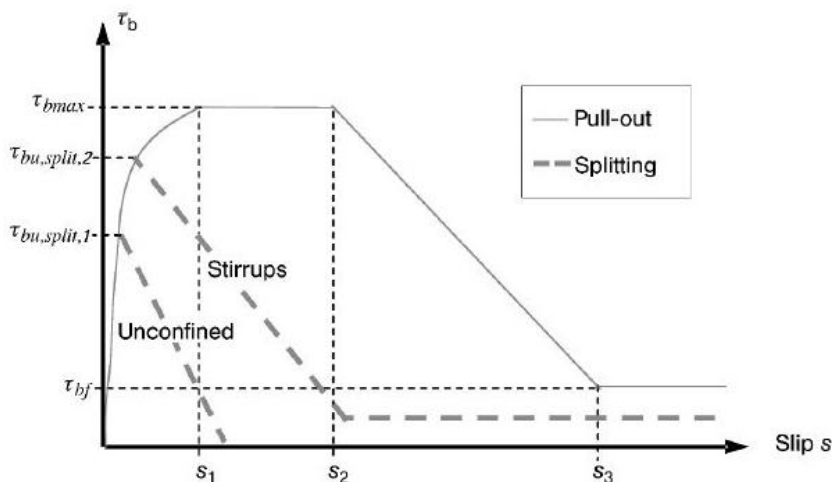


Fig. 1.1. Model Code 2010 Analytical bond stress-slip relationship under monotonic loading (Fédération Internationale du Béton – International Federation for Structural Concrete (fib) 2013)

Table 1.1. Model Code 2010 prediction of bond stress and slip

Slip s	Bond stress τ
$0 \leq s \leq s_1$	$\tau = \tau_{max} \left(\frac{s}{s_1} \right)^\alpha$
$s_1 < s \leq s_2$	$\tau = \tau_{max}$
$s_2 < s \leq s_3$	$\tau = \tau_{max} - (\tau_{max} - \tau_f) \left(\frac{s - s_2}{s_3 - s_2} \right)$
$s > s_3$	$\tau = \tau_f$

Table 1.2. Model Code 2010 prediction parameters for bond stress and slip

Parameter	1	2	3	4	5	6
	Pull Out		Splitting			
	Good bond cond.	All other bond cond.	Good bond conditions		All other bond cond.	
			unconfined	stirrups	unconfined	stirrups
$\tau_{\max}$	$2.5\sqrt{f_{ck}}$	$1.25\sqrt{f_{ck}}$	$7.0 \left(\frac{f_{ck}}{20}\right)^{0.25}$	$8.0 \left(\frac{f_{ck}}{20}\right)^{0.25}$	$5.0 \left(\frac{f_{ck}}{20}\right)^{0.25}$	$5.5 \left(\frac{f_{ck}}{20}\right)^{0.25}$
s1	1.0 mm	1.8 mm	$s(\tau_{\max})$	$s(\tau_{\max})$	$s(\tau_{\max})$	$s(\tau_{\max})$
s2	2.0 mm	3.6 mm	s1	s1	s1	s1
s3	$c_{\text{clear}}^{1)}$	$c_{\text{clear}}^{1)}$	1.2 s1	0.5 c_{clear}	1.2 s1	0.5 c_{clear}
α	0.4	0.4	0.4	0.4	0.4	0.4
t_f	0.4 $\tau_{\max}$	0.4 $\tau_{\max}$	0	0.4 $\tau_{\max}$	0	0.4 $\tau_{\max}$

With c_{clear} being the clear distance between the ribs. Other bond/slip relationship models are available in the research community such as Orangun, Esfahani and more (Darwin et al. 1992; Harajli 2004; Fédération Internationale du Béton – International Federation for Structural Concrete (fib) 2013).

1.1.2. Cracking in Reinforced Concrete structures

As established above the serviceability performance of RC structures hinges on the interaction between its two constitutive materials dictating both its behavior and performance. The integrity of such interaction, though, can break when cracks start forming in the RC structures, radically altering its nature. Indeed, the brittle nature of concrete combined with its limited tensile strength capacity makes cracking a crucial factor for the serviceability of RC structures.

Concrete cracking occurs once the tensile stresses transmitted to the former surpasses its maximum tensile capacity (Leonhardt 1988). Yet, despite the vast number of studies even currently going on aimed at gauging all of the different concrete fracture mechanics' complex aspects, its serviceability limit state analyses in the design codes and standards are still inconsistent and complicated (Ramanauskas 2019).

It has been experimentally proved that the kind of rebar (plane or deformed) embedded in the rebar plays a significant role in the crack formation. The primary crack spacing for plain bars is considerably bigger than the deformed bars which further leads to higher crack widths. At relatively higher steel stresses, new primary cracks stop appearing whilst new smaller transversal crack start appearing henceforth referred to as secondary cracks. These usually do not extend to the

concrete surface, as they are densely packed and stemming around the bar ribs (Goto 1971) where the latter literally crushes its surrounding concrete (as visible in Figure 1.2).

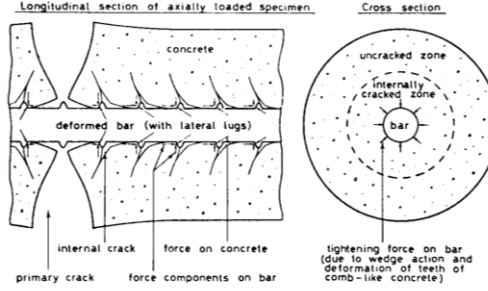


Fig. 1.2. Deformation of concrete around reinforcing steel after formation of internal cracks (Goto 1971)

The extension of these internal cracks can deteriorate the existing bond mechanism between concrete and reinforcement surface, which naturally may challenge the structural serviceability of the structure concerned. The internal cracking may cause acute deformation in the concrete, particularly around the deformed bar, resulting in non-planar sections (Gribniak et al. 2018). In order to further elucidate the cracking mechanism, it is important to explain the stress transfer approach's take on serviceability of a RC member. According to it, a RC structure's behavior is generally divided in three stages: the elastic stage, the crack formation stage and the stabilized cracking stage. Figure 1.3 depicts a RC tensile member's theoretical rebar strain profile $\varepsilon_s(x)$ during the above mentioned three stages, together with the corresponding steel-concrete bond stresses profile $\tau_b(x)$.

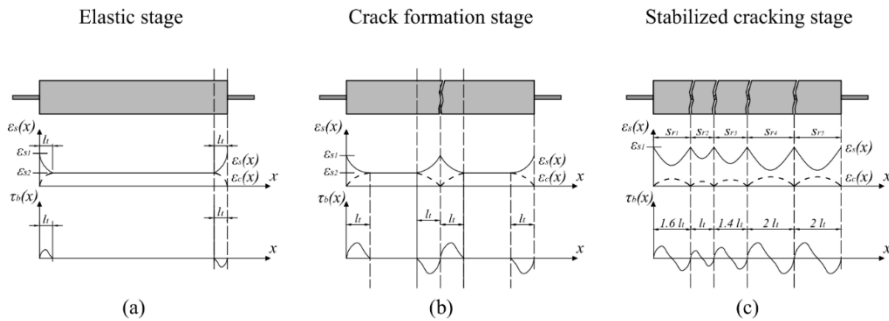


Fig. 1.3. RC tensile member three stages behavior: elastic (a), crack formation (b) and stabilized cracking (c)

The first stage is referred to as elastic stage as no permanent deformations are present, neither in the concrete (cracks) nor in the steel (yielding). At this stage, bond and slip occur within localized regions at the ends of the element. These regions, of length l_t (defined as transmission length), is where the stresses are transferred from the steel rebar to the surrounding concrete through bond stress τ_b . As a result of this mechanism defined as tension-stiffening, the rebar is increasingly less stressed. Beyond the l_t its strains ε_{s2} are decidedly inferior compared to the naked rebar's ε_{s1} . In the region between the latter two, both steel and concrete share the same strain level (strain compatibility area) and the resulting steel strain profile is flat. The main parameters of the strain profile are given by Equation 1.2 and 1.3 while their ratio is equals to Equation 1.4.

$$\varepsilon_{s1} = \frac{\sigma_s}{E_s} = \frac{P}{A_s E_s}; \quad (1.2)$$

$$\varepsilon_{s2} = \varepsilon_c = \frac{P}{A_c E_c + E_s A_s}; \quad (1.3)$$

$$\frac{\varepsilon_{s2}}{\varepsilon_{s1}} = \frac{n \cdot \rho}{1 + n \cdot \rho}. \quad (1.4)$$

Where A_s is the steel rebar's cross-sectional area, ε_c the concrete's strain and A_c is the concrete's cross-sectional area, ε_{s1} is the reinforcement strain at the end of the member in the naked rebar and ε_{s2} is the reinforcement strain within the region where the strain compatibility between the concrete and bar is restored. As the applied tensile load increases, eventually the stress transferred to the concrete surpasses its tensile strength f_{ct} . The next stage, crack formation, is then initiated. As visible in Figure 1.4, in the cracked section (point A) the steel rebar cannot transfer anymore the stresses to concrete (as it is absent) and will have to carry again its entirety alone, similarly to point A on the edges of the member.

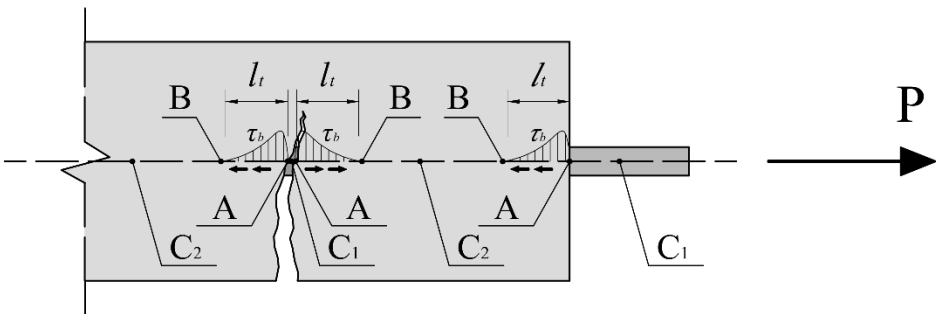


Fig. 1.4. Evolution of bond stresses inside a cracking RC structure behavior

The formation of a strain peak in correspondence to a crack is quite sudden as the bar abruptly passes from carrying a total strain of ε_{s2} to a much larger one (ε_{s1}) in an instance (a jump corresponding equivalent to Equation 1.4). This is translated into a cresting of the strains in correspondence to the crack until value ε_{s1} . As we move away from point A in the crack, concrete is present again and once more the steel is able to slowly transfer to it part of the stresses. Steel strains automatically decrease up until points C₁ where the strain compatibility is fully re-acquired. Meanwhile, in the DOFS sections where the concrete is still fully intact, the compatibility between the two materials is also still intact resulting in a uniformly flat line as in Figure 1.3b.

The third and last stage is the one of stabilized cracking. With the evolution of the tensile load in the member, the latter is split into various segments (defined by the cracks) which do not crack any further as their length s_r is insufficient to allow the stresses in the concrete to reach the tensile strength f_{ct} ($l_t < s_r < 2l_t$) (as in Figure 1.3c). According to literature (Gintaris Kaklauskas et al. 2017), the average length of such a block corresponds to the mean distance value between cracks s_{rm} and was determined to be in range of $1.3l_t - 1.5l_t$.

1.1.3. Influence of concrete shrinkage on the performance of Reinforced Concrete structures

Concrete shrinkage and creep, often referred to as time dependent deformations, can be responsible for structural safety and serviceability concerns of Reinforced Concrete (RC) structures even when designed according to design codes. Comprehensive reviews of the concrete structures' creep and hygrothermal effects can be found in (Chiorino 2019; Sassone & Chiorino 2005) and in (Bažant & Jirásek 2018) along with the detailing of analyses methodologies and mathematical models for structures highly sensitive to creep and shrinkage. Generally, whenever studying the effects of shrinkage on the performance of a RC structure, it should be kept in mind that, if to consider an isolated plain concrete element, shrinkage alone would simply shorten it without causing any stresses. Instead, in RC structures the presence of reinforcement embedded inside concrete, effectively restrains the concrete's shrinkage deformation, leading to the rise of a compressive state in the reinforcement and tensile stresses in the concrete surrounding the rebar.

Generally, shrinkage of concrete is considered as the alteration in its volume in unloaded and unrestrained conditions at a constant temperature. It manifests itself as a compressive deformation whose magnitude, even in its earlier stages, can be larger than the cracking load of concrete structures (Kaklauskas & Gribniak 2011). Literature suggests that shrinkage of concrete occurs in two different stages: early (within the first 24 h) and later stage. Shrinkage at both stages is an

overlapping of the effects of autogenous, drying and thermal shrinkage. In long term, the carbonation shrinkage is also added which leads to an accumulated effect (Wu et al. 2017). The first kind of shrinkage is the autogenous shrinkage which occurs in the early days after concrete casting and is predominantly responsible for the structural volume change (Lura 2003) in particular for high strength concrete (Holt 2005). It is a result of chemical shrinkage affiliated with the hydration of cement particles (withdrawal of water from the capillary pores due to the hydration of the previously un-hydrated cement), a process known as self-desiccation. This kind of shrinkage occurs in absence of moisture exchange (it is measured with sealed concrete specimens in order to prevent any moisture exchange with the surrounding environment (Liu et al. 2016)) and it is mainly a function of temperature and the water/cement ratio (Loukili et al. 1999; Jensen & Hansen 2001; Oliveira et al. 2017; Lura 2003). The second, defined as drying shrinkage, is a volume reduction of concrete which occurs due to the evaporation of moisture stored in gel pores of the concrete in unsaturated air environments with no applied stress. It is a function of the concrete's structural element thickness and exposed surface, porosity, free water content and fineness of the binder (Acker & Ulm 2001). In addition to drying and autogenous shrinkage, the concrete is also subjected to volume reductions due to chemical, thermal changes (deemed as driving force to the autogenous shrinkage (Bullard et al. 2011)) and carbonation reactions.

Among several external factors influencing the concrete shrinkage, ambient relative humidity and water-cement ratio are the most predominant. As a matter of fact, Havlasek (Havlásek & Jirásek 2016) and Gribniak (Gribniak et al. 2007) expressed shrinkage strain as non-linear function of relative humidity while Jasiczak (Jasiczak et al. 2015) and J.J. Brooks (Brooks 2015) showed that shrinkage is directly proportional to the water-cement ratio. Lesser but still relevant influence on shrinkage is attributed to improper curing of concrete in a dry environment (Henkensiefken et al. 2009; Collepardi et al. 2005; Folliard & Berke 1997), intrinsic properties of the concrete mix (self-compacting concrete exhibits higher shrinkage due to its higher amount of fines compared to conventional vibrated concrete (Oliveira et al. 2017) or presence of fly ash and silica fumes reduce the pore size in concrete and lower its permeability (Neville 2011)), aggregate properties and structural geometry (development rate of shrinkage is higher in thinner or smaller elements).

The compressive state that rises in the concrete surrounding a rebar and restraining its shrinkage is the core of what makes shrinkage such a harmful phenomenon for RC structures. Indeed, if the tensile stresses surpasses the concrete's tensile resistance, cracking would start occurring and its development, if left uncontrolled, could affect the structural serviceability and durability (ACI 2008; Sakata 1996; Liu et al. 2016; Seddik Meddah & Tagnit-Hamou 2011; Goel

et al. 2007). Additionally, cracks may allow the ingress of chlorides towards the reinforcement which results in corrosion and the potentially premature rupture of the steel reinforcement (Wu et al. 2017; Craeye et al. 2011; Ghourchian et al. 2013). Even when early age cracking does not occur, the residual stresses due to restrained early-age deformation can substantially reduce the remaining tensile-carrying capacity of concrete (Nguyen et al. 2017). Finally, being tension-stiffening (contribution from concrete to member stiffness) amongst the most influent parameters on RC structures deformations (Gilbert 2007; Gribniak et al. 2010), the necessity to assess the large influence that shrinkage exercises on it has been recognized, among others, in Bischoff (Bischoff 2003), Gilbert (Gilbert 2001) and Kaklauskas et al. (Kaklauskas & Gribniak 2011). The present thesis authors (Kaklauskas et al. 2018) and Bischoff (Bischoff 2001) have attempted to establish the notable effect of shrinkage on the tension stiffening in both an empirical and numerical manner. It is common ground that the influence of shrinkage on tension stiffening estimates is an underestimation of its value.

The effects of shrinkage on the performance of RC structures may not be limited to long drying periods of time. Indeed, in many applications freshly poured concrete is loaded after only three to four days later, making it essential to study the extent of shrinkage-induced strains present in the constitutive materials at this stage (Claisse & Dean 2016). For example, “urban verticalisation” is steadily expanding all over the world thus seeing an increased erection of high-rise buildings whose construction process, often referred to as high-speed construction, sees a 2-3 floors casting per week (Bester & Nicholas 2013). Also, individual structural elements may not be allowed to fully develop their characteristic strength before being loaded in order to meet project time targets. For example, a slab may be used to support the formwork for another floor or other structural elements, or it may be loaded with masonry blocks for walls before being laid (Shema & Abejide 2009).

1.2. Distributed Sensing for the assessment of the strain profiles of rebars embedded in Reinforced Concrete structures for the study of bond stress and slip between concrete and steel

The importance of monitoring the strains of rebars embedded inside RC structures for understanding and confronting their serviceability issues is a well-established concept. Yet, performing such monitoring in an efficient manner has always represented a challenge. By “efficient manner” the following points are contemplated:

- The measurements need to be highly accurate.
- The spacing between monitored points should be as small as possible in order to detect all the strain profile trend variations no matter how minimal they are.
- The deployment of the sensors should be as un-intrusive as possible in order not to alter the performance of RC structures.
- The sensor should report strictly the strains of the element on which it is bonded, i.e., the rebar. Therefore, it is crucial to isolate it from any shear deformation transmitted by the surrounding concrete.
- The measurable strain threshold should be as high as possible in order to allow for the monitoring of high-magnitude strain phenomena such as the steel yielding stage of RC structures.

In the past numerous attempts were developed with the goal of measuring the strains in a RC member's rebar. In order to do so, Mains (Mains 1951) introduced the idea of performing the strain measurements from the inside of a rebar. First, the latter had to be cut in two halves, then their middle had to be milled out in order to make space for the sensor of choice and finally the two halves had to be glued back together. The groove had to be sufficiently wide to provide enough space for the wiring. This technique was used by several more researchers such as Bresler and Bertero (Bresler, Boris 1968) and Kankam (Kankam 1997). The latter reportedly instrumented Ø25 rebars by first sawing them in two longitudinally and later milling them in the middle and providing channels 6 mm wide and 5 mm deep to install 3 mm long strain gauges and 9.5 mm wide and 4 mm deep for 5 mm long strain gauges. Also Houde (Houde 1974) and Scott and Gill (Scott & Gill 1987) proceeded in their tests in a very similar manner. Despite the revolutionary aspect of such test at the time, the technique is time-consuming, labor-intensive, and rather invasive. All of this in addition to being punctual in nature (Kankam positioned the strain gauges at 25 mm distance while Houde at 1.5'' corresponding to 38.1 mm).

A different approach to the monitoring of rebar strain profiles sees the employment of quasi-distributed sensing systems. Its most advanced example are Fiber Bragg Gratings (FBGs) i.e., quasi-distributed optical fiber sensors in which a characteristic wavelength is used to simultaneously provide its address in the sensor network, and the measurement (temperature and strains) (Ferdinand 2014). Davis et al. (Davis et al. 1997) used FBG in order to monitor the strains of reinforced concrete beams and decks tested to failure. Kaklauskas et al. (Kaklauskas et al. 2019) proceeded to use FBG with the goal of analyzing different experimental setups (Strain Gauges versus FBG) for the acquisition of reinforcement strain distribution data. In particular, the authors reported FBGs measuring strains with a spatial resolution of 20 mm which began diverging from

the expected results at higher loading stages. Yet, FBG, just like strain gauges, have one evident shortcoming, namely the limited strain location range monitoring. This shortcoming prevents the study of the overall global strain state of the structure in addition to the impossibility of precisely pointing out the location where the damage first occurs and prevents the linking of local damage mechanisms to the global condition of the structure (Deif et al. 2010).

This limitation is entirely surpassed by Distributed Optical Fiber Sensors (DOFS). These sensors are very thin glass wires (125 μm of diameter, similar to a hair's thickness as in Figure 1.5) able to accurately measure strains (1 $\mu\epsilon$), temperature and vibration in 2 or even 3 dimension and in a completely-distributed manner (every 0.6 mm, 1.3 mm and 2.6 mm) with flexible spatial resolution and of the desired frequency (down to 250 Hz) (Luna Innovations Incorporated 2020).

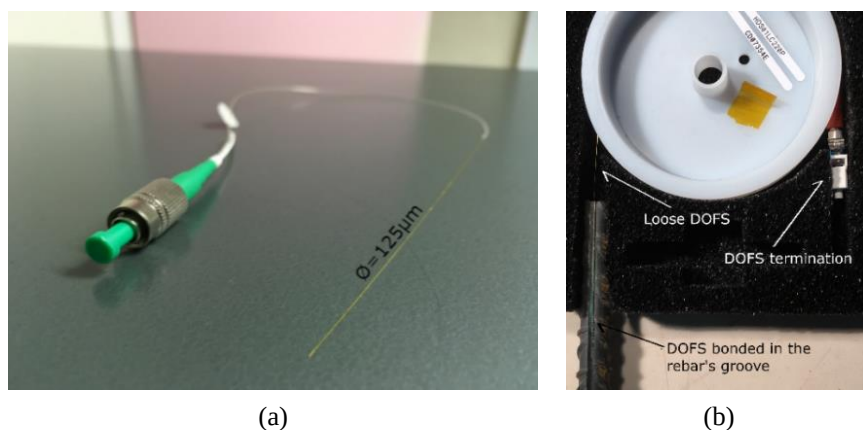


Fig. 1.5. (a) Distributed Optical Fiber Sensor (DOFS) with loose termination (b) and bonded to a rebar

Figure 1.6 summarizes the performance difference between DOFS and the other previously presented monitoring tools. As previously mentioned, traditional monitoring tools can carry out measurements only on a punctual scale, as represented in Figure 1.6a for a whole and for a cracked RC element. Whilst the situation can be improved by installing several sensors along the monitored axis, thus using a quasi-distributed sensing monitoring system (illustrated in Figure 1.6b), the resulting profile would still lack sufficient accuracy to properly represent an actual strain profile. Distributed sensing, instead, provides complete access to this data (Figure 1.6c). On the grounds of DOFS' sensitivity and elevated spatial resolution, the small but important strain profile variations (such as the stress transfer transmission zones as in Figure 1.6c) are finally entirely detectable.

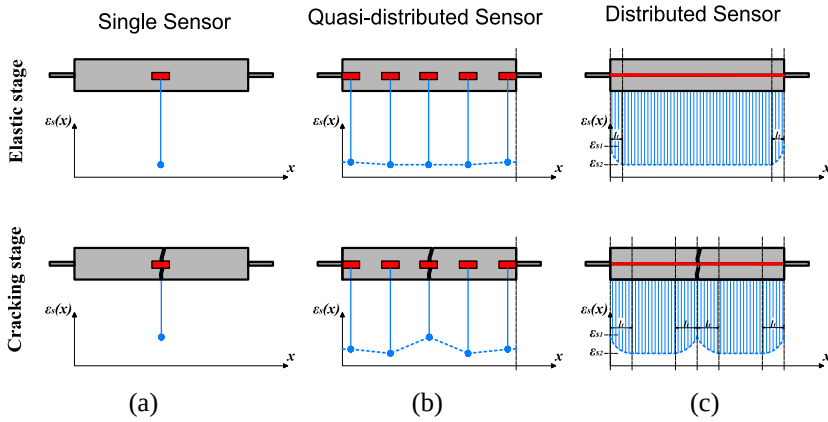


Fig. 1.6. Illustration on the performance difference between (a) Single or Punctual monitoring sensors (b) Quasi-distributed monitoring sensors or (c) Distributed monitoring sensors

Single and distributed sensors have the disadvantage of not only failing to capture small profile variations but also requiring a non-neglectable installation work load.

In order to obtain a high spatial resolution with OFDR-based sensors, a very narrow light pulse is originated by an Optical Backscatter Reflectometer (Figure 1.7), resulting in a proportionally lower level of the backscattering signal and, at the same time, an increased receiver bandwidth, essential requirement in order to detect such signals (Barrias et al. 2016).



(a)



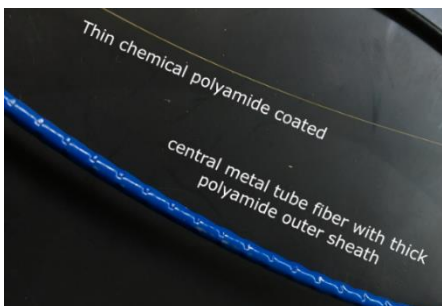
(b)

Fig. 1.7. (a) ODiSI-A OBR (b) ODiSI 6000 OBR

In more detail, Rayleigh scattering originates from the interaction between the electromagnetic pulse propagating in the fiber silica core and the latter's inherently random silica impurities. A sampled Rayleigh backscattering, measured whenever a DOFS is in an unstrained state, constitutes a “fingerprint” against which any later backscattering measurements are compared against in order to determine the strains that the fiber has been subjected to. Indeed, whenever a DOFS is subjected to an external stimulus (like strain), the backscatter pattern presents a spectral shift that the OBR can convert into a strain variation value.

Three different types of scattering processes may occur in a DOF sensor, Raman, Brillouin and Rayleigh scattering, but all hold particular optical features that make one more suitable than others relative to the research objectives. While the Raman scattering is characterized by high dependence on temperature it can also be used to extract physical and chemical information of a material (Cantarero 2015) such as food quality or explosive materials (Gillibert et al. 2018). Brillouin backscattering, thanks to its extended measurement range capability (up to several kilometers) is the most studied and used DOFS system in civil and geotechnical engineering (Bado & Casas 2021). The third scattering phenomenon is Rayleigh scattering. Despite its 70 m sensing range limit, it can provide a high spatial resolution (up to 1 mm), which is ideal for strain and damage monitoring in concrete structures. The latter is often used in experimental laboratory investigations having the DOFS bonded either to its surface, to monitor its strains and damages and/or on the reinforcement bars that will later be embedded in the concrete.

A fiber silica (SiO_2) glass core is usually encapsulated in a cladding made of polymeric material or nylon which can vary in diametrical size, shape and manufacturing process. The external coating can be constituted by a numerous kinds of materials, layering, thicknesses and external textures. Figure 1.8 displays the two kinds of fiber used in the present thesis.



(a)



(b)

Fig. 1.8. (a) Thin and thick polyamide jacket covered DOFS and (b) thick DOFS

In particular, the first (yellow) has a chemical polyamide cladding whilst the second (blue) has a central metal tube and a thicker polyamide outer sheath adding up to a nominal diameter of 3.2 mm.

Whenever dealing with DOFS with thicker coatings, it should be assured that no shear lag occurs in the strain transmission between the monitored element and the fiber core (Bado & Casas 2021). Indeed, a shear lag induced by the shear deformation of intermediate layers (adhesives, fiber cladding, fiber coatings, etc.) alters the measured strains. In order to compensate for such, the shear lag should either be accounted for or a DOFS with minimal coating should be employed.

The increasing popularity of Optical Fiber Sensors (OFS) is evident in their equally increasing number of OFS-related articles published in conference proceedings (Ferdinand 2014) between 1983 to 2012 as in Figure 1.9.

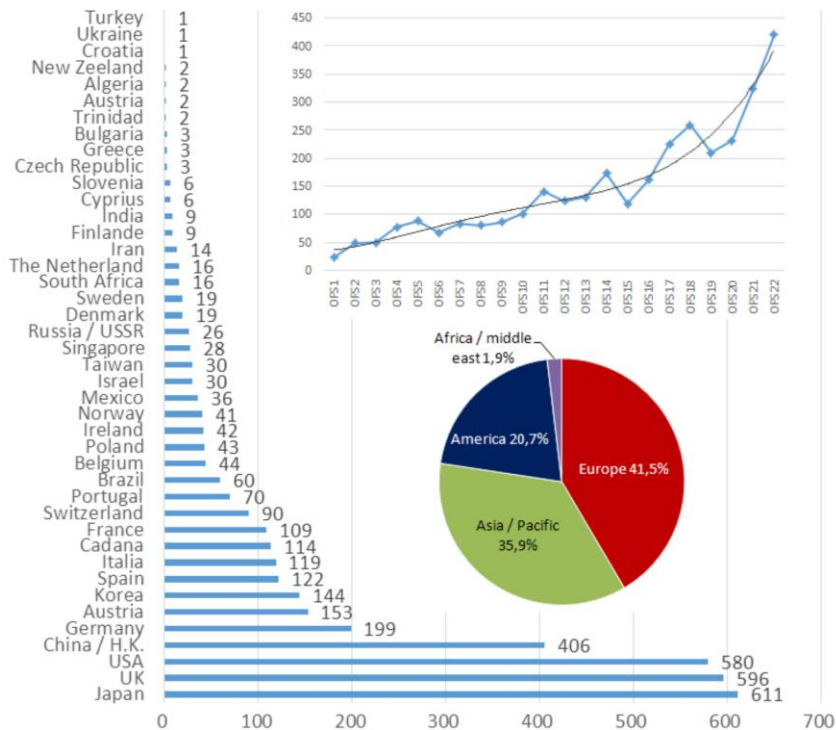


Fig. 1.9. OFS papers per origin (countries from 1983 to 2012)

Furthermore, Figure 1.10 displays that, according to the data presented by a market survey report conducted by the Photonic Sensor Consortium and published by Information Gatekeepers, the growth of the DOFS technological market is projected to be over \$1.3 billion by 2023.

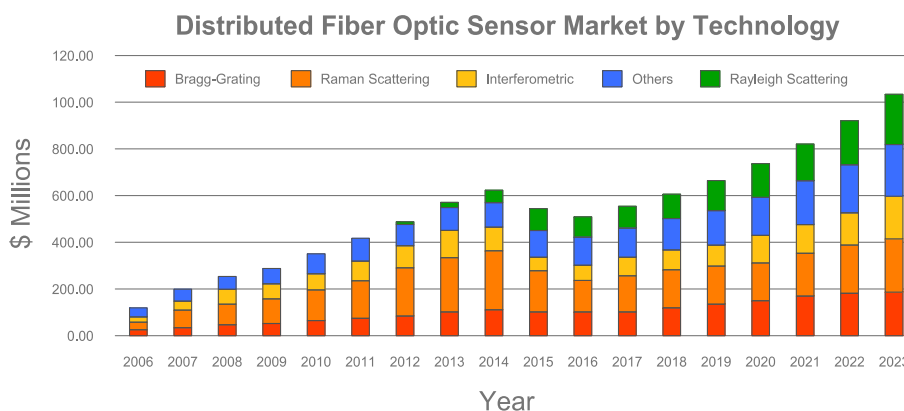


Fig. 1.10. Growth of the DOFS technological market

Henceforward a brief review of experimental campaigns with DOFS-instrumented RC tensile members is presented.

Some authors (Davis et al. 2016) used thin DOFS fibers to monitor the impact of corrosion on the performance of RC structures. Tension tests were conducted on bare reinforcement and RC tensile members subjected to various stages of (accelerated) corrosion. The first stage, 0 corrosion, was fundamentally a DOFS monitored double pullout test on a RC tensile member. The combination of the two fibers was again used by the same authors (Davis et al. 2017) in an attempt to measure the shrinkage and tension stiffening in RC tensile members.

Sieńko et al. (Sieńko et al. 2018) glued thick DOFS to the degreased reinforcement bar of a RC tensile member in order to verify the suitability of a standard “tight jacket” OFDR DOFS for the strain analysis of RC members. The fiber used in the present publication is different from the ones used in the tests present in this thesis due to the presence of a protective jacket that increases the DOFS cable diameter around 7 folds (0.125 mm versus 0.9 mm). According to the authors, the results showed good measurement accuracy for micro-cracks and narrow crack widths (range up to 0.05 mm) and only moderate accuracy in the case of wider cracks (up to 0.30 mm). This was attributed to the slip between the DOFS core and its external jacket leading to fiber strains not corresponding fully with the actual deformation state of the concrete member.

Zhang et al. (Zhang et al. 2020) investigated the ability of DOFS to detect concrete cracking and large strain steel deformation of six 120x120x1000 mm RC tensile members with different OFDR DOFS cable structures, coating materials and diameters which were both embedded in the concrete and bonded to the rebar. The article reported how OFDR successfully detected continuous rebar deformations and the evolution of the concrete-steel bond stress up until and

beyond the yielding range as well as identifying the crack-indicative strains. The authors highlighted the potential of DOFS combined with double pullout tests for the evaluation of existing RC structure behavioral predictive models through the bond stress analysis.

Quiertant (Quiertant et al. 2012) and Michou (Michou et al. 2015) introduced a novel technique for protecting the DOFS against external hampering factors, consisting in incising a groove on the surface of the rebar and positioning the DOFS inside it before securing it with adhesive. Cantone et al. (Cantone et al. 2021) embraced this technique and provided a comprehensive work on the determination of the rebar-concrete interaction by means of OFDR DOFS and DIC monitored RC structures tested under pullout, double pullout and bending conditions. In particular, the authors positioned the DOFS inside 1 mm-depth grooves performed along two or four rebar sides. During the multi-cycle double pullout tests the authors were successfully able to track the increasing bond degradation providing results in sound agreement with the literature.

Finally, Malek et al. (Malek et al. 2019)'s work is also worth of mention as they utilized OFDR DOFS for the study of the steel/concrete bond degradation due to reinforcement yielding in RC beams subjected to lateral loading. The performed experimental program, representing a drastic departure from the common bond/slip determination tests (i.e., the single and double pull-out tensile tests), was based on a pseudo-static pushover test of four 25x35x285 cm cantilever RC beams tested vertically. Instrumentation-wise, two different types of optical fibers were used, a PVC-coated tight buffer OFS cable (for both internal and external sensing) and a polyimide-coated fiber (bonded on the longitudinal rebars). Thus, during the test, the strains were captured continuously at a spacing resolution of 2.6 mm and with an accuracy of 25 $\mu\epsilon$ both on the outside and on the inside of the specimen. This allowed for the determination of the concrete/steel slip, steel stress, bond stress, bond deterioration length and the locations of cracks.

1.3. Conclusions of Chapter 1 and formulation of the tasks of the thesis

The theoretical concepts introduced in the present chapter on represent the backbone of the present thesis of which the following key points were:

1. Between the stress transfer and mean strain approach, the former, albeit more demanding, is undoubtedly the superior method for the serviceability analyses of RC structures, providing a realistic reflection of their behavioral nature, of its bond-slip mechanisms, of the effect of tension stiffening and of the crack development.

2. In order to determine the key parameters of the stress transfer approach i.e., concrete-reinforcement bars (rebars) bond stress and slip, a correct assessment of the strains of the rebars embedded in RC structures is required. A novel tool able to achieve this objective with unprecedented accuracy and performance (strain resolution $1\text{ }\mu\epsilon$, spatial resolution 0.63 mm and measurement frequency 250 Hz) are Distributed Optical Fiber Sensors (DOFS).
3. The viability of DOFS for the validation, integration or dismissal of modern-day RC structures' serviceability models is yet to be determined, thus the present doctoral work.

In order to achieve this objective, the following tasks were undertaken:

1. DOFS' strain monitoring performance was compared against that of other classic tools as a starting point for the analysis.
2. DOFS-monitored experimental tests were performed in order to extract the strain profiles of the steel rebars embedded inside RC ties subjected to tensile testing and shrinkage-induced compression. The performance of these tests were progressively boosted thanks to express research on DOFS bonding techniques and measurement cleansing algorithms.
3. The test data was processed in order to assess the magnitude of the steel-concrete bond stress/slip, crack spacing and concrete shrinkage-induced tensile strength reduction.
4. The experimentally extracted serviceability parameters were finally used to validate, integrate or dismiss several RC structures' serviceability models.

Experimental Campaigns for Reinforced Concrete Structures' Serviceability Analyses

The present section will introduce the reader to the authors' experimental efforts aimed at establishing the performance of DOFS for RC structures' serviceability analyses. The two comprised sub-sections will first tackle the technological aspects behind a DOFS-monitored experimental campaign and then describe the actual campaigns performed by the author. This chapter includes the material presented in journal publications (Bado et al. 2018; Bado et al. 2020; Bado & Casas & Dey et al. 2020; Bado & Casas & Kaklauskas 2020; Bado & Casas 2021).

2.1. Towards a standard technology for Reinforced Concrete structures' Distributed Sensing-monitored tests

Being DOFS such a modern technique, whilst their working principle is well known, many uncertainties still exist on how to guarantee their functioning and maximum performance. The years separating the present moment to the one that

the author first started working with DOFS were crucial to the definition of a methodology aimed at guaranteeing the best outcome of experimental DOFS-monitored campaigns. The author's work and publications have contributed to the slow unraveling of such quandary and the present section intends to define exactly how.

The first matter at hand consisted in checking the performance of DOFS and its ability to report strain readings similar to the ones sampled by other monitoring tools such as strain gauges and extensometers. Whilst the monitored strains were not expected to be identical (strain gauges also suffer a certain degree of uncertainty mainly connected to the adhesive used to bond them to the structural surface) the order of magnitude was expected to be similar. The second issue was the lack of a standard DOFS/structural surface bonding technique which guarantees the successful strain measurement in each of the fibers' sensing points. The third issue was connected to the recurrence of anomalistic readings (see Figure 2.1) in every previously reported experimental campaign with DOFS which, if left unaddressed, could hamper any future experimentations.

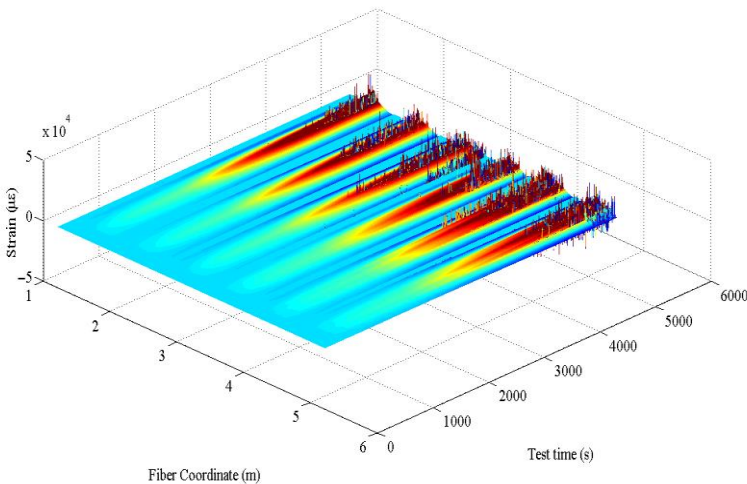


Fig. 2.1. Three-dimensional representation of the strains measured by the DOFS over time along the fiber.

In order to avoid such, it was crucial to establish the reason behind the insurgence of such Strain Reading Anomalies (SRAs). The three previously mentioned issues were the grounding of the author's first experimental campaign. The latter and its main conclusions are shortly described in the following. Multiple publications based on DOFS-monitored experimental campaigns report coming across strain-reading anomalies (SRA) in the form of localized and excessively

large strain peaks (Barrias et al. 2017; Brault et al. 2017) which have seemingly little to no physical meaning or explanation. For instance, the DOFS may report in a single point of the fiber large tensile strain peaks followed by equally large compressive strain peaks in the span of a few seconds. These anomalies do not seem to be restricted to a particular model of Optical Backscatter Reflectometer (OBR) as they seem to rise up in both ODiSI-A model (Barrias et al. 2017) and OBR 4600 (Davis et al. 2016; Brault et al. 2017), both manufactured by LUNA Technologies (Roanoke, VA, USA). Such readings can be considered abnormal for the following reasons:

- their sheer size.
- the large difference with the previous readings despite the load difference between the two being almost negligible.
- the occurrence of large compressive peaks in a homogenously tense area of the structure.

This acute proximity among tensile and compressive strain peaks can also be present geometrically speaking. Indeed, such opposite strain peaks may be reported in neighboring sections of the monitored structure even a few millimeters distant. This is abnormal considering that usually both sections are entirely and homogenously in tension or compression and further illogical considering their co-existence at a very short distance from one another. The phenomenon here described has little to no physical meaning suggesting that it might simply be an erroneous DOFS strain-reading. This assumption is confirmed when comparing the monitored structure's behavior under loading with theoretical models and simultaneous strain gauges monitoring.

In the present case, none of the above methods concurs with the DOFS's reported strain peaks suggesting that these are in fact SRAs. These anomalies should obviously be avoided as they prevent the possibility of getting reliable measurements in the single location of the anomaly (time-wise and geometrical-wise anomalies) and extensively increase the level of unreliability of the surrounding measurements.

As an answer the author conceived an experimental test aimed at pin-pointing the motive behind the SRAs. The test intends on inducing these in a controlled environment under different conditions to isolate and identify the cause of their origin. The experiment sees the loading of seven parallel S500 steel reinforcement Ø20 bars, six of which were instrumented with a single 5 m long DOFS (Bars 1–6) and one is instrumented with strain gauges for comparison purposes (Bar 7). The test surveyed the DOFS performance for different bonding conditions, rebar sections and loading speed. Indeed, two kinds of adhesives were used to bond the DOFS to the bars. Cyanoacrylate for Bar 2 and Bar 5; silicone for Bar 3 and Bar 6; cyanoacrylate with an extra silicone layer on top for Bar 1 and Bar 4. Furthermore,

Bars 4, 5, 6 were incised along their upper face to locally reduce their sectional area in order to simulate different transmission lengths present around a RC member's cracked section. The reduced area of the rebars leads to increased carried stresses in the incised sections, similarly to what happens in RC beam sections that have cracked compared to those that have not. A steep edge of the incision can be compared to a short transmission length l_t while a sloping one can be compared to a long l_t . The strains in these three rebars are closely monitored to verify whether a drastic section gradient causes SRAs first compared to softer incision edges. During the test the specimens are subjected to two loading speeds and two impact loads. Initially the specimen is loaded at a speed of 0.35 kN/min. At 880 s an impact load of 0.1 kN is applied followed by a 60s hold. At its conclusion, the specimen is again loaded at 0.35 kN/min until, at 1835s, a second impact load (1.0 kN) is applied. As before, a 60s hold follows, at the end of which the specimen is loaded at a quicker speed of 1.12 kN/min until the specimen's plasticity is reached. Figure 2.2 displays some of the results of the present test.

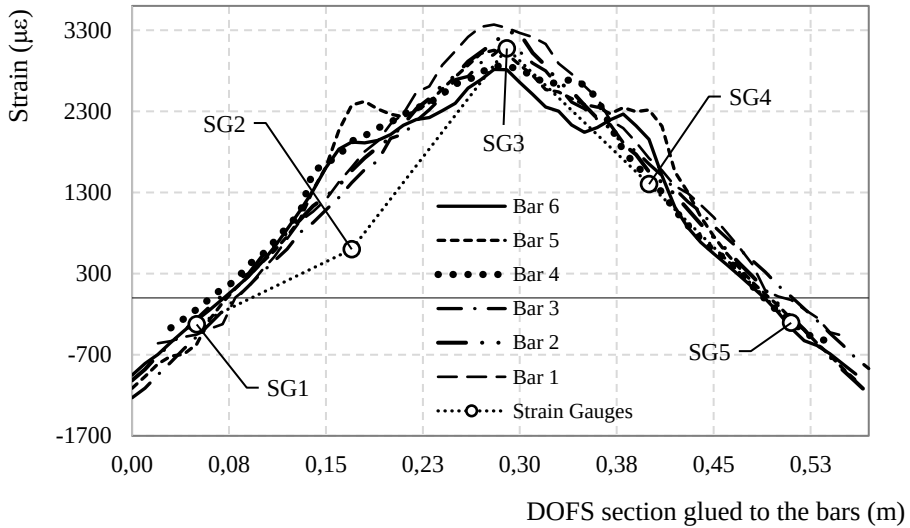


Fig. 2.2. DOFS strain profiles at specific time and global load instances along the test

Figure 2.2 compares the strain profile of all the seven rebars at a global load of 30.0 kN (3089 s). The similarity among them is evident (except for the local increase in strain in the location of the incisions) along with the correspondence between the strain gauge's and the DOFS's measurements. Strain gauge 2 seems to be the only one not reporting correct values, probably due to a slight damage occurred during the positioning process. This embodies one more advantage of

the DOFS. Indeed, differently with the strain gauges, the latter does not require fixing of every single sensing point, making it so that the correct positioning of the fiber is enough to ensure the reading capacity and reliability of each point. The comparisons among each bar's DOFS strain profile along with their coherence with the strain gauges' measurements is a testament to the performance and the reliability of the DOFS strain sensing technique. This conclusion is not unique to the author's research as other authors reached similar conclusions in their work (Barrias et al. 2019b; Barrias et al. 2019a).

In conclusion the test demonstrated the ability of DOFS to perform reliable measurements up until around 4000 $\mu\epsilon$ despite the application of several impact loads and the presence of transversal incisions along the fibers. Concerning the adhesives, the outcome of the test sees silicone to be the most resistant adhesive therefore ensuring a delayed appearance of SRAs and in smaller number to the expense of the precision of the strain measurements. Oppositely, the rigid but fragile nature of the cyanoacrylate provides the results with highest precision to the expense of resistance and endurance. In fact, the SRAs seem to spring up earlier and in bigger quantity in the latter. Meanwhile the combination of the two gives birth to a hybrid behavior. Regarding the adhesives, it was concluded that there was not a particularly better one, but it was rather a function of the objectives of the researcher. Concerning the insurgence of the SRAs it was concluded that the main cause is the friction between concrete (especially when cracking) and the DOFS.

The author further compared the proximity of measurements performed by the DOFS and an extensometer for multiple DOFS-instrumented rebars put into tension. The results of such comparison are visible in Figure 2.3.

As visible, the strains sampled by the two measuring tools are also very similar. We can go even as far as to say that in this particular case the DOFS fiber performed better than the extensometer as its reported strain value at the end of the test has gone back to zero whilst the extensometer hasn't completely (Figure 2.3a). This is due to the uncertainties connected to how you attach the extensometer to the bar and the slip that might occur because of any unintentionally clamped rib. Otherwise the DOFS' measurements, being the tool completely bonded to the rebar in a mechanical manner, do not suffer of this uncertainty. Once again this is in agreement with other researcher's work (Barrias et al. 2018).

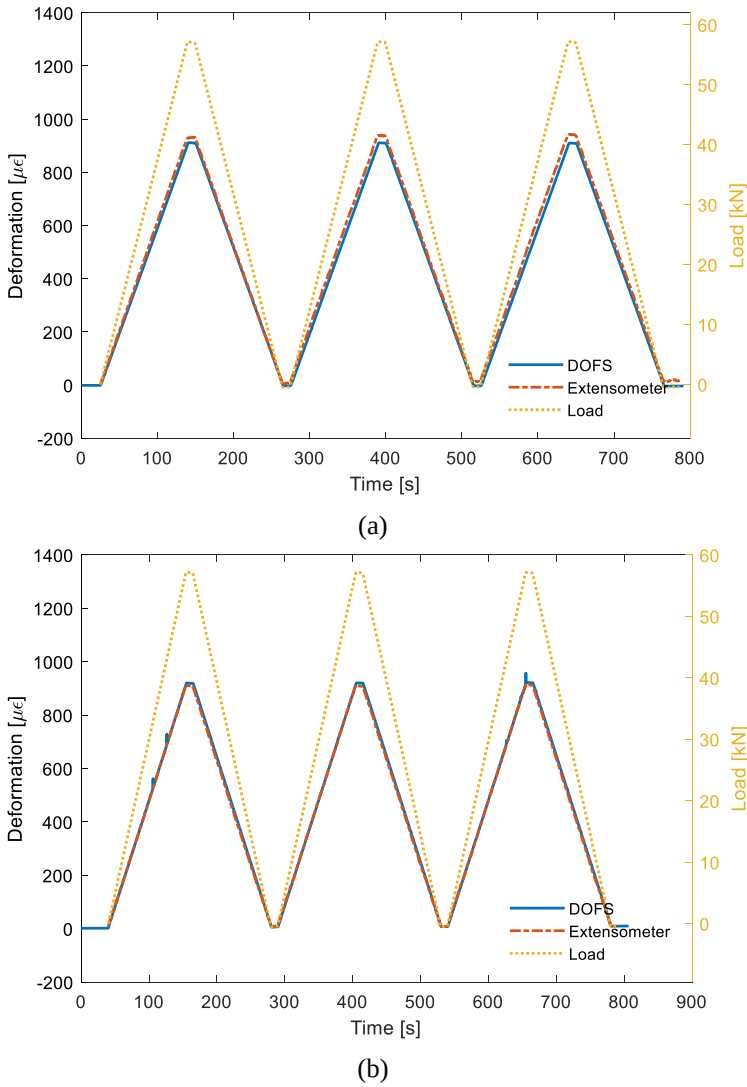


Fig. 2.3. Strain measurements as sampled by DOFS and an extensometer for a rebar put into tension with DOFS bonded with just cyanoacrylate (a) and with a combination of cyanoacrylate and silicone (b)

DOFS' results proximity to other more conventional monitoring tools establishes the reliability of the measurements performed by the latter, practically elevating it to the most accurate, simple in its use and performant tool on the market.

In order for the author's successive DOFS-monitored experimental campaigns to be successful it was necessary to define which DOFS/structural surface bond technique represented the best compromise between measurement accuracy and protection of the fiber. Furthermore, it was essential to create an algorithm able to remove the anomalies from the DOFS-extracted test data when and if these were to appear. Starting from the former, the main challenge was the lack of reported experimental campaigns with DOFS that included the latter being embedded inside the RC structure itself. Most applications of such sensors revolved around simply bonding on the surface of a structural element to monitor the superficial deformations and opening of cracks. Consequently, with no indication of which techniques seems to consistently work, the author had to come up with different bonding techniques. These are:

- Cyanoacrylate (CYN) on bare rebar.
- Cyanoacrylate on bare rebar with a protective silicone (SI) layer on top.
- Cyanoacrylate on incised rebar.
- Cyanoacrylate on an incised rebar with a protective silicone layer on top.

A graphical representation of such techniques can be found in Figure 2.4 below whilst pictures taken during the experiment are displayed in Figure 2.5.

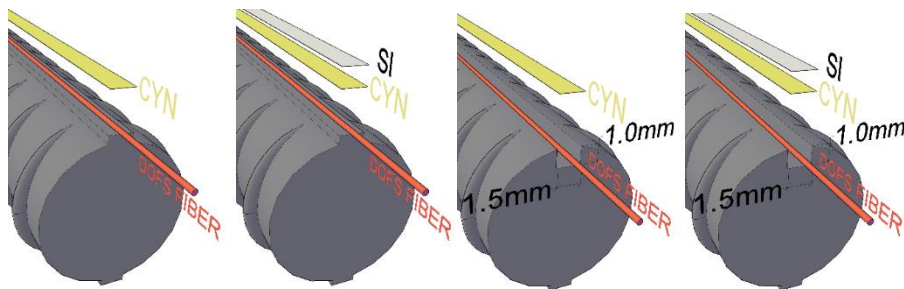


Fig. 2.4. Four DOFS/rebar bonding and protection techniques

As mentioned previously in Chapter 1.2 the idea of incising a groove in the rebar in order to position the fiber inside it is not novel but inspired by Michou's work (Michou et al. 2015). Over the tests these techniques performed differently thus providing alternate quality results but the inclusion that currently seems to yield the best results is the silicone's. The test data presented in the current thesis reflects the quality difference of the various DOFS/rebar bond techniques. Consequently, some measurements include Strain Reading Anomalies (SRAs) which, in the post-processing are to be eliminated.



(a)



(b)

Fig. 2.5. Different bonding techniques (a) no groove + CYN + SI and
(b) no groove + CYN

In order to achieve such result, different SRA-cleansing algorithms were created but only the most fitting to laboratory testing will be reported here. The Polynomial Interpolation Comparison Method (PICM) is a routine based on the very simple concept of what is an error. The core idea is based on the development of an approximated polynomial curve of the study case DOFS section's ε -t profile which encompasses the maximum number of reliable strain readings of the profile (effectively representing its trend). The polynomial degree can vary to provide improved fitting even though, generally talking, it should be neither too low (which would lead to non-representative tendencies), nor too high (because the trend would in return be affected by the punctual anomalies). Following the definition of a maximal error between the two curves' points, whenever the error is higher than such established limit, the reading is considered as an SRA. This is achieved through a very simple function that checks the difference between the reading and its corresponding expected value (belonging to the ε -t curve approximation) as in Equation 2.1.

$$\begin{aligned}
 Error_i &= |\varepsilon_{DOFS,i} - \varepsilon_{APROX,i}| \\
 \text{if } Error_i > Error_{max} &\Rightarrow \varepsilon_{DOFS,i} = SRA \\
 \text{if } Error_i \leq Error_{max} &\Rightarrow \varepsilon_{DOFS,i} \neq SRA
 \end{aligned}
 \tag{2.1}$$

The choice of the threshold value $Error_{max}$ has to be, as always, a result of an in-depth study of the measured tendencies and peaks. Figure 2.6 depicts the algorithm required to perform a PICM data post/processing. The latter requires as input data the raw strain matrix, its previously developed polynomial fit and a maximum error that is accepted between them (maxError).

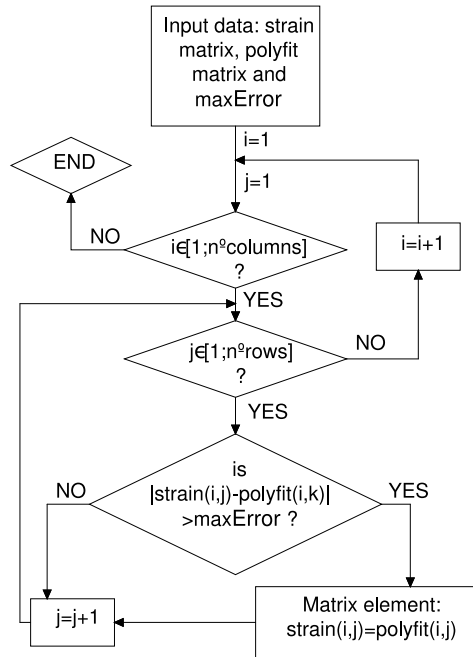


Fig. 2.6. SRAs detection algorithm with a PICM tool

As mentioned, the algorithmic post-processing is finalized to cleaning the raw data provided by the OBR from as many SRAs as possible. The outcome of this process helps the data interpretation by removing fictitious and erroneous light backscattering algorithmic interpretation-based strain readings. In order to understand its functioning it is important to distinguish harmful SRAs (HF-SRAs) from harmless ones (HL-SRAs) (Bado et al. 2018). The former is a kind of anomaly which, whenever it first occurs in a specific DOFS coordinate, is then followed strictly by flawed strain measurements for the remainder of the experimental test. It goes without saying that HF-SRAs are particularly disruptive for DOFS-monitored campaigns. Instead when HL-SRAs first occurs in a particular DOFS coordinate it may still be followed by reliable readings. Their elimination procedure differs substantially. Starting from the latter, being the

nature of this anomaly punctual in nature, the entire trend of every DOFS sensing section has to be studied. Figure 2.7 reports the strain measurement taken by DOFS coordinates 1.1775 m of a member defined as 8x8_D8 whose geometrical and mechanical characteristic will be defined later.

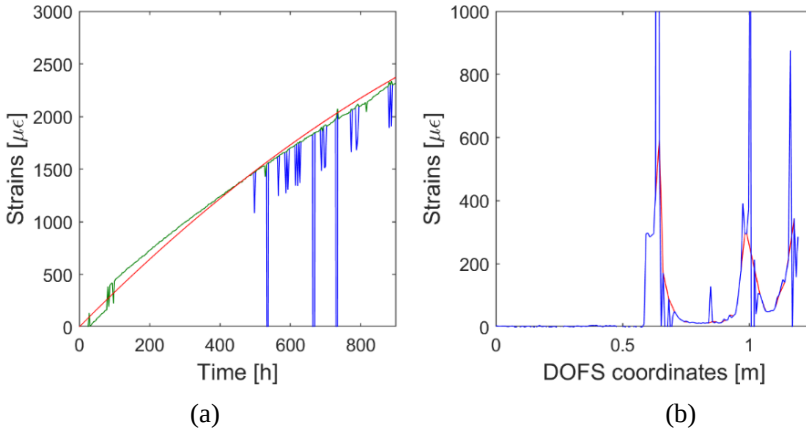


Fig. 2.7. DOFS measured steel strains versus (a) time and (b) DOFS coordinates

Using the computational algorithm, the global trend is fitted and plotted (in red) by means of a polynomial and compared with the original OBR measured raw data (in blue). If the difference between them is less than a defined threshold ($100 \div 200 \mu\epsilon$), equivalent to the desired sensibility of the comparison, then the raw data is considered valid (in green). If this condition is not met the strain measurement under scrutiny is replaced by a linearly interpolated value of the first reliable strain reading measured before and after the specific DOFS coordinate. HF-SRAs, instead, are much harder to diagnose by means of a computational algorithm. Hence, DOFS coordinates displaying the mentioned signs of such anomaly are manually substituted, once again, by a linearly interpolated value of the first reliable strain reading measured before and after the specific DOFS coordinate.

In conclusion, the definition of a methodology aimed at preventing the appearance of SRAs in sampled DOFS strains and the creation of an algorithm to eliminate the ones have appeared represent the two main pillars on the grounds of which DOFS can finally become a completely reliable and flexible tool for strain monitoring inside RC structures. The author's work on these two topics is aimed at making that a reality thus creating the perfect tool for the future of RC serviceability analysis.

2.2. The experimental campaigns

The current section is aimed at describing the experimental campaigns by the means of which DOFS' potential for the extraction of valuable and innovative data for serviceability analyses of RC structures.

Multiple specimens were designed, produced and tested by the author in different experimental campaigns. All are RC tensile members (henceforth referred to as RC ties) with different rebar diameters, concrete prism transversal and longitudinal dimensions and cracking behavior. RC ties are often used to illustrate cracking, deformation and bond behavior of RC structures thanks to their simplicity and reasonably good representation of the distribution of internal forces and strains in the tensile zone of RC structures (Ruiz et al. 2007). This last point essentially represents the reasoning behind the choice of this particular RC structure as focus of the present experimental investigation.

The tested RC tie specimens will be henceforward referred to through a code such as 10x10_D12 where the first two numbers are indicative of the cross sectional dimensions (in cm) and the last of the embedded rebar diameter (in mm). Specimen 10x10_D12, for example, is characterized by a 10x10 cm cross section with a 12 mm diameter rebar embedded inside it. Figure 2.8 below illustrates the general geometry of RC tie with a DOFS deployed on their rebar.

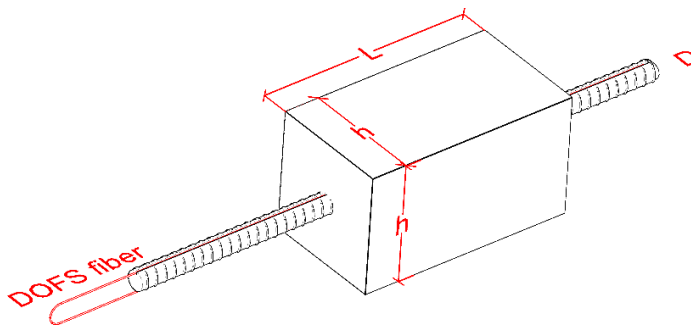


Fig. 2.8. RC tie standard geometry

The author performed three experimental campaigns on RC ties. These varied in the tested samples, scale, OBR interrogator model and overall objectives. Indeed, whilst the core principle remains the same i.e., monitoring of the steel rebar strains during the testing of the RC ties, the possible implications of the results are numerous and varied. By means of the DOFS strain profiles the author was able to:

- Learn the stress state present in the steel and in the surrounding concrete at any instance of the test.
- Study the performance of various bonding techniques in order to assess which performs best, namely the technique that ensures the best insulation of the DOFS from the concrete friction whilst guaranteeing the least amount of measurement alteration.
- Assess the occurrence of concrete shrinkage-induced cracking and residual tensile strength reduction.
- Gain insight on the bond stress and slip between the concrete and steel rebar occurring on their interface.
- Analyze the crack spacing.
- Analyze the crack width.

The performed experimental monitoring campaigns encompassed all these goals. Moving on to the description of the tested specimens, their geometrical features are listed in Table 2.1 and Figure 2.9 as per the parameters illustrated in Figure 2.8.

Table 2.1. Geometrical features of the tested RC ties

Experi- mental campaign	Specimen	h, mm	L, cm	Rebar diameter D, mm	Number of specimens
1	10x10_D12	100	60	12	1
2	8x8_D8	80	16, 33	8	1
2	9x9_D12	90	30, 15	12	1
2	10x10_D16.01	100	14.5, 14.5, 30	16	1
2	12x12_D12	120	19, 24	12	1
2	12x12_D16	120	40, 20	16	1
3	15x15_D20	150	21	20	1
3	10x10_D16.02	100	60	16	3

As visible the specimens are characterized by different geometrical features such as rebar diameter $\varnothing_s$, cover size, reinforcement ratio ρ_s and prismatic length L. Additionally, the specimens were designed to have two different cracking behaviors, namely a cracking one (thus being characterized by longitudinal sizes superior to $2l_t$) whilst others not (longitudinal size inferior to $2l_t$). Furthermore, some RC ties are composed by a single concrete prism whilst others by more.

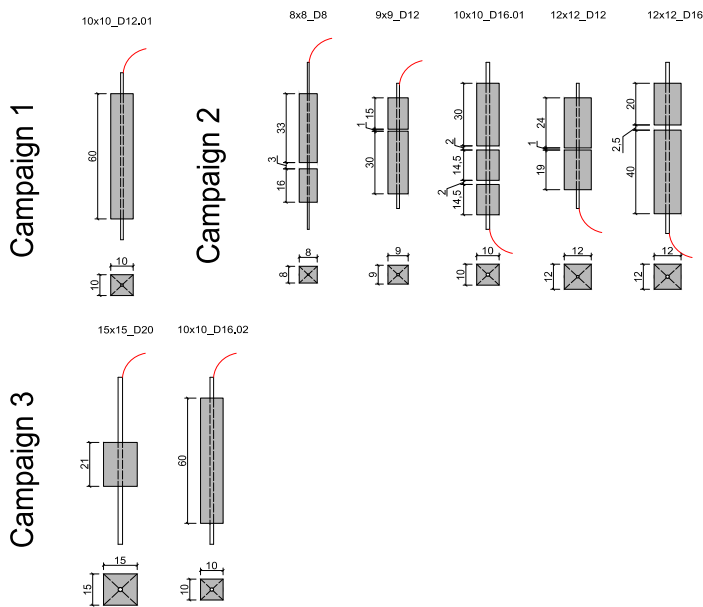


Fig. 2.9. Geometrical features of the tested RC ties

Member 8x8_D8, for example, is included in the former category whilst member 10x10_D12.01 in the latter. This geometrical peculiarity allowed for the studying of the strains inside two differently sized RC ties (one short and one long) by means of a single rebar and of a single DOFS. As a matter of fact, the tensile value applied on the edges of a RC ties is uniformly transmitted all along the rebar, hence it is equal in every concrete prism located on the same rebar. Figure 2.10 displays the RC ties before testing.



Fig. 2.10. Pictures of the DOFS-instrumented RC ties

Table 2.2 details the concrete mixes with which the specimens were obtained, whilst Table 2.3 and Table 2.4 specify the concrete and steel mechanical properties.

Table 2.2. Concrete mixes for the different campaigns

Campaign	1	2	3
Cement, kg/m ³	363	353	363
Water, kg/m ³	182	148	182
Fine aggregate .04, kg/m ³	484	475	484
Fine aggregate .02, kg/m ³	484	475	484
Coarse Aggregate, kg/m ³	968	950	968
Super Plasticizer, kg	0.5% weight of cement		

Table 2.3. Mechanical properties of the RC ties constitutive materials

Campaign	1	2	3
Compressive strength f_c , MPa	37.65	40.94	47.89
Tensile strength f_{ct} , MPa	2.96	3.18	2.89
Modulus of Elasticity E_c , MPa	34939	31198	28159

Table 2.4. Steel rebar's mechanical properties

Member	Steel yielding stress f_y , MPa	Modulus of Elasticity E_s , MPa
10x10_D12	541.22	200000
8x8_D8	561.50	192800
9x9_D12	541.22	197770
10x10_D16.01	506.50	191250
12x12_D12	541.22	204953
12x12_D16	551.51	192271
10x10_D16.02	533.31	200000
15x15_D20.01	550.00	200000

The average concrete mechanical characteristics were established at 28 days in accordance with BS EN 12390. The characteristics of Campaign 1 and Campaign 2 were assessed with 150 mm sided cubic samples (f_c) and with 150x300 mm cylinder samples (f_{ct} , E_c) whilst Campaign 3's with 100 mm sided cubic sample (f_c), 150 mm sided cubic sample (f_{ct}) and 100x200 mm cylinder samples (E_c). The noticeable difference in compressive strength between G1 and G3 (produced with the same formula) is mostly attributable to the difference in specimen size (the measured strength of a specimen decreases with an increase in its size (Neville 2011)), but also to possible differences in the aggregate properties (Campaign 1 and Campaign 3 were produced in different laboratories). The reinforcement mechanical characteristics (f_{sy} , f_{su} and E_s) were instead obtained in accordance with ISO 6892-1:2009 (specified in the standard BS EN 10025).

Moving on to the description of the test setups, each campaign attempted to monitor the concrete-shrinkage induced compressive strains occurring in the rebars as a consequence of its restraint of the concrete contraction. In particular, these have many similarities and some differences, the biggest of which are the members' geometries and the OBR used to measure the strains inside the DOFS. In particular the first two campaigns used an ODISI-A (LUNA Technologies 2013) model whilst the second an ODISI 6000 (Luna Innovations Incorporated 2020) model, both developed by LUNA Technologies. The latter, being a much more modern OBR model, not only provides higher performances but also higher reading frequencies and spatial resolutions.

In order to ensure an easy member demolding process, polyester or plywood sections were chosen as delimitation of the different prisms. The use of these materials is advantageous as it secures the correct positioning of the rebars on each member's centroid axis through the drilling of symmetrical holes for the passage of the rebar in each polyester/wood prism. After casting and demolding the specimens (1 day later), the ones pertaining to Campaigns 1 and 2 were kept cured at a controlled temperature of averagely 24 °C and 59% of relative air humidity whilst the ones pertaining to Campaigns 3 were kept at a temperature of averagely 20 °C and 35% of relative air humidity. In order to ensure that the water evaporation occurs in all longitudinal faces, rollers were placed beneath the specimens for proper ventilation of the bottom face (as illustrated in Figure 2.11). This serves also the purpose of minimizing the friction between the specimen and its support.

The members of Experimental campaign 1 and 3 were continuously monitored for a 28 days time span whilst the ones of Experimental campaign 2 for only 130 hours (approximately 6 days). Consequently, the latter were only used for short-term drying monitoring. All the specimens were later subjected to monotonic tensile stress at a constant loading speed of 0.1 mm/min by means of a Universal Testing Machine.

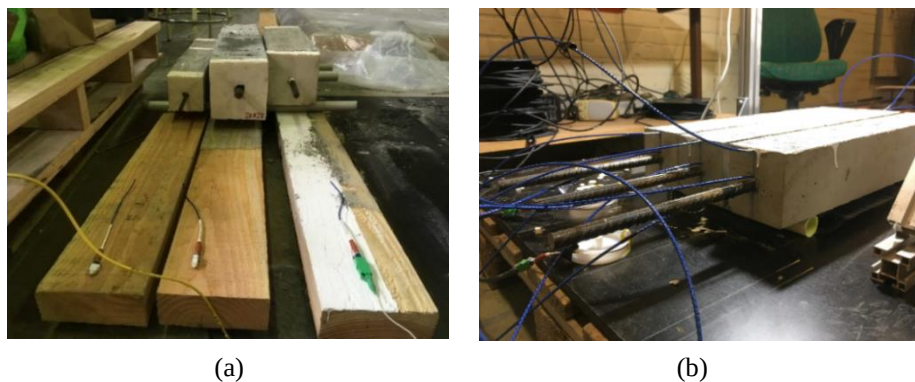


Fig. 2.11. RC members subject of the experimental campaign (a) 1 and 2 and (b) 3

The strains inside the members were monitored through 1.2 m long DOF sensors (bonded to the rebars) by means of an OBR ODiSI-A (campaigns 1 and 2) and ODiSI 6000 (campaign 3). The selected spatial resolutions were 5 mm and 7.5 mm with measurements every 3 to 5 seconds with ODiSI-A and 1.3 mm with measurements every 0.5 seconds with the ODiSI-6000 model.

The concrete prisms' superficial displacements were also measured. This was achieved through Digital Image Correlation (DIC) technology (Figure 2.12).

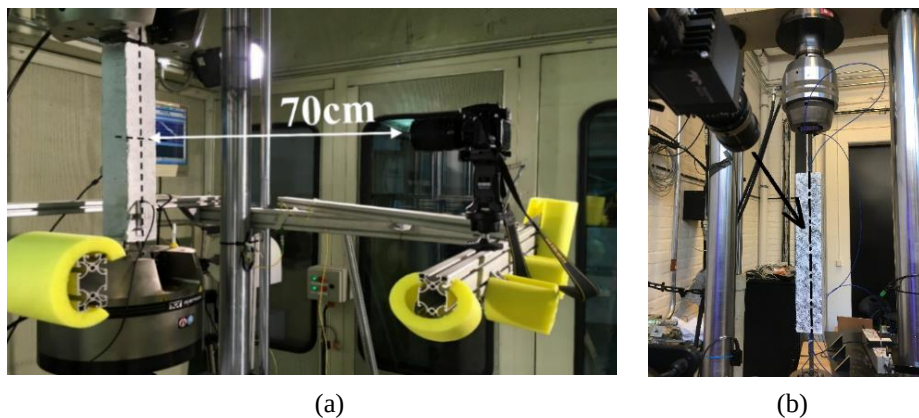


Fig. 2.12. Digital Image Correlation setup (a) with a camera and (b) with a GOM manufactured DIC setup

The latter is an innovative, cheap non-contact optical technique for measuring strain and displacements of a previously primed surface. DIC works by

comparing digital photographs of a component or test piece at different stages of deformation. By tracking blocks of pixels of a complex stochastic pattern (Figure 2.13a), the system can measure surface displacement and build 2D and 3D deformation vector fields and strain maps (McCormick & Lord 2010; Fayyad & Lees 2014) as visible in Figure 2.13b and c.

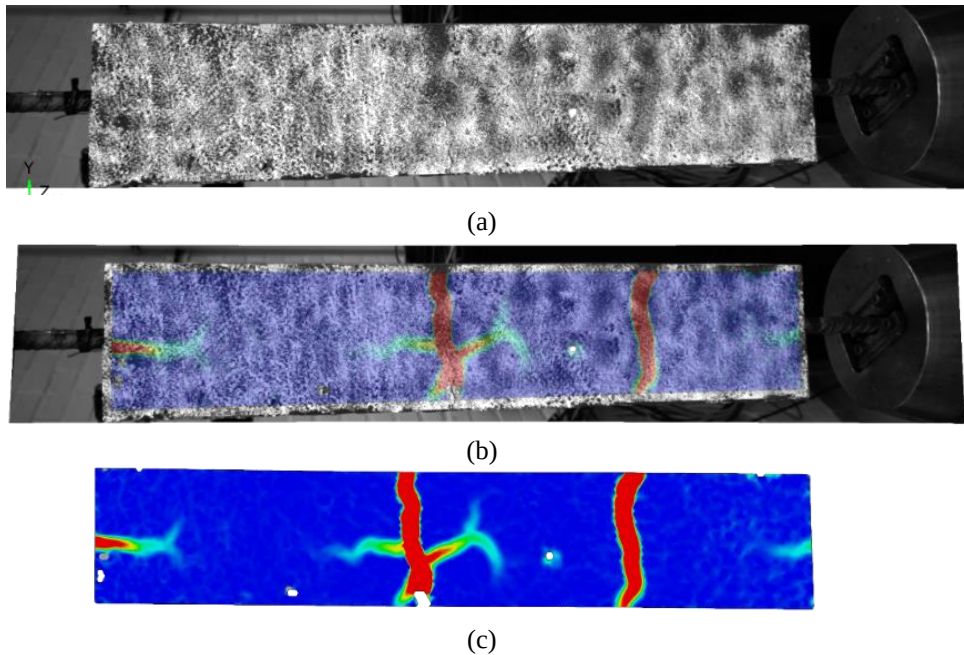


Fig. 2.13. (a) Stochastic pattern (b) 2D deformation vector field overlapped with the picture of the specimen and (c) the 2D deformation vector field only

DIC utilizes a correlation algorithm to obtain displacements and deformations which identifies areas of matching grey-scale values in the stochastic pattern of the deformed and un-deformed images (Antoš et al. 2017). As mentioned earlier, in order to perform a DIC analysis, each image must have a clear pattern that is later divided into a grid of interrogation cells containing a finite number of pixels. Therefore, the resolution and accuracy of the displacement and deformation fields depend on the total number of pixels within the images and on the level of suitability of the pattern to the expected displacements. In order to act on the first point, a Canon EOS 77D camera was utilized. The latter was able to take pictures up to 24 megapixels (6000x4000pixels). The camera was positioned 70 cm away from the members (Figure 2.12a), perpendicularly to the intersection between its vertical and horizontal axis.

The stochastic pattern, instead, was sprayed with black mate paint but only after the member's concrete surface was primed with white paint in order to create an even background. Finally, an LVDT was glued to the member's surface in order to ascertain the correctness of the DIC measurements. Finally, the software analysis was developed with the help of GOM Correlate software by GOM Precise Industrial 3D Metrology.

2.3. Conclusions of Chapter 2

The findings and observations from the author's experimental investigation on RC tensile elements with DOFS-instrumented rebars can be summarized as follows:

1. DOFS strain monitoring readings correlate well with the ones extracted by conventional strain monitoring tools such as strain gauges and extensometers.
2. The DOFS deployment technique that ensured its best insulation from the concrete friction - thus safeguarding the measurement performance- whilst guaranteeing the least amount of interference possible was combination of a groove incised in the rebar, cyanoacrylate and silicone.
3. Eventual anomalistic strain readings sampled by DOFS can be compensated for and corrected by means of mathematical algorithms.
4. Experimental efforts demonstrated DOFS is capable of extracting crucial parameters for serviceability analyses of RC structures i.e., (a) the stress state present in the steel and in the surrounding concrete at any instance of the test -even beyond yielding- (b) the occurrence of concrete shrinkage-induced cracking and tensile strength reduction (c) the bond stress and slip between the concrete and steel rebar occurring on their interface (d) crack spacing and (e) crack width.

Distributed Sensing for the Validation, Integration or Dismissal of Reinforced Concrete Structures' Serviceability Models

Classical approaches to the serviceability of RC structures have either considered a perfect interaction between steel and concrete or a simplified constant bond relation. Either way, due to the lack of tools that could accurately assess the strains inside RC structures, they adopted theoretical, empirical or numerical solutions to define the analyses' parameters. Modern research, instead, advocates the use of the stress transfer approach which suggests an interaction between the rebars and its surrounding concrete in the form of a force transfer (bond stress). The latter provides a realistic reflection of the behavioral nature of RC, of its bond-slip mechanisms, of the effect of tension stiffening and of the crack development. Still, the issue of accurately investigating the strains inside of a RC structure persists. With the introduction of DOFS, accurate, completely-distributed, experimentally measured strain profiles of the RC structure's constitutive materials are finally possible. DOFS finally fills the gap that ever-present instrumental gap holding back the reliability of RC structure serviceability models. The current chapter will show how to marry the potential of DOFS with the one of the stress transfer for

the models' substantiation, integration and/or provision of data for the formulation of new ones. This chapter includes the material presented in journal publications (Kaklauskas et al. 2017; Bado & Kaklauskas 2017; Kaklauskas et al. 2018; Bado et al. 2018; Bado et al. 2019; Bado et al. 2021; Bado et al. 2021).

3.1. Assessment of the concrete-steel bond stresses and slip through Distributed Sensing

The stress transfer approach (also referred to as partial interaction or discrete crack based approaches) portrays in a realistic fashion the inner-workings of a RC structure and is crucial for any accurate serviceability analysis. Therefore it is of key importance to be able to assess as accurately as possible its constitutive parameters: concrete/steel bond stress τ and slip s .

The most recurrent experimental technique for studying bond-slip is the direct pull-out test (Figure 3.1a), in which an embedded reinforcement bar is directly pulled out from a concrete interface. Yet, according to a few modern research, the conventional pullout test fails to provide the real bond condition (Fédération Internationale du Béton 2000) because of its:

- Support-induced compressive stress field in the concrete.
- Non-uniform distribution of bond stress around the anchorage length.
- Instigation of lateral confining pressure at the reinforcement-concrete interface.

Alternatively, a more realistic way to study bond/slip is the double pullout test as is the case for the current experimental campaigns. This test is essentially a tensile RC tie test (Figure 3.1b) integrated with monitoring sensors able to sample and report the evolution of the strains along the segment of the reinforcement bars embedded inside concrete. Thus, being the local response of the bond affected in each point by the longitudinal strain state of the bar $\varepsilon_s(x)$, the bond slip behavior is simply obtained by the governing equivalence (Ruiz et al. 2007) indicated in Equation 3.1.

$$\frac{d\varepsilon_s(x)}{dx} = \frac{4}{E_s \phi_s} \tau(x), \quad (3.1)$$

where E_s represents the steel's modulus of elasticity and ϕ_s the steel rebar's diameter. Whilst some experimental campaigns (Houde 1974; Scott & Gill 1987; Murray et al. 2018) perform such study on a concrete block defined by two consecutive RC ties' cracks, some others (Kankam 1997; Kaklauskas 2017) do

the same on members of length sufficiently reduced not to welcome any transversal cracks during its testing ($L < \text{mean crack spacing value } s_{rm}$).

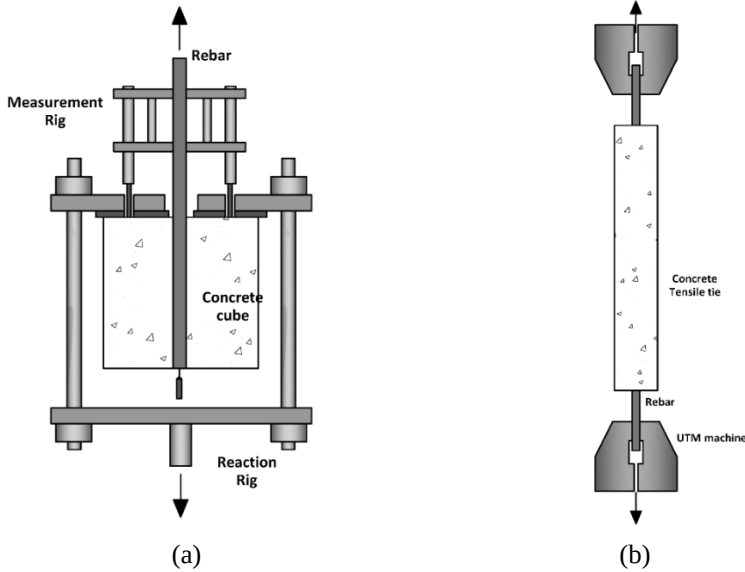


Fig. 3.1. Different methodologies of assessing the bond stress and slip in a RC member: (a) pull-out test and (b) double pull-out test

Effectively, the stress transfer phenomenon occurring on the edges of the former occurs in a similar fashion on the edges of the latter. In both cases the bond action is significantly reduced by the presence of slip-induced localized concrete damage. Therefore, it is safe to assume that the bond stress present in a cracking RC tie can be simulated by a smaller one with identical cross-sectional dimensions which will additionally be free of cracking-induced uncertainties (such as oblique and/or not fully developed cracks, steel redistribution and double pointed peaks). In either case, an accurate measurement of the variation of ϵ_s along the rebar is the key to an equally accurate bond stress/slip depiction.

In conclusion, the experimental campaigns described in the following chapters are aimed at assessing the stress transfer approach's main parameters, namely the bond stress and slip present between steel and concrete, by means of DOFS-extracted steel strain profiles of rebars embedded inside multiple RC ties. These campaigns' experimental efforts are intended to represent the beginning of a novel DOFS-aided bond/slip quantification trend which should be as broad and universal as possible in order to encapsulate all the various facets of these phenomena. Consequently, they not only embrace both above described bond/slip

testing methodologies (cracking and non-cracking members) but adds further diversity in its RC samples by varying both their transversal and longitudinal dimensions (Figure 2.8).

3.1.1. Distributed Sensing monitoring for bond stress/slip assessment: *modus operandi*

The present sub-chapter will elucidate the *modus operandi* for the quantification of bond/slip present in the interface between steel reinforcement bars and concrete on the grounds of DOFS extracted strain profiles. In order to achieve this, a full analysis on a specific member 8x8_D8 will be developed. In the next chapters, instead, the analysis of each of the other members will be bypassed for conciseness purposes and just the final bond stress/slip values will be displayed.

Before starting, the reader should keep in mind that Ø8 is a rebar diameter whose bond stress has hardly been investigated due to its reduced size preventing the allocation of relatively bulky sensors (strain gauges with their large cabling or FBG sensors) neither on its surface, nor in its core. Therefore, DOFS is possibly the best and only way to instrument such rebars.

Figure 3.2 represents the strain evolution of a Ø8 rebar embedded inside member 8x8_D8's concrete prisms at several instances along the test, as reported by the OBR.

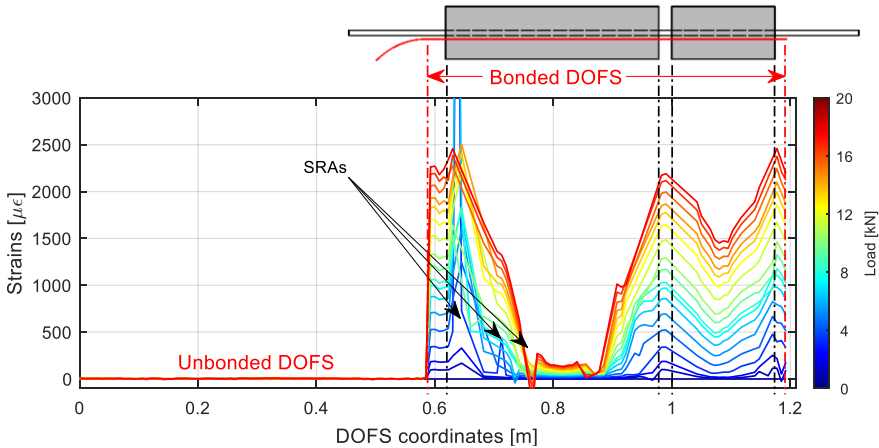


Fig. 3.2. Steel strains as measured by DOFS at various load stages of the tensile test inside member 8x8_D8

In order to help the interpretation of this figure and all the following DOFS strain profiles, Table 3.1 summarizes the DOFS coordinates of the beginning and end of each DOFS monitored segment.

Table 3.1. Significant DOFS coordinates for the study case RC ties

Label	First DOFS monitored section, m	Segment 1		Segment 2		Last DOFS monitored section, m
		Length, mm	DOFS coordinates, m	Length, mm	DOFS coordinate, m	
8x8_D8	0.58	330	0.63–0.96	160	1.00–1.16	1.19

The strain readings related to DOFS coordinates preceding the “First DOFS monitored section” (usually oscillating around the value of $0 \mu\epsilon$) are irrelevant as they are originated in un-bonded and loose fiber segments (labeled as “unbonded DOFS” in Figure 3.2). Furthermore, the table elucidates the DOFS coordinates corresponding the start and end of the different concrete blocks. When a member includes two blocks, their intermediary DOFS coordinates, being outside of any concrete prism, report the tension-stiffening-deprived bare rebar’s strain.

A correct reading of Table 3.1 should go as follows. The first DOFS coordinate bonded to the rebar and effectively reporting useful strain measurements is 0.58 m (as expected, only strain values oscillating around $0 \mu\epsilon$ are present before such coordinate). The fiber keeps monitoring the bare rebar until DOFS coordinate 0.63 m (flat segment in Figure 3.2) after which it starts reporting the rebar’s strains when embedded inside a 330 mm long concrete prism. From 0.96 m to 1.00 m the measurements correspond to the bare rebar situated between the two concrete prisms (again a tiny flat segment in Figure 3.2). Between 1.00 m and 1.16 m the fiber reports the rebar’s strains when embedded inside a 160 mm long concrete prism and finally, between 1.16–1.19 m the fiber is once again bonded to the bare rebar on the opposite end of the RC tie.

It should be kept in mind that, in the present test, different adhesives have used to bond DOFS which resulted in differently performing monitorings. Consequently, the amount of SRAs may differ drastically in different regions of each member. Despite the DOFS extracted strain profiles having been subjected to the process of SRA-removal, in some occasion the sheer number and size of the anomalies makes it impossible to plot completely SRA-free strain profiles. This is particularly evident in member Figure 3.2’s DOFS sections 0.63–0.76 m where hardly any data can be considered reliable, especially compared to its much improved symmetrical counterpart (0.85–0.98 m). Oppositely the entirety of the following 160 mm segment presents optimal strain evolution readings.

Unexpectedly, member 8x8_D8's larger segment (330 mm) didn't crack transversally as by design; instead, only splitting cracks appeared on its edges. Indeed, Figure 3.2 shows no crack-indicative strain peaks effectively suggesting that both segments of 8x8_D8 behave as non-cracking members. This can be attributed to the lack of sufficient transversal pressure, usually provided by thick concrete covers or transversal reinforcement (both lacking in 8x8_D8), which delays the commencement of splitting failure thanks to its induced frictional force in the steel-concrete contact surface (resulting in higher bond stresses and in reduced values of slip).

Moving on to the bond/slip calculations, the stress transfer considers the strains to be directly governed by the bond stress between the rebar and concrete on the surface of the former. The opposite is also valid and the bond stress τ can be calculated simply rearranging Equation 3.1 into Equation 3.2.

$$\tau_b(x) = \frac{E_s \phi_s}{4} \frac{d\varepsilon_s(x)}{dx}. \quad (3.2)$$

It should be considered, though, that the DOFS extracted strain profiles aren't fit to be directly used as input for Equation 3.2's $\varepsilon_s(x)$. Indeed, due to the DOFS' intrinsic sensitive nature, its profiles are never perfectly smooth but rather "lumpy" as visible, for example, in Figure 3.3a.

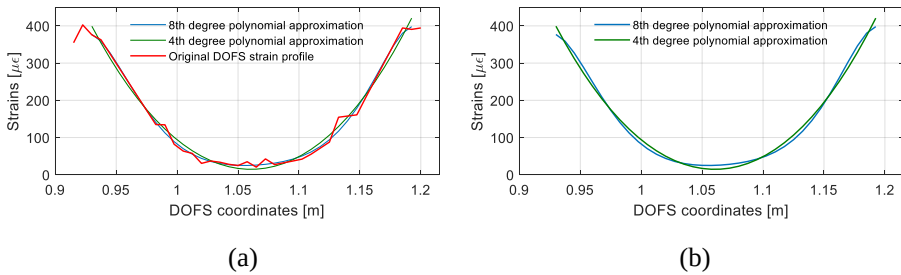


Fig. 3.3. DOFS extracted strain profile and its different polynomial approximations (a) and just the latter two for better comparison (b)

The reason for such can span from the DOFS measurement noise to the presence of aggregates in monitored cross-sections creating extra friction on the fiber. However, the overall trend is never less than evident and can be easily captured by means of simple polynomial approximations of any specific degree. For example, Figure 3.3 represents the use of a 4th and 8th degree and, as evident in Figure 3.3b, their resulting profiles display some dissimilarities. In the light of such polynomial-degree profile differences, the choice of the correct degree is crucial for a performant analysis and thus should be undertaken with care.

The steel/concrete slip, over a segment n (underscore letter in the following equations) of length Δl , is calculated as in Equation 3.3 while the total slip over half of the member $l/2$ is a simple integral as in Equation 3.4.

$$\Delta s_n = \Delta l \left(\frac{\varepsilon_{s,n} + \varepsilon_{s,n-1}}{2} - \frac{\varepsilon_{c,n} + \varepsilon_{c,n-1}}{2} \right); \quad (3.3)$$

$$\int_0^{l/2} s(x) dx. \quad (3.4)$$

The mean strain in the concrete $\varepsilon_{c,n}$ is calculated from rearranging the equilibrium in Equation 3.5 into Equation 3.6:

$$P = N_{s,n} + N_{c,n} = \varepsilon_{s,n} E_s A_s + \varepsilon_{c,n} E_c A_c; \quad (3.5)$$

$$\varepsilon_{c,n} = \frac{P - \varepsilon_{s,n} E_s A_s}{E_c A_c}. \quad (3.6)$$

The resulting bond-slip laws will be compared in the following with the Model Code 2010's as in Equation 3.7 with parameters selected considering pull-out failure (double pull-out in our case) and good bonding conditions as in Equation 3.8.

$$\tau_0 = \tau_{max} \left(\frac{s}{s_1} \right)^\alpha \text{ for } 0 < s < s_1; \quad (3.7)$$

$$\tau_{max} = 2.5 \sqrt{f_{ck}}, \quad s_1 = 1 \text{ mm and } \alpha = 0.4. \quad (3.8)$$

With f_{ck} being the characteristic compressive strength of concrete but in our case being equals to the mean compressive strength $f_{cm} = 40$ MPa. By means of these equations, Figure 3.4 plots the bond stress, slip and bond-slip curves of the study case member 8x8_D8 and, in particular, of the strains included in its DOFS coordinates interval 0.8775-0.9600 m. Furthermore, in order to clearly see the profile differences arising when using a specific polynomial degree approximation versus all the others, Figure 3.4 reports all the curves as calculated by means of 4th, 6th and 8th degree polynomial approximations. Their dissimilarity is immediately evident as the ones resulting from higher degree polynomial approximations present more fluctuations as a testament to their increased sensitivity to the strain profiles' own pendence variations.

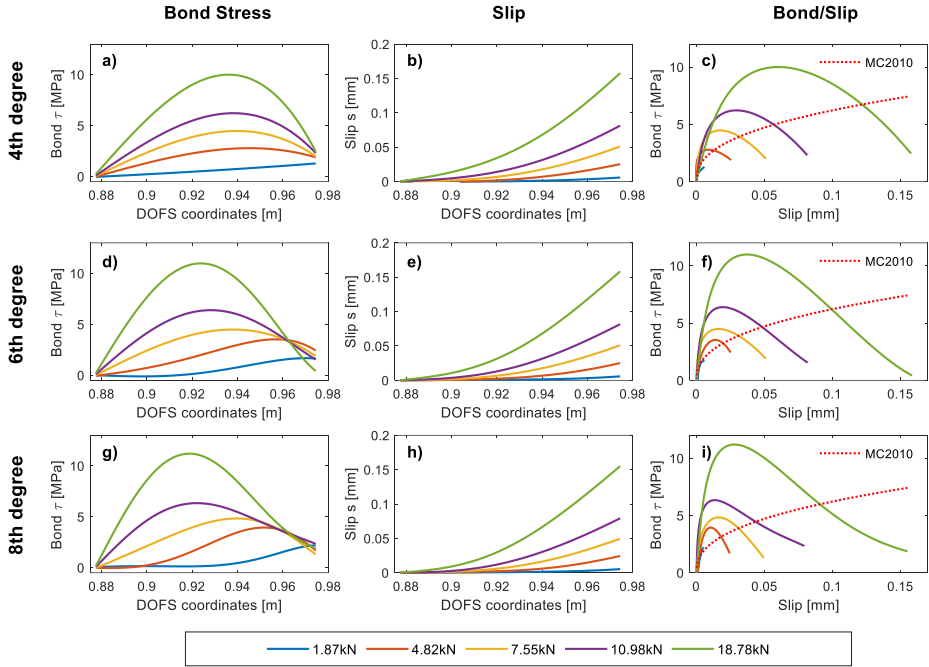


Fig. 3.4. Bond, slip and bond-slip calculation for 4th, 6th, 8th degree polynomial approximated strain profiles for member 8x8_D8 segment 1

The advantages of higher degree polynomial approximations can be perceived when comparing Figure 3.4's first and last line. Indeed, in the latter, the graduality with which the steel/concrete strain compatibility is lost along the test is more evident. As a matter of fact, with an 8th degree polynomial approximation, the bond stress graph (Figure 3.4g) clearly shows how the first DOFS section where $\tau \neq 0$ is progressively shifting inwards with the load, until this condition is only met in the member's mid-span. Differently so, with a 4th degree polynomial approximation, such evolution is undetectable. What is also more discernible in Figure 3.4g, compared to Figure 3.4a, is the progressive shift inwards of the bond profiles' maximal points. A correct evaluation of their position is crucial for the definition of the *debonding area* (also defined as bond deterioration zone or damage zone). The latter, comprised between the maximal point on one side and the contiguous member edge on the other, progressively expands inwards as the slip gradually deteriorates the bond. Finally, a higher degree polynomial approximation also represents the rise in the slip (Figure 3.4b, e and h) in a more realistic fashion.

Oppositely so, the same sensitivity that allows to better represent bond/slip of one RC member, can be counterproductive in another, especially if the latter is “lumpier” than the former. Indeed, with a higher degree polynomial approximation the resulting bond/slip curves of the latter would be characterized by curving and twisting that would not be representative of the global evolution of the bond stress but rather of the influence of punctual factors such as aggregate friction on the fiber and DOFS reading issues. In this case then, a lower degree polynomial approximation is preferable.

The bond/slip curves represented in the upcoming sub-section will follow the above elucidated guidelines.

3.1.2. Distributed Sensing monitoring for bond stress/slip assessment: cracking members

The present sub-chapter follows up the previous’ *modus operandi* applying all the elucidated operating steps to the tested cracking RC ties visible in Figure 2.8. In particular, Figure 3.5a and b display the DOFS-captured profiles of member 10x10_D12 and 9x9_D12 together with their respective substantiating DIC-based major strains analyses.

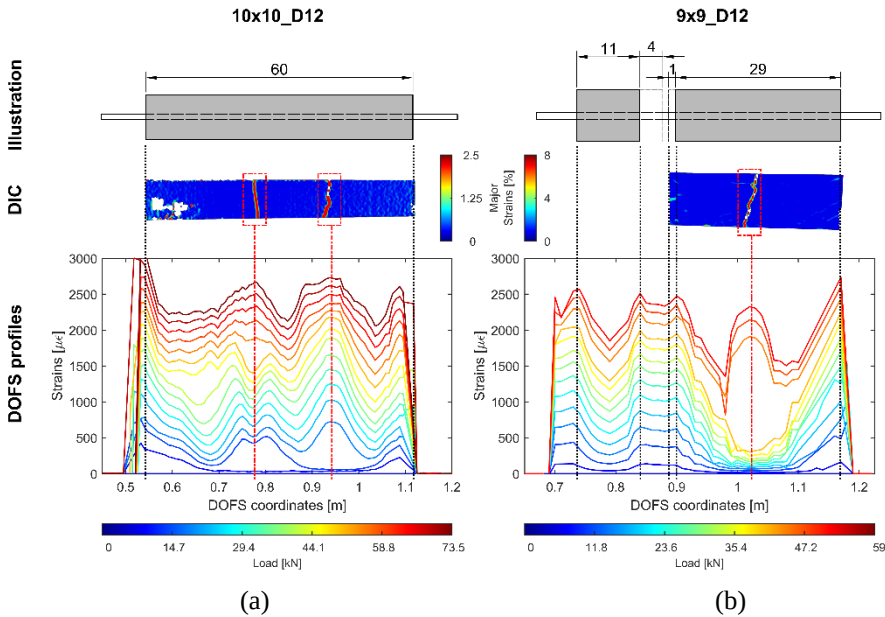


Fig. 3.5. 10x10_D12 (a) and 9x9_D12 (b) correlation between their designed dimensions, DIC analysis and DOFS extracted strain profiles

The latter are aligned with their corresponding DOFS coordinates in order to create a tight geometrical correlation between the two experimental outputs. Figure 3.5's profiles display precisely the three RC structures cracking phases described previously: the elastic, cracking and stabilized cracking phases. A few paragraphs will henceforth be dedicated to comparing the DOFS profiles with Figure 1.3's theoretical ones with the intention of assessing the former's ability to diagnose the latter. Figure 3.5's lower strain profiles are expectedly flat in the central compatibility area (ε_{s2}) whilst on the edges, in their transfer lengths l_t , they curve upwards until reaching, at its extremities, the strain value of the bare rebar ε_{s1} (as in Figure 1.3's elastic phase). 10x10_D12's third profile (corresponding to a load of 9.89 kN) and 9x9_D12's seventh (45.60 kN) present newly formed peaks suggesting the appearance of cracks and the start of the crack formation phase. It should be noted that these cracks-induced peaks appear very similarly in Michou (Michou et al. 2015) who uses an identical cross-section/rebar-diameter member. The figure demonstrates that, in reality, the strains in cracked cross-sections do not immediately reach a value equivalent to ε_{s1} as cracks often do not develop entirely in a single plane but, instead, obliquely over multiple ones, as visible in Figure 3.5's DIC images. This is due to the heterogeneous distribution of aggregates and lack of tensile strength symmetry across a RC tie's cross-section. Hence a certain amount of steel stress is transferred to the adjacent parts of concrete that are still whole thus delivering a small amount of tension stiffening.

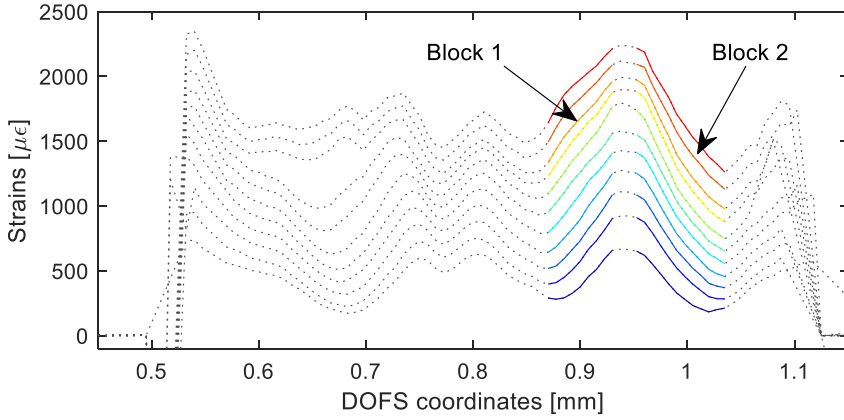
Consequently, the steel strains do not reach ε_{s1} as in Figure 3.5's strain profiles. Moving away from the crack, concrete is again present around the rebar. Consequently, the steel is able, once more, to transfer to it part of its stresses. This occurs until a point (distant l_t from the crack) beyond which the strain compatibility is fully re-acquired, translating again in a uniformly flat strain profile (as the one visible on the right of 10x10_D12's double-pointed peak). Soon after the advent of the first crack, though, crack 2 appears in the middle of such compatibility area and develops much faster than crack 1 due to its superior width. The third and last stage is the one of stabilized cracking. Here, despite the increase of the tensile load, the RC ties do not crack any further. This is due to insufficient length of the cracks-defined chunks to allow for the stresses in the concrete to surpass the tensile strength f_{ct} . According to the literature (Gintaris Kaklauskas et al. 2017), the average length of such blocks s_{rm} is considered to be comprised in the interval $l_t < s_{rm} < 2l_t$. Indeed, experimentally wise, the higher profiles of Figure 3.5a and b lack any further crack-induced peaks suggesting that the stabilized cracking stage is effectively ongoing. The profiles simply keep increasing in magnitude shifting upwards in the graph. It is also noticeable how, at higher loads, the strain peaks values become similar and equivalent to the bare rebar's due to the full opening of the cracks.

It should be noted that some of the above mentioned issues with the monitoring of cracking elements are noticeable between DOFS coordinate 0.5 m and 0.8 m where the strain profiles are disturbed by the presence of extra peaks and pendence variations that can only be explained by the obliqueness of the crack or DOFS reading issues.

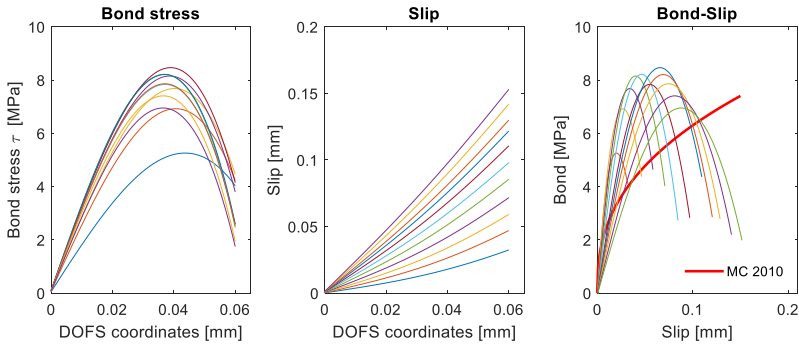
Overall, the above DOFS measurements are in good agreement with the theoretical RC tie behavioral phases. Indeed, the various stages are accurately reported both at low and high strain gradient and absolute value. On the topic of the maximum measurable strain threshold both embedded DOFS continue sampling reliable measurements beyond the cracking stage, deep in the stabilized cracking stage up until the steel yielding strain. This testifies to the resistance of the DOFS as embedded sensors. Finally, a good agreement can also be found between the crack positioning as reported by DOFS and by DIC demonstrating each other's compatibility and integration potential.

Unfortunately, of the two above displayed cracking members, only 10x10_D12's strains are accurate enough to warrant a bond stress and slip analysis therefore it will be the only one analyzed in the present sub-chapter. 9x9_D12's non-cracking member, instead, will be studied in the following one. Figure 3.6a displays 10x10_D12's two regions that are sufficiently un-affected by crack-induced alterations to allow for a bond/slip analysis. The latter's results are reported in Figure 3.6b and c. Being the two blocks characterized by different crack spacing length, their bond/slip values are directly affected and differ in both magnitude and evolution shape. Indeed, Block 2's larger slip suggests an inferior bond condition between the two constitutive materials. Seen the presence of such differences, in the bond-slip analysis of double pullout test profiles it is common practice to consider the central crack-defined block as it is frequently of length similar to the average crack spacing. This will henceforth be applied to the following cracking members.

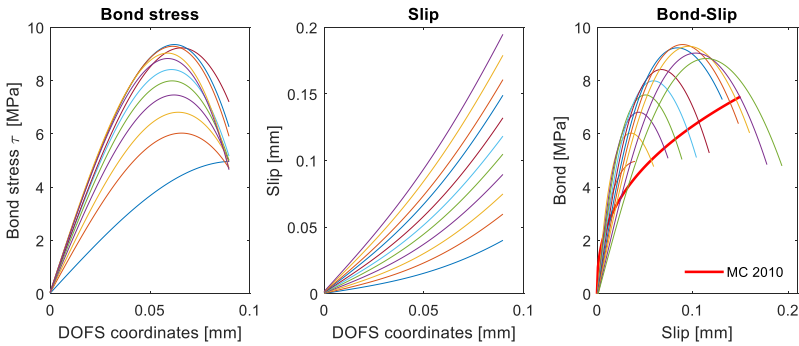
Moving on to member 10x10_D16.02, Figure 3.7 displays its DOFS sampled steel strain profiles and highlights with black rectangles some zones of strain uncertainty probably due to the sudden local under-performance of the DOFS. Once again two cracks separate member 10x10_D16.02 into three parts limited by DOFS coordinates 135.2–257.4 mm (Block 1), 312.0–455.0 mm (Block 2), 500.5–669.5 mm (Block 3).



(a) Blocks definition



(b) Block 1: 0.870-0.930



(c) Block 2: 0.930-1.075

Fig. 3.6. DOFS-extracted strain and bond-slip profiles for member 10x10_D12 for (b) Block 1 and (c) Block 2 as defined in (a)

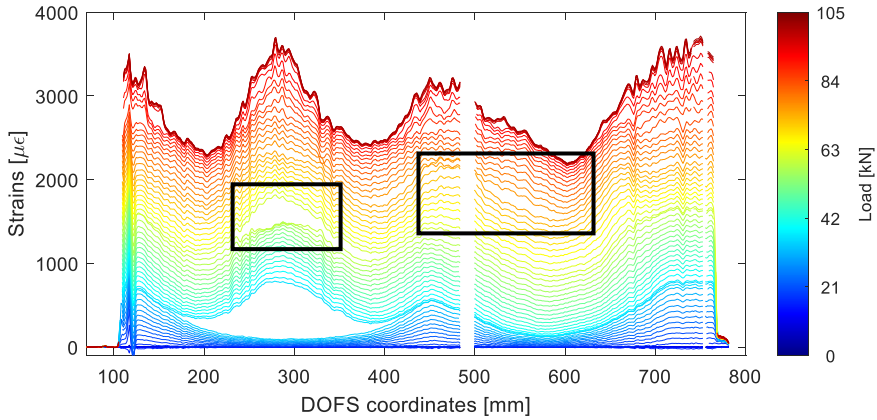


Fig. 3.7. DOFS-extracted strain profiles for member 10x10_D16.02

As previously mentioned, the focus of the bond/slip analysis will be the central block which, as visible in Figure 3.7, is characterized by two rather different halves (statement valid for all the blocks). This displays, once more, the difficulty of studying the bond stress for a cracking RC members with the variability that a crack formation process brings to the table compared to non-cracking members. Figure 3.8 displays the extracted bond/slip curves. The bond stress curves have similar maximal bond stress and average bond stress values which, as seen further on, is the crucial parameter for the definition of an average bond stress value per structural configuration. The presence of the aforementioned DOFS strain reading uncertainties leads to a lack of continuity in Figure 3.8's curves. This withstanding the fact that such a monitoring was possible to be performed well beyond cracking and even beyond the steel yielding strain is in itself a great success and a demonstration of the viability of such kind of monitoring for further testing. Overall, despite the profile differences between the left and right branches of a single block are smaller than the difference between different blocks, it is still cannot be over looked.

In conclusion, the present sub-section has demonstrated the overall complexity of a bond/slip analysis especially when based on cracking RC members. The complexity's core lays in the diversity of the profiles not only among different crack-defined blocks but also between the left and right branches of a single block's strain profile.

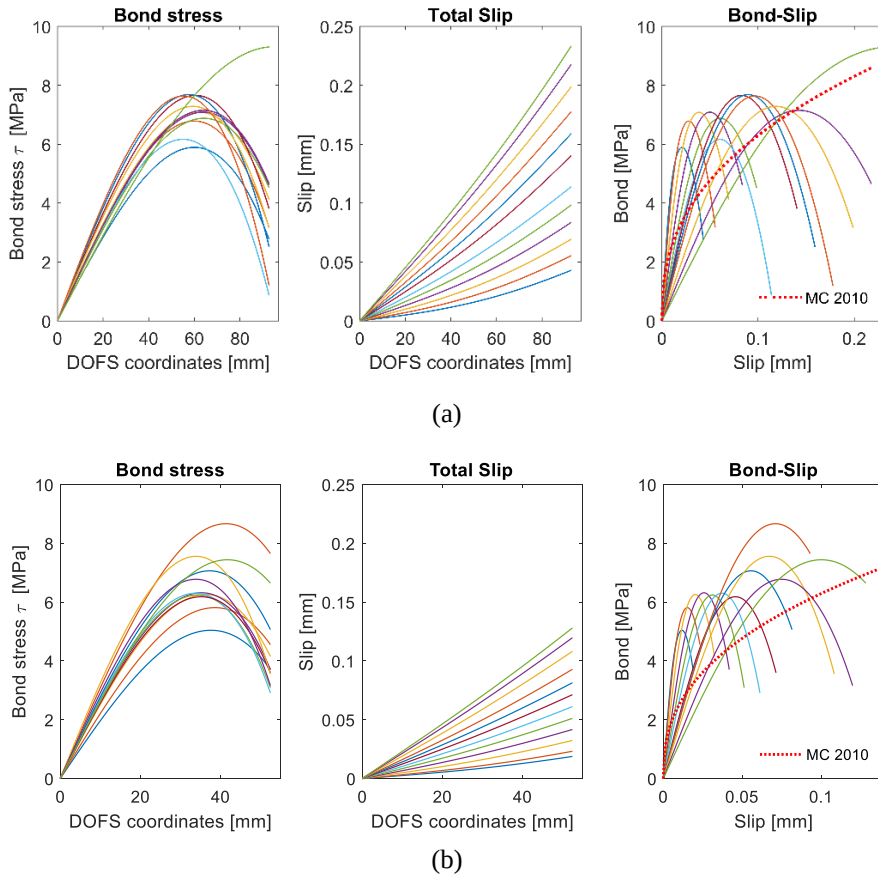


Fig. 3.8. DOFS-extracted bond, slip and bond/slip profiles of the (a) right and (b) left half of the second crack-delimited block of member 10x10_D16.02

3.1.3. Distributed Sensing monitoring for bond stress/slip assessment: non-cracking members

The present sub-chapter follows up displaying, this time, the bond stress/slip of non-cracking RC ties. Figure 3.9 presents the DOFS-profiles of members 10x10_D16.01 and 12x12_D12.01. As visible in Figure 2.8, on member 10x10_D16 different DOFS/rebar bonding techniques were used. Its first segment (145 mm) is the only one including a silicone-based protection layer and ends up being the only one free of excessive HF-SRAs. Consequently, only the latter is represented in Figure 3.9.

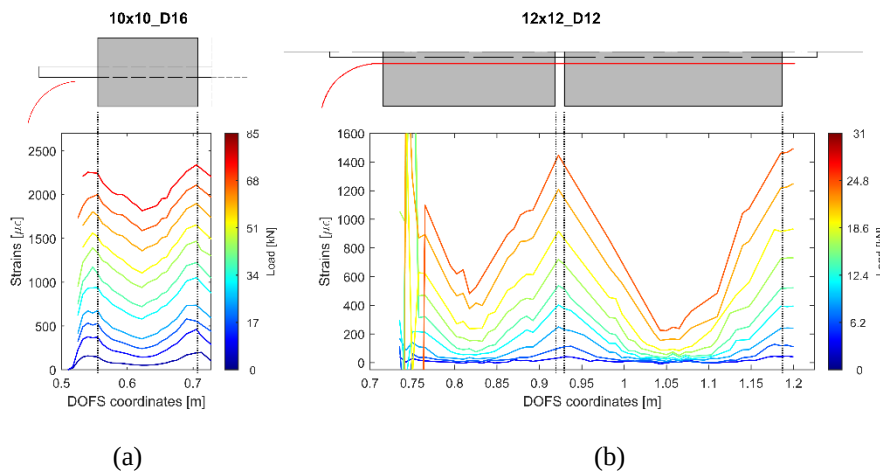


Fig. 3.9. DOFS steel strains at various load stages of the tensile test inside member (a) 10x10_D16.01 and (b) 12x12_D12.01

This result, despite having come at the cost of most of the member's measurements, is crucial for the definition of a universally performant DOFS bonding methodology. As expected, all of Figure 3.9's strain profiles present no strain peaks (indicative of cracks) at any load but do, instead, behave in two seemingly different ways. The first, hereon defined as Behavior A and exemplified by member 10x10_D16 (Figure 3.9a), sees a geometrically uniform evolution of the strain profiles (practically offsetting upwards) proportional to the applied load. Such behavior is also present in the second segment of 8x8_D8 (Figure 3.2) and in the first segment of 12x12_D12 (Figure 3.9b). In all of these cases, the strain profiles are initially characterized by a parabolic shape with a flat segment in its center (analogous to the cracking members' elastic stage). Soon after, when the compatibility areas' flatness is lost, they are characterized by two linearly-approximable profiles (crossing in the mid/span) which simply shift upwards remaining almost unaltered. Only the first segment of 12x12_D12, slightly gains pendency with the increasing load. These results are in line with the experimental work from Kankman (Kankam 1997), Beeby and Scott (Beeby & H. Scott 2005), Scott and Gill's (Scott & Gill 1987) and Kaklauskas et al. (Kaklauskas et al. 2019). Behavior A is reminiscent of the steel rebar's strain profile during the stabilized cracking stage (Figure 1.3c).

Behavior B, instead, is exemplified by the first segment of 8x8_D8 (Figure 3.2) and is characterized by a strain profile whose central zone hardly grows along the duration of the test whilst its edges do, proportionally to the load. Such behavior is also evident in the second segment of 12x12_D12 (Figure 3.9b)

even though not as prominently. Behavior B's profiles are reminiscent of Figure 1.3b's cracking stage ones despite the formers' central zone strains are not sufficiently large for the concrete to actually crack.

The most plausible reason for Behavior A and B's resemblances to the stabilized cracking profiles and cracking stage profiles, respectively, lays in the RC tie lengths. Indeed, member 12x12_D12 second segment's length is critically close to surpassing the member's maximum crack spacing, effectively landing its behavior between a non-cracking and a cracking RC tie. Consequently, its strain profile will be closer to Figure 1.3b's (cracking stage) than Figure 1.3c's (stabilized cracking stage) despite never reaching the point that a crack-induced peak appears in its mid-section. Oppositely, member 12x12_D12's first segment, being 50 mm shorter, completely misses out on this hybrid behavior therefore being characterized by strain profiles closer to Figure 1.3b's. In order to assess the correctness of these novel assumptions, further studies should be dedicated to the influence of the tested RC members' longitudinal dimensions on the steel strain profiles.

Figure 3.10 displays member 12x12_D16' strain profiles along the test together with their polynomial interpolation.

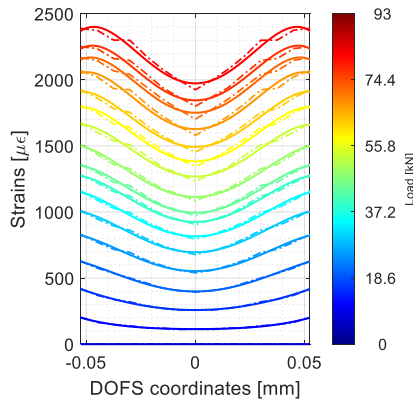


Fig. 3.10. DOFS steel strains at various load stages of the tensile test inside member 12x12_D16

The evolution of such strains can be considered pertaining to the behavioral category B as distinct pendence variations are diagnosable along the entire duration of the test. Figure 3.11 displays member 15x15_D20.01' strain profiles as sampled by the new OBR model ODISI-6000 (Figure 1.6). It should be noticed that hardly any SRAs are visible and the strains are monitored all the way to the yielding load (around 167.5 kN) and beyond.

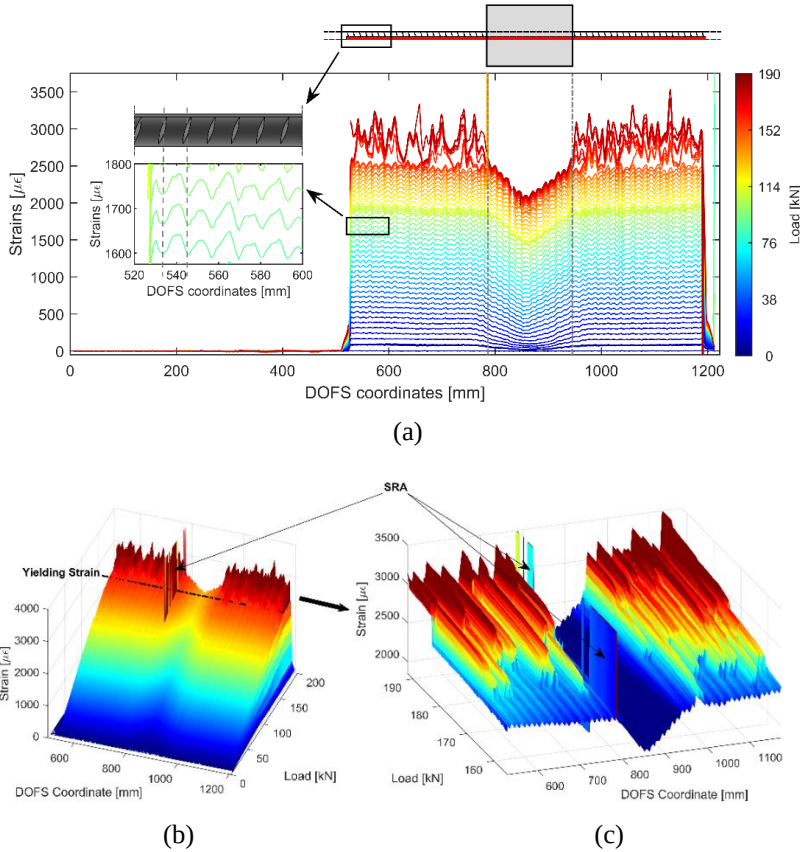


Fig. 3.11. (a) DOFS steel strains at various load stages of the tensile test inside member 15x15_D20 and (b and c) a three dimensional representation of their evolution

The peaks present at the higher loads are not SRAs but rather the physical representation of the rebar's different sections slowly plasticizing one after the other. Figure 3.11b and c provide a three dimensional representation of member 15D20_21's strains displaying a smooth increase up until the yielding strain ϵ_{sy} beyond which it is visible how little by little all the rebar's cross sections progressively plasticize (indicated by a sudden peaking of the strains). This by itself is a significant step forward from many literature's DOFS embedded tests which stop producing reliable results at loading stages numerous folds inferior to the present one. This is, furthermore, very encouraging when it comes to the DOFS/rebar bonding technique performance. Moving back to Figure 3.11a, a

small zoom is provided on its left of the rebar's strains profile outside of the concrete as reported by DOFS. An oscillation of the profile is visible which could easily be confused with signal noise. Yet, it is not noise but rather the influence of the transversal rib on the performance of the rebar. As a matter of fact, whenever a rib is present, the cross-section of the bar is automatically larger therefore the carried stress (and consequent steel strains) is inferior. Oppositely, whenever the rib is absent the carried stress is higher. The reiteration of this pattern leads to the above mentioned oscillating profiles. Nonetheless, in signal processing, noise is a general term for unwanted (and, in general, unknown) disturbance that a useful information signal may suffer during capture, storage, transmission, processing, or conversion. Therefore, unless a researcher is interested specifically in the rib's influence on the bar's performance, the latter can be considered noise as the useful information signal that is usually pursued is strictly the global profile of the strain curve. Concerning the evolution of member 15x15_D20's strain profiles, their behavior is similar to the above mentioned Behavior A.

In the following we will build on the SRA-free strain profiles of all the above non-cracking members to extract their experimental bond stress and slip curves by means of polynomial degree approximations that represent the best compromise of quality and accuracy. Figure 3.12 displays such results and further compares them with the ones calculated following the guidelines of the Model Code 2010. Note that this comparison's quality index is not a point-by-point similarity between the DOFS and MC2010 curves, but rather a similarity of their magnitude, trend and load-dictated variation. Also, some of Figure 3.12's profiles are characterized by smaller final loads compared to the others, namely member 12x12_D12's, due to the disruptive presence of SRAs at larger loads. The use of only low-load profiles is directly reflected in the lower slip values.

With these aspects in mind, starting from the concrete/steel bond stress of each member, different development behaviors are noticeable. The first, present in Figure 3.4, Figure 3.12a, g, j and p, begins with a few increasingly pendent plots (indicative of the equivalent elastic and cracking phases), but soon after becomes characterized by small variations of mostly linear profiles. Whilst this might not be immediately discernible from Figure 3.12's bond representations, attention should be put to the load corresponding to each plot. The reader would then see that the ones corresponding to the larger ones are moderately grouped. Differently so, the second development behavior, evident in Figure 3.12d, m and s, is characterized by a gradual curvilinear evolution all along the test duration. It should be remarked that member 12x12_D12's segment 1 (Figure 3.12g) and 9x9_D12 display a hybrid behavior, starting with the former and subsequently becoming resemblant to the latter.

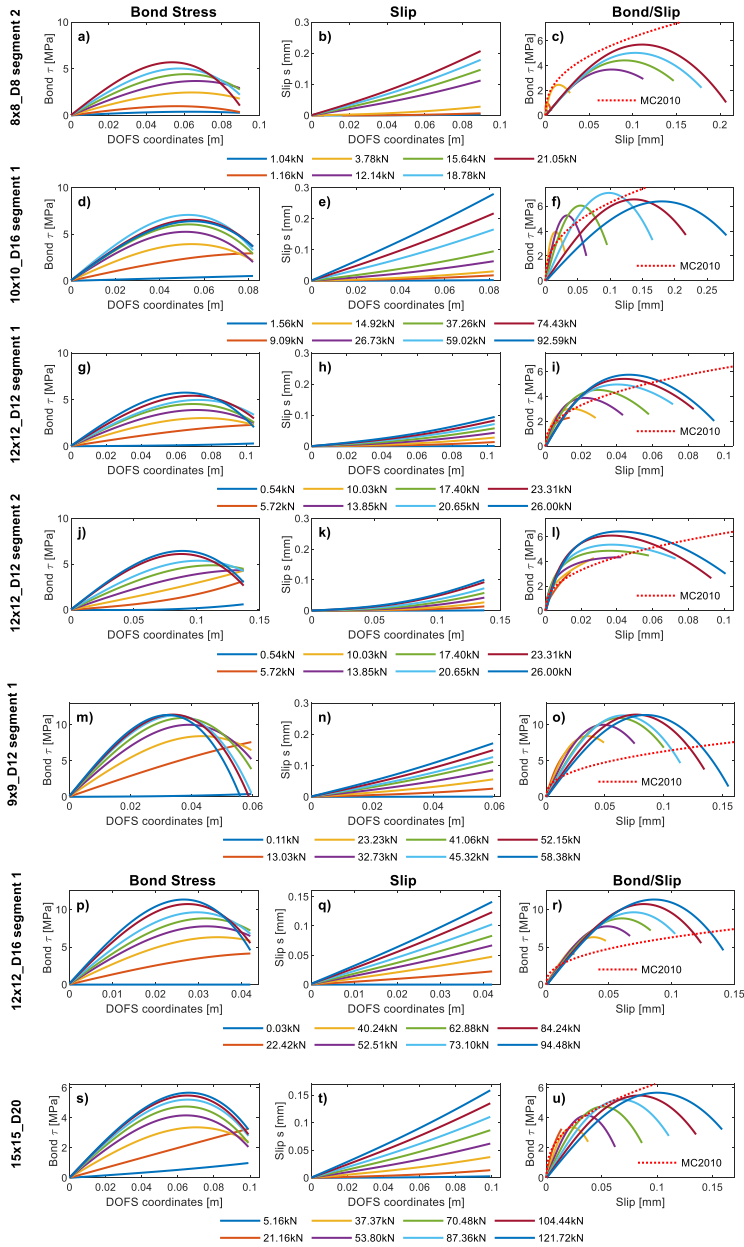


Fig. 3.12. Bond, slip and bond-slip profiles of non-cracking members

Consequently, their bond/slip profiles are simultaneously compact and fanning outwards. This, as discussed above, is potentially a consequence of the concrete block's length and its proximity to the minimum crack spacing. The behavioral difference was actually foreshadowed by the dissimilarity of their relative strain profiles and therefore can be tightly correlated to the above defined Behavior B and A respectively. This difference is also translated in the members' bond-slip laws and is particularly evident when compared against the ones calculated by means of the Model Code 2010. Indeed, Behavior B leads to a compact and almost uniform bundle of ascending branches whilst Behavior A leads to profiles that, despite still enveloping the MC2010 curve, tend to fan outwards thus losing Behavior B's compactness. This was to be expected as Behavior A leads by definition to very similar bond stresses but to different slips, thus resulting in ascending branches which tend to distance each other, fanning out, as the load increase. Behavior B, though, can also be found in literature for other bond/slip modeling techniques such as finite elements analysis (Jakubovskis & Kaklauskas 2019). The hybrid behavior of Figure 3.12i's curves is also noticeable as they aren't as compact as in Figure 3.12l but do not fan out as much as the previous member's.

Concerning the bond/slip laws' proximity to the MC2010's, generally stronger bond stresses are sampled by DOFS (versus the model code's) per identical slip values. This could suggest an under-estimation of the concrete/steel bond from the MC2010's part. Among all the studied members the ones whose bond/slip laws is closer to the code's are 10x10_D16, 12x12_D12 and 15x15_D20.

On the grounds of the above extracted data, the reinforcement ratio and crack spacing (length of the non-cracking member in case of shorter ones) seem to exercise a hefty influence on the bond stress/slip curves.

3.1.4. Single Distributed Sensing bond stress values per load

The bond stress curves displayed in the previous sub-chapter efficiently described the evolution of the bond stresses with load. When performing serviceability analyses, though, it is often convenient to have a single bond stress value per applied load instead of a full curve. In order to turn the latter in the former, the bond stress curves should be averaged, resulting in a single value. Then, if to collect all these single bond stress values across all the loads, an average bond stress/load curve can be extracted.

An example of it is displayed in Figure 3.13 which displays the evolution of the average bond stress values of the left branch of the 15x15_D20 strain profiles, of its right one and of the average of the two.

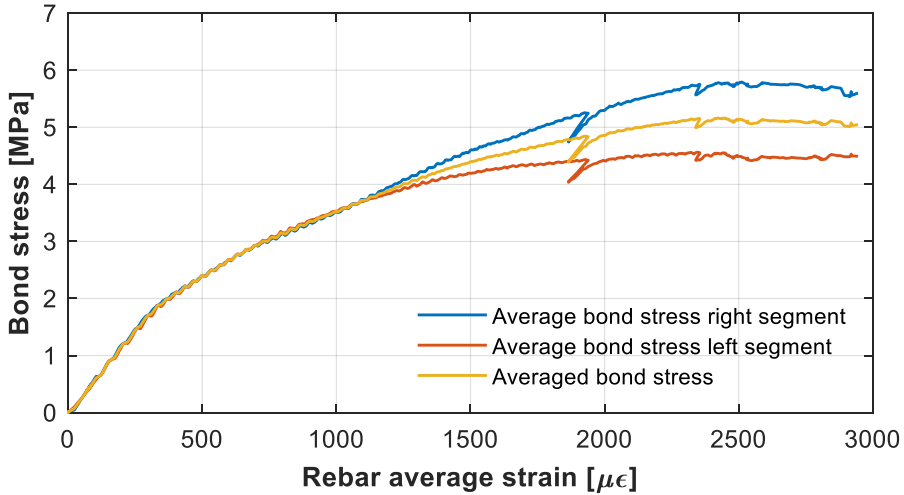


Fig. 3.13. Average bond stresses per different rebar strain levels for member 15x15_D20

It is to be expected that, when the stabilized cracking stage is reached in the RC member, the bond stress values vary very little. This is confirmed by Figure 3.13's graphs which seem to flatten at around 2500 $\mu\epsilon$. The corresponding bond stress value would be 5.16 MPa. This value, representative of the stabilized cracking stage, is crucial for serviceability analyses and will henceforth be referred to as "serviceability bond stress".

Note that, commonly, bond stress values are referred to as a function of the concrete's tensile strength f_{ct} (for member 15x15_D20 corresponding to 2.89 MPa) as visible in Equation 3.9.

$$\tau = \frac{\text{average of the bond stress curve}}{f_{ct}} \quad (3.9)$$

Thus, the corresponding value of the serviceability bond stress of 15x15_D20 is equals to $5.16/2.89 \text{ MPa} = 1.78 \text{ MPa}$. Table 3.2 reports the similarly calculated serviceability bond stresses of the other members together with their corresponding reinforcement ratio. Figure 3.14 summarizes the above data and correlates it with their relative reinforcement ratios ρ . Curiously a decreasing trend can be spotted for increasingly larger reinforcement ratios. The corresponding linear regression is reported in Equation 3.10.

$$\tau_{DOFS} = (-0.72 \rho + 3.04) * f_{ct} \quad (3.10)$$

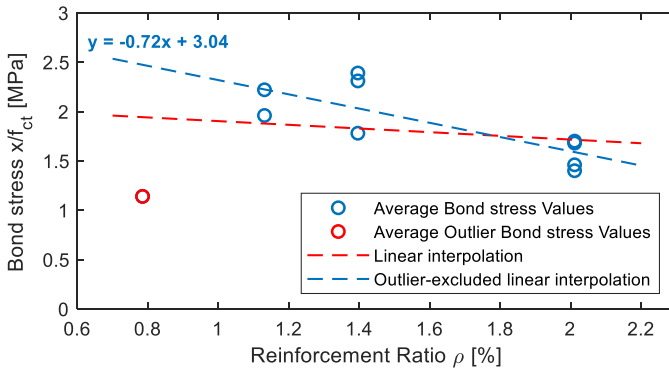
Table 3.2. Bond stress per different load levels as a function of the concrete's tensile strength

Member	Bond/ f_{ct} , MPa	Bond, MPa	Reinforcement ratio
10x10_D12.01	5.3/2.7	$1.96 f_{ct}$	1.131%
10x10_D12.02	6.0/2.7	$2.22 f_{ct}$	1.131%
8x8_D8	3.5/3.07	$1.14 f_{ct}$	0.785%
9x9_D12	7.10/3.07	$2.31 f_{ct}$	1.396%
10x10_D16.01	4.48/3.07	$1.46 f_{ct}$	2.011%
12x12_D16	7.35/3.07	$2.39 f_{ct}$	1.396%
15x15_D20	5.16/2.89	$1.78 f_{ct}$	1.396%
10x10_D16.02 block 1	4.05/2.89	$1.40 f_{ct}$	2.011%
10x10_D16.02 block 2	4.90/2.89	$1.70 f_{ct}$	2.011%
10x10_D16.02 block 3	4.86/2.89	$1.68 f_{ct}$	2.011%
10x10_D16.02 average	4.60/2.89	$1.59 f_{ct}$	2.011%

This equation is effectively the first ever DOFS-based bond stress model and represents one of the main novelties of the present work. Obviously, a much larger DOFS based experimental database would be required for the definition of an effective bond stress model but, nevertheless, the steps illustrated so far constitute a valid *modus-operandi*, demonstrating the potential of DOFS for serviceability modeling.

For the sake of simplification and ease of use, Equation 3.10 can be simplified into Equation 3.11.

$$\tau_{DOFS} = \left(3 - \frac{\rho}{4}\right) * f_{ct} . \quad (3.11)$$

**Fig. 3.14.** Bond stress values as a function of f_{ct} for different reinforcement ratios

3.2. Revisiting the Strain Compliance crack spacing Model enhancing it through Distributed Sensing

Despite the vast number of studies on the cracking of RC structures, the complexity of this phenomenon is ample enough to warrant the furtherance of its investigation. The latter actually represents one of the most prominent applications of DOFS (Berrocal et al. 2021; Berrocal et al. 2020; Bado et al. 2021). On its back, the author explored DOFS' potential to formulate, integrate, validate or dismiss modern RC structures cracking serviceability models. In particular, the present chapter will report a DOFS integration of a recently introduced average crack spacing predication model.

The latter is the *Strain Compliance Method (SCM)*, originally introduced by Prof. Dr Gintaris Kaklauskas (Kaklauskas 2017) in 2017 for bending RC members. Here, the author argued that, despite much is known on the topic of cracking in RC structures, a completely rational and general analysis method is still unavailable. The author further stated that, up until the present day, the most widely used models are the mean strain approach and discrete crack-based approach (also called partial interaction or bond-slip). The first reportedly boasts features of simplicity and numerical effectiveness, whilst the second portrays in a more realistic fashion the inner-workings of a RC structure thanks to the stress transfer method. The SCM, instead, innovatively suggested a combined use of the two in an attempt to merge their benefits, effectively becoming “the best of two worlds”. This union was defined as *compatibility of discrete crack-based and mean strain approaches* or, in brief, *Strain Compliance*. It should be noted that the SCM bears similarities to earlier proposed approaches (Kaklauskas & Gribniak 2011; Kaklauskas, Gintaris Ghaboussi 2001) dedicated to the constitutive modelling of tension stiffening in bending members.

Multiple publications and thesis spun up from prof. Kaklauskas' and his team's work. One of such was the doctoral thesis of Dr. Regimantas Ramanauskas. Here, a reference element (standard rebar diameter size, concrete prism dimensions and mechanical characteristics) was chosen on the grounds of which the SCM analysis was developed with an iterative approach. Such approach was used to establish the mean strain of alternative member configurations, namely members with different reinforcement ratios ρ and rebar diameter $\varnothing_s$. A further iterative solution (for the maximum reinforcement strain) was also employed to ensure the mean strain compatibility.

The present chapter builds on the originally proposed version of the SCM providing a new and more performant approach. It should be kept in mind that a complete interconnection between the SCM and experimental DOFS results is only possible if the amount of experimental data is large enough. As mentioned earlier, at the present moment in time, DOFS is a rather young monitoring tool

therefore a sufficiently large database is still to be created. Consequently, the present work displays only how DOFS can integrate the SCM in such a way as to improve its predictions and not as a final definition of the latter.

The following chapters will introduce the key concepts of this new version of the SCM, starting from its base assumptions all the way to the detailing its constitutive parameters. The model will not only be presented in its integral form but also in a simplified but similarly performant version. Its parameters will further be integrated with the data provided by DOFS testing in order to provide SCM with experimental grounding. Finally, the prediction capacity of the SCM will be put to the test and will be compared against the ones of the Model Code 2010 and Eurocode 2.

3.2.1. Strain Compliance Model bases: methodology and assumptions

The SCM lays on certain assumptions which automatically outline the development methodology. Such assumptions are as follows:

- It is designed to perform crack analysis for loads corresponding to the Model Code 2010's Stabilized Cracking Stage.
- In Stabilized Cracking Stage, the crack pattern of the RC tie is fully developed and includes both primary and secondary cracks. The former kind fully extends from the rebar to the concrete prism's surface whilst the latter can be located in correspondence to the rebars' ribs even in the middle of adjacent primary cracks but has a limited extension. Whilst the secondary cracks can continue increasing in both size and width, no supplementary primary crack appears at this stage. For the SCM, only the primary crack pattern is considered whilst the secondary ones are neglected.
- The range of the equivalent service loads is relatively broad and will be an important parameter in the following work.
- The tension softening effect (representing the ability of concrete to transfer stresses in the cracks) is omitted.
- A RC tie's segment of length equals to the spacing between two primary cracks (henceforth defined as block) is considered representative as the average behavior of the entire RC member. Thus, assuming satisfactory the mean strain approach's performance in providing an accurate average deformation behavior of a full RC element, it can be assumed that also the mean behavior of a short RC block is an accurate representation of the mean behavior of the entire RC element. In conclusion, the deformations of a single RC block having the length of mean crack

spacing is assumed to represent the averaged behavior of the cracked member.

- The development (not the use) of the SCM requires the knowledge of the rebar's reinforcement strain profile when embedded in concrete and in particular in the study case RC block.
- The steel rebars' strain profiles are linearly approximable to a specific shape.

As explained in the introduction, over the years researchers have attempted to study the strain distribution of steel rebars embedded inside RC structure by means of multiple monitoring technologies such as strain gauges, Fiber Bragg Gratings and Distributed Sensing. On the possibility of linearly approximating the steel strain profile delimited by two primary cracks, the above mentioned research presents opposite results. Whilst on one side a linear approximation seems possible (Scott & Gill 1987; Beeby & H. Scott 2005), on the other the best approximation is suggested to be a second order polynomial one (with the introduction of local effects near the normal cracks). The solution to such quandary may be a hybrid behavior. Indeed, in Kaklauskas et al. (2019), the authors suggest that at lower loads the strain profile resembles a parabola while, as the load increase, it progresses in a linearly approximable outline.

Thus, keeping in mind the SCM performance range (high load levels corresponding to the stabilized cracking stage), the strain profile should be approximated linearly. Henceforth the reinforcement strains in the study concrete block are assumed to follow a simplified linear expression over the distance of the element. Figure 3.15 and Figure 3.16 experimentally measured the steel strain profiles at different loads for both older experimental campaigns and the more recent ones performed with the help of DOFS.

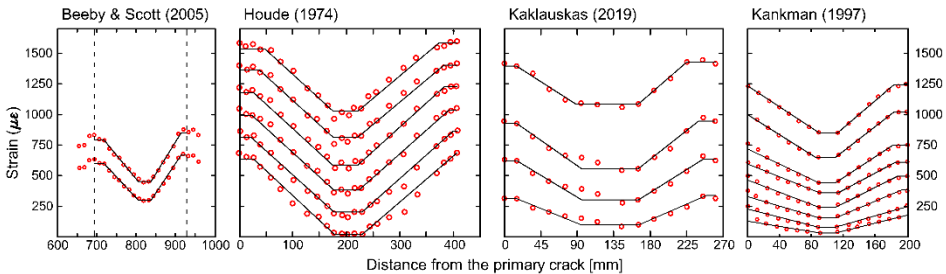


Fig. 3.15. Experimentally extracted strain profiles of rebars embedded in RC ties from four different campaigns

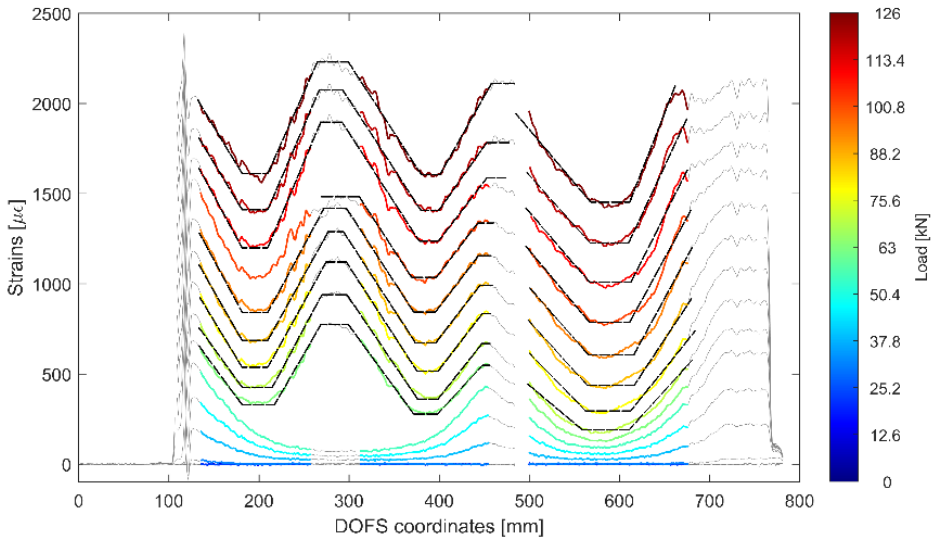


Fig. 3.16. DOFS extracted strain profile for a Ø16 rebar embedded in member 10x10_D16.02

The best linear approximation of the profiles is composed by two flat lines on the edges, two oblique lines and a central flat line in the middle. This is the case not only for tests with low spatial resolution (which may leave space to some doubts due to the lack of monitoring point) but also with DOFS with a spatial resolution of 1.3 mm. By itself this last point already represents a validation of the SCM form DOFS' part.

Each of the above mentioned segments delimits specific zones characterized by different bond conditions between concrete and steel. The higher flat lines (on the edges of the v-shaped profile) are indicative of the crack location and of the neighboring areas where the concrete/steel bond was deteriorated by the development of slip. Here the strain on the steel is practically equivalent to the one of the external bare rebar. Oppositely, the central zone is indicative of the area where the concrete/steel bond is the strongest. Finally, the oblique lines represent the connection between these last two zones, effectively showing how the concrete/steel bond progressively improves with the distance from the crack.

3.2.2. Strain Compliance Model: integral version

On the grounds of the linear approximations performed in the previous sub-chapter, Figure 3.17 depicts, in a simplified manner, the resulting geometry of the steel strain profile between two primary cracks.

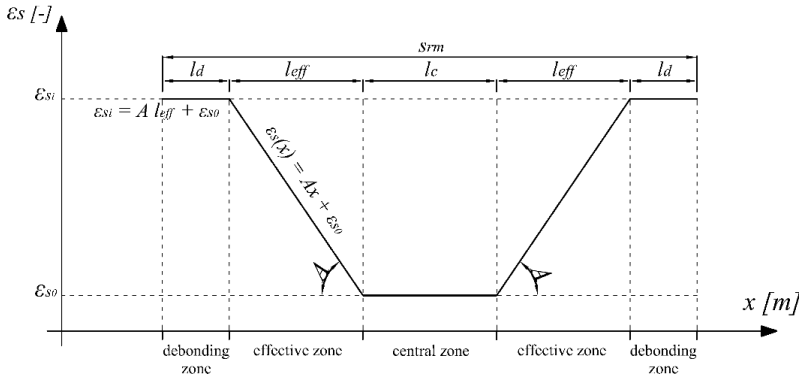


Fig. 3.17. The first approach to the linear approximation of a rebar's strain profile

Similarly to what was discussed previously, in Figure 3.17 the following zones can be identified:

- *Debonding zone* whose length is defined as debonding zone length (l_d).
- *Effective zone* whose length is defined as effective zone length (l_{eff}).
- *Central zone* length whose length is defined as central zone length (l_c).

Consequently, the sum of these zones' lengths corresponds to the average crack spacing s_{rm} as in Equation 3.12:

$$s_{rm} = 2l_d + 2l_{eff} + l_c. \quad (3.12)$$

From Figure 3.17 it can be directly deduced that the length of the effective zone is equivalent to Equation 3.13:

$$l_{eff} = \frac{\varepsilon_{si} - \varepsilon_{s0}}{A}. \quad (3.13)$$

Being A the strain slope coefficient and ε_{si} the strain of a bare rebar and/or of a rebar embedded in a RC element in a transversally cracked section and ε_{s0} the lowest strain on the rebar located in the segment where the most tension stiffening occurs (the central area). The slope coefficient A can be directly connected to the concrete/steel bond/stress as in Equation 3.14. Being τ the concrete/steel bond stress, E_s the modulus of elasticity of steel and $\varnothing_s$ the steel rebar's diameter. Instead, ε_{si} can be calculated as in Equation 3.15. Being P the study case load and A_s the cross-sectional area of steel.

$$A = \frac{4\tau}{E_S \theta_s}; \quad (3.14)$$

$$\varepsilon_{si} = \frac{P}{E_S A_s}. \quad (3.15)$$

Note that ε_{si} and ε_{s0} and the bond stress τ present on the concrete/steel contact surface are unknown at a design stage of a RC structure, thus the need to define a crack spacing model based on strictly geometrical and mechanical characteristics of the composite materials. The SCM is introduced on purpose to tackle this lacking. As previously mentioned, SCM acknowledges a synergy between the average strain value provided by the stress transfer approach (also referred to as partial interaction or discrete crack based approaches) and the mean reinforcement strains of the embedded bars (also known as the smeared crack or mean strain-based approaches). Not that the latter can be estimated by any mean strain approach whether analytical, numerical, experimental, etc. Thus, this model's governing parameters are obtained by equating the mean strains of the tensile reinforcement as defined by these two concepts. In the present work the mean reinforcement strain, that is to be compared to with the strain-profile-extracted average, will be calculated by means of the Eurocode 2 model code. Through this equality, the distance between cracks of any RC tension element can be calculated as function of strictly geometrical and mechanical features of the members. In order to further explain this concept, a graphical explanation will be provided in the following. Given the simplified strain profile indicated in Figure 4.3, if to consider the surface delimited by each zone, three distinct surfaces can be identified, S1, S2 and S3 corresponding to the above mentioned zones as in Figure 3.18.

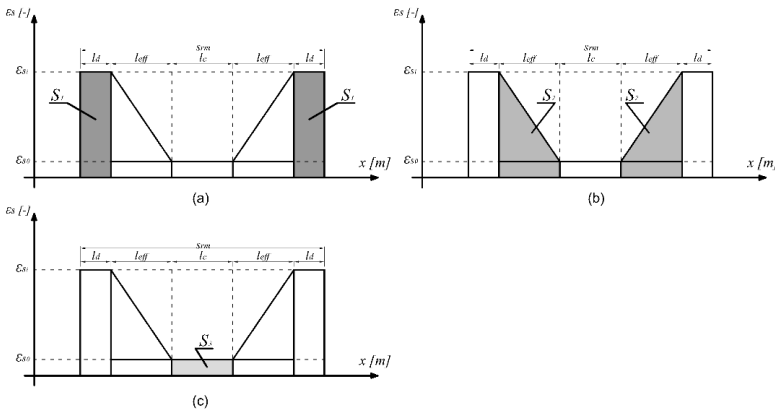


Fig. 3.18. Geometrical definition of areas S_1 , S_2 and S_3

The SCM approach dictates that 1) the mean strains of the smeared crack method and 2) the stress transfer approach are effectively equivalent. This suggests that the mean strain over a specific crack spacing $\varepsilon_{sm} s_{rm}$ is equivalent to the strain amount defined by the three above defined zones as in Equation 3.16 being S_1 , S_2 and S_3 defined in Equations 3.17 to 3.19.

$$2S_1 + 2S_2 + S_3 = \varepsilon_{sm} s_{rm}; \quad (3.16)$$

$$S_1 = \varepsilon_{si} l_d; \quad (3.17)$$

$$S_2 = \frac{(\varepsilon_{si} - \varepsilon_{s0}) l_{eff}}{2} + \varepsilon_{s0} l_{eff} = \varepsilon_{si} l_{eff} - \frac{\Delta l_{eff}^2}{2}; \quad (3.18)$$

$$S_3 = \varepsilon_{s0} l_c. \quad (3.19)$$

This concept is further illustrated in Figure 3.19. As visible, the geometrical shapes in Figure 3.19a and Figure 3.19b are composed of the same geometrical sub-shapes just rearranged differently.

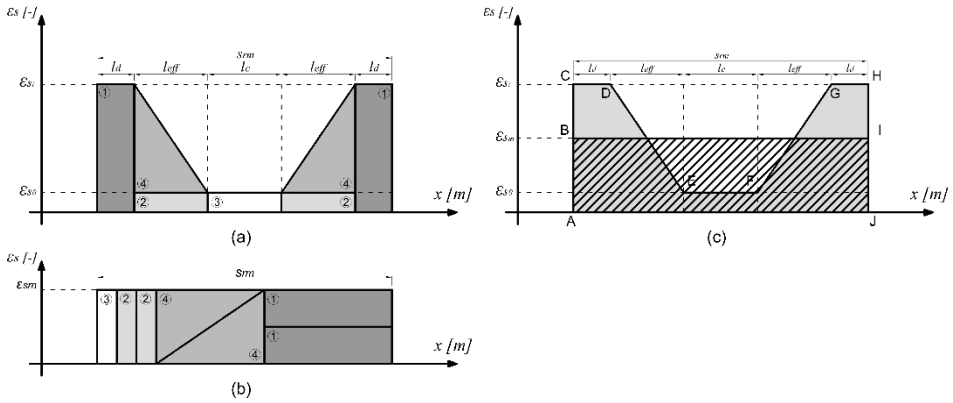


Fig. 3.19. Geometrical demonstration of the equality between (a) the strain area delimited by the stress transfer and (b) mean strain approach as in (c)

This suggests their area equality in a similar manner to the total strain equality as calculated by the stress transfer and mean strains of the smeared crack approach. By substituting the values of S_1 , S_2 and S_3 into Equation 3.16 we get the strain compliance equation (Equation 3.20) and later, by defining s_{rm} , the final Equation 3.21:

$$2(\varepsilon_{si}l_d) + 2\left(\varepsilon_{si}l_{eff} - \frac{Al_{eff}^2}{2}\right) + (\varepsilon_{s0}l_c) = \varepsilon_{sm} s_{rm}; \quad (3.20)$$

$$\frac{2\varepsilon_{si}l_d + 2\left(\varepsilon_{si}l_{eff} - \frac{Al_{eff}^2}{2}\right) + \varepsilon_{s0}l_c}{2l_d + 2l_{eff} + l_c} = \varepsilon_{sm}. \quad (3.21)$$

In order to calculate the mean strain value ε_{sm} , Eurocode 2 interpolates the strain values of uncracked and fully cracked segments of the reinforced concrete tensile member in question thus extrapolating $\varepsilon_{sm,EC2}$. This is done through Equation 3.22:

$$\varepsilon_{sm} = \varepsilon_{sm,EC2} = (1 - \xi)\varepsilon_{el} + \xi\varepsilon_{si}, \quad (3.22)$$

where ξ is the interpolation coefficient, ε_{el} is the elastic strain of the uncracked section and ε_{si} the reinforcement strain in the cracked section (thus in the bare rebar). The interpolation coefficient ξ can be extracted as in the following Equation 3.23:

$$\xi = 1 - \left(\frac{P_{cr}}{P}\right)^2, \quad (3.23)$$

where P_{cr} represents the cracking load of the tensile element and P the study case load. P_{cr} can be extracted from Equation 3.24 as in Equation 3.25:

$$\sigma_{cracking} = f_{ct} = \frac{P_{cr}}{A_c + nA_s}; \quad (3.24)$$

$$P_{cr} = A_c f_{ctm}(1 + n\rho). \quad (3.25)$$

Where A_c stands for the cross-sectional area of concrete, and f_{ctm} for the concrete tensile strength n modular steel/concrete ratio and ρ the reinforcement ratio over concrete area as indicated respectively in Equation 3.26 and 3.27:

$$n = \frac{E_s}{E_c}; \quad (3.26)$$

$$\rho = \frac{A_s}{A_c}. \quad (3.27)$$

Where E_C stands for the concrete's modulus of elasticity. Back to Equation 3.22, the elastic strain of the uncracked section ε_{el} can be calculated according to Equation 3.28:

$$\varepsilon_{el} = \frac{P}{A_C E_C + A_S E_S}. \quad (3.28)$$

3.2.2.1. Zone lengths definition

Some authors (Kaklauskas 2017) suggested the length of the debonding zone to be as in Equation 3.29 below and, seen the correctness of their hypothesis and results, a similar equation is adopted here:

$$l_d = \frac{\varnothing_s}{2}. \quad (3.29)$$

Moving on to the definition of the parameters for the calculation of the length of the effective zone l_{eff} . If to redefine Equation 3.13, Equation 3.30 can be extracted:

$$\varepsilon_{s0} = \varepsilon_{si} - A l_{eff}. \quad (3.30)$$

Now, substituting set equation in the main definition of the SCM (Equation 3.21) and extrapolating the value of l_{eff} as in Equation 3.31 to 3.33, the outcome would look like Equation 3.34:

$$\frac{2\varepsilon_{si}l_d + 2(\varepsilon_{si}l_{eff} - 0.5Al_{eff}^2) + (\varepsilon_{si} - Al_{eff})l_c}{2l_d + 2l_{eff} + l_c} = \varepsilon_{sm}; \quad (3.31)$$

$$\begin{aligned} 2\varepsilon_{si}l_d + 2\varepsilon_{si}l_{eff} - Al_{eff}^2 + \varepsilon_{si}l_c - Al_c l_{eff} \\ = 2l_d \varepsilon_{sm} + 2l_{eff} \varepsilon_{sm} + l_c \varepsilon_{sm}; \end{aligned} \quad (3.32)$$

$$l_{eff}^2 (A) + l_{eff} (A l_c + 2 \varepsilon_{sm} - 2 \varepsilon_{si}) + (\varepsilon_{sm} - \varepsilon_{si})(l_c + 2l_d) = 0; \quad (3.33)$$

$$l_{eff} = \frac{-B + \sqrt{B^2 - 4AC}}{2A}; \quad (3.34)$$

$$\text{with } B = A l_c + 2(\varepsilon_{sm} - \varepsilon_{si}) \quad \text{and} \quad C = (\varepsilon_{sm} - \varepsilon_{si})(l_c + 2l_d).$$

As visible the effective length is a function of the debonding zone length l_d and the central zone length l_c which still remains to be defined. Moving on to the definition of the parameters for the calculation of the length of central zone l_c , the latter is calculated on the grounds of experimental observation. In particular, with a database of average crack spacing measurements on RC ties at hand, it is possible to extract a value of l_c for each member in the following way. On the ground of the SCM and therefore Equation 3.20, substituting the theoretical average crack spacing s_{rm} with an experimental one $s_{rm,exp}$ (extracted from the tensile member database) in the strain compliance equation can help redefining l_c as in Equation 3.35.

$$2\varepsilon_{si}l_d + 2\left(\varepsilon_{si}l_{eff} - \frac{Al_{eff}^2}{2}\right) + \varepsilon_{s0}l_c = \varepsilon_{sm} s_{rm,exp}. \quad (3.35)$$

If to redefine Equation 3.12 with the purpose of extracting the value of l_{eff} the result would be as follows in Equation 3.36:

$$l_{eff} = \frac{s_{rm}}{2} - l_d - \frac{l_c}{2}. \quad (3.36)$$

If to redefine Equation 3.13 with the purpose of extracting the value of ε_{s0} as a function of a $s_{rm,exp}$ (as defined in Equation 3.36) the result would be as in Equation 3.37:

$$\varepsilon_{s0} = \varepsilon_{si} - Al_{eff} = \varepsilon_{si} - A\left(\frac{s_{rm,exp}}{2} - l_d - \frac{l_c}{2}\right). \quad (3.37)$$

Finally, if to integrate these equations into Equation 3.35 would turn into Equation 3.38:

$$2\varepsilon_{si}l_d + 2\left[\varepsilon_{si}\left(\frac{s_{rm,exp}}{2} - l_d - \frac{l_c}{2}\right) - \frac{A}{2}\left(\frac{s_{rm,exp}}{2} - l_d - \frac{l_c}{2}\right)^2\right] + \left[\varepsilon_{si} - A\left(\frac{s_{rm,exp}}{2} - l_d - \frac{l_c}{2}\right)\right]l_c = \varepsilon_{sm} s_{rm,exp}. \quad (3.38)$$

Solving Equation 3.38 for l_c would give us Equation 3.39:

$$l_c = \sqrt{\frac{s_{rm,exp}(\varepsilon_{sm} - \varepsilon_{si}) + A l_d^2 + \frac{A s_{rm,exp}^2}{4} - A s_{rm,exp} l_d}{A}} \quad (3.39)$$

With l_c defined for each member of the database a simple linear regression of all the calculated l_c values plotted against key parameters (rebar diameter or reinforcement ratio for example) provides a single value of l_c as a function of set parameter/s.

With all the zone lengths defined, it is then possible to calculate the theoretical crack spacing according to Equation 3.12.

3.2.2.2. Construction of the central zone length l_c equation

As seen, the first step for the construction of the current version of the SCM was the formulation of the steel strain profiles' zone lengths, namely, the debonding, effective and the central one. The flow chart in Figure 3.20 summarizes the relation between the model's various parameters. What is pretty clear is the high level of inter-dependence between the various parameters. This is particularly evident for l_{eff} which is grounded on the knowledge of both l_d and l_c . As seen in the previous section (Equation 3.39), the formulation of l_c should be grounded on a large experimental database of RC-ties' average crack spacings $s_{rm,exp}$. This is scope of the present sub-chapter.

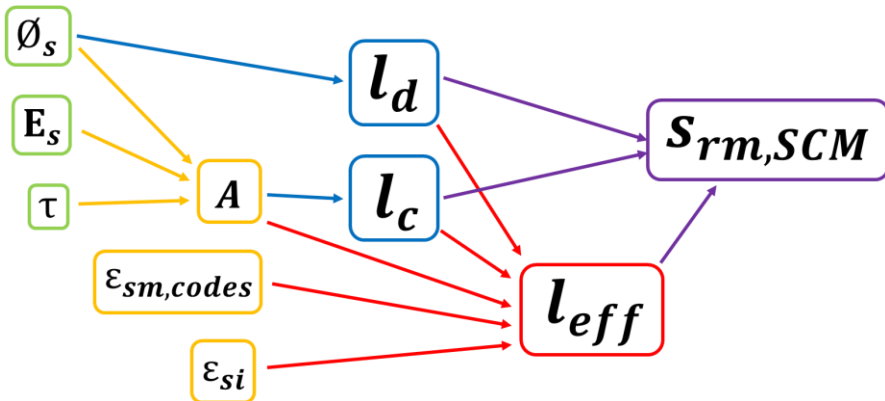


Fig. 3.20. SCM formulation flow chart

The database employed for the construction of the l_c equation is displayed in Annex A. The latter includes RC ties of cross section 100x100 with different rebar

diameters $\varnothing_s$. A graphical representation of its data, as a function of $\varnothing_s$, is reported in Figure 3.21.

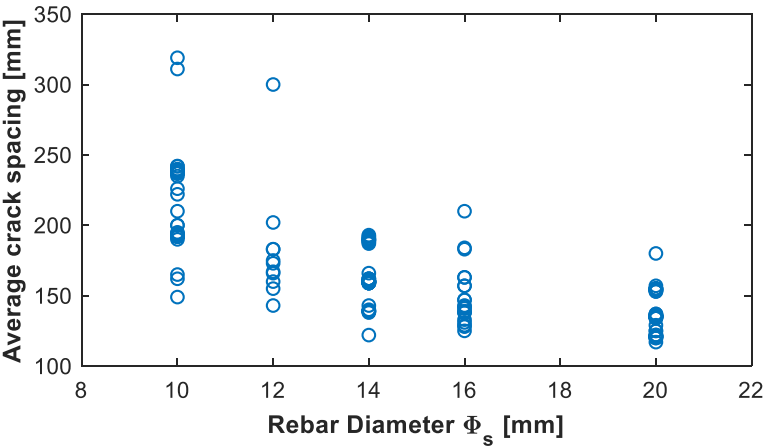


Fig. 3.21. Plot of the average crack spacing s_{rm} per reinforcement diameter

The mean values and variance of Figure 3.21’s data is displayed in Table 3.3, grouped per rebar diameter size, and plotted in Figure 3.22 in the form of normal distribution graph.

Table 3.3. Average crack spacing and variance per rebar diameter

Percentile	Rebar diameter $\varnothing_s$, mm	Average, mm	Variance
[0 100]	10	216.2	1.055
	12	182.5	1.767
	14	162.4	0.324
	16	149.9	0.479
	20	136.6	0.205

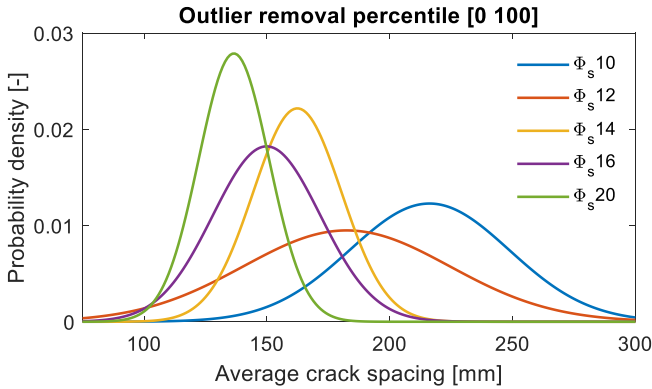


Fig. 3.22. Normal distribution of the average crack spacing per different rebar diameter size

As visible some diameters seem to have a larger data scatter than others ($\Phi 12 > \Phi 20$ for example) due to the presence of outlier values at their extremities. The outlier values strongly affect the average and standard deviation of the data set. This is not a self-contained issue as it represents a limit for the whole model development, thus the need to proceed to their removal. The outlier detection and removal method cannot be developed on the whole database as the scatter percentile would be calculated compared to the total average which is not necessarily indicative of the correct value. Instead, the process should be developed separately per every diameter size for different percentiles. The present database will therefore be submitted to a process of data cleaning of all the experimental readings outside different percentile intervals. The unfiltered database has a percentile interval of [0 100] and the filtering process will be developed for intervals [10 90], [15 85], [20 80] and [30 70]. Figure 3.23 below displays the difference between the original database and the filtered out ones when using such percentiles together with the normal distribution analysis. What can be extracted from the previous table is that, despite the large improvement in terms of average and variance that the database goes through by means of a simple process of outlier removal, the use of the experimental data in the [30 70] percentile is too restrictive as too few data is included inside it. Despite interval [20 80] provides clear indication of the superiority of the clear indication of the superiority of the $\Phi 14$ data pool, the difference with interval [10 90] is not large enough to justify the loss of experimental data. Finally, for all intervals it is possible to evince how the increase in rebar diameter size lead to progressively smaller crack spacings. Indeed, the spacing between the average value of the curves is rather even.

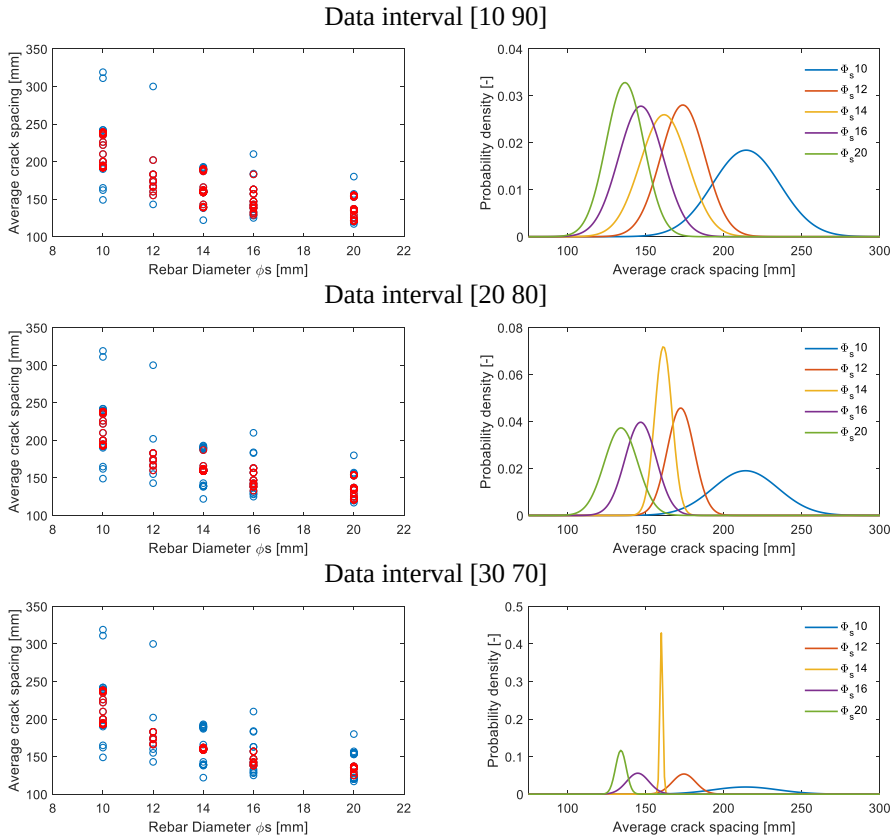


Fig. 3.23. Scatter graphs and normal distribution graphs of the database filtered out with different outlier removal percentiles

It should be kept in mind that the correct choice of input data for the calculation of l_c is a very delicate step. Indeed, referring back to Equation 3.28, the elements under the square root could result negative leading to complex numbers which would alter the calculation of l_c . Different factors could be the cause for such outcome. These are:

- Length of the debonding zone l_d .
- Bond stress factor τ .
- Reference load P .
- Percentile of the selected database for the model creation.

- The difference between the real-life average mean strain present in each tested RC tie and the model codes' predictions (effectively the shortcomings of the model codes could influence the performance of the SCM).

Whilst the former four can be corrected with a numerical variation of their constitutive parameters, the latter requires a different approach. For example, if to use the Eurocode 2 to extract ε_{sm} , knowing that its crack spacing predictions are consistently higher than the actual experimental ones, the ones that would lead to complex numbers would be strictly the smaller ones. In terms of percentile, if the database's [0 100] percentile leads to complex numbers, [10 100] percentile could already solve the issue. Similarly, a [10 90] percentile not only would provide a solution to the complex numbers issue but would also remove the top outlier values thus becoming the preferable choice. With an outliers-free database it is possible to proceed to the calculation of different values of central zone length l_c for each member through Equation 3.28. The selected parameters are the result of an iterative research process aimed at unearthing the most performant ones.

These are as follow:

- Length of the debonding zone $l_d = \frac{\emptyset_s}{2}$.
- Bond stress factor $\tau = 2.72 f_{ct}$.
- Reference load $P = 2.65 P_{cr}$.
- Percentile [10 90].

These values allow to use the smallest of the fractile [10 90] and extract all non-complex l_c numbers. The outcome is summarized in Figure 3.24 where the l_c scatter is represented together with its linear regression. The resulting function of the linear regression provides a formula that defines l_c as dependent strictly on the rebar diameter $\emptyset_s$ (Equation 3.40).

$$l_c = 46.55 - 0.91 \emptyset_s. \quad (3.40)$$

With l_c defined, it possible to proceed to the formulation of both l_{eff} and, finally, the SCM average crack spacing predictions $s_{rm,SCM}$.

3.2.3. Strain Compliance Model: simplified version

The simplified version of the SCM is a slight variation of the above one which, in return, consistently eases the average crack spacing calculation $s_{rm,SCM}$. The most crucial difference lays in the removal of the debonding zone and, in return, the expansion of the central area of an amount equivalent to debonding zone (Equation 3.41).

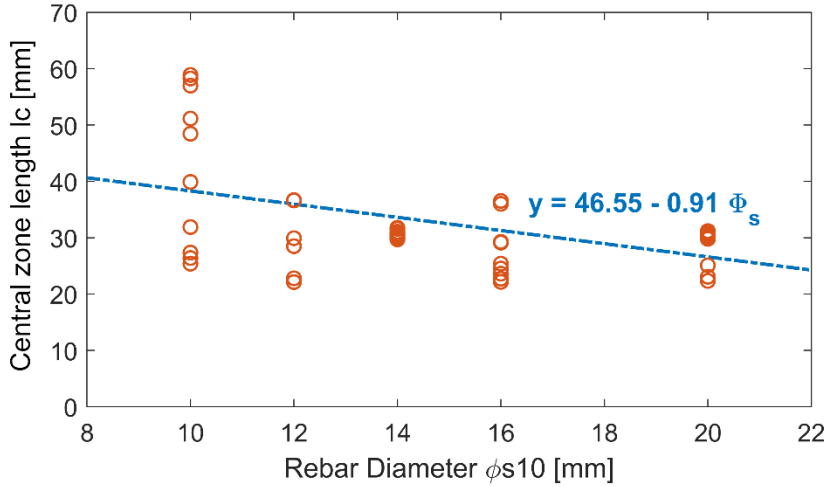


Fig. 3.24. Scatter of the l_c predictions and its linear regression

$$s_{rm,SCM} = 2l_{eff} + l_{c_simp}. \quad (3.41)$$

Here l_{eff} is expressed in a very similar manner to the previous SCM except for the condition of $l_d = 0$ as in Equation 3.42 below:

$$l_{eff} = \frac{-B + \sqrt{B^2 - 4AC}}{2A} \quad (3.42)$$

$$\text{with } B = A l_c + 2(\varepsilon_{sm} - \varepsilon_{si}) \quad \text{and} \quad C = (\varepsilon_{sm} - \varepsilon_{si}) l_c.$$

Also similarly, l_c is calculated as in Equation 3.43:

$$l_{c_simp} = l_c + l_d = l_c + \frac{\phi_s}{2}. \quad (3.43)$$

The performance of such simplified version of the SCM will be studied in the following chapter.

3.2.4. Contribution of the Distributed Sensing to the Strain Compliance Model

The contribution that DOFS can bring to the present formulation of the SCM is significant when considering that some of the parameters that it would help assessing (which are still at the present day under investigation) directly influence

its performance. In particular, the introduction in the model of DOFS-based parameters is aimed at yielding crack spacing predictions that are as close as possible to the experimental ones. The present sub-chapter will delve on the performance of this SCM-DOFS integration.

Starting from the above mentioned key parameters, the debonding length l_d represented in Equation 3.18, will find continuation in the present DOFS-SCM model too. The steel-concrete bond stress, instead, which was previously considered $\tau = 2 f_{ct}$ (as suggested by Bischoff & MacLaggan 2006) or $\tau = 1.8 f_{ct}$ (as in the Model Code 2010) finds a new DOFS-based definition in Equation 3.11. Finally, referring back to the SCM performance range (high load levels corresponding to the stabilized cracking stage), the characteristic load at which these analyses were usually run considered $P = 3 P_{cr}$ but, through an iterative procedure, it was evinced by the author that, for the present DOFS-SCM model, the best formulation is $P = 2.3 P_{cr}$.

Concerning τ , Figure 3.25 displays how the average bond stress values per the most common reinforcement ratios is extracted from the DOFS-extracted bond stress law (Equation 3.10).

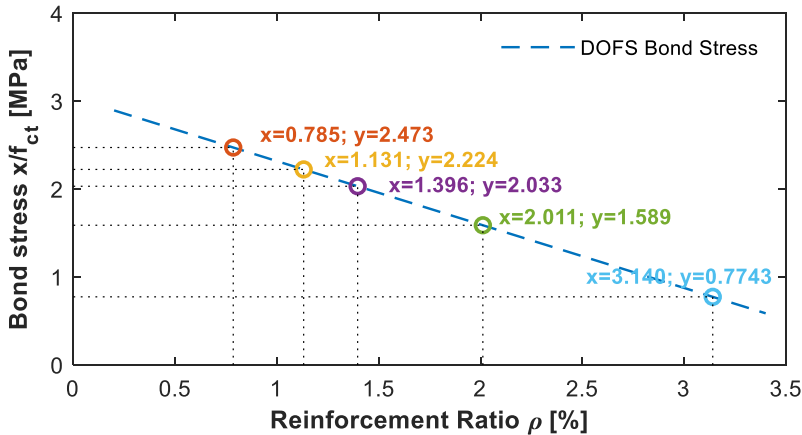


Fig. 3.25. Values of the average bond stresses per different reinforcement ratios extracted from the DOFS-defined bond law

Now, if to insert these parameters in the previous sub-chapter's SCM equations, it is possible to extract an SCM-based average crack spacing value $s_{rm_{SCM}}$ per all the reinforcement ratios. Table 3.4 displays these values for both the simplified and integral version of the SCM.

Table 3.4. SCM-based average crack spacing value $s_{rm_{SCM}}$ per specific reinforcement ratios

Reinforcement ratio ρ , %	Bond Stress τ , MPa	$s_{rm_scm_int}$, mm	$s_{rm_scm_si}$, mm
0.785	2.473	208.72	210.22
1.131	2.224	182.22	184.37
1.396	2.033	154.88	157.13
2.011	1.589	151.52	154.69
3.140	0.7743	134.65	139.45

In order to check the performance of the SCM, thus the accuracy of its average crack spacing predictions, Table 3.4's values will be compared to experimental ones (extracted from RC ties) collected in a large database detailed in Annex B. For each of these elements, a newly introduced parameter called Correlation Ratio (CR), detailed in Equation 3.44, is calculated.

$$Correlation\ Ratio\ CR = \frac{s_{rm_{SCM}}}{s_{rm_{exp}}}. \quad (3.44)$$

This could range from 0 or 2, both values corresponding to an extremely poor correlation, whilst 1 represents a perfect one. Furthermore, in order to compare the novel SCM's performance against the standard codes', the CR values will also be calculated for the MC2010 and EC2. Figure 3.26 is a scatter graph plotting all the above mentioned CRs.

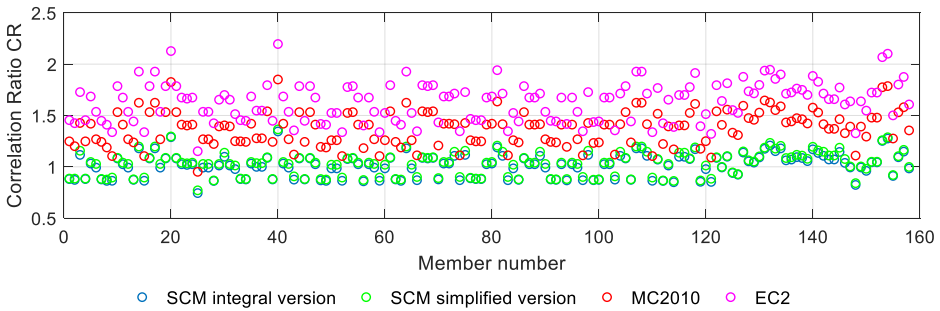


Fig. 3.26. Correlation Rate CR between the experimentally calculated one $s_{rm_{exp}}$ and the theoretically calculated average crack spacing $s_{rm_{SCM}}$ with the integral and simplified SCM versions, with the MC2010 and EC2 code

Once again, the model providing scatter points closer to value 1 is also the one providing the most accurate predictions. From Figure 3.26, it can be evinced

that both versions of the SCM provide predictions with significantly superior CR values compared to the model codes. This stands as a testament to the performance of the former’s predictive skills. In the following, a statistical investigation is performed based on Figure 3.26’s data. Table 3.5 reports the average and standard deviation values of the above crack spacing predictions.

Table 3.5. Average and standard deviation of the results provided by the integral and simplified versions of the SCM, the MC2010 and the EC2 model codes

Average crack spacing s_{rm} prediction method	Average	Standard Deviation
SCM Integral	1.0117	0.1118
SCM Simplified	1.0305	0.1138
Model Code 2010	1.3745	0.1650
Eurocode 2	1.6315	0.1891

As foreshadowed by Figure 3.26 the SCM models have not only a mean value very close to 1 but also have the smallest standard deviations of the four predictive models. Table 3.5’s data can be graphically represented in the format of a normal distribution graph (Figure 3.27) or of a box and whiskers graph (Figure 3.28).

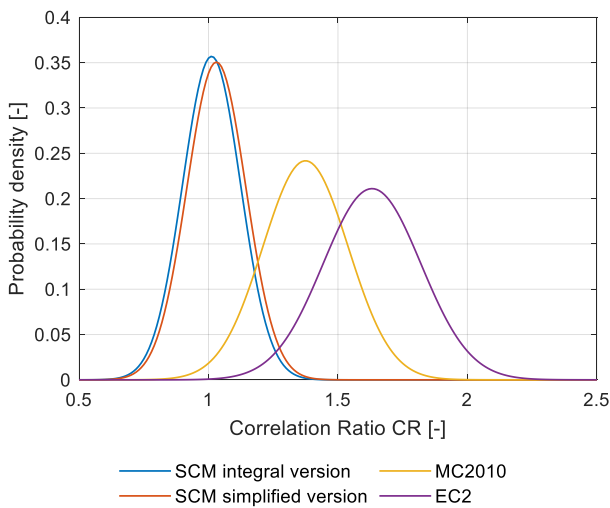


Fig. 3.27. Normal distribution plot of the Correlation Rate CR of the predictions extracted with the integral and simplified SCM versions, the MC2010 and EC2 code

Please note that on each box of Figure 3.28, the central mark indicates the median, and the bottom and top edges of the box indicate the 25th and 75th percentiles, respectively. The whiskers extend to the most extreme data points not considered outliers, and the outliers are plotted individually using the '+' symbol.

Overall, the performance difference between predicting models is entirely in favor of the newly introduced Strain Compatibility Model SCM. The average is not only significantly closer to 1 but the data spread is considerably inferior too.

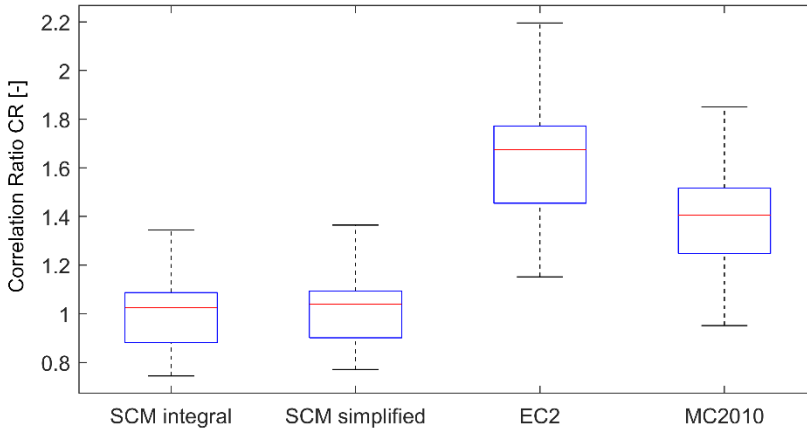


Fig. 3.28 Box and whiskers plot of the Correlation Rate CR of the predictions extracted with the integral and simplified SCM versions, the MC2010 and EC2 code

Comparing the integral and simplified versions of the SCM, the similarity of the results is quickly noticeable suggesting their dual performance as a theoretical prediction model along with each other's substitutability. It should be kept in mind though that such similarity is strictly a consequence of the selected parameters. Indeed, if to vary the debonding zone length l_d , bond stress value τ and reference load P this similarity would be lost. Lastly, Figure 3.29 uses a box and whiskers graph to represent the accuracy with which the combination of DOFS and SCM predicted the crack spacing for Figure 3.26's members sorted per reinforcement ratios. As visible, whilst $\rho = 0.79$ and 0.314 have average CR values around 1, they are also the one with the largest variance. Furthermore, reinforcement ratio $\rho = 0.154$ has a large number of outlier (indicated by the large amount of '+' symbols). In order to tackle both these issues and achieve the maximum performance of the SCM it is important to obtain a rather large DOFS extracted result database. This essentially represents the scope of future research.

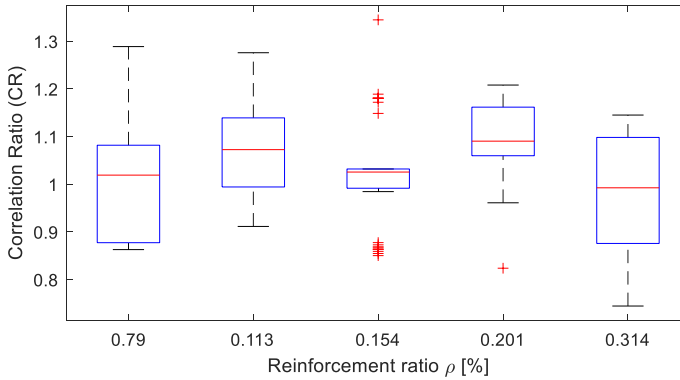


Fig. 3.29. Box and whiskers plot of the Correlation Rate CR of the predictions extracted with the integral SCM version for different reinforcement ratios

3.3. Assessment of the shrinkage-induced performance reduction of Reinforced Concrete structures through Distributed Sensing

The constant consumption growth of concrete across the globe has steadily increased the awareness over its behavior in both early and later stages. Indeed, its time dependent deformations can lead to serious issues such as malformation and member shortening when the concrete is still unloaded, excessive deflection in its service life and early cracking in both stages. Among the concrete time dependent behavior of concern for durable structures, shrinkage plays a crucial role (Al-Saleh 2014; Goel et al. 2007; Jasiczak et al. 2015; Yazdizadeh et al. 2017). Indeed, it can be responsible for structural safety and serviceability concerns of Reinforced Concrete (RC) structures even when designed according to design codes. With the increasing application of high-strength materials resulting in longer spans and smaller sections, the above mentioned issues are more relevant than ever as they literally affect the governing criterion used to design such structures.

One of these criterion is tension stiffening; the ability of concrete to carry a minor portion of the tensile stresses inside a RC member. The present section introduces a new tension stiffening law that accounts for the effects of concrete shrinkage and was developed by the authors before starting testing with Distributed Sensing. As such, its basis are empirical/statistical in nature. The curve fitting approach is used to analyze a large amount of test data reported in the literature review and, on its ground, a new shrinkage-free tension stiffening law is proposed.

The model completely relies on the accuracy of the predictions of concrete shrinkage and creep developed according to the codes (Eurocode 2 or MC2010) with no experimental proof on how the overlapping of the combined effects predicted by the code effectively translates to real RC members. In order to substantiate the present model, it is therefore crucial to assess the model codes' predictive capacities of the effects of concrete shrinkage and creep on the residual performance of a RC member. In order to achieve this, a DOFS monitoring campaign was designed to weigh such effects inside real life RC tie along the duration of the drying period of time. Finally, a comparison between the experimental results and the codes predictions is developed in an attempt to establish their applicability to the novel tension stiffening model.

3.3.1. Shrinkage-free tension stiffening law for various concrete grades

The tension stiffening effect on the serviceability analysis of RC structures has been the object of numerous studies (Gilbert & Ranzi 2010; Bischoff 2001; Gilbert & Warner 1978). The stress-transfer approach based on the bond-slip interaction between concrete and reinforcement most realistically deals with it. However, the lack of adequate bond-slip models and complex mechanisms of analysis limit a wider application of the latter approach. Models with smeared (averaged) representation of deformations and cracks due to their simplicity are most extensively used in the numerical applications (Murray et al. 2018). Gribniak et al. (Gribniak & Mang 2015) has proposed a stochastic approach in assessing tension stiffening. A simplified approach suggested by Eurocode 2 based on the interpolation formula relating non-cracked and fully cracked states is frequently applied for the deformation analysis of RC ties. However, a statistical analysis (Kaklauskas et al. 2015) has shown that it provides a far too stiff deformation response, particularly for lightly reinforced members. The prediction errors at various load levels ranged from 25 to 61 % for lightly reinforced tensile members ($\rho < 1.6 \%$) and from 14 to 30 % for the members with larger amounts of reinforcement ($\rho \geq 1.6 \%$). The above inaccuracies to a significant extent were due to the shrinkage effect occurring prior to loading that was not taken into account by the Eurocode 2. Due to the restraining action of reinforcement or supports, shrinkage induces tension stresses in the concrete, which might significantly reduce crack resistance and increase deformations of the member. Investigations (Kaklauskas & Gribniak 2011; Bischoff 2001) have shown that shrinkage may indeed have a significant effect on deformations of RC members subjected to short-term loading. However, very few tension stiffening models were proposed (Bischoff 2001) that take into account or remove the shrinkage effect before

calibrating the model; moreover, such models were generally based on a limited amount of test data.

The proposed tension stiffening model is based on the test data of seven experimental programs reported in the literature, including 108 RC tension members covering a wide range of concrete grade, reinforcement ratio and bar diameter. The test data covers a wide range of geometrical and material properties such as concrete grade, reinforcement ratio and bar diameter. The proposed tension stiffening relationship has the removed shrinkage effect occurring prior to the external loading.

The techniques to remove the shrinkage effect from load-deformation and tension stiffening relations have been proposed by Bischoff (Bischoff 2001) and Kaklauskas and Gribniak (Kaklauskas & Gribniak 2011). The latter approach is more general as it is applicable not only to tensile members, but also to a combined action of axial force and bending moment. The removal of the shrinkage effect from tension stiffening relations of tensile RC members can be performed in a simple way by lifting up the stress–strain diagram by stress $\sigma_{ct,cs}$ corresponding to the shrinkage-induced tensile stress in the concrete surrounding the rebar. In such way the experimental load-strain curves of the RC ties were adjusted to eliminate the shrinkage effect. For that, the resultants of the shrinkage-free tension stiffening stresses were summed up with the internal forces of tensile reinforcement. The principle of superposition was applied to every experimental point with increasing load and strain, resulting in the load-strain diagrams.

In such a way shrinkage-free tension stiffening relations were derived for each of the test specimen employed in the current study. Figure 3.30 plots the descending branches of the shrinkage-free stress-strain tension relations obtained for all the data set.

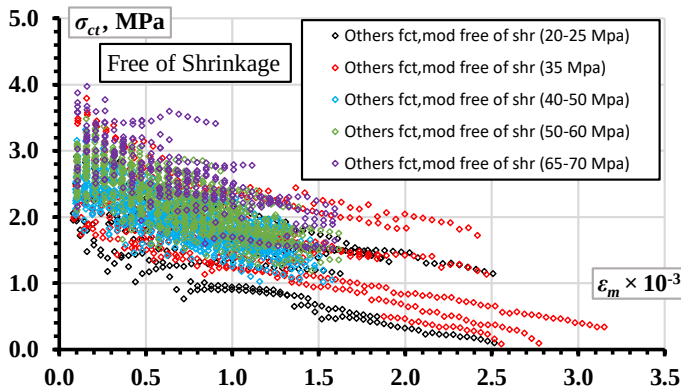


Fig. 3.30. Experimental data set for deriving shrinkage-free stress-strain tension stiffening law

To have equal representation of different loading levels of the members with different concrete strengths, the data set included the test points corresponding to certain levels of normalized strains ($\varepsilon_m = \varepsilon_{ct}/\varepsilon_{cr} = 1.0, 1.5, 2.0, 2.5, \dots$). Where ε_{cm} is the member mean strain and $\varepsilon_{cr} = f_{ct}/E_c$ is the cracking strain of concrete with f_{ct} the tensile strength of concrete defined according to EC2 and E_c the modulus of elasticity of concrete. The tension stiffening diagrams were terminated at the yield load. The current study follows the traditional approach of plotting the concrete tensile behavior in RC members by splitting the mean stress-mean strain relationship into two parts, the ascending branch and the descending one. The first one represents its elastic behavior assuming that the concrete is still intact whilst the second branch examines the RC member's behavior subsequent the surpassing of the concrete's cracking stress. The geometry of the ascending branch will be dictated by the material's elastic constitutive law as per Equation 3.45:

$$\sigma_{ct} = \varepsilon_{ct} E_c \quad (\varepsilon_{ct} \leq \varepsilon_{ct,max}). \quad (3.45)$$

The elastic behavior will be terminated at the point representing the maximum stress, $\sigma_{ct,max}$ (Equation 3.46), and the respective strain, $\varepsilon_{ct,max}$ (Equation 3.47):

$$\sigma_{ct,max} = 0.025 f_{ct} + 1.54; \quad (3.46)$$

$$\varepsilon_{ct,max} = \frac{\sigma_{ct,max}}{E_c}. \quad (3.47)$$

Where f_c is the compressive cylinder strength of concrete. The descending tension stiffening curve was obtained by means of curve fitting using the test points given in Figure 3.30. Among a wide range of possible fitting curves, the one shown in Equation 3.48 has been selected as a sensible compromise between accuracy and simplicity:

$$\sigma_{ct} = 0.025 \cdot f_c - \frac{0.85 \cdot (\varepsilon_{ct} \cdot 10^3)^{0.8} - 1.5}{0.25 \cdot (\varepsilon_{ct} \cdot 10^3)^{0.3} + 0.8}, \quad (\varepsilon_{ct} \geq \varepsilon_{ct,max}). \quad (3.48)$$

Figure 3.31 shows the variation of the descending branches of the proposed tension stiffening law for different concrete strengths: 35 MPa, 45 MPa, 55 MPa and 65 MPa and are compared against the test points representing different intervals of concrete strength. A noteworthy feature of the proposed constitutive law is its relation to the compressive strength of concrete instead of the tensile strength as in most tension stiffening laws.

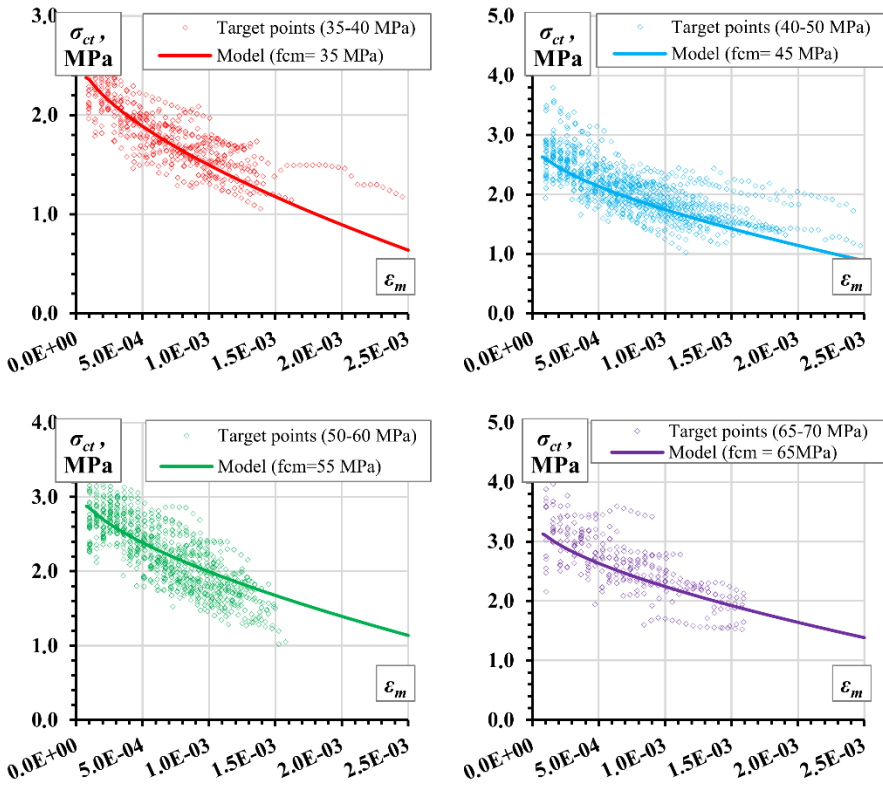


Fig. 3.31. Proposed tension stiffening law against experimental target points grouped in the ranges of concrete strength f_c between: (a) 35–40 MPa; (b) 40–50 MPa; (c) 50–60 MPa and (d) 65–70 MPa

This way the proposed model avoids the inaccuracies associated with the uncertainties related to empirical expression of the tensile strength of concrete. On the other hand, this removes the ability of using the actually measured tensile strength.

The authors of such model recognized the latter deficiency as a crucial one and decided to tackle it directly in the later years. This time instead, a different philosophy was adopted. Instead of resorting to statistics applied to the average behavior of RC ties, a monitoring of the actual shrinkage occurring in every RC tie section was planned with the strains present in the concrete surrounding the rebar being calculated inversely.

3.3.2. Distributed Sensing contribution to studying the effect of concrete-shrinkage in a Reinforced Concrete member

In the previous chapter, in order to proceed to the removal of the shrinkage effect from tension stiffening relations of tensile RC members, a simple lift up of the stress-strain diagram by stress $\sigma_{ct,cs}$ was performed. This value was calculated on the grounds of the model codes Eurocode 2 and Model Code 2010 hence fully relying on its accuracy. This, despite being a valid methodology, is a mere mathematical trick to investigate the predicted average performance of any RC member beyond its drying period of time.

With the goal of actually studying the real physical and experimental shrinkage of dried RC members and check the correctness of the predictions of the model on which the novel tension stiffening model is based, an in depth monitoring is required. Given DOFS' great advantages as a monitoring tool, it is the ideal pick for achieving such goal. Thus, an experimental campaign was designed was designed with this express purpose.

The campaign's objective is the continuous monitoring of several rebars' strains when embedded inside concrete from the moment the latter is poured onwards. This should provide a clear picture of the strains that shrinking concrete transfers to the embedded rebar due to the latter's restraint. These strains will further be studied under two frameworks; short-term and long-term shrinkage. Whilst the former tackles the issue of strains in the constitutive materials in a potential real-life construction time span, the latter confronts this matter for a 28-days drying/hardening period of time. Indeed, as mentioned above, it is crucial to evaluate the possibility of short-term concrete cracking and its reduction of tensile-carrying capacity in both stages. The present chapter is the result of a collaborative effort between research teams of Universitat Politècnica de Catalunya (UPC), CHALMERS University of Technology and Vilnius Gediminas Technical University (VGTU). Thus, the presented experimental campaigns, despite being conceptually identical, differ in materials and setup.

As previously discussed, the experimental results on shrinkage pertain to different experimental campaigns performed by the author. The first was based on the single RC tie (10x10_D12), henceforth defined as Group A, monitored through strictly thin DOFS. The second campaign was made up of three thin DOFS instrumented members (8x8_D8, 12x12_D12 and 12x12_D16), henceforth defined as Group B. Lastly, the third campaign monitored the shrinkage induced strains inside three identical 10x10_D16 members (henceforth defined as Group C) with two different kinds of fiber, one thin and one thickly coated. The thick fibers monitoring the three 10x10_D16 members were effectively one single 30 m long fiber monitoring all three members simultaneously.

3.3.2.1. Short term shrinkage

The study case DOFS-extracted steel rebar strain profiles will be reported hereinafter for a period of time of approximately 6 days. In this time interval it is possible to witness the initial rise of strains due to heat being released by the hydration reaction of cement and the consequent trend inversion whenever the latter slows down to the extent that concrete has time to dissipate the heat produced during the exothermic reactions. Before proceeding to the results, the following aspects should be kept in mind:

First of all, when analyzing early-age shrinkage results, it is necessary to have a consistent reference point for its start. Yet, this is a very aleatory choice as it is close to impossible to guess the precise time when the concrete has sufficiently hardened to start compressing the embedded rebar. The task is rendered even harder by the thermal reactions occurring in the first 24 h. For uniformity purposes, especially when comparing the shrinkage measurements of various mixtures, it was assumed that any shrinkage occurring prior to the initial monitoring time was insignificant because of the fluid nature of the concrete. Furthermore, the strain measurements were performed by two different OBRs. Consequently, the output plots of the Group 3 (G3) members are composed of many more points due to the ODiSI 6000's higher spatial resolution.

Figure 3.32 represents such DOFS recorded strain profiles along the members for five instances along the duration of the short term shrinkage monitoring (2 h, 24 h, 33 h, 54 h, 129 h). Unfortunately, member 8x8_D8 was successfully monitored in just the first half of its length as its second one (dashed lines) is dominated by SRAs in the form of large, unrealistic and thus unreliable strain peaks. However, the mid-section of its largest segment was not affected by such SRAs and can provide reliable data on the member's maximum concrete shrinkage-induced strains (DOFS section 0.8250). The sum of these results gives a clear picture and an experimental confirmation on what is the standard shape of an increasingly compressed rebar embedded inside shrinking concrete. Indeed, it is evident in Figure 3.32 that in most of the curves different strain values are recorded in the center and on the edges of the members. For example, the strain profile of member 12D12 at 24 hours has a strain value of approximately $150\mu\epsilon$ at DOFS coordinate 0.9225 m (mid-section) and $90\mu\epsilon$ at DOFS coordinate 0.7050 m (edges). Similarly, at 129 hours at the respective strains are approximately $-50\mu\epsilon$ and $-70\mu\epsilon$. Overall it can be assumed that the central part is averagely having larger strains than the edges as the stresses of the rebar tend to accumulate from the member's edges to the middle portion. Furthermore, the members pertaining to Group 2 are still in tension at 24 h whilst the others have already expired this initial phase. For simplicity's sake, the results of the three members 10D16 extracted by means of the thin and thick DOFS in Figure 3.32's last two frames are their average.

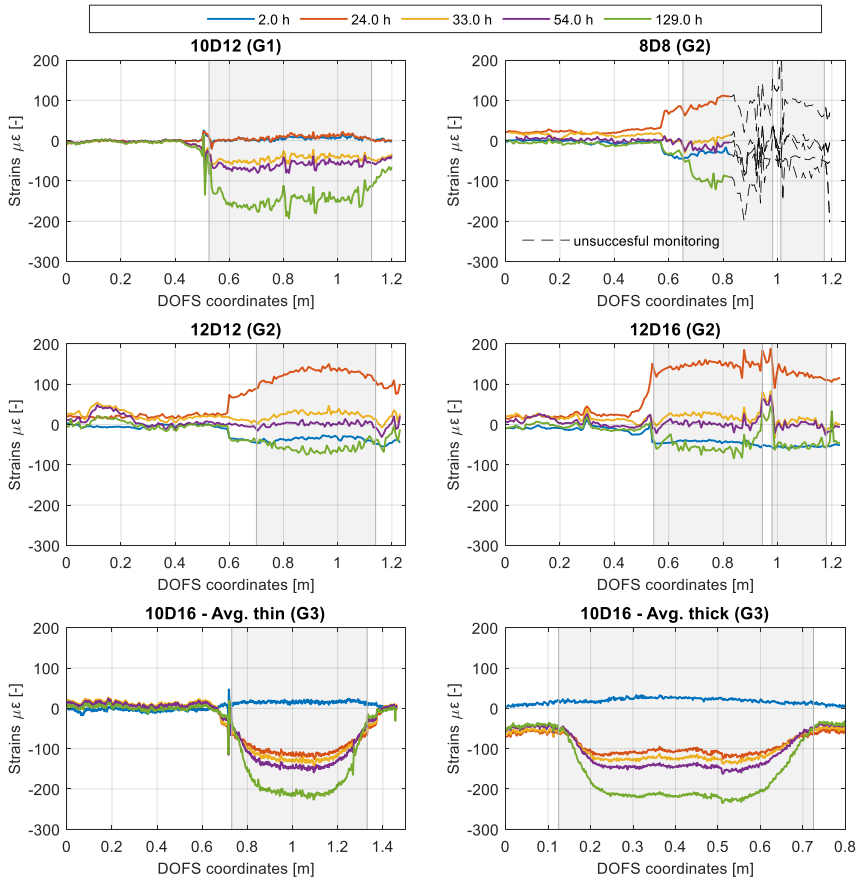


Fig. 3.32. DOFS sampled strain profiles at 5 instances during the short-term shrinkage in all six RC ties

Reporting strictly the latter is conceptually correct thanks to the small difference between the three 10D16 members' readings.

Figure 3.33 displays the strain evolution of a single DOFS coordinate, corresponding to the mid-span, per each of the Ø16 rebars.

Figure 3.34 expands Figure 3.33's concept to all the members thus reporting their rebar mid-section strains (the largest segments' in case of multiple-segmented members) in order to highlight any common trend. In Figure 3.34 the dots are representative of the data scatter as measured along time whilst the dashed lines are the result of an algorithmic curve fitting interpolation of the measured data. The curves pertaining to Group 2's members do not start from 0 $\mu\epsilon$ but from a certain compression value pre-existent in the rebar.

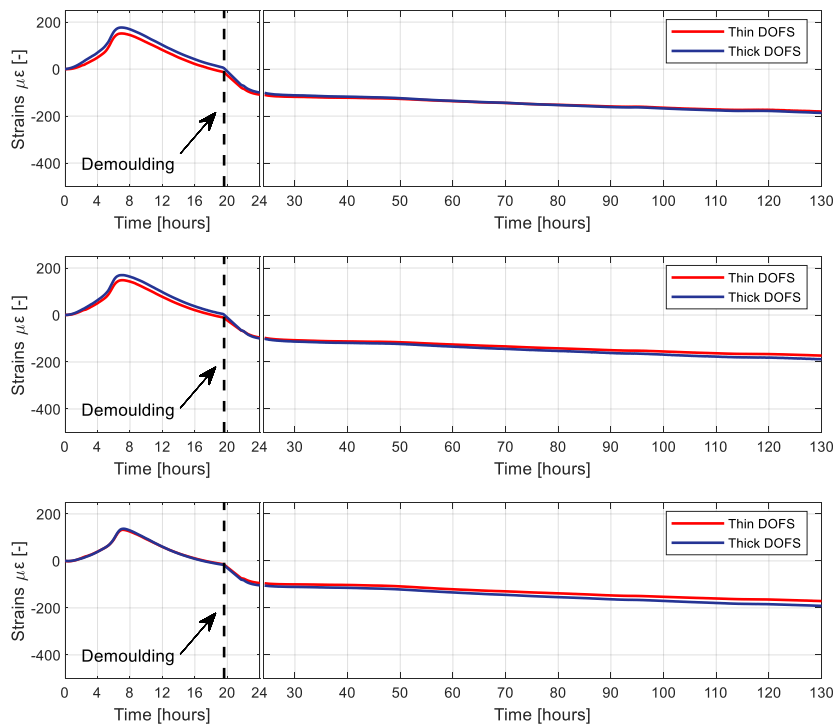


Fig. 3.33. Rebar strain profiles extracted from the mid-span of the three 10D16 members (G3) through the thin and thick DOFS

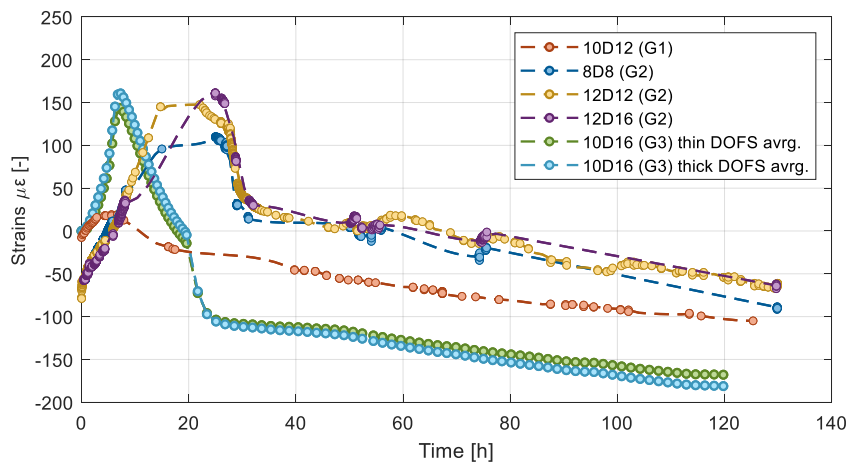


Fig. 3.34. Concrete shrinkage-induced strain on the rebar over a 130 hours monitoring time span

The latter was due to rebar's thermal contraction (with respect to the moment that the DOFS were glued corresponding to the 0 strain state) that occurred when casting the RC ties in an external winter environment.

It is distinctly seen that all members stage a similar pattern of strain development. Expectedly, in the initial hours of concrete shrinkage the heat expelled by the cement hydration reaction, combined with its consequent local expansion of concrete (Holt 2005), causes local tension in the embedded rebar (initial lump). Note that it is up for discussion whether the concrete at this early stage is hard enough to be able to transmit stresses to the embedded rebar i.e., the thermal expansion-induced tension. This positive strain growth slows down after certain hours post-casting (when the hydration slows down) until it fully stops and the trend is reversed. The amount of time required for this to occur varies among the three member groups due to the difference in their constitutive materials, such as cement type and amount of fines. Member 10D16 stands out for its rapid rise in strains and equally fast trend inversion. Member 10x10_D12, instead, stands out for its reduced and rather blunt strain peak. Beyond this point, though, drying shrinkage acquires an increasingly larger role in the concrete's total shrinkage which, once again, is restrained by the embedded rebars. As visible in the experimental data, such restraining effort from the rebars' part causes the latter to enter a compressive state (negative increase in strains).

Interestingly, the experiment displays how the early shrinkage trends (whole 130 h monitoring) seems to be influenced almost exclusively by the constitutive materials versus the almost trivial influence of the bar diameter and cross-sectional dimension. Indeed, the plotted graphs of members 8x8_D8, 12D12 and 12x12_D16 are almost equivalent after the initial lump and furthermore slowly approaching member 10x10_D12's graph.

3.3.2.2. Long term shrinkage

In the present section the long-term (28 days) shrinkage monitoring of members 10x10_D12 and the three 10D16 will be displayed. Starting from the former, Figure 3.35 shows its longitudinal strain evolution at multiple instances along the 28 days. As observed, the long-term shrinkage induced strain profiles do not significantly change in shape compared to the short term drying except for the rebar's average strain in its final stage (approximately $225 \mu\epsilon$ in the long term versus $150 \mu\epsilon$ in the short term). Furthermore, the profile's horizontal trend is disrupted by two alterations in the member's DOFS coordinate 0.814 m and 0.91 m, which, as discussed later on, could be attributed to concrete-shrinkage induced micro-cracks. Figure 3.36 below provides similar graphs to Figure 3.35 for the strains inside the three 10D16 members as sampled by the thin and thick DOFS fibers.

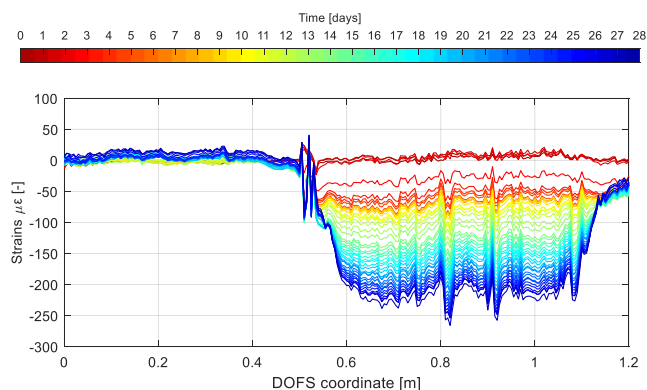


Fig. 3.35. Concrete shrinkage-induced strain inside member 10x10_D12 over a 28 days monitoring time span

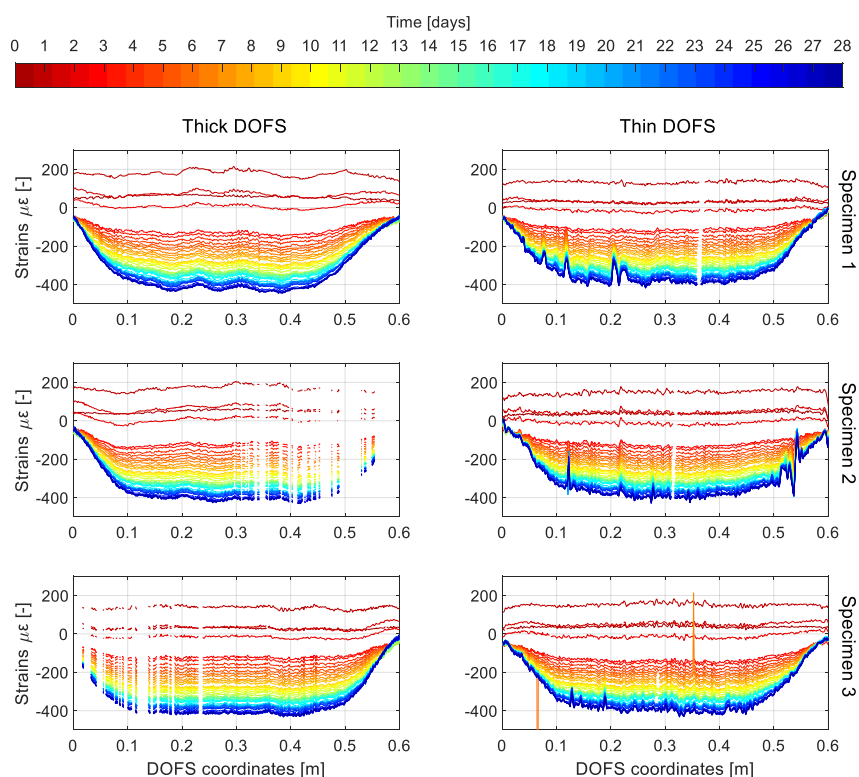


Fig. 3.36. Strains inside the three 10D16 specimens as sample by both thin and thick DOFS

Figure 3.36's thick fiber results display some data gaps which were erroneously left out by the DOFS. Notwithstanding the profiles and its trends are still clear and thus the authors' decision of plotting the actual raw data. Figure 3.36's strain profiles are not only similar to each other (testament of the quality of test and of the performance of the two types of DOFS) but also to their short-term counterpart and, geometrically-wise, to member 10x10_D12's profiles. Figure 3.37 summarizes the two previous figures in an attempt to grasp the performance differences of the thin and thick fibers and compare their results to member 10x10_D12's.

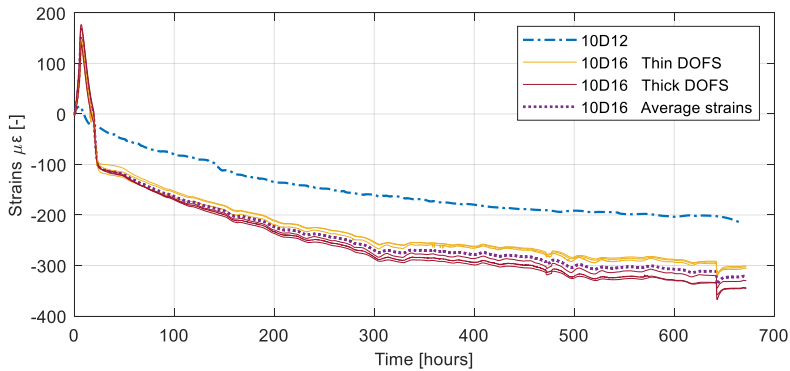


Fig. 3.37. The RC ties' mid-section strains as measured by thin and thick DOFS embedded inside members 10x10_D12 and 10D16

The RC ties' mid-section strains are henceforth plotted along with an average of all six 10D16 readings. As visible, the strains measured by the 10D16's thin DOFSs are very consistent among each other with a maximal difference of $5 \mu\epsilon$ after 672 hours. Less consistent (maximal difference of $15 \mu\epsilon$ after 672 hours) but still satisfying are the thick DOFSs' strain measurements. Whilst the plots of the two kinds of fiber report simultaneous strain variations along time, their uniformity (initially strong) slowly degrades with time reaching a maximal difference of $44 \mu\epsilon$ at 672 hours. Despite the slight degradation with time, the correlation between the measurements performed by the thin and thick fiber is still quite tight. This is encouraging considering the thickly coated fiber's potential of being embedded inside concrete without any extra support or protection. It could, indeed, give access to cross-sectional strain variation data inside the concrete itself.

Finally, as per the short-term shortening, it is clear that both 10x10_D12 and 10D16s members stage a similar pattern of strain development. Despite such, their difference (up to $100 \mu\epsilon$ at 672 hours) is evident and can be attributed mainly to

the constitutive materials and drying conditions since, as seen before, the difference in rebar diameter covers very little influence.

3.3.2.3. Comparison with the Eurocode 2 and the Model Code 2010 predictions

In the present section the authors intend to compare the experimental results with theoretical ones extracted from the Model Code 2010. By means of such comparison it will be possible to assess the code's prediction power of the shrinkage-induced RC members' inner-workings.

In order to achieve such objective, the following theoretical approach is considered. The shrinkage induced axial strain on the steel rebar $\varepsilon_{s,shk}(t, t_0)$ (as a function of time t and the initial 0-stress time t_0) is modelled as the result of a fictitious force P_{shk} that tends to compress the study case RC member as in Equation 3.49:

$$\varepsilon_{s,shk}(t, t_0) = \frac{P_{shk}(t, t_0)}{A_c E_c + A_s E_s} = \frac{\varepsilon_{shk}(t, t_0) A_c E_c}{A_c E_c + A_s E_s}. \quad (3.49)$$

Where A_c and A_s are the transversal area of concrete and steel relatively, $\varepsilon_{shk}(t, t_0)$ is the concrete free shrinkage strain (calculated also according to the MC2010) and E_c and E_s are the modulus of elasticity of concrete and steel relatively. In order to take in consideration the strain relieving effect of tensile creep, E_c should be replaced with the Age Adjusted Modulus of Elasticity (AAEM) $E_{ca}(t, t_0)$ as in Equation 3.50.

$$\varepsilon_{s,shk}(t, t_0) = \frac{\varepsilon_{shk}(t, t_0) A_c E_{ca}(t, t_0)}{A_c E_{ca}(t, t_0) + A_s E_s}. \quad (3.50)$$

This is a product of a method developed by Trost-Bazant (Bazant 1972) in order to study the long term strains under varying stresses and can be calculated as in Equation 3.51:

$$E_{ca}(t, t_0) = \frac{E_c(t)}{1 + \varphi(t, t_0) \chi(t, t_0)}. \quad (3.51)$$

Where $\varphi(t, t_0)$ is the related creep factor and $\chi(t, t_0)$ the aging coefficient. The latter is studied for a double value of 0.5 and 0.8 (the latter being suggested by the MC2010 for simpler analyses). It should be kept in mind that, as suggested by the MC2010 model itself, after short durations of loading or drying, the prediction error is higher than after long durations.

Figure 3.38 and Figure 3.39 plot the shrinkage and creep-induced strain curves, as calculated from the above method with the EC2 and MC2010 code,

with the members 10x10_D12's and the average of 10D16's experimentally extracted ones for different values of the creep factor (0.8 and 0.5 respectively).

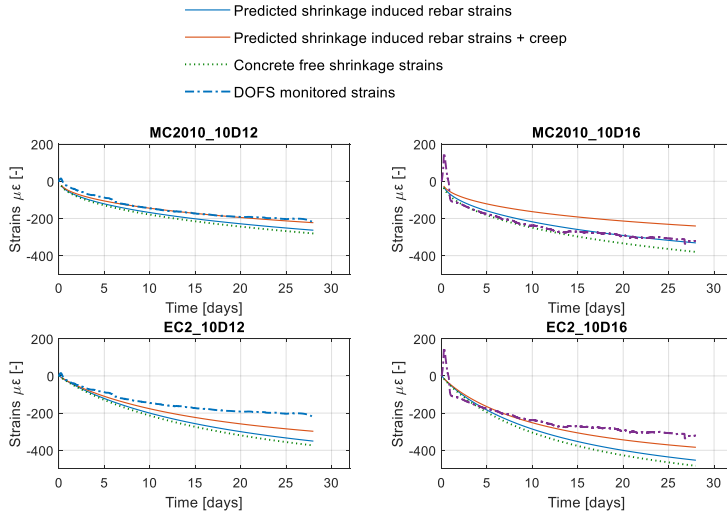


Fig. 3.38. Concrete shrinkage and creep induced steel strains on 10x10_D12 and 10x10_D16's rebars according to the EC2 and MC2010 codes with $\chi = 0.8$

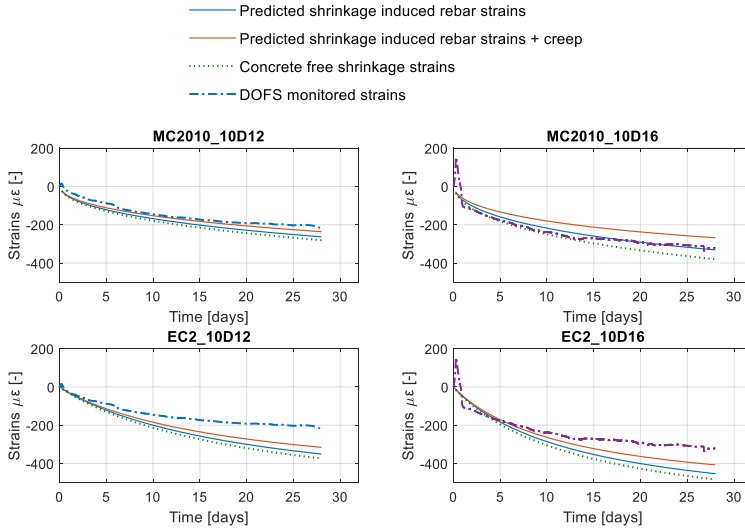


Fig. 3.39. Concrete shrinkage and creep induced steel strains on 10x10_D12 and 10x10_D16's rebars according to the EC2 and MC2010 codes $\chi = 0.5$

Both Figure 3.38 and Figure 3.39 suggest that for both ageing coefficients 0.5 and 0.8 the correlation between the predicted and experimental results is superior for the MC2010. Yet, considering the latter model code, member 10x10_D12's DOFS plot coincides with the theoretical curve that includes the relaxing effect of tensile creep, on the other hand member 10D16's is closer to the predicted curve that excludes the latter effect. Finally, of the two ageing coefficients used, 0.8 seems to prove more efficient in predicting the concrete-shrinkage induced strains on the steel rebars. Finally, it can be stated

This analysis has now been performed for only two members but through the creation of a large database of DOFS monitored shrinkage campaigns, it would be possible to accurately establish if the EC2 and the MC2010's predictions of shrinkage and creep effectively averagely underestimate, overestimate or are on a par with the real strains inside the RC member.

By means of the DOFS extracted steel strain profiles and the codes' free shrinkage strain predictions, it is also possible to calculate the stresses in the concrete surrounding the rebar and check if its tensile strength has already been exceeded. In the absence of external forces, the equilibrium of the internal forces sees the concrete's and the reinforcement's equal in magnitude and opposite in sign, therefore, the resulting concrete stress at 28 days can be calculated by means of the Eurocode 2, $\sigma_{c,28_EC2}$ as per Equation 3.52 or Model Code 2010 $\sigma_{c,28_MC2010}$ as per Equation 3.53 or directly from the DOFS fibers $\sigma_{c,28_DOFS}$ as per Equation 3.54.

$$\sigma_{c,28_E2} = - \frac{\varepsilon_{shk,EC2}(t,t_0) E_s \rho}{1 + \frac{E_s}{E_{ca}(t,t_0)} \rho}; \quad (3.52)$$

$$\sigma_{c,28_MC2010} = - \frac{\varepsilon_{shk,MC2010}(t,t_0) E_s \rho}{1 + \frac{E_s}{E_{ca}(t,t_0)} \rho}; \quad (3.53)$$

$$\sigma_{c,28_DOFS} = - \frac{\varepsilon_{s,28_DOFS} E_s A_s}{A_c}. \quad (3.54)$$

Being ρ the element's reinforcement ratio. With these equations and the values extracted from Figure 3.37 and Figure 3.38, $\sigma_{c,28}$ can be calculated. The resulted are listed in Table 3.6 below. Overall it seems that once again the MC2010 code is the extracting results closer to the experimental ones, solidifying itself as the reference model and grounding of shrinkage free serviceability analyses of RC structure. The analysis was performed for both Age Adjusted Modulus of Elasticity (AAEM) and normal modulus of elasticity of the concrete at 28 days.

Table 3.6. Concrete stresses predicted according to the EC2 and MC210 code

$\sigma_{c,28}$ [MPa]		MC210		EC2		DOFS
		$\chi = 0.5$	$\chi = 0.8$	$\chi = 0.5$	$\chi = 0.8$	
D12	E_c	0.59		0.81		0.49
	E_{ca}	0.54	0.51	0.73	0.73	
D16	E_c	1.40		1.78		1.36
	E_{ca}	1.12	1.01	1.52	1.39	

Whilst for 10x10_D12 the use of the former suggests an improved prediction, this is the opposite for the latter. Further research on the topic is required. If to compare these values to Table 2.3's f_{ct} , it is clear that the present members' concrete tensile strength is not surpassed in the entire shrinkage phase.

Concluding the present section, overall it can be stated that an evident congruence was found between the experimental data extracted with DOFS fibers during the RC ties' hardening with the theoretical predictions developed by the Eurocode 2 and Model Code 2010 (more for the latter). Further congruence can be found in the prediction of the stress suffered by the concrete surrounding the rebar which, in both cases, is smaller than its tensile strength. This stands as testament of the performance of the predictive model in calculating the shrinkage induced strains in a RC member steel rebar.

3.4. Conclusions of Chapter 3

The findings and observations from the integration of the DOFS-powered experimental campaigns results to both present-day and novel RC structures serviceability analyses can be summarized as follows:

1. It was observed that the geometry of the experimental mean bond stress curves was substantially influenced by the RC specimens' reinforcement ratio ρ and crack spacing s_{rm} .
2. The existence of a correlation between the bond stress and the specimens' ρ was revealed as per the function $\tau_{DOFS} = (3 - \rho/4) * f_{ct}$. This novel bond stress law represented one of the biggest findings of the present research.
3. The present work embraces the superiority of the stress-transfer approach for the formulation of RC structures' serviceability models and, on the back of the revolutionary ability of DOFS, proposes a

revisitation of the Strain Compliance crack spacing Model (SCM). By means of DOFS, the author demonstrated the correctness of SCM's core hypothesis, namely that the strain profile of a rebar embedded in a RC tie follows a regular linear profile constituted by three zones whose total length is equals to the average crack spacing.

4. Integrating the previous DOFS-powered bond stress law in the revisited version of the SCM, the latter's crack spacing prediction power was plainly superior to that of the fib MC2010 and EC2 (62% more accurate than the MC2010 and 73% more accurate than the EC2).
5. A novel shrinkage-free tension stiffening serviceability model was established by means of a curve fitting approach on point clouds of shrinkage-free RC ties' average tensile strains values. Shrinkage was removed using the model codes' shrinkage predictions, thus assuming their correctness. The veracity of the latter was demonstrated by multiple DOFS-monitored experimental campaign performed on shrinking RC ties, thus validating the new tension stiffening model.
6. The data extracted from the above DOFS monitoring campaign not only substantiated the newly introduced tension stiffening and crack spacing models but further fine-tuned them, increasing their performance and providing the Structural Engineering field with novel, practice based, non-theoretical and non-empirical RC structures' serviceability models.

General Conclusions

Originating from the literature review and the investigation of the background of the study of serviceability for Reinforced Concrete (RC) structures, the following key points can be conveyed:

1. Between the stress transfer and mean strain approach, the former, albeit more demanding, is undoubtedly the superior method for the serviceability analyses of RC structures, providing a realistic reflection of their behavioral nature, of its bond-slip mechanisms, of the effect of tension stiffening and of the crack development.
2. In order to determine the key parameters of the stress transfer approach i.e., concrete-reinforcement bars (rebars) bond stress and slip, a correct assessment of the strains of the rebars embedded in RC structures is required. A novel tool able to achieve this objective with unprecedented accuracy and performance (strain resolution $1\text{ }\mu\epsilon$, spatial resolution 0.63 mm and measurement frequency 250 Hz) are Distributed Optical Fiber Sensors (DOFS).

The findings and observations from the consequent experimental investigation on RC tensile elements with DOFS-instrumented rebars can be summarized as follows:

3. The DOFS deployment technique that ensured its best insulation from the concrete friction - thus safeguarding the measurement performance-

- whilst guaranteeing the least amount of interference possible was combination of a groove incised in the rebar, cyanoacrylate and silicone.
4. Experimental efforts demonstrated DOFS is capable of extracting crucial parameters for serviceability analyses of RC structures i.e., (a) the stress state present in the steel and in the surrounding concrete at any instance of the test -even beyond yielding- (b) the occurrence of concrete shrinkage-induced cracking and tensile strength reduction (c) the bond stress and slip between the concrete and steel rebar occurring on their interface (d) crack spacing and (e) crack width.

The findings and observations from the integration of such experimental results to both present-day and novel RC structures serviceability analyses can be summarized as follows:

5. The mean bond stress between concrete and the steel rebars was found to be dependant on the length of the reinforced concrete elements (length of the RC blocked comprised between two adjacent cracks) and their reinforcement ratio ρ . Based on these results, a novel mean bond stress law τ_{DOFS} was introduced as a function of the reinforcement ratio ρ .
6. With the integration of the novel DOFS-based bond stress law, the revised Strain Compliance Model (SCM) was found to provide more accurate crack spacings predictions than the widespread Model Code 2010 and Eurocode 2 methods. The values of the ratio (theory/experiment) of the SCM method were much closer to 1 and its standard error lower. The mean value for the SCM is 1,0117, and MC2010 and EC2 - 1,3745 and 1,6315 respectively. The standard deviation for these methods were - 0.112, 0.165 and 0.189 respectively.
7. It was assessed that a newly introduced tension stiffening model succesfully quantifies the structural phenomenon without the altering influence of shrinkage. The removal of such effect is performed by means of the MC2010 and the EC2 whose shrinkage predictive power was validated by means of DOFS.
8. DOFS innovatively makes it possible to estimate the extent to which the tensile strength of reinforced concrete elements decreases due to the shrinkage of concrete in addition to detecting shrinkage-induced cracks.

Overall, this thesis demonstrated the potential of DOFS to validate, integrate or dismiss present day RC structures' serviceability models. These results can be considered as the first step towards a re-visiting of the modern way of understanding the behavior of RC structures under serviceability conditions.

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Summary in Lithuanian

Ivadas

Problemos formulavimas

Projektuojant gelžbetonines konstrukcijas, du svarbiausi kriterijai yra saugumas ir tinkamumas. Pastarasis yra susijęs su konstrukcijų eksploatacija viso jų naudojimo laikotarpiu. Gelžbetoninių konstrukcijų analizę ir modelių kūrimą apsunkina betono elgsenos netiesiškumas, kurią salygoja betono valkšnumas, traukimasis bei plyšių susidarymas. Pasak klasikinio požiūrio, gelžbetoninių konstrukcijų tinkamumas skaičiuojamas laikantis prielaidos, kad armatūros ir betono sąveika yra ideali arba tariant, kad tarp armatūros ir betono veikia pastovūs sukibimo įtempiai. Nagrinėjant gelžbetoninių konstrukcijų tinkamumą naudoti, šios prielaidos nėra tikslios. Tikslesnių analizės modelių kūrimą riboja nepakankamai toboli eksperimentiniai konstrukcijų elgsenos stebėsenos įrankiai: nėra tokių deformacijų matavimo metodų, kurie leistų visuose gelžbetoninių konstrukcijų taškuose „matyti“ tai, kas vyksta jų viduje. Dėl to kuriami modeliai, netiksliai atspindintys konstrukcijų elgseną. Gelžbetoninių konstrukcijų tyrimuose taikomas įtempių perdavimo metodas, kuriuo apibūdinama sąveika tarp armatūros ir ją supančio betono. Ši sąveika apibrėžiama kaip sukibimo įtempis τ_b (arba šlyties įtempis), atsirandantis armatūros strypo ir betono kontakto zonoje. Įtempių perdavimo metodu galima nustatyti gelžbetoninių konstrukcijų

deformacijas, tempiamąjį sustandėjimą, atstumą tarp plyšių bei plyšių plotį. Pagrindinis šio metodo trūkumas – tai, kad iki dabar nėra sukurtas tikslus sukibimo įtempių – slinkties dėsnis. Viena to priežasčių – eksperimentinių deformacijas matuojančių priemonių, galinčių deformacijas nustatyti konstrukcijų viduje, stoka.

Darbo aktualumas

Šiame darbe nagrinėjami gelžbetoninių konstrukcijų tinkamumo naudoti klausimai. Darbe parodyta, kaip tinkamumo modelių kūrimui gali būti naudojami ištisinio matavimo optinio pluošto jutikliai (DOFS). Tokie jutikliai gali įvertinti besikeičiančius įtempių perdavimo parametrus gelžbetoninių konstrukcijų viduje, kintant apkrovoms. Tai reiškia, kad taikant DOFS galima tiksliai (matavimo jautrumas 1 $\mu\epsilon$) ir ištisai (matavimai atliekami kas 0,63 mm) tirti gelžbetoninių konstrukcijų deformacijas. Deformacijų stebėseną gali būti atliekama ne tik gelžbetoninių konstrukcijų paviršiuje, bet ir jų viduje – armatūros ir betono sąlyčio zonoje. Deformacijas galima tirti veikiant skirtingoms apkrovoms, prieš atsirandant plyšiams betone ir jiems atsiradus, ir netgi tuomet, kai viršijamas plieno takumas. Ši technologija leis pilniau panaudoti įtempių perdavimo metodo potencialą.

Tyrimo objektas

Šio darbo tyrimo objektas yra dvejopas: pirma, tai įvairūs gelžbetoninių konstrukcijų tinkamumo ribinio būvio mechaninės elgsenos aspektai (armatūros ir betono kontakto zonos sukibimo įtempiai/slinktis, atstamai tarp plyšių, tempiamasis sustandėjimas, suvaržytas traukumas); antra, tai eksperimentinis gelžbetoninių elementų strypinės armatūros deformacijų nustatymas ištisinio matavimo optinio pluošto jutikliais (DOFS).

Darbo tikslas

Šio darbo tikslas yra įvertinti ir pagrįsti ištisinio matavimo optinio pluošto jutiklių (DOFS) taikymo galimybes, kuriant gelžbetoninių konstrukcijų tinkamumo ribinio būvio analizės modelius (sukibimo įtempių – slinkties, atstumo tarp plyšių, tempiamojo sustandėjimo, suvaržyto traukumo).

Darbo uždaviniai

Tyrimo tikslui pasiekti keliama šie uždaviniai:

1. Įvertinti DOFS savybes, palyginant jas su klasikiniais deformacijų matavimo įrankiais.
2. Nustatyti metodus ir priemones, tinkamus panaudoti DOFS gelžbetoninių konstrukcijų tyrimui bei padedančius išvengti galimų netikslumų, tiriant deformacijų pasiskirstymą elementuose.
3. Taikant DOFS, eksperimentiškai nustatyti betono traukumo sukeltų gelžbetoninių konstrukcijų armatūros deformacijų profilius matuojamajame ilgyje.
4. Išanalizuoti eksperimentiškai gautus duomenis ir įvertinti plieno ir betono sukibimo įtempių / slinkties diagramas, atstumus tarp plyšių ir betono traukumo sukeltus įtempius.

Tyrimo metodika

Pirma, apibrėžiami tie gelžbetoninių konstrukcijų tinkamumo analizės modeliai, kuriuos reikia patikslinti. Antra, vykdomi nauji DOFS kontroliuojami eksperimentiniai tyrimai, būtini nustatyti kuriamų modelių parametrus. Trečia, gautų eksperimentinių duomenų apdorojimui, modelių pagrindimui, kūrimui ir integravimui panaudojama skaičiavimo programinė įranga. Ketvirta, pateikiamos išvados apie DOFS taikymo galimybes tikslu pagerinti gelžbetoninių konstrukcijų eksploatacines savybes.

Darbo naujumas

1. DOFS savaime yra naujovė, nes šie jutikliai neseniai pradėti taikyti statybos konstrukcijų tyrimams. Jie greitai tapo populiarūs, nes yra labai tikslūs, patikimi, jų taikymas nesudėtingas ir, svarbiausia, leidžia nustatyti deformacijas viso optinio sensoriaus ilgyje (matavimai atliekami kas 0,63 mm). Šiuo metu DOFS laikomi pažangiausiu įrankiu, atliekant konstrukcijų būklės stebėseną ir eksperimentiškai tiriant konstrukcijų deformacijas.
2. Eksperimentinių statybos konstrukcijų tyrimų, atliekant deformacijų stebėseną DOFS, yra labai nedaug, o tokių gelžbetoninių konstrukcijų tyrimų yra dar mažiau. Dauguma eksperimentinių tyrimų atlikta, jutiklius fiksuojant konstrukcijų paviršiuje. Šiame darbe eksperimentinis tyrimas atliekamas, armatūros deformacijas matuojant konstrukcijos viduje.
3. Tvirtinant optinį jutiklį prie armatūros, jos paviršius pažeidžiamas minimaliai. Taip yra todėl, kad DOFS jutiklio skersmuo yra labai mažas, ši savybė yra labai svarbi, atliekant numatytą armatūros ir

betono sukibimo tyrimą, ypač tuomet, kai naudojama mažo skersmens armatūra (pvz., Ø8). Anksčiau, taikant tradicinius deformacijų stebėsenos įrankius, buvo neįmanoma atlikti tokių tyrimų.

4. Iki šiol DOFS nebuvo taikomi tikslu sukurti, patobulinti arba patvirtinti/atmesti gelžbetoninių konstrukcijų modelių adekvatumą. Šiame darbe buvo kuriami nauji modeliai arba nurodytos gairės jiems kurti taikant DOFS.
5. DOFS dažniausiai taikomi vykdant konstrukcijų būklės stebėseną. Retesniais atvejais šie jutikliai buvo taikomi atliekant konstrukcijų bandymus. Šiandien žinomi tik kelių tyrėjų atlikti eksperimentiniai tyrimai, tačiau iki šiol DOFS nebuvo taikomi, vertinant betono ir armatūros sukibimo įtempius ir slinktį.
6. Šiame tyrime pateiktas naujas atstumų tarp plyšių prognozavimo modelis. Šis modelis remiasi deformacijų darnos prielaida, kuri yra grindžiama DOFS taikymu.
7. Iki šiol betono traukumo sukeltų armatūros deformacijų stebėseną taikant DOFS pasaulyje buvo atlikta tik vieną kartą. Šiame darbe gauti nauji eksperimentinių tyrimų rezultatai apie armatūros ir betono įrašų kitimą laike, traukiantis betonui.
8. DOFS gauti eksperimentiniai duomenys leido pagrįsti naują tempiamojo sustandėjimo modelį, kuriame eliminuota betono traukumo įtaka.

Darbo rezultatų praktinė reikšmė

1. Šiame darbe pateiktos eksperimentinių tyrimų atlikimo gairės, taikant ištisinio matavimo optinio pluošto jutiklius, atspindi būsimų statybos konstrukcijų stebėsenos ir tyrimo pagrindus.
2. Šiame tyrime pateiktas modifikuotas atstumo tarp plyšių modelis leidžia tiksliau nustatyti gelžbetoninių elementų plyšių plotį bei sutaupyti statybines medžiagas.

Ginamieji teiginiai

Ginamieji teiginiai grindžiami atlikto tyrimo rezultatais ir darbo autoriaus sukaupta patirtimi:

1. DOFS gali padėti tirti klausimus, susietus su gelžbetoninių konstrukcijų tinkamumo elgsena. Šiame darbe pateikiami keli tinkamumo modelių

- kūrimo, tobulinimo ir patvirtinimo / paneigimo pavyzdžiai, taikant DOFS.
2. DOFS minimalaus invazinio poveikio (kai beveik nepažeidžiamas armatūros strypas, prie kurio tvirtinamas jutiklis) savybė yra labai svarbi, bandymus atliekant su mažu skersmens armatūros skersmeniu.
 3. Taikant DOFS atlikta tiksli armatūros deformacijų gelžbetoninėse konstrukcijose stebėsena. Tai leido tiksliai apibrėžti plieno ir betono sąveiką per sukibimo įtempius, slinktį ir įtempių perdavimo ilgus.
 4. DOFS grįstais eksperimentiniais tyrimais buvo pademonstruotas deformacijų darnos modelio (SCM) adekvatumas. Gavus naują vidutinių sukibimo įtempių modelį, buvo modifikuoti SCM parametrai. Pastarasis modelis tiksliau prognozuoja atstumus tarp plyšių nei EC2 bei MC2010 metodai.

Darbo rezultatų aprobavimas

Iš viso buvo publikuota 14 mokslinių publikacijų, susijusių su šios disertacijos tema, iš kurių 8 – žurnaluose, turinčiuose cituojamumo rodiklį, 6 – įvairių tarptautinių konferencijų rinkiniuose. Doktorantūros studijų laikotarpiu (2016-2021) disertacijos rezultatai buvo paskelbti 6 konferencijose:

- G. Kaklauskas, M. F. Bado. A novel concept of crack analysis: prediction of mean crack spacing in FRP reinforced concrete beams. COST Action TU1207 Next Generation Design Guidelines for Composites in Construction. Budapest, Hungary. April 2017.
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Disertacijos struktūra

Disertaciją sudaro įvadas, 3 skyriai, bendrosios išvados, literatūros sąrašas (105 šaltiniai), autoriaus mokslinių publikacijų sąrašas (14 publikacijų), santrauka lietuvių kalba bei 5 priedai. Darbo apimtis 145 puslapiai. Darbe pateikti 62 grafikai, 12 lentelių per keturis skyrius.

Padėka

Autorius yra be galo dėkingas už paramą, kantrybę bei paskatinimą šiame kelyje, rengiant disertaciją, savo vyresniems ir jaunesniems šeimos nariams – Francesca, Renato, Irene, Greta, Aurora, Giulio ir savo partnerei gyvenimo ir nuotykių kelyje Alba.

Už vadovavimą ir konsultacijas rengiant šią disertaciją autorius išreiškia padėką savo mokslinio darbo vadovui – Gelžbetoninių konstrukcijų ir geotechnikos katedros profesoriui habil. dr. Gintariui Kaklauskui. Taip pat už konsultacijas, patarimus ir pagalbą autorius išreiškia dėkingumą Katalonijos politechnikos universiteto (Universitat Politècnica de Catalunya, UPC) profesoriui Joan Ramon Casas. Jis taip pat nori išreikšti nuoširdžią padėką dr. Regimantui Ramanauskui, dr. Pui Lam NG ir Alinda Dey už jų draugystę ir patarimus, taip pat prof. dr. Dariui Bačinskui bei dr. Aleksandrui Sokolov. Autorius taip pat dėkingas Vaivai Miškinytei už konsultacijas ir draugišką palaikymą. Jis išreiškia dėkingumą Gelžbetoninių konstrukcijų ir geotechnikos katedros darbuotojams, su kuriais teko dalintis šia doktorantūros patirtimi. Autorius išreiškia padėką Lietuvos mokslų tarybai už finansinę paramą doktorantūros studijų metu.

1. Gelžbetoninių konstrukcijų tinkamumo analizės modelių ir ištisinio dformacijų matavimo metodų apžvalga

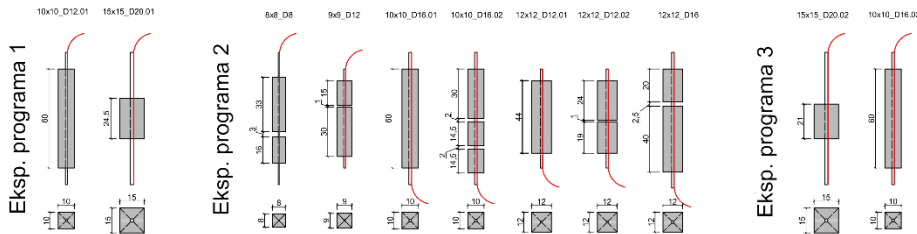
Pirmajame skyriuje pateikiama trumpa tyrimų apžvalga apie gelžbetoninių konstrukcijų tinkamumo ribinį būvio analizės metodus ir DOFS taikymą. Ši apžvalga parodė, kad klasikiniuose konstrukcijų tinkamumo analizės metoduose taikomi pernelyg supaprastinti plieno ir betono sąveikos dėsniai. Gelžbetoninių konstrukcijų elgseną, veikiant eksploatacinei apkrovai, tikroviškai atspindi įtempių perdavimo metodas, kuris grindžiamas sukibimo įtempių – slinkties dėsniu. Manoma, kad norint tiksliai apibrėžti įtempių perdavimo parametrus, reikia nuodugniai ištirti gelžbetoninių konstrukcijų deformacijas. Dėl gaunamų rezultatų tikslumo (1 $\mu\epsilon$), matavimų greičio (250 Hz), ištisinio matavimų pobūdžio (matuojama taškuose kas 0,63 mm), nesudėtingo taikymo ir minimalaus invazinio poveikio (jutiklių skersmuo 125 μm) DOFS yra idealus deformacijų nustatymo įrankis, padedantis eksperimentiniu būdu identifikuoti įvairius parametrus.

2. Eksperimentiniai gelžbetoninių elementų tyrimai tinkamumo analizei

Antrojo skyriaus pradžioje aprašomi iššūkiai, su kuriais susidūrė šio darbo autorius, pradėjęs gelžbetoninių konstrukcijų deformacijų stebėseną, taikant DOFS. Iki tol DOFS kaip stebėsenos įrankis buvo tik kelis kartus taikytas civilinės inžinerijos srityje. Todėl autorius pirmiausia turėjo įvertinti DOFS optinių jutiklių perspektyvumą lyginant su įprastomis deformacijų stebėsenos priemonėmis, tokioms kaip tenzometriniai jutikliai ir ekstensometrai. Autoriaus atlikti preliminarūs deformacijų tyrimai skirtingais metodais parodė gerą rezultatų sutapimą, tad padaryta išvada, kad DOFS yra tinkamas deformacijų stebėsenos įrankis, kurio galimybės yra daug didesnės nei kitų metodų.

Pažymėtina, kad DOFS dažniausiai yra tvirtinami prie gelžbetoninių konstrukcijų paviršiaus. Autorius DOFS ėmė naudoti strypinės armatūros dformacijų matavimui. DOFS tvirtinimo prie armatūros strypo tvirtinimo būdas yra nesudėtingas ir beveik nepalieka pažeidimų armatūroje, be to, sudaro sąlygas nuolat ir įvairiose gelžbetoninių konstrukcijų vietose stebėti armatūros deformacijas per visą eksploatacijos laiką. Autorius išbandė skirtingus DOFS tvirtinimo prie armatūros būdus: sėkmingiausias rezultatas buvo gautas, DOFS tvirtinant nedidelėse armatūros strypo įpjovose, naudojant cianoakrilatą ir silikoną. Ši metodika sėkmingai pasiteisino, atlikus eilę eksperimentinių tyrimų. Darbo autorius atliko trijų skirtingų eksperimentinių programų tempiamųjų gelžbetoninių elementų tyrimus. Tyrimų tikslas - atlikti šių elementų armatūros deformacijų stebėseną naudojant DOFS. Eksperimentuose naudoti šie kintamieji

parametrai (S2.1 pav.): armavimo koeficientas, armatūros skersmuo ir elemento ilgis.



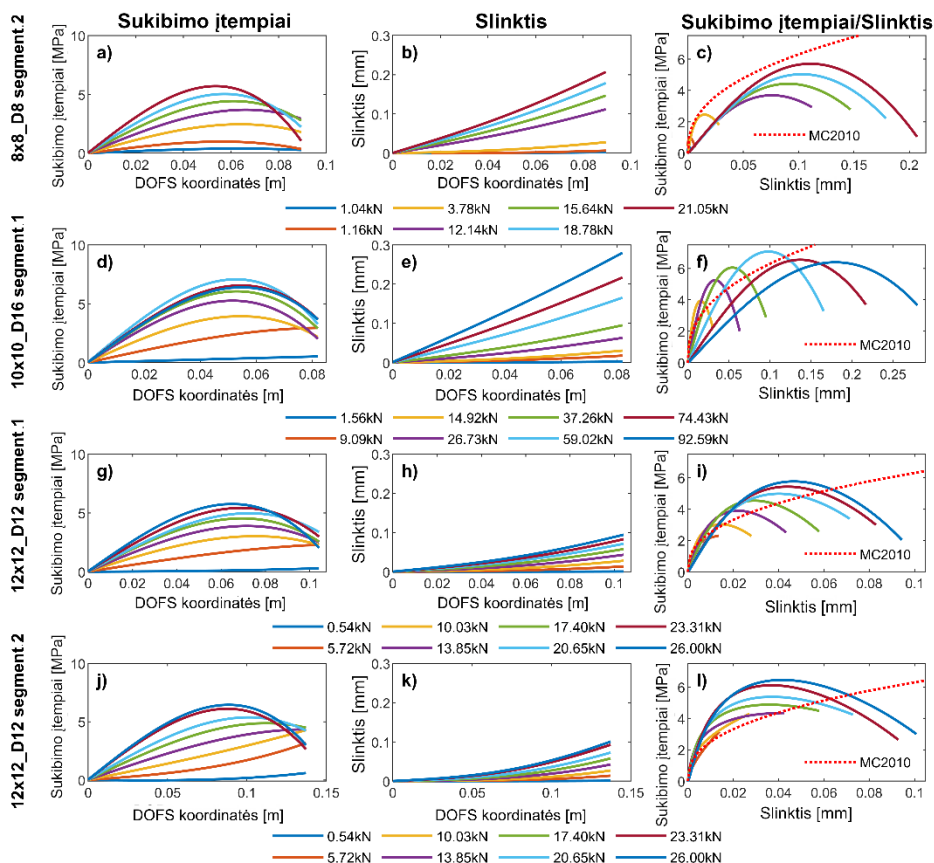
S2.1 pav. Eksperimentiniai tempiamieji gelžbetoniniai elementai, armatūros deformacijos matuojant DOFS

Šie eksperimentiniai tyrimai leido:

- bet kuriuo laiko momentu nustatyti armatūros ir betono deformacijų būvį.
- išanalizuoti optinių jutiklių tvirtinimo prie armatūros strypo būdus ir pagal mažiausios paklaidos kriterijų išrinkti geriausiąjį.
- įvertinti betono traukumo sukeltas deformacijas armatūroje ir pleišėjimo apkrovos sumažėjimą.
- gauti žinių apie betono ir armatūros kontakte atsirandančius sukibimo įtempius ir slinktį.
- išanalizuoti atstumus tarp plyšių.
- išanalizuoti plyšio plotį.

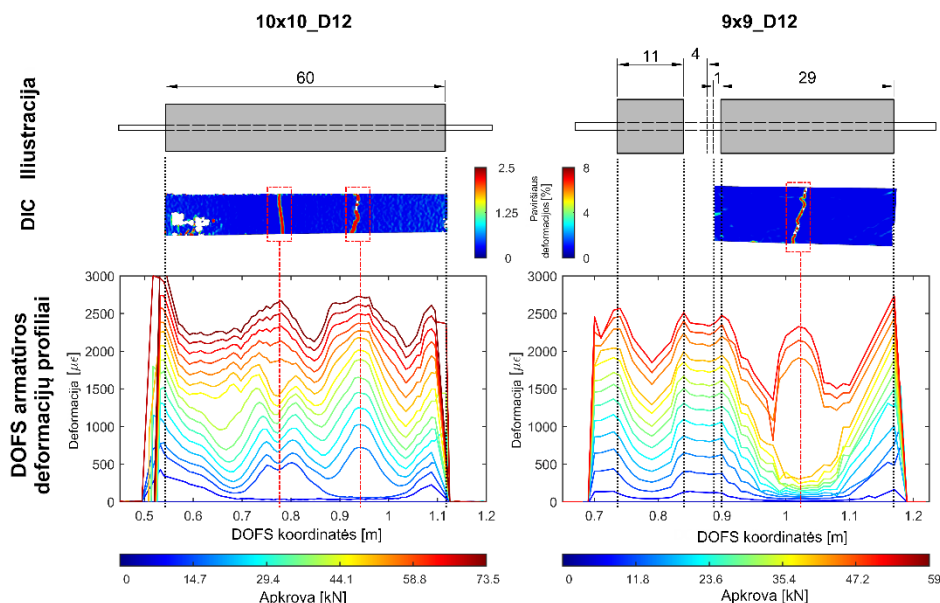
3. Ištisinio deformacijų matavimo taikymas, gežbetoninių konstrukcijų tinkamumo analizės modelius bei tikrinant esamų modelių adekvatumą

Vienas iš šio darbo uždavinių – taikant DOFS eksperimentiškai nustatyti gelžbetoninių konstrukcijų tempiamųjų elementų su plyšiais ir be plyšių armatūros deformacijas. Pasitelkus deformacijų nuskaitymą ir įvairaus laipsnio polinomų aproksimacijas bei apdorojus eksperimentinius duomenis, gautos armatūros deformacijų diagramos skirtingoms apkrovos pakopoms (žr. S3.1 pav.). Iš armatūros deformacijų diagramų buvo nustatomos sukibimo įtempių - slinkties priklausomybės. DOFS buvo tiksliai išmatuotos armatūros deformacijos nesupleišėjusiuose ir supleišėjusiuose tempiamuose gelžbetoniniuose elementuose įvairiuose apkrovos lygiuose iki plienui pasiekiant takumo įtempius.



S3.1 pav. Sukibimo įtempiai, slinktis ir sukibimo įtempių / slinkties diagramos, paskaičiuotos pagal eksperimentinius armatūros deformacijų profilius

Bandymai buvo sėkmingi tiek didesnio ($\varnothing 16$), tiek mažesnio ($\varnothing 8$) armatūros skersmens atveju. Buvo nustatyta, kad ir DOFS, ir DIC (skaitmeninės vaizdo koreliacijos metodas) vienodai gerai nustatė plyšių vietas. Tai rodo, kad taikant DOFS ir DIC derinį, galima gauti tikslus ir informatyvius įtempių - deformacijų bei plyšių parametrų rezultatus (žr. S3.2 pav.). Atskiriems tempiamiesiems elementams skirtingose apkrovos pakopose buvo nustatytos sukibimo įtempių diagramų vidutinės reikšmės. Buvo gautas ryšys tarp vidutinių sukibimo įtempių ir armatūros koeficiento. Šis vidutinių sukibimo įtempių dėsnis - vienas svarbiausių šio tyrimo rezultatų. Taip pat buvo gauta, kad vidutiniai sukibimo įtempiai priklauso ir nuo gelžbetoninio elemento ilgio (kai elementas nepleišėja) arba atstumo tarp plyšių.

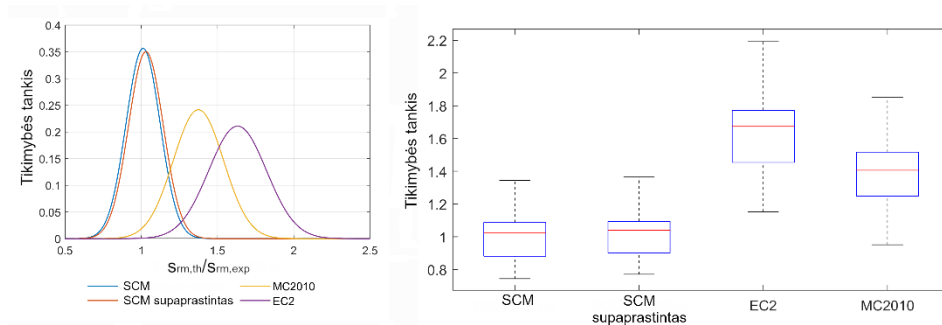


S3.2 pav. Tempiamųjų gelžbetoninių elementų plyšių identifikavimas DIC ir armatūros deformacijų diagramų gavimas taikant DOFS

Darbe pademonstruotas DOFS potencialas, tiriant gelžbetoninių konstrukcijų atstumus tarp plyšių bei tobulinant deformacijų darnos modelį (SCM). Disertacinio darbo vadovas ši analitinį modelį pasiūlė lenkiamiesiems elementams, o šiame darbe modelis pritaikytas tempiamiesiems elementams. Pastarasis modelis remiasi kokybiškai nauja koncepcija, kurios esmė yra ta, kad vidutinis atstumas tarp plyšių nustatomas iš tempiamosios armatūros deformacijų profilio, taikant dviejų skirtingų deformacijų analizės metodų (vidutinių deformacijų ir diskrečių plyšių) sinergiją. Deformacijų darnos metodas pagrįstas vidutinių armatūros deformacijų lygybe, taikant šias dvi skaičiavimo koncepcijas. Priimama, kad deformacijos kinta tiesiškai atskirose efektyvaus sukibimo (efektyvioji zona) ir pakitusio sukibimo (pažeidimo ir centrinė) zonose. Šlyties įtempiai tarp armatūros ir betono perduodami tik efektyviojoje zonoje, o likusiose zonose šlyties įtempiai priimami lygūs nuliui. Modelyje priimtas armatūros deformacijos profilis yra laužytos tiesinės formos, išskiriant minėtas tris zonas, turinčias skirtingus sukibimo įmpius: pažeidimo, efektyvioji ir centrinė zonos. Jų bendras ilgis yra lygus vidutiniam atstumui tarp plyšių.

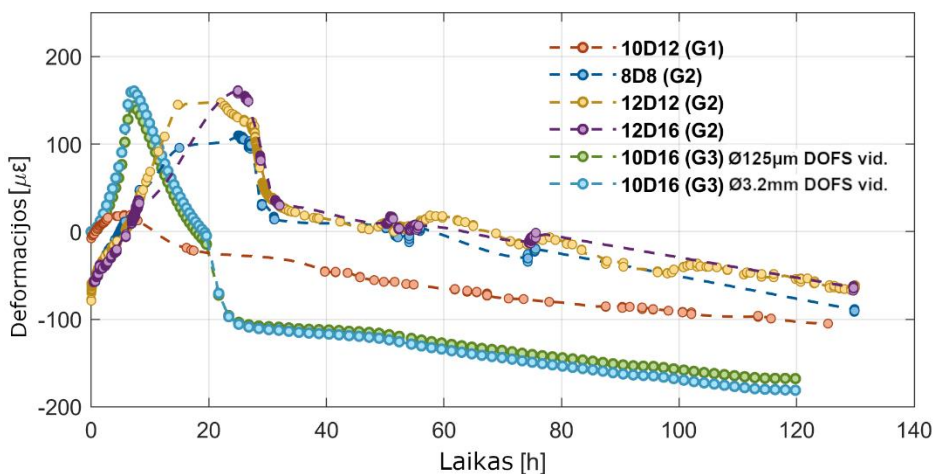
DOFS eksperimentiškai nustačius tempiamųjų gelžbetoninių elementų armatūros deformacijos profilius, buvo parodyta, kad SCM modelio prielaidos yra teisingos. Modelyje priimtas armatūros deformacijų profilis gerai atitiko

eksperimentiškai gautuosius deformacijų profilius. Patikslinus zonų parametrus pagal DOFS eksperimentinius rezultatus, buvo sukurta nauja SCM atstumo tarp plyšių modelio versija. Šis modelis buvo palygintas su MC2010 ir EC2 metodais, atstumus tarp plyšių skaičiuojant kaip normalizuotas jų vertes ($S_{rm,th}/S_{rm,exp}$), kur $S_{rm,th}$ ir $S_{rm,exp}$ yra apskaičiuotas ir eksperimentinis atstumas tarp plyšių. Atstumo tarp plyšių prognozė taikant deformacijų darnos metodą (SCM) buvo tikslesnė nei minėtų projektavimo normų. SCM metodu nustatytos vidurkio reikšmės buvo daug artimesnės 1, o standartinė paklaida - mažesnė. Deformacijų darnos modeliu gautos vidurkio reikšmės, imant trijų armatūros deformacijų zonų modelį bei supaprastintą dviejų zonų modelį, buvo atitinkamai lygios 1,0117 ir 1,0305, o MC2010 ir EC2 - atitinkamai 1,3745 ir 1,6315 (žr. S3.3 pav.). Rezultatų sklaida SCM metodu taip pat buvo mažesnė: standartiniai nuokrypiai dviejų ir trijų zonų modeliui buvo atitinkamai lygūs 0,1118 ir 0,1138, tuo tarpu MC2010 ir EC2 - atitinkamai 0,1650 ir 0,1891.



S3.3 pav. Tempiamųjų gelžbetoninių elementų atstumo tarp plyšių prognozavimo EC2, MC2010 ir pasiūlytuoju deformacijų darnos modeliu (CSM) statistinių rodiklių grafinė iliustracija: normalinis pasiskirstymas ir standartinis nuokrypis

Paskutinė trečiojo skyriaus dalis skirta betono traukimo tyrimams taikant DOFS. Optinius sensorius fiksuojant prie armatūros strypo, gauti nauji eksperimentinių tyrimų rezultatai apie armatūros deformacijų ir įrąžų kitimą laike, traukiantis betonui. Buvo parodyta, kad ankstyvojoje cemento teslos rišimosi stadijoje dėl hidratacijos proceso ir jį lydinčio šilumos poveikio armatūroje pasireiškė teigiamos deformacijos. Vėliau, atslūgus temperatūrai ir betonui kietėjant ir traukiantis, armatūra buvo suspaudžiama, t.y. joje atsiradavo gniuždymo deformacijos (žr. S3.4 pav.). DOFS gauti eksperimentiniai duomenys taip pat leido pagrįsti naują tempiamojo sustandėjimo modelį, kuriame eliminuota betono traukimo įtaka. Šis modelis buvo gautas, taikant literatūroje paskelbtų tempiamųjų gelžbetoninių elementų eksperimentinius apkrovos - deformacijų diagramas.



S3.4 pav. DOFS pamatuotos tempiamųjų gelžbetoninių elementų armatūros deformacijos, sukeltos betono traukumo

Bendrosios išvados

Išanalizavus mokslinę literatūrą gelžbetoninių konstrukcijų tinkamumo (pleišėjimo ir deformacijų) klausimais, galima teigti, kad:

1. Palyginus gelžbetoninių konstrukcijų tinkamumo analizei naudojamus vidutinių deformacijų ir įtempių perdavimo metodus, akivaizdu, kad pastarasis yra pranašesnis dėl realistiško mechaninės elgsenos (sukibimo įtempių – slinkties, tempiamojo sustandėjimo, diskretaus pleišėjimo pobūdžio) atspindėjimo.
2. Norint nustatyti svarbiausius įtempių perdavimo metodo parametrus (armatūros ir betono sukibimo įtempius ir slinktį), būtina turėti tikslus eksperimentinius armatūros strypų deformacijų pasiskirstymo visame elemento ilgyje duomenis. Toks tyrimas tik neseniai tapo įmanomu, statybos konstrukcijų bandymuose pradėjus taikyti ištisinio matavimo optinio pluošto jutiklius (DOFS). Dėl gaunamų rezultatų tikslumo (1 $\mu\epsilon$), matavimų greičio (250 Hz), ištisinio matavimų pobūdžio (matuojama taškuose kas 0,63 mm), nesudėtingo taikymo ir minimalaus invazinio poveikio (jutiklių skersmuo 125 μm) DOFS yra idealus armatūros deformacijų nustatymo įrankis.

Atlikus eksperimentinius tempiamųjų gelžbetoninių elementų armatūros deformacijų tyrimus, naudojant optinius jutiklius, atskleista, kad:

3. Iki autoriui pradedant tyrimus, nebuvo standartinės optinių jutiklių tvirtinimo prie armatūros strypo metodikos, kuri apsaugotų jutiklį nuo

trinties su betonu poveikio. Išbandęs įvairius DOFS tvirtinimo prie armatūros strypo būdus, kaip efektyviausią autorius pasirinko metodą, kuriame jutiklis tvirtinamas nedidelėje armatūros strypo įpjovoje, naudojant cianoakrilatą ir silikoną.

4. Atliktieji supleišėjusių ir nesupleišėjusių tempiamųjų gelžbetoninių elementų eksperimentiniai tyrimai pademonstravo, kad optiniais jutikliais galima gauti tokius svarbius rezultatus tinkamumo ribinio būvio analizei: (a) armatūros ir betono įtempių ir deformacijų būvį bet kuriame apkrovos lygyje (net ir viršijus takumo ribą); (b) betono traukumo sukeltus armatūros ir betono įtempius bei skerspjūvio pleišėjimo atsparumo sumažėjimą; (c) armatūros ir betono sukibimo įtempių ir slinkties parametrus; (d) atstumus tarp plyšių; (e) plyšio plotį.

Paminėtus eksperimentinius tyrimo rezultatus panaudojus gelžbetoninių elementų tinkamumo analizei atlikti, gauta, kad:

5. Vidutiniai armatūros ir betono sukibimo įtempiai tempiamuose gelžbetoniniuose elementuose priklauso nuo armavimo koeficiento ir elemento bloko, kurį apriboja du gretimi plyšiai, ilgio. Remiantis šiais rezultatais, pasiūlytas naujas vidutinių sukibimo įtempių modelis, priklausantis nuo armavimo koeficiento.
6. Didelei eksperimentinių duomenų imčiai atlikta tempiamųjų geležbetoninių elementų atstumo tarp plyšių prognozė patikslintuoju deformacijų darnos modeliu (SCM) buvo tikslesnė nei plačiai pasaulyje taikomais Model Code 2010 ir Eurocode 2 metodais. SCM metodu nustatytos vidurkio (teorija/eksperimentas) reikšmės buvo daug artimesnės 1, o standartinė paklaida - mažesnė. Deformacijų darnos modeliu gauta vidurkio reikšmė - 1,0117, o MC2010 ir EC2 - atitinkamai 1,3745 ir 1,6315. Standartinis nuokrypis šiems metodams – atitinkamai 0,112, 0,165 ir 0,189.
7. Optiniais jutikliais įvertinus betono traukumo poveikį armatūros deformacijoms, buvo parodyta, kad darbe pasiūlytasis tempiamojo sustandėjimo modelis, susietas su MC2010 ir EC2 traukumo metodais, tiksliai įvertina tempiamųjų gelžbetoninių elementų deformacijas.
8. Optiniai jutikliai leidžia įvertinti, kokia dalimi dėl betono traukumo sumažėja gelžbetoninių elementų atsparumas pleišėjimui.

Apibendrinant galima teigti, kad DOFS yra pažangiausias gelžbetoninių konstrukcijų deformacijų stebėsenos įrankis. Šis tyrimas parodo, kad taikant DOFS galima patvirtinti, atmesti arba patobulinti gelžbetoninių konstrukcijų tinkamumo modelius. Šio tyrimo rezultatai gali būti laikomi pirmuoju žingsniu, naujai pažvelgiant į gelžbetoninių konstrukcijų elgseną ir tinkamumo modelius.

Annexes¹

- Annex A.** Database for the construction of parameter l_c
- Annex B.** Database of RC ties for the comparison between experimental RC ties' crack spacings and the SCM extracted crack spacing predicted values
- Annex C.** Declaration of academic integrity
- Annex D.** The co-authors' agreements to present publications material in the doctoral dissertation defense
- Annex E.** Copies of scientific publications by the author on the topic of the dissertation

¹The annexes are supplied in the enclosed compact disc.

Mattia Francesco BADO

DISTRIBUTED SENSING FOR SERVICEABILITY
ANALYSIS OF REINFORCED CONCRETE
STRUCTURES

Doctoral Dissertation

Technological Sciences,
Civil Engineering (T 002)

IŠTISINIO DEFORMACIJŲ MATAVIMO
TAIKYMAS GELŽBETONINIŲ KONSTRUKCIJŲ
TINKAMUMO ANALIZEI

Daktaro disertacija

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