

Performance of RC Columns Retrofitted with UHPFRC Jackets Under Cyclic Displacement Reversals

by

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Abstract

It is the purpose of this study to evaluate, through combined experimental and analytical research, the performance of reinforced concrete columns with or without prior damage, after retrofitting with ultra-high-performance fiber-reinforced concrete (UHPFRC) jackets and subjected to cyclic loading that simulates earthquake effects. A series of conventionally designed RC column elements were constructed, based on detailing methodologies of older code standards. Preliminary damage was imposed, simulating the effects of previous seismic excitation on the columns, and the plastic hinge zone was repaired through cover replacement with a thin commercial-grade UHPFRC jacket, as well as bond breakers placed on the starter bars of the lap-spliced connection. Through experimental testing, the behavior of the columns was studied under reversed lateral cyclic displacements with the objective to obtain quantified evidence of the performance of the retrofit methodology. The UHPFRC-retrofitted columns demonstrated substantial improvement in strength and ductility, over the pre-damaged columns, where the repaired and strengthened specimens showed similar failure modes and comparable responses. Detailed FEA simulations of the experimental program were conducted, where cyclic performance of the UHPFRC-retrofitted models was defined in accordance with the pertinent experimental data, and the broader distribution of reinforcement strains was validated.

Companion small-scale material characterization experiments were prepared, to obtain the constitutive hysteretic properties of concrete confined with UHPFRC jackets, under axial monotonic and cyclic loading regimes, where the remarkable enhancement of the ductile response of the encased concrete was illustrated. Analytical models based on classical mechanics were applied, and the constitutive material laws were formulated, based on parameter calibration with the results collected from the experimental component.

An extensive database on bond experiments from the literature was assembled, including specimens fabricated with UHPFRC and engineered cementitious composites (ECC). The database reveals the existing trend of discrepancy in the experimentally measured bond, dependent on specimen geometry. The results of a combined experimental study, containing direct tension pullout (DTP) and anchored four-point bending (AFPB) using UHPFRC-fabricated specimens confirm the passive contribution of curvature in bond strength, and parametric sensitivity FEA was

conducted, where the spurious lateral effects occurring on the various bond test setups were captured.

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1 Introduction

1.1 Background

Structural performance of concrete structures under the influence of seismic excitation has received substantial attention since the 1980's. Damage is an intrinsic attribute of seismic design: expected controlled damage through ductility in the design earthquake characterizes modern seismic resistant structures. However, older structures, with insufficient detailing in accordance with former design practices, develop brittle patterns of damage when subjected to severe ground motions, often with non-reversible, and disproportional consequences for the entire structure. Therefore, retrofitting and repairing such structures to improve their prospective performance in a future event is and has been, for a long time, a priority in research and practice of Structural Engineers, particularly in light of the vast inventory of such structures in the urban centers in regions of high seismicity around the world.

The need to mitigate catastrophic damage, and to increase the safety of existing vulnerable structures has spearheaded and motivated innovation in material technologies as well as in the development of novel retrofitting methods over a very broad spectrum of applications. In the period 2000-2010 the emergence of fiber-reinforced polymers (FRPs) dominated the field of seismic retrofit; since 2010, however, several issues related to the unsatisfactory performance of FRP-encased concrete under conditions that induced environmental degradation and propelled the exploration of alternative solutions. Following the advancement of novel technologies in the field of strain hardening cementitious materials such as engineered cementitious composites (ECC) and ultra-high-performance fiber-reinforced concrete (UHPFRCs), and the emergence of Code standards and design criteria have enabled exploitation of their special properties in seismic resistant construction. Although UHPFRCs were originally used primarily for deck repairs through overlays for in situ casting of joints in precast panels recently their use in the development of engineering retrofitting solutions towards improved and durable seismic response of vulnerable or previously damaged structures are becoming increasingly attainable. Combined with the extensive feedback from past reconnaissance reports from major earthquakes in the last 30 years which have

put to the test the older engineered structures, great efforts have been made towards improved structural resistance specifications to protect new construction against severe lateral loading such as earthquake action. Cyclic loading reversals under displacement control have been used extensively by researchers as a means of simulating seismic excitation to investigate damage in lateral load resisting structural elements. Cyclic performance is evaluated based on a multitude of technical criteria among which, driving considerations are the protection against extreme lateral deformations, and the development of ductile response through development of plasticity in inherently deformable components of the structures and, eventually, avoidance of collapse.

The response of reinforced concrete (RC) members to cyclic displacement reversals is affected by an array of factors including the system formation, the material properties and the buildup of hysteretic damage, the reinforcement detailing, the quality and condition of the exposed materials, the loading history, and the overall member geometry (e.g., aspect ratio). Due to the inherent limitations of Normal Strength Concrete (NSC), in the absence of systematic detailing or after degradation due to environmental exposure, the seismic performance of older, (pre-1980's) RC members is often hampered by inevitably low margin, premature failures, in which members fail before the attainment of their full ductile response.

An effective means of prolonging the service life of existing structures and ensuring compliance with modern seismic codes is the retrofitting and rehabilitation of older construction. By this approach, weakened members can be reverted to operational capacity, via partial or complete strengthening of damaged subcomponents. Retrofitting is classified by the effective implications it has on the dynamic characteristics and deformation capacity of the structural system. If the retrofit alters the structural stiffness as a whole (reducing the structural period) then it not only increases the strength of the system, but it also reduces the deformation demand. Retrofit schemes that affect both sides of the design inequality, reducing the demand and increasing the capacity are known as global interventions (Thermou and Elnashai 2006). Reinforced Concrete or Steel belong to this category. Local interventions are retrofitting solutions that only enhance the deformation capacity without altering the demand – FRP jacketing is a characteristic example of this type of retrofit.

Within the scope of the present study, retrofitting by jacketing is investigated in the context of (UHPFRCs), where very thin jacketing solutions are possible in the form of cover replacement, or

in partial replacement of the plastic hinge (PH) region of columns. The study focusses in column specimens that are representative of older buildings. In the experimental program the performance of the retrofit method is explored by conducting tests under simulated earthquake effects.

1.1.1 Ultra High Performance Fiber Reinforced Concrete (UHPFRC)

Ultra-high-performance fiber-reinforced concrete (UHPFRC, or UHPC for short), is a relatively novel branch of the strain-hardening cementitious composite (SHCC) category of structural materials, and directly descending from a combination of Reactive Concrete Technologies and the emergence of Fiber Reinforced Engineered Cementitious Composites (ECC) (Naaman and Reinhardt 2006; Graybeal and Tanesi 2007; Toutlemonde et al. 2018; Azmee and Shafiq 2018; Saikali 2019). These materials employ a dense particle matrix (max. aggregate size $< 2\text{-}3\text{ mm}$, but most often less than 1 mm), along with an array of supplementary binder materials such as silica fume and slag, among others, to improve particle packing density and compressive strength. Chemical admixtures and superplasticizers are added to imbue certain properties to the mixture, such as flowability and resistance to environmental agents. However, the most characteristic property of these composites is the addition of mass reinforcement in the form of fibers, which is distributed throughout the cementitious matrix. The incorporation of fiber reinforcement, (steel, synthetic, carbon or wood) as part of the composite enables the passive confinement of the concrete core, due to the tensile resistance of the fibers in particle separation and, by extension, the pseudo-ductile, strain-hardening response of the material. This behavior of UHPFRC sets a very attractive prospect for the enhancement of concrete as a structural material, as it eliminates the two inherent weaknesses of the conventional form: the low tensile capacity and brittleness.

Preliminary work on the definition of the mechanical characteristics has been conducted by Bache 1981; Richard and Cheyrezy 1995; Naaman and Reinhardt 1996; Chanvillard and Rigaud 2003; and by Graybeal 2006. UHPFRCs are typically classified as materials capable of developing compressive strengths greater than $120\text{-}150\text{ MPa}$, while tensile strength, measured against direct tension has to exceed $5\text{-}7\text{ MPa}$ (Resplendino and Toutlemonde 2014; FHWA HRT-06-103 2006/HRT-13-060 2013; AFNOR NF P18-470/P18-710 2016; SIA 2052:2016; CSA-S6 Annex 8 2019). Thanks to their high particle density, UHPFRC materials are additionally characterized with favorable qualities such as increased durability, impermeability to external chemical and environmental agents as well as reduced susceptibility to long term effects (Naaman and Reinhardt

2003; Graybeal and Tanesi 2007; Wille et al. 2011; Baby et al. 2013; Yu et al. 2014; Yoo et al. 2016; Azmee and Sha 2018). The reduced susceptibility to long term effects is owing to the limited occurrence of capillaries in the material microstructure, and is valuable in marine and coastal environments, where ingress and diffusion of oxidized ions accelerates the corrosion of exposed reinforcement. UHPFRC also forms strong chemical bonds with existing RC allowing for synergistic application of the former in older construction in the rehabilitation of degraded structural members.

An array of structural projects has already been completed using UHPFRC, involving complete structural entities, infrastructure, and architectural designs (Resplendino and Toutlemonde 2013; Azmee and Sha 2018). Throughout these projects, the unique composition of this material allows for significant reduction in required volume of material and for milder maintenance, as well as it alleviates reinforcement congestion, thanks to the participation of the fibers in the tensile response of the member. However, field application of UHPFRC in structures is still limited, in comparison to conventional structural materials, owing to the absence of established standards and dissemination of the *modus operandi* for preparation and use in the industry. Code provisions are currently in development from Technical Committees and Task Groups, such as the French Standards AFNOR NF P18-470/P18-710, the Swiss Standards SIA 2052:2016, the Canadian Standards Annex U of CSA-A23.1 & Annex 8.1 of CSA-S6, and the American Standards ACI 544 and ACI 239, for fiber-reinforced concrete and UHPFRC, respectively, FHWA-HRT-13-060, FHWA-HRT-18-030, FHWA-HRT-19-011, AASHTO (2020), and ASTM C1856 / C1856M-17.

1.1.2 Structural Retrofit and Rehabilitation of Older Construction

Use of established standards for the protection against earthquake damage is one of the major considerations in RC design. Provisions have been improved in newer versions of engineering codes, such as the European standards EN 1998-1, with seismic design shifting towards reinforcing targeted primary members and structural systems by linking design considerations to displacement-based objectives (performance-based design) instead of the traditional force-based approach (capacity design). This allows for the attainment of more economically viable and effective seismic resisting systems, where deformation capacity becomes the controlling property of the members. As a requirement of the above, the structural integrity of RC members becomes critical in the presence of large plastic deformations and cyclic load reversals. Conventional

concrete, if lightly reinforced, exhibits brittle and weak behavior in such phenomena particularly, due to the inherently low fracture energy and inability to develop tensile deformations, limiting the efficiency of RC members under seismic excitation.

With a large fraction of the existing construction approaching or having exceeded its service life limit, intervention techniques to restore structural capacity and preserve functionality become pivotal, not only for protection of the population but also for sustainability – prolonging the use of the built inventory of structures.

Jacketing has prevailed as one of the most popular retrofit methods. The advantage of this method is that it can be implemented with minimal interference to the operation of the structural system, while member properties and geometry are preserved. Jacketing with steel is capable of restoring stiffness and strength to the pre-damaged state, whereas FRP jackets preclude premature modes of failure and can only enhance the deformation capacity of the member, however, the latter are susceptible to stress concentrations that may lead to sudden jacket rupture and longitudinal bar buckling, while at the same time offering no protection in the event of fire. Steel jackets, on the other hand, are a very expensive solution as they require perfect matching of the prefabricated jacket with the encased structural element and seam-welds along the member length which also affect the reliability in field applications – this is why this type of solution has only been used in high budget retrofits, such as bridge piers in the Northridge area after the 1999 earthquake (Priestley et al. 1996).

The use of concrete jackets is a natural alternative, more suitable for seismic retrofitting of concrete and masonry structures due to material compatibility and synergy with the encased concrete, maintaining structural integrity and restoring ductility (Lampropoulos et al. 2012; Farzad et al. 2019). This method has commonly been implemented with steel reinforced NSC; however, it is not aesthetically optimal (as the usual jacket thickness must be more than 70 mm to accommodate longitudinal and transverse reinforcement leading to significant change of geometry and area of the structural member), nor is it easy to implement: being a global intervention, the longitudinal reinforcement is expected to continue from floor to floor through perforations made in the floor slab adjacent to the beam column joint. The introduction of novel structural materials, such as UHPFRC and ECC, which exhibit superior mechanical properties and post-cracking ductility, mitigate the problem of section enlargement because it has been found that very small amounts

(thin jackets) perform very well; at the same time, they seem to be a hybrid solution to conventional jacketing between global and local intervention. Being moderately invasive and not requiring addition of reinforcement – they also combine the benefits of concrete jacketing in terms of material compatibility and the facility of FRP as a local means of enhancement of member deformation capacity. Existing studies have reported complete restoration, as well as enhancement, of the flexural and shear stiffness of impaired RC members retrofitted with UHPFRC and ECC jackets, which will be discussed in Chapter 2 of the thesis.

1.2 Objectives of Research

It was the purpose of this study to evaluate, through a combined experimental and analytical research framework, the performance of UHPFRC jackets under cyclic loading. This was achieved through an experimental component, where a series of conventionally designed RC vertical column elements were constructed, based on detailing methodologies of older code standards. The effect of damage history was studied by applying preliminary damage, simulating the after-effects of seismic excitation, and repairing the plastic hinge zone with a commercial-grade UHPFRC jacket. Through the tests, the behavior of the columns under reversed lateral cyclic displacements after retrofitting was investigated, with the objective to obtain quantified evidence of the performance of the retrofit methodology.

The second objective was to obtain quantified evidence of the bond of anchorage in UHPFRC matrices was through comparison of experimental studies, where the contribution of specimen form to bond strength was considered. Results from two parallel experimental studies were described and a database of existing experimental studies on bond behavior of UHPFRC-embedded anchorage was assembled to obtain evidence of the sensitivity of measured bond across the various experimental setups. Parametric sensitivity analysis was conducted using 3D FEA modeling of the various bond test setups, using to capture the spurious lateral phenomena acting on the anchorage.

Thirdly, companion material characterization experiments were performed, where the confining effectiveness of thin UHPFRC jackets on the compression behavior of encased concrete was studied, with parameters including the jacket thickness, loading history, and encased concrete cross section shape. The constitutive properties of UHPFRC-confined concrete were determined based

on the experimental response and analytical equations were formulated to describe the compressive stress-strain law of the material.

Lastly, numerical simulation of the cyclic behavior of the UHPFRC-repaired and strengthened column specimens was performed using detailed 3D FEA on the combined ATENA/GiD platform, where model input was validated across the obtained experimental data. Objective of the analytical component was to expand the evidence of the experimental database in order to completely characterize the contribution of the retrofitting methodology on strength and deformation capacity of the repaired columns with reference to the important design parameters of the retrofit.

1.3 Thesis Contents

Each chapter of the manuscript corresponds to the individual experimental and analytical procedures conducted in this thesis. The scope and objectives of the research are described in Chapter 1, along with an overview of the thesis contents.

In Chapter 2, a literature review on seismic applications of UHPFRC materials is conducted, and the current state-of-the-art is discussed. Previously done studies including the use of UHPFRC jackets for retrofit and repair of cyclically loaded columns are discussed, as well as topics regarding the confinement effect on NSC imparted by UHPFRC. Analytical models on the cyclic behavior of UHPFRC, as well as the cyclic compressive behavior of confined concrete, are shown in this chapter.

Chapter 3 addresses the influence of bond-behavior in the synergistic performance of embedded steel reinforcement and UHPFRC jacket. An extensive database of experimental setups on bond is assembled, particularly on tests including HPFRC and ECC materials, and the importance of test setup selection is discussed. Detained numerical solutions via FEA simulations are provided, where the effect of the passive lateral stresses imparted by the fibers, on bond performance of embedded reinforcement is validated.

Chapter 4 contains the experimental investigation of confinement of UHPFRC jackets on NSC cores. As part of this investigation, several cylindrical and rectangular NSC specimens were confined with UHPFRC jackets of two thicknesses and tested under monotonic and cyclic

compressive axial load. Results are gathered using externally attached instrumentation and digital image correlation (DIC) analysis.

Chapter 5 contains the analytical portion of the investigation of the confinement effect of UHPFRC jackets, where analytical interpretations are used in correlation to the obtained experimental data of Chapter 4. The analytical equations used were adapted from existing formulations originally developed for conventionally confined concrete, and the parameters are updated based on the experiments of this thesis.

Chapter 6 describes the large-scale testing of RC column specimens, which are designed to be retrofitted with UHPFRC jackets. In this chapter, the procedure to conduct testing on the non-retrofitted specimens is described, including the preparation of the lab setup and necessary equipment, and results are reported regarding the performance of the columns under combined axial loading and cyclic lateral displacement reversals. The fabricated column specimens included reinforcement detailing from older standards of seismic design, to simulate the detailing conditions encountered in existing older construction.

Chapter 7 contains the second part of the large-scale experimental procedure, where the RC column specimens are retrofitted using UHPFRC jackets as cover replacement. Two retrofitting scenarios are included, one for repair and one for strengthening, containing four pre-damaged specimens as tested in the previous chapter for the first and two intact specimens for the second. The test arrangement and loading procedure was identical to the pre-damaging phase, with loading being applied to larger displacement. Results are obtained using the same external instrumentation and the undamaged strain gauges of the previous chapter.

Chapter 8 describes the numerical simulation of the UHPFRC-retrofitted column specimens of this study, using detailed 3D FEA with the combined ATENA/GiD platform. The procedure of the model assembly is shown, as well as the material model properties used. Results are compared with the experimental evidence and validated. The strain distributions on the concrete elements, as well as the longitudinal strains of the embedded reinforcement elements, are displayed and conclusions are drawn in reference to the experimental data.

Chapter 9 contains the main conclusions and outcomes of this study, as well as suggestions for future work and continuation of research to further developments of the studied concepts.

2 Literature Review

The application of UHPFRC in seismic retrofit and upgrading of older construction represents one of the most prominent potential uses of the material qualitatively determined by the material volume used as compared to the benefit in terms of deformation capacity and service life enhancement. To better understand the ability of UHPFRC to improve the strength of conventional structural members or to recover strength and ductility in damaged concrete, it is essential to fully understand the mechanical behavior and development of strain capacity of the material, both under monotonic and cyclic loading. Considering the novelty of the material, emphasis is placed on the establishment of code provisions that are accessible to practitioners and engineers in the field. In this section, the above topics will be discussed, based on the knowledge and findings available from published literature.

The improved cyclic performance and ductility capacity of ECC and UHPFRC column elements, over conventional RC elements, under combined shear-flexure and axial load action has been experimentally investigated in the literature (Fisher and Li 2003; Sun et al. 2015; Nozawa et al. 2017; Xu et al. 2017; Wu et al. 2017; Wang et al. 2018; Marchand et al. 2019; Ren et al. 2019; He and Deng 2019; Mahmoud et al. 2020). Prefabricated bridge column specimens containing UHPFRC segments in the plastic hinge (PH) region has also been investigated under simulated seismic loading, which showed reduced damage and higher lateral deformation capacity compared to RC columns (Zhang et al. 2019 ; Chan et al. 2020). Konstantinidis et al. (2017) replaced the pre-damaged concrete in the plastic hinge region of a pre-1970s designed column specimen using HPFRC in which they observed an increase in lateral strength and energy dissipation capacity after extending the jacket through the clear height. Post-tensioning combined with UHPFRC segmental construction of hollow-section bridge piers showed that the higher axial load capacity of the material enabled greater drift when tested under cyclic loading (Ichikawa et al. 2016; Wang et al. 2018). Comparison of column elements jacketed with fiber-reinforced polymers (FRP) and fiber-reinforced cementitious composites, exhibited similarities in cyclic performance, with an increase in load carrying capacity in the latter case (Zoppo et al. 2018). In addition, the composite application of FRP and ECC/UHPFRC jackets allowed for reduction of residual displacements

and elimination of interfacial bond failures (Fisher and Li 2003; Zohrevand and Mirmiran 2012). Finally, the monotonic performance of confined concrete cores, using reinforced/unreinforced HPFRC and UHPFRC jackets, showed improved strength and axial deformation capacity, in comparison to NSC specimens, which is optimized by the presence of transverse reinforcement (Rabehi et al. 2014; Wu et al. 2018; Gholampour et al. 2019; Fantilli et al. 2020; Dadvar et al. 2020; Malak et al. 2021).

2.1 Constitutive Properties of UHPFRC

The exceptional mechanical characteristics and durability of UHPFRC have been reported consistently throughout the literature (Naaman & Reinhardt 2003; Habel et al. 2006; Graybeal 2011; Yoo and Yoon 2016; Zhang et al. 2017). Although numerous experimental studies exist regarding the determination of core properties and constitutive behavior of the material, more hands-on evidence is required to establish acceptable and efficient design standards. Classification of the material properties and experimental practices still vary across countries (AFNOR NF P18-470/P18-710, SIA 2052:2016-F, Annex U of CSA-A23.1, ACI 239, FHWA-HRT-13-060) and compatibility between provisions of different Codes has yet to be established. It is important to ensure predictability and consistency of the material performance, especially in the application of UHPFRC in seismically resistant and repaired systems.

2.1.1 Classification & Material Laws

Naaman et al. (2006) proposed the classification of HPFRC composites to be determined by the combination of the following properties: 1) post-cracking tensile strength (f_{pc}), 2) minimum strain at peak tensile stress (ϵ_{pc}) and 3) the elastic modulus of elasticity (E_t). The generalized proposed analytical model for HPFRC contained a combination of seven points, that described the complete stress-strain response of the strain hardening fiber reinforced composite (SH-FRC); however, only the coordinates of two points, namely the cracking and ultimate, are necessary for classification (i.e., first cracking stress and peak stress). The material is classified with the nomenclature “Class T-”, where the dash is followed by the numbers 2.5, 5, 10, 15, 20 or 25 (in MPa), representing the peak tensile stress f_{pc} in each category. For all cases, it is required that the minimum values for ϵ_{pc} and E_t are 0.005 (0.5%) and 10.5GPa, respectively. To obtain the above characteristics, the

researchers recommend the use of direct tension tests on prismatic specimens of square or circular cross-sections.

Wille et al. 2014 used the dissipated energy per unit volume at the strain-hardening branch to categorize UHPFRC materials, using the level-based classification of post-peak behavior defined by Naaman et al. 2006. According to this classification, Level 3 energy dissipation corresponds to tensile strain-hardening materials and Level 4 to high-energy absorbing materials. Lower levels (1 and 2) correspond to categories of strain softening materials. Levels 3 and 4 are distinguished by the magnitude of energy dissipated during the hardening branch, with the latter exceeding 50 kJ/m³. This methodology is based on results obtained from direct tension tests (DTTs) on unnotched dog-bone specimens.

The French standards, AFNOR NF P18-470 (2016) and AFNOR NF P18-710 (2016), provide guidelines for material testing and design, respectively. Characterization of UHPFRC materials requires that the minimum compressive strength (f'_c) is 130 MPa and 150 MPa, for the former and latter case, respectively. In both standards, the tensile cracking strength (f_{cr}) of UHPFRC has to be a minimum of 6 MPa, and categorization of the tensile behavior is defined according to two parameters, the tensile cracking strength (f_{cr}) and the ratio between cracking strength and ultimate tensile strength (f_{tu}). The latter parameter can be referred to as the hardening ratio, symbolized here as γ_c for the characteristic curve and γ_m for the mean curve. In AFNOR NF P18-470 (2016), three classes of UHPFRC are distinguished: i) strain-softening (Class T1), where $\gamma_m, \gamma_c \leq 1.25$, ii) limited strain-hardening (Class T2) where $\gamma_c \geq 1.25$ and $\gamma_m \geq 1.25$, iii) significant strain-hardening (Class T3) where $\gamma_m, \gamma_c \geq 1.25$. In AFNOR NF P18-710 (2016), the hardening ratio is additionally divided by the fiber orientation factor K to account for the dispersion of fibers in directions other than the principal tensile direction. For Classes T1 and T2, AFNOR NF P18-470 (2016) recommends four-point bending tests (FPBTs) on unnotched prism specimens, to obtain the elastic properties, while three-point bending tests (TPBTs) on notched specimens combined with inverse analysis methods should be used to determine the post-cracking branch. For Class T3 UHPFRC, four-point bending tests on thin plate specimens are recommended, in the same standard, in combination with inverse analysis.

The Swiss standard, SIA 2052:2016-F, characterize UHPFRC as having compressive strength (f'_c) and tensile strength (f_t) greater than 120 MPa and 7 MPa, respectively. Based on the tensile

behavior, three classes are distinguished: i) strain-softening material (UO), ii) low-hardening (UA) and iii) improved strain-hardening material (UB). In addition to the cracking strength (f_{cr}) and hardening ratio (γ), this standard provides a criterion for the strain at the end of the hardening phase ($\varepsilon_{t,u}$). For the first two classes, a minimum cracking strength of 7.0 MPa is required, whereas for the latter 8.5 MPa is the minimum required limit. Hardening ratios (γ) are required to be 0.7, 1.1 and 1.2 for classes UO, UA and UB, respectively. For strain-hardening materials of the UA and UB classes, the minimum $\varepsilon_{t,u}$ strain is 0.15% and 0.25%, respectively. The above criteria correspond to the characteristic values obtained from FPBT on unnotched prisms or DDTs on dog-bone specimens.

The Canadian standards, Annex U of CSA-A23.1 and Annex 8.1 of CSA-S6, require that UHPFRC has a minimum compressive strength (f'_c) of 120 MPa. In tension, UHPFRC materials are categorized as either tension softening (TSFRC) or tension hardening (THFRC). For the former, the minimum tensile strength (f_{cr}) is 4.0 MPa, whereas it is 5.0 MPa for the latter. For THFRC, the hardening ratio γ is expected to be greater than 1.1 and the ultimate strain ($\varepsilon_{t,u}$) greater than 0.1. For TSFRC materials, the crack-opening branch is described by a minimum of six points corresponding to crack width $w = 0.15, 0.25, 0.50, 1.50, 2.50, \text{ and } 3.50$ (in mm). For THFRC materials, the same branch is described by a minimum of four points at locations corresponding to 80%, 60%, 40%, 20% of the peak tensile stress (f_{tu}). For both categories, the elastic branch is described by a minimum of three points, as per section U.4.1.2.2.8 of Annex U of CSA-A23.1, with the corresponding stress-strain diagrams being provided in accordance with Fig. 2.1 below. The above values are obtained from FPBTs on unnotched specimens combined with inverse method of analysis.

From American Standards, ACI 239-18 categorizes UHPFRC materials as having compressive strengths (f'_c) between 150-250 MPa and direct tensile strength (f_t) between 6-12 MPa. The elastic modulus (E_c) is defined based on ASTM C469/C469M, with values between 40-50 GPa. Strain hardening is characterized by the formation of multiple fine cracks prior to the attainment of the post-cracking stress σ_{pc} , as shown in Fig. 2.2, where crack localization and softening occur. The energy dissipation of UHPFRC is expected to be 50-100 times that of conventional concrete.

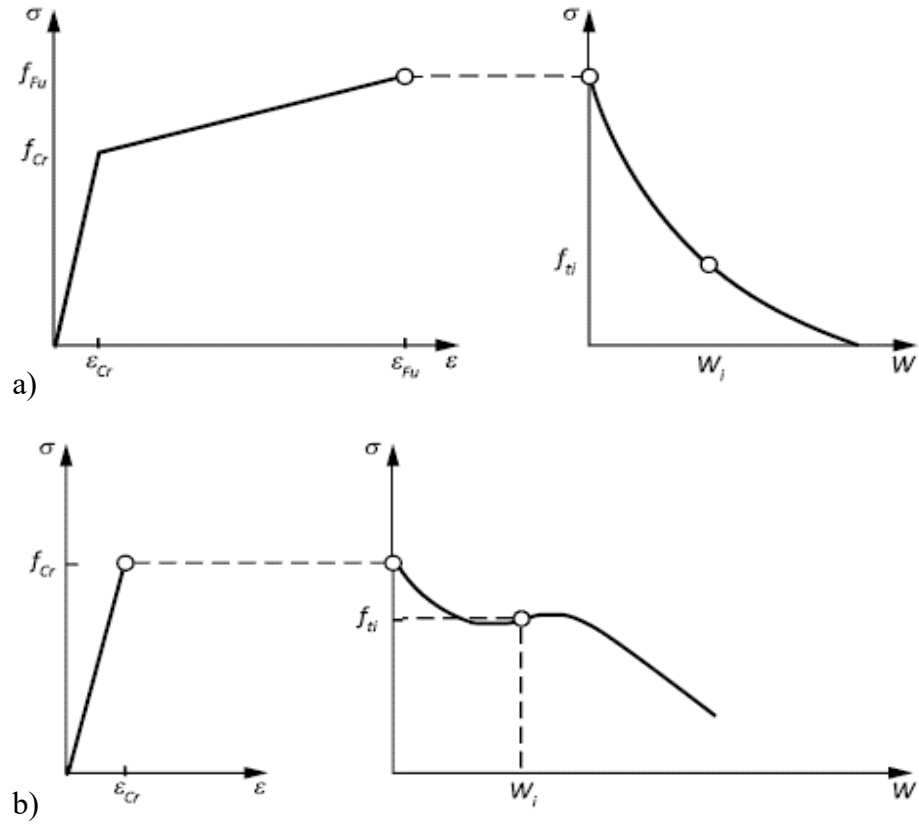


Figure 2.1. Tensile stress-strain diagrams for: a) THUFRC; b) TSUHPC (Figure U.3, obtained from Annex U of CSA-A23.1).

Based on the FHWA-HRT-13-060 report, an extensive experimental series involving approximately 1000 specimens showed that the compressive strength of the material is between 126-193 MPa. The tests followed the procedure denoted in ASTM C39, using cylindrical and cubic specimens of various sizes; it was shown that the measured strengths were within the 8% margin of the conventional cylinder size of 76 -x 152- mm. Test conducted to measure the first cracking strength (f_{cr}), showed that the expected strength was at 9 MPa, ranging from values between 6-11 MPa; specimens included flexural prisms, split cylinders, mortar briquettes and direct tension test of cylinders. The modulus of elasticity was measured according to ASTM C469, with values ranging from 42.7-51.9 GPa.

branch can be linearized (shown in dashed lines in Fig. 2.3) for simplicity. The strain at peak stress ε_{cc} and the modulus of elasticity are calculated based on Eqs. 2.2 and 2.3, respectively.

$$\sigma_a = f'_c \cdot \left[\frac{n \cdot (\varepsilon_a / \varepsilon_{co})}{n-1 + (\varepsilon_a / \varepsilon_{co})^{nk}} \right] \quad (2.1)$$

$$\varepsilon_{co} = f'_c / E_c \quad (2.2)$$

$$E_c = 4070 \sqrt{f'_c} \quad (2.3)$$

The internal confining pressure provided by the dispersed fiber reinforcement is considered by defining the internal confining stress $q_{f,lat}$, as shown in Eq. 2.4.

$$q_{f,lat} = \varphi_c \cdot f_{td} \quad (2.4)$$

Where φ_c is the material resistance factor used for high performance concrete in bridge design, denoted in Annex 8 of CSA-S6 at 0.75. The strain at the end of the plateau at peak stress is determined by strain magnitude $\varepsilon_{cc,o}$, which is obtained from Eq. 2.5 upon substitution of Eq. (2.4).

$$\varepsilon_{cc,o} = \varepsilon_{co} \left(1 + 6 \left(\frac{q_{f,lat}}{f'_c} \right) \right) \quad (2.5)$$

In the descending branch, a strain capacity ε_{cu} at 75% of the peak stress is calculated, considering the confined strength and strain enhancement of concrete, as per Eq. 2.6.

$$\varepsilon_{cu} = \varepsilon_{co} \left(1 + 20 \left(\frac{q_{f,lat}}{f'_c} \right) \right) \geq 0.004 \quad (2.6)$$

The descending branch extends linearly from the end of the plateau at peak stress and is intercepted the point defined by stress $0.75f'_c$ and strain ε_{cu} , until the residual branch is reached, at $0.2f'_c$; the last branch is sustained to large strains, for this model.

2.1.2 Cyclic Constitutive Laws for UHPFRC

Paschalis and Lampropoulos (2016) proposed a cyclic constitutive model for tension of UHPFRC, which was based on damage theory, and was determined from the experimental responses of DTTs on unnotched dog-bone specimens. The model comprises a total of five components: i) the linear-elastic branch, ii) the nonlinear hardening branch, iii) the nonlinear descending branch, iv) the unloading branch, and v) the reloading branch (Fig. 2.4). Table 2.1 summarizes the constitutive

equations for each branch; the notation has been adapted from the original publication to be consistent with the present study. Linearity is terminated at stress-strain coordinates $(f_{cr}, \varepsilon_{cr})$, whereas the peak stress is located at $(f_{fu}, \varepsilon_{fu})$. The initial tangent modulus of elasticity is symbolized by E_t . Following the peak stress, the model follows an exponentially descending branch, until the location of the first unloading cycle, namely $(f_{un}, \varepsilon_{un})$. In Eqs. 2.10-2.12, axial strain ε_{op} and stress f_{op} symbolize the corresponding values at any location on the descending branch. For the unloading branch (Eqs. 2.13-2.18), axial strain $\varepsilon_{un,0}$ is the residual strain, at the end of the branch, for zero stress, and $E_{un,0}$ is the corresponding modulus of elasticity. Finally, for the reloading branch, an exponential set of equations is adopted, as shown in Eqs. 2.19-2.21, where ε'_t and f'_t symbolize maximum strain and corresponding stress on the descending branch and E'_t is the initial tangent modulus of elasticity. Finally, strain ε'_{op} is the corresponding maximum strain for the tangent modulus of elasticity.

Table 2.1. Cyclic tension constitutive equations for tension of UHPFRC (Paschalis and Lampropoulos, 2016).

<p>Elastic branch ($\varepsilon \leq \varepsilon_{cr}$):</p> $\sigma = \varepsilon \cdot E_t \quad (2.7)$	<p>Descending branch ($\varepsilon > \varepsilon_{fu}$):</p> $\sigma = \left(B + C \cdot \varepsilon \cdot e^{\frac{(\varepsilon_{cr}-\varepsilon)}{\varepsilon_{fu}}} \right) \cdot E_t \quad (2.10)$
<p>Ascending branch ($\varepsilon_{cr} \leq \varepsilon \leq \varepsilon_{fu}$):</p> $\sigma = \varepsilon_{cr} \cdot (1 - A) + A \cdot \varepsilon \cdot e^{\frac{(\varepsilon_{cr}-\varepsilon)}{\varepsilon_{fu}}} \quad (2.8)$ $A = \frac{f_{cr}-\varepsilon_{cr} \cdot E_t}{E_t \cdot \left(\varepsilon_{fu} \cdot e^{\frac{(\varepsilon_{cr}-1)}{\varepsilon_{fu}}} - \varepsilon_{cr} \right)} \quad (2.9)$	$B = \frac{f_{op} \cdot \varepsilon_{fu} \cdot e^{\frac{(\varepsilon_{cr}-1)}{\varepsilon_{fu}}} - \varepsilon_{op} \cdot f_{fu} \cdot e^{\frac{\varepsilon_{cr}}{\varepsilon_{fu}} \left(1 - \frac{\varepsilon_{op}}{\varepsilon_{cr}} \right)}}{E_t \cdot \left(\varepsilon_{fu} \cdot e^{\frac{(\varepsilon_{cr}-1)}{\varepsilon_{fu}}} - \varepsilon_{op} \cdot e^{\frac{\varepsilon_{cr}}{\varepsilon_{fu}} \left(1 - \frac{\varepsilon_{op}}{\varepsilon_{cr}} \right)} \right)} \quad (2.11)$ $C = \frac{f_{fu}-f_{op}}{E_t \cdot \left(\varepsilon_{fu} \cdot e^{\frac{(\varepsilon_{cr}-1)}{\varepsilon_{fu}}} - \varepsilon_{op} \cdot e^{\frac{\varepsilon_{cr}}{\varepsilon_{fu}} \left(1 - \frac{\varepsilon_{op}}{\varepsilon_{cr}} \right)} \right)} \quad (2.12)$
<p>Unloading branch:</p> $\sigma = D \cdot e^{\frac{F \cdot (1 - \frac{\varepsilon - \varepsilon_{un,0}}{\varepsilon_{un} - \varepsilon_{un,0}})}{22}} \cdot E_t \cdot (\varepsilon - \varepsilon_{un,0}) \quad (2.13)$ $D = \frac{r \cdot (1 - \delta_{un})}{r - 1} \quad (2.14)$	<p>Reloading branch:</p> $\sigma = \left(\varepsilon'_t \cdot (1 - A') + A' \cdot \varepsilon \cdot e^{\frac{(\varepsilon'_{op}-\varepsilon)}{\varepsilon'_t}} \right) \cdot E'_t \quad (2.19)$

$$F = \ln\left(\frac{R \cdot (1 - \delta_{un}) \cdot (r - 1)}{r}\right) \quad (2.15)$$

$$\delta_{un} = 1 - \frac{B}{\varepsilon} - C \cdot e^{\frac{(\varepsilon_{cr} - \varepsilon)}{\varepsilon_{fu}}} \quad (2.16)$$

$$r = \varepsilon_{un} / \varepsilon_{un,0} \quad (2.17) \quad R = E_{un,0} / E_t \quad (2.18)$$

$$A' = \frac{f'_t - \varepsilon'_{op} \cdot E'_t}{E'_t \cdot \left(\varepsilon'_t \cdot e^{\left(\frac{\varepsilon'_{op}}{\varepsilon'_t} - 1 \right)} - \varepsilon'_{op} \right)} \quad (2.20)$$

$$\varepsilon'_t = \varepsilon - \varepsilon_{un,0} \quad (2.21)$$

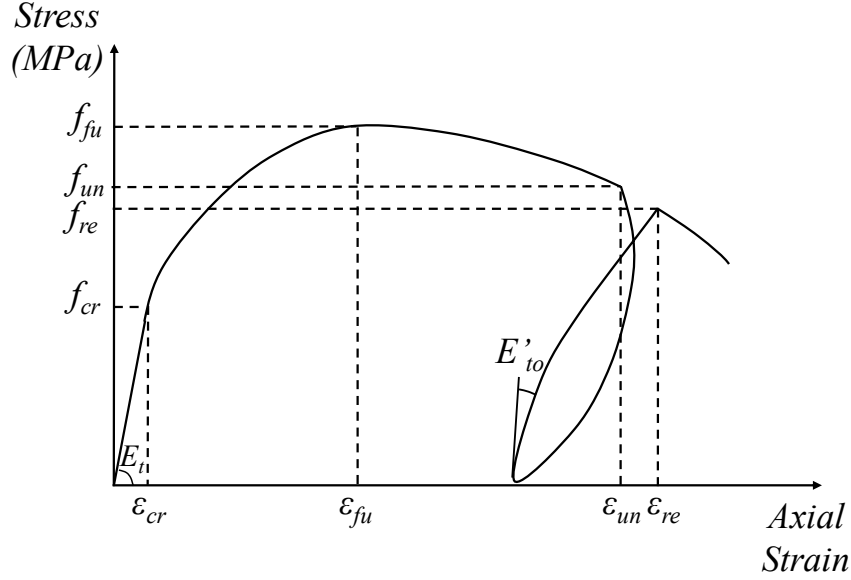


Figure 2.4. Cyclic constitutive model for UHPFRC in tension (adapted from Paschalis and Lampropoulos, 2016).

Wu et al. (2016) proposed a combined cyclic constitutive model for both tension and compression, which considers partial unloading/reloading; a phenomenon expected to be encountered in seismic nonlinear analysis. Figure 2.5 describes the backbone curves for tension and compression. Equations 2.22-2.23 represent the backbone curves for tension and compression, respectively. The trilinear tensile model is distinguished in three branches, namely the elastic, ascending and descending, and terminates at zero stress after the exceedance of the ultimate axial strain ε_{tu} . Stress-strain points $(f_{to}, \varepsilon_{to})$ and $(f_{tu}, \varepsilon_{tu})$ signify the initiation of the ascending and descending branches, respectively. For the compressive backbone curve, a parabolic curve is assumed up to the maximum stress f_c , followed by a linear descending branch and a horizontal residual branch. Axial stress-strain coordinates $(f_{cu}, \varepsilon_{cu})$ signify the initiation of the residual branch.

$$\text{For tension: } \sigma = \begin{cases} \varepsilon \cdot E_o, & \varepsilon \leq \varepsilon_{to} \\ f_{to} + (f_t - f_{to}) \cdot \left(\frac{(\varepsilon - \varepsilon_{to})}{(\varepsilon_t - \varepsilon_{to})} \right), & \varepsilon_{to} \leq \varepsilon \leq \varepsilon_t \\ f_{tu} \cdot \left(\frac{(\varepsilon - \varepsilon_{tu})}{(\varepsilon_t - \varepsilon_{tu})} \right), & \varepsilon_t \leq \varepsilon \leq \varepsilon_{tu} \\ 0, & \varepsilon > \varepsilon_{tu} \end{cases} \quad (2.22)$$

$$\text{For Compression: } \sigma = \begin{cases} f_c \left[k \cdot \left(\frac{\varepsilon}{\varepsilon_c} \right) + (1 - k) \cdot \left(\frac{\varepsilon}{\varepsilon_c} \right)^2 \right], & \varepsilon \leq \varepsilon_c \\ f_c + (f_{cu} - f_c) \cdot \left(\frac{(\varepsilon - \varepsilon_c)}{(\varepsilon_{cu} - \varepsilon_c)} \right), & \varepsilon_c \leq \varepsilon \leq \varepsilon_{cu} \\ f_{cu}, & \varepsilon > \varepsilon_{cu} \end{cases} \quad (2.23)$$

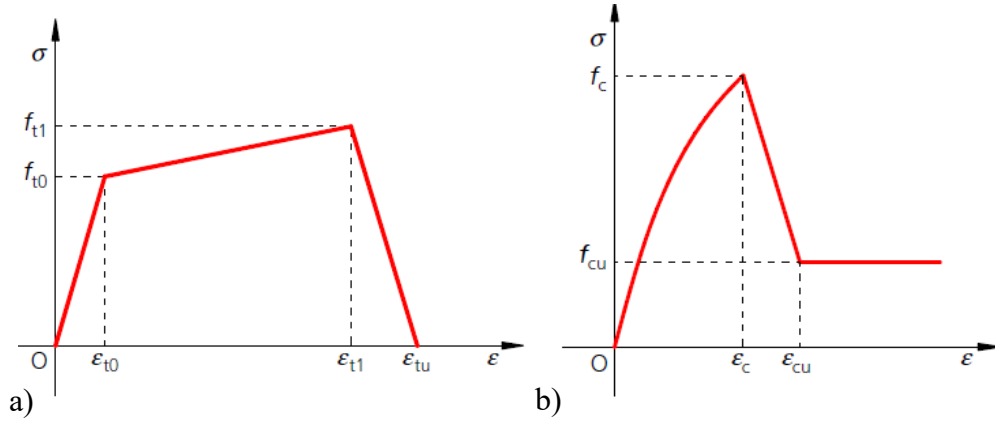


Figure 2.5. Envelope curves for THFRC in a) tension; b) compression (Wu et al. 2016).

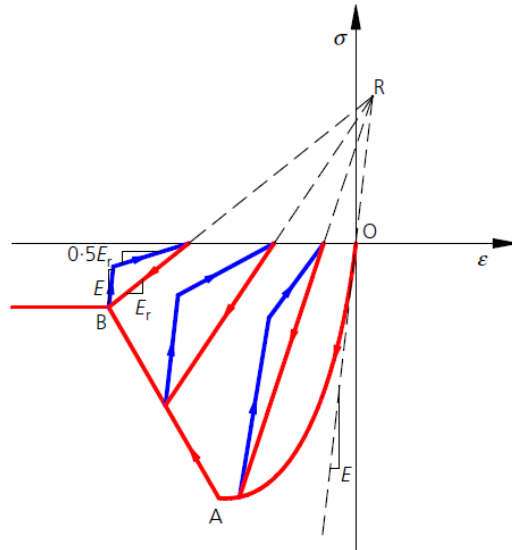


Figure 2.6. Definition of reference point for uniaxial cyclic constitutive model proposed by Yassin 1994 (image obtained from Wu et al. 2016, blue and red lines refer to unloading and reloading branches, respectively).

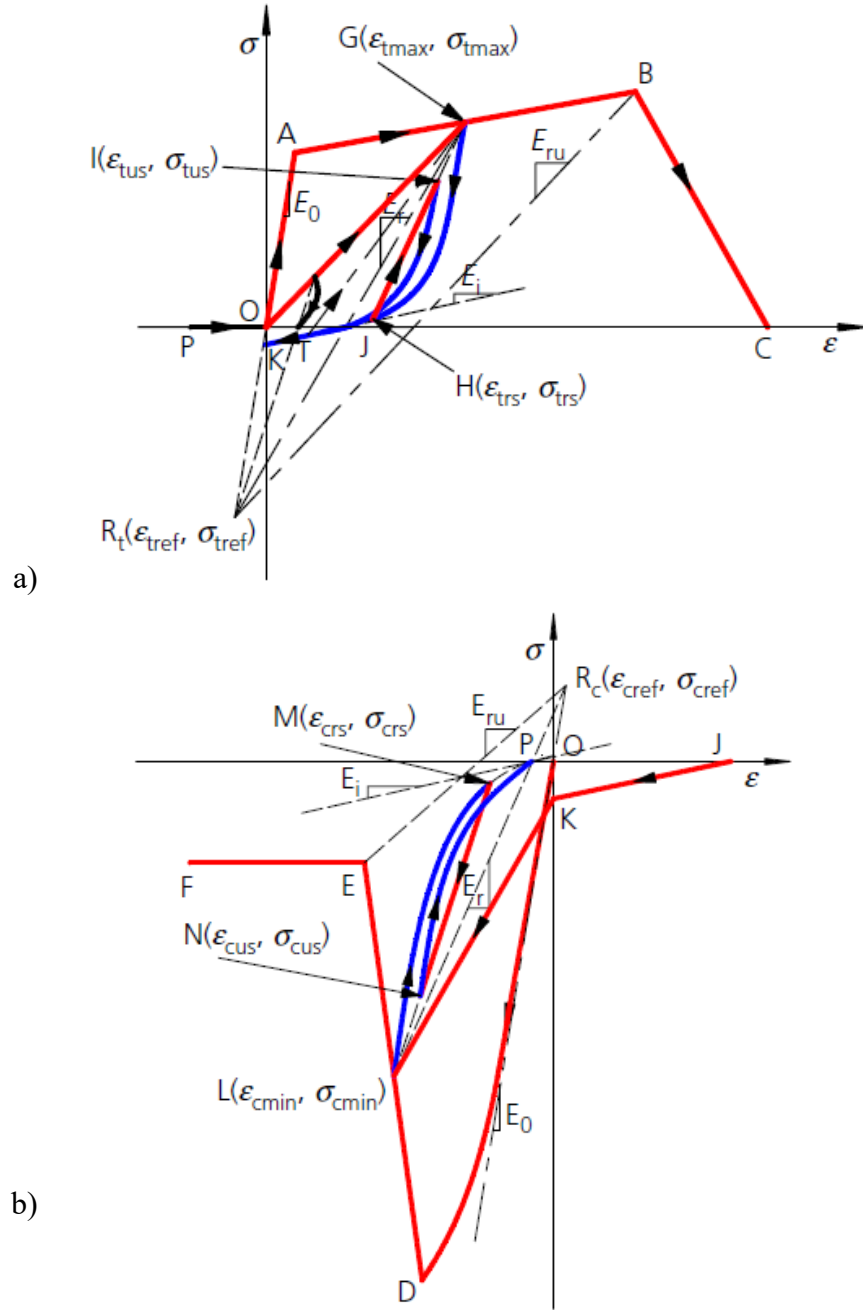


Figure 2.7. Cyclic constitutive model for THFRC in a) tension; b) compression (Wu et al. 2016, blue and red lines refer to unloading and reloading branches, respectively).

For the unloading and reloading branches, represented by the blue and red lines, respectively, in Fig 2.6, nonlinear functions were assumed for the model. In both cases, the reference point $R(\sigma_{ref}, \epsilon_{ref})$ is considered, which is the intersection point of the reverse extension lines from axis of the reloading lines, as proposed by Yassin (1994), which is illustrated in Fig. 2.6. The reverse extension lines are defined by the points at the start and end of the unloading branch (or

correspondingly the start and end of the reloading branch). The unloading branch consists of two straight lines, with slopes E and $0.5E_r$, where E is the initial modulus of elasticity and E_r is the slope of the reloading line. Extension of the reloading lines on the opposite side of the horizontal axis intercepts the extension line with slope E at the start of the backbone curve, at the origin, where reference point R is defined. The curves representing the cyclic constitutive response for tension and compression are shown in Fig. 2.7; the unloading and reloading are represented by the blue and red lines, respectively.

In the following equations (Eqs. 2.24-2.25) axial strain ε_{us} and stress σ_{us} correspond to the coordinates at the initiation of the unloading, which can originate on the backbone curve (full unloading - σ_{tmax} for tension, σ_{cmin} for compression) or from the line between points L and R (partial unloading - σ_{tus} for tension, σ_{cus} for compression). Parameters α and p are obtained from experimental data. In Eq. 2.26, axial strain ε_m and stress σ_m correspond to the coordinates of reloading.

$$\sigma^* = a \cdot \varepsilon^* + (1 - a) \cdot (\varepsilon^*)^p, \text{ and } \begin{cases} a = \frac{E_i}{E_t}, & \varepsilon^* = \frac{(\varepsilon - \varepsilon_o)}{(\varepsilon_{us} - \varepsilon_o)}, & \sigma^* = \frac{\sigma}{\sigma_{us}} \\ \varepsilon_o = \varepsilon_{us} - \frac{\sigma_{us}}{E_t}, & E_t = \frac{\sigma_{us} - \sigma_{ref}}{\varepsilon_{us} - \varepsilon_{ref}} \end{cases} \quad (2.24)$$

$$\sigma_{ref} = E_o \cdot \varepsilon_{ref}, \text{ and } \varepsilon_{ref} = \frac{\left(\frac{\sigma_{us}}{E_o}\right) - \left(\frac{E_{ru}}{E_o}\right) \cdot \varepsilon_{us}}{1 - \left(\frac{E_{ru}}{E_o}\right)} \quad (2.25)$$

$$\sigma = \sigma_{rs} + (\sigma_m - \sigma_{rs}) \cdot \frac{(\varepsilon - \varepsilon_{rs})}{(\varepsilon_m - \varepsilon_{rs})} \quad (2.26)$$

The transition between tension and compression, and vice versa, is represented through Eqs. 2.27 and 2.28, respectively. A bilinear stress-strain response is assumed for the former case, based on experimental data. For the latter case, the stress remains at zero until the strain reaches the original curve and a linear path defined by the secant to the original point and maximum strain of the previous load history is followed (lines PO and OG in Fig. 2.7a).

$$\sigma_{tc} = \begin{cases} E_i \cdot (\varepsilon - \varepsilon_o), & \varepsilon_o \geq \varepsilon > 0 \\ \frac{\varepsilon}{\varepsilon_{c,min}} \cdot (\sigma_{c,min} + E_i \cdot \varepsilon_o) - E_i \cdot \varepsilon_o, & \varepsilon \leq 0 \end{cases} \quad (2.27)$$

$$\sigma_{ct} = \begin{cases} 0, & \varepsilon < 0 \\ \frac{\sigma_{c,max}}{\varepsilon_{t,max}}, & \varepsilon \geq 0 \end{cases} \quad (2.28)$$

Krahl et al. (2018) implemented a plastic-damage model (Fig. 2.8a) to simulate the hysteresis loops in the backbone curves of tensile and compressive constitutive relationships. Figure 2.8b describes the backbone curves for tension and compression, represented by Equations 2.29 and 2.30. In Eq. 2.29, variables W_t and I_p represent the total tensile energy and the strain at the inflection point, respectively. The strain at peak is symbolized by ε_t and the stress at the end of the curve is σ_u . In Eq. 2.30, the stress-strain coordinates at peak are symbolized by σ_c and ε_c , respectively, and E_o is the initial tangent modulus of elasticity. Parameters k_1 and k_2 are calibrated based on experimental results, using the total compressive energy W_c (Eq. 2.31).

$$\sigma_t = \frac{W_t}{I_p \sqrt{\pi/2}} \cdot e^{\frac{-2(\varepsilon - \varepsilon_t)^2}{I_p^2}} + \sigma_u \quad (2.29)$$

$$\sigma = \sigma_c \left[\frac{k_1 \cdot \beta \cdot \frac{\varepsilon}{\varepsilon_c}}{k_1 \cdot \beta - 1 + \left(\frac{\varepsilon}{\varepsilon_c}\right)^{k_2 \cdot \beta}} \right], \text{ and } \beta = \frac{1}{1 - \frac{\sigma_c}{\varepsilon_c E_o}} \quad (2.30)$$

$$\begin{cases} k_1 = 0.42539 - 0.04942 - 0.2071W_c + 0.00186\beta^2 + 0.16163W_c^2 \\ k_2 = 1.19603 - 0.09059 - 0.82313W_c + 0.00329\beta^2 + 0.37862W_c^2 \end{cases} \quad (2.31)$$

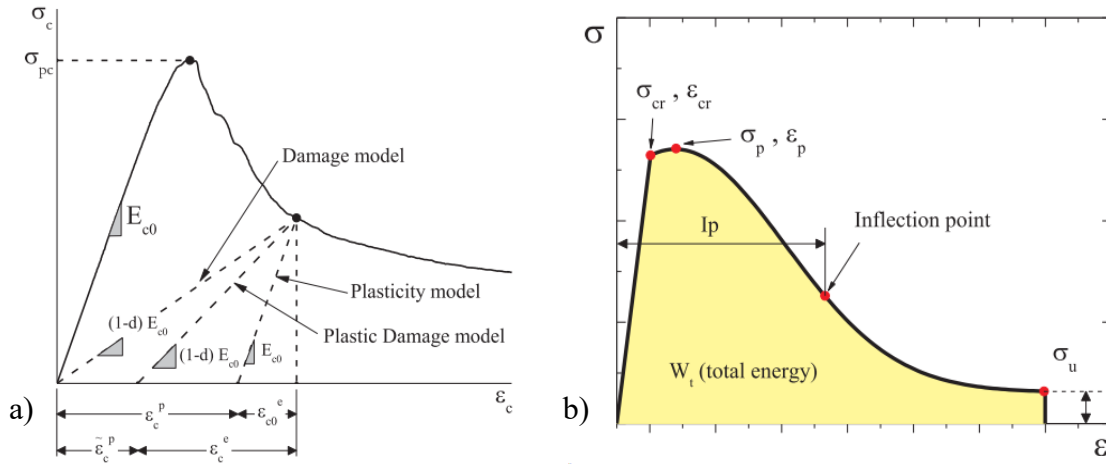


Figure 2.8. a) Plastic damage model curves; b) Backbone curve of cyclic model (Krahl et al. 2018).

Plasticity is also considered with the equivalent plastic strain ε_p , which is subtracted from the recoverable strains to represent the permanent damage of the material (Eq. 2.32). Damage evolution is represented by internal damage variable d (d_t for tension and d_c for compression), which allows to incorporate the stiffness degradation in the modulus of elasticity (E_c), as shown in

Eq. 2.32. Variables d_t and d_c are calculated from Eqs. 2.33 and 2.34, where constants y_o , A_1 , A_2 , t_1 , and t_2 are obtained experimentally.

$$E_c = (1 - d) \cdot E_o, \text{ and } \sigma = E_c \cdot (\varepsilon - \varepsilon_p) \quad (2.32)$$

$$d_t = y_o + A_1 \left(1 - e^{\frac{\varepsilon}{t_1}}\right) + A_2 \left(1 - e^{\frac{\varepsilon}{t_2}}\right) \quad (2.33)$$

$$d_c = y_o + A_1 \left(1 - e^{\frac{\varepsilon}{t_1}}\right) \quad (2.34)$$

Menna et al. (2022) investigated the flexural response of strain-hardening steel fiber reinforced concrete (SFRC) under cyclic loading, where the material contained various types of double-hooked-end steel fibers. They conducted four-point bending tests on prisms composed of SFRC, which were designed based on ASTM C1609, by applying loading and unloading cycles at increments of 0.5mm, starting at 0.25mm. Based on these tests, the stiffness degradation between subsequent cycles was defined, expressed in terms of the cyclic modulus E_{cyc} and the degree of reversibility R ; the former term is the slope defined by the coordinates at the start of the unloading branch and the start of the reloading branch (Fig. 2.9), and the latter term is the ratio of the reversible deformation to the maximum deformation prior to unloading, as expressed in Eq. 2.35.

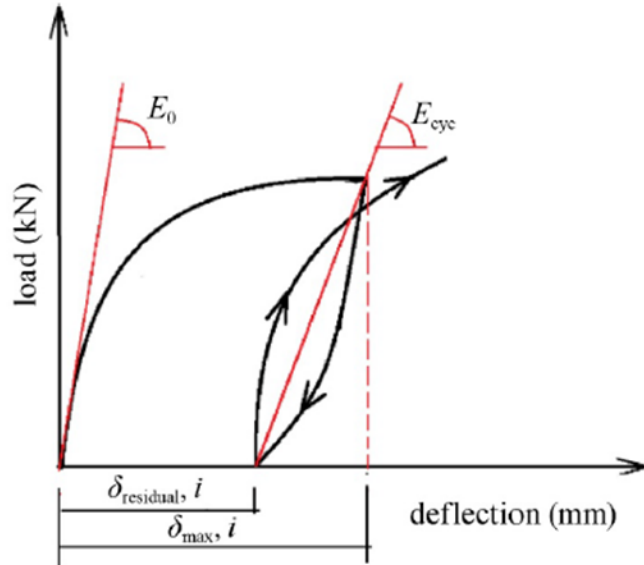


Figure 2.9. Definition of E_{cyc} during the flexural cyclic response of SFRC (Menna et al. 2022).

$$R = (\delta_{max} - \delta_{residual}) / \delta_{max} \quad (2.35)$$

In Eq. 2.35, δ_{max} is the deflection up to the start of the unloading branch and $\delta_{residual}$ is the deflection at the start of the reloading branch. The degree of reversibility R expresses the magnitude of reversible deformation, which decreases for larger number of subsequent cycles. It was also shown, by the experimental work of Menna et al. (2022), that higher energy dissipation in SFRC increased the value of R , which is a crucial property for structural performance under cyclic loading.

2.2 Seismic Performance of UHPFRC

Based on the strategy for seismic retrofitting, the application of UHPFRC jackets, typically containing steel fibers, is categorized in two approaches: a) repair and b) strengthening. In the former, artificial damage representing seismic impact is simulated on “as-built” specimens, followed by the application of the UHPFRC jackets; the retrofitted specimens are subsequently loaded under a reversed cyclic displacement regime. In the latter, the jackets are applied prior to seismic damage and cyclic loading is applied directly. Below, experimental investigations on seismic retrofitting with UHPFRC materials that are pertinent to the present are discussed and the repair and strengthening retrofit scenarios are distinguished. In the notable studies section, more specialized cases of the application of UHPFRC jackets are presented.

2.2.1 Repair

Farzad et al. (2019) simulated damage due to corrosion on circular columns (having a diameter of 305mm and a shear span of 1320 mm) by removing the concrete cover and reinforcement from the plastic hinge zone. They then repaired the specimens using two types of UHPFRC, one with 2% ($f'_c = 174\text{MPa}$, $f_t = 22\text{MPa}$ (from flexural tests)), and one with 4% ($f'_c = 179\text{MPa}$, $f_t = 25\text{MPa}$ (from flexural tests)), steel straight fibers and NSC ($f'_c = 48\text{MPa}$, $f_t = 5.5\text{MPa}$ (from flexural tests)). Continuous and lap spliced bars ($\varnothing 16$) were used at the column-footing connection, with longitudinal reinforcement ratio at 2.2%. Transverse bars ($\varnothing 10$) were placed at 100mm spacing. The repair zone extended to 35% and 86% of the clear height and, in some specimens, extended to the footing surface. The jacket thickness was between 13 and 51mm. For the specimens repaired with UHPC, superior load capacity and ductility, as well as controlled cracking, were observed in comparison to the unstrengthened counterparts, whereas even greater improvement was noted for the specimens with footing repair. It was noted by the researchers that relocating the plastic hinge

away from the footing and within the repaired zone of the jacket is critical in decreasing damage concentration in the footing. Selection of a pertinent jacket thickness, and the inclusion of the footing in the repair scheme, are critical to this phenomenon. Finally, the higher fiber content showed negligible contribution to the initial stiffness and energy dissipation.

Xue et al. (2018) used HPFRC jacketing, containing either steel (2% by volume, $f'_c = 110\text{MPa}$ (approx.), $f_t = 5\text{MPa}$ (approx.)) or polymer (4% by volume, $f'_c = 52\text{MPa}$ (approx.), $f_t = 3.6\text{MPa}$ (approx.)) fibers, to repair previously damaged circular RC columns (diameter of 420mm $\times$ shear span of 1170 mm, including the cap beam) and reloaded them with reversed cyclic loading. In this study, the repair scheme included the use of turned steel rebar to replace the damaged reinforcement (by the term “turned rebar” the authors refer to smoothened steel bars after removal of the ribs). An older configuration of bars was used, with design criteria preceding the modern Chinese seismic code of JTG/T-B02/01/2008. Damage was simulated via the application of preliminary cyclic loading, and the two types of HPFRC materials mentioned in the preceding were used for retrofitting. In both cases, column capacity was restored, and adequate shear strength was provided by the jackets, indicating reduced requirements for additional reinforcement. However, steel fibers were shown to be more efficient in the energy dissipation and damping, in comparison to the polyvinyl alcohol (PVA) fibers. It should be noted that PVA fibers are more commonly implemented in ECC and HPFRC materials, whereas UHPFRCs contain steel fiber reinforcement.

Li et al. (2017) applied cyclic loading on rectangular NSC column specimens (having a cross section of 200 mm $\times$ 200 mm $\times$ 820 mm) repaired with HPFRC jackets, after initially imposing artificial damage through preliminary cyclic loading. The HPFRC material ($f'_c = 42\text{MPa}$, $f_t = 6\text{MPa}$ (approx.)) contained PVA fibers at 1.7% of the volume percentage. The section dimensions were remained unaltered after repair, while anchored longitudinal bars ensured the connection to the rigid footing, according to modern seismic codes. Two repair heights were considered, at 33% and 55% of the clear height. Conventional detailing was applied to the sections, with 1% longitudinal reinforcement ratio ($\varnothing 16$) while $\varnothing 6$ bars spaced at 80mm were used for transverse reinforcement. In all HPFRC-repaired specimens, it was found after testing that the load resistance was enhanced, and improved displacement capacity was observed in comparison to their original counterparts, with greater improvements provided by the jacket with the greater height. The researchers

recommended the repair height to extend as far as 1.5 times the larger of the dimensions of the column cross-section. Reduced crack propagation and the horizontal orientation indicated adequate shear resistance from the jacket in the repaired specimens.

2.2.2 Strengthening

Dagenais et al. (2018) applied a UHPFRC ($f'_c = 93\text{-}128\text{MPa}$, $f_t = 6.5\text{-}10.7\text{MPa}$) pre-packaged mix, with 3% straight steel fibers, on rectangular bridge piers (having a cross section of $600\text{mm} \times 1200\text{mm}$) constructed with deficient detailing and tested them under reversed cyclic loading. Lap spliced connections were used at the lower end, at the column-footing interface, as it was customary in design guidelines preceding the more modern seismic codes of the 1970s. Various longitudinal reinforcement ratios, bar geometries and lap splice lengths were evaluated. Artificial damage was imposed with demolition of the cover concrete around the lap splices, up to $1d_b$ depth, with UHPFRC jackets covering the whole extent of the spliced connection. In all specimens, ductile responses were reported during the test, with ultimate damage determined by the rupture of the starter bars. Damage was shown to extend from the column to the footing, due to plastic deformations penetrating from the starter bars into the base, which was manifested by one large, localized crack at the column-footing connection.

Chan et al. (2020) investigated the performance of precast circular column specimens (diameter of 508mm and shear spans of 2032mm and 1270mm , including the cap beam) with UHPFRC ($f'_c = 137.3\text{-}140.1\text{MPa}$, $f_t = 24.9\text{-}28.2\text{MPa}$ (from flexural tests)) lap splice connections, with high strength starter bars extending into the footing, and compared the results with those of conventional cast-in-place (CIP) specimens constructed by Al-Jelawy et al. (2018). Comparable responses under cyclic excitation were obtained between the two types of specimens, with the CIP case exhibiting bond failure and pullout of the starter bars. It was noted by the researchers that the improved bond between connection bars and the UHPFRC, combined with the shifted plastic hinge design, were sufficient for the precast specimens to develop sufficient lateral resistance and improve their ductility capacity.

Tong et al. (2019) applied combined cyclic and axial loading on two UHPFRC ($f'_c = 129.7\text{MPa}$, $f_t = 5.52\text{MPa}$) jacket-retrofitted rectangular RC piers ($500\text{mm} \times 450\text{mm} \times 2300\text{mm}$). The jackets had a thickness of 50mm , thereby enlarging the combined cross-sectional dimensions, and extended up to 20% and 44% of the clear height. Longitudinal reinforcement ratio ($\phi 16$) was

0.71%, as denoted in Chinese standards prior to the 1980s, and transverse reinforcement ($\varnothing 8$) was spaced at 80mm at the PH location. Sufficient anchorage was provided with continuous longitudinal bars in the footing. Peak strength was increased for the specimen with the longer jacket; however, it was noted that ductility was reduced for increased jacket lengths.

Cho et al. (2012) applied HPFRC with PVA fibers ($f_t = 3.7\text{-}4.4\text{MPa}$) in the plastic hinge region of intact NSC columns ($300\text{mm} \times 300\text{mm} \times 1540\text{mm}$) and tested them under a history of reversed cyclic displacements of increasing intensity. The fiber content, either 1.5 or 2.0% by volume; length of the jacket ($1.5\text{-}2.0d_b$) and presence, or not, of shear reinforcement ($\varnothing 10@100\text{mm}$) were parameters of this study. Straight, anchored bars were used for the column-footing connection, without any lap splice connections that were prevalent in older structures. In comparison to the control NSC specimen, all HPFRC-strengthened specimens produced stable hysteresis and greater deformation capacity, as well as improved energy dissipation. Damage phenomena were delayed by the HPFRC jacket, allowing for overall improved lateral performance of the specimens.

Considering the above, it was shown that the performance of UHPFRC-retrofitted column specimens, both in the repair and strengthening scenarios, has not been sufficiently investigated in existing literature. In the former case, preliminary damage has been applied in the form of artificial removal of concrete cover and reinforcement in the plastic hinge region (Farzad et al. 2019), while and cyclic loading histories on columns subsequently retrofitted with HPFRC materials have been studied (Xue et al. 2018, Li et al. 2017). In the latter case, application of UHPFRC jackets in the plastic hinge region of column specimens has been investigated in the case of footing connections containing lap splices (Dagenais et al. 2018, Chan et al. 2020), complete replacement of the plastic hinges (Chan et al. 2020, Cho et al. 2012) and external jacketing with increase of the cross-sectional dimensions (Tong et al. 2019).

As such, in this study, column specimens retrofitted with UHPFRC jackets in the plastic hinge region, as well as lap-spliced connections and deficient detailing in accordance with older design standards, were tested under combined constant axial and reversed cyclic loading conditions. Direct comparison with identical and undamaged specimens, which were strengthened with UHPFRC jackets in the plastic hinge region, was performed. The effectiveness of this retrofit method to enhance the impaired seismic performance and mitigate the effects of previous damage history was investigated herein.

2.2.3 Notable Studies

Reggia et al. (2020) constructed a large scale rectangular hollow bridge pier specimen (1500mm $\times$ 625 mm $\times$ 5830mm) with conventional detailing and tested it under combined constant axial load and cyclic horizontal load in the original state, up to the 50% the theoretical load prior to reinforcement yielding. After the initial test the specimen was retrofitted using a thin HPFRC jacket (30mm thickness, $f'_c = 136.3\text{MPa}$, $f_t = 4.5\text{MPa}$) and the strengthened specimen was tested until failure. The HPFRC jacket was applied at almost the full height of the pier specimen, with the axial load being applied throughout the retrofit procedure. It was found that in the presence of the HPFRC jacket, both the strength and ductility showed substantial improvement, with the former property increased by 57% from the theoretical corresponding value of the unstrengthened pier. Yielding of the longitudinal reinforcement also occurred at a higher load, for the strengthened specimen, while longitudinal cracking was more diffused. Despite the thin layer of the HPFRC jacket, the pier was able to reach an ultimate drift of 3.4%.

Aviram et al. (2014) constructed circular columns (diameter of 406mm; shear span of 1625mm) containing conventional reinforcement detailing, as well as lap-spliced connections at the column-footing interface. In this study, one of the columns was constructed with regular concrete and two others, with identical geometric properties, were constructed completely with HPFRC ($f'_c = 46.7\text{MPa}$, $f_t = 6.69\text{MPa}$). In addition, bond “breakers” were utilized on the starter and main longitudinal bars (13M) of the shear span, in each HPFRC column specimen, respectively. The debonded segments were intended to allow for more evenly distributed reinforcement strains in the longitudinal bars, and to avoid localization, and subsequent premature fracture, at the end of the lap-splice connection; this approach is also beneficial in transferring the formation of the plastic hinge further from the critical section at the column-footing interface. For both HPFRC columns, apart from the improvements in strength and ductility, compared to the control case, a stable hysteretic response was shown despite the large demands in drift and shear force. Failure in the HPFRC specimens was attributed to loss of flexural capacity while no evidence of critical shear failure was observed, despite the lax spacing of transverse reinforcement. The specimen with the bond “breaker” placed in the starter bars showed the most extensive distribution of bar yielding.

Yuan et al. (2022) constructed three circular column specimens (diameter of 320mm; shear span of 1300mm), containing conventional reinforcement detailing based on the seismic design

provisions for highway bridges of the Ministry of Transport of China, two of which were retrofitted with UHPFRC ($f'_c = 110.6\text{MPa}$, $f_t = 5.45\text{MPa}$) in the plastic hinge location. The jacketed portion extended to 46% of the specimen shear span, with two specimen cases with jacket thicknesses of 25mm and 50mm, respectively. In the former case, the jacket replaced the cover of the section in the plastic hinge region, without increasing the section dimensions. In the latter case, the jacket resulted in the increase of the cross-sectional dimensions, including the cover replacement. All specimens were tested under combined constant axial load and cyclic lateral displacement reversals. Compared to the unjacketed specimen, the UHPFRC-retrofitted columns showed improved hysteretic response, with larger areas enclosed in the unloading/reloading hysteresis loops, as well as reduced residual displacements, with the greater reduction observed for the thicker jacket.

Hong et al. (2021) constructed rectangular column specimens ($300\text{mm} \times 300\text{mm} \times 1230\text{mm}$), containing conventional reinforcement detailing (8 x D-19 long., D-10/300mm transv.) which were retrofitted with an external UHPFRC jacket throughout the whole shear span, as well as the combination of UHPFRC jackets ($f'_c = 172\text{MPa}$, $f_t = 6.4\text{MPa}$) and textile reinforcement. The control case consisted of the equivalent geometrical properties without the UHPFRC jacket and textile reinforcement. For this study, all specimens were designed to be controlled by shear failure, owing to the combination of high longitudinal reinforcement ratio and sparse transverse reinforcement. All column specimens were subjected to a constant axial load ratio of 30% and 45% of $A_g f_c$ ($f'_c = 24\text{MPa}$ and $f'_c = 60\text{MPa}$), combined with cyclic lateral displacement reversals. Compared to the control case, the UHPFRC retrofitted columns showed reduced damage and controlled cracking, with shear cracking ultimately controlling the failure. The specimens containing both the UHPFRC jacket and textile reinforcement showed improved lateral performance, particularly in ductility, altering the governing failure mode from shear to flexural shear, and reducing the opening of vertical cracks. It was concluded by the researchers that the combination of the textile reinforcement and UHPFRC jackets is a viable option to mitigate failure in shear critical members.

2.3 Confinement Mechanism

The passive confinement provided on the concrete core of a column as a result of encasement by confining mechanism has been approximated using Eq. 2.36, as proposed by Richart et al. (1928). Variables f_{cc} , and f_{co} correspond to the compressive strengths of confined concrete and unconfined concrete, respectively, and f_l represents the lateral confining pressure. Parameter K is an experimentally derived constant which depends on the effectiveness of the confinement.

$$f_{cc} = f_{co} + K \cdot f_l \quad (2.36)$$

Rabehi et al. (2014) evaluated the performance of previously damaged cylindrical (unreinforced) concrete cores, repaired with UHPFRC jackets, and proposed the modification shown in Eq. 2.37. Variable f_{cd} corresponds to the compressive strength of the damaged concrete. Variables t_j , d and f_{tj} correspond to the thickness of the jacket, the diameter of the concrete core and the tensile strength of the jacket, respectively.

$$\frac{f_{cc}}{f_{cd}} = 2.5 + \frac{f_l}{f_{cd}}, \text{ and } f_l = \frac{2 \cdot t_j}{d} \cdot f_{tj} \quad (2.37)$$

Tong et al. (2019) assumed plain strain conditions to evaluate the passive confinement of UHPFRC jackets on strengthened rectangular RC piers. The confined stress state, in the xy plane (where x and y the principal axes of the cross section of the pier), is described by Eq. 2.38 and the expressions for A and B are given by Eq. 2.39. Variables E_u , t_u , and ε_z correspond to the modulus of elasticity for UHPFRC, the thickness of the jacket, and axial strain, respectively.

$$\begin{cases} \sigma_x = A \cdot (x^2 - y^2) - B \cdot l^2 \\ \sigma_y = A \cdot (y^2 - x^2) - B \cdot l^2 \\ \tau_{xy} = -2 \cdot A \cdot x \cdot y \end{cases} \quad (2.38)$$

$$\begin{cases} A = 21 \cdot E \cdot E_u \cdot t_u \cdot \nu \cdot \varepsilon_z / (l^2 \cdot [25 \cdot E \cdot l + 12 \cdot E_u \cdot t_u \cdot (2 \cdot \nu + 5)]) \\ B = 18 \cdot E \cdot E_u \cdot t_u \cdot \nu \cdot \varepsilon_z / l^2 \cdot [25 \cdot E \cdot l + 12 \cdot E_u \cdot t_u \cdot (2 \cdot \nu + 5)] \end{cases} \quad (2.39)$$

In the analytical solution proposed, considering both the confinement provided by stirrup reinforcement and the UHPFRC jacket, the peak strength and corresponding strain at peak of the core confined concrete core are given by Eq. 2.40.

$$\frac{f_{cc}}{f_{co}} = 1 + 8 \cdot \frac{f_l}{f_{co}} - 4 \cdot \left(\frac{f_l}{f_{co}} \right)^{1.2}, \text{ and } \varepsilon_{cc} = \varepsilon_{co} \cdot \left(\frac{f_l}{f_{co}} \right)^{1.1} \quad (2.40)$$

Malak et al. (2021) defined a multi-stage model for the compressive stress-strain relationship of confined concrete using SIFCON jackets around cylindrical specimens. Five stages were recognized by this model (Fig. 2.10): 1) The confinement region, where the jacket prevents lateral expansion prior to activation, 2) the transition point, which signifies the initiation of the jacket activation, followed by 3) the first crack/multi-crack region, 4) the fiber pullout region, which begins after the attainment of the peak stress, and 5) the inner core failure/rupture region. The model distinguishes the effectiveness of the confinement in “full” and “partial” to account for the randomness associated with the fiber dispersion and orientation.

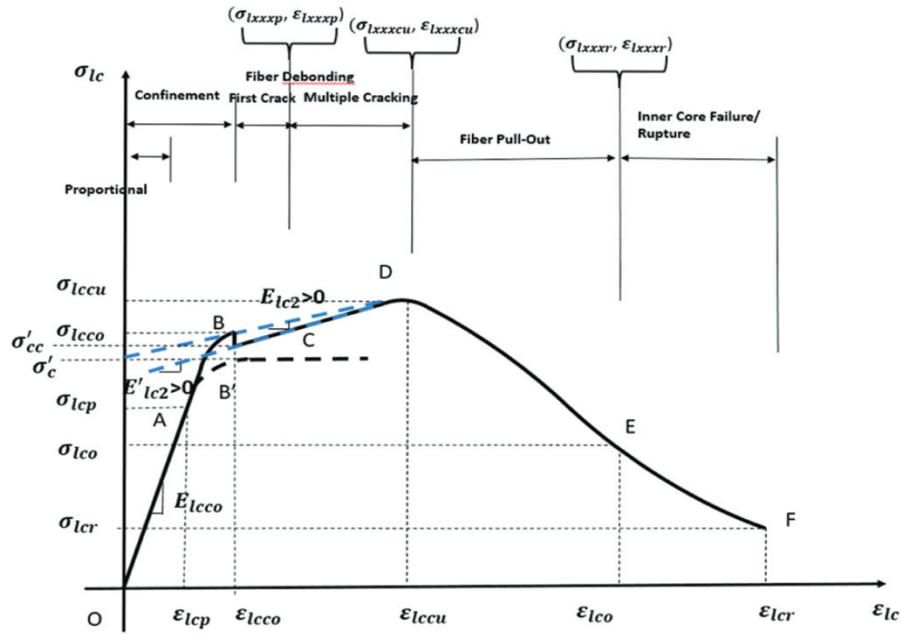


Figure 2.10. Visual representation of the multistage stress-strain model for the a) longitudinal direction (image obtained from Malak et al. 2021).

2.3.1 Compressive behavior of confined concrete

One of the most prominent applications of UHPFRC is the confinement of existing reinforcement concrete members, through external application in the form of jackets. The resulting mechanical performance of the confined concrete is improved, both in terms of strength and ductility, and pertinent analytical laws are required to describe the constitutive behavior. Existing models, such as the Modified Kent and Park (1982), are implemented to describe the compressive stress – strain relationship of the confined concrete. This model is characterized by efficiency and ease of implementation. Additional models, such as the one described by Darwin and Pecknold (1974), modified from Karsan and Jirsa (1969), provide an analytical approach for determining the

compressive constitutive properties under cyclic axial loading. The models are described in this section.

2.3.1.1 Modified Kent and Park Model (1982) for Confined Concrete

For this model, the material's ascending part of the uniaxial compressive stress-strain relationship is defined according to Hognestad's parabola (Hognestad 1951), as shown in Eq. 2.41 below. For the post-peak, descending branch of the envelope, the modified constitutive model by Scott et al. (1982) is used, as shown in Eq. 2.42, until the residual branch which is described in Eq. 2.43.

$$f = f_{cc} \cdot [2 \cdot (\varepsilon/\varepsilon_{cc}) - (\varepsilon/\varepsilon_{cc})^2], \varepsilon \leq \varepsilon_{cc} \quad (2.41)$$

$$f = f_{cc} \cdot [1 - Z \cdot (\varepsilon - \varepsilon_{cc})], \varepsilon_{cc} < \varepsilon \leq \varepsilon_{cu} \quad (2.42)$$

$$f = 0.2 \cdot f_{cc}, \varepsilon > \varepsilon_{cc} \quad (2.43)$$

In the equations above, f_{cc} and ε_{cc} are the stress and strain of confined concrete as per Eqs. 2.44 and 2.45. Parameter Z describes the slope of the descending branch, which is defined in Eq. 2.46 shown below.

$$\varepsilon_{cc} = \varepsilon_{co} \cdot K \quad (2.44)$$

$$f_{cc} = f_c \cdot K \quad (2.45)$$

$$Z = 0.5 \cdot \left(\frac{3+0.29[0.0285] \cdot f_c}{145[14.2] \cdot f_c - 1000} + 0.75 \cdot \rho_v \cdot \left(\frac{h}{s} \right)^{\frac{1}{2}} - \varepsilon_{co} \cdot K \right)^{-1}, \text{ in MPa[PSI]} \quad (2.46)$$

For Eq. 2.46, empirical parameters vary according with the system of units; the expression is intended to be used in the SI system (stresses in MPa); for the Imperial System of units the corresponding empirical constants are given in brackets “[]”. Parameter ρ_v is the volumetric ratio of transverse reinforcement, h is the width of the concrete core and s is the spacing of the stirrups.

The lateral confinement stresses are defined by parameter $\sigma_{tot,lat}$, which is the sum of the lateral confinement stresses contributed by the transverse reinforcement ($\sigma_{st,lat}$) and the UHPFRC jacket ($\sigma_{j,lat}$), as described by Eq. 2.47.

$$\sigma_{tot,lat} = \sigma_{j,lat} + \sigma_{st,lat} \quad (2.47)$$

The effective confining pressure on the encased concrete, contributed by the UHPFRC jacket is obtained as follows:

$$\sigma_{j,lat} = 2k_{e,f}\varphi_c f_t \left(\frac{t}{h} \right) \quad (2.48)$$

Where, t is the jacket thickness. The confinement effectiveness $k_{e,f}$ is obtained from the ratio of the sectional area that is effectively confined, divided by the total encased area (assuming a rectangular cross section with dimensions $b \times h$). Research on confinement effectiveness using UHPC jacketing on columns (Rabehi et al. 2014) has led to an estimate for the confinement effectiveness coefficient, $k_{e,f}$, that is practically identical to that obtained for FRP jackets (Pantazopoulou et al. 2016; fib Bulletin 40):

$$k_{e,f} = 1 - \frac{(b^2 + h^2)}{3 \cdot (1 - \rho_{sc})bh} \quad (2.49)$$

For the case of the simultaneous presence of confining reinforcement (hoops and stirrups), the contribution of stirrup confinement to the confining pressure is accounted for, as follows:

$$\sigma_{st,lat} = 0.5 \cdot k_{eff} \cdot \rho_w \cdot f_{y,st} \quad (2.50)$$

Where, k_{eff} is the confinement effectiveness factor, ρ_w is the volumetric ratio, and $f_{y,st}$ is the yield stress for the confining reinforcement (Eurocode 8).

2.3.1.2 Chang and Mander (1994) for Confined Concrete

The model proposed by Chang and Mander (1994) for the monotonic envelope of confined concrete in compression consists of a continuous smooth curve, which is replaced by a straight line after the onset of cracking. At spalling, it is assumed that the concrete strength is zero. The complete sets of equations that describe the stress and tangent modulus of elasticity are shown in Eqs. 2.51-2.53.

$$\begin{cases} f = f_{cc} \cdot y(x) \\ E = E_c \cdot z(x) \end{cases}, \quad x < x_{cr} \quad (2.51)$$

$$\begin{cases} f = f_{cc} \cdot [y(x_{cr}) + n \cdot z(x_{cr}) \cdot [x - x_{cr}]] \\ E = E_c \cdot z(x) \end{cases}, \quad x_{cr} \leq x \leq x_{sp} \quad (2.52)$$

$$f = E = 0, \quad x > x_{sp} \quad (2.53)$$

Where $x = \varepsilon/\varepsilon_{cc}$, $n = Ec \cdot \varepsilon_{cc} / f_{cc}$ and r is a parameter for adjustment of the shape of the curve. Functions $y(x)$ and $z(x)$ are based on the model proposed by Tsai (1988) and are described in Eqs. 2.54-2.56.

$$y(x) = n \cdot x / D(x) \quad (2.54)$$

$$z(x) = (1 - x^r) / (D(x))^2 \quad (2.55)$$

$$D(x) = 1 + \left(n + \frac{r}{r-1}\right) \cdot x + \frac{x^r}{r-1}, \quad r \neq 1 \quad (2.56a)$$

$$D(x) = 1 + (n - 1 + \ln x) \cdot x, \quad r = 1 \quad (2.56b)$$

The spalling strain, expressed in the non-dimensional strain x_{sp} can be obtained from Eq. 2.57. However, greater values can be assumed for the case of confined concrete that does not spall.

$$x_{sp} = x_{cr} - y(x_{cr}) / n \cdot z(x_{cr}) \quad (2.57)$$

For the hysteretic response, the model proposed by Chang and Mander (1994) consists of three main components, namely, the envelope curves, connecting curves, and transition curves, which are shown in Fig. 2.11.

Envelope curves determine the “back-bone” of the hysteretic response, which are described by equations such as the ones shown above. Connecting curves represent the connection between envelope curves, such as the unloading and reloading curves, where pinching, softening or hardening phenomena are represented. Transition curves are used when a reversal is made at the connecting curve, i.e., the reversal from the compression to the tension envelope, and vice versa, where these curves are used for stability. The detailed equations, summarized by a set of 15 “rules”, which correspond to each branch, as shown in Fig. 2.11. that describe the hysteretic properties of concrete were obtained through statistical regression analyses on experimental data and are included in Chang and Mander (1994).

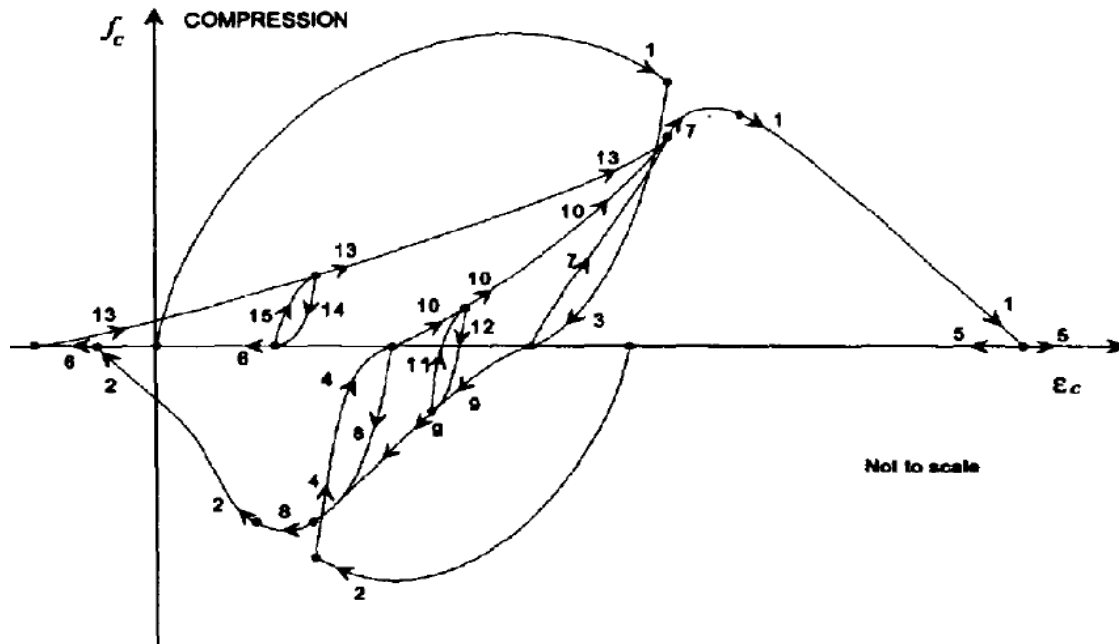


Figure 2.11. Sample of relationships between the model components (Chang and Mander 1994).

2.3.1.3 The Cyclic Model for Concrete in Compression by Darwin and Pecknold (1974)

To obtain a complete stress-strain law for concrete under cyclic compression, the hysteresis loops need be defined. In experimental curves, the hysteresis loops are defined by the upper and lower bounds of the envelope, or skeleton curve, and by the strain, at the end of the unloading loop, respectively (Fig. 2.12).

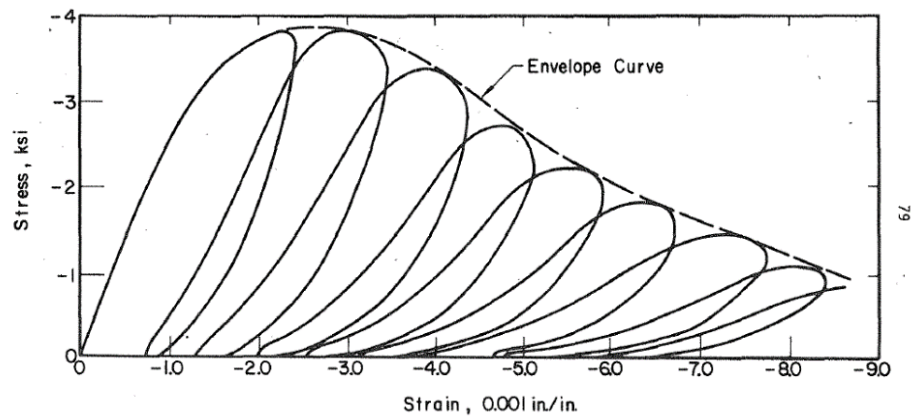


Figure 2.12. Hysteretic response of concrete to uniaxial cyclic loading (Darwin and Pecknold (1974)).

Karsan and Jirsa (1969) determined that the coordinates of the points that influence the hysteresis loop shape and size exist in a band of points below the envelope curve. For the case of unloading

under the lower limit of the band, the stress – strain curve forms a closed hysteresis loop. For the case of unloading above the lower limit and on the upper limit of the band, permanent deformations are accumulated in the material. If the loop is maintained above the upper limit, no additional deformation is accumulated.

The upper and lower band limits are reduced to single line curves, based on the work of Darwin and Pecknold (1974), which are defined by the common points and stability (or turning) points curves; the analytical definitions of the idealized envelope curves are defined based on the experimental response of concrete in compression, as depicted in Fig. 2.13. The two envelopes, referred to as “Common Points” and “Turning Points” are used to guide the formation of the hysteresis. The equations describing these curves are discussed further in Chapter 5.

Initially, unloading from the envelope at a strain ϵ_{eni} (at coordinates $(\sigma_{eni}, \epsilon_{eni})$) follows a line parallel to the elastic branch (with slope E_o), until it intersects the turning points envelope; beyond that point, unloading continues with reduced slope to the residual strain $\epsilon_{un,0}$ for zero residual compressive stress – further reduction of compressive strain to ϵ_{rev} follows without giving any rise in stress. Reloading from that strain towards the envelope follows the slope of the last segment of the unloading branch, till it crosses the unloading line in the “Common Points” envelope.

The coordinates of the envelope curve are provided, for this study, from the Thorenfeldt et al. (1987) model. Based on the envelope curve, the coordinates of the common points and turning points curves can be obtained.

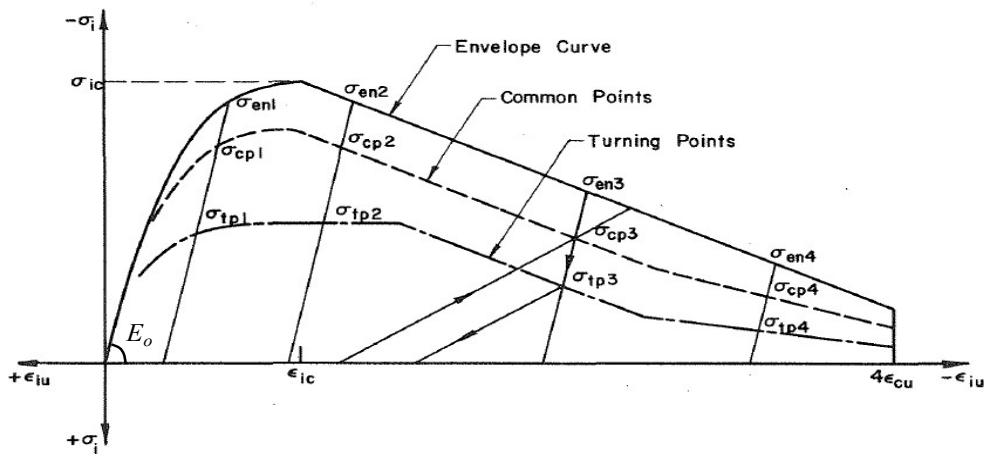


Figure 2.13. Envelope curve, common points and turning points, as defined in Darwin and Pecknold (1974).

It was also shown, by Karsan and Jirsa (1969), that the strain of the envelope curve prior to unloading, namely ε_{en} , and the residual strain after the load has been completely unloaded, namely the plastic strain ε_{pl} , are closely related. The following equations (Eqs. 2.58-2.59) were determined to describe the relationship between the strains, for the unloading and reloading branches, respectively:

$$\frac{\varepsilon_{pl,un}}{\varepsilon_{cu}} = 0.145 \cdot \left(\frac{\varepsilon_{en}}{\varepsilon_{cu}}\right)^2 + 0.13 \cdot \frac{\varepsilon_{en}}{\varepsilon_{cu}} \quad (2.58)$$

$$\frac{\varepsilon_{pl,re}}{\varepsilon_{cu}} = 0.093 \cdot \left(\frac{\varepsilon_{en}}{\varepsilon_{cu}}\right)^2 + 0.091 \cdot \frac{\varepsilon_{en}}{\varepsilon_{cu}} \quad (2.59)$$

In the context of the present study, Equation 2.58 is modified, in order to obtain the plastic strain ε_{pl} for confined concrete with UHPFRC jackets. The monotonic envelope, as defined by the Thorenfeldt et al. (1987) model, as well as the hysteretic loop coordinates shown in Darwin and Pecknold (1974) were combined as part of this study, to obtain the constitutive properties of confined concrete with UHPFRC jackets, under cyclic axial loading. This is described in detail in Chapter 5. From the available models in the literature, the model by Darwin and Pecknold (1974) was selected for this study, to account for the significant deformation capacity imparted by the UHPFRC material. The coordinates of the hysteresis loops were determined based on the common points and turning points curves, which were calculated based on the envelope curve and, thus, the contribution by the UHPFRC jacket was considered. In previous models, such as the one proposed by Chang and Mander (1994), the hysteretic properties were based on tests on specimens with conventional concrete materials, which were not considered in this study. This selection was based on observations made by this researcher.

3 Bond of Reinforcement in UHPFRC

The method of column rehabilitation through cover replacement requires removal of the existing cover to a depth of about $1D_b$ behind the longitudinal bars (where D_b is the diameter of the bars) and replacement of the cover concrete with UHPFRC. In doing so, the intent is to confine the encased core by taking advantage of the tensile resistance of the replaced cover. Confinement is also exerted on the longitudinal bars which are now developed in the UHPFRC cover. This opens two research questions that need to be understood for the benefit of modelling and interpretation of the retrofit scheme applied on a structural component. This and the next chapter of the present thesis are dedicated to the investigation of these two aspects: Chapter 3 explores development of reinforcement bars embedded in UHPFRC covers, in order to determine the local bond stress – slip relationship and the pattern of strain penetration over the development under large pullout slip at the critical section (i.e., a crack location). Chapter 4 explores the behavior of concrete encased in UHPFRC jackets when subjected to cyclic load reversals. Results from these two investigations are used in the development of Finite Element models for the analysis of retrofitted columns.

Bond and strain penetration of reinforcement embedded in UHPFRC materials is studied using data from experiments conducted previously at the High Bay Lab and from published literature, followed by a detailed analytical investigation. The presentation below comprises two parts: i) the assembly of an experimental database on existing bond tests conducted on tension-hardening materials (this term encompasses UHPFRC, high performance fiber-reinforced concrete (HPFRC) and engineered cementitious composites (ECC) materials), where the effects of cementitious cover and boundary conditions on the bond-slip law are quantified with the objective to understand how the confinement imparted by the cover material affects the bar development capacity, which is the development of the strain capacity of the reinforcement, ii) a detailed three-dimensional numerical analysis of the actual state of stress along a bar embedded in UHPFRC where the bar profile is modeled explicitly and the bond slip law is the output of the analysis rather than the input at the bar – cover interface. Among the outcomes of this study is to provide experimental evidence, as well as confirm via computational simulation, the internal confinement effect owing to the increased tensile performance of the tension-hardening materials that contributes to the bond

between reinforcement and concrete. This effect, provided by the unique mechanical characteristics of tension hardening cementitious materials, is pivotal to the confining action of the jacket on existing RC columns.

Tension hardening materials present a prolonged post-cracking range of response to tension, marked by sustained or even increased tensile resistance with increasing deformation. This behavior is owing to the presence of fibers which intercept the propagation of cracks and delay the separation of the crack surfaces to larger stress, leading to apparent ductile responses and sustained damage. The dense structure of the matrix and the low fraction of capillaries in the pore volume protects the matrix from penetration of external environmental and chemical agents, thereby providing durability and robustness. The tensile resilience is valuable in many aspects of structural concrete design, particularly, in the improvement of deformation capacity and mechanical performance of plastic hinge locations and in retrofitting inadequate structural components, either suffering from previous damage (e.g., due to corrosion) or due to the absence of sufficient reinforcement details. Another area where tension hardening materials have gained wide applicability and acceptance is in continuity joints between precast decks in ABC (Accelerated Bridge Construction). In this form of construction, the gap between successive panels which is relatively narrow (200 mm or less) is filled usually with UHPFRC in order to provide sufficient lap-splice capacity to the bars extending from adjacent panels thereby ensuring continuity.

The introduction of tension hardening materials in structural design as in the examples described in the preceding requires the synergistic contribution of reinforcement with the cementitious fiber reinforced matrix, so as to support actions such as flexural moment and shear. This function is based on the bond stress – slip mechanics that develop when steel reinforcement is embedded in HPFRC. The ABC form of construction described above underscores that a much smaller anchorage length is needed to lap or anchor bars in UHPFRC than in conventional concrete; lengths in the range of $8D_b$ have been proposed as being adequate to yield a bar, however the mode of failure (pullout vs splitting or a combination thereof), and the amount of slip that follows the onset of failure, all of which depend on the UHPFRC cover confinement and strength are still to be determined. At the onset of the slippage of reinforcement due to tension, the deformed surface of the bar imposes bursting pressures on the surrounding cementitious matrix, creating radial tensile stresses and longitudinal compressive stresses. In conventional concrete the magnitude of these stresses is limited due to the very low fracture energy of plain concrete. The UHPFRC

improves the bond performance of the reinforcement-to-concrete interface, by enabling tensile stresses of higher magnitudes to be developed around the bar, and further enhances development capacity thanks to its ductile response.

As the UHPFRC applications in Structural Engineering have recently gained traction, more experimental evidence is necessary, for understanding the mechanisms of the interaction between this class of materials and steel reinforcement. Experimental evaluation of bond and bar development is a key underlying requirement. To this end, various bond test standards are available in the literature, some having been standardized for characterization by the industry of bond properties of new reinforcement products, however embedded in conventional concrete. Most commonly used are the standard pullout (SP) test (Rilem 1978) and beam-end test (BE) (ACI 408). Previous studies have illustrated that the selection of test arrangement is a pivotal matter in the experimental investigation of bond, as the bonded interface is sensitive to lateral restraining effects acting on the bar, as a result of test boundary conditions; vastly different bond slip laws may be extracted keeping all parameters of an experiment the same, except for the test setup. A question of relevance is whether this sensitivity to test setup extends also to the case of tension hardening materials. Extensive investigation of the experimental literature, which is assembled below in the form of a database of the available published tests has shown that this bias is also occurring in the case of tension hardening matrices. This is surprising given the internal confinement of the material, which is imparted by the fibers.

3.1 Assembly of an Experimental Database of Tests investigating bond of bars embedded in Tension Hardening Materials

Georgiou and Pantazopoulou (2018) quantified experimentally the absolute magnitude of the internal confining pressure that arises by virtue of the fibers is of the same order of magnitude as the tensile cracking stress. Therefore, as the bar ribs translate relative to the concrete it is expected that the radial outwards displacement will be to a certain extent restrained by the internal confining pressure whereas the magnitude of the bond strength will be enhanced by the hoop tensile strength of the material which is now sustained to large levels of strain. In light of this mechanistic model for the role of the internal confining pressure, it could be argued that the confining restraint of the setup boundary conditions would not have discernible effects in the case of tension hardening

materials. To explore this question a database was assembled containing bond tests on reinforcement embedded in various tension-hardening materials. Specimens were included from many different investigations that had used one of five distinct experimental test setups, namely, i) the Standard Pullout (SP) test depicted in Fig. 3.1(a), where the concrete block encasing the anchored rebar bears directly on a steel plate during bar pullout; the modified standard pullout (MSP) test was also included, where a Teflon layer is used in the SP setup under the bearing surface to serve as a friction breaker (specimen form depicted in Fig. 3.1(a)); ii) the so-called Beam-End (BE) test depicted in Fig. 3.1(b)), iii) the non-contact lap-splice direct tension pullout test – a test setup wherein a long strip of the HPFRC containing several bars being pulled sequentially, provides partial restraint to expansion in the longitudinal direction as shown in Fig. 3.1(c) (NCLP-DTP) (Tsotsias and Pantazopoulou, 2021) iv) the contact lap-splice direct tension pullout test (CLP-DTP) shown in Fig. 3.1(d), and v) a direct tension pullout test (DTP) depicted in Fig. 3.1(e-f),. The data were organized based on bond strength (f_b) normalized by the square root of the concrete compressive strength (f_c), as it is customary in established standards (ACI 408, Model Code 2010). The normalized bond values are plotted in Figs 3.2 and 3.3, against the normalized anchorage length (L_b / D_b) and normalized cover (C / D_b), respectively.

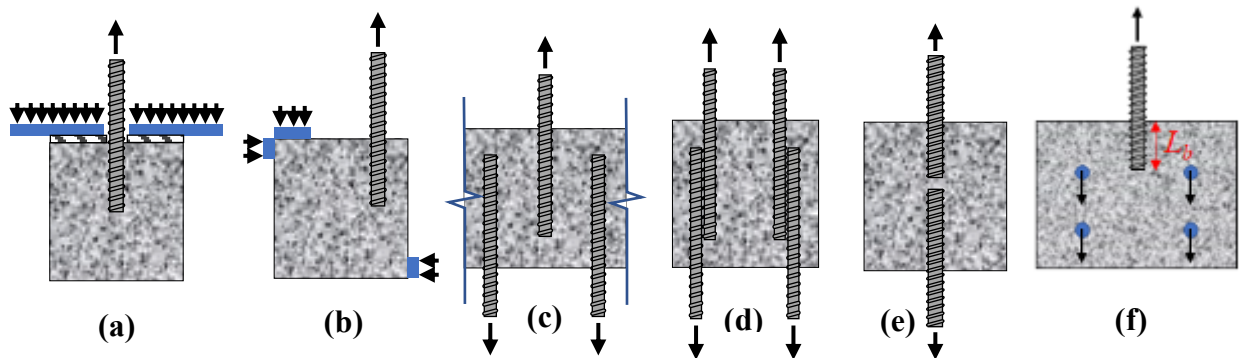


Figure 3.1. Bond test configurations: a) Standard Pullout (SP) Test, b) Beam-End (B-E) Test, c) Non-Contact Lap Spliced DTP (NCLS-DTP) Test [11], d) Contact Lap Splice DTP (CLP-DTP) Test [13], e) Direct Tension Pullout (DTP) Test, (f) Modified DTP Test.

For tension hardening High performance or Ultra High Performance materials (referred to for brevity in the remainder as HPFRCs), information from the literature was collected for 604 specimens, comprising a range of material types according to their compressive strength, i.e., steel-fiber reinforced self-consolidating concretes with compressive strengths in the range of 70 – 200 MPa. Of those, 258 belong to the SP/MSP test category, 24 specimens for the B/BE test, while for

the DTP test 25 specimens were included. For the CLP-DTP and NCLP-DTP tests, 32 specimens were included, containing HPFRC materials. To determine the normalized values, the bond strength measurements were divided by $\sqrt{f_c}$, and the embedment length L_b and cover thickness C values were divided by D_b . It was noted that a vast difference is observed in the obtained normalized values from the different test setups indicating a persistent effect of the specimen form and boundary conditions on the obtained results. The shaded areas in Fig. 3.2a and Fig. 3.2b indicate the pattern of influence of bond strength in terms of the anchorage length and the cover thickness, respectively.

In Fig. 3.2a, the bond strength obtained from tests with HPFRC materials is shown against embedment length of the anchorage. For values of embedment length in the range of $L_b \leq 3D_b$ (short range) bond strengths of the included SP/MSP tests are averaged at $3.52\sqrt{f_c}$, while for values in the range of $3D_b < L_b \leq 10D_b$ (medium range), the average value of bond was $1.6\sqrt{f_c}$. In the case of longer embedment lengths, such as $L_b > 10D_b$, the average bond strength of the specimens included, was $0.85\sqrt{f_c}$. For the B/BE tests, in the short, medium and long embedment length ranges, the average values of bond are measured at $2.35\sqrt{f_c}$, $2.4\sqrt{f_c}$, and $0.67\sqrt{f_c}$, respectively. For the DTP tests included in this database, bond strengths were measured for short and medium embedment lengths, with average bond strengths at $2.0\sqrt{f_c}$ and $1.45\sqrt{f_c}$, respectively. For the CLP-DTP and NCLP-DTP tests, studies containing medium and long embedment lengths were included. In the former case, the average bond strength was $0.75\sqrt{f_c}$ and $2.5\sqrt{f_c}$, respectively, while for the latter, the values are $0.80\sqrt{f_c}$ and $1.5\sqrt{f_c}$. Based on the above data, for the short range of embedment lengths, higher bond strengths are obtained for the SP/MSP test setup (Fig.3.1a), while the lowest are shown in the DTP test (Fig. 3.1e-f). In the medium range, the B/BE and NCLS-DTP tests show the highest bond, while the CLS-DTP tests show the lowest average value. In the range of long embedment lengths, the highest and lowest average values are obtained by the NCLS-DTP and B/BE tests, respectively.

In Fig. 3.2b, the same bond tests on HPFRC materials are depicted against normalized cover C / D_b . For the SP/MSP test, for cover in the range of $C \leq 2D_b$ (small covers), average bond strength was measured at $1.70\sqrt{f_c}$, while for $2.0 < C / D_b \leq 4.0$ (medium cover thickness), bond was $2.65\sqrt{f_c}$. For cover in the range of $C > 4D_b$, bond strengths averaged at $3.25\sqrt{f_c}$. For the B/BE test, in the short range of normalized D_b cover, the average bond strength was $2.23\sqrt{f_c}$. For the DTP test, for the short and medium ranges, bond strength was measured at $1.37\sqrt{f_c}$ and $4.60\sqrt{f_c}$, respectively. For

the CLS-DTP and NCLS-DTP tests, for the short range of normalized cover, bond strength was measured at $0.72\sqrt{f_c}$ and $1.87\sqrt{f_c}$, respectively, while for the medium range of cover, the values were $0.9\sqrt{f_c}$ and $2.41\sqrt{f_c}$, respectively. It was observed that for smaller cover thickness, the highest bond was obtained by the B/BE tests (Fig. 3.1b), followed by the NCLS-DTP tests (Fig. 3.1c), while the lowest boundary of bond strength was obtained from the CLS-DTP tests (Fig. 3.1d). For the medium range of the cover thickness, the highest values were obtained from the DTP test, while the lowest values corresponded to the CLS-DTP test.

For this ECC materials, a total of 154 specimens were included, with materials fabricated with PVA fibers and compressive strengths in the range of 40 – 80 MPa. Of those, 110 belong to the SP/MSP test category, 38 specimens for the B/BE test, while for the DTP test 4 specimens were included.

In Fig. 3.3a, the bond strength obtained from tests with ECC materials is shown against embedment length of the anchorage. For values of embedment length in the range of $3D_b < L_b \leq 10D_b$, bond strengths of the included SP/MSP tests are averaged at $1.5\sqrt{f_c}$, while for values in the range of $L_b > 10D_b$ the average value of bond was $2.0\sqrt{f_c}$. For the B/BE tests, in the medium embedment length range of $3D_b < L_b \leq 10D_b$, the average of the values of bond was measured at $1.1\sqrt{f_c}$. For the DTP tests, bond strengths in the same the medium embedment lengths range, were averaged at $0.7\sqrt{f_c}$. It can be noted here that the SP/MSP provides the highest values for bond strength (Fig. 3.1a), for the various embedment lengths. The B/BE test (Fig. 3.1b) arrangement provides values approximately equal to $\sqrt{f_c}$, while the lowest bound was provided by the DTP test (Fig. 3.1e-f).

In Fig. 3.3b, the same bond tests on ECC materials are depicted against normalized cover C / D_b . For the SP/MSP test, for cover in the range of $C \leq 2D_b$ (small covers), average bond strength was measured at $1.23\sqrt{f_c}$, while for $2.0 < C / D_b \leq 4.0$ (medium cover thickness), bond was $1.46\sqrt{f_c}$. For cover in the range of $C > 4D_b$, bond strengths averaged at $2.71\sqrt{f_c}$. For the B/BE test, in the short and medium ranges of normalized D_b cover, the average bond strength was $1.0\sqrt{f_c}$ and $1.36\sqrt{f_c}$, respectively. For the DTP test, for the short range of normalized cover, bond strength was measured at $0.7\sqrt{f_c}$. The same pattern as denoted above was observed for the ECC materials, where the SP/MSP tests provide the least conservative values for normalized bond strength. Here, the B/BE test provides values equal to $\sqrt{f_c}$ for the smallest cover thicknesses, while for the same range, the DTP provides the lowest bound of bond strength.

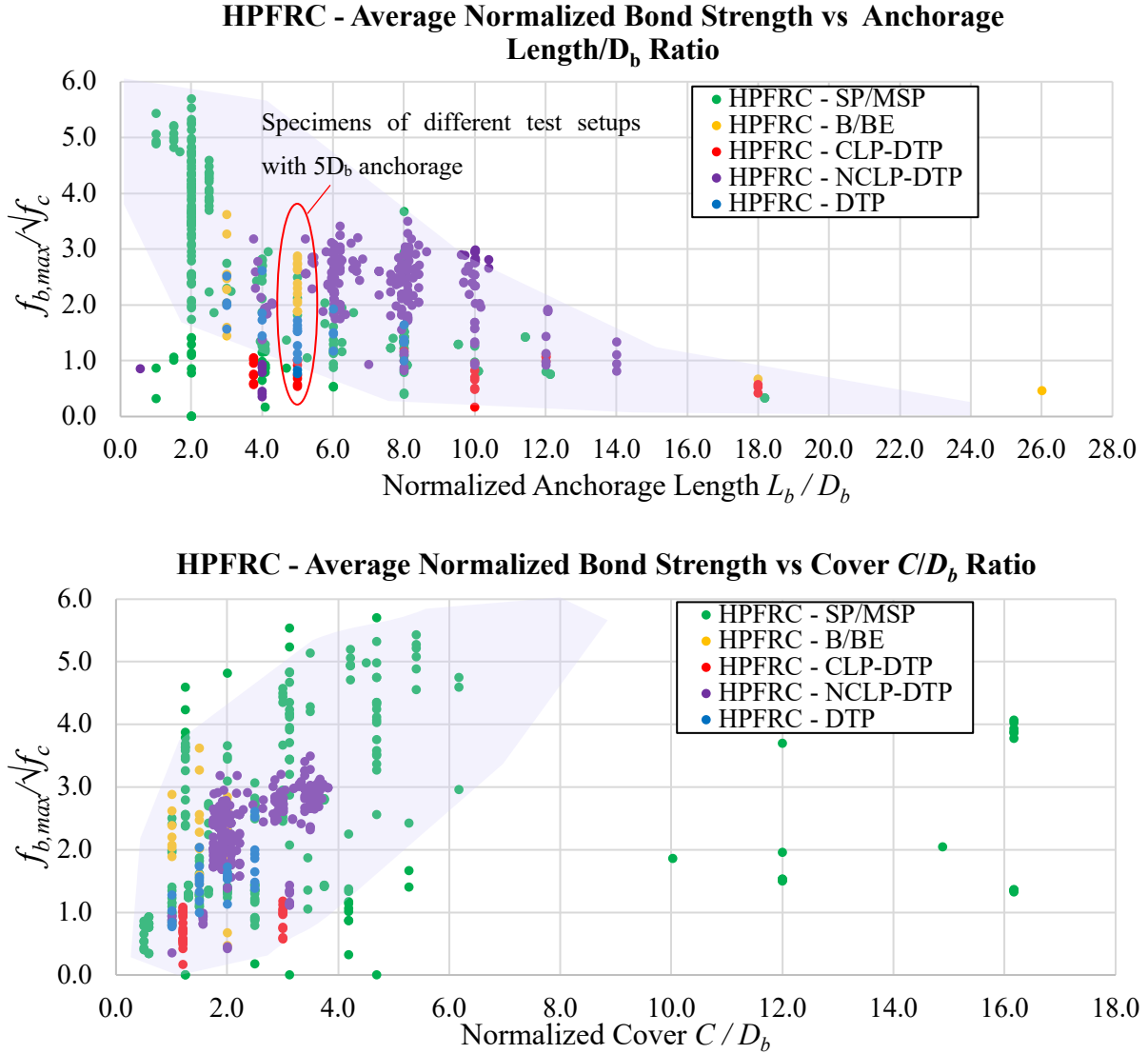


Figure 3.2. Database of HPFRC Bond Tests: (a) Effect of anchorage length and (b) Effect of concrete cover.

Based on all the above, including the databases for both the HPFRC and ECC materials, it was clearly shown that the effect of spurious confinement stresses contributes to the experimental bond measurements, for both material types and affects the magnitude of bond that can be counted on for calculating bar development and strain penetration effects. Setup bias was more pronounced for conventional bond test setups, such as the SP test, where the concrete matrix surrounding the bar was enveloped in compressive stress fields. For test setups where the concrete matrix is enveloped in tensile stresses, as was the case with the CLP, NCLP and DTP tests, the contribution of lateral stresses was diminished, and the lowest bounds of bond strength are obtained. Therefore,

the patterns obtained from the database confirm the sensitivity of measured bond based on the type of test setup.

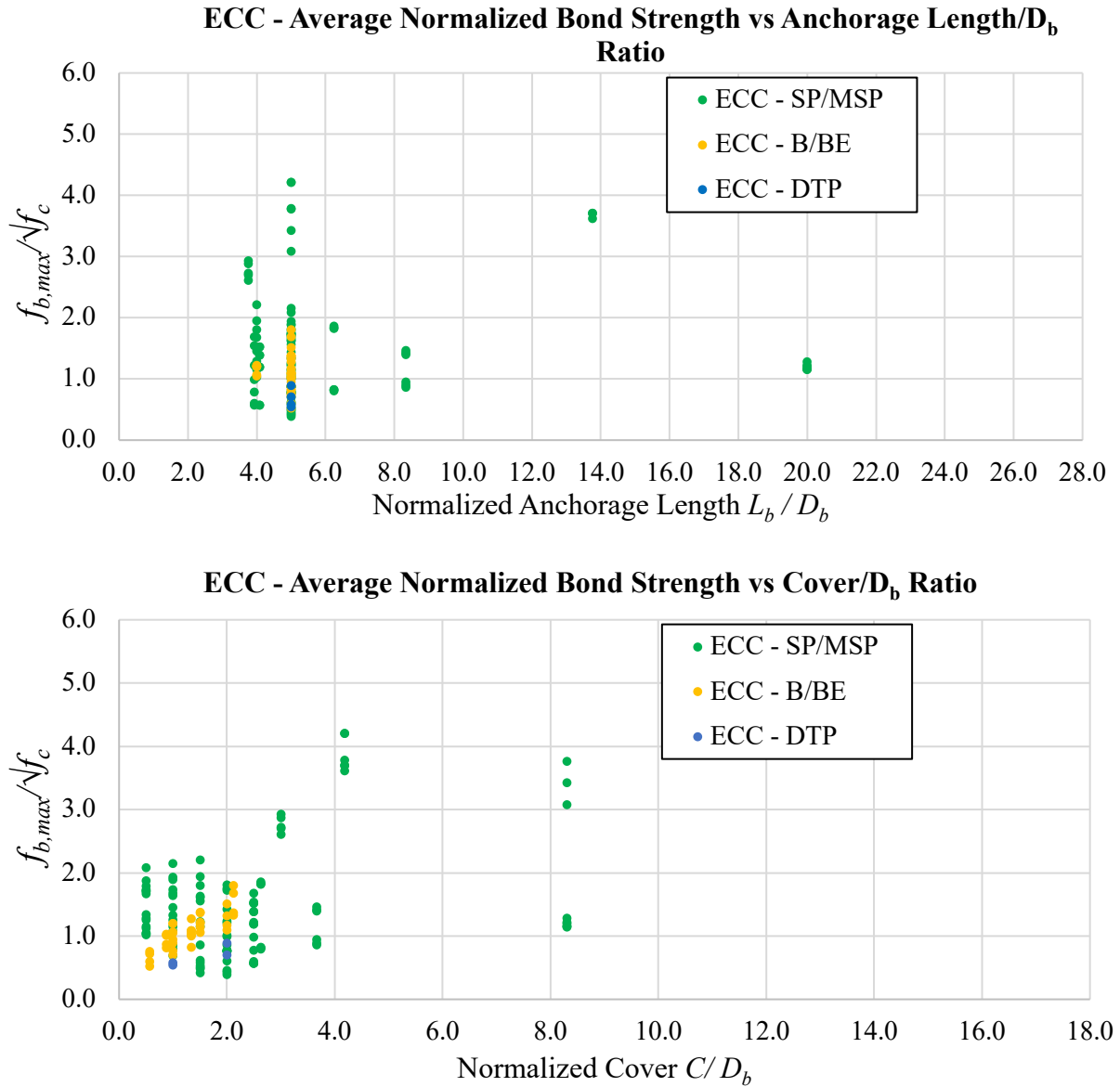


Figure 3.3. Database of ECC Bond Tests: (a) Effect of anchorage length and (b) Effect of concrete cover.

Lastly, for the case of the BE test, where bond is measured under the flexural response of the test bar, the obtained bond strengths were shown to exceed the ones obtained from direct tension tests, such as the CLP and DTP tests. This was attributed to the contribution of curvature on the test bar, which further improves bond performance. This effect was further studied in an experimental setting, in the next section below, where DTP specimen tests are compared with anchored four-point bending tests.

3.2 Comparison between Flexural and Direct Tension Tests on Bars

Experimental evidence from setups that yield conservative estimates of bond strength is considered in the present section: two series of bond tests on bars embedded in HPFRC mortars conducted in parallel in the High Bay lab using the same batched materials (Saikali 2019, Tsiotsias 2019) are used here to explore the differences between the anchored Beam (B) and the DTP test, respectively. A total of 13 and 9 specimens were fabricated, for each test setup, respectively. The HPFRC material properties were obtained from characterization tests as shown in Table 3.1 (Pantazopoulou et. al 2019). In Table 3.1, f_c is the compressive strength, f_f the tensile strength obtained from flexural tests, and f_{cr} and f_t the cracking and tensile strengths obtained from direct tension tests. In both sets of experiments, short embedment lengths of $5d_b$ were assumed, with the aim of being able to approximate local bond strength by averaging the bond stress values along the length of the anchorage, as per the following equation:

$$f_b = F_b / \pi D_b L_b \quad (3.1)$$

In Eq. 3.1, f_b is the bond stress, F_b is the force acting on the loaded end of the anchorage, D_b is the diameter, and L_b is the embedment length of the anchorage. Parameters of this study are the HPFRC material type and cover thickness ($1D_b$ or $2D_b$).

Table 3.1. Material parameters, fiber content and mechanical properties.

Material ID	Fiber Type (Vol. content %)	Fiber Properties	Fiber Count (x 10^6 m ³)	f_c / f_f (MPa)	f_{cr} / f_t (MPa)	ϵ_{cr} / ϵ_{tu} (x 10^{-4})
Commercial Mat. K	Steel (2%)	Straight; $l_o = 13\text{mm}$; $d_f = 0.2\text{mm}$	49.0	122.6 30.1	10.6 13.5	3.7 47.4
Commercial Mat. F	Steel (2%)	50% Straight: $l_f: 20\text{ mm}$; $d_f: 0.2\text{ mm}$ 50% Hooked: $l_f: 25\text{ mm}$; $d_f: 0.3\text{ mm}$	16 / 5.7	128.4 35.3	11 16	6.1 120
In-House Mat. R	Steel (2.5%)	Straight; $l_o = 13\text{mm}$; $d_f = 0.2\text{mm}$	61.2	125 26.4	9.2 10.5	3.2 47

3.2.1 Direct Tension Pullout (DTP) Test

The DTP test was designed and conducted using the setup depicted in the schematic of Fig. 3.2a (Tsiotsias 2019). In this version of the test, coaxial support to the test bar was provided through an

external mechanical base, with a rotating convex fastener on the support bar, as shown in Figs. 3.4a-c. The support bar was allowed to rotate on the surface of the steel plate to ensure alignment with the test bar. The base was attached to the specimen with four transverse rods, penetrating through the specimen and covered with custom steel sleeves. This arrangement was selected to minimize potential eccentricities between the test and support bars during casting and to reduce the size of the specimen. Further discussion on the test design may be found in Tsiotsias and Pantazopoulou (2021). To obtain the pullout slip relative to the concrete, a pair of linear potentiometers were attached to the steel base, as shown in Fig. 3.4d.

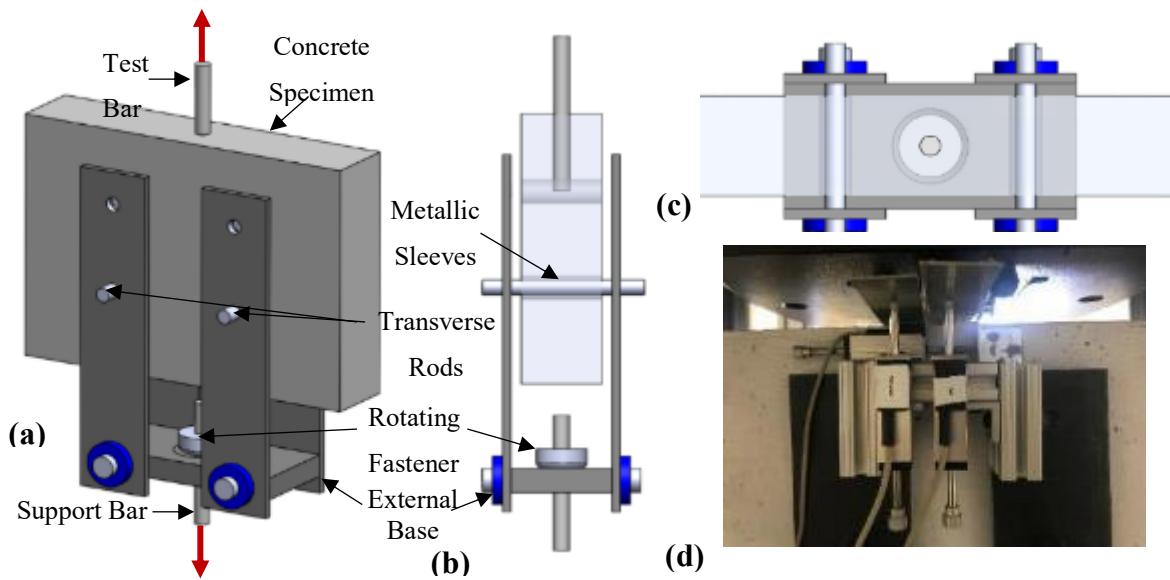


Figure 3.4. Modified version for DTP test: (a) Overview of setup; (b) Side view of specimen and metallic base; (c) Top view of setup; (d) Attachment of DTs to obtain bar slip.

3.2.2 Beam (B) Test

For the beam test, an anchored beam specimen was used, as shown in Saikali (2019). This flexural test consists of a beam specimen subjected to four-point bending with symmetrical load, in which the test bar was anchored within the constant moment zone, as shown in Fig 3.5a. To create the start of the bond embedment length, a notch was created by a foam insert in the formwork. On one side of the notch the test bar was fully anchored, but on the other side, only a length of $5D_b$ was embedded – the rest of the bar extended to the end of the specimen, but it was covered in a PVC tube as bond breaker. The symmetric loads were placed so that the embedment length was within the constant moment region. The bar protruded beyond the end of the specimen where the

longitudinal translation of the free end was measured with a DT. In this design, two auxiliary bars were placed on each side of the test bar, to eliminate flexural failure of the segment beyond the end of the embedment. The section of the specimen at the test zone is shown in Fig. 3.5b. Loading was applied with rollers, attached using a custom steel jig, as shown in Fig. 3.5c.

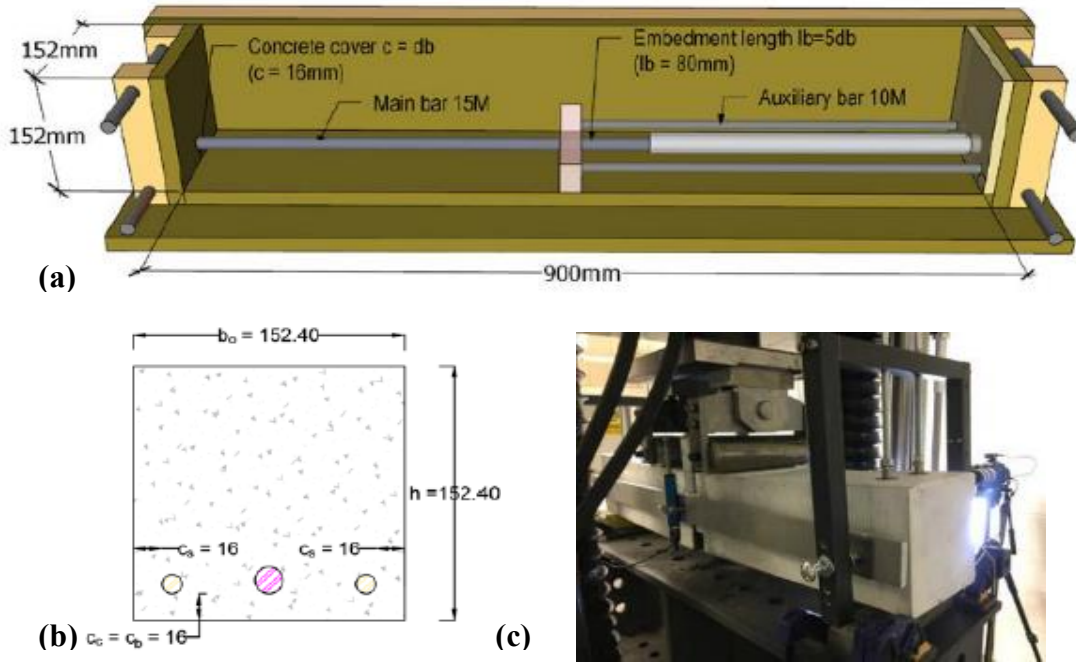


Figure 3.5. Schematic of the AFPB test specimen: (a) Formwork and sample specimen dimensions; (b) Specimen section; (c) Placement of AFPB specimen in loading apparatus.

3.2.3 Comparison of the Test Results

In Figs. 3.6-8, the cumulative results from both the Anchored Beam and DTP tests are shown. The tabulated data are shown in Table 3.2. Both types of tests show comparable results in the maximum obtained bond stress. For material K, the specimens with cover thickness of $C_c = 1D_b = 16$ mm, showed an average bond stress in each type of test of 13.4 MPa and 8.9 MPa, respectively. For the case of the cover $C_c = 2D_b = 32$ mm for the same material, the average bond was 11.2 MPa and 14.6 MPa, respectively for each type of test. For material F, the specimens with cover thickness of $C_c = 1D_b = 16$ mm, showed an average bond stress in each type of test of 20.2 MPa and 13 MPa, respectively. For the case of the cover $C_c = 2D_b = 32$ mm for the same material, the average bond was 17.85 MPa and 18.44 MPa, respectively for each type of test. For material R, the specimens with cover thickness of $C_c = 1D_b = 16$ mm, the average bond stress in the Anchored Beam test was

16 MPa. For the case of the cover $C_c = 2D_b = 32$ mm for the same material, the average bond was 19.2 MPa and 18.2 MPa, for the Anchored Beam and DTP tests, respectively. It can be observed that the Anchored Beam tests shows higher bond strength for the cover thickness of $C_c = 1D_b = 16$ mm. The opposite however was observed for specimens for $C_c = 2D_b = 32$ mm, where bond strength was higher in the DTP test, with the exception of the in-house material IR.

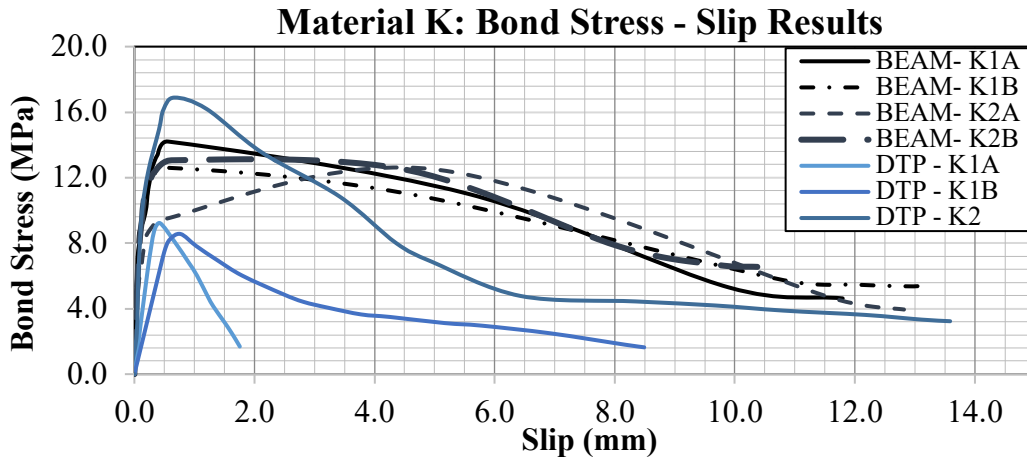


Figure 3.6. Bond Stress – Slip results for Anchored Beam and DTP tests.

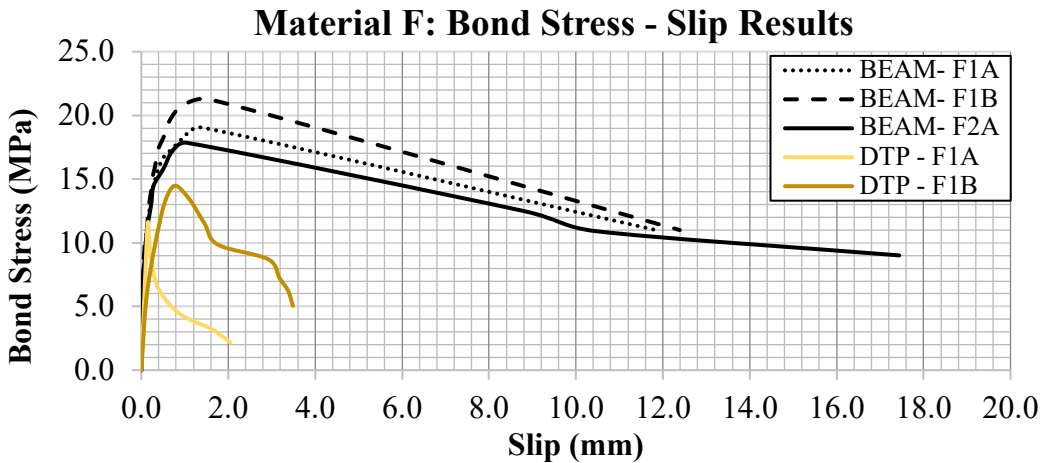


Figure 3.7. Bond Stress – Slip results for Anchored Beam and DTP tests.

Material IR: Bond Stress - Slip Results

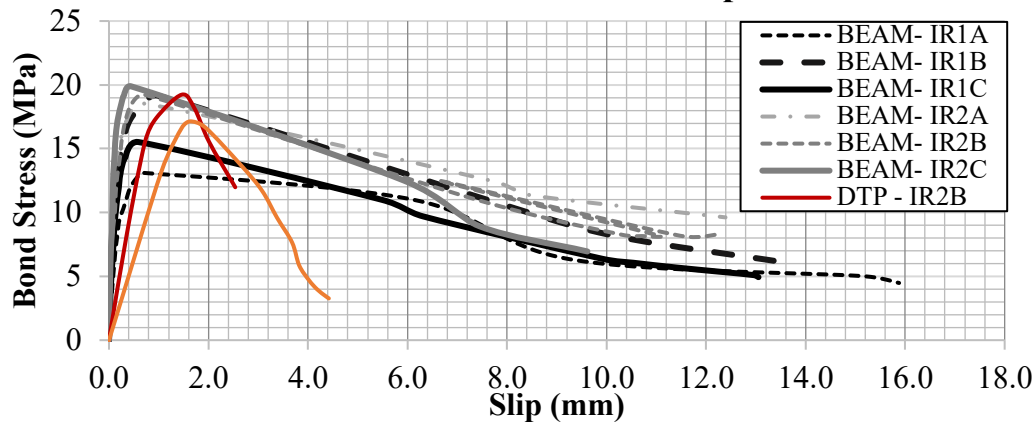


Figure 3.8. Bond Stress – Slip results for Anchored Beam and DTP tests.

Table 3.2. Tabulated values of test results for the anchored Beam and DTP tests.

Specimen ID	Cover (mm)	F_{peak} (kN)	$\Delta_{@peak}$ (mm)	$f_{b,peak}$ (MPa)	$f_{b,peak}/\sqrt{f_c}$	slip@peak (mm)	Failure Mode
BEAM – K1A	16	75.1	1.62	14.21	1.28	0.56	P-S
BEAM – K1B	16	69.7	1.54	12.61	1.14	0.52	P-S
BEAM – K2A	32	56.4	1.50	9.39	0.85	0.42	P-S
BEAM – K2B	32	68.7	1.86	13.06	1.18	0.62	P-S
DTP – K1A	16	37.10	3.67	9.22	0.83	0.43	S
DTP – K1B	16	34.41	3.77	8.55	0.77	0.77	P-S
DTP – K2A	32	50.30	4.00	12.5	1.13	-	P-S
DTP – K2B	32	67.91	5.43	16.89	1.53	0.67	P-S
BEAM – F1A	16	92.1	2.96	19.05	1.68	1.39	P-S
BEAM – F1B	16	101.0	3.24	21.25	1.88	1.58	P-S
BEAM – F2	32	83.7	3.13	17.85	1.58	1.04	P-S
DTP – F1A	16	46.63	5.37	11.6	1.02	0.15	S
DTP – F1B	16	58.11	5.10	14.45	1.28	0.74	P-S
DTP – F2	32	74.14	5.92	18.44	1.63	-	P-S
BEAM – IR1A	16	72.6	2.00	13.10	1.17	0.81	P-S
BEAM – IR1B	16	91.5	2.17	19.15	1.71	0.89	S
BEAM – IR1C	16	79.5	1.78	15.52	1.39	0.58	S
BEAM – IR2A	32	80.3	2.04	18.53	1.66	0.71	P-S
BEAM – IR2B	32	83.3	2.61	19.21	1.72	0.73	P-S
BEAM – IR2C	32	86.0	2.20	19.89	1.78	0.44	P-S
DTP – IR2A	32	77.34	5.10	19.23	1.72	1.46	S
DTP – IR2B	32	68.92	4.94	17.13	1.53	1.63	S

Note: P = Pullout; S = Splitting; P-S = Pullout and splitting. 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi; 1 kN = 0.2248 kips.

In addition, in almost all specimens concurrent splitting and pullout failure were observed, with multiple branches of cracks appearing on the surface of the concrete. Compared to the beam

specimens, DTP tests show steeper softening branches and less post-peak resilience and energy absorption. The difference was clearly owing to the different state of stress in the two setups; note that the curvature which occurs in the beam, particularly in the larger deflection levels, is a passive but important source of bond strength resistance which has not been accounted for explicitly to date.

3.3 Finite Element Analysis (FEA) of Bond Tests

To simulate and better understand the confining influence of boundary conditions on the development capacity of reinforcement and the resulting bond-slip relationship, a series of bond test setups were modeled using the combined GiD/ATENA Studio software package (ATENA Program Documentation, 2020). The various setups were chosen to represent the tests encountered in the literature and include the tests described in the Database of section 3.1. Among the tests included are the standard pullout test (SP), the beam-end test (BE), the contact lap-splice pullout test (CLSP, or CLP, for short), the non-contact lap-splice pullout test (NCLSP, or NCLP, for short), the restrained non-contact lap-splice pullout test R-NCLP), which consists of a variation of the NCLP test where specimens are fabricated in continuous concrete strips, and the DTP test, in the variation of the test described in Tsotsias and Pantazopoulou 2021. Schematics of the model sections, for each type of test, are shown in Fig. 3.9.

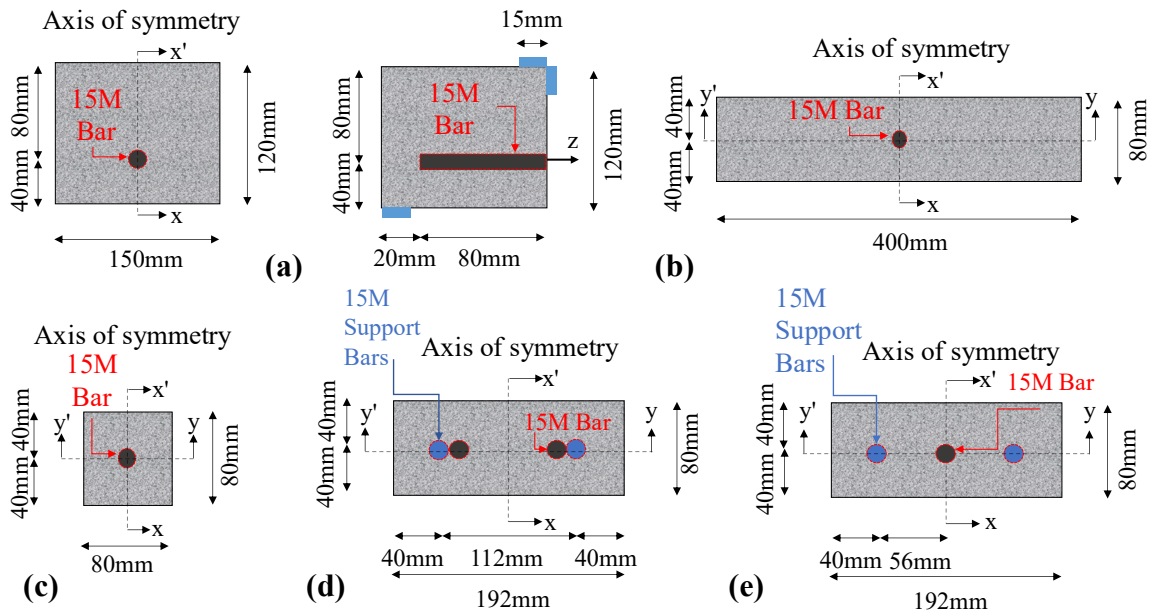


Figure 3.9. Model sections for the various types of bond tests: (a) Beam End (BE); (b) Direct Tension Pullout (DTP); (c) Standard Pullout (SP); (d) Contact Lap Splice (CLSP); (e) Non-Contact Lap Splice (NCLP/R-NCLP).

For better comparison all other non-parametric details are therefore kept the same. Parameters of this analysis consist of the anchorage length and cover thickness – the former was considered at values of $5D_b$ and $10D_b$, and the latter at values of $1D_b$ and $2D_b$.

3.3.1 The ATENA-GiD Interface

The modeling segment of the FEA was conducted using the commercial software GiD©. For this program, the process of modeling supports incremental definitions of geometrical properties, i.e., definitions of points, lines, surfaces and then volumes, which facilitates the modeling process. Once the three-dimensional geometrical entities are defined (i.e., volumes), the material properties and support conditions are applied to their respective entities, as well as the interval data, which contain information about the loading stages. Once the pre-processing is complete, the model is “sent” to ATENA Studio for analysis and post-processing.

Both the reinforcement and concrete were modeled using 3D solid elements, in order to capture lateral stresses that develop due to the boundary conditions of the test. Bar ribs are modeled directly on the bar surface, as one continuous solid (Fig. 3.10). The concrete body was modeled around the bar surface and double symmetry was considered on two orthogonal planes going through the bar longitudinal axis in order to reduce the computational effort and improve robustness of the solution. The geometry of the bar element was adapted from 15M reinforcing bar sample from the lab facilities and was normalized, as shown in Fig. 3.10. The mesh was selected such that the elements were able to fit into the bar and concrete macroelements, without imposing undue computational load. It was determined, through trial and error, that the optimal size was 0.05 m.

For the analysis, displacement control with the Newton-Raphson solution algorithm was implemented, with 30 iterations per step. Displacement was applied on the top surface of the element, on the loaded end of the bar.

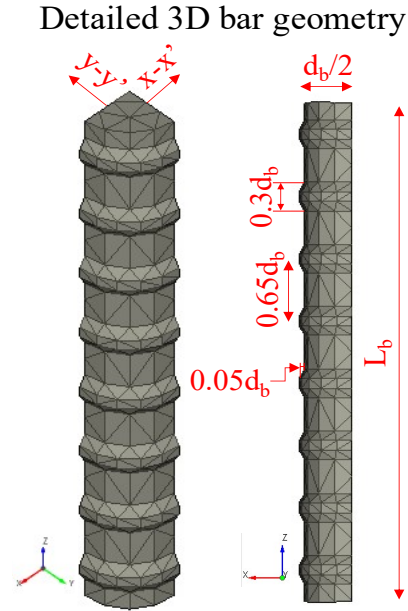


Figure 3.10. Surface profile of the modeled bar.

3.3.2 Material parameters

To model HPFRC concrete, the material model *Cementitious2 SHCC/HPFRC* was used, with $f'_c = 120$ MPa, $f_t = 7.5$ MPa and $E = 45$ GPa. Nonlinearity was considered in the functions for tension and compression in the HPFRC material, as shown in Tables 3.3 and 3.4. The non-linear functions were obtained through calibration of the model to the experimental data of the DTP tests (Tsiotsias 2019), which are shown in Section 3.2. Material K was modeled, which contained steel fibers with diameter $d_f = 0.2$ mm and length $l_o = 13$ mm, at a fraction of 2%. For the bar, an elastic stress-strain material model was used, where $E = 200$ GPa and $\mu = 0.3$, to ensure that failure in the models occurs due to pullout (and not yielding of the bar).

Table 3.3. Tension function for HPFRC material model.

Coordinate ID	ε_f	σ_t/f_t
1	0	1
2	0.00002	0.6
3	0.00005	0.2
4	0.0001	0.125
5	0.01	0.075
6	0.12	0.0

Table 3.4. Compression function for HPFRC material model.

Coordinate ID	ε_{pl}	σ_c/f_c
1	-0.01	0
2	-0.002	0.8
3	0	1

3.3.3 Results

The results of all the analyses are shown in tabulated form Table 3.5, while in Figs. 3.11-3.14, the strength curves for each parametric scenario are shown for the various test setups. In Table 3.5, the model ID includes the specimen type, the normalized cover and normalized anchorage length, for example, model “SP-1C-5L” corresponds to the model with the Standard Pullout specimen configuration, cover thickness of $C_c = 1D_b$, and anchorage length of $L_b = 5D_b$. For the specimen types, BE corresponds to the Beam-End test, CLP to the Contact Lap Splice Direct Tension Pullout test, NCLP to the Non-Contact Lap Splice Direct Tension Pullout test, R-NCLP to the Restrained Non-Contact Lap Splice Direct Tension Pullout test, and DTP to the Direct Tension Pullout test. The responses of the DTP-1C-5L and DTP-2C-5L models were obtained in accordance with the bond stress- slip relationships of the DTP1-K1 and DTP-K2 specimen cases, as shown in Table 3.1, respectively.

For the parametric scenario of $C_c = 1D_b$ and $L_b = 5D_b$, it can be seen that the SP and R-NCLP tests result in the upper bound in the force vs displacement curves. While in both cases, the bar force was reduced with a steep slope after the peak, in the latter case shows a ductile post-peak response, for up to 6.8 mm. The BE test represents a medium range of the force vs displacement curves, with a ductile post-peak response as well. The lowest bounds are occupied by the CLP, NCLP and DTP tests, respectively. The CLP test shows an immediate loss of the bar force at post-peak, with no ductility in the descending branch. In the NCLP and DTP case, a prolonged post-peak response was observed, up to 4.25 mm and 8.125 mm, respectively. Lastly, in the DTP case, the maximum bar force was the lowest of the lower bound responses, at 65% and 73% the bar force of the CLP and NCLP tests.

For the parametric scenario of $C_c = 1D_b$ and $L_b = 10D_b$, the highest bounds of bar force were shown for the SP, BE and R-NCLP tests. With the exception of the SP test, the responses of these tests were ductile in the post-peak range. The CLP and NCLP tests showed higher strength than the

DTP test, at values approximately 153% and 126% the bar force of the DTP test, while all three tests showed a ductile post-peak response.

Table 3.5. Summary of results from FEA of bond pullout tests.

Model ID	Bond Test Setup	Type of Bar	Cover (mm)	Embedment Length (mm)	F_{peak} (kN)	D_{peak} (mm)	Average f_b (MPa)	Failure Type
SP-1C-5L	Standard Pullout	15M	16	80	185.4	0.3	46.11	P
SP-2C-5L			32	80	226.3	0.39	56.28	P
SP-1C-10L			16	160	195.6	0.375	24.32	P
SP-2C-10L			32	160	334.8	0.89	41.63	P-S
BE-1C-5L	Beam-End	15M	16	80	108.9	0.5	27.08	P-S
BE-2C-5L			32	80	121.5	0.525	30.21	P-S
BE-1C-10L			16	160	215.8	0.775	26.83	P-S
BE-2C-10L			32	160	244	0.875	30.34	P-S
CLP-1C-5L	Contact Lap Splice Pullout	15M	16	80	53	0.25	13.18	S
CLP-2C-5L			32	80	84.3	0.325	20.96	P-S
CLP-1C-10L			16	160	131	0.575	16.29	P-S
CLP-2C-10L			32	160	163.4	0.75	20.32	P-S
NCLP-1C-5L	Non-Contact Lap Splice Pullout	15M	16	80	47.4	0.25	11.79	P-S
NCLP-2C-5L			32	80	58.8	0.35	14.62	P-S
NLCP-1C-10L			16	160	116.4	0.55	14.47	P-S
NCLP-2C-10L			32	160	156.4	0.75	19.45	P-S
R-NCLP-1C-5L	Restrained	15M	16	80	194.8	0.65	48.44	P-S
R-NCLP-2C-5L	Non-Contact		32	80	230.5	0.725	57.32	P-S
R-NLCP-1C-10L	Lap Splice		16	160	272.3	1.125	33.86	P-S
R-NCLP-2C-10L	Pullout		32	160	400.5	1.5	49.80	P-S
DTP-1C-5L	Direct Tension Pullout	15M	16	80	34.7	0.35	8.63	P-S
DTP-2C-5L			32	80	58.4	0.475	14.52	P-S
DTP-1C-10L			16	160	51.6	0.55	6.42	P-S
DTP-2C-10L			32	160	67.6	0.5	8.41	P-S

Comparison of Resistance Curves (1C -5L)

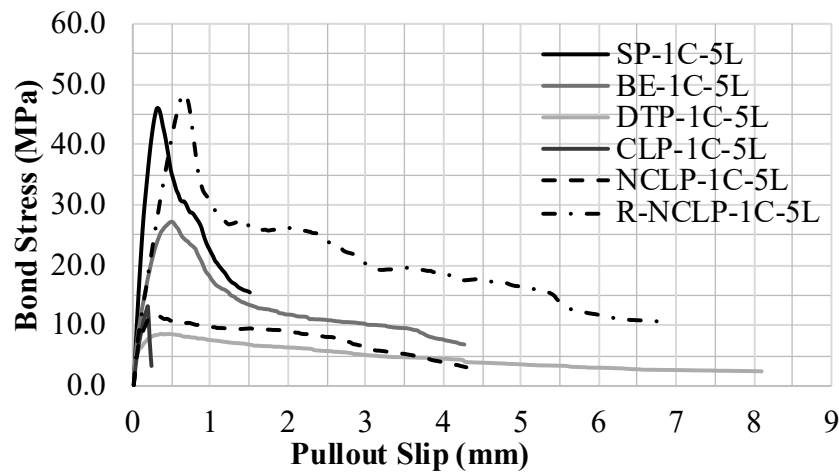


Figure 3.11. Resistance curves of the various models for the parametric scenario 1C-5L.

Comparison of Resistance Curves (1C -10L)

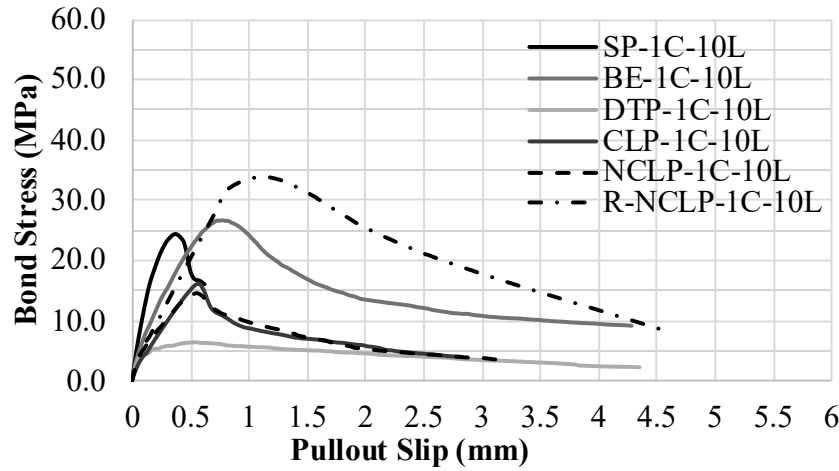


Figure 3.12. Resistance curves of the various models for the parametric scenario 1C-10L.

For the parametric scenario of $C_c = 2D_b$ and $L_b = 5D_b$, the highest range of bar force development was attained for the SP and R-NCLP tests. The medium range of developed bar force values was represented by the BE test, which showed also a ductile post-peak response. The lowest bounds of developed bar force values were obtained for the CLP, NCLP and DTP tests, with the latter two showing approximately identical post-peak responses, up to 4.1 mm of pullout slip. Compared to the NCLP and DTP tests, the CLP tests showed a steeper descending branch, and corresponding loss of strength, in the post-peak range.

Comparison of Resistance Curves (2C -5L)

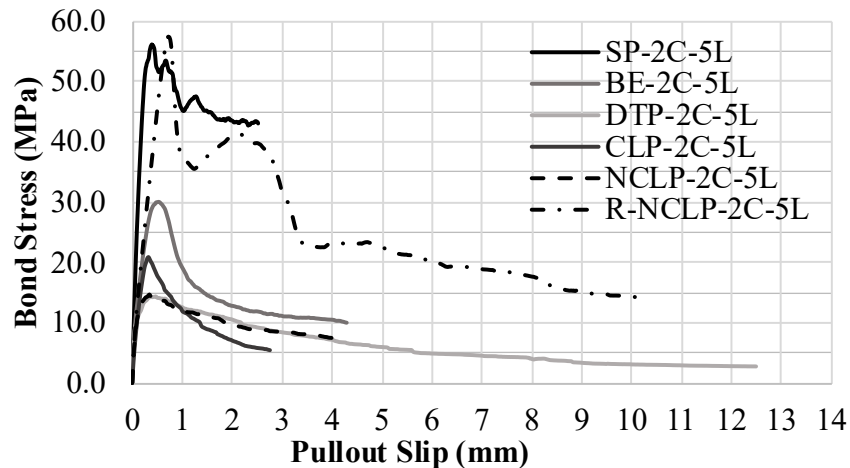


Figure 3.13. Resistance curves of the various models for the parametric scenario 2C-5L.

Comparison of Resistance Curves (2C -10L)

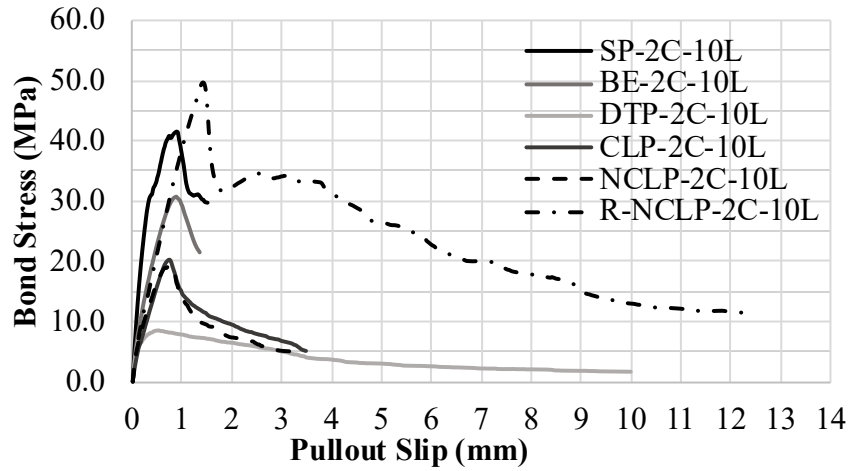


Figure 3.14. Resistance curves of the various models for the parametric scenario 2C-10L.

For the parametric scenario of $C_c = 2D_b$ and $L_b = 10D_b$, the highest bar force development was shown for the SP and R-NCLP tests. The medium range of bar force development was obtained for the BE test, whereas the lowest range was obtained from the CLP, NCLP and DTP tests. The CLP and NCLP tests showed an almost identical ascending branch, while in the post-peak range, the NCLP tests showed faster reduction of the developed bar force. The lowest value of bar force was obtained for the DTP test for this case.

Based on the above, it was observed that the bar forces developed from the SP and R-NCLP tests showed the highest sensitivity to the anchorage length and cover thickness. The latter test showed more sensitivity to the anchorage length, with the bond strength increasing by 40%, with the increase to the $L_b = 10D_b$ length. The BE test also showed higher sensitivity to the anchorage length, with the increase of the $L_b = 10D_b$ length leading to approximately double the bar force from the control case. Similarly, the CLP and NCLP tests showed approximately the same increase of developed bar force, by 145%, with change of the anchorage length to $L_b = 10D_b$. On the contrary, the DTP test showed more sensitivity to the cover thickness, with the increase of bar force, for the cover thickness of $C_c = 2D_b$ at 68%.

3.3.3.1 Stress distributions for the various models

The stress states of the concrete around the bar, for the various types of models are shown in Figs. 3.15-3.20. The control case models with $C_c = 1D_b$ and $L_b = 5D_b$ are shown in the present section,

while the remaining cases are included in Appendix A. The stress states are shown for the peak longitudinal (bar) axis, at the peak load and the end of the analysis, as well as the stress and strain of concrete around the bar at the peak load, in the longitudinal and transverse directions. Lastly, the shear stress τ_{xz} (or τ_{yz}) is shown for the concrete around the bar, at the peak load, which represents the equivalent bond stress acting on the bar-concrete interface. The observations collected from the analyses are discussed in Section 3.3.3.2.

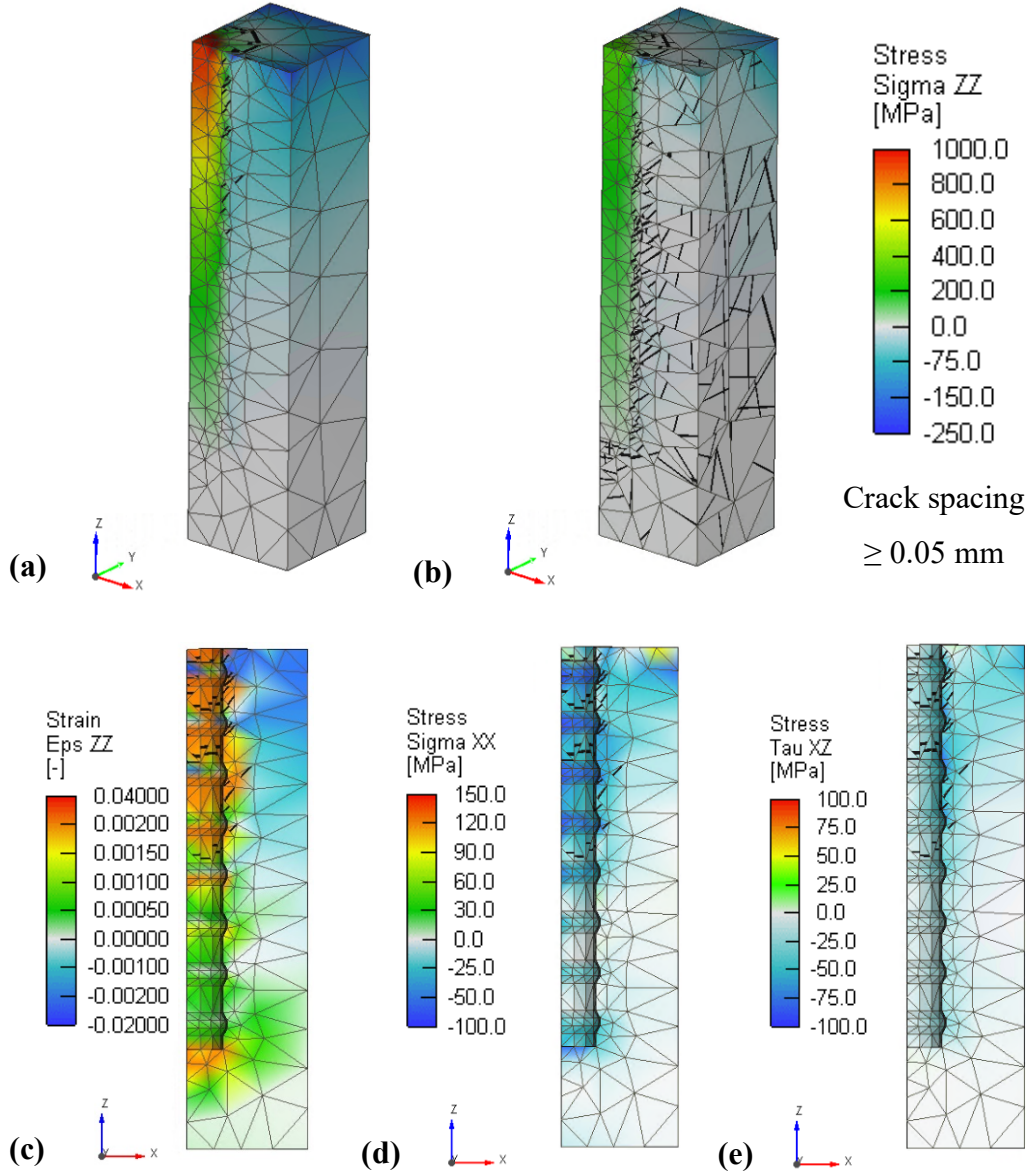


Figure 3.15. Finite element model for SP-C1-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{xx} at peak; e) Concrete Stress τ_{xz} at peak.

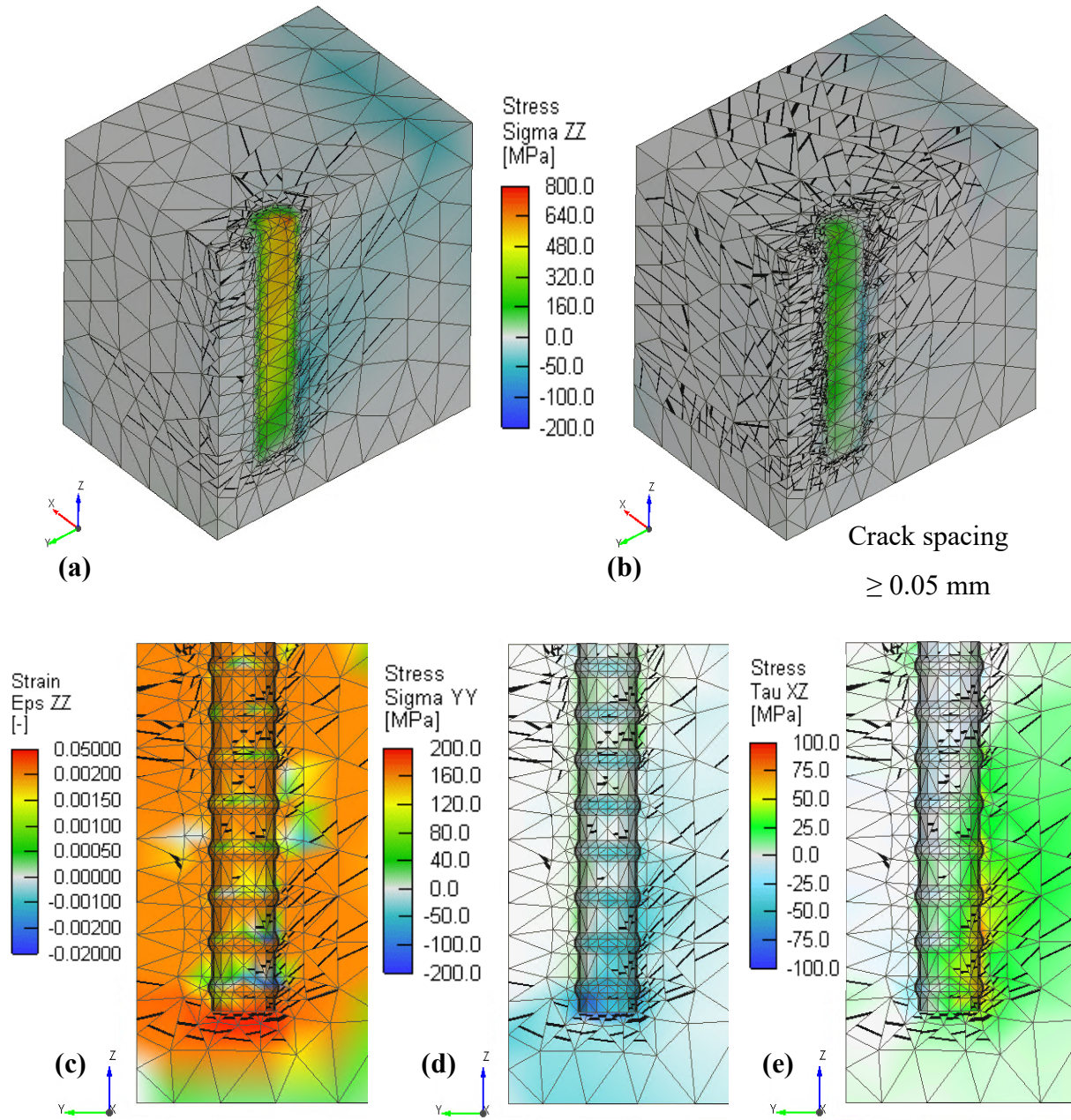


Figure 3.16. Finite element model for BE-C1-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

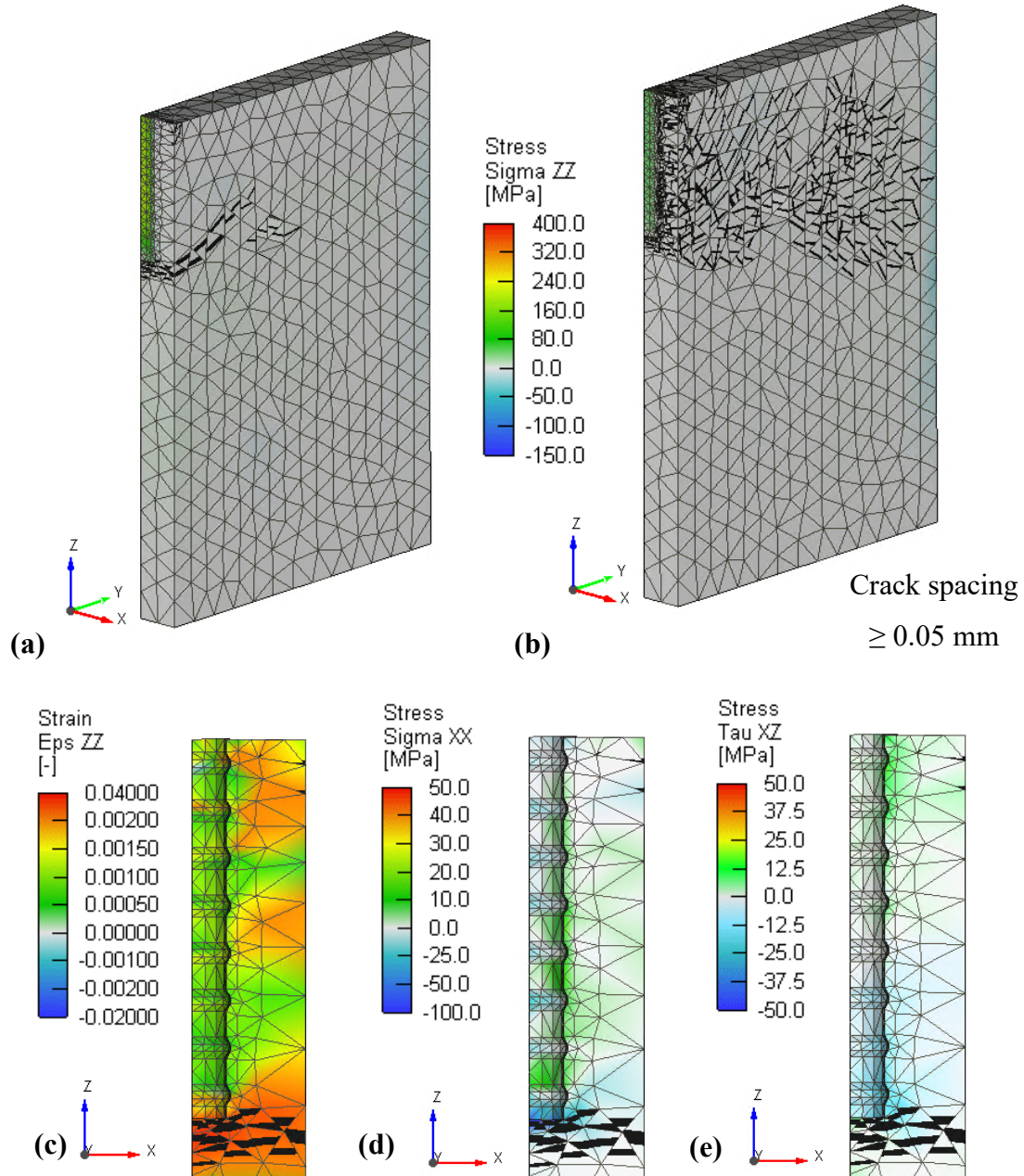


Figure 3.17. Finite element model for DTP-C1-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{xx} at peak; e) Concrete Stress τ_{xz} at peak.

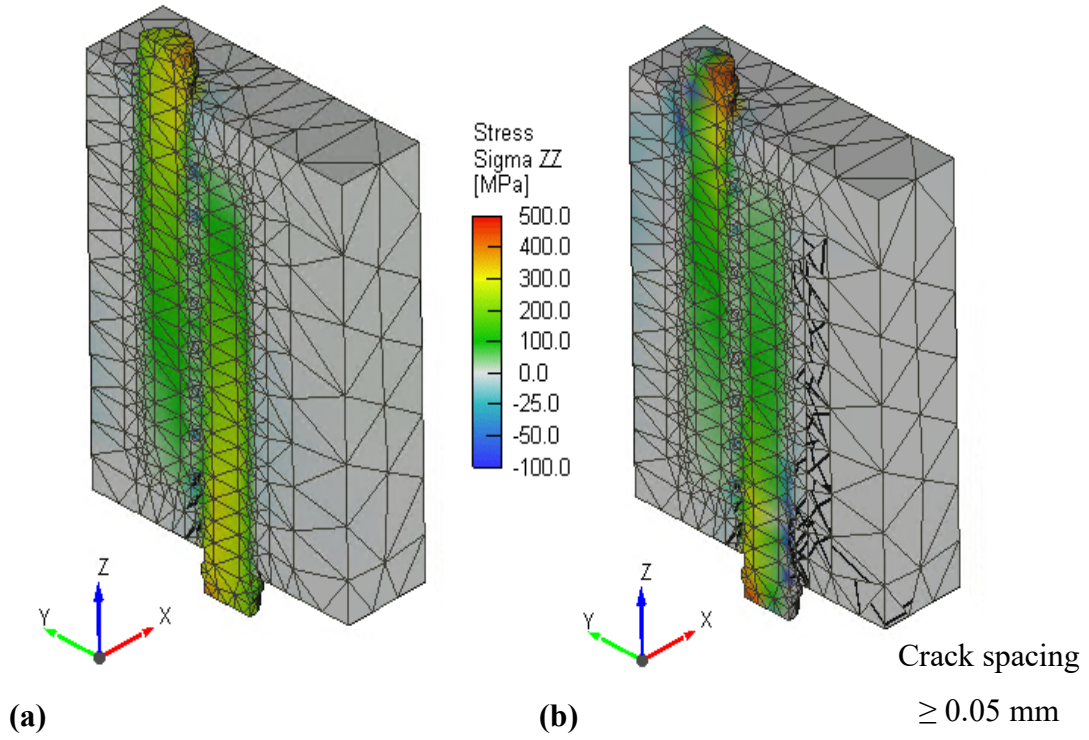
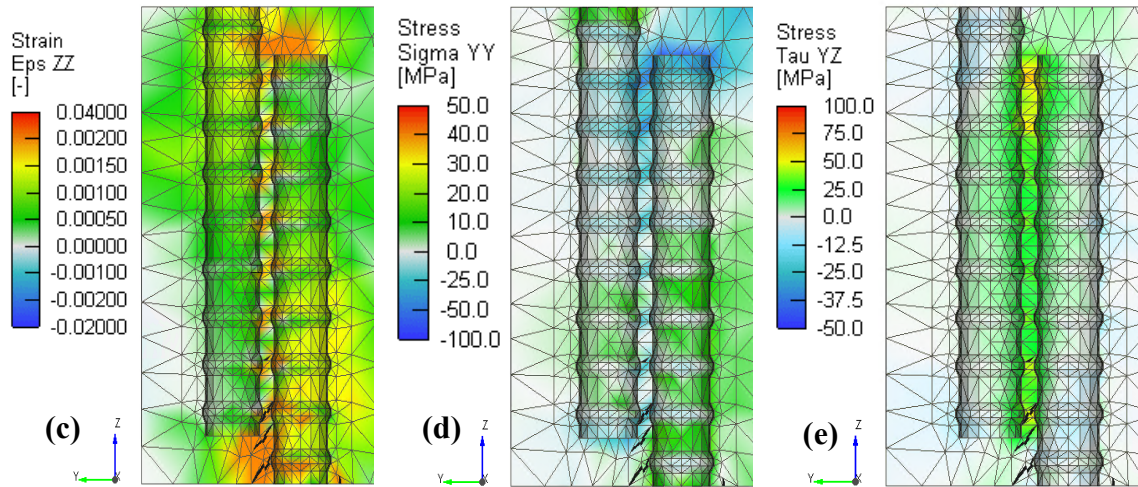


Figure 3.18. Finite element model for CLP-C1-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

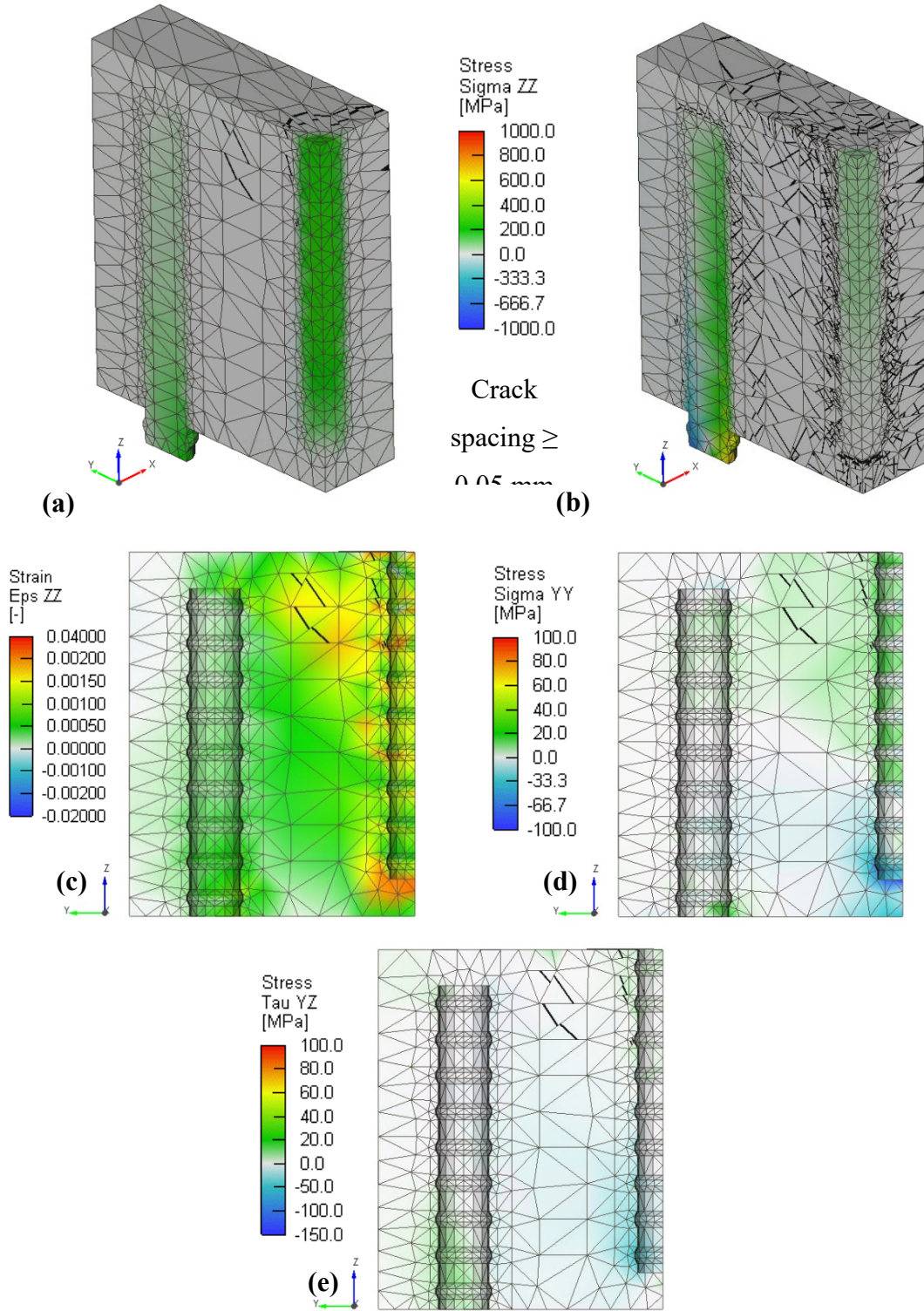


Figure 3.19. Finite element model for NCLP-C1-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

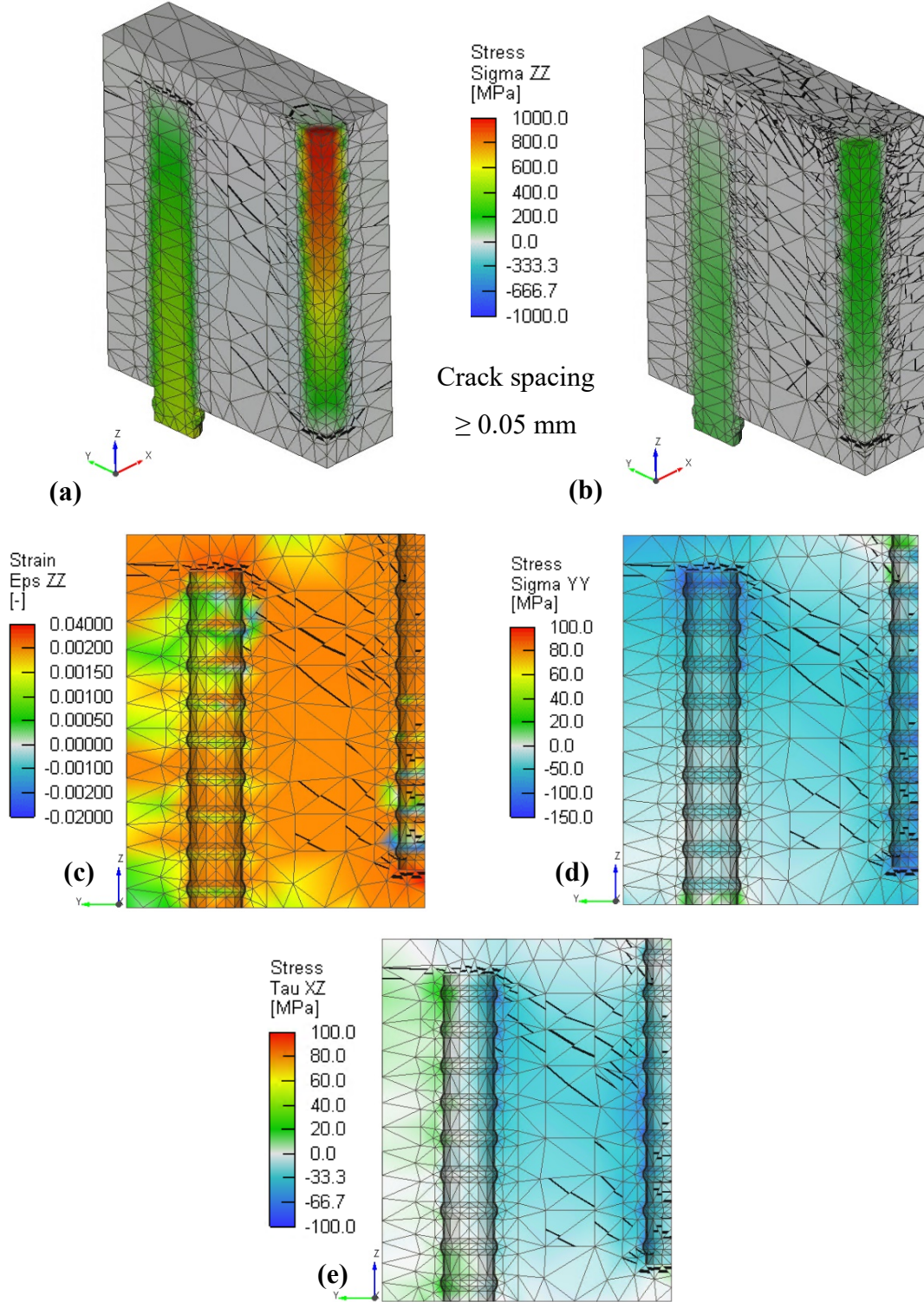


Figure 3.20. Finite element model for R-NCLP-C1-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{xz} at peak.

3.3.3.2 Parametric Sensitivity of the Model and Comparison with the Experimental Database Trends

In Figs. 3.21-3.24, a sensitivity study based on the above models, including all parameter scenarios, was assembled. This collection of analytical results was discretized based on the normalized bond strength, as obtained by shear stress τ_{xz} (or τ_{yz}) at peak load, the normalized anchorage length L_b/D_b and normalized cover thickness C/D_b .

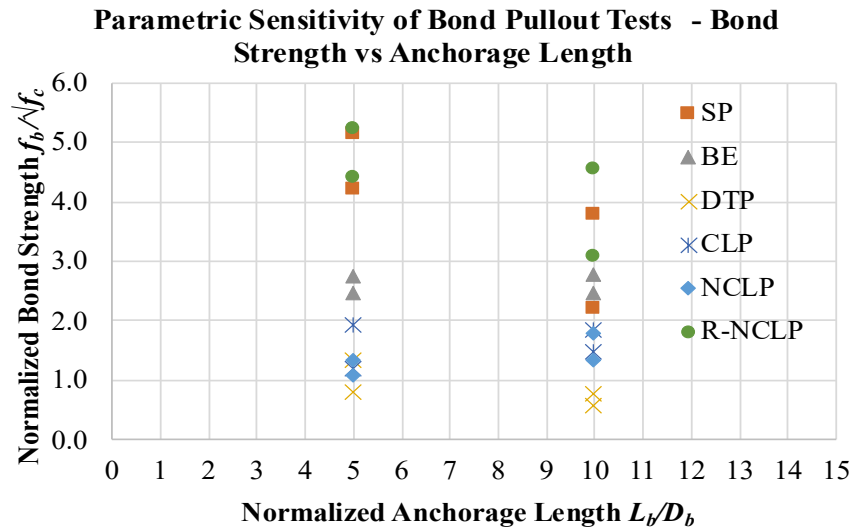


Figure 3.21. Analysis Results that relate the estimated normalized bond strength $f_b/\sqrt{f_c}$ with the normalized anchorage length L_b/D_b .

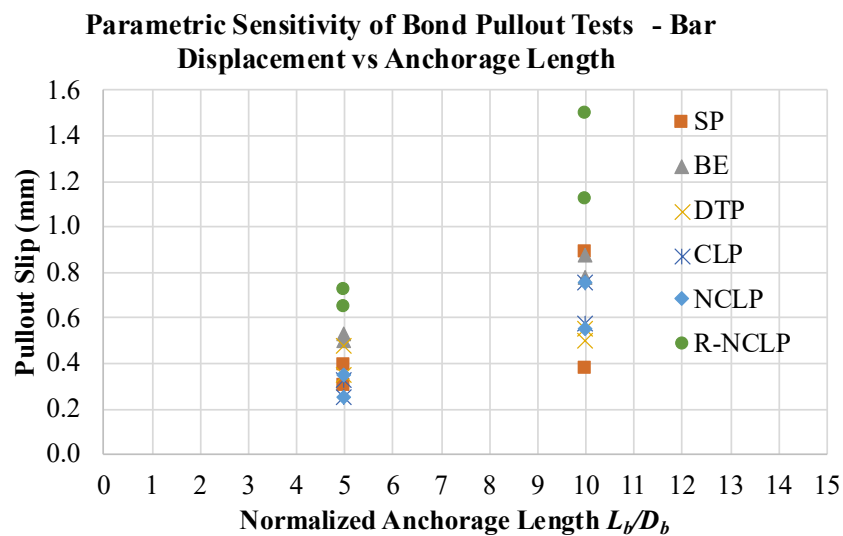


Figure 3.22. Results for pullout slip as a function of normalized anchorage length L_b/D_b .

From Fig. 3.21, it was observed that, the SP tests are in the upper to medium range of bond strength, for the various anchorage lengths considered, reaching up to average normalized bond strength values of 4.67 and 3. For anchorage lengths of $L_b = 5D_b$ and $L_b = 10D_b$, the highest average normalized bond strength was obtained from the R-NCLP test, at 4.83 and 3.82, respectively. The BE test showed consistent values in the average normalized bond strength at 2.62 for all cases of anchorage length. The lowest values were obtained from the CLP, NCLP and DTP tests, with the average normalized bond strength values at 1.56, 1.21 and 1.06, respectively, for the case of $L_b = 5D_b$. For anchorage lengths of $L_b = 10D_b$, the pertinent values for the CLP, NCLP, and DTP tests were 1.67, 1.55, and 0.68, respectively. The DTP test showed the lowest magnitudes of bonds while tests SP, R-NCLP and DTP showed decrease of bond with corresponding increase of the anchorage length.

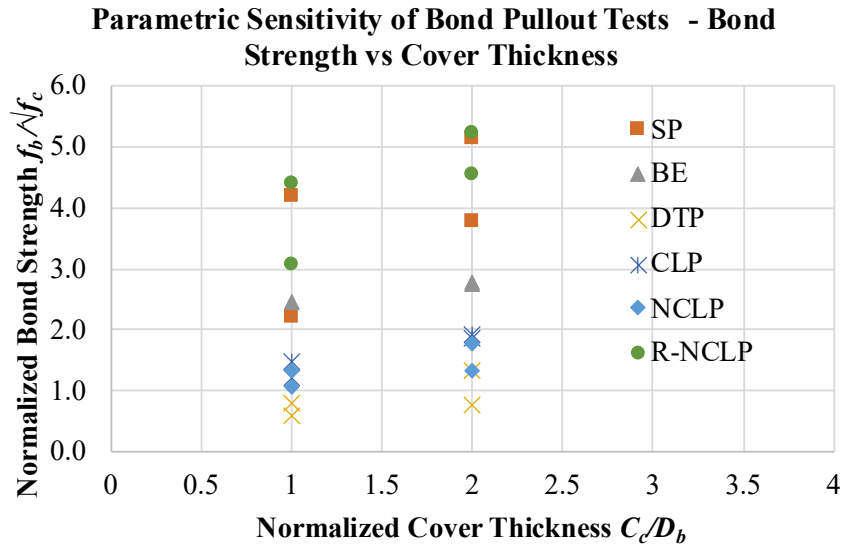


Figure 3.23. Parametric sensitivity analysis on normalized bond strength $f_b/\sqrt{f_c}$ and normalized cover thickness C_c/D_b .

From Fig. 3.22, the pullout slip at peak load vs the normalized anchorage length can be observed. For anchorage lengths of $L_b = 5D_b$, the highest pullout slip at peak was obtained from the R-NCLP test, at an average value of 0.69 mm. The next highest value for the pullout slip was obtained from the BE and DTP tests, at 0.51 mm and 0.41mm, respectively. The average pullout slip at peak for the SP test was 0.35 mm, while for the CLP and NCLP tests, this value was almost identical at 0.3mm. For anchorage lengths of $L_b = 10D_b$, the two highest values of the pullout slip were obtained for the R-NCLP and BE tests, at 1.31 mm and 0.83 mm. Following, the SP, CLP, and

NCLP tests showed almost identical pullout slip values at peak, at approximately 0.65 mm. The lowest value of average pullout slip at peak was obtained from the DTP test, at 0.53 mm.

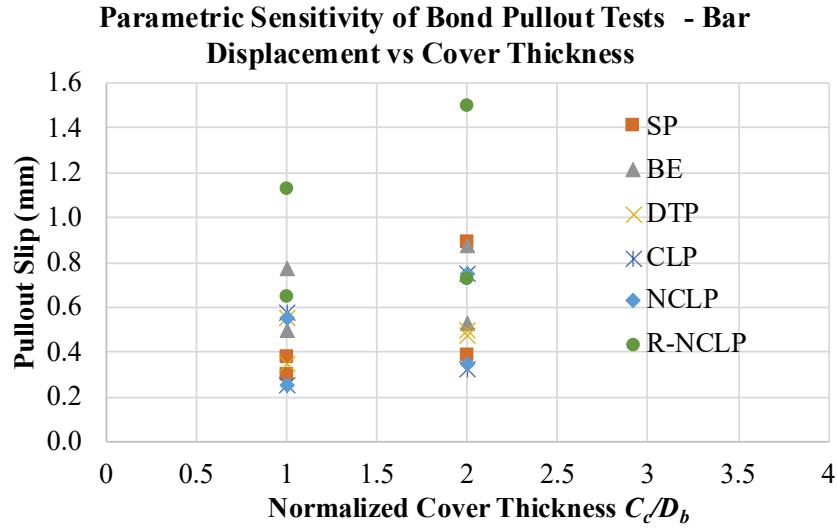


Figure 3.24. Model database based on pullout slip and normalized cover thickness C_c/D_b .

From Fig. 3.23, it was observed that the average normalized bond strength obtained for the SP and R-NCLP tests, considering the cover thickness of $C_c = 1D_b$, was 3.21 and 3.76, respectively. For the same cover thickness, the average normalized bond strength for the BE test was 2.46. The lowest bound of average normalized bond strength values was obtained for the CLP, NCLP and DTP tests, at 1.35, 1.20 and 0.69, respectively. For the cover thickness of $C_c = 2D_b$, the highest values of average normalized bond strength were obtained from the SP and R-NCLP tests, at 4.47 and 4.89, respectively. Following, the next highest values were obtained from the BE, CLP and NCLP tests, at 2.76, 1.88 and 1.56, respectively. The lowest bound of the average normalized bond strength was obtained from the DTP test, at 1.05.

In Fig. 3.24, the average pullout slip at peak vs the normalized cover thickness can be observed. For the cover thickness of $C_c = 1D_b$, the upper bound of the average pullout slip values was obtained from the R-NCLP test, at 0.89 mm. Following, the next highest value was obtained from the BE test, at 0.64 mm. The average pullout slip at peak for the CLP, NCLP and DTP tests were 0.41 mm, 0.4 mm and 0.45 mm, respectively. The lowest value for the pullout slip at peak was obtained for the SP test, at an average of 0.34 mm. For the cover thickness of $C_c = 2D_b$, the highest pullout slip at peak was obtained by the R-NCLP test, at 1.11 mm. Tests BE and SP showed similar

values for the pullout slip at peak for this cover thickness, at 0.7 mm and 0.64 mm, respectively. Lastly, tests CLP, NCLP and DTP also showed comparable values for the pullout slip at peak, at 0.54 mm, 0.55 mm and 0.49 mm. The DTP test showed the lowest value between all tests, for this cover thickness.

Based on all the above, the model database results in a similar pattern as the database described in Figs. 3.2 and 3.3. In reference to the embedment length, the SP and R-NCLP tests showed the highest values of normalized bond stress, in accordance with experimental evidence in the literature. Of the tests included, a similar pattern such as the shown in the experimental database was observed. For the range of anchorage lengths between $5D_b$ and $10D_b$, the normalized bond strength of the BE test exceeded the values obtained from the CLP, NCLP and DTP tests. The last three tests showed similar values in the normalized bond strength, which was also observed in the experimental database.

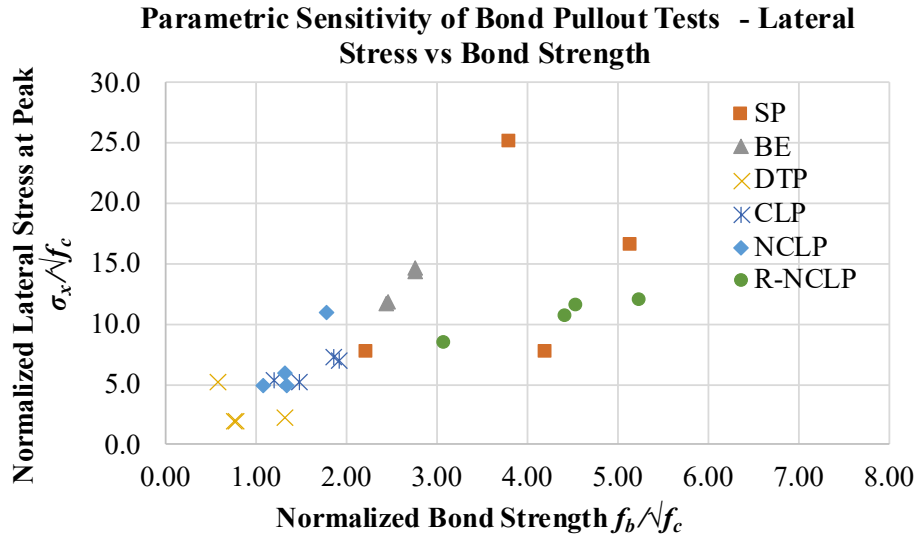


Figure 3.25. Model database based on normalized lateral stress at peak $\sigma_x/\sqrt{f_c}$ and normalized bond strength $f_b/\sqrt{f_c}$.

For the discretization based on the cover thickness, it was shown that the SP, BE and R-NCLP tests result in higher normalized bond strength, compared to other test setups. The lowest values were obtained by the CLP, NCLP and DTP tests, which showed comparable results for the various cover thicknesses. The CLP test exceed the NCLP and DTP test in the average normalized bond

strength for both values of the cover thickness. Overall, the upward trend for the increase in cover thickness that was observed in the experimental database is also validated in the model analysis.

In Fig. 3.25, the relationship between the normalized lateral stress at peak load and the normalized bond strength can be observed. The trend of the graph also shows a commensurate increase in both the lateral stress and bond strength, for the various setups. It was clear that the test setups that induce passive lateral stresses, such as the SP, BE and R-NCLP test, results in higher values of normalized bond strength. The BE test results in increased confining stresses generated from the flexural response of the test bar, which further contribute to bond strength, as it was the observation based on experimental evidence. The lowest bounds, located on the bottom left of the graph, correspond to the CLP, NCLP, and DTP tests, where the DTP test results in values between the CLP and NCLP tests. These patterns of the normalized bond strength show the reduced effect of lateral stresses on these tests. In the case of the DTP test, the absence of passive lateral stresses and flexural response of the bar, leads to the most conservative estimate of the obtained normalized bond strength values.

3.4 Summary

In the above chapter, the bond behavior between reinforcement and UHPFRC was investigated by conducting a two-part study: i) a database of experimental studies on bond between reinforcement and HPFRC and ECC materials was assembled, containing a variety of bond tests conducted several different test setups, and ii) an extensive parametric study using detailed 3-dimensional FEA was conducted to validate and further demonstrate the stress state of the reinforcement-to-HPFRC interface of the various experimental bond test setups. The outcomes of this study are the following:

1. The database investigation illustrated that there is excessive sensitivity of the measured bond strength on the type of setup chosen – using as a benchmark anchorage length a value of $5D_b$ it was found that the maximum value for bond strength from all specimens was up to $1.8\sqrt{f_c}$ whereas the corresponding magnitude in the case of DTP was only up to $1.3\sqrt{f_c}$. The source of this effect was owing to the different boundary conditions of the two test setups a sensitivity and bias that was found to persist in the case of HPFRC despite the significant internal confinement as compared to conventional concrete.

2. A similar pattern to the one described above was observed for the database consisting of ECC specimens where, considering the anchorage length of $5D_b$, the DTP test showed an average value of bond strength of $0.7\sqrt{f_c}$, compared to the values of $1.2\sqrt{f_c}$ and $1.9\sqrt{f_c}$, for the BE and SP tests, respectively. As such, bond tests with the ECC materials also showed sensitivity to the passive confinement due to support conditions.
3. To derive conservative values for bond, experiments conducted including both DTP and beam specimens (with the bar anchored in the constant moment region) were reviewed and their results compared. In these tests three types of materials from the same batches had been used. It was found that the two tests produced comparable values for bond, in the range of $1.3\sqrt{f_c}$ and $1.45\sqrt{f_c}$, for the range of the two cover thicknesses of $1D_b$ and $2D_b$ and considering all materials. Both of these tests corresponded to the lower bound for short anchorage lengths ($5D_b$) based on the database evidence.
4. The experimental outcomes of the DTP test were implemented in the FEA modeling of UHPFRC-retrofitted columns, in combination with the constitutive relationships based on the Model Code 2010, for the description of bond behavior within the UHPFRC jacket. This is described in detail in Chapter 8.
5. A noticeable difference between the two test setups was a significantly higher tensile resilience in the case of the beam specimens as compared to the DTP specimens, particularly in the post-peak response envelope. This difference was attributed to the curvature of the beam at large levels of deflection; the bond stresses attributed to curvature are analogous to the frictional losses owing to curvature in prestressed concrete (loss of tendon force occurs due to transfer of forces to concrete – which, in non-prestressed concrete, is the bond stress). In fact, this secondary source of resistance was enough to suppress the appearance of the effect of the cover thickness in the beam specimens, whereas the thicker cover led to higher bond strength in the DTP specimens.
6. Extensive parametric FEA analysis of the various bond test setups using a detailed three-dimensional model for the bar demonstrated the effect of spurious stresses generated by boundary conditions on the specimen model. Laterally restrained models, such as the SP and R-NCLP resulted in greater values of bond strength, compared to unrestrained tests such as the CLP, NCLP and DTP tests. The contribution of flexural effects was also shown in the BE test model, which corresponded to the medium range of bond strength values.

The CLP test showed the least sensitivity both in the anchorage and cover thickness increases, whereas the NCLP test showed sensitivity to the cover thickness increase. The DTP test provided, on average, the lowest bound of bond strength values.

7. Results from a parametric analysis using the model followed the trends of the experimental database assembled earlier. Discretization based on the anchorage length showed a similar trend for specimens modeled with a $5D_b$ anchorage length, with conventional pullout tests (SP, BE) exceeding the direct tension pullout tests (CLP, NCLP, DTP), while the sensitivity of the results to cover thickness resulted in increased average bond strength for most model cases, with greater increases observed in the cases of laterally restrained models.
8. Comparison of the lateral stress σ_x (or σ_y) at peak to the bond strength f_b of the various models confirmed there a relationship between the magnitude of passive confinement, i.e., the spurious lateral stresses generated by the presence of boundary conditions, and bond strength as seen in the Database Results.
9. Based on the above experimental and computational evidence, it was shown that the internal confinement provided by the cementitious matrix of HPFRC and ECC materials contributes to the bond performance of embedded reinforcement. As such, to further understand and quantify the magnitude of the confinement, a combined experimental and analytical study was conducted, with UHPFRC jackets placed around conventional concrete cores, which is described in the next section.

4 Compressive Testing of UHPFRC-Confined NSC cores

In this chapter, the results of axial compressive tests of normal strength concrete (NSC) cores confined with UHPFRC jackets are shown. The axial load is applied directly on the inner core surface and the jacket is activated only in the lateral tension. Both monotonic and cyclic axial load patterns are applied, and the results obtained through external instrumentation and digital image correlation (DIC) analysis are used to assess the intensity and effectiveness of passive confinement imparted by the UHPFRC jacket.

4.1 Preparation of Specimens

Two series of the material characterization experiments were conducted, containing cores of unreinforced NSC confined with thin UHPFRC jackets and two types of sections. The inner core was confined triaxially, by the combined action of the loading platens in the vertical direction, and by the action of the jacket in the lateral direction. Cyclic displacement histories were applied to assess the hysteretic response and strength degradation. The main purpose of this experiment was to define the constitutive stress-strain relationship of NSC in compressive cyclic loading, with lateral confinement provided by the UHPFRC jackets. The jackets were applied throughout the whole length of the specimens, with the top and bottom surfaces of the cores remaining uncovered to fit the loading platens (Fig. 4.1).

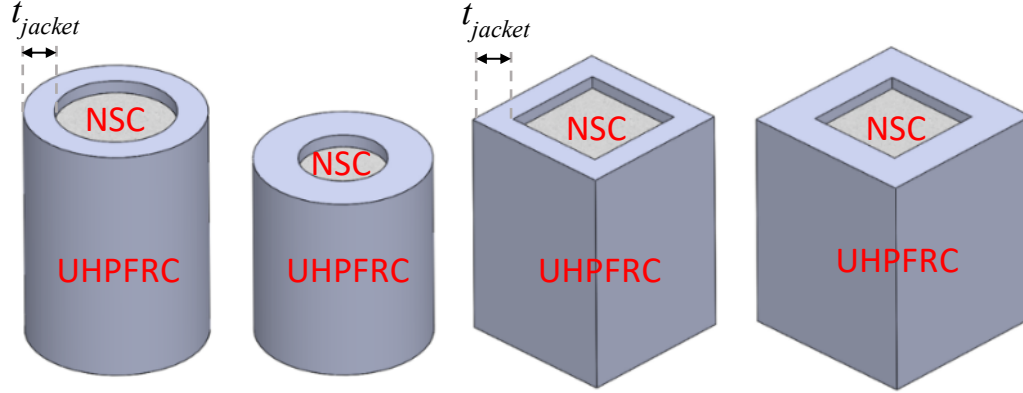


Figure 4.1. Specimen schematic: a) UHPFRC-Confined Cylinders; b) UHPFRC-Confined Prisms.

A total of 30 specimens were constructed, of which 18 had the confined-cylinder (CC) configuration and 12 were confined prisms (CP) as shown in Fig. 4.1. The jackets were cast around concrete cores for each specimen type, using the setups shown in Fig. 4.2. For the case of the cylindrical specimens, the inner cores were cast using conventional PVC forms. After removal of the formwork, the cores were ground and cured in water for 14 days. Subsequently, the confining UHPFRC jacket was applied using a second PVC formwork of appropriate size, to create jacket thicknesses of 25 mm and 37.5 mm, as shown in Fig 4.2a. To ensure central placement within the jacket, the inner cores were attached on a layer of foam which was centered with respect the axis of the cylindrical formwork (Fig. 4.2a). An additional layer of foam was placed on the top surface of the inner core, such that the surface would remain exposed after casting of the UHPFRC jacket. The formwork was adequately oiled to allow for smooth removal after hardening. During casting of the jacket, the UHPFRC material was poured only from one side of the cylinder, aiming to obtain an optimal orientation of the steel fibers ($l_f = 13\text{mm}$, $\varnothing = 0.2\text{mm}$) in the horizontal direction, which would ensure radial confinement during the lateral expansion of the jacket. The fiber volume was 2% for this material. The available space was filled without requirement for vibration, thanks to the flowability and self-consolidating properties of the material. After two days, the specimens were removed from the formwork, the foam surfaces disconnected from the inner cores, and the specimens were soaked under wet burlap for 14 days.

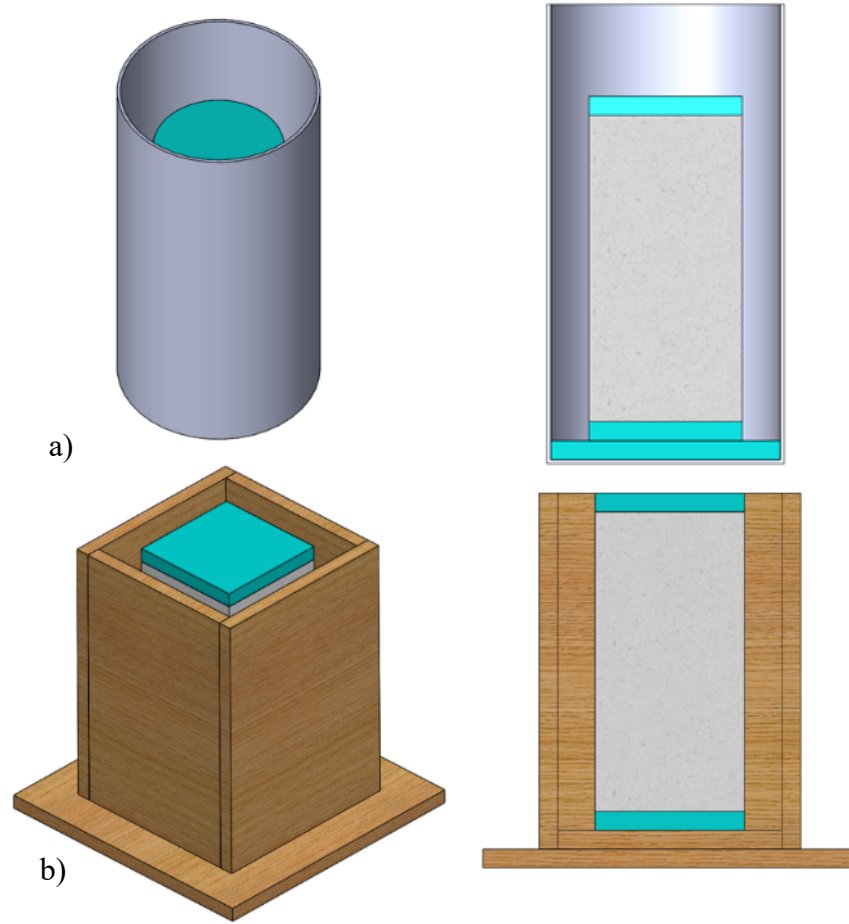


Figure 4.2. Specimen formworks: a) Confined Cylinders series; b) Confined Prisms series.

Due to restrictions to accessing facility equipment during the COVID-19 pandemic, access to large capacity mixers was not available at the time of preparation. As a result, casting procedures were completed in a small capacity mixer (6L), producing two specimens for each batch (total of 9 castings) and one 75 x 150 (mm) cylinder; the latter was used for testing of the UHPFRC compressive properties. To ensure consistency in the subsequent batches, the pertinent equipment was cleaned and dried manually, and excess water was removed prior to each casting. The control parameters in both the cylindrical and prismatic specimen series include the cylinder diameter (D_c) and cover thickness (C_t). The material properties of the NSC cores were obtained from the testing of 100 x 200 (mm) cylinders, in an MTS testing frame, which is discussed in the following sections. For the UHPFRC material, the mix designs are provided in Appendix B. The compressive strength was determined based on three cylinders of the 75 x 150 (mm) cylinders, which were tested in another facility, and resulting in an average compressive strength of 140MPa. Finally, the cracking and ultimate tensile strengths were determined based on inverse analysis of four-point bending

tests conducted on 75 x 75 x 280 (mm) prisms, resulting in $f_{cr} = 9.32$ MPa and $f_t = 11.29$ MPa, respectively. The cracking strain ε_{cr} and the strain at ultimate tensile strength ε_t were 0.00026 and 0.00457, respectively.



Figure 4.3. Confined cylinder specimens; Use of wet burlap for hydration; Custom steel cylinders for testing.

For the rectangular prisms, the formworks for the inner cores were fabricated using commercial plywood, using the arrangement shown in Fig. 4.4. The cores consisted of beam-like prisms with dimensions of 100mm x 100mm x 200mm, and two jacket thicknesses were considered, as in the CC specimen series, at 25mm and 37.5mm. After removal of the formworks, the rectangular cores were soaked in water for 14 days.

To prepare the rectangular prism cores for casting of the jacket, the formworks shown in Fig. 4.2b were used, with the specimen cores centered to the formwork space on a layer of plywood. Both outer surfaces of the prisms, at the top and bottom, were covered with foam to ensure that the surfaces were not enclosed by the UHPFRC material. Casting of the jackets is shown in Fig. 4.4 and was followed by removal of the formworks after 2 days and submersion of the specimens in water for 28 days.



Figure 4.4. Specimen formworks for Confined Prisms: Casting of inner cores and UHPFRC jackets.

4.2 Testing Procedure

The specimens were prepared prior to testing by clearing the debris on the exposed surfaces of the inner cores and painting of the jacket surface. To ensure leveling of the inner core surface, a layer of commercial grade plaster was placed on the exposed surfaces, as shown in Fig. 4.5. The external UHPFRC jacket surface was subsequently painted white with a water based Kilz primer, as shown in Fig. 4.6. A pattern of black dots was additionally sprayed throughout the painted jacket surface (Fig. 4.6), to observe the strain field and surface displacements of the jackets using DIC analysis.

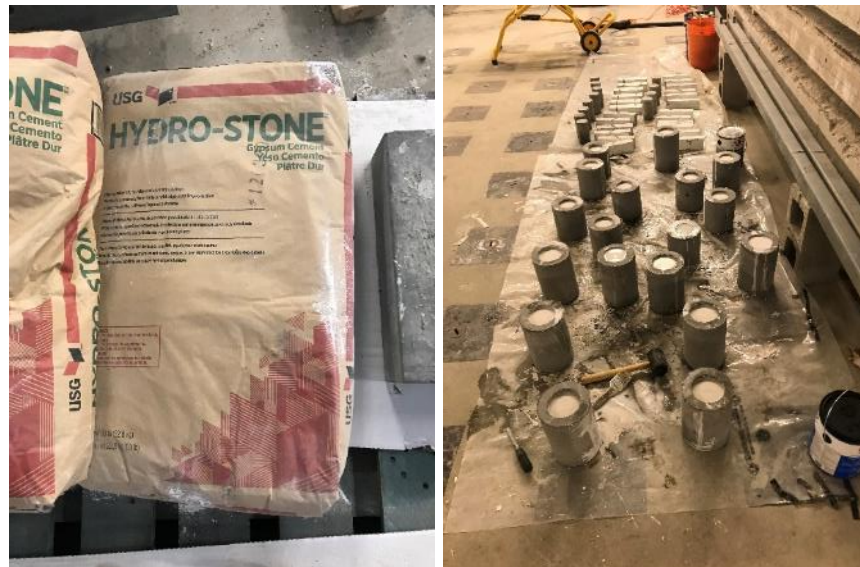


Figure 4.5. Hydro-stone plaster layer cast on inner core surfaces.



Figure 4.6. Left: Black-speckled surface for DIC analysis. Top-Right: Rust-Oleum black spray used for speckles. Bottom-Right: White Kilz primer used for jacket surface.



Figure 4.7. Left: Custom-fitted steel prisms for CC and CP specimens. Right: CC and CP specimens with plaster layer and speckled surface for DIC.

Testing was conducted on an MTS Universal Testing frame. Direct compressive loading was applied to the inner cores with the custom-fitted, loading platens, bearing on the layer of plaster. The platens were machined in such a way that they would be “snug” fit on the inner core surface (Fig. 4.7). For the cylindrical specimens, circumferential dilatation was measured using a chain-like extensometer that was placed around the specimen and connected directly to the DAQ system (Fig. 4.8). For the CP specimens, dilatation is measured using the DIC analysis measurements.



Figure 4.8. Epsilon chain-link circumferential extensometer for CC specimens.

In addition, two pairs of strain gauges were placed on opposite sides of each specimen, to measure axial and lateral strains (Fig. 4.8). The mechanical apparatus was attached to the MTS testing frame using a magnetic connector (Fig. 4.9). The final setup is shown in Fig. 4.10.

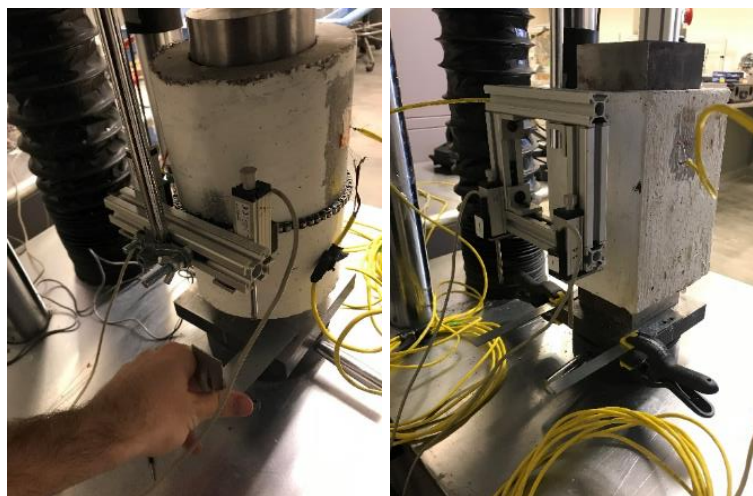


Figure 4.9. Left: CC specimens on testing frame setup; Right: CP specimens on testing frame setup.

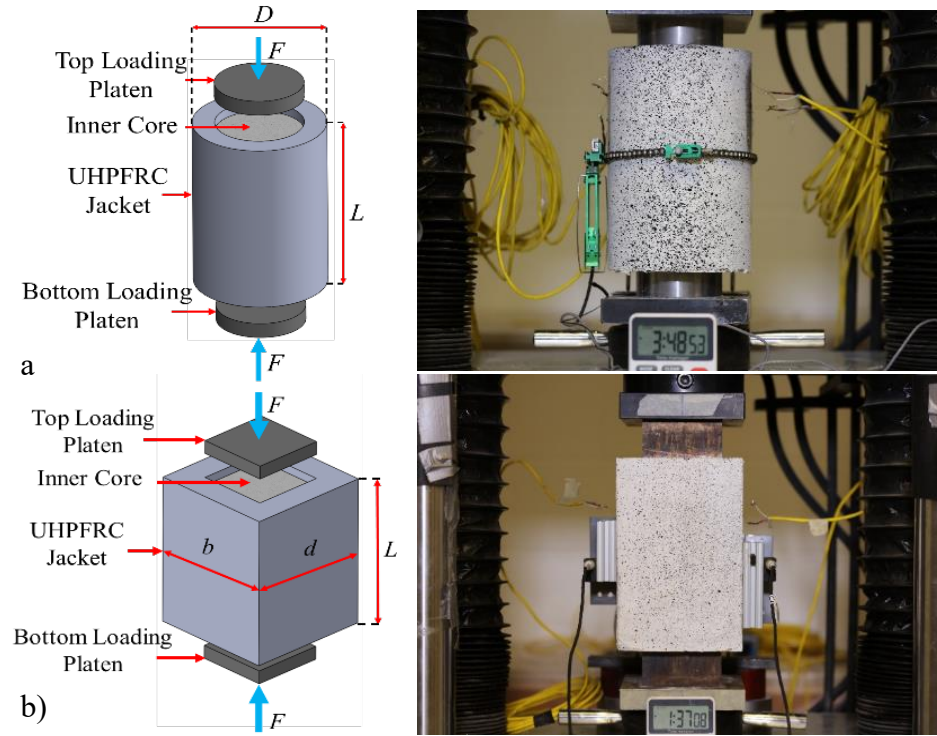


Figure 4.10. Confined specimen geometry and test arrangement for: a) CC specimens and b) CP specimens.

4.3 Results

From the testing procedure described above, results are obtained in the form of force vs. displacement curves using the measurements from the attached instrumentation. Expansion curves measured in the CC specimens are also discussed, as well as measurements from the horizontal DTs attached on the CP specimens. Material parameters and mechanical properties are presented for each specimen series separately.

4.3.1 Confined Cylinder (CC) Series

The results of all the tests in the CC specimen series are shown in Table 4.1. In all CC specimens, failure was characterized by a distinct splitting crack on the UHPFRC jacket (Fig. 4.11). Multiple finer cracks also appeared throughout the jacket top surface, without developing into complete splitting of the surfaces. At the termination of the test, the inner core was shown to be crushed to the largest extent, and the weaker concrete of the core was reduced to a “sand”-like texture. Upon closer inspection of the cracked surfaces, fiber distribution was shown to be inconsistent, leading to a weakened section of the jacket. For this experiment, the pouring location during casting was

random, to ensure objectivity of the observations. The tests were terminated at approximately ~25% of the peak load, to avoid sudden failures of the specimens. It is worth noting at this point that at the residual branch of the specimen response, when complete separation of the splitting crack surfaces had occurred, substantial ductility was shown by the confined core; no abrupt reduction in strength and ductility was observed.



Figure 4.11. Distinctive vertical crack due to lateral expansion of the jacket; Left: CC25 specimens. Right: CC375 specimens.

Table 4.1. Summary of UHPC-confined cylinder specimens.

Specimen Types	Loading Type	F _{peak} (kN)	Displacement (mm)	Stress (MPa)	Long. Strain (%)	Expansion (%)
NSC-M1	Monotonic	150.8	1.78	19.2	0.89	1.057
NSC-M2	Monotonic	233.0	0.45	29.7	0.23	0.045
NSC-M3	Monotonic	169.7	1.30	21.6	0.65	0.675
CC25-M1	Monotonic	264.3	-	33.7	-	-
CC25-M2	Monotonic	223.4	1.67	28.5	0.83	0.127
CC25-M3	Monotonic	230.6	1.49	29.4	0.75	0.23
NSC-C1	Cyclic	196.3	0.54	25.0	0.27	0.339
NSC-C2	Cyclic	225.7	0.48	28.7	0.24	0.046
NSC-C3	Cyclic	186.5	0.19	23.8	0.10	0.056
CC25-C1	Cyclic	272.3	1.60	34.67	0.80	0.427
CC25-C2	Cyclic	244.6	1.35	31.14	0.68	0.207
CC25-C3	Cyclic	266.7	1.48	33.96	0.74	0.137
CC25-C4	Cyclic	208.9	2.29	26.60	1.15	0.314
CC25-C5	Cyclic	221.1	2.07	28.14	1.04	0.130
CC375-M1	Monotonic	169.1	1.66	38.28	1.11	0.206
CC375-M2	Monotonic	205.3	1.74	46.48	1.16	0.268
CC375-M3	Monotonic	255.0	1.77	57.73	1.18	0.101
CC375-C1	Cyclic	179.5	1.68	40.63	1.12	0.173
CC375-C2	Cyclic	216.2	1.63	48.94	1.09	0.262

CC375-C3	Cyclic	187.5	2.03	42.45	1.35	0.372
CC375-C4	Cyclic	195.2	1.69	44.19	1.13	0.251
CC375-C5	Cyclic	176.9	2.10	40.04	1.40	0.529

4.3.1.1 Monotonic Loading

Prior to testing of the confined cores, an initial set of experiments was conducted on unconfined NSC cylinders, using the same setup as in confined cylinder tests. In these tests, loading was applied directly to the specimens using the testing frame load plates. The force – displacement response was obtained, as well as the expansion curves, using the chain-link circumferential extensometer as shown in Fig. 4.8. The cumulative results of the monotonic tests on NSC cylinders are shown in Fig 4.12.

For the NSC cylinders, the average strength was 184.5 kN (23.5 MPa). The average strain at peak stress was 0.0059, normalized to the height of the cylinder, which is 200mm. The expansion of the cylinder at peak load was averaged at 1.86 mm (average lateral strain = $1.86/3.14/100 = 0.0059$, i.e., an equivalent Poisson’s ratio of 1 at failure). Excluding specimen NSC-M2, which showed higher strength than anticipated, the remaining cylinders performed at the expected levels for a conventional concrete with a nominal strength of 25 MPa.

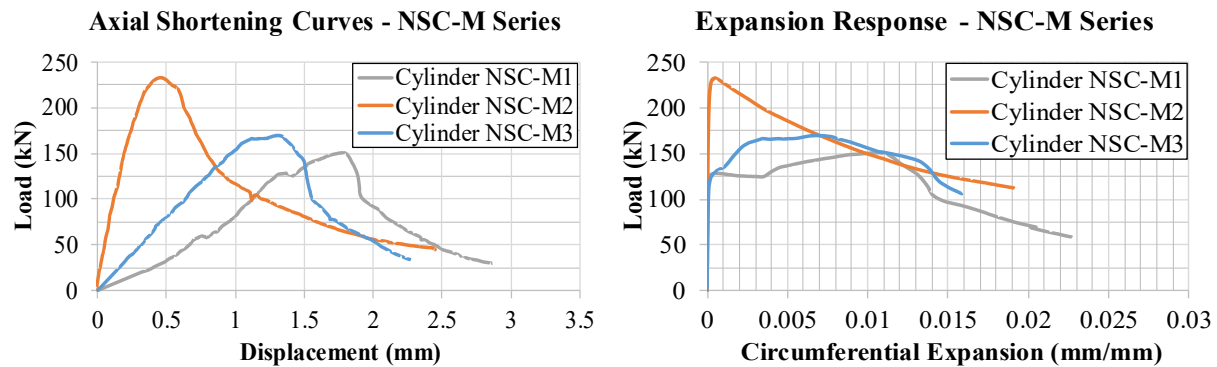


Figure 4.12. Axial shortening (left) and expansion curves (right) for the NSC-M specimen series.

For the first tests on confined cylinders the cumulative results of the CC25-M series are shown in Fig. 4.13, including both the response and expansion curves. Results from specimen CC25-M1 are not shown in the figure due to a software error at testing, however, the peak load is included in Table 4.1. The small “drop” in the response curves, in the post-peak branch, signifies the position

where the circumferential extensometer is removed, at approximately 45% of the peak load, to avoid damaging the equipment at the end of the experiment.

For the CC25-M cylinders, the average strength was 239.4 kN (30.5 MPa), including specimen CC25-M1. The average strain at peak stress was 0.008 and the expansion at the peak load was averaged at 0.0018 (0.56 mm); average lateral strains are shown in reference to the inner core. Compared to the unconfined cylinders, these specimens show an average peak stress 30% higher, with the axial strain at peak to have similar increase (~35%). The expansion at the peak load of these specimens is substantially lower, which is ~30% of the expansion of the unconfined cylinders. It is apparent that the presence of the jacket improves the performance of the NSC cylinders, both in strength and ductility, thanks to the confining action and significantly high tensile capacity of the jacket. This result is in accordance to experimental evidence regarding the strength of confined concrete with UHPFRC materials.

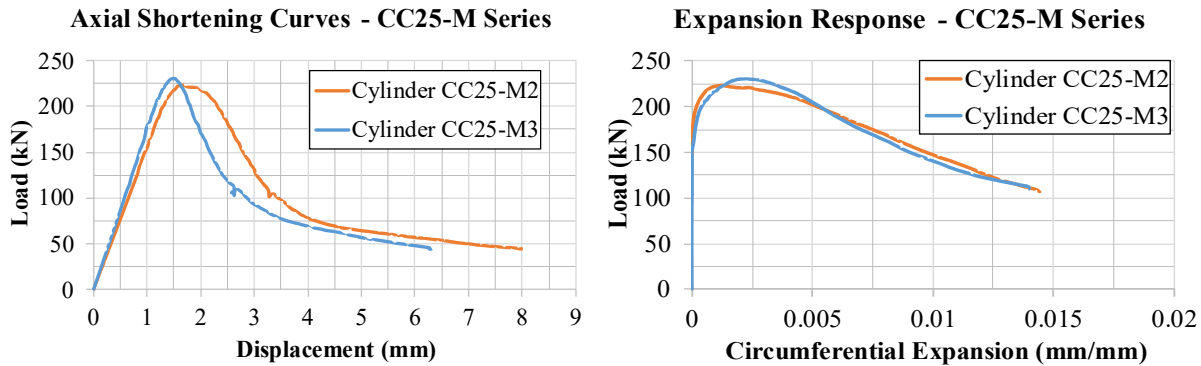


Figure 4.13. Axial shortening (left) and expansion curves (right) for the CC25-M specimen series.

For the tests on the confined cylinders of the CC375-M series, the cumulative results of the monotonic tests are shown in Fig. 4.14, including both the response and expansion curves. The average strength for this series was 209.8 kN (47.5 MPa) and was obtained at an average axial strain of 0.0115. Expansion at peak was 0.00195 (0.45 mm). The above values correspond to strength and strain (at peak) increases by 102% and 95%, respectively. The circumferential expansion at the peak load was reduced by 25% from the average expansion of the unconfined cylinders. It is evident at this stage that the increase in jacket thickness further improved the mechanical performance of the confined concrete, approximately doubling the strength of the unconfined specimens. It should be noted here that the cylinder size of the inner core, for the CC375 series, is 75 mm x 150 mm, less than the 100 mm x 200 mm size of the unconfined

cylinders; however, for it can be reasonably assumed that this small difference in size produced negligible variations in the obtained response in terms of a size effect (aggregates < 13mm were used).

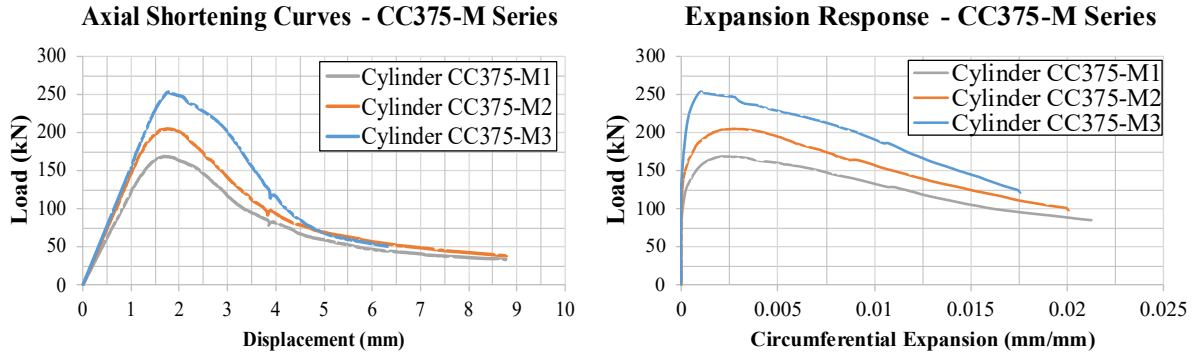


Figure 4.14. Axial shortening and expansion curves for the CC375-M specimen series.

4.3.1.2 Cyclic Loading

For the cyclic loading, the loading regime was programmed to produce two loading cycles at increments of 0.5 mm. In each cycle, the test algorithm applied displacement-controlled load until the target actuator displacement was reached (i.e., 2.0 mm, 2.5 mm, 3.0 mm, etc.). Upon unloading, the algorithm switched to load-control and the load direction was reversed until the applied actuator load would reach only 5.0 kN; this value of the load was chosen to avoid disconnection of the loading platens from the specimen surface. In the case of the unconfined cylinders, the displacement increment was selected to commence at the target displacement of 0.5 mm, to account for the brittleness of conventional NSC. Based on observations from the monotonic performance of the confined cylinders, displacement increments were selected to commence at 2.0 mm, to obtain cycles beyond the elastic range.

The cumulative results of the cyclic tests on NSC cylinders are shown in Fig 4.15. Average strength of this specimen series was 202.8 kN (25.8 MPa). The average strain at peak stress was 0.0020, normalized to the height of the cylinder (100 mm x 200 mm specimens). The average expansion of the cylinder at peak load was 0.00147 (0.462 mm). In this series of specimens, the NSC concrete strength was closer to the target value of 25 MPa. It was observed that the ductility of the unconfined cylinders under cyclic loading was reduced substantially, with the strain at the residual stage reaching 1.05%, contrary to the maximum strain of 1.4% obtained for monotonic tests. Expansion measurements were higher for cyclic tests reaching a maximum value of 0.041 (13

mm), while the same parameter for the monotonic tests was 0.023 (7.2 mm). It became evident that the application of cyclic loading impacts the deformation capacity of the NSC cylinders; the increased expansion also supports this observation, as the load reversals further incurred damage in the concrete specimen and, owing to Poisson's effect, led to larger expansions of the circumference.

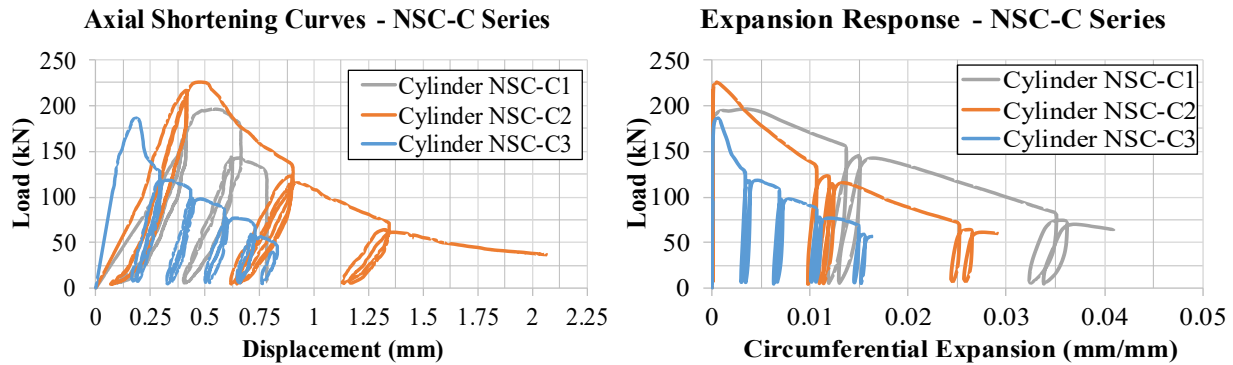


Figure 4.15. Axial shortening and expansion responses for the NSC-C specimen series.

For the cyclic tests on the confined cylinders of the CC25-C specimen series, the cumulative results of the response and expansion curves are shown in Fig. 4.16. Average strength for this series was 242.7 kN (30.9 MPa), obtained at an average axial strain of 0.0088. Expansion at the peak load was 0.00242 (0.76 mm). Compared to the unconfined NSC cylinders, the strength in the CC25-C specimens increased by 20%, at a strain that was 4.4 times higher than the strain at peak for the control specimens. While the increase in strength appears to be less than what was found in the monotonic loading case, the increase in ductility, represented by the significant strain at peak and subsequent branches, shows very favorable results. Axial strain in the residual branch reached a maximum of 3.85 % for the CC25-C specimens and expansion at peak was increased by 65%, compared to the unconfined cylinders. The superior mechanical performance was evident throughout the confined cylinder arrangement, with higher values of axial strain sustained at the ultimate (residual) stage, without abrupt loss of strength. It is evident that a thin UHPFRC jacket of 25 mm is sufficient to impart significant benefits in terms of strength and ductility to conventional NSC axial members.

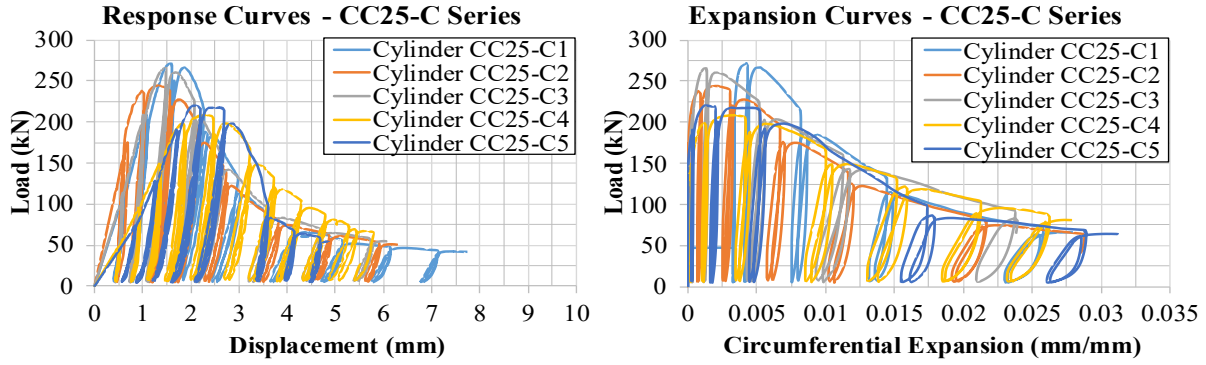


Figure 4.16. Axial shortening and expansion curves for the CC25-C specimen series.

For the cyclic tests on the confined cylinders of the CC375-C specimen series, the cumulative results of the response and expansion curves are shown in Fig. 4.17. The average strength for this series was 191.1 kN (43.3 MPa) and was obtained at an average axial strain of 0.0122. The expansion of the circumference at the peak was 0.0032 (0.75 mm). Compared to the unconfined cylinders, the strength in the CC375-C series shows an increase by 68%, whereas the average axial strain at the peak load is 6.1 times higher than the control specimens. Similar to the thinner jacket of 25 mm, it is observed that further increase of the jacket thickness produces greater improvement in strength and ductility; compared to the CC25-C series, the strength and strain at peak are 40% and 38% higher, respectively. Expansion at the peak is also greater for the CC375-C case, than for the unconfined specimens. The expansion in this series is approximately the same as in the CC25-C case, however the reduced circumference of the 75 mm x 150 mm cylinder must be considered.

Based on the above, the volumetric strains can be estimated as $\varepsilon_{vol} = \varepsilon_{11} + \varepsilon_{22} + \varepsilon_{33}$. It is also assumed that lateral strains ε_{11} and ε_{22} are equal. By normalizing to the corresponding inner cores (cylinders of 100 mm x 200 mm and 75 mm x 150 mm, for the CC25 and CC375 specimen series, respectively), the obtained volumetric strains of the NSC, CC25, and CC375 experiment series, under monotonic loading are 0.0059, -0.0044 and -0.0076, respectively. In similar fashion, the volumetric expansions under cyclic loading are 0.00094, -0.004, and -0.0058; positive sign for expansion, negative sign for shrinkage. It is evident that in the UHPFRC-jacketed specimens, the presence of the jacket results in restriction of lateral expansion, leading to reduction of the total volume of the inner cores. In addition, in the case of cyclic loading, the value of the volumetric strain for the unconfined cylinders, is substantially reduced, compared to the same value for monotonic loading; this is an indication of uncontrolled failure which has been shown to occur

when the volumetric strain approaches zero (Imran and Pantazopoulou 1996). Lastly, for all confined cylinder cases, the volumetric strains show comparable values under the monotonic and cyclic loading regimes, with greater reduction of volume occurring in the former case.

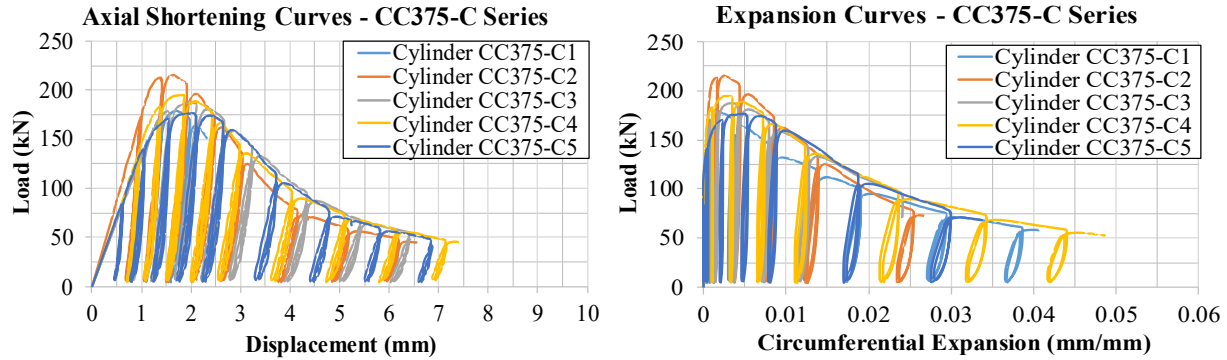


Figure 4.17. Axial shortening and expansion curves for the CC375-C specimen series.

4.3.2 Confined Prism (CP) Series

The results of unconfined cylinder tests, as well as all the tests in the CC specimen series are shown in Tables 4.2 and 4.3, respectively. Specimens of the CP series were tested using the arrangement shown in Fig. 4.10. As in the case of the cylindrical specimens, failure was signified by a distinct splitting crack across one of the sides of the jacket. In all specimens, the splitting crack appeared along one of the faces of the specimens, while vertical cracks developed from the inside corners of the jacket and spreading towards the outside corner (see Fig. 4.18(c)). Due to localization of the splitting cracks on the faces, the corner cracks never developed completely through the jacket thickness to the outer surface. In most of the specimens, further observation of the crack opposite surfaces shows unequal distribution of the fibers; this indicates the potential weakening of the crack section due to this effect. Additional finer cracking was also observed at the top and bottom surfaces of the jacket, without fully developing into splitting failures.

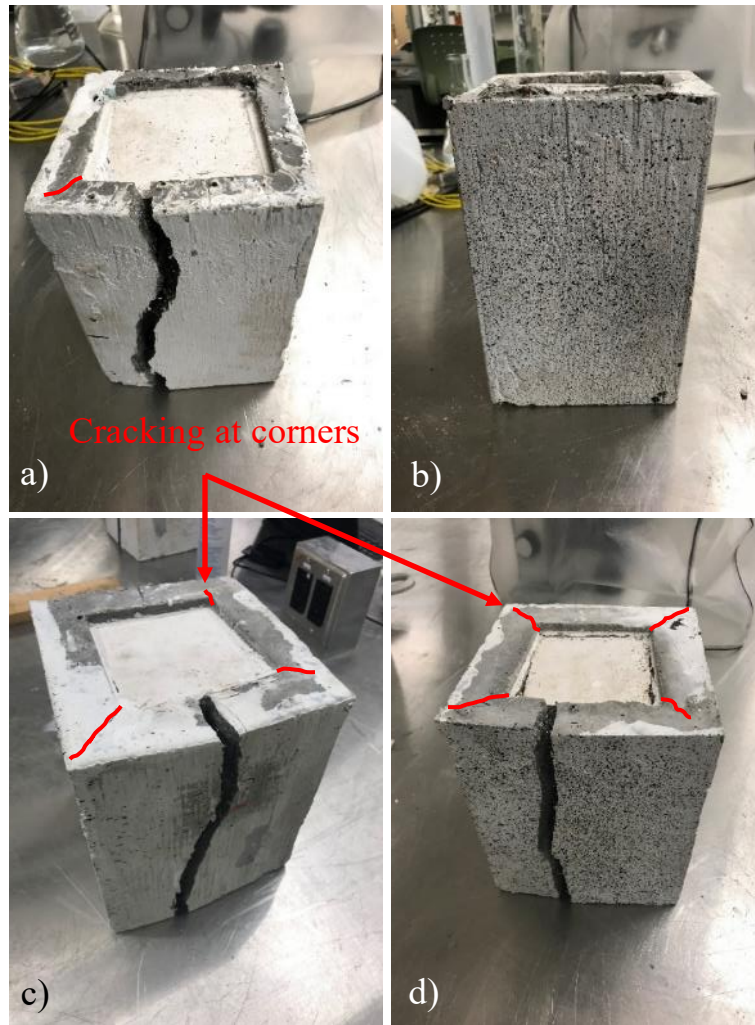


Figure 4.18. Splitting cracks appearing on the outer UHPFRC jacket of CP specimens.

Table 4.2. Summary of results for unconfined NSC cylinder specimens at peak.

Specimen Types	Loading Type	F_{peak} (kN)	Displacement (mm)	Stress (MPa)	Long. Strain (%)	Expansion (%)
NSC-M1	Monotonic	287.8	0.75	36.65	0.38	0.038
NSC-M2	Monotonic	272.1	0.66	34.65	0.33	0.089

Table 4.3. Summary of results for UHPFRC-confined prism specimens at peak.

Specimen Types	Loading Type	F_{peak} (kN)	Displacement (mm)	Stress (MPa)	Long. Strain (%)
CP25-M1	Monotonic	373.2	1.57	37.32	0.79
CP25-M2	Monotonic	338.9	2.10	33.89	1.05
CP25-M3	Monotonic	346.0	1.80	34.60	0.90
CP25-CA1	Cyclic	369.2	2.41	36.92	1.21
CP25-CA2	Cyclic	327.0	1.95	32.70	0.98
CP25-CA3	Cyclic	345.9	1.69	34.59	0.85

CP25-CB1	Cyclic	463.1	1.38	46.31	0.69
CP25-CB2	Cyclic	410.6	1.30	41.06	0.65
CP25-CB3	Cyclic	386.6	1.88	38.66	0.94
CP375-C1	Cyclic	433.9	2.15	43.39	1.08
CP375-C2	Cyclic	461.3	1.61	46.13	0.81
CP375-C3	Cyclic	409.8	2.23	40.98	1.12

4.3.2.1 Monotonic Loading

For this series, a set of unconfined cylinders, cast using conventional PVC formwork (100 x 200 (mm)), was tested under monotonic loading to obtain the compressive properties of the inner concrete. The tests were conducted in the same manner as in the CC specimen series. The cumulative results of the monotonic tests on the NSC-MP series are shown in Fig. 4.19. The average peak load for this series is 280 kN (35.65 MPa), and the average axial strain at the peak is 0.0036. Expansion at the peak load is averaged at 0.00064 (0.2 mm). At this stage, the inner cores exhibit the mechanical performance characteristic of NSC.

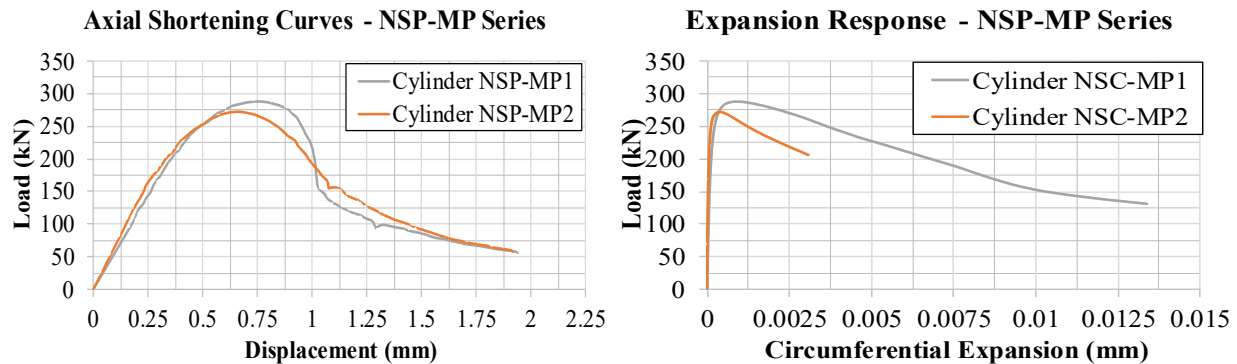


Figure 4.19. Axial shortening and expansion curves for the NSC-MP specimen series.

For the monotonic tests on confined prisms, the cumulative results of the response curves for the CP25-M series are shown in Fig. 4.20. The average strength is 352.7 kN (35.27 MPa), and the average axial strain at the peak, normalized to the height of the prism at 200 mm, is 0.0091. Compared to the unconfined cylinders, the prisms sustained a peak load that is higher by 26%. The axial strain is measured to be substantially higher, at 2.5 times the value of the average axial strain of the NSC cylinders. It is observed at this stage that the confined specimens exhibit higher load and strain capacity, with the later property being increased substantially; this observation is in accordance with the results obtained in the confined cylinder specimens.

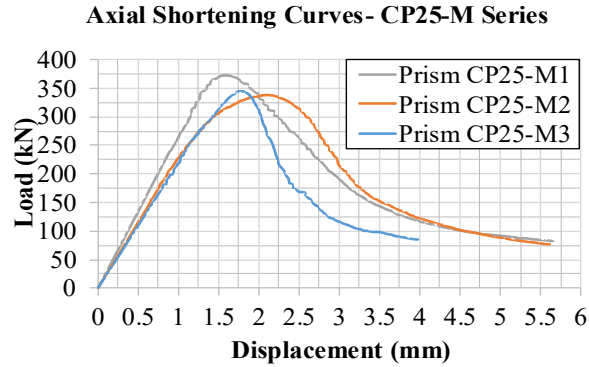


Figure 4.20. Axial shortening curves for the CP25-M specimen series.

4.3.2.2 Cyclic Loading

For the cyclic tests on confined prisms, the cumulative results of the response curves for the CP25-CA and CP25-CB series are shown in Fig. 4.21. Both specimen series CA and CB contain specimens of equivalent properties; the former was tested soon after hydration of the specimen, at the 28 days milestone, and series CB was tested after a long time (~150 days) due to difficulties operating the equipment during the COVID-19 pandemic.

In series CP25-CA, the average strength is measured to be 347.3 kN (34.7 MPa), obtained at an average axial strain of 0.01. In series CP25-CB, the average strength is measured to be 420 kN (42.0 MPa), obtained at an average axial strain of 0.0086. Compared to the unconfined cylinders, this series shows an increase in load bearing capacity of 24% and 50%, for series CP25-CA and CP25-CB, respectively. Axial strain at peak is also increased, reaching 2.81 and 2.14 times the average axial strain of the unconfined cylinders, similarly. It is observed that in the case of the CP25-CB series, the axial deformation capacity is reduced in return for an increase in strength, which can be attributed to the continued maturity of the concrete. Therefore, even at the presence of cyclic loading, both specimen series exhibit superior performance, compared to the unconfined control specimens, providing greater load bearing and axial deformation capacity.

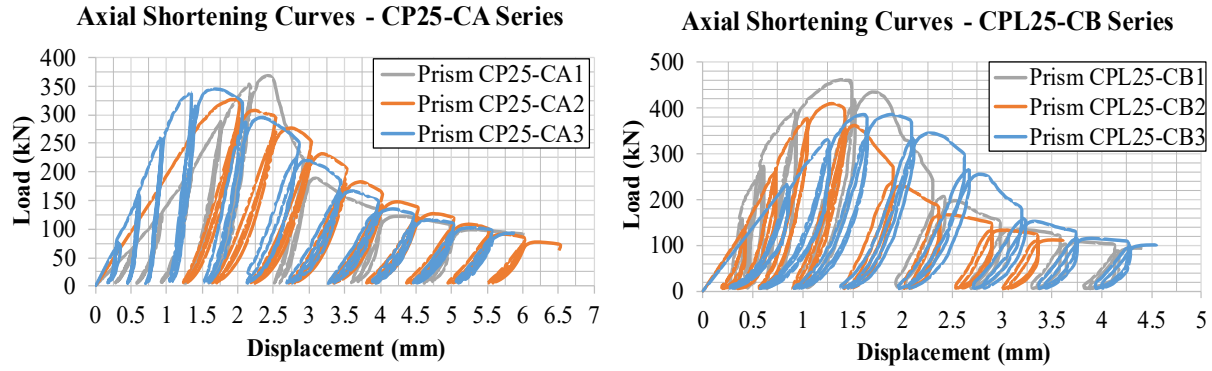


Figure 4.21. Axial shortening curves for the CP375-CA and CP375-CB specimen series.

For the cyclic tests on the CP375-C specimen series, the cumulative results of the response curves are shown in Fig. 4.22. The average strength is measured to be 435 kN (43.5 MPa), obtained at an average axial strain of 1.00%. Compared to the unconfined cylinders, this series shows an increase in load bearing capacity by 55% and an axial strain at peak to be 2.78 times higher. Similar to the confined cylinder specimens, further increase of the jacket thickness contributes to increase in load bearing capacity. The axial deformation capacity of this series is close to that obtained from specimen series CP25-CB. It is evident at this stage that increasing the jacket thickness to 37.5 mm provides further improvements to the mechanical performance of confined concrete.

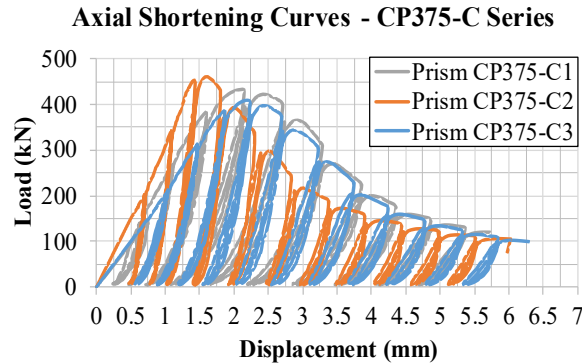


Figure 4.22. Axial shortening curves for the CP375-C specimen series.

4.4 Digital Image Correlation (DIC) Analysis

For the DIC analysis, the open source 2D-DIC MATLAB software Ncorr was used, combined with the post-processing tool of Nezerka (2014). The later tool was used to obtain the strain across splitting cracks on the jacket surface via virtual extensometers. To prepare the input for the analysis, each specimen was photographed during testing, using a Canon DSLR camera, which

was mounted on a tripod in front of the testing frame. Additional light was provided with a portable light projector, placed next to the camera. To obtain the pictures, a remote-controlled timer was attached on the camera and was programmed to take pictures every 10 seconds. The complete setup of the specimen surface as captured by the camera is as shown in Fig. 4.10. In this chapter, a sample analysis is shown, as well as the pertinent analysis parameters and the collective results are discussed.

4.4.1 Preparation of input

The preparation of the software input and analysis parameters of specimen CP25-C1 are shown below. The steps followed to conduct the DIC analysis are included the Ncorr instruction manual v1.2.2. Initially, the Reference Image is selected, which corresponds to the beginning of the experiment, as well as the Current Images of the analysis, which include images throughout the complete testing sequence of the specimen. Each image was renamed with the prefix “010A_” followed by the number of the picture used. The images were selected to be on the backbone curve of the specimen response. After the selection of the images, the software screen is as shown in Fig. 4.23.

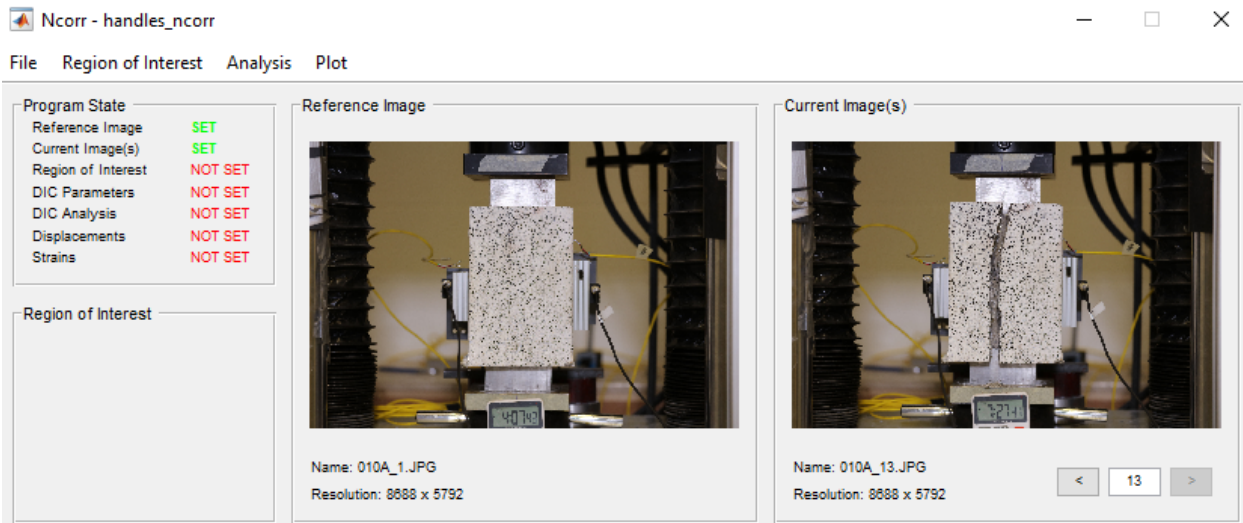


Figure 4.23. Selection of Reference Image and Current Images in Ncorr software screen.

Next, the Region of Interest (ROI) was drawn, which includes the test area of the jacket surface. Figure 4.24 shows the software screen for the definition of the ROI, which was defined by drawing a rectangular shape on the specimen speckled surface.

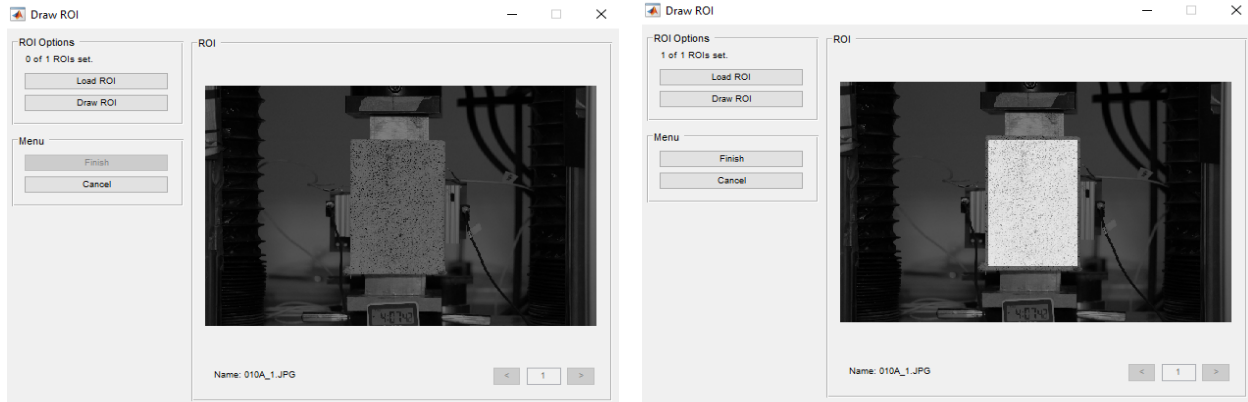


Figure 4.24. Determination of Region of Interest (ROI) in Ncorr software screen.

For the definition of the DIC Parameters for analysis, some iteration was necessary to define the appropriate combination of subset radius and spacing (Fig. 4.25). For optimal results, the subset radius was selected to be the smallest possible, with the aim of not leading to “noisy” data after the analysis, as per the Ncorr instruction manual (Blaber et al. 2015). Such data appear as individual points of strain and displacement of opposite sign, i.e., individual compressive strain points within the tensile strain field, and do not accurately represent the strains developed on the jacket surface. For a larger subset radius, the strain distribution appeared to be smoothed, with overestimation of the strains in proximity to the crack path. To account for the above criteria, the subset radius was selected to be 50 pixels and the spacing was selected at 3 pixels; the later property dictates the spacing between the subsets, with values ranging from 1-20.

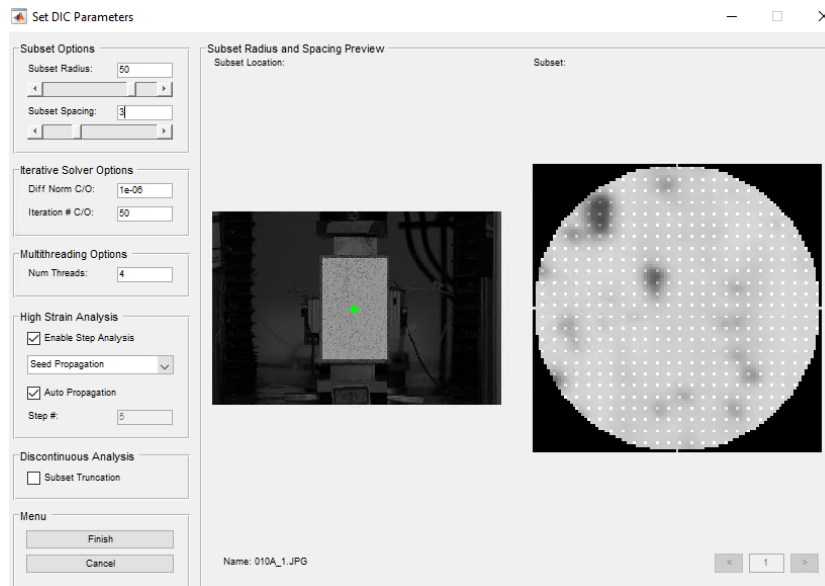


Figure 4.25. Selection of subset radius and subset spacing in Ncorr software screen.

After determination of the DIC parameters, the placement of the seeds and evaluation of the convergence criteria were defined. The number of seeds is determined by the number of available computer cores for multi-threading, which are determined during installation of the program; the main purpose in selecting multiple cores is to divide the computational load. Seed placement is critical for this analysis as it determines the initial estimation on which the DIC analysis is performed. The seeds are to be placed within the boundaries of the ROI throughout the execution of the analysis, in an arrangement that allows for evenly distributed proportions of the ROI, as such an approach facilitates convergence. The placement of the seeds is shown in Fig. 4.26.

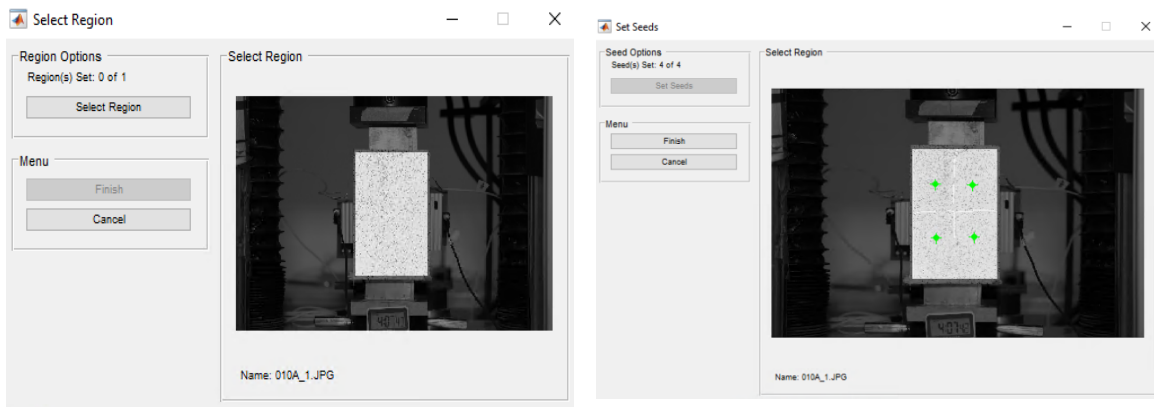


Figure 4.26. Placement of seeds for multi-thread analysis in Ncorr software screen.

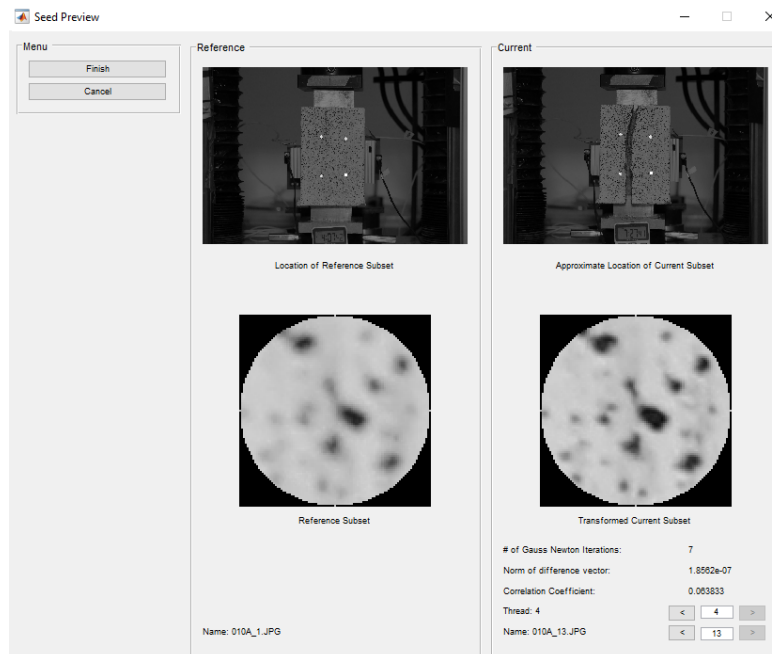


Figure 4.27. Evaluation of convergence criteria in Ncorr software screen.

In Fig. 4.27, the convergence criteria are displayed. It is shown that for the sequence of pictures included, the number of iterations performed (7) is less than the maximum number of iterations per image (50), which signifies early converge of the DIC analysis. Additionally, the correlation coefficient is within the bounds of $10^{-1} - 10^{-3}$, which is the acceptable range for this parameter. This is the final step prior to the execution of the analysis.

After execution of the analysis, the displacements are formatted (Fig. 4.28) and the strain fields calculated (Fig. 4.29), in accordance with the Ncorr manual. For the displacement, the unit-to-pixel ratio is defined, which ensures that the measured displacements are scaled to the objects shown in the pictures. This is achieved by manually defining two points of known distance in the Reference Picture, using an on-screen selection tool. After definition of the conversion rate, the correlation coefficient cutoff value is defined, which is used to remove points with extremely high or low values. For this example, this value is selected as 1.0, such that the points within the cracked zone are not considered.

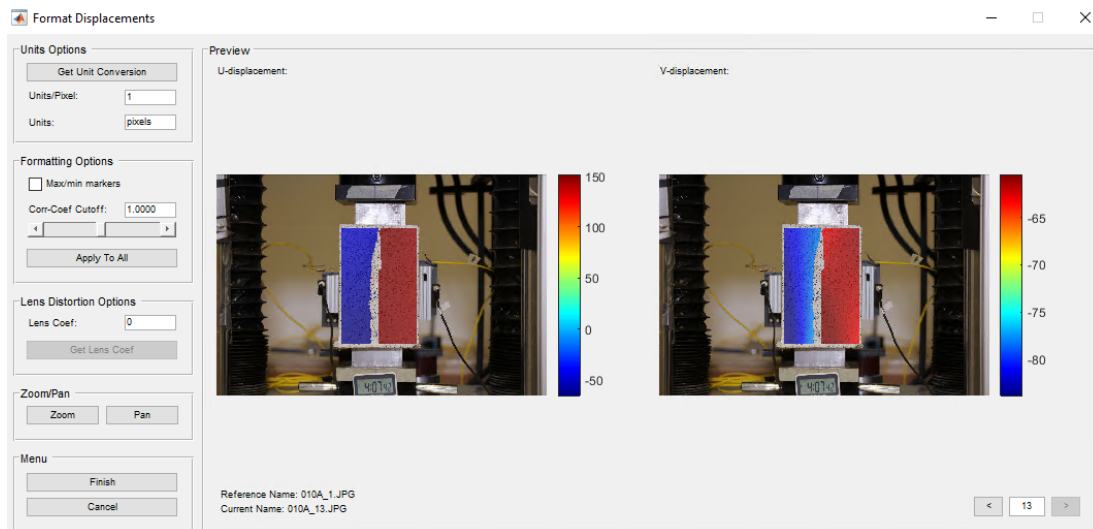


Figure 4.28. Formatting of displacements in Ncorr software screen.

For the calculation of strains, the Green-Lagrangian strains are considered. Parameter strain radius is selected to be in the default value of 15 (Fig. 4.29). At this point, processing of data in the Ncorr software is complete and the remainder of the analysis is conducted in the post-processing tool of Nezerka (2014).

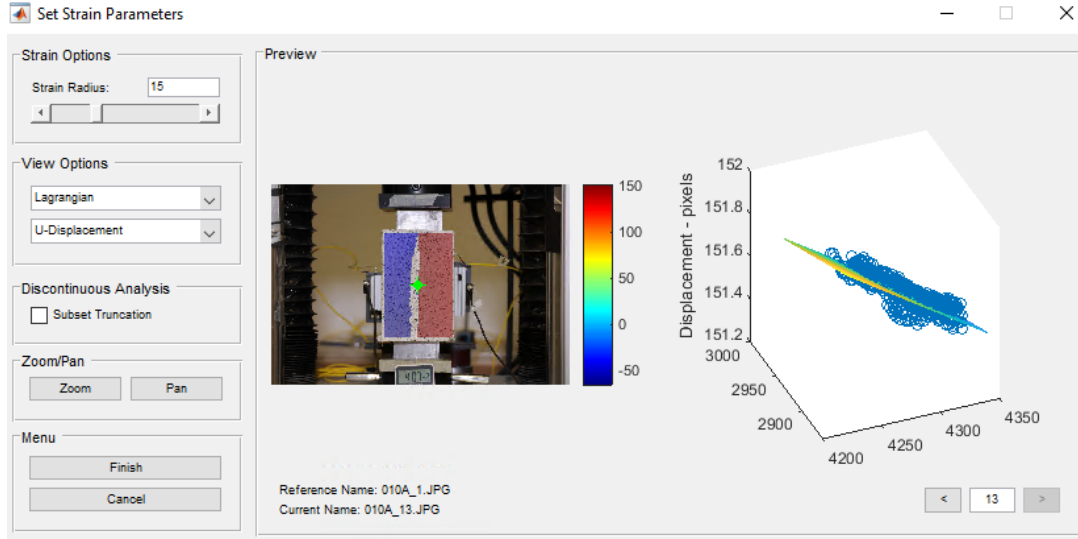


Figure 4.29. Calculation of strains in Ncorr software screen.

To ensure proper transfer of the data from Ncorr to the post-processor, the unit/pixel ratio in the formatting of the displacements was selected to be 1 (Fig. 4.28); the scaling of the displacements is subsequently performed in the post-processor based on the dimensions of the loading plate of the testing frame. The results of the lateral and axial strain fields are discussed further in the following section. Lastly, a set of virtual extensometers are placed laterally on the surface of the jacket, one on each side of the crack path, as shown in Fig. 4.30. This method allows for measurement of the crack opening, as well as description of the jacket tensile strain across the defined extensometer length.

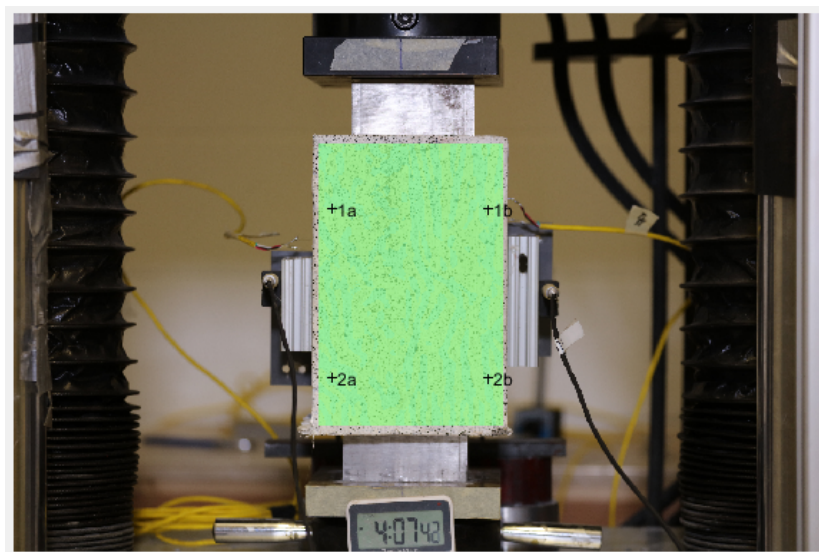


Figure 4.30. Placement of virtual extensometers in Ncorr post-processor.

4.4.2 Splitting failure within DIC zone

Of the specimens tested, three developed splitting crack failures within the DIC analysis surface, including CP25-M3, CP25-CA1 and CP374-C2. In Figs. 4.31-4.33 the lateral strain fields for each of the specimens CP25-M3, CP25-CA1, and CC375-C2 are shown, respectively. The results of the crack widths and strains measured from the virtual extensometers are shown in Fig. 4.34, whereas the tabulated results are given in Table 4.4.

In all cases of the three specimens, it was observed that significantly increased localized strains developed on the crack path, with tensile deformation exceeding the value of 5%. In specimen CC25-M3, at a lateral strain at 75% of the peak load, in the pre-peak branch, the outline of the splitting crack became apparent. At that stage, lateral strains reached a maximum of 2% and at peak load and, at peak, specimen CC25-M3 reached a maximum tensile strain of 4%. In the post-beak branch, at 75% of the peak load, the strain had localized and the surface of the UHPFRC disconnected entirely. In the following stages of the analysis, lateral strains exceeded conventional values in cementitious materials, due to the further opening of the splitting crack. Throughout the remainder of the jacket surface, compressive strains were developed, which did not exceed 1%.

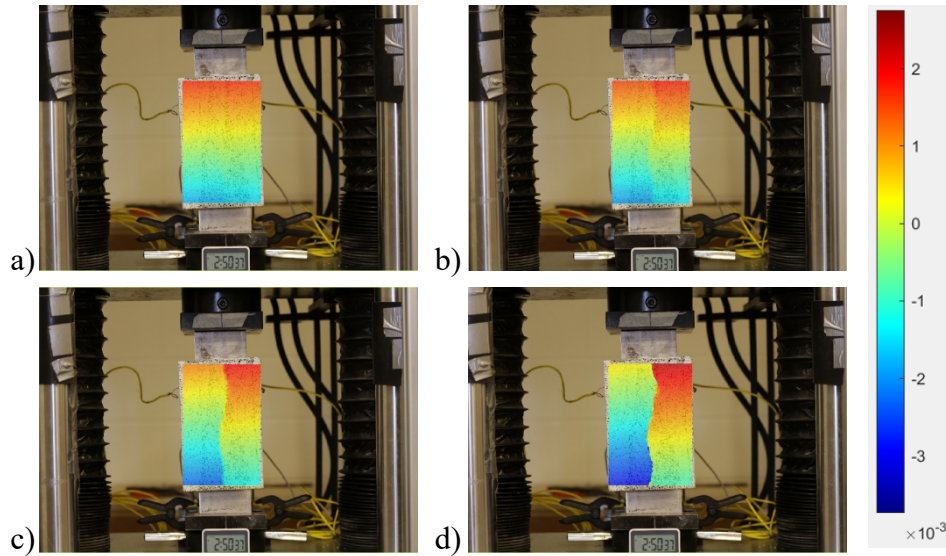


Figure 4.31. Lateral strain fields for specimen CP25-M3 at: (a) 75% F_{peak} in pre-peak, (b) F_{peak} , (c) 75% F_{peak} in post-peak, and (d) 50% F_{peak} in post-peak.

For specimens of the cyclic testing series CP25-CA1 and CP375-C2, the lateral strain at 0.7 of the peak load, in the pre-peak branch, reached 1.7% and 0.4%, respectively, which corresponded to 85% and 20% of the strain of specimen CP25-M3 at this stage. For both specimens, the distribution

of lateral strains was similar to specimen CP25-M3, with the governing failure mode determined by the splitting crack. For specimen CP25-CA1, at peak load, the strain is 3.3% (82.5% of specimen CP25-M3), whereas for specimen CP375-C2, the strain reached values of 1.4% (35%, similarly). In the post-peak branch, at 75% and 50% of the peak load, specimen CP25-CA1 reached lateral strain values of 6.2% and 40%, beyond which the splitting crack had appeared throughout the whole surface of the jacket. At this stage of the test, specimen CP25-M3 had already exceeded the strains encountered in CP25-CA1. Lastly, for specimen CP375-C2, at 75% and 50% of the peak load, the tensile strains reached 6.4% and 63.6%. Beyond the latter stage, the width of the dominant localized splitting crack continued to increase. It was found that in the post-peak branch, a parallel path developed close to the initial crack path, through which the splitting failure eventually localized.

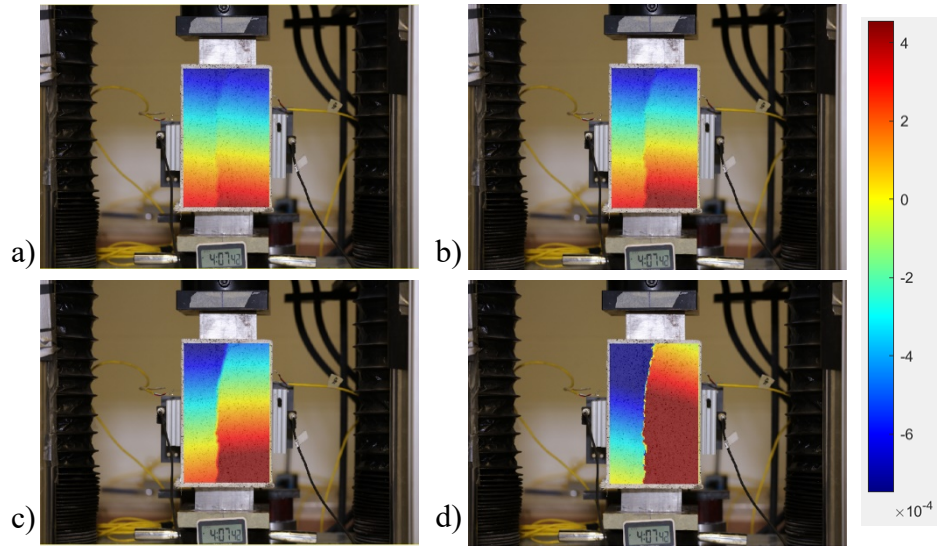


Figure 4.32. Lateral strain fields for specimen CP25-CA1 at: (a) 75% F_{peak} in pre-peak, (b) F_{peak} , (c) 75% F_{peak} in post-peak, and (d) 50% F_{peak} in post-peak.

Based on the above data, it was found that while in the ascending branch, the specimens tested in cyclic loading reached lower surface strains in the lateral direction, compared to the monotonically loaded specimen. Strains in the cyclically loaded specimens increased further in the post-peak branch, beyond the threshold of residual load equal to 50% of peak, whereas the monotonically loaded specimen localized to smeared strains greater than 40% at 75% of the peak load in the post-peak envelope.

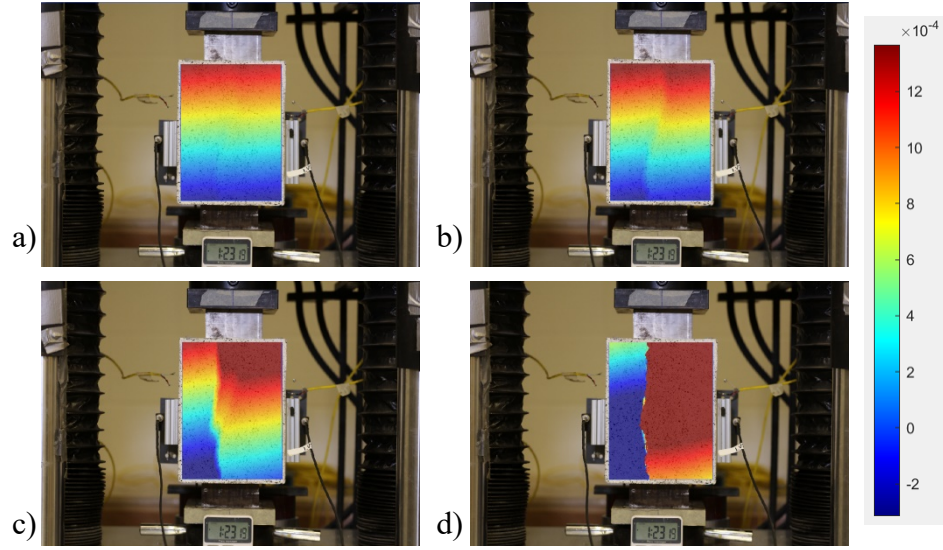


Figure 4.33. Lateral strain fields for specimen CC375-C2 at: (a) 75% F_{peak} in pre-peak, (b) F_{peak} , (c) 75% F_{peak} in post-peak, and (d) 50% F_{peak} in post-peak.

Table 4.4. Tabulated results of crack width and lateral strain measurements from the virtual extensometers.

Specimen Types	Loading Type	Crack Width (mm) / V.E. Strain (%)			
		75% F_{peak} Pre-peak	F_{peak}	75% F_{peak} Post-peak	50% F_{peak} Post-peak
CP25-M3	Monotonic	0.34 / 0.30	0.89 / 0.78	1.87 / 1.63	5.07 / 4.42
CP25-CA1	Cyclic	0.37 / 0.31	1.01 / 0.84	2.14 / 1.79	6.87 / 5.73
CC375-C2	Cyclic	≈ 0 / ≈ 0	≈ 0 / 0.036	0.50 / 0.402	4.34 / 3.47

For the lateral VEs, the gauge length for the CP25-M3 and CP25-CA1 specimens was 110mm and for specimen CP375-C2, the length was 125 mm. At 75% of the peak load, specimens with $t_{jacket} = 25$ mm, developed similar crack widths and lateral strains, at an average value of 0.35 mm. For specimen CP375-C2 the crack width was of negligible magnitude. At peak, the monotonically loaded specimen developed a crack width of 0.89 mm, which was 12 % lower than the cyclically loaded specimen of equivalent jacket thickness. For the cyclically loaded specimen of $t_{jacket} = 37.5$ mm, the crack width remained at negligible value. At 75% of the peak load in the post peak range, the crack width of the monotonic test was lower than that of the cyclic test by 12.6%, however, it was 3.74 times the strain of the cyclic test of higher jacket thickness. Lastly, at residual load equal to 50% of the peak, the crack width of the monotonic test was lower than that of the cyclic test by 26.2% and higher than the crack width of the cyclic test of higher jacket thickness by 16.8 %.

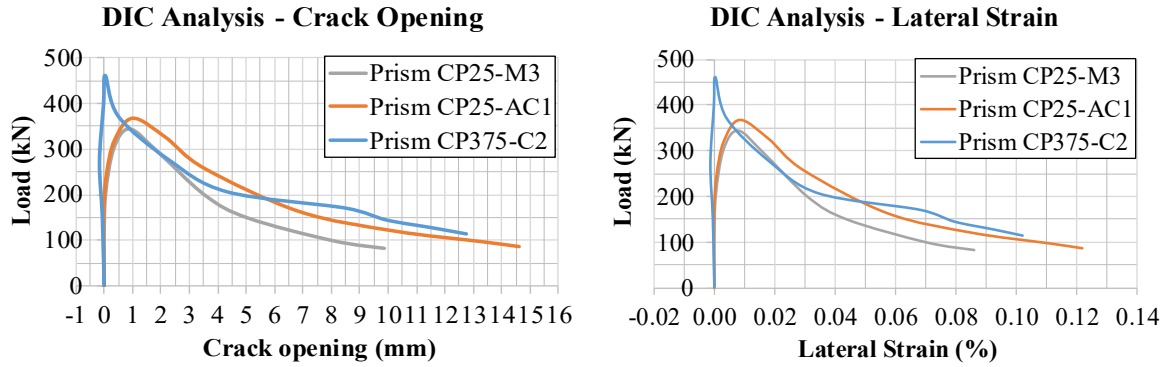


Figure 4.34. Crack opening (left) and lateral strains (right) curves obtained from VEs.

It is evident that higher strain magnitudes were obtained from the cyclic tests as compared with the monotonic ones; however, the lateral strains and vertical splitting crack localization was mitigated effectively by the thicker jackets.

4.4.3 Splitting failure outside DIC zone

The tabulated results of the maximum lateral and axial strains obtained from the DIC analysis on the CP specimen series are shown in Table 4.5. Figures 4.35-4.38 display the lateral strain fields for one specimen of each series. To account for the volume of the specimens, the remaining results are included in Appendix B.

As shown in Table 4.5, lateral strains in cyclically loaded specimens exceed the ones obtained from monotonic tests. On average, tensile strains on CP25-M and CP25-C specimens were 0.785% and 1.044%, respectively, corresponding to an increase by 33% in the cyclic tests. Specimens of the CC375 series display an average lateral tensile strain of 0.54%, which corresponds to 51% of the average lateral tensile strain the CC25-C specimens. Compressive lateral strains were markedly lower in most specimens, with an average value of 0.0046, 0.0052%, and 0.0017, for the CP25-M, CP25-C and CP375-C specimen series, respectively. Axial strains on the UHPFRC jacket surface were predominantly in tension, with averages to be 0.0049, 0.0075, and 0.0047, similarly. Compressive strains in the axial direction were substantially lower, due to the lateral expansion response of the jacket. The average values are measured to be 0.0008, 0.00224 and 0, for the CP25-M, CP25-C and CP375-C specimen series, respectively.

Table 4.5. Summary of maximum lateral and axial strains for CP specimens.

Specimen Types	Loading Type	Lateral Strain ϵ_{lat} (%)		Axial Strain ϵ_{lat} (%)	
		Tension	Compression	Tension	Compression

CP25-M1	Monotonic	0.90	0.58	0.4	0.16
CP25-M2	Monotonic	0.67	0.33	0.57	0.0
CP25-CA2	Cyclic	0.79	0	0.89	0.0
CP25-CA3	Cyclic	1.03	0.51	0.53	0.16
CP25-CB1	Cyclic	1.78	0.46	0.55	0.09
CP25-CB2	Cyclic	0.73	1.00	1.01	0.70
CP25-CB3	Cyclic	0.89	0.63	0.75	0.17
CC375-C1	Cyclic	0.35	0.01	0.37	0.0
CC375-C3	Cyclic	0.72	0.33	0.57	0.0

In the specimens described in this section, the splitting failure developed on the surfaces to the side of the DIC zone, or at the surface opposite to the one in the image. Strain fields in all cases were characterized by both tension and compression throughout the surface of the jacket; the distribution was affected by the location of the splitting crack in the neighboring faces. As it is shown in Figs. 4.37 and 4.38, tensile strains accumulated on the side of the surface next to the splitting failure, whereas on the opposite side, compressive strains developed. On the other hand, in specimens where failure occurred on the opposite side of the jacket, as shown in Figs. 4.39 and 4.40, the strain distribution was characterized by predominantly compressive values. Tensile strains also developed on these surfaces, in longitudinal-oriented areas, and showed greater magnitude towards the edge of the jacket surface; this is owed to the eccentricity of the splitting crack on the opposite side of the jacket.

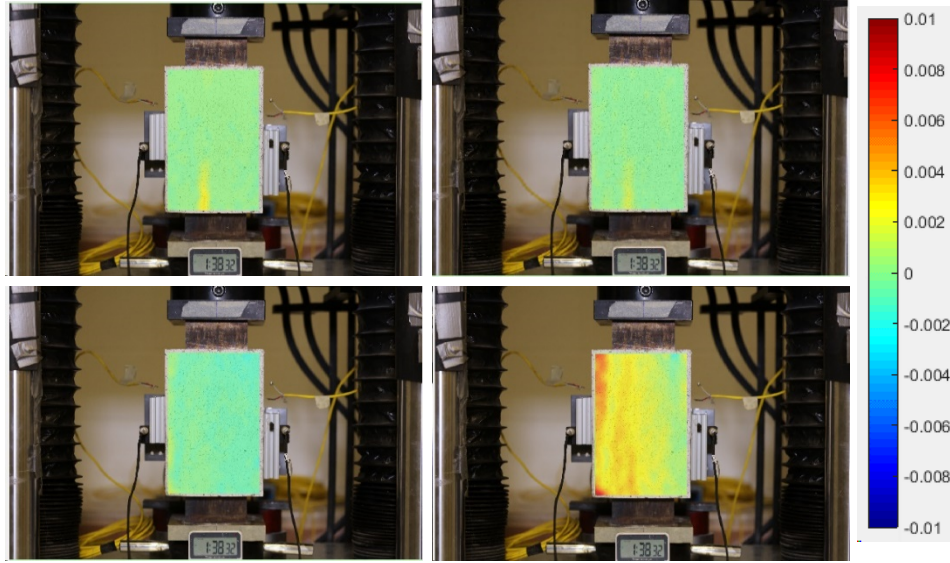


Figure 4.35. Lateral strain fields for specimen CP25-CA3 at: (a) 75% F_{peak} in pre-peak, (b) F_{peak} , (c) 75% F_{peak} in post-peak, and (d) 50% F_{peak} in post-peak.

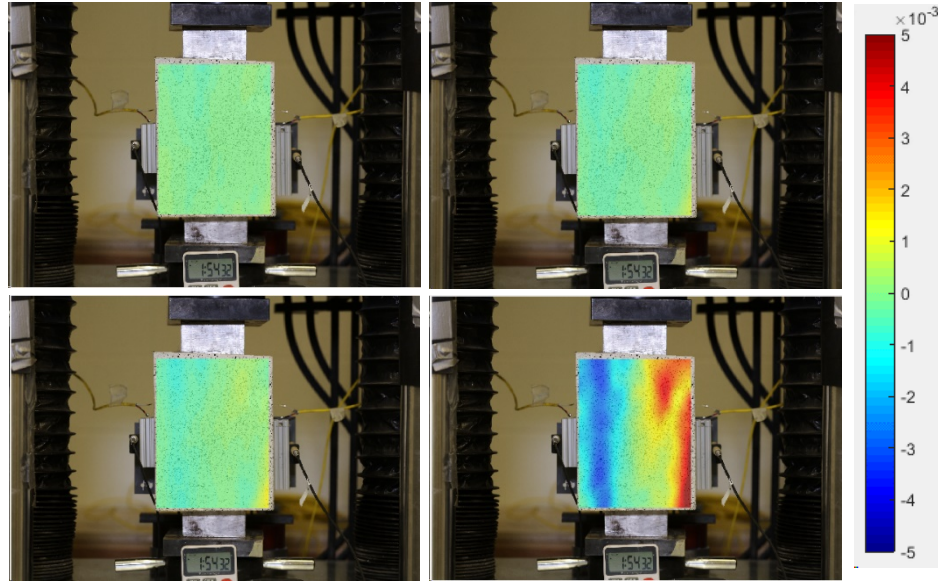


Figure 4.36. Lateral strain fields for specimen CP375-C3 at: (a) 75% F_{peak} in pre-peak, (b) F_{peak} , (c) 75% F_{peak} in post-peak, and (d) 50% F_{peak} in post-peak.

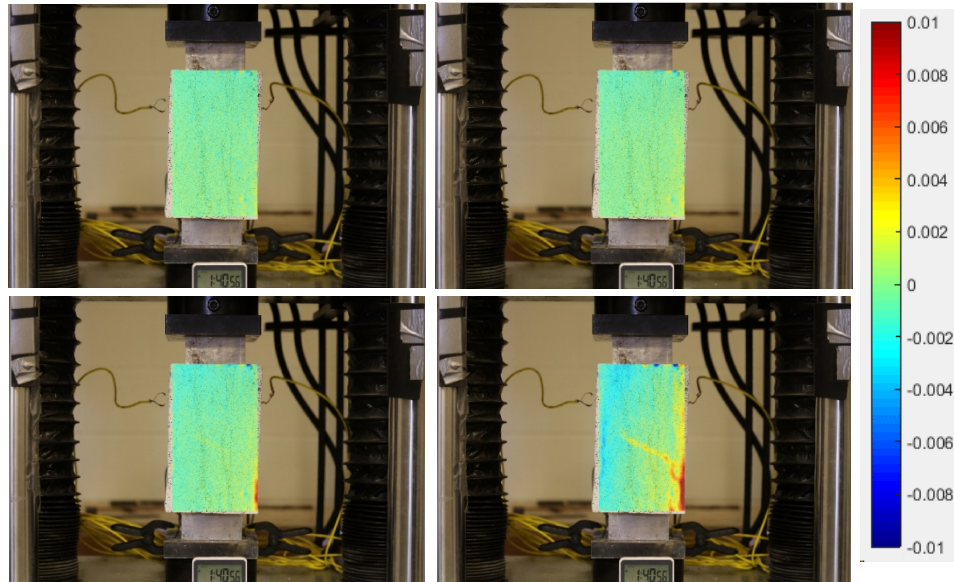


Figure 4.37. Lateral strain fields specimen CP25-M2 at: (a) 75% F_{peak} in pre-peak, (b) F_{peak} , (c) 75% F_{peak} in post-peak, and (d) 50% F_{peak} in post-peak.

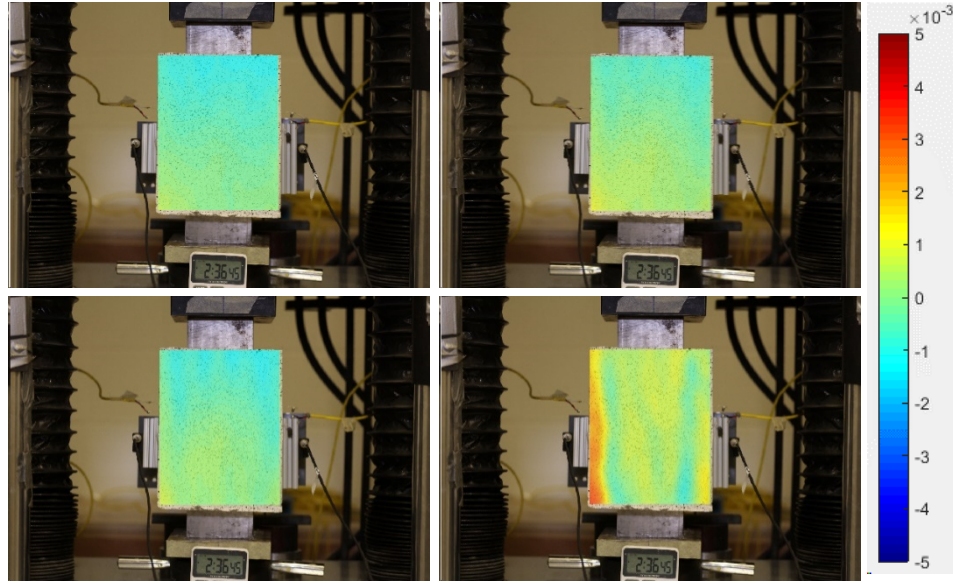


Figure 4.38. Lateral strain fields for specimen CP375-C1 at: (a) 75% F_{peak} in pre-peak, (b) F_{peak} , (c) 75% F_{peak} in post-peak, and (d) 50% F_{peak} in post-peak.

4.5 Summary

This experimental series was focused on determining experimentally the contribution of the confining effect of UHPFRC jackets on NSC cores. Two section geometries were evaluated, using jackets of thicknesses 25 mm and 37.5 mm, and tests were conducted using both monotonic and cyclic loading excitations. Jacket engagement occurred by the dilating pressures of the encased NSC core, leading to lateral tensile transverse strains to be developed throughout the jacket. Both external instrumentation and computational analysis via digital image correlation (DIC) were used to investigate this experiment.

1. The addition of the UHPFRC jackets improved the mechanical performance of the confined NSC cores, both in terms of strength and ductility. In the case of monotonic loading, the stress and strain at peak of cylindrical specimens were improved by approximately 30% in the presence of the thin ($t_{jacket} = 25$ mm) jacket. For the thickness of $t_{jacket} = 37.5$ mm, both the strength and strain were doubled at peak. For prismatic sections, the improvement in strength for the same jacket thickness was similar to what was observed in the case of the cylindrical specimens, at approximately 25%. Improvement of the strain at peak state was substantially higher, at 2.53 times the strain of the unconfined NSC cores.

2. Cyclic loading severely impaired the deformation capacity of the unconfined NSC cores, which was measured to be 1/3 of the strain at peak for monotonically loaded specimens. The contribution of the UHFPRC jackets allowed for the cylindrical confined specimens to reach 4.4 (0.88%) and 6.1 (1.22%) times the strains of the unconfined NSC cores, for the two jacket thicknesses studied, respectively. The strength was also increased, albeit more mildly, up to 20% and 60%, for the same jacket thicknesses, similarly. In the case of the prismatic specimens, strains at the peak reached values of 0.94% and 1.00%, for the two thicknesses of the jacket, respectively. Strength was increased, compared to the strength of the NSC-MP series, by 7% and 22%, similarly. Based on the above, it can be reasonably inferred that for higher strengths of the core concrete, the relative contribution in mechanical performance imparted by the jacket is reduced. However, the contribution of the confining action in deformation capacity showed impressive enhancement under cyclic displacement reversals, where the ductility of the specimens showed improvement even under the effects of cyclic excitation.
3. Expansion of the cylindrical specimens was markedly higher for unconfined specimens. In the CC25 series, for monotonic loading, the expansion was 73% lower, including both thicknesses of the jacket. For the cyclic loading specimens, this pattern was not apparent at the peak state. The unconfined specimens reached larger expansions at the residual branch almost entirely, reaching approximately 0.016 and 0.026, for monotonic and cyclic loading, respectively. In comparison, confined specimens for jacket thicknesses of 25 mm and 37.5 mm, showed an ultimate expansion of 0.014 and 0.018, for monotonic loading. The same measurements for cyclic loading were 0.015 and 0.029.
4. Based on the measurements obtained from the virtual extensometers during the DIC analysis of CP25 specimens, crack widths were shown to be larger for cyclically loaded specimens, by 13% at peak stress and by 33% at the ultimate stage, respectively. For the larger thickness of the jacket, crack widths at peak were measured to be zero, and were lower by 35.5% than the CP25 specimen at the ultimate stage. It was shown therefore, that increasing the jacket thickness could mitigate the lateral deformations throughout the jacket surface of the specimen.
5. All specimens, using both the cylindrical and prismatic sections, failed by one major localized crack. Multiple cracks can be seen developing on the top and bottom surfaces of

the jacket, without developing fully into complete separation of the surfaces. Based on observation of exposed cracked surface, the weak sections of the specimens are characterized by reduced volume of fibers in lateral orientation. Pouring of the concrete during casting was performed from one side, to ensure optimal placement of the fibers, making the mixing and casting process of the UHPFRC material pivotal to the performance of the jacket.

6. In prismatic specimens, the surface containing the splitting crack was characterized by higher tensile lateral deformation in the proximity of the crack. While minor compressive lateral strains were developed in the remainder of the surface, the majority of severe tensile deformation was shown to be along the primary crack path. Surfaces to the side of the crack surface, developed both compressive and tensile transverse strains, in response to the expansion of the jacket. The side next to the cracked surface of the specimen contained the tensile surface strains. In the case of the surfaces on the opposite side of the jacket, compressive strains were developed, along with tensile strains owing to the eccentricity of the crack path.

5 Analytical Compression Model for UHPFRC-Confined Concrete under Cyclic Loading

In this chapter, a constitutive relationship for confined concrete was formulated, based on the experimental results of the UHPFRC-confined cores of the previous chapter. The model was based on the confinement models established for conventionally confined concrete and was modified to account for the confinement effect imparted by the tension hardening property of the jacket. A modified compression envelope was defined based on the model by Thorenfeldt et al. (1987), while the rules for the formulation of the hysteretic loops in the unloading/reloading regime were determined based on the work of Darwin and Pecknold (1974); these models were chosen as benchmark from among the various alternatives, based on the review of the available models in literature, which is described in Chapter 2. The combined model was calibrated to the recorded axial shortening responses of the cylindrical and prismatic specimens described in Chapter 4. Adjustment of parameters of the models was performed to fit the experimental curves, with the purpose of determining an analytical model capable of reproducing the compressive behavior of UHPFRC-confined concrete.

5.1 Compression Model for Confined Concrete

The model described in this chapter consists of the envelope stress-strain curve in compression, which was obtained separately from the monotonically and cyclically loaded specimens, using the same approach. For the cyclically loaded specimens, the addition of the common point and turning point (referred to also as stability) curves is provided, as described in Chapter 2, as well as the definition of coordinates of the unloading/reloading hysteretic cycles, so as to simulate the response under the passive effect of the jacket.

5.1.1 Monotonic Envelope

The monotonic envelope was defined as per the model for compression proposed by Thorenfeldt, Tomaszewicz & Jensen (1987), as it is shown in Fig. 2.3 in Chapter 2, where the contribution of confinement from the UHPFRC jacket was explicitly considered. Modifications are introduced to the original model, to better correlate to the experimental responses of the specimens described in Chapter 4.

For the ascending branch of the stress-strain curve in compression, the function shown in Eq. 5.1 was used, where coordinates of stress and strain (σ_{en} , ε_{en}) represent any point of the branch on the envelope curve. The value of parameter n was modified so that the slope of the ascending branch was equal to the initial tangent modulus of elasticity of the experimental curves; parameter k was taken equal to 1.

$$\sigma_{en} = f'_{cc} \cdot \left[\frac{n \cdot (\varepsilon_{en} / \varepsilon_{cc})}{n - 1 + (\varepsilon_{en} / \varepsilon_{cc})^{nk}} \right] \quad (5.1)$$

In Eq. 5.1, f'_{cc} is the compressive strength of confined concrete and ε_{cc} is the corresponding strain at peak stress. The contribution of the confinement imparted by the UHPFRC jacket is considered through the confinement factors, K for the strength increase, and K_{strain} for deformation capacity increase, as shown in Eqs. 5.2 and 5.3, respectively.

$$f'_{cc} = f'_c \cdot K \quad (5.2)$$

$$\varepsilon_{cc} = \varepsilon_{co} \cdot K_{strain} \quad (5.3)$$

Confinement factors K and K_{strain} were obtained from Eqs. (5.4-5.5), where the empirical factors λ and λ_{strain} were introduced to account for the confinement effectiveness based on the type of section geometry. These factors were determined experimentally.

$$K = 1 + \lambda \frac{\sigma_{j,lat}}{f_c} \quad (5.4)$$

$$K_{strain} = 1 + \lambda_{strain} \frac{\sigma_{j,lat}}{f_c} \quad (5.5)$$

In Eqs. (5.4-5.5), $\sigma_{j,lat}$ is the lateral confining pressure imparted by the UHPFRC jacket, as it is described in Chapter 2. The material safety factor φ_c is taken to be 1 for this model since the objective is to fit the experimental values. The confinement effectiveness coefficient, k_{ef} , is estimated from Eq. 5.7, also described in Chapter 2.

$$\sigma_{j,lat} = 2k_{e,f}\varphi_c f_t \left(\frac{t}{h} \right) \quad (5.6)$$

$$k_{e,f} = 1 - \frac{(b^2 + h^2)}{3 \cdot (1 - \rho_{sc})bh} \quad (5.7)$$

The strain at the end of the plateau at peak stress is determined by strain magnitude $\varepsilon_{cc,o}$, as shown in Fig. 5.1, which is obtained from Eq. 5.8. This equation is adapted from Eq. 2.5, which is described for the same property in Chapter 2.

$$\varepsilon_{cc,o} = \varepsilon_{cc} \left(1 + \lambda_o \left(\frac{\sigma_{j,lat}}{f'_c} \right) \right) \quad (5.8)$$

In Eq. 5.8, parameter λ_o is an experimentally derived value. The descending branch starts at the end of the peak plateau, at coordinate $(f'_{cc}, \varepsilon_{cc,o})$. To determine the slope of this branch, a strain capacity ε_{cu} at 75% of the peak stress is calculated from Eq. 5.9, which was originally adapted from Eq. 2.6.

$$\varepsilon_{cu} = \varepsilon_{cc} \left(1 + \lambda_u \left(\frac{\sigma_{j,lat}}{f'_c} \right) \right) \quad (5.9)$$

In Eq. 5.9, parameter λ_u is an experimentally derived variable. The descending branch ends when the envelope stress reaches the value of $0.25f'_{cc}$, and the residual branch, assumed to be a plateau, starts from the same stress value.

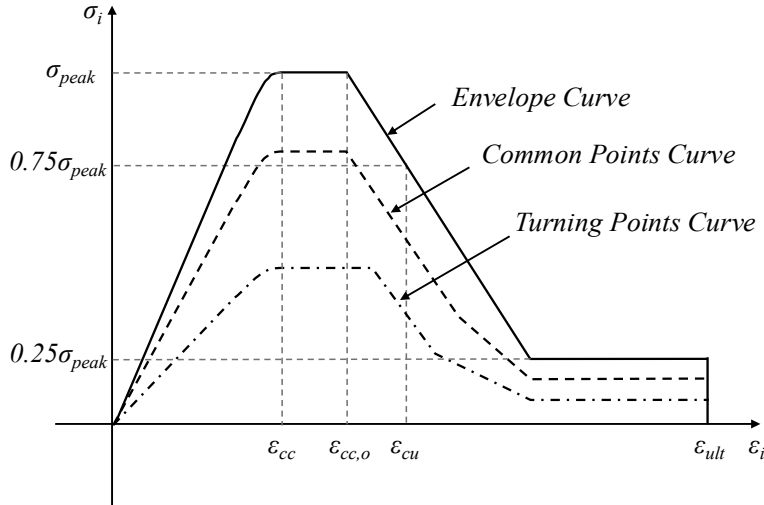


Figure 5.1. Envelope curve, common points curve and turning points curve (adapted from Darwin and Pecknold (1974)).

It is assumed that the residual branch continues for very large strains, however, for this study, it is considered that the envelope curve terminates at the end of the experiment. The value of ε_{ult} , which is the ultimate strain at the termination of the model, was determined experimentally, based on the corresponding strain from the experimental data; this is discussed below.

5.1.2 Cyclic Envelope

The envelope of the cyclic stress-strain response was defined in the same manner as the monotonic envelope described above, in Section 5.1.1, using the model by Thorenfeldt et al. (1987). The coordinates of the hysteretic loops were determined using the model of Darwin and Pecknold (1974). The common point and turning point curves were defined in this section (Fig. 5.1), as well as the coordinates of the hysteretic loops, which were adapted to correlate to the experimental evidence obtained in the experimental study.

In Fig. 5.1, the common point and turning point curves were divided into four branches, as described below by Equations 5.10-5.19. In these equations, $\sigma_{en\#}$ is the stress of the envelope curve (the # corresponds to the branch number), $\sigma_{cp\#}$ is the stress of the common points curve, and $\sigma_{tp\#}$ is the stress of the turning points curve. The first branch was defined by Eqs. 5.10 and 5.11, for the common point and turning point curves, respectively. This branch terminates for the value of strain on the envelope lying at the end of the ascending branch, at peak stress; thus, this refers to hysteretic loops that are formed by unloading from the envelope at axial compressive strains less than ε_{cc} .

$$\sigma_{cp1} = \frac{5}{6}\sigma_{\varepsilon n1}, \text{ for } \varepsilon \leq \varepsilon_{cc} \quad (5.10)$$

$$\sigma_{tp1} = \frac{1}{2}\sigma_{\varepsilon n1}, \text{ for } \varepsilon \leq \varepsilon_{cc} \quad (5.11)$$

The second branch was defined by the mathematical expressions given by Eqs. 5.12 and 5.13, similarly. These hysteretic loops occur when unloading from the envelope at a point on the descending branch, but the difference between the common point stress and turning point stress is *greater* than the difference between envelope stress and common point stress, as described in Darwin and Pecknold (1974). An illustration of the branch distinctions is provided in Fig. 5.2.

$$\sigma_{cp2} = \sigma_{\varepsilon n2} - \max\left(\frac{1}{6}\sigma_{\varepsilon n2}, \frac{1}{6}f_{cc}\right), \text{ for } \varepsilon_{cc} < \varepsilon \leq \varepsilon_{cu} \quad (5.12)$$

$$\sigma_{tp2} = \max\left(\frac{1}{2}\sigma_{\epsilon n2}, \frac{1}{2}f_{cc}\right), \text{ for } \epsilon_{cc} < \epsilon \leq \epsilon_{cu} \quad (5.13)$$

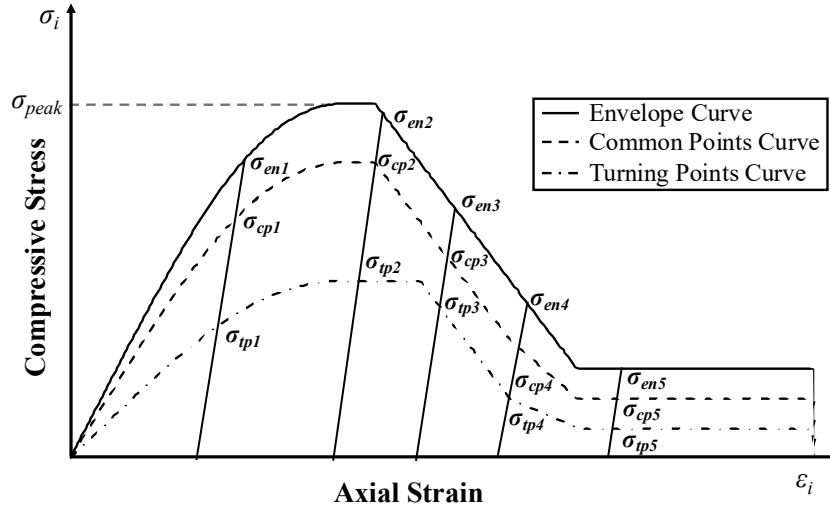


Figure 5.2. Envelope curve, common points curve and turning points curves for the model of this study.

The third branch is defined by Eqs. 5.14 and 5.15, similarly. These are hysteretic loops that begin from an envelope point that lies on the descending branch of the stress-strain response, as a continuation of the previous branch, while the difference between the common point stress and turning point stress is *less* than the difference between envelope stress and common point stress, as is illustrated in Fig. 5.2.

$$\sigma_{cp3} = \sigma_{\epsilon n3} - \frac{1}{6}f_{cc}, \text{ for } \epsilon_{cc} < \epsilon \leq \epsilon_{cu} \quad (5.14)$$

$$\sigma_{tp3} = \sigma_{\epsilon n3} - 2(\sigma_{\epsilon n3} - \sigma_{cp3}) = \sigma_{\epsilon n3} - \frac{1}{3}f_{cc}, \text{ for } \epsilon_{cc} < \epsilon \leq \epsilon_{cu} \quad (5.15)$$

The fourth branch was defined by Eqs. 5.16 and 5.17, similarly. This branch was defined after the third branch, while the turning point stress becomes *less* than 1/3 of the envelope stress σ_{en} as shown in Fig. 5.2;

$$\sigma_{cp4} = \frac{2}{3}\sigma_{\epsilon n4}, \text{ for } \epsilon_{cc} < \epsilon \leq \epsilon_{ult} \quad (5.16)$$

$$\sigma_{tp4} = \frac{1}{3}\sigma_{\epsilon n4}, \text{ for } \epsilon_{cc} < \epsilon \leq \epsilon_{ult} \quad (5.17)$$

In this model, it was assumed that the fourth branch of the descending branch is continued through the residual branch as well. As such, the following expressions (Eqs. 5.18-5.19) were used to obtain the common points and turning points in the fifth branch.

$$\sigma_{cp5} = \frac{2}{3} \sigma_{\varepsilon n5}, \text{ for } \varepsilon_{cc} < \varepsilon \leq \varepsilon_{ult} \quad (5.18)$$

$$\sigma_{tp5} = \frac{1}{3} \sigma_{\varepsilon n5}, \text{ for } \varepsilon_{cc} < \varepsilon \leq \varepsilon_{ult} \quad (5.19)$$

5.1.2.1 Calculation of the hysteretic loops' coordinates

The mathematical form of the hysteretic loops was determined based on the values obtained from the experimental data, according to the model of Darwin and Pecknold (1974). A schematic for the loop coordinates used in this study is provided in Fig. 5.3. The full path of a hysteretic loop consists of the unloading point on the envelope (U_0) following an unloading line with a slope proportionally reduced from the initial (tangent) modulus, until the intersection of the first unloading branch with the common points curve (U_1) and the intersection with the turning points curve (U_2); from that point, the second unloading branch extends to the axis of zero compressive stress (U_3), where the coordinates of the residual plastic strain on the axis of zero compressive stress are defined. Further straining in the same direction does not produce any stress, but the residual plastic strain magnitude is reduced to the value less than U_4 . Upon reversal of strain no stress is developed until the compressive strain reaches the value of U_4 , beyond which the compressive stress builds up following a linear reloading branch that goes through the last common point, U_1 ; and rejoins the nonlinear envelope after intersection with it at point U_5 . These five points define a complete hysteretic loop, each of which is discussed in this section.

It was stated that at any of the unloading points from the envelope curve (U_0), the unloading stress-strain response is divided into two branches: the first follows a path with a slope equal to the initial modulus of elasticity E_c , attenuated by factor γ_u , which is used to adjust the shape of the hysteretic loop (Fig. 5.3). Thus, the final slope of the unloading path, symbolized with E_u , is obtained according to Eq. 5.20:

$$E_u = \gamma_u \cdot E_c \quad (5.20)$$

Factor γ_u was introduced, based on observations and on the experimental evidence, to account for the degradation of the initial unloading slope of hysteretic loops of the model with progressive

damage accumulation for large axial strains, typically exceeding values of 0.015; loops become wider at that stage as more and more energy is dissipated due to damage. The width of the hysteretic loop in this instance is defined as the horizontal distance on the curve between U_3 and U_4 , as displayed in Fig. 5.3. The values of γ_u were manually adjusted, through a trial-and-error approach, so that the areas between the unloading/reloading path of the experimental hysteretic loops were equal to the ones obtained from the corresponding paths of the model. The values of γ_u ranged between 0 and 1, where the latter corresponds to $E_u = E_c$ and the procedure for the definition of this parameter is discussed in detail in Section 5.2.5.

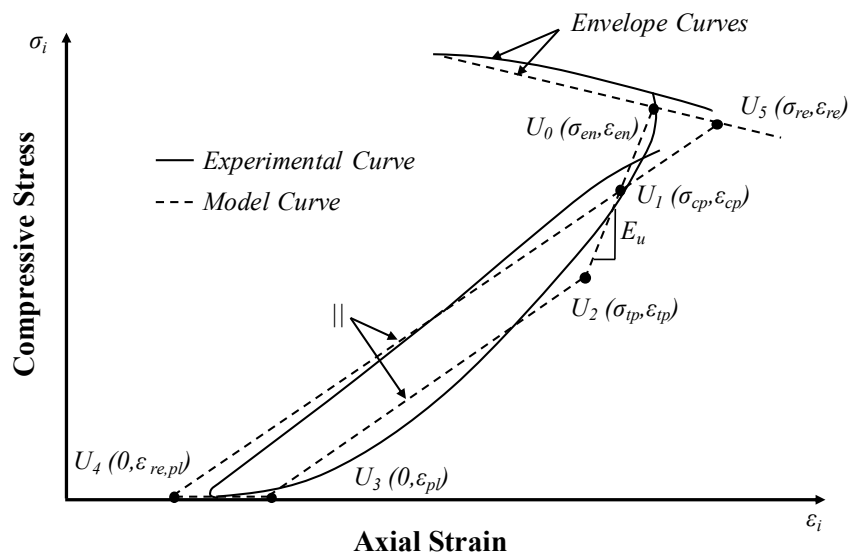


Figure 5.3. Demonstration of influence of shape parameter n in the ascending branch of the model.

The path of the first unloading branch intersects with the common point and turning point curves, at U_1 and U_2 , respectively. After the second intersection, the second branch of the unloading path is defined by the slope parallel to the slope between the plastic strain coordinates U_4 and U_1 . The branch extends until the intersection with the axis of zero compressive stress, at point (U_3), the horizontal ordinate of which is the unloading plastic strain ε_{pl} . This strain consists of one of the two definitions of plastic, or residual, strain, as proposed by Karsan and Jirsa (1969), which corresponds to the end of unloading path; the pertinent definition of the unloading plastic strain is provided by Eq. 2.58 of Chapter 2. For this study, a new expression was derived, which was based on the experimental evidence obtained from Chapter 4. This formula followed the basis of the original work by Karsan and Jirsa (1969), who correlated the plastic strain ε_p with the strain on the

envelope curve ε_{en} , at the start of the unloading. The procedure for the definition of the plastic strain ε_p is provided in Section 5.2.5 and the complete equation is described in Section 5.3.

The horizontal distance between points U_3 and the onset of reloading, U_4 represents further strain reversal at zero stress (i.e., unloading without resistance). Point U_4 is the beginning of the reloading branch, where the horizontal ordinate is the reloading plastic strain $\varepsilon_{re,pl}$, as defined by Eq. 2.59 of Chapter 2. Finally, the reloading branch is defined as the line between points U_4 and U_5 . The slope of this line is equal to the slope of the line defined between points U_3 and U_2 and intersects with the common points curve at point U_1 . Point U_5 is obtained when the reloading line intersects with the envelope curve.

5.2 Influence of Model Parameters

In this section, the influence of the pertinent parameters that influence the compression model behavior are discussed. These parameters include the confinement factor for strength K , the confinement factor for strain K_{strain} , the shape factor for the ascending branch of the envelope curve n , the parameter for the determination of $\varepsilon_{cc,o}$ (thus the length of the peak plateau) λ_o , the parameter for the determination of ε_{cu} (thus the slope of the descending branch) λ_u , the plastic strain ε_p at each hysteretic loop, factor adjustment of the width of the hysteretic loop γ_u , and the strain at ultimate stage (and termination of the model) ε_{ult} . The effect of each of the parameters was investigated, by formulating the model curves and comparing with the experimental response curves of the various specimens, until correlation of the curves was obtained. Optimal values for the parameters were proposed Section 5.3, based on the corresponding criteria for each branch.

5.2.1 Parameter n

This parameter affects the shape of the envelope curve, up to the ascending branch, as per Eq. 5.1. To obtain the optimal value, an iterative procedure was implemented, where various values were applied, until the initial stiffness of ascending curve of the model resulted in the same value obtained from the experimental response. An example of one of the specimens is illustrated in Fig. 5.4, where the ascending branch was defined from the origin to the peak coordinates $(\sigma_{peak}, \varepsilon_{cc})$.

In the example illustrated in Fig. 5.4, values of n were assumed ranging between 2 and 15. For the lower values of this range, i.e., for $n=2$, the model showed larger initial modulus of elasticity, thus overestimating the path of the ascending branch. The value of 15 was proposed by Mohammed et

al. (2022) for the ascending branch of UHPFRC materials in compression. For this value, the ascending branch underestimated the experimental response and approached an almost linear response for the greater portion of the branch. After trial and error, the optimal value for this example was taken equal to 8.

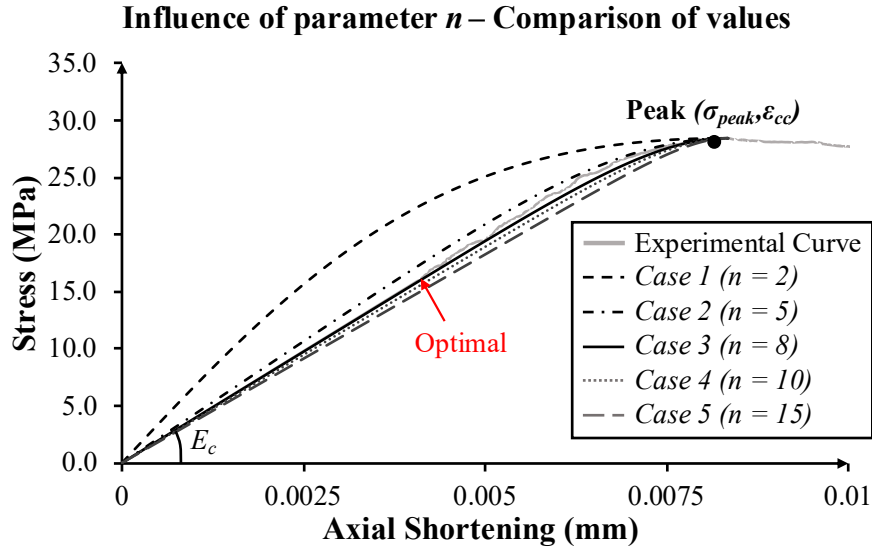


Figure 5.4. Demonstration of influence of shape parameter n in the ascending branch of the model.

5.2.2 Parameter λ_o

This parameter was used for the calculation of strain $\varepsilon_{cc,o}$, which corresponded to the end of the peak plateau. For the value of this parameter to be obtained, the envelope curve of the experimental response was drawn, up to the experimental strain ε_{cu} , at 75% of the peak load. The segment of the experimental response from peak load to that strain was then determined via linear approximation. An example of the calculation of $\varepsilon_{cc,o}$ is provided in Fig. 5.5.

Through iterations, the value of λ_o was altered until the end of the peak plateau matched with the start of the descending branch. For the value of 0.5, the length of the peak plateau was too short, as the start of the descending branch was located at a lower strain than the experimental response, while the opposite holds true for the value of 2. It should be noted that the value of 6 was proposed for UHPFRC materials in Mohammed et al. (2022). Finally, the value of 1 was selected as optimal for the length of the peak plateau, for this example.

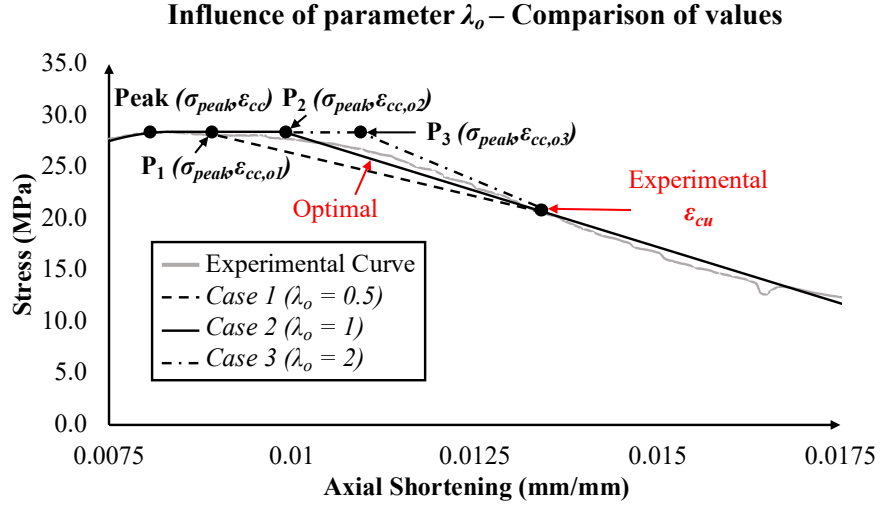


Figure 5.5. Demonstration of influence of parameter λ_o in the peak plateau of the envelope curve.

5.2.3 Parameter λ_u

This parameter was used for the calculation of ε_{cu} , which is the axial strain at 75% of the peak stress σ_{peak} , in the descending branch, as per the model proposed by Mohammed et al. (2022). This parameter was investigated after defining the end of the peak plateau, as per Section 5.2.2. By altering this parameter, through an iterative approach, the slope of the descending branch was affected, and the optimal value was selected by ensuring the area of the model curve matched the one obtained from the experimental response.

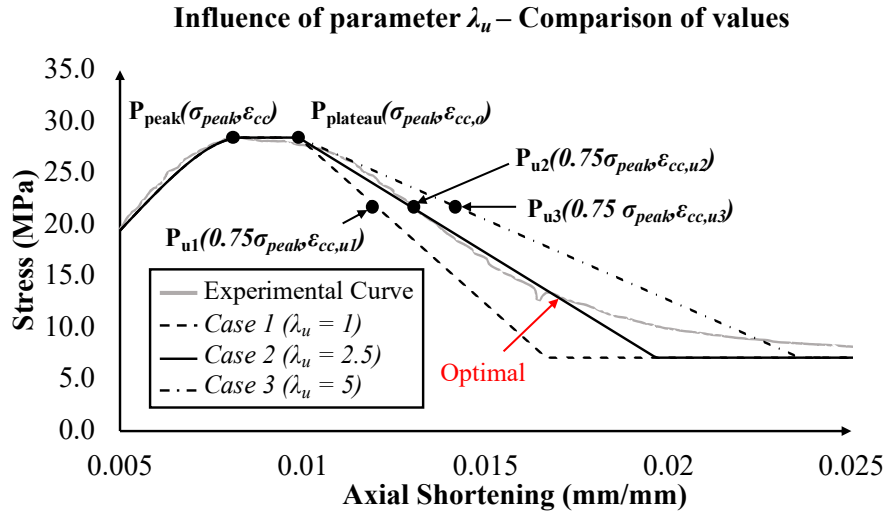


Figure 5.6. Demonstration of influence of parameter λ_u in the descending branch of the envelope curve.

An example of this approach is depicted in Fig. 5.6, where the lower value of λ_u , overestimates the slope of the descending branch, while the opposite holds true for the higher value of 5. For this example, Case 2 where the value of λ_u was 2.5 resulted in a descending branch that matched the optimal slope, i.e., equal areas between the model and experimental curves.

5.2.4 Factors K , K_{strain} , and ε_{ult}

The confinement factors described in Eqs. 5.2-5.3 were used to account for the increase in strength and axial deformation capacity, due to the presence of the UHPFRC jacket. These factors were determined by comparing the compressive strength of the unconfined specimens described in Chapter 4, with the corresponding UHPFRC-confined specimens, from which parameters λ and λ_{strain} were determined. While in classical models, such as the Modified Kent and Park model (Scott et al. 1982), it is assumed that the same confinement factor K is utilized for both strength and deformation capacity, for this study, they were defined separately due to the disproportionate contribution of the UHPFRC jacket in deformation capacity; this holds true particularly in the cyclic loading cases. As such, factors K and K_{strain} were obtained as the ratio of the confined strength over unconfined strength, and confined deformation at peak over unconfined deformation at peak, respectively. The resulting values are shown in Section 5.3, for each specimen category.

The ultimate strain ε_{ult} is the strain at which the model response is terminated. It should be noted here that, in the experiments described in Chapter 4, the tests were terminated at approximately 25% of the peak stress, to ensure safety of the loading apparatus. Regardless, for this study, it was assumed that the ultimate strain ε_{ult} corresponded to the end of the range of experimental results.

5.2.5 Plastic Strain ε_p and Factor γ_u

After the envelope curve parameters were determined, for both the monotonic and cyclic loading cases, the coordinates of the model hysteretic loops were defined. The first step was to define the common points and turning points curves, as described in Section 5.1.2. The unloading strains were obtained from the experimental tests of Chapter 4, where the strains at the start of the model unloading branches were the same as in the experiments.

To obtain the value of plastic strain ε_p , both the second unloading branch (from U_2 to U_3) and the reloading branch (from U_4 to U_1) were approximated with linear segments. An iterative procedure was then implemented, where the slope of the second unloading branch was adjusted until it

matched the slope of the reloading branch. Thus, the plastic strain ε_{p1} and the reloading plastic strain $\varepsilon_{re,pl1}$ were defined at the intersection the x-axis (axis of zero compressive stress), as per the example provided in Fig. 5.7. The same procedure was conducted for the second cycle of the same hysteretic cycle (ε_{p2} and $\varepsilon_{re,pl2}$ in Fig. 5.7) and this approach was implemented for all hysteretic loops recorded during the experiments. The results of this process are shown, for all specimens, in Section 5.3.

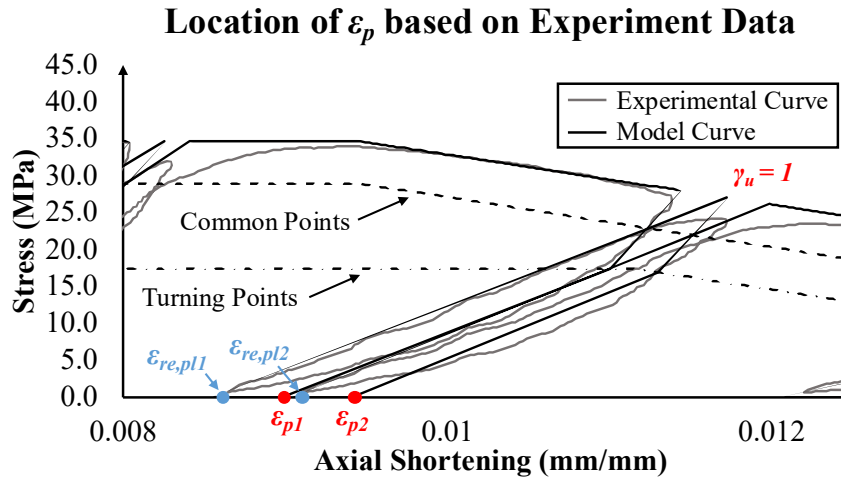


Figure 5.7. Demonstration of determination of plastic strain ε_p ordinates for the model hysteretic loops.

After the determination of the plastic strain ε_p values, factor γ_u was defined, following an iterative procedure. This factor affected the slope of the first unloading segment of the hysteretic loop, thus affecting the intersection between the first unloading branch and the turning points curve (U_1 in Fig. 5.3). An example of the various values of this factor is illustrated in Fig. 5.8. In the case of $\gamma_u = 1$, it can be seen that the cycle width is overestimated, which results in a larger hysteretic loop. This, in turn, also affects the starting point and the width of the second cycle of the same loop, resulting in a larger total dissipated energy (area enclosed by the loop), compared to the experimental response. By reducing the value of γ_u to 0.25, as shown in Fig. 5.8, the optimal hysteretic loop width was achieved, and the slope of the reloading branches of both cycles matched the experimental response.

As mentioned previously, the phenomenon described above was observed for large strains (> 0.015), and typically after the second half of the descending curve. This was attributed to the fact that the adopted hysteretic model was originally tailored for normal concrete constitutive laws,

which, as per the unconfined cylinder tests shown in Chapter 4, were able to reach axial smeared strains up to approximately 0.0125 (effectively 1/3 of the strains attained in the UHPFRC-confined specimens). This factor was therefore implemented to adjust the slope of the first unloading branch and, by extension the width of the hysteretic loops.

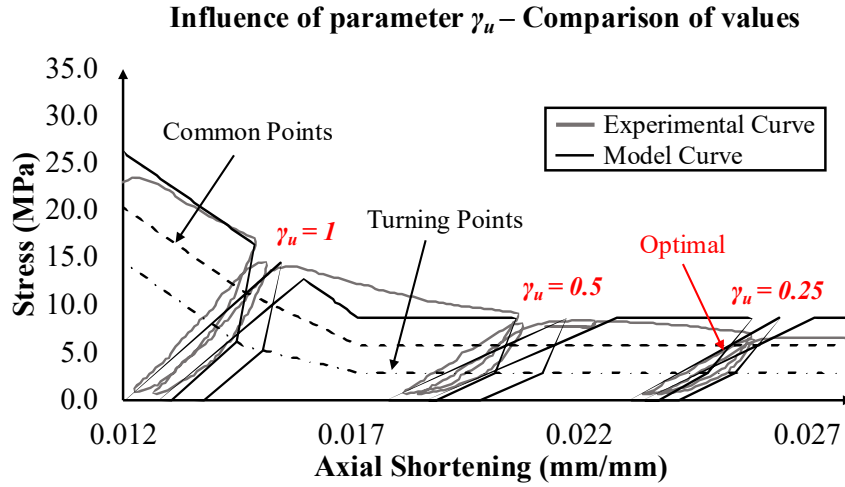


Figure 5.8. Demonstration of influence of factor γ_u on the slope of the first unloading branch of the hysteretic loops.

5.2.6 Sample Complete Curves

Based on the model equations and parameters described above, samples from the complete envelope curves and hysteresis loops are provided in this section, for reference. Figure 5.9 shows the envelope curve assumed for specimen CC25-M2, Fig. 5.10 shows the envelope curve and hysteretic cycles for specimen CC25-C1 and Fig. 5.11 shows the same for specimen CC375-C2. The same approach was implemented for all specimens, with the exception of specimen CC375-C1, which was not included in the calculations, on account of inconsistency in the recorded data.

5.3 Results for Model Parameters

The results of the values for the various parameters mentioned above are included in this section. Among these parameters, the following are presented: K , K_{strain} , n , λ_o , λ_u , ϵ_p , γ_u , and ϵ_{ult} . The parameters are distinguished between the envelope and hysteresis loops parameters and the significance of their values is discussed.

5.3.1 Envelope Curve

The results for the parameters for the envelope curves are presented in Table 5.1. Among all types of specimens, almost all parameters showed approximately the same values, between the monotonic and cyclic loading cases. The ratio λ_{strain} was defined separately for each loading case. This illustrates the disproportionate contribution of the UHPFRC jacket to deformation capacity, compared to the contribution in strength.

Table 5.1. Calibration of parameters for each specimen type.

Specimen Type	Loading Type	λ	λ_{strain}	n	λ_o	λ_u	$\varepsilon_{ult}/\varepsilon_{cc}$
Confined Cylinder	Monotonic	1.5	3	7	0.5	2	4
	Cyclic		20				
Confined Prisms	Monotonic	10.5	60	7	1	10	3
	Cyclic		140				

The parameters listed in Table 5.1 may be used in general application of the model. It is recommended that ratio λ_{strain} is considered separately for the two types of loading, to account for the significant contribution in deformation capacity in the cyclic loading case.

5.3.2 Hysteresis Loops

For the plastic strain ε_p , the evolution of values obtained throughout the experiments is shown in Fig. 5.9. The magnitudes of the plastic strain ε_p , in comparison with the axial strain ε_i , are presented normalized to the strain at peak, for each corresponding experiment. From the pattern of values presented in Fig. 5.9, the function that describes the evolution of ε_p was obtained, as it is described in Eq. 5.21. For both the first and second cycles of the performed experiments, it was determined that the plastic strain values were practically identical, thus the same function was assumed for subsequent cycles.

The function displayed in Eq. 5.21 can be applied for the effectively confined part of a UHPFRC-confined concrete cross section.

$$\frac{\varepsilon_p}{\varepsilon_{cc}} = -0.018 \cdot \left(\frac{\varepsilon_{en}}{\varepsilon_{cc}}\right)^3 + 0.14 \cdot \left(\frac{\varepsilon_{en}}{\varepsilon_{cc}}\right)^2 + 0.68 \cdot \frac{\varepsilon_{en}}{\varepsilon_{cc}} \quad (5.21)$$

For factor γ_u , the pattern of evolution throughout the tests of all specimens is displayed in Fig. 5.10. The results are expressed in reference to the axial strain ε_{en} normalized to the strain at peak

ε_{cc} , for each specimen, and Eq. 5.22 was defined according to the pattern of the experimental response. The maximum value of γ_u is 1, and therefore, Eq. 5.22 was separated in two components, as they are shown respectively in Eq. 5.22a and 5.22b.

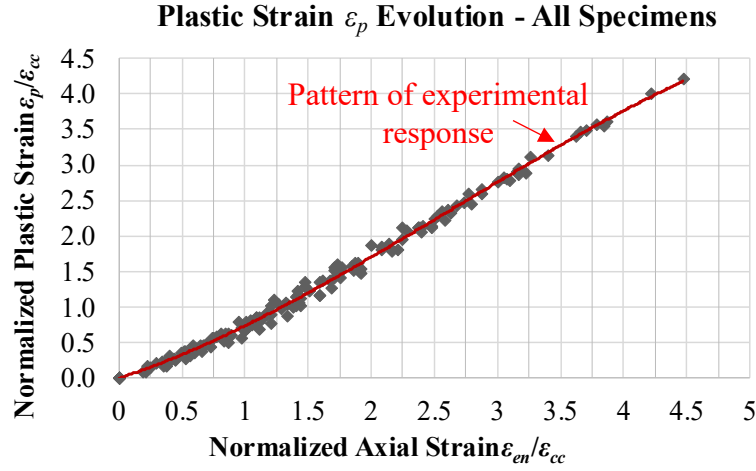


Figure 5.9. Pattern of plastic strain ε_p evolution for all specimens.

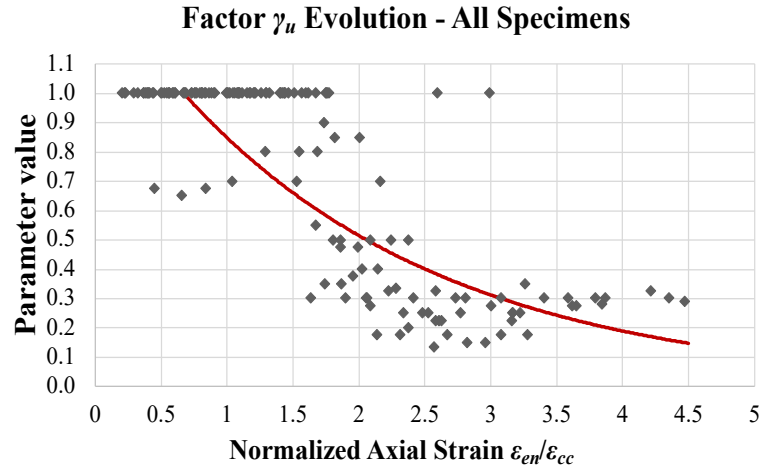


Figure 5.10. Pattern of factor γ_u evolution for all specimens.

The following function is valid for all types of specimens included in this study, with circular or rectangular sections.

$$\gamma_u = 1, \text{ for } \varepsilon_{en}/\varepsilon_{cc} \leq 0.7 \quad (5.22a)$$

$$\gamma_u = 1.4 \cdot e^{-0.5 \cdot \frac{\varepsilon_{en}}{\varepsilon_{cc}}}, \text{ for } \varepsilon_{en}/\varepsilon_{cc} > 0.7 \quad (5.22b)$$

5.4 Application of the model

Two examples of application of the model are provided, to illustrate the process of calculation. A schematic of the sections for each example is shown in Fig. 5.11. For the first case, a circular section with diameter $D = 500\text{mm}$, is assumed, confined with a UHPFRC jacket of $t_j = 37.5\text{mm}$. The section is composed of normal strength concrete with $f'_c = 25\text{MPa}$, $E = 31000\text{MPa}$ and $\varepsilon_{co} = 0.0021$. The UHPFRC has a tensile strength of $f_t = 11\text{MPa}$. It is assumed that the inner core undergoes cyclic axial strains with increments of strain $\varepsilon_{in} = 0.0001$, and unloading, with two cycles, commencing at the following strains $\varepsilon_{un} = [0.0025, 0.005, 0.075, 0.01, 0.0125, 0.015]$.

For the second case, a square section with $b = h = 450\text{mm}$, is assumed, confined with a UHPFRC jacket of $t_j = 25\text{mm}$. The section is composed of normal strength concrete with $f'_c = 30\text{MPa}$, $E_c = 33000\text{MPa}$ and $\varepsilon_{co} = 0.0022$. The UHPFRC has a tensile strength of $f_t = 11\text{MPa}$. It is assumed that the inner core undergoes cyclic axial strains with increments of strain $\varepsilon_{in} = 0.0001$, and unloading, with two cycles, commencing at the following strains $\varepsilon_{un} = [0.0025, 0.005, 0.075, 0.01, 0.0125, 0.015]$. The pertinent calculations are shown below.

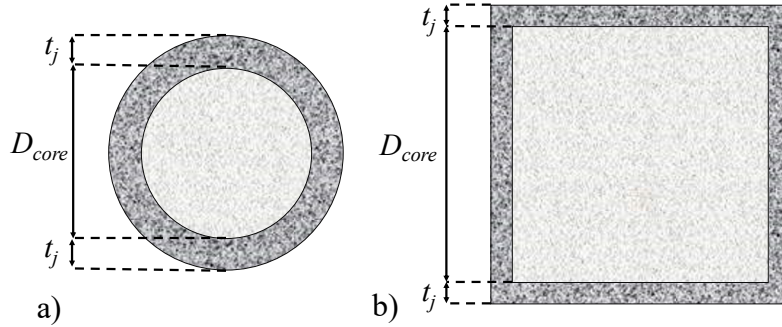


Figure 5.11. UHPFRC-confined section for examples: a) Circular; b) Square.

Case 1: UHPFRC-confined cylindrical section:

$$k_{e,f} = 1 \quad (\text{Eq. 5.7})$$

$$\sigma_{j,lat} = 2 \cdot 0.99 \cdot 1 \cdot 11 \cdot \left(\frac{37.5}{500} \right) = 1.65 \quad (\text{Eq. 5.6})$$

$$K = 1 + 1.5 \cdot \frac{1.65}{25} = 1.1 \quad (\text{Eq. 5.4})$$

$$K_{strain} = 1 + 20 \cdot \frac{1.65}{25} = 2.32 \quad (\text{Eq. 5.5})$$

$$f'_{cc} = 25 \cdot 1.1 = 27.5 \text{ MPa} \quad (\text{Eq. 5.2})$$

$$\varepsilon_{cc} = 0.0021 \cdot 2.32 = 0.0049 \quad (\text{Eq. 5.3})$$

$$\varepsilon_{cc,o} = 0.0049 \cdot \left(1 + 0.5 \cdot \left(\frac{1.65}{25}\right)\right) = 0.00506 \quad (\text{Eq. 5.8})$$

$$\varepsilon_{cu} = 0.0049 \cdot \left(1 + 2 \cdot \left(\frac{1.65}{25}\right)\right) = 0.00555 \quad (\text{Eq. 5.9})$$

$$f_{res} = 25\% \cdot 27.5 \text{ MPa} = 6.875 \text{ MPa}$$

$$\varepsilon_{ult} = 4 \cdot 0.0049 = 0.02 \quad (\text{Table 5.1})$$

Plastic strains ε_{un} were calculated based on Eq. 5.21, for each strain increment listed in ε_{un} . Factor γ_u was calculated based on Eq. 5.22, also for each strain increment listed in ε_{un} . The envelope curve was defined as described in Section 5.1. The final result is illustrated in Fig. 5.12.

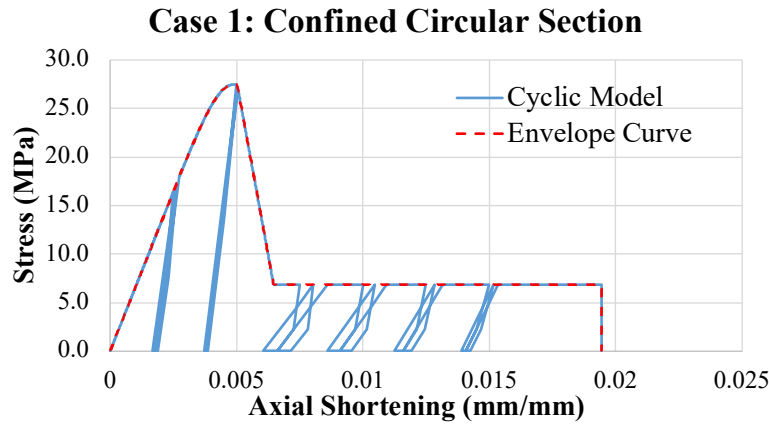


Figure 5.12. Envelope curve and hysteretic loops for the first example.

Case 2: UHPFRC-confined square section:

$$k_{e,f} = 1 - \frac{(200^2 + 200^2)}{3 \cdot 200 \cdot 200} = 0.33 \quad (\text{Eq. 5.7})$$

$$\sigma_{j,lat} = 2 \cdot 0.33 \cdot 1 \cdot 11 \cdot \left(25/450\right) = 0.4 \text{ MPa} \quad (\text{Eq. 5.6})$$

$$K = 1 + 10.5 \cdot \frac{0.4}{30} = 1.14 \quad (\text{Eq. 5.4})$$

$$K_{strain} = 1 + 140 \cdot \frac{0.4}{30} = 2.87 \quad (\text{Eq. 5.5})$$

$$f'_{cc} = 30 \cdot 1.14 = 34.2 \text{ MPa} \quad (\text{Eq. 5.2})$$

$$\varepsilon_{cc} = 0.0021 \cdot 2.87 = 0.006 \quad (\text{Eq. 5.3})$$

$$\varepsilon_{cc,o} = 0.006 \cdot \left(1 + 1 \cdot \left(\frac{0.4}{30}\right)\right) = 0.00608 \quad (\text{Eq. 5.8})$$

$$\varepsilon_{cu} = 0.006 \cdot \left(1 + 10 \cdot \left(\frac{0.4}{30}\right)\right) = 0.0068 \quad (\text{Eq. 5.9})$$

$$f_{res} = 25\% \cdot 34.2 \text{ MPa} = 8.55 \text{ MPa}$$

$$\varepsilon_{ult} = 3 \cdot 0.006 = 0.018 \quad (\text{Table 5.1})$$

Similar to the previous example, plastic strains ε_{un} were calculated based on Eq. 5.19, for each strain increment listed in ε_{un} . Factor γ_u was calculated based on Eq. 5.20, also for each strain increment listed in ε_{un} . The envelope curve was defined as described in Section 5.1. The final result is illustrated in Fig. 5.13.

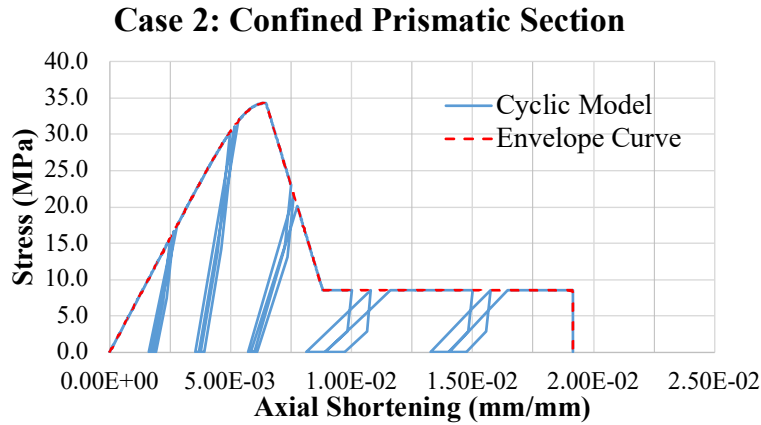


Figure 5.13. Envelope curve and hysteretic loops for the second example.

5.5 Summary

In this chapter, an analytical model was developed and applied, based on the experimental data obtained from the Confined Cylinder and Confined Prism tests of Chapter 4. The model used the compression envelope proposed by Thorenfeldt, Tomaszewicz & Jensen (1987), as it was shown in Mohammed et al. (2022), while the hysteresis loop coordinates, and cyclic parameters were defined based on the approach shown in Darwin and Pecknold (1974). The model was applied to

both the monotonically and cyclically loaded specimens. Parameter definition and proposed values were described, such that the model can be used to simulate the compressive behavior of UHPFRC-confined concrete. It should be noted here that the model should be validated with data from an independent experimental investigation pertinent to this study – this will be performed when such data become available.

1. For the monotonically loaded specimens, the envelope was distinguished into four main branches, the ascending, the peak plateau, the descending and the residual. The ascending was defined up to the peak stress coordinates, as obtained in the experimental curves, where the peak plateau extended until the start of the descending branch. The end of the descending branch was reached at 25% of the peak stress, in the post-peak range, and the residual branch extends until the end of the experiments presented in this study. Adjustment of the curves was matched with the experimental data, by determining each branch separately.
2. For the cyclically loaded specimens, the envelope was defined as in the monotonic tests, with the inclusion of the hysteretic loops in the corresponding location. The loops were defined based on the common points and turning points curves, as described in Eqs. 5.10-5-19, as they were presented in the work of Darwin and Pecknold (1974). The coordinates of the unloading and reloading paths were obtained from the experimental data, specifically the location of the plastic strain ε_p . The slope of the first unloading path from the envelope to the intersection with the turning points curve was manually adjusted to match the hysteresis loops of the experimental data.
3. The plastic strain ε_p showed dependency on the magnitude of axial shortening, requiring alteration from the original formula, to the one described in Eq. 5.19, which was defined based on the present experimental data. The equation proposed is suitable for both circular and rectangular sections.
4. Parameter γ_u was introduced to modify the slope of the unloading path from the envelope curve to the turning points curve, to better match the hysteretic response from the experimental data, and thus adjusting the width of the hysteresis loops. For all specimens, the proposed equation was given from Eq. 5.20, which was derived from the pattern of the experimental response.

5. Two examples were provided, containing one circular and one rectangular section of normal concrete confined with UHPFRC jackets, and the compressive stress -strain relationship under cyclic axial deformations were provided.

6 Design of Structural Component Tests

The structural-scale experimental part of the thesis was conducted in the strong floor / strong wall system of York's High Bay Structural Lab. The program is described in the present and the following chapter, in three main stages: 1) The design and casting of RC columns, using older seismic design principles and sections with deficient reinforcement detailing, 2) The first phase of testing to cause initial damage of a number of the column specimens by subjecting them to combined axial load and reversed cyclic lateral displacement of increasing magnitude, simulating earthquake action, and 3) The main phase of the experimental program comprising repair or strengthening of the columns with thin UHPFRC cover-replacing jackets and testing them under axial load and reversed cyclic lateral displacement histories of increasing amplitude, to failure. Details of the experimental program that are strictly relevant to the third stage are discussed further in Chapter 7. For the use of the UHPFRC jackets, the same type of prepackaged cementitious material that was used in conducting the confinement investigation of Chapter 4 (Densit[®]) is used in the structural component tests. The aim of this series of tests was to provide tangible evidence regarding the performance of cover-replacement with UHPFRC jackets in retrofitting and strengthening applications, and to evaluate the potential of the material for incorporation in seismic retrofitting design provisions.

6.1 Preparation and Design of RC Column Specimens

Two section types are investigated for the column specimens, both in rectangular form with equal external dimensions (cross section of 300mm $\times$ 300mm), one containing 4 $\times$ 20M bars (=1200 mm²) for longitudinal reinforcement and one containing 6 $\times$ 15M bars (same total bar area =1200 mm²), lap spliced above the footing, over a length of $24D_b$ in order to represent detailing practices used in pre-1980's construction (Franz 1972). The difference in the bar sizes was meant to investigate the effect of different bond demands along the lap splices under cyclic loading. Stirrups were provided using the same arrangement for both section types, comprising 8 mm diameter ($\varnothing 8$) smooth bars spaced at 250mm. The first stirrup was placed at 100mm from the upper surface of

the footing. Two clear column heights were considered, designed to induce different types of failure: equal to $L_s=1450$ mm and 850 mm; thus, shear demand in the shorter columns was almost double that of the taller columns. The footing and cap beam sections were 300mm wide, i.e., equal to the column dimensions. The footing length was 900 mm, encompassing two vertical openings symmetric with respect to the column axis, with center to center spacing of 600 mm which is the dimension of the grid of openings in the High Bay Strong Floor. Footing thickness was 500 mm. The cap beam was 300 mm wide, 400 mm thick and 500 mm long symmetric to the column axis. Footings and cap beams were heavily reinforced to ensure rigidity in their response and the same cross-sectional properties were used in all specimens. Specimen details are shown in Figs. 6.1-6.3. Table 6.1 lists the characteristics and identification code of each specimen.

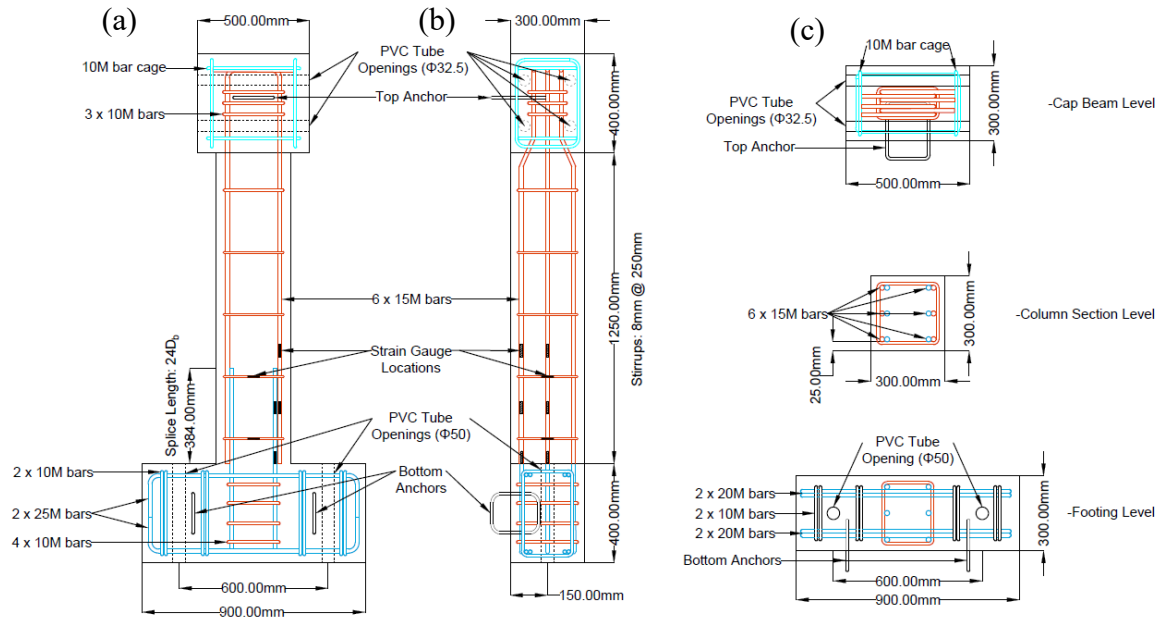


Figure 6.1. Geometry and reinforcement details of CT1-15M. (a) plane of action; (b) transverse to the plane of action; (c) sectional arrangement. Note: These details concern specimens CT1-15M-R and CT1-15M-S prior to any intervention.

Identification nomenclature was as follows: specimen “CT1-15M-R” corresponds to a column with a clear height of 1250 mm (“CT”), prepared in batch “1”, containing “15M” longitudinal reinforcement bars; the last letter, (R or S), if it exists, it refers to either retrofitted or strengthened specimens with cover replacement using UHPFRC material. Columns with the clear height equal to 650 mm are symbolized with “CS”. During testing, all columns were subjected to a constant

axial load ratio equal to $\nu = 15\%$, which corresponds to approximately 300kN. (The axial load ratio is defined as the ratio of the axial load magnitude divided by the gross sectional area and by the concrete uniaxial compressive strength: $\nu = P/(A_g f'_c)$).

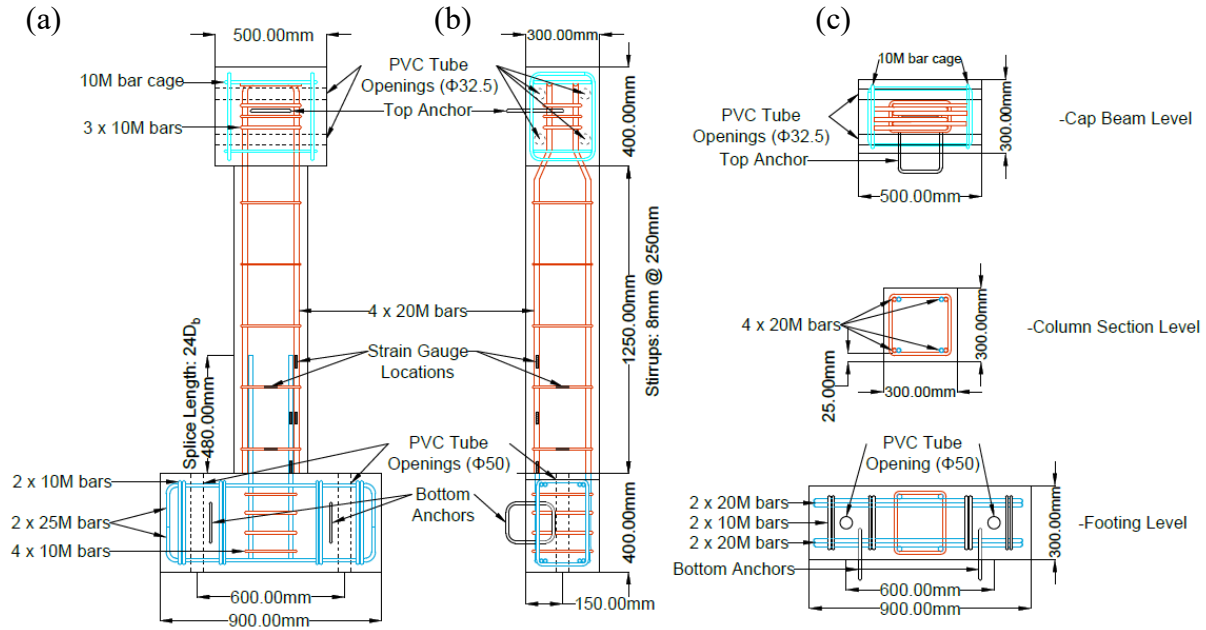


Figure 6.2. Geometry and reinforcement details of CT2-20M. (a) plane of action; (b) transverse to the plane of action; (c) sectional arrangement. Note: These details concern specimens CT2-20M-R and CT2-20M-S prior to strengthening or repair.

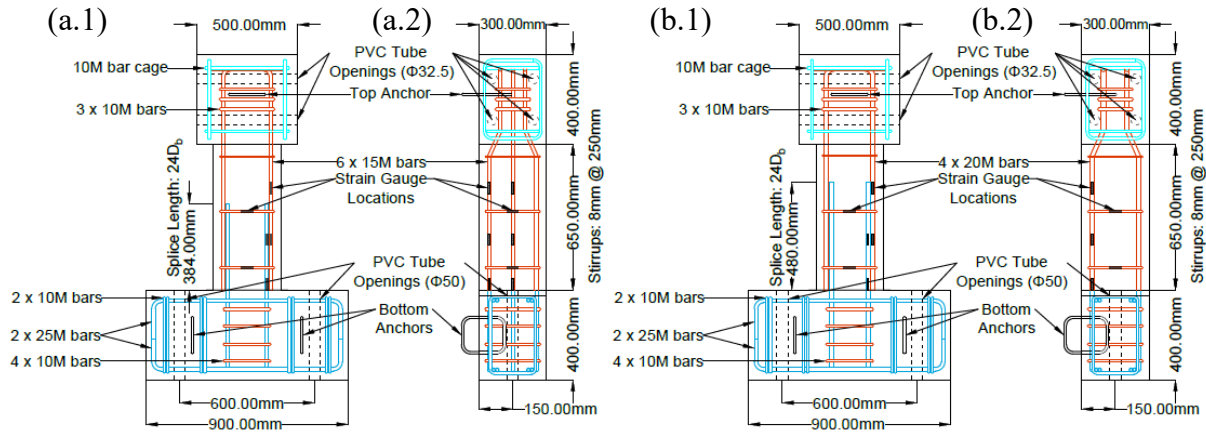


Figure 6.3. Geometry and reinforcement arrangement of (a.1) specimen CS3-15M(in the plane of action of the test), (a.2) transversally to the plane of action; Note that CS3-15M-R column had these details in the initial state; (b.1) specimen CS3-20M in the plane of action, (b.2) transversally to the plane of action. These are also the initial details of CS3-20M-R columns.

Table 6.1. Column specimen identification and parameters.

Column ID	Retrofit Type	Long. Bars (mm)	Lap Splice Length (mm)	Aspect Ratio
CT1-15M	Initial State	15M (16)	384	4.2
CT1-15M-R	Repair	15M (16)	384	4.2
CT1-15M-S	Strengthening	15M (16)	384	4.2
CT2-20M	Initial State	20M (19.9)	480	4.2
CT2-20M-R	Repair	20M (19.9)	480	4.2
CT2-20M-S	Strengthening	20M (19.9)	480	4.2
CS3-15M	Initial State	15M (16)	384	2.1
CS3-15M-R	Repair	15M (16)	384	2.1
CS3-20M	Initial State	20M (19.9)	480	2.1
CS3-20M-R	Repair	20M (19.9)	480	2.1

*All stirrups were smooth wires having 7.8 mm diameter; Shear reinforcement ratio was 0.133%.

The flexural capacity of the RC columns was calculated using Response 2000 (Bentz (2000)). Cover was 25 mm to the surface of the stirrup, and the material properties were $f'_c = 25$ MPa and $f_y = 400$ MPa, for the concrete and steel, respectively. The concrete model implements the base curve as defined by Popovic (1973), with the modifications proposed by Porasz (1989); default properties were used for the steel model. For all cases, the axial load ratio was $\nu = 0.15f'_c A_g$. The complete moment curvature response curve is shown in Fig. 6.4, where the maximum calculated flexural moments were 92 kNm and 97.7 kNm, for the section types with 15M ($D_b = 16$ mm) and 20M ($D_b = 20$ mm) longitudinal reinforcement, respectively. The section types with 20M bars showed the highest curvature at peak, at a value of 32.8×10^{-3} rad/m (61% higher than the $D_b = 16$ mm case).

The resistance curves were also estimated for each case, using the Response 2000 software, as shown in Fig. 6.4- without, however, taking into account the limitation on the post yielding strain development capacity of the lap splices imparted by the short lap length. Column specimen series such as the CT1-15M and CT2-20M ($L_v = 1450$ mm) showed a more ductile response, with the maximum shear force up to 67kN, while series CS3-15M and CS3-20M ($L_v = 850$ mm) showed brittle behavior, with the maximum shear force up to 108kN. The total expected drift was obtained

by considering the contribution of lateral displacement owing to bar slip at the plastic hinge location, which was obtained using the graphs proposed by Tastani and Pantazopoulou (2013), as shown in Eq. 6.1.

$$\Delta_{slip} = slip / (d - d') \quad (6.1)$$

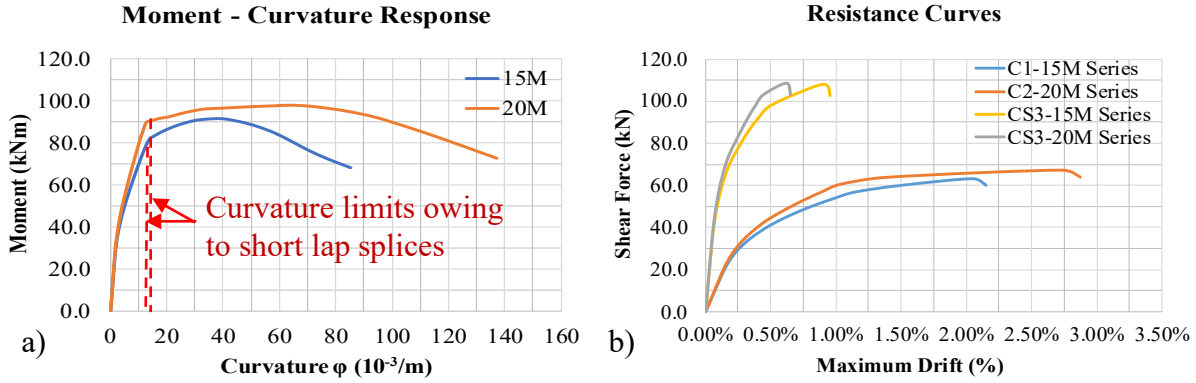


Figure 6.4. Moment-curvature (a) and resistance curves (b) for the RC column specimens.

For the CT1-15M and CT2-20M columns, the total drift at the ultimate stage was 2.7% and 3.3%, respectively. In the same fashion, for the CS3-15M and CS3-20M columns, the expected drift at peak was at 1.5% and 1.1%, respectively.

When considering the limit imposed by the short lap splice on the strain development capacity of the reinforcement based on the design charts developed by Tastani and Pantazopoulou (2015) as depicted in Fig. 6.5(b) it follows that the bar can hardly support a strain that is $1.2\epsilon_y$ for a low bond strength of 5MPa, and 2.4 to $4.8\epsilon_y$ for reinforcement with high to low slope of strain hardening. The corresponding limiting values of curvature, for low bond strength, are 11.51×10^{-3} rad/m and 12.65×10^{-3} rad/m, for the 15M and 20M cross sections, respectively (shown in Fig. 6.4). Beyond these limiting values, the bars within the short lap spliced connection have reached their maximum sustainable slip, at which point the plastic and residual branches of the bond stress – slip relationship have been exhausted, and no higher strains are developed.

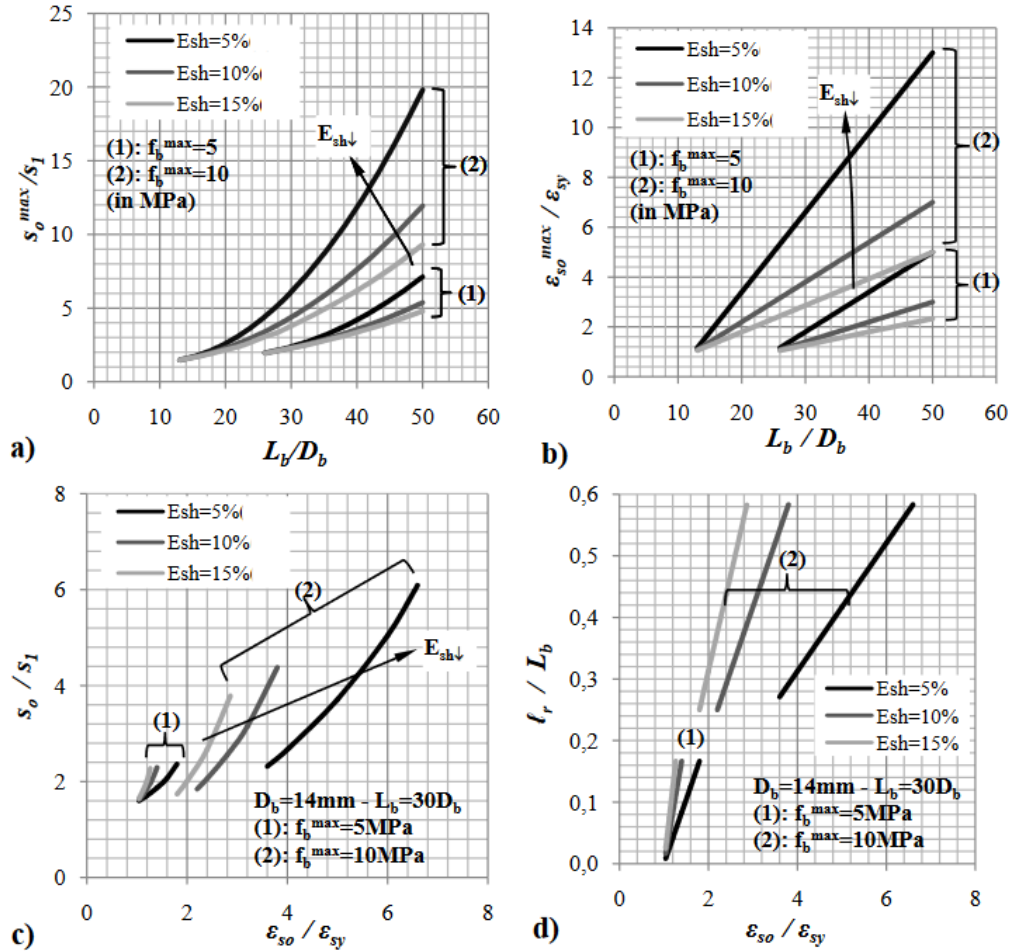


Figure 6.5. Charts for the definition of a) the slip at loaded-end of the bar anchorage and b) the corresponding strain capacity of the anchored bar versus the embedded length for several values of steel hardening modulus (E_{sh} : 5, 10, 15% E_s) and two values of bond strength ($f_{b,max}$ =5, 10MPa). For a specific bar anchorage geometry and given the imposed strain at the loaded-end c) the corresponding slip and d) the debonded length are calculated (charts obtained from Tastani and Pantazopoulou (2015)).

The shear capacity of the web of the RC columns was also estimated using the recommended equations by ACI 318 (shown in Eq. 6.2), which are also recommended by ASCE/SEI 41-17, by EN1998 Part-III (Eq. A.12 in original document), and the expression proposed by Pardalopoulos and Pantazopoulou (2019), as shown in Eq. 6.3. The detailed calculations are shown in tabulated form in Appendix C. Based on these equations, the column specimens with the large shear span ($L_v = 1450$ mm) showed web capacities of approximately 98 kN, 88 kN and 76 kN, according to ACI 318, EN1998-3, and Pardalopoulos and Pantazopoulou (2019), respectively. In the same fashion, the short shear span specimens ($L_v = 850$ mm) showed web capacities of 98 kN, 82 kN and 103 kN, respectively.

$$V_{v,ACI} = 0.66 \cdot \lambda_s \cdot \lambda \cdot \rho_w^{1/3} \cdot \sqrt{f'_c} + N_u / (6 \cdot A_g) \cdot b \cdot d + A_v \cdot f_y \cdot d / s \quad (6.2)$$

$$V_v = v \cdot b \cdot d \cdot f'_c \cdot \tan \alpha + A_{tr} \cdot f_{st} \cdot d \cdot (1 - 0.4 \cdot \xi) \cdot \cot \theta_v / s \quad (6.3)$$

Considering the presence of the lap-spliced connection, at the column-footing interface, calculation of the lateral capacity of the lap-spliced portion was performed. The set of equations were adapted from Pardalopoulos and Pantazopoulou (2019), as described in Eqs. 6.4-6.6. The detailed calculations and parameters are included, in tabulated form, in Appendix C. For the column specimens with the large shear span ($L_v = 1450\text{mm}$), the obtained lateral capacity of the lap-spliced connection was 77kN; the same property for the short span ($L_v = 850\text{mm}$) was 130kN.

$$V_{lap} = \left(C_{min} \cdot d \cdot (1 - 0.4 \cdot \xi) + v \cdot b \cdot d^2 \cdot f_c \cdot \left(0.5 \cdot \frac{h}{d} - 0.4 \cdot \xi \right) \right) / (H_{cl}/2) \quad (6.4)$$

$$C_{min} = \min \{ (C_{lap}); N_b \cdot A_b \cdot f_y \} \quad (6.5)$$

$$C_{lap} = \mu_{fr} \cdot L_{lap} [A_{tr} \cdot f_{st} / s + a_b \cdot (b - N_b \cdot D_b) \cdot f_t] \quad (6.6)$$

6.2 Preparation of Specimens and Lab facilities for Testing

For installation of the specimens on the Strong Wall-Strong Floor facility, two hollow PVC tubes had been used in the footing during casting to create the necessary openings. To fasten the specimens on the Strong Floor, two threaded steel rods were used, with steel nuts placed on the top footing surface and under the floor (Fig. 6.6). The steel nuts were placed to bear on separately-cast, UHPFRC plates, one on each side of the footing top face, which served to distribute the bearing forces throughout the footing surfaces. The UHPFRC-plates were ground to ensure leveling of their surfaces, using a high-power grinding tool.

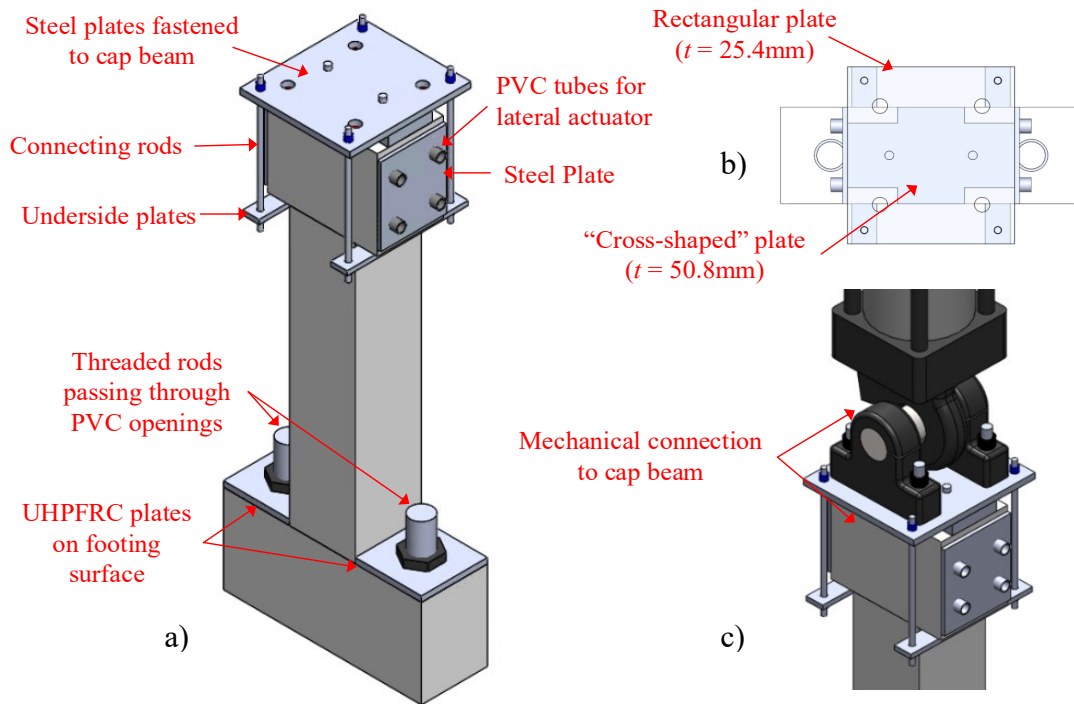


Figure 6.6. a) Overview of specimen design with connection details; b) Top view of steel plate connection; c) Attachment of axial actuator.

The axial and cyclic lateral loads were imposed by a set of two MTS[®] hydraulic actuators, one with 250kN and one with 1000kN capacity. The latter actuator was used for axial loading. Due to size restrictions, an external mechanical attachment was designed for the columns, to ensure fastening to the actuators. For the axial actuator, the mechanical attachment contained two bearing plates, one rectangular plate ($t = 25.4\text{mm}$) attached to the actuator head and one “cross-shaped” plate ($t = 50.8\text{mm}$), placed on the top of the cap beam (Fig. 6.6). The latter plate was designed to allow room for the fastening nuts on the underside of the former plate. For the lateral actuator, two steel plates were placed on each side of the cap beam and were fastened using threaded steel rods. The openings for the rods for the lateral actuator were included during casting of the column, using PVC tubes in the cap beam formwork. The complete mechanical attachment is shown in Fig. 6.6.

In addition, to obtain the strain development on the reinforcement bars within the spliced region, instrumentation was placed on both the longitudinal and transverse bars in that region. Twelve strain gauges were installed in columns containing 15M bars for longitudinal reinforcement, and eight for the columns containing 20M. To prepare the surface of the bars for placement of the strain gauges, a high-speed grinding tool was used, and the smoothened surface was cleaned to

remove debris. The strain gauges were placed using superglue and the connecting wires were separated to protect the gauge from short-circuits. Lastly, to prevent damage to the strain-gauges during casting, a mix of epoxy and sand was prepared and placed above each gauge at least one day prior to casting, to create a protective layer.

Casting of all specimens was completed in three separate batches. During each batch, two column specimens were cast, using commercial low-strength concrete, at a target strength of 25MPa so as to emulate old construction. Formwork was removed after two days, and the columns were kept under humid conditions for 28 days using wet burlap. During casting, a vibrating tool was used to ensure proper placement of the concrete in the formwork. Additional lateral supports were provided to the formwork, along with clamps placed on the top sides of the plywood walls. Plastic sheets were used to cover the top surface after casting. At demolding, the formwork was carefully removed, in order to be re-used for the subsequent casts. Figures to illustrate the above processes are included in Appendix C.

To prepare for testing, the lab facilities were adjusted, and the pertinent equipment was placed on the Strong Wall - Strong Floor (SW-SF) system of the High-Bay Lab. Initially, the blue mounting steel component was placed on the Strong Wall, on which the MTS actuator for the lateral cyclic loading was supported (Fig. 6.7). The mounting unit was moved using the crane mechanism of the lab and was fastened to the wall via four threaded steel rods. Upon fastening, the MTS actuator was also lifted using the crane and fastened to the middle of the mounting unit. The arrangement of the openings in the SW-SF system allowed for alignment of the actuator with the middle plane of the column specimens.

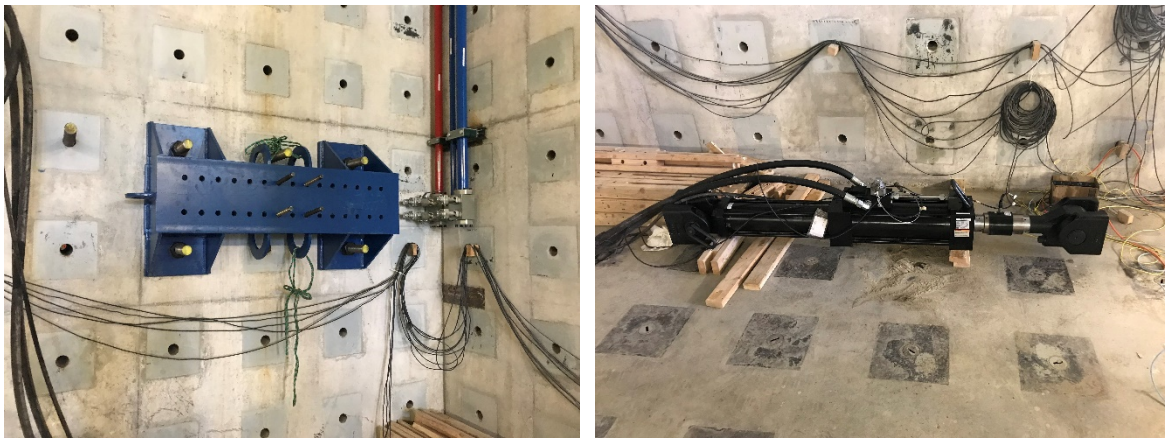


Figure 6.7. Placement of mount for lateral MTS actuator.



Figure 6.8. Placement of mount for vertical MTS actuator.

Secondly, the cantilever mounting unit for the axial actuator was placed on the wall segment parallel to the horizontal actuator, using the crane mechanism. To account for the height of the column specimens, the cantilever was fastened to the wall at an elevated position (Fig. 6.8), such that it enabled the maximum available space under the combined mounting unit and actuator system. The steel cantilever was attached to the wall via threaded steel rods; side flanges were also attached afterwards, which were provided by design, to eliminate sway on the cantilever.

Prior to lifting the axial actuator, the already fastened horizontal actuator was lifted to the elevated position shown in Fig. 6.9, to ensure adequate working space around the cantilever mounting area. The axial actuator was lifted in a vertical position using the crane mechanism and the metallic plate, which served as connection to the column cap beams and was attached to the bottom head, using four threaded rods. To enable proper grip on the actuator handles, a spreader beam was used (Fig. 6.9) on which two hooks were attached, on each side of the actuator. To attach the actuator on the cantilever mount, the Sky-Jack lift was used, with two researchers on top, to ensure proper fastening of the steel rods. During fastening, the crane was used to lift the actuator slightly, in order for the actuator surface to be aligned with the cantilever. After fastening, the crane was slowly lowered, and the bottom head of the actuator was rotated, such that the swivel movement was in the same direction as the lateral loading of the columns.

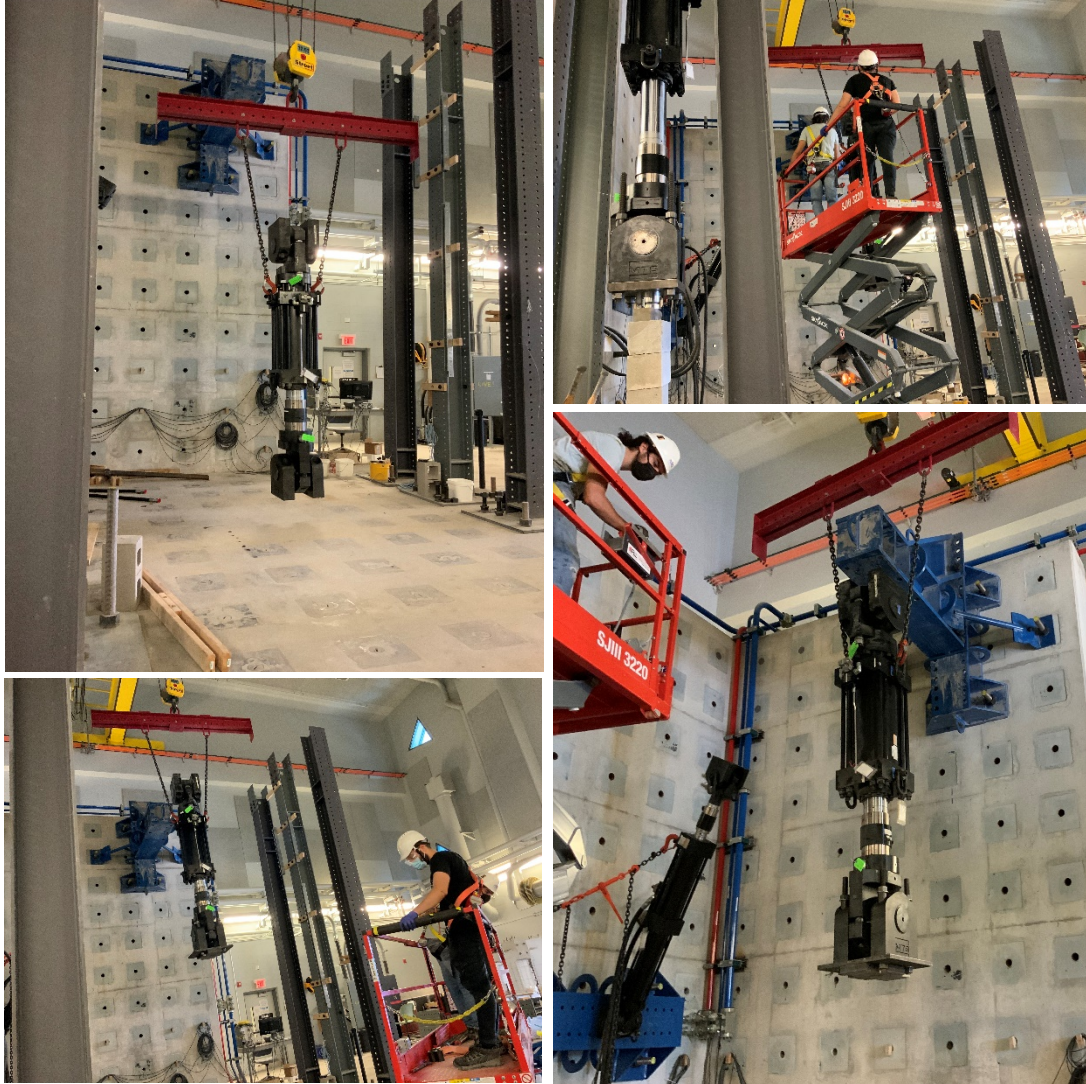


Figure 6.9. Placement of MTS axial actuator on the Strong Wall mount.

Prior to placement of the columns, it was necessary to test the functionality of the actuators, by extending them to approximately the full stroke, for a few times. This step was also beneficial in releasing entrapped air throughout the shaft of the actuator, which might lead to faulty readings during testing.

Prior to placement of the columns in the test zone, a layer of plaster was cast under each column, to ensure leveling of the footing surface. To cast the plaster, a custom rectangular wooden base was fabricated, which was placed under a plastic sheet (Fig. 6.10). Strain gauge cables were also covered for protection. The plastic sheet ensured no spillage of the plaster during casting. To protect the steel rod openings from being filled with plaster, wooden pieces were attached around

the openings and were glued under the footing prior to placement on the casting base. In addition, to ensure alignment of the column axis with the axial actuator, the columns were connected to an adjacent vertical steel column via straps, which were adjusted until leveling of the top surface was achieved. Plaster was poured from all sides of the column, so that the plaster was adequately distributed throughout the bottom surface. Once the layer was solidified, the columns were lifted with the crane system, and the plastic sheet was removed. The column specimens were then placed immediately on the test zone and under the axial actuator, with the plaster layer attached.



Figure 6.10. Placement of plaster layer under the column footings for leveling.

To place the columns in the test zone, the crane mechanism of the lab was utilized. The same spreader beam was used to enable placement of the columns in alignment with the actuator. Once the column was placed under the axial actuator, and aligned with the openings of the Strong Floor, the UHPFRC hollow plates were placed on the footing top surfaces, and the specimen was fastened using threaded steel rods, one on each side of the footing (Fig. 6.11). The steel nuts were tightened manually, using steel wrenches, until the footing was sufficiently fastened to the floor, to eliminate displacement at testing. At rest, the horizontal actuator was maintained in the elevated position, as shown in Fig. 6.11, and the axial actuator was supported with a wood column, when there were no specimens in the test zone.



Figure 6.11. Setup of column specimens with only the axial actuator (left) and with both actuators (right). After fastening of the base, the columns were attached to the double actuator system, as shown in Fig. 6.11. The horizontal actuator was extended, using the MTS proprietary software for control, and the threaded steel rods were fastened through the two steel plates, on each side of the cap beam. For the axial actuator, the bottom head was lowered and fastened to the cap beam, using the threaded rods and plate connection shown in Fig. 6.11.

External instrumentation was placed on the column hinge region and throughout the footing and cap beam of the specimens, as shown in Fig. 6.12. Three string potentiometers were placed on each side of the column, at successive levels of 150 mm, to obtain the curvature along the height of plastic hinge region. Two pairs of string potentiometers were also placed diagonally on the front face of the column, at levels of 150 mm apart, height-wise. To account for potential uplift of the footing, two linear potentiometers were placed vertically on the front face of the footing, one on each axis of the supporting rods. Additionally, to measure the potential slip of the footing in the lateral direction, two linear potentiometers were placed on the sides of the footing and were connected to the strong floor. On the back face of the column, and along the central axis, one string potentiometer was placed vertically, from which the axial elongation or shortening of the column was measured. The lateral displacement of the cap beam during testing was measured with a pair of string potentiometers, attached laterally to the middle of the side surface, and fixed on a

stationary steel column. An additional potentiometer was placed on the middle of the clear height and extended laterally up to the steel element.

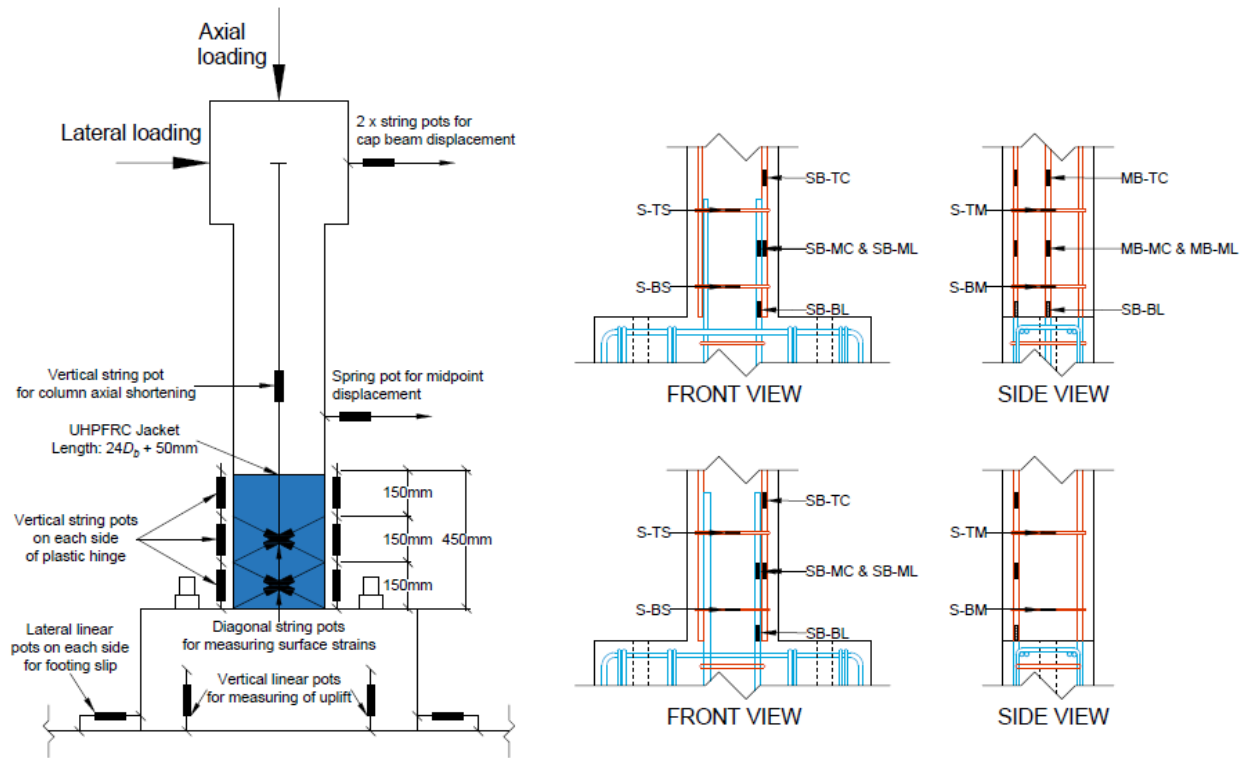


Figure 6.12. Schematic of instrumentation placement in any specimen along with name IDs for strain gauges.

All connections of the external instrumentation were ensured with hot glue. After placement of the instrumentation, the front face of the column was as shown in Fig. 6.13. The potentiometers were connected to one of three DAQ systems, also shown in Fig. 6.13, with conventional connecting wires.

Prior to testing, the area around the columns was cleaned and the instrumentation calibration secured. To record the experiments, a digital single-lens reflex camera (DSLR) and video recording camera were used, facing the front face of the column specimens. The former was set to capture pictures at time intervals of 10 sec, using an external attachment and controller on the camera, as shown in Fig. 6.14. To ensure adequate lighting, the front face of the columns was brightened using portable light projectors, also shown in Fig. 6.14.

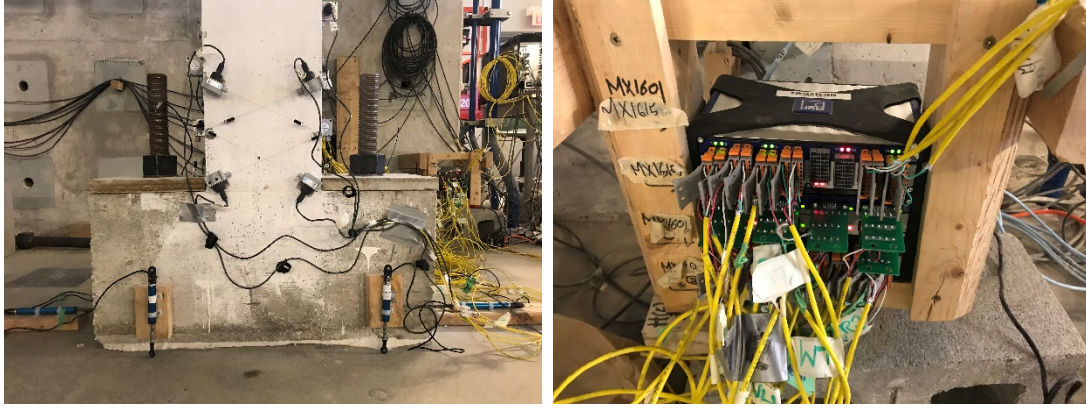


Figure 6.13. Attachment of instrumentation on column specimens, as shown in Fig. 6.12.



Figure 6.14. Complete setup of column specimens with actuators and instrumentation attached.

After testing, the actuators were reverted to zero load position and the mechanical attachments were released. The instrumentation was disconnected from the columns and debris around the column due to delamination of the cover was cleaned. To relocate the column specimens to their original position, the spreader beam was connected to the crane mechanism of the lab and the columns were lifted from the cap beam. The spreader beam was connected to two steel chains, that were extended with straps, and were used to transfer the columns, in the arrangement shown in Fig. 6.15. The columns were subsequently prepared for repair, the procedure of which is discussed in the next chapter.



Figure 6.15. Column specimen after testing (left); Relocation of damaged columns to repair area (right).



Figure 6.16. Complete modified setup for short column specimens (left); Attached formwork to improve leveling of side surface (right).

The same process was performed for both the tall and short column specimens. For the latter, the horizontal actuator and mounting unit were moved to a lower elevation, to be aligned with the cap

beam axes (Fig. 6.16). Since the axial actuator was not long enough to reach the shorter specimens, a steel plate was also placed on the cap beam to provide adequate height to the specimen.

In specimen CS3-20M, where one of the side surfaces of the cap beam was not leveled, an additional layer of plaster was cast, so that the horizontal actuator surface was evenly connected. To cast the vertical layer, a custom formwork was fabricated, with the top surface exposed, and was attached to the cap beam via mechanical clamps and tightened with rope (Fig. 6.16). Once the plaster was solidified, the formwork was removed and the steel connecting plate for the actuator was placed on top of the layer.

6.3 Experimental Parameters

Prior to each test, the compressive strength of concrete was confirmed by testing sets of three 100 mm x 200 mm cylinder specimens, on the Pilot Controls testing frame (Fig. 6.17). The results of each cylindrical tests are listed in Table 6.2, in tabulated form along with the average compressive strength for each case.



Figure 6.17. Compression testing of cylinders (left); Samples of cylinder specimens for testing (right).

Table 6.2. Cylinder compressive strength for normal concrete in column specimens.

Cylinder ID	f_c' (MPa)	f_c' (MPa) - Average
C1-1	20.4	20.4
C1-2	20.2	
C1-3	20.6	
C2-1	25.77	24.35
C2-2	24.33	

C2-3	22.94	27.24
CS3-1	28.11	
CS3-2	25.7	
CS3-3	27.91	

For the cyclic loading, a displacement-controlled scheme was programmed into the MTS proprietary software, with cycles repeated at pre-selected lateral displacement increments defined in terms of drift ratio, with three cycles performed at each increment; exception of specimen CT1-15M which was tested using two cycles per increment. The maximum drift level of the first set of cycles was set at 0.25%, the drift amplitude of the next set of three cycles was set at 0.5% and was increased by 0.5% for each set of three cycles after that (Fig. 6.18). This scheme was selected based on ACI 307 (2008), with the aim of simulating the softening effects of repeated seismic loading on reinforced concrete structures. For specimen CT1-15M the two-cycle loading scheme was implemented, to ensure the safety of the loading frame. The targeted drift levels and corresponding displacements are shown in Table 6.3, in tabulated form. The axial load was imposed at a fixed value of $0.15f_c A_g$.

Of the four specimens tested in this experimental phase (intended to cause initial damage to the columns), two were tall columns, having a shear span $L_v = 1450$ mm, and two were short columns with $L_v = 650$ mm. The tests were conducted until loss of lateral strength was evident, and extensive damage had developed on the specimens. Damage was manifested as “flattening” on the force-displacement curves at a reduced load level as compared with the strength, with the increase of lateral displacement, indicating a frictional dragging along the tension reinforcement. Overall, both loading directions exhibited similar behavior and symmetry, however, with occasionally the pushing side (positive) showing greater resistance (load) than the pulling side (negative). This could be attributed to initial damage in the compression side of the lap-splices during the previous cycle which might have rendered the bond more compliant and weaker for the reversed load.

Table 6.3. Drift ratios for tall and short column specimens and equivalent lateral translation.

Drift Ratios (%)	<i>Top Displacement (mm): Tall Columns - ($L_v = 1450$ mm)</i>	<i>Top Displacement: Short Columns - ($L_v = 850$ mm)</i>
0.25	3.625	2.125
0.5	7.25	4.25
1.0	14.5	8.5
1.5	21.75	12.75

2.0	29	17
2.5	36.25	-
3.0	43.5	-

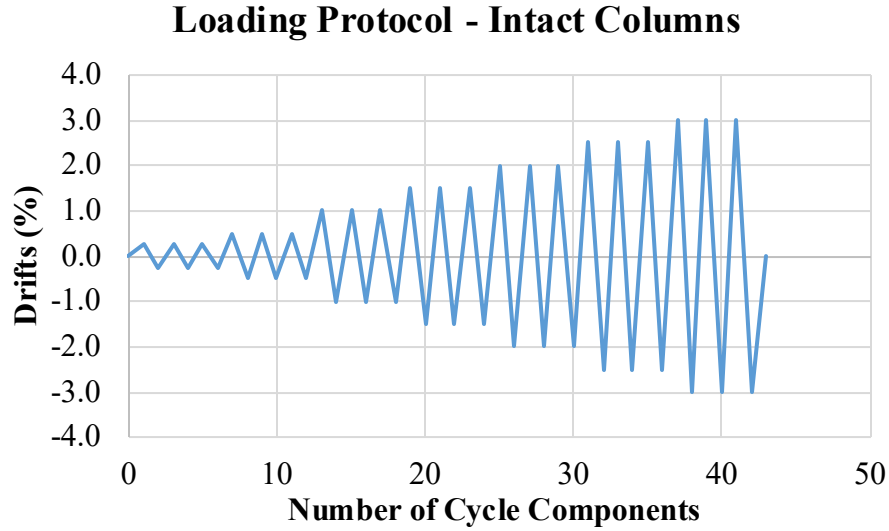


Figure 6.18. Loading protocol adopted for intact column specimens.

6.4 Results

6.4.1 Force-Displacement Response Curves

The complete force-displacement responses of the tall ($L_v = 1450\text{mm}$) and short ($L_v = 850\text{mm}$) column specimens are shown in Figs. 6.19 and 6.20, respectively. Specimen CT1-15M was tested until a maximum drift level of 2.0%, where reduction in the maximum lateral load capacity was observed, whereas specimen CT2-20M was tested to 3.0% drift, which was pushed to the stage of significant loss of strength. Both short column specimens, CS3-15M and CS3-20M, were tested until the 2.0% drift level, and the tests were terminated to avoid collapse of the specimens. Pictures of the observed cracking during the tests are shown in Figs. 6.21-6.25.

The results shown in Figs. 6.19 and 6.20 were obtained from the force applied by the horizontal actuator and the measured lateral displacement of the cap beam, using the string potentiometer setup, as shown in Fig. 6.12. To obtain the lateral resistance of the column, V_{res} , the lateral actuator force was adjusted to account for the contribution of the axial actuator, due to deflection from the column axis, as in Eq. 6.7:

$$V_{res} = H_{piston} + P \cdot \theta \cdot \cos(\theta_{act}) + P \cdot \sin(\theta_{act}) \quad (6.7)$$

In Eq. 6.7, H_{piston} is the force measured by the horizontal actuator, P is the axial load of the vertical actuator, θ is the rotation at the base of the column and θ_{act} is the rotation of the actuator from the top connection to the tip of the column cantilever.

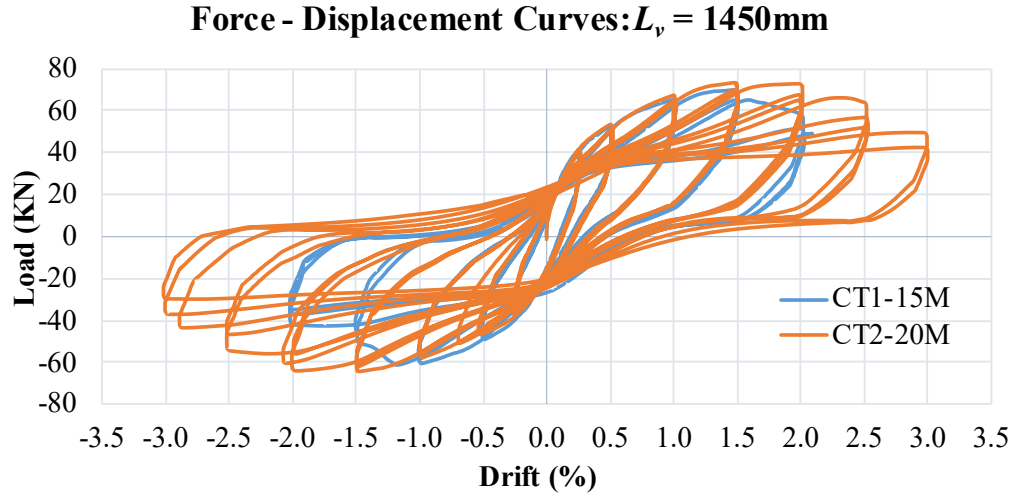


Figure 6.19. Force – Displacement curves for CT1-15M and CT2-20M specimens.

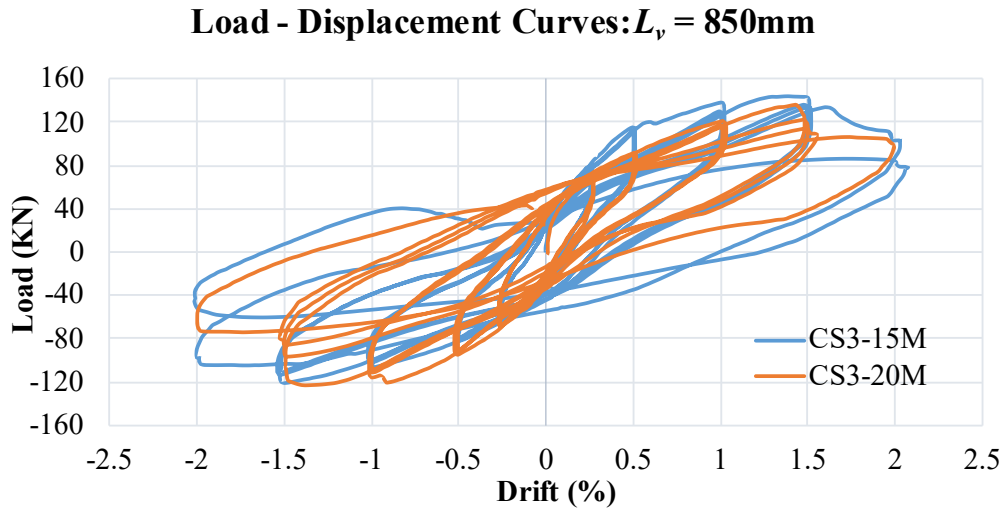


Figure 6.20. Force – Displacement curves for CS3-15M and CS3-20M specimens.

Specimen CT1-15M reached a maximum lateral load of 70kN, at the drift level of 1.5% and the test was terminated at the second cycle at 2.0% drift, with a residual lateral load of 34kN (~50% of strength). Until reaching 1.0% drift, the loss of strength between successive cycles at the same

drift level was not substantial (less than $\sim 5\text{kN}$), while the strength dropped significantly between the drift ratios from 1.5% to 2%. The negative side showed lower strength than the positive.

Throughout the test, lateral cracks appeared on the surface of the column with increasing drift magnitudes. The first horizontal cracks appeared close to the location of the stirrups and the crack spacing was reduced as the displacement increased. Inclined cracks appeared as well, indicating the increased presence of shear, although they appeared at higher displacements of the column. Severe cracking was observed at the base of the column, where the connection to the footing showed the most damage. After reaching a drift of 2%, vertical cracks in alignment with the longitudinal bars were observed on the sides of the column, within the lap-spliced zone. The latter observation indicates the exhaustion of the deformation capacity of the lap-spliced connection and increased strain on the longitudinal reinforcement.

Specimen CT2-20M reached a maximum lateral load of 73.5kN, at the drift level of 1.5%, similarly to specimen CT1-15M. The test was initially terminated at the first cycle of 2.0% drift, with a lateral load of 62kN (83% of capacity), as the column showed increased signs of crack propagation and loss of integrity (Fig. 6.22). However, the test was continued, after removal of the external instrumentation in the plastic hinge zone, until the strength was reduced to less than 1/3 of the maximum load. The column was tested until a drift ratio of 3%, at which stage severe cracking and loss of strength occurred (at 3.0% drift the lateral resistance was reduced to less than 1/3 of the strength) and the experiment was terminated (Fig. 6.23).

Similar to specimen CT1-15M, flexural cracks (lateral orientation) appeared throughout the column, particularly in the plastic hinge zone, with increasing lateral displacements. The spacing of the cracks was slightly larger at drift levels below 1.5%, compared to CT1-15M, however, the density of cracking increased substantially at higher drift ratios. Failure localized at the column-footing connection, where large cracks developed at the corner of the column base, indicating the penetration of strain within the bottom lap-spliced bars. In this specimen, the inclined cracks within the column were more pronounced than in the previous case.

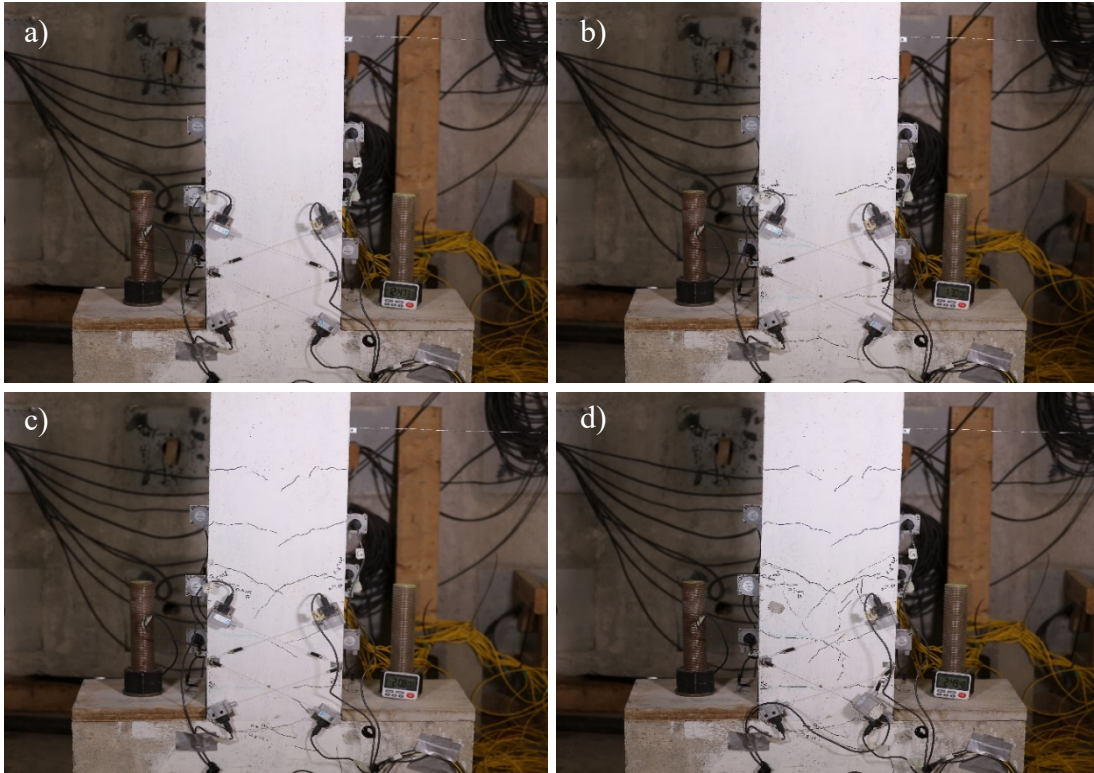


Figure 6.21. CT1-15M:- a) Pristine Condition; b) 1%; c) 1.5%; d) 2%.

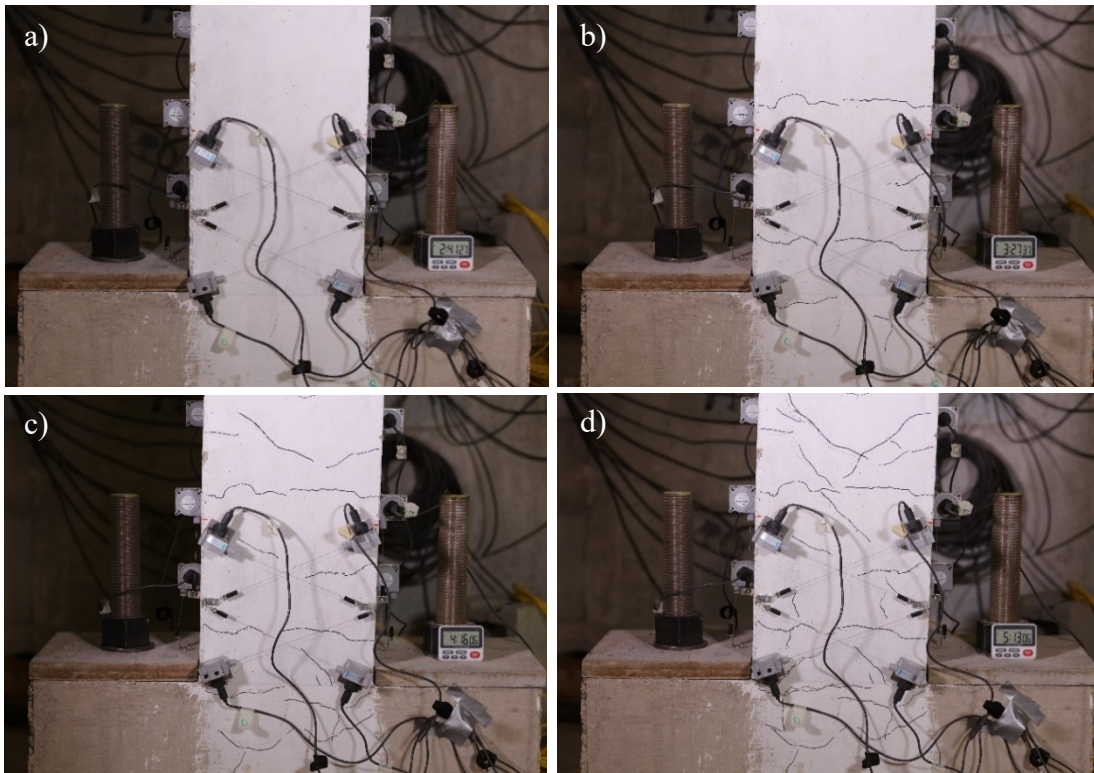


Figure 6.22. CT2-20M: a) Pristine Condition ; b) 1%; c) 1.5%; d) 2%.

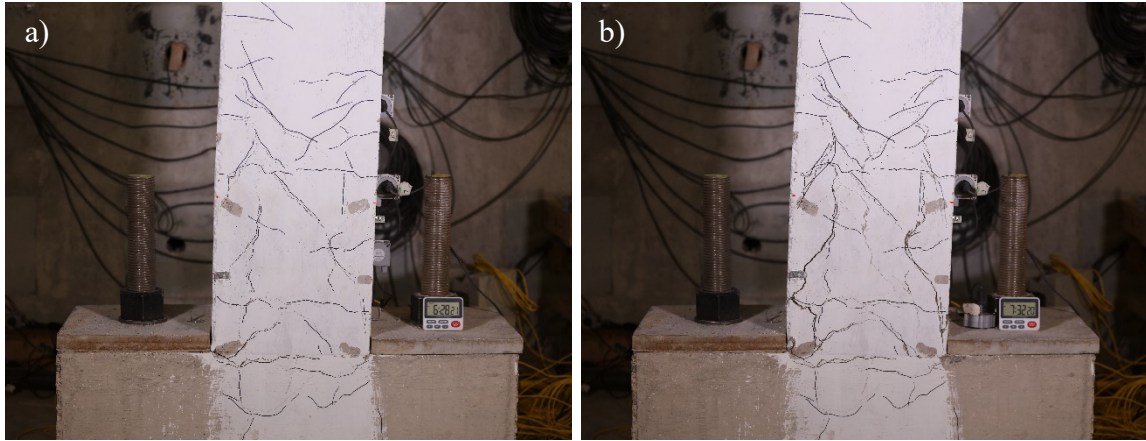


Figure 6.23. CT2-20M: a) 2.5%; b) 3%.

Of the short span specimens, the maximum lateral loads attained were 132.2kN and 114.8kN, for specimens CS3-15M and CS3-20M, respectively. Despite the reduced deformation capacity, the tests were continued to 2% drift ratio, with specimen CS3-15M reaching up to 33.1kN (25% of capacity), at the maximum lateral displacement, and specimen CS3-20M reaching 51.8kN (45% of capacity), similarly, during the first cycle at that level of drift.

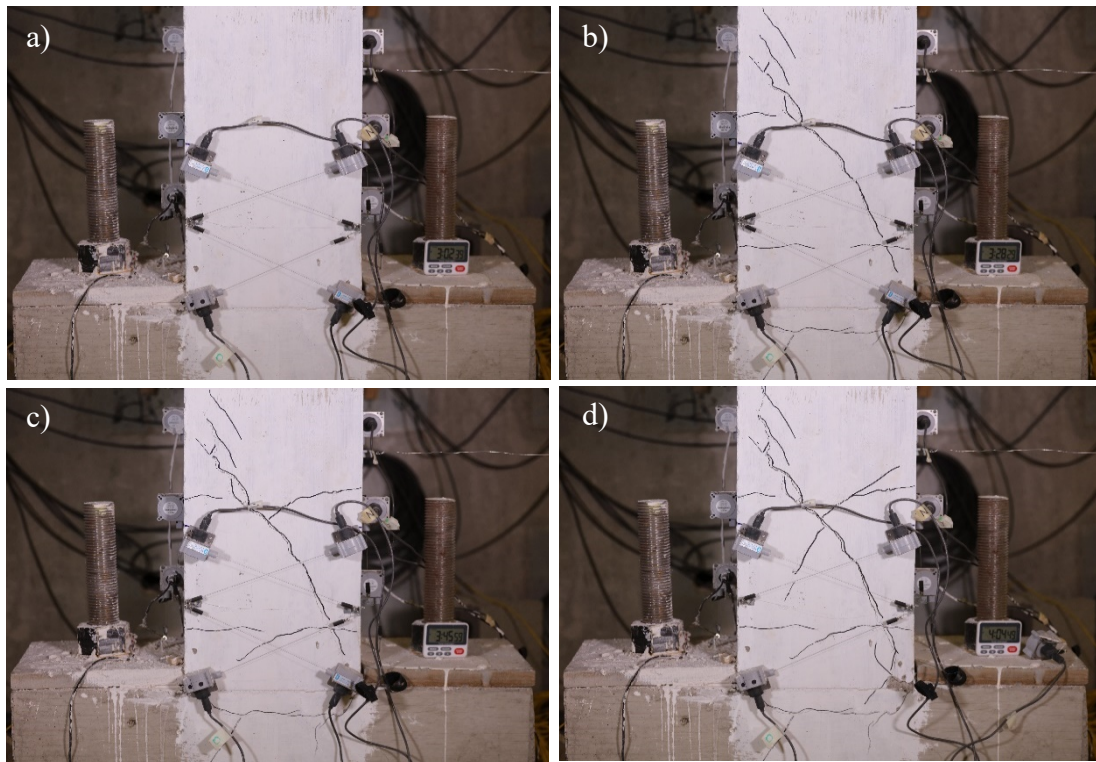


Figure 6.24. CS3-15M: a) Pristine Condition ; b) 1%; c) 1.5%; d) 2%.

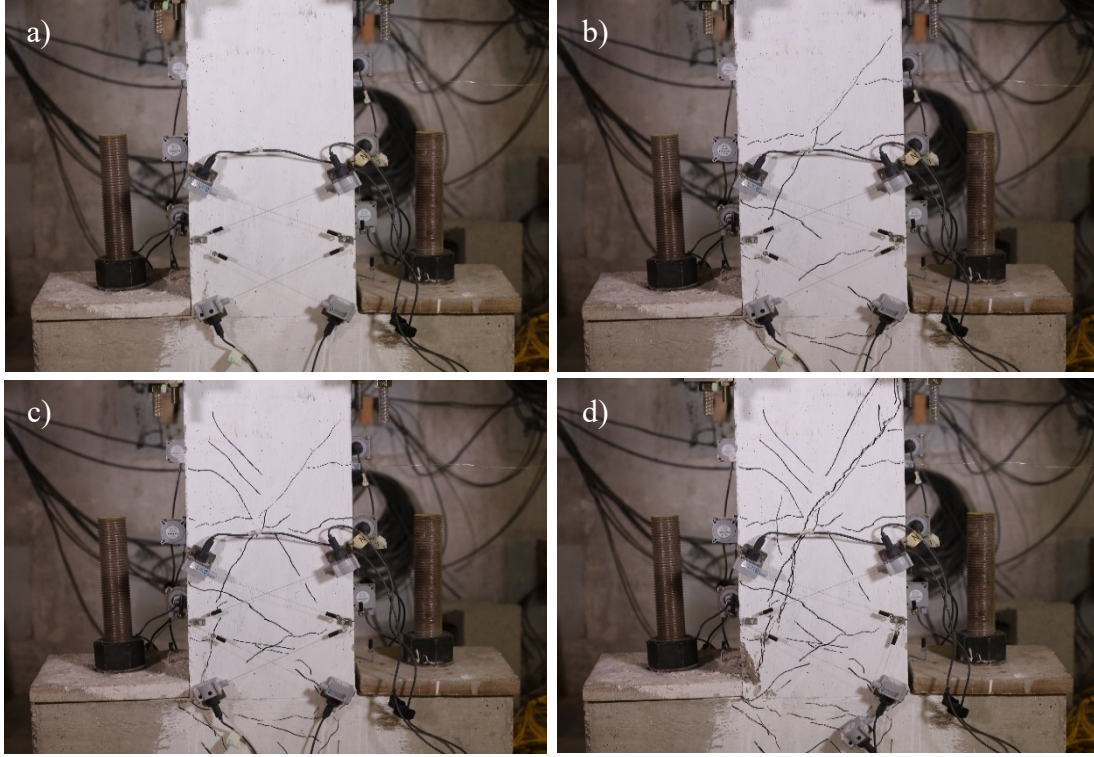


Figure 6.25. CS3-20M: a) Pristine Condition ; b) 1%; c) 1.5%; d) 2%.

At drift levels lower than 1%, minor flexural cracks were observed on the sides of each specimen, while, at peak load a larger shear crack appeared on the front surface, on both specimens. Failure localized on the shear crack, during the cycles exceeding 1% drift, with multiple damage states appearing on the specimens such as cover delamination, crushing of the concrete at the column base, pullout failure of the column-footing interface, and longitudinal cracking along the lap splice bars. Closer to the termination of the test, the shear crack width increased almost to the point of complete separation of the surfaces. At this stage, both the cover and core concrete were significantly damaged.

6.4.2 Curvature Distribution

The external instrumentation shown in Fig. 6.12 was used to obtain the rotation and curvature, at the base of the specimen. The rotation was calculated as in Eq. 6.8, where δ_t and δ_c correspond to the displacements measured from the string potentiometers, on the tension and compression side of the column, respectively, l_c is the vertical gauge length, and h_c is the horizontal center-to-center distance between the measurements.

$$\varphi = (\delta_t - \delta_c) / (l_c \cdot h_c) \quad (6.8)$$

Based on the arrangement of Fig. 6.12, the curvature was obtained on three regions along the column height: at 0-150mm, 150-300mm and 300-450mm. The curvature distribution diagrams are also shown in Figs. 6.26-6.27. The magnitudes of the curvature along the column height for the drift ratios of $\pm 0.5\%$ and $\pm 2\%$, for each of the intact specimens. The curvatures of the short columns are not shown due to inconsistent data. The dashed lines correspond to specimens with the 15M longitudinal reinforcement and the continuous line to the 20M case.

For the tall columns (Fig. 6.26) it was observed that the curvatures of specimen CT1-15M, were of lower magnitude than the CT2-20M case. In both cases both the negative and positive sides of loading showed an almost symmetric response. The maximum curvature was obtained at the lower segment of column height (0m – 150m), with 0.059 1/m and 0.089 1/m for columns CT1-15M and CT2-20M, respectively. The distribution of curvature, for both cases, captures the response observed in the experiment, where the curvatures of the larger magnitude were located within the plastic hinge region.

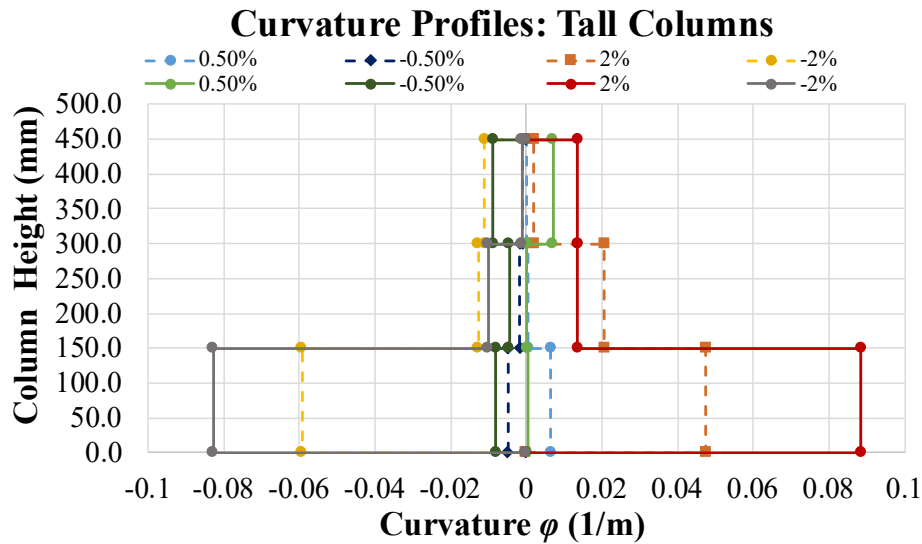


Figure 6.26. Curvature – Time Step history for specimen CS3-15M.

6.4.3 Strain Gauge Measurements

Strains on the embedded reinforcement were obtained according to the arrangement shown in Fig. 6.12. The strains at peak load are shown in this chapter, in Tables 6.4 and 6.5, for strain gauges (SGs) placed on the longitudinal and transverse reinforcement, respectively. In addition, diagrams of strain versus time step diagrams are shown in Fig. 6.27.

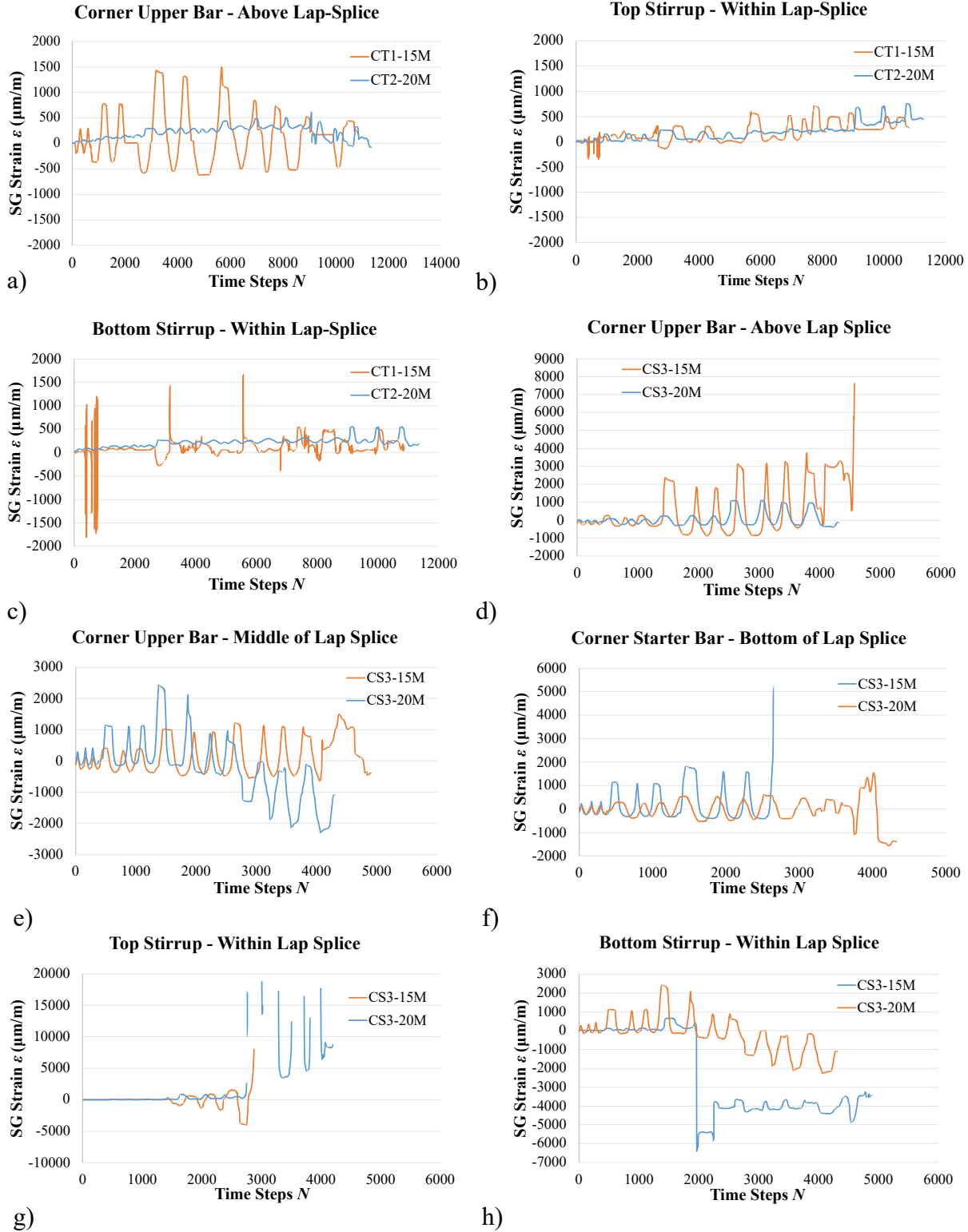


Figure 6.27. Strain – Time Step history for various locations in column reinforcement: a) Corner Upper Bar – Above Lap Splice; b) Top Stirrup – Within Lap Splice; c) Bottom Stirrup – Within Lap Splice; d) Corner Upper Bar – Above Lap Splice; e) Corner Upper Bar – Middle of Lap Splice; f) Corner Starter

Bar – Bottom of Lap Splice; g) Top Stirrup – Within Lap Splice (front face of column); h) Bottom Stirrup – Within Lap Splice (front face of column).

Based on the strain measurements on the longitudinal reinforcement, there was indication of yielding of the bars, with strains approaching the value of 2200 $\mu\epsilon$. In particular, locations MB-TC and MB-MC on specimen CT1-15M reached 2000 $\mu\epsilon$ and 1860 $\mu\epsilon$, respectively, while on specimen CT2-20M, location SB-MC reached 2326 $\mu\epsilon$. Strains on the shorter specimens were higher, with specimens CS3-15M and CS3-20M reaching values of 3249 $\mu\epsilon$ and 3277 $\mu\epsilon$, respectively.

Table 6.4. Strain gauge measurements at peak load on longitudinal reinforcement on various locations.

Column ID	MB-TC ($\mu\epsilon$)	MB-MC ($\mu\epsilon$)	SB-TC ($\mu\epsilon$)	SB-MC ($\mu\epsilon$)	MB-ML ($\mu\epsilon$)	MB-BL ($\mu\epsilon$)	SB-ML ($\mu\epsilon$)	SB-BL ($\mu\epsilon$)
CT1-15M	-2001	-1860	-630	0	-16703	-516	0	0
CT2-20M	0	0	434	681	0	0	1291	2326
CS3-15M	1903	1380	3249	1245	1467	1455	963	0
CS3-20M	0	0	1118	579	0	0	3277	658

***MB-TC**: Intermediate Upper Bar – Above Lap Splice; **MB-MC**: Intermediate Upper Bar – Middle of Lap Splice; **SB-TC**: Corner Upper Bar – Above Lap Splice; **SB-MC**: Corner Upper Bar – Middle of Lap Splice; **MB-ML**: Intermediate Starter Bar: Middle of Lap Splice; **MB-BL**: Intermediate Starter Bar: Bottom of Lap Splice; **SB-ML**: Corner Starter Bar – Middle of Lap Splice; **SB-BL**: Corner Starter Bar – Bottom of Lap Splice.

Table 6.5. Strain gauge measurements at peak load on transverse reinforcement.

Column ID	S-TM ($\mu\epsilon$)	S-TS ($\mu\epsilon$)	S-BM ($\mu\epsilon$)	S-BS ($\mu\epsilon$)
CT1-15M	-271	16	32	566
CT2-20M	0	198	209	256
CS3-15M	4260	-3813	412	-3657
CS3-20M	266	516	363	652

***S-TM**: Top Stirrup – Side Face of Column; **S-TS**: Top Stirrup – Front Face of Column; **S-BM**: Bottom Stirrup – Side Face of Column; **S-BS**: Bottom Stirrup – Front Face of Column.

Strains measured on the stirrups were markedly lower for the taller specimens, with the maximum values reaching up to 566 $\mu\epsilon$ and 256 $\mu\epsilon$, for specimens CT1-15M and CT2-20M, respectively.

For the short specimens, higher values were obtained for specimen CS3-15M, with strains reaching a maximum of 4260 $\mu\epsilon$. Specimen CS3-20M showed lower strains, at values of 652 $\mu\epsilon$.

6.4.4 Shear Strains and Drift

Based on the configuration of diagonal string potentiometers, the surface shear strains were calculated by averaging the displacement measurements over the inclined distance, as shown by Eq. 6.9:

$$\gamma_{shear} = (\delta_{left} - \delta_{right})/l_{incline} \quad (6.9)$$

In the above equation, ϵ_{shear} is the shear strain, δ_{left} and δ_{right} are the displacement measurements of the intersecting inclined potentiometers and $l_{incline}$ is the hypotenuse defined by the center-to-center horizontal distances between the potentiometers and the vertical gauge length, as it is shown in Fig. 6.12. The calculations are performed for the two sets of intersecting potentiometers, placed at two levels, with the first at 300mm from the footing surface and the second at 150mm, so that the second level is closer to the critical section, also shown in Fig. 6.12. For each level, the shear drift was calculated, as it is shown in Eq. 6.10.

$$\theta_{shear} = \gamma_{shear} \cdot l_c / (L_v) \quad (6.10)$$

Where θ_{shear} is the shear drift, l_c is the vertical gauge length, and L_v the shear span. The total shear drift was calculated by the summation of the corresponding values of the two levels of potentiometers, and the diagram of total shear drift versus drift ratio is shown in Fig. 6.28.

Compared to the tall columns, it is evident that the short columns resulted in significantly larger shear drift ratios. The CS3-15M and CS3-20M specimens exceeded the CT1-15M and CT2-20M cases by approximately 5.6 and 17 times, respectively. The maximum values of the short columns were 0.054% and 0.072%, for the cases of CS3-15M and CS3-20M, respectively.

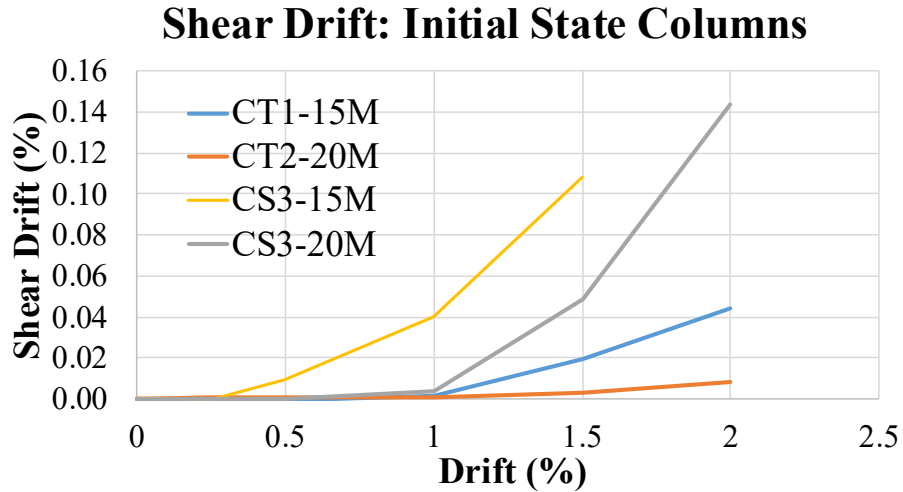


Figure 6.28. Shear drifts versus drift ratios for intact columns.

6.4.5 Axial Elongation of the Columns

The axial displacement was measured with the vertical string potentiometer that was placed on the cap beam and along the column axis, as was shown in Fig. 6.12. The elongation is plotted against the applied lateral drift ratio in Fig. 6.29.

For all specimens, the displacement was negligible at very low drift ratios of 0.25%. Of the taller columns, specimen CT1-15M showed compressive axial displacement (shortening) up to 0.5% drift, with the maximum value being 0.12mm, while, at the same drift magnitude, specimen CT2-20M showed 0.45mm. In the former case, for drift ratios exceeding the magnitude of 0.5%, displacement in tension (elongation) was also observed, on both directions of loading, and reached values of 1.5mm at 2% drift. Specimen CT2-20M showed only shortening throughout the test, with the maximum attained value of 0.52mm obtained at 1%, whereas, for higher magnitudes of drift the compressive axial displacement was reduced to 0.24mm.

Specimens CS3-15M and CS3-20M showed comparatively low compressive axial displacement, with values reaching 0.26mm and 0.83mm, at 1.5% drift. Prior to failure and the termination of the experiment, the compressive displacement reached 6.07mm and 2.86mm for the former and latter cases, respectively.

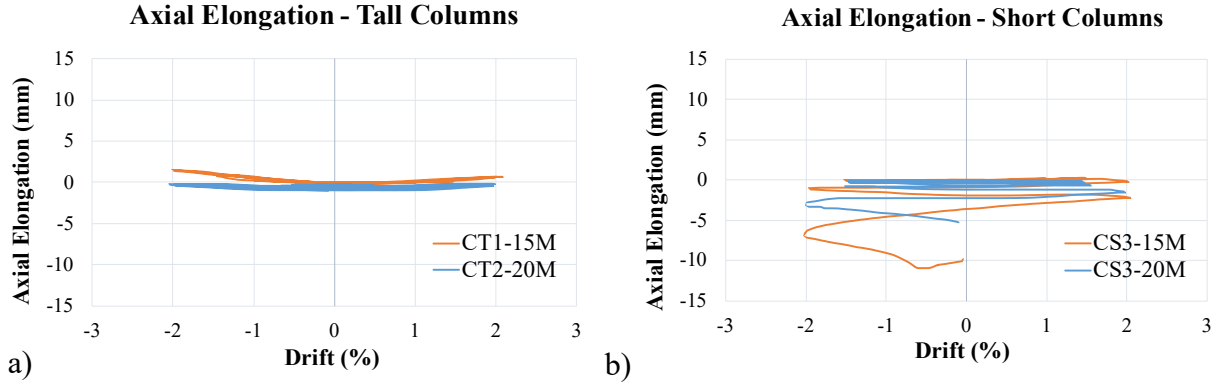


Figure 6.29. Axial elongation versus drift ratio for: a) Tall Columns; b) Short Columns.

6.5 Summary

In the structural-scale experimental program of this chapter, a total of four NSC column specimens were tested in combined constant axial and cyclic lateral loading. Tests were conducted in the Strong Wall – Strong Floor structural facility, using two high-capacity hydraulic actuators. The specimens comprised two columns with a clear height of $L_v = 1450\text{mm}$ (designated at “flexural” columns) and two with $L_v = 650\text{mm}$ (designated as “shear” columns), with each specimen in both cases being detailed with 6 x 15M bars and 4 x 20M bars, lap spliced above the footing over a lap length of $24D_b$. Transverse reinforcement was sparse, with 8mm smooth wires at 250mm spacing, to represent older seismic design procedures. Loading was applied using displacement control, with two and three cycles performed at preselected drift increments. The outcomes of this study are aimed to be utilized in correlation with cyclic tests on the same specimens, after being retrofitted with UHPFRC jackets, which are described in the following chapter. Among the observations of the above tests are the following:

1. Column specimens with conventional concrete of $f'_c = 25\text{MPa}$ and mild reinforcement detailing reached their strength at lateral drift ratios of 1.5%. Loss of strength was rapid in the post-peak branch, reducing by approximately 1/3 of the load capacity in the first cycle to the next drift increment. Tests were terminated at 2% drift, to avoid brittle failures of the specimens. Flexural columns showed an average of 71.5kN, which was half the average peak load of the shear columns, at 125kN. Despite the difference in clear span, all specimens performed until the 2% drift milestone.

2. Failure in all specimens appeared on the column-footing interface, with a horizontal crack localized at the base of the columns. Multiple flexural cracks appeared throughout the column span, with more dense propagation on the flexural specimens. Shear columns showed both flexural and shear cracks, with the latter appearing at higher drift ratios and governing the failure of the specimens. In addition, vertical cracks along the lap-splice axes appeared on the sides of the columns, at drift ratios exceeding 1.5%, indicating failure of the spliced connection. Lastly, crushing of the concrete was observed closer to the failure of the specimens and severe cover spalling appeared prior to the termination of the experiments.
3. Based on the curvature measurements, in the flexural specimens, the values were mostly symmetric in the positive and negative directions of loading, for drift ratios up to 2%. Curvature distributions in both flexural columns were similar, with larger magnitudes of measured curvature closer to the plastic hinge region. In the short columns, curvatures were of lower magnitudes, compared to the tall columns, and the curvature distributions more pronounced the upper segments throughout the column height.
4. Input from strain gauges placed on the reinforcement shows that longitudinal bars reached yielding at the peak load, in all specimens.
5. Shear drift ratios on the surface of the columns showed the maximum values for the short columns, which significantly exceeded the shear drift ratios of the tall columns.
6. All specimens showed shortening behavior in the first stages of the experiment, as shown by the vertical string potentiometer, with the flexural specimens reverting to elongation for increasing drift ratios. Specimen CT1-15M was subjected to tension upon reaching 1% drift, while specimen CT2-20M remained in compression until the end of the test. Both shear columns remained in compression and at 2% drift sudden reductions of the column length were observed, reaching -6.07mm and -2.86mm for columns CS3-15M and CS3-20M, respectively.
7. Overall, the specimens were tested until severe damage and complete reduction of strength occurred. Failures were noted on the column-footing interface, with simultaneous extensive cracking on the columns. At the end of the experiments all columns were inoperable, with the damaged concrete reaching up to the cores of some of the specimens.

The next step for the specimens, which is discussed in the next chapter, is the repair with UHPFRC jackets, after removal and cleaning of the damaged parts.

7 Testing of UHPFRC-Retrofitted Columns

This chapter describes the third stage of the structural-component experimental program, consisting of testing the column specimens of the previous chapter, after these were retrofitted with thin UHPFRC jackets in the plastic hinge region. Repair of the damaged specimens, as well as retrofitting of the previously untested specimens was performed using the prepackaged cementitious material Densit[®] which had been used also in the previous experimental components of this thesis. Prior to application of the jackets, the concrete surrounding the plastic hinge zone was mechanically removed, including both the previously tested and untested specimens, and casting was performed in separate batches. All specimens were tested in the same combination of constant axial and cyclic lateral load. Load cycles were performed at higher drift ratios than the control specimens, to obtain insights and practical evidence on the improvement of performance imparted by the UHPFRC jackets. External and internal instrumentation were used to measure the displacements and deformation of the column components. The performance of the retrofitting method with thin jackets to avoid alteration of the geometric details of the structural components and the required commensurate details in order to control the localization of failure are discussed in detail in the following sections.

7.1 Preparation of NSC Columns for Retrofit

The design schematics and technical characteristics of the column specimens in their pristine condition were described in Chapter 6 of this thesis. Figures 7.1-7.3 describe the updated column geometry details, after placement of the UHPFRC jackets. A characteristic of this retrofitting approach is casting of very thin jackets – for the size of columns of the study this corresponds to a thickness equivalent to the existing cover. To avoid alteration of the component geometry, the retrofitting procedure aims at cover replacement with UHPFRC material in the critical region of the member (around the plastic hinge region), up to the extent of the lap-spliced bar connection. Where prior damage had led to disintegration of the core concrete, then this was also replaced with UHPFRC material. To discourage formation of a single crack at the critical section, as is often seen in columns jacketed by other systems (e.g. FRP wraps), the jacket was extended to the upper

part of the footing, reaching down to a depth of up to $2 \cdot D_b$ and $5 \cdot D_b$, in the lateral and axial directions, respectively.

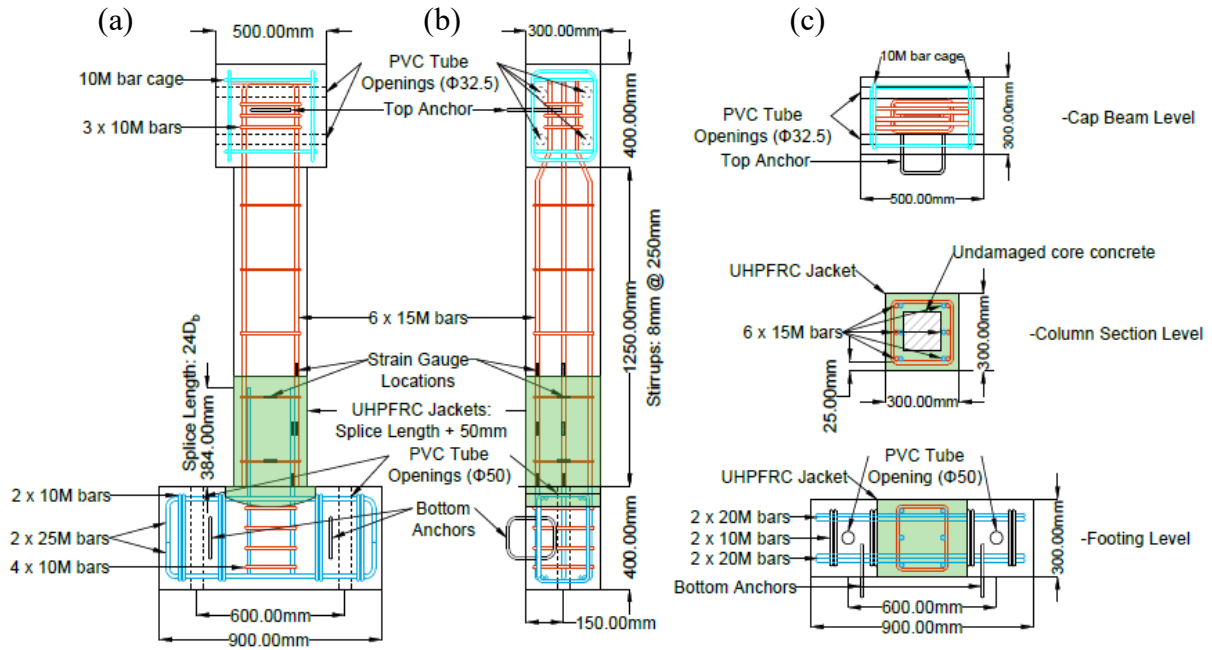


Figure 7.1. Geometry and reinforcement details of CT1-15M-R/S. (a) plane of action; (b) transverse to the plane of action; (c) sectional arrangement.

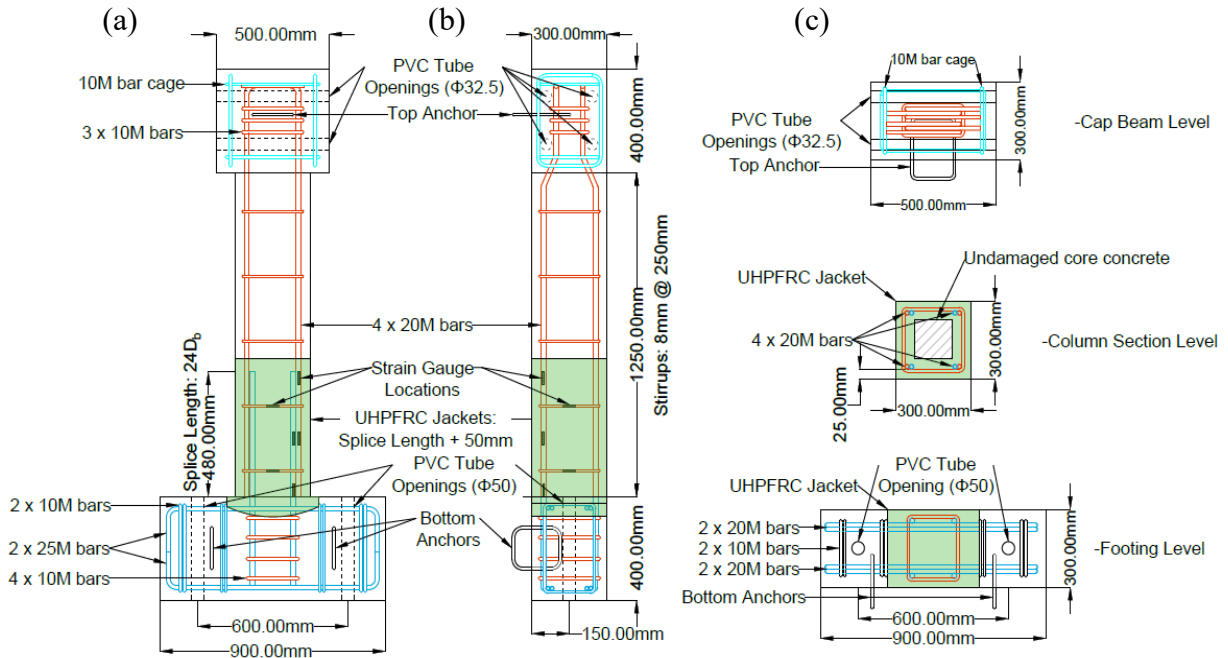


Figure 7.2. S Geometry and reinforcement details of CT2-20M-R/S. (a) plane of action; (b) transverse to the plane of action; (c) sectional arrangement.

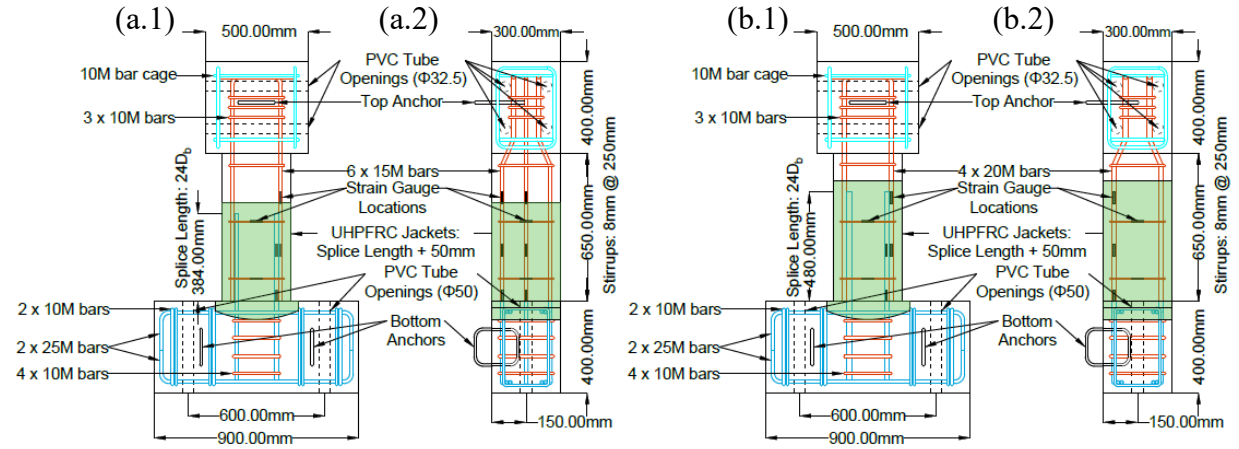


Figure 7.3. Geometry and reinforcement arrangement of (a.1) specimen CS3-15M(in the plane of action of the test), (a.2) transversally to the plane of action; (b.1) specimen CS3-20M in the plane of action, (b.2) transversally to the plane of action.

The Identification Codes of the specimens described in this chapter were included in Table 6.1 of Chapter 6; the last letters (R or S) correspond to Repaired (i.e., previously damaged column), and Strengthened (column in pristine condition) cases, respectively. In the repaired specimens, CT1-15M-R, CT2-20M-R, CS3-15M-R, and CS3-20M-R the damaged concrete was removed first, which included the cover and where needed, the disintegrated core of the specimens. Similarly, the cover around the longitudinal reinforcement on specimens CT1-15M-S and CT2-20M-S was removed and the intact core was maintained. Due to the magnitude of the damage sustained by the pre-damaged specimens, as well as the inherent brittleness of conventional concrete, the final dimensions of the cores were eventually defined by the total undamaged concrete body of the column. In addition, the largest portion of the strain gauges on the embedded reinforcement was rendered inoperable, as a result of the damage imparted by the mechanical impact of the retrofitting tools, and information was gathered from the remaining intact instruments. The embedded reinforcement was left as it was (no alterations) and the footing connection to the floor remained intact.

7.1.1 Determination of bond breaker location with preliminary FEA model

Prior to application of the UHPFRC jackets, the option to install bond breakers at the starter bars of the lap-spliced connection was considered. As it was shown in the literature (Stojadinovic et al. 2019) the use of the bond breakers may improve the spread of longitudinal strains in the lap-spliced connections of flexural members, by alleviating the tendency of strain localization in the critical section on the starter bars of the footing and enabling broader distribution of longitudinal strains in shear span reinforcement. This topic was discussed in Chapter 2 of this study.

A preliminary FEA model was made, using geometric properties similar to the CT2-20M specimen, on the ATENA 3D/ATENA Studio combined platform. The schematic of the non-retrofitted model is shown in Fig. 7.4a). The jacket was modeled by dividing the macroelement topology to contain separate elements for the jacketed and the non-jacketed column sections. The jacket macroelement extended within the footing at the appropriate length. For this version of the FEA model, the footing was assumed to be of complete thickness, to ensure adequate stiffness.

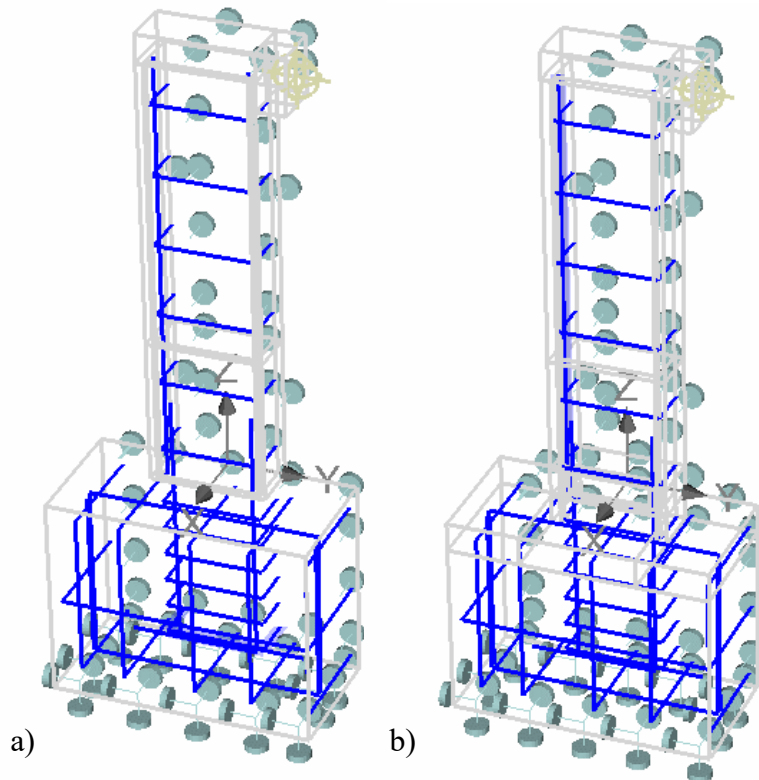


Figure 7.4. Preliminary FEA model for column a) without the UHPFRC jacket and b) with UHPFRC jacket and bond “breaker” openings.

For normal strength concrete, the *3DNonLinCementitious2User* model was used (ATENA Manual 2019), with compressive strength $f'_c = 25\text{MPa}$, modulus of elasticity $E = 30000\text{MPa}$ and Poisson's ratio $\nu = 0.2$. All other parameters were left at default values. For UHPFRC, the *3DNonLinCementitious2* material model was used $f'_c = 120\text{MPa}$, $f_t = 12\text{MPa}$, $E = 45000\text{MPa}$, and Poisson's ratio $\mu = 0.2$. The fracture energy G_f was assumed to be 1.5kN/m for this material (Pantazopoulou et al. 2019; ATENA Program Documentation 2019) – about 15 times the value used for normal strength concrete. For the reinforcement, a bilinear material model was assumed with $f_y = 400\text{MPa}$ and $E_s = 200\text{GPa}$. The bond stress – slip laws were activated for the reinforcement, as it is shown in Fig. 7.5. For the UHPFRC material, the bond stress – slip law was obtained from the work of Tsiotsias (2019).

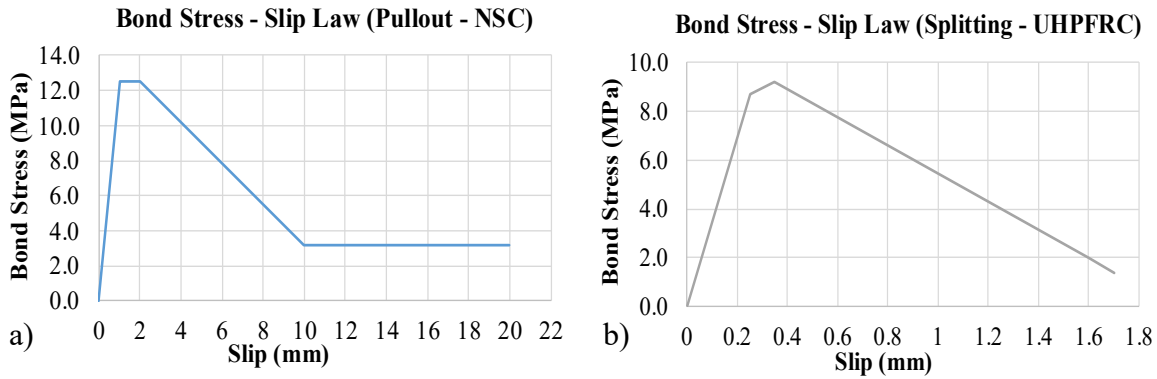


Figure 7.5. Bond stress-slip law for a) footing reinforcement and b) shear span reinforcement.

For the needs of the preliminary investigation of the jacketing/bond breaker scheme four case studies were prepared: i) the column model with no jacket, ii) the column model with the UHPFRC jacket, iii) the column model with the UHPFRC jacket and a bond breaker on the starter bar, at $5D_b$ length within the column shear span (case B1), and iv) the column model with the UHPFRC jacket and a bond breaker on the starter bar, at $5D_b$ length within the footing (case B2). The bond breakers were modelled by modifying the macroelements to contain a “gap” at the desired length of the bars; this approach was recommended by the ATENA 3D manual for the modeling of unbonded segments on reinforcement (ATENA Program Documentation 2019). A schematic of this macroelement topology is shown in Fig. 7.4b, and the unbonded segments, for the case of the bond breakers within the shear span are shown in Fig. 7.6. The column models were analyzed using the default solver properties (Newton-Raphson iterative solver) and the post-processing procedure was conducted in the ATENA Studio platform.

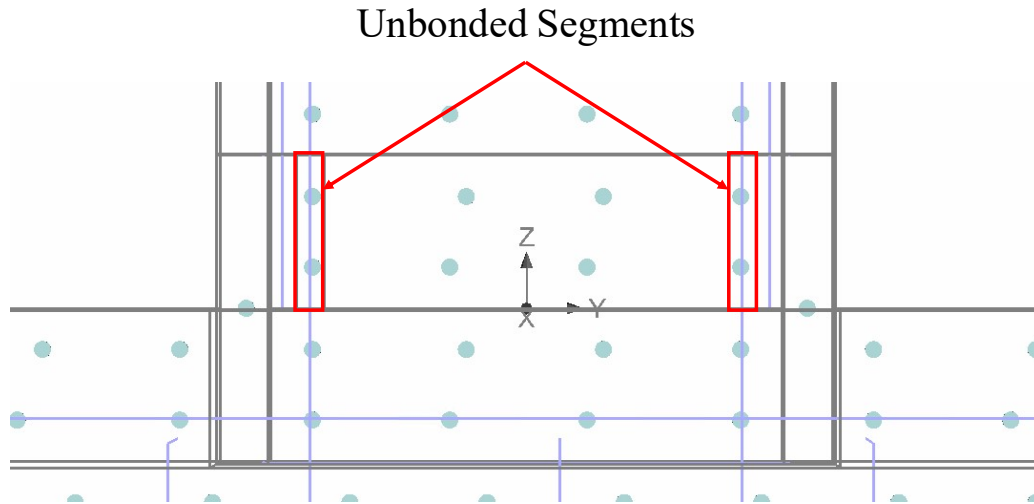


Figure 7.6. Schematic of unbonded segments on the starter bars, within the shear span.

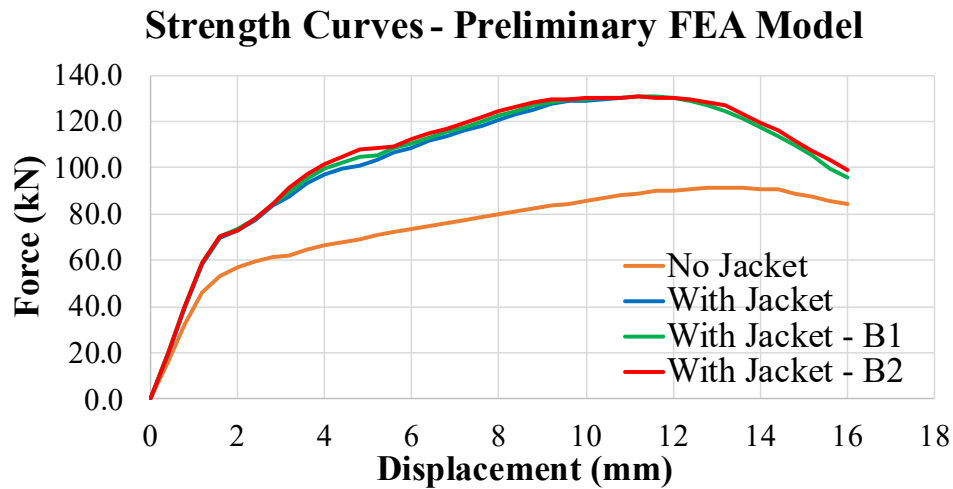


Figure 7.7. Strength curves of the preliminary FEA models.

The results of the analyses are shown in Fig. 7.7, in the force – displacement curves of the columns. The first two diagrams correspond to the models with no jacket and with the UHPFRC jacket, whereas models B1 and B2, correspond to the columns with UHPFRC jackets with bond breakers placed within the column shear span and within the footing, respectively. It was shown that all cases of columns containing the UHPFRC jacket reached greater strength by as much as 43% higher than that of the same column with no jacket. The model with the UHPFRC jacketed and without any bond breakers, reached the peak load at slightly lower lateral displacement and the analysis was terminated at the peak. Both cases of the jacketed columns with bond breakers showed greater ductility, albeit small, with approximately the same peak load.

In Fig. 7.8, the distributions of the longitudinal strains on the reinforcement are shown, for each model case described above. For the control case, containing only conventional concrete, it was seen that yielding occurred within the plastic hinge region, at approximately 50mm above the column-footing interface (displayed with cyan color on the tension side). On the compression side, the longitudinal strains of the reinforcement reached values up to approximately 0.0015. For all the jacketed cases, it was observed that the yielding of the reinforcement was attained further away from the column-footing interface and above the jacketed region. In the case of no bond breakers, the strains in the footing reinforcement were observed to be larger, than the cases with bond breakers, particularly on the transverse reinforcement. However, the strains on the tension side of the shear span reinforcement, were double in the cases where bond breakers were present; the UHPFRC with no bond breakers reached a tensile strain of 0.004 in the shear span reinforcement and the UHPFRC with bond breakers reached tensile strains up to 0.0088. Lastly, in the UHPFRC-jacketed specimens the longitudinal strains on the compression side were significantly reduced, compared to the non-jacketed case, without exceeding the value of 0.0005.

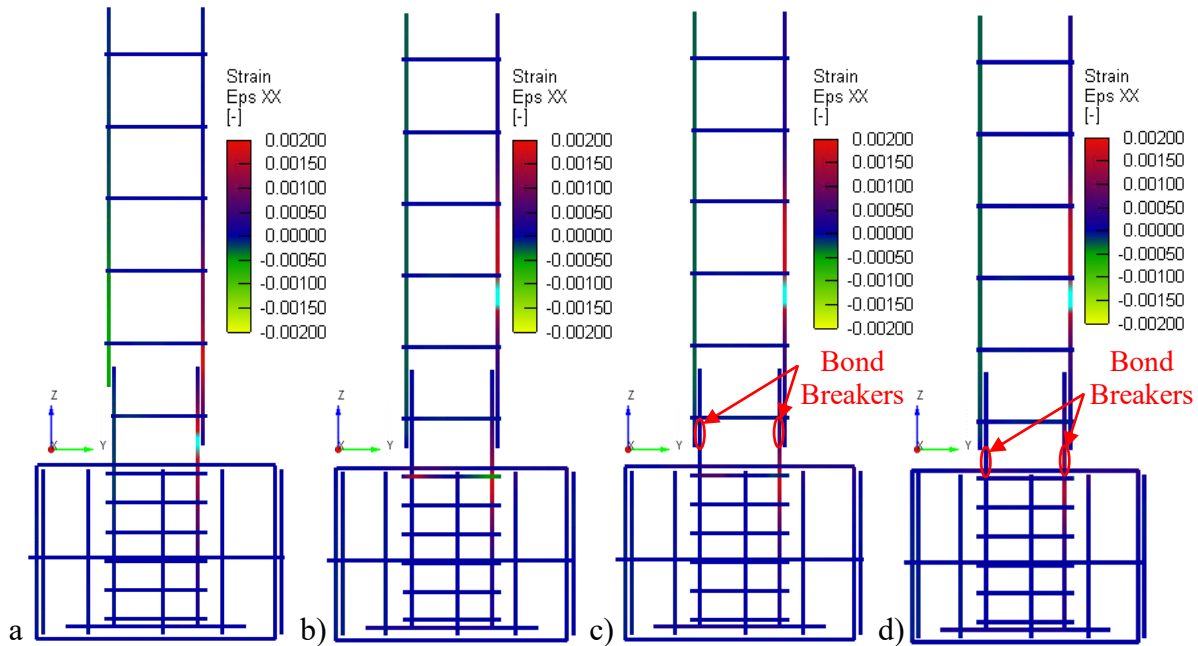


Figure 7.8. Axial strain and cracking distributions for preliminary FEA models for a) the column without jacket, b) the column with the UHPFRC jacket, c) the jacketed column with bond “breakers” placed within the shear span, and d) the jacketed column with bond “breakers” placed within the footing.

Between cases B1 and B2, the distribution of the longitudinal strains was virtually identical, with the strains on the tension side of the shear span to be slightly higher in the former case. As such, it was selected that bond breakers would be placed within the shear span.

7.1.2 Pre-Damaged Specimens for Repair

Of the specimens included in the pre-damage stage, the surface concrete was removed initially with power drills, until the reinforcement was revealed. To ensure stability of the specimens, vertical wooden studs were placed, on each side of the column, by resting under the armpit of the cap beam – column connection. The specimens were also connected with lateral straps that were placed around the cap beams and wooden beams which were placed between them for additional stability. Pertinent pictures illustrating these processes are included in Appendix C.

To reduce the effect of strain concentration on the bottom bars of the lap-splice connection, the segment at the upper end of the starter bars was covered with plastic tape, at a length of approximately $5D_b$. Only the starter bar was covered with the tape. This method was selected in order to eliminate the bond connection between the bottom bar and the concrete, within the taped segment, allowing for the spread of strains to occur on the column bar and further away from the footing.

The same process was applied on all pre-damaged specimens, including CT1-15M-R, CT2-20M-R, CS3-15M-R, and CS3-20M-R (pertinent figures in Appendix C). The concrete in the column-footing connection was sufficiently removed, until the embedded reinforcement was revealed. The damaged core was dislocated and removed, and the intact concrete was cleaned using the power drills. Sufficient space was allowed for the UHPFRC jacket to be poured around and behind the bars. The inner surface of the column-footing connection was also cleaner, in order to create a consistent surface for the placement of the UHPFRC jacket. The sides and front and back surfaces of the footing were also removed, in the aforementioned distances, and with sufficient care to avoid sudden delamination of the cover. Prior to casting, the surface concrete was cleaned for residual particles and the surface concrete was lightly wetted, to facilitate the cohesion to the UHPFRC jacket.

7.1.3 Cover Removal in Pristine Specimens for Strengthening

The preparation of the intact specimens was similar to the process followed for the pre-damaged specimens. Cover concrete was removed up to a depth of $\sim 1D_b$, while the core was maintained throughout the length of the retrofitted region. The surface of the core was cleaned from loose fragments, to ensure consistent connection with the UHPFRC jacket. Concrete removal for these specimens required greater effort, due to the undamaged state of the material. Sufficient care was required such that no sudden spalling occurred penetrating into the core of the column. The column-footing connection was also constructed to be in the same manner as in the repaired specimens. Both the sides and front and back surfaces were removed to the depth dimensions mentioned in the preceding.

The starter bars were covered with plastic tape as a bond breaker, in the same manner as in the pre-damaged specimens, to ensure better strain spreading and avoiding strain localization. Strain gauges were maintained where possible, and the connecting wires were placed outside the jacket zone. Residual components and dust were cleaned, and the exposed concrete was wetted.

7.2 Casting of UHPFRC Jackets

For casting of the UHPFRC jackets, customised plywood formworks were constructed, which were fastened on the columns with mechanical grips (Fig. 7.9). The formwork consisted of 18mm thick plywood panels, placed on each side of the column, on which custom openings were provided for pouring of the UHPFRC jacket. Mechanical clamps were placed on the sides of the columns, two pairs on each side, to ensure stability of the formwork during casting. This arrangement allowed for versatile placement and removal of the formwork, on account of the restricted access to the lab facilities at the time of completion.

Casting of the UHPFRC jackets was completed in four separate batches. During each batch, the UHPFRC was prepared in a high-capacity mixer, with a 40L rotating drum. The mix design used is described in Table 7.1. During the preparation process, the prepackaged dry mix was initially placed in the mixer for up to 2 minutes. The water content divided into two proportions (half each), and each portion was poured slowly, and mixing continued for 5 minutes, for each case. After mixing with the water was complete, the steel fibers were slowly added, and were manually

dispersed, to ensure even distribution in the mixture. Finally, after all the fibers were added, mixing was continued until the optimal material consistency was achieved.

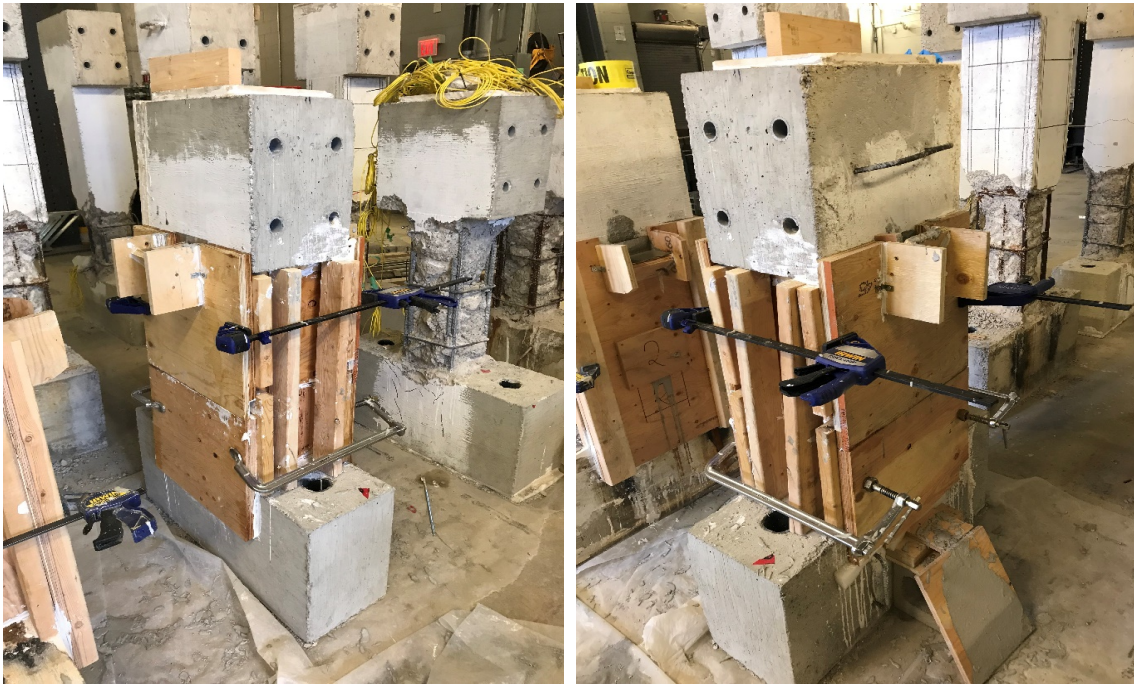


Figure 7.9. Attached plywood formwork panels, using mechanical grips, and custom openings for concrete pouring.



Figure 7.10. Retrofitted specimen with UHPFRC jacket, after removal of formwork, at two days.

Table 7.1. Material Mix Design for Densit© UHPFRC mixture.

Material Type	Proportions by Mass (kg)	Density (kg/m ³)
Prepackaged Dry-Mix (Densit)	0.84	2860
Steel Fibers ($d_f = 0.2\text{mm}$; $l_f = 13\text{mm}$)	0.11	
Water	0.055	

The UHPFRC jackets were cast immediately after mixing, through the openings provided on the formwork. Pouring was terminated once the level of the UHPFRC within the formwork reached the capacity of the formwork. After casting, the formwork was inspected to ensure no leakage occurred. The formwork was maintained for two days, as per the optimal curing instructions for the material (Fig. 7.10). At removal of the formwork, the jackets were covered with soaked burlap, which was wrapped around the columns. To maintain moisture, the burlaps were covered with plastic sheets (Fig. 7.11), and the columns remained covered until 28 days had passed. Additional water was added to the burlap throughout this time to ensure adequate moisture on the jacket.



Figure 7.11. Covering of UHPFRC jacket with soaked burlap and plastic sheets for moisture retention.

7.3 Preparation of UHPFRC-Retrofitted Columns for Testing

At removal of the jacket coverings, the surfaces of the specimens were cleaned and subsequently painted white, using the white primer paint, as mentioned in Chapter 6. The procedure following the preparation of the jackets was the same as in the phase I testing of specimens: firstly, the specimens were aligned in the axial direction by casting a layer of plaster under the footing. Then, using the crane mechanism, the specimens were transferred in the test zone and under the axial actuator, where they were fastened to the Strong Floor using threaded steel rods. The rods were fastened by steel nuts that bore on the UHPFRC plates used in the previous tests. Once fastening of the footing was completed, the cap beam was connected to the horizontal and axial actuators, using the threaded rods and metallic plates, as described in Chapter 6, respectively. Instrumentation was also added in the similar arrangements as shown in Fig. 6.12; string potentiometers were placed in the same locations, as in the conventional concrete specimens, on the jacket surface. Tests were repeated, in the same process as with the specimens of the previous chapters.

7.4 Experimental Parameters

Prior to each test, a set of 75mm × 150mm UHPFRC cylinders were tested in the Pilot Controls testing frame, to measure the compressive strength of the material. Four sets of cylinders were tested, each set containing three cylinders. The cylinders were shown to fail abruptly and with a loud noise, which was owing to the high compressive capacity of the UHPFRC material. Failure was characterized with distinct near vertical cracks along the cylinder surface with cone formation at the top, as shown in Fig. 7.12. The cumulative results of the compressive tests on the UHPFRC cylinders are shown in Table 7.2.

Due to the expected improved performance of the retrofitted specimens, the loading regime was updated to reach drift ratios to 5%. The corresponding target values for the drift increments are listed in Table 7.3 and a schematic of the loading protocol is shown in Fig. 7.13. It was noted, however, that due to restrictions of the geometry of the experimental testing frame, only the specimens of the shorter shear span ($H_v = 850\text{mm}$) were able to reach drift ratios higher than 4%. The column specimens of the larger shear span ($L_v = 1450\text{mm}$) reached up to a maximum of 3.5% drift, corresponding to ~ 50mm, at which stage the tests were terminated.



Figure 7.12. Cracked 75mm × 150mm UHPFRC cylinder after compression test.

Table 7.2. Cylinder compressive strengths for UHPFRC cylinders.

Column ID	Cylinder ID	f_c' (MPa)	f_c' (MPa) - Average
Ct1-15M-R	CU1-1	175.05	172.48
	CU1-2	166.51	
	CU1-3	158.23	
Ct2-20M-R	CU2-1	169.44	169.23
	CU2-2	171.83	
	CU2-3	166.69	
CS3-15M-R	CU3-1	143.03	142.36
	CU3-2	145.13	
	CU3-3	138.93	
CS3-20M-R	CU4-1	152.31	152.54
	CU4-2	147.08	
	CU4-3	158.23	

Similar to the tests performed on the pre-damaged specimens, three complete cycle reversals were completed at each target displacement increment, manually triggered on the software interface. The tests were recorded using the combined camera and light projector setup described in Chapter 6. Both the DSLR and video cameras were used. Results were obtained through the attached

instrumentation and the response of the specimens was measured via the string potentiometers attached on the specimens.

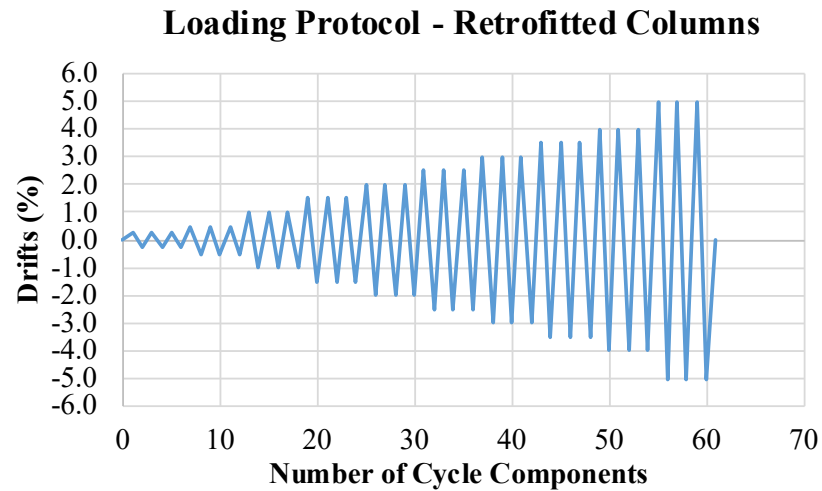


Figure 7.13. Loading protocol for the retrofitted columns.

Table 7.3. Drift ratios for tall and short column specimens and equivalent lateral translation, updated for tests on the UHPFRC-enhanced specimens.

Drift Ratios (%)	<i>Tall Columns</i> ($L_v = 1450$ mm) (mm)	<i>Short Columns</i> ($L_v = 850$ mm) (mm)
0.25	3.625	2.125
0.5	7.25	4.25
1.0	14.5	8.5
1.5	21.75	12.75
2.0	29	17
2.5	36.25	21.25
3.0	43.5	25.5
3.5	50.75	29.75
4.0	-	34
5.0	-	42.5

7.5 Results

In this section, the results from the experimental testing are presented. The data are discussed based on the results obtained from the attached instrumentation.

7.5.1 Force-Displacement Response Curves

The complete force-displacement curves of the CT1-15M-R/S and CT2-20M-R/S specimen series are shown in Figs. 7.14 and 7.15, respectively. The same curves for the specimen series of CS3-15M-R and CS3-20M-R are also shown in Figs. 7.16 and 7.17, respectively. Both the curves of the conventional and retrofitted columns were included in the graphs, to compare the contribution of the jacket to the specimen performance. With the exception of specimen CT1-15M-S, all experiments were conducted in one continuous curve, up to the maximum value of drift; specimen CT1-15M-S was tested in two separate sessions, due to the presence of off-plane eccentricity in the axial actuator. In this particular case, the first session was terminated at 2% drift, at which point the specimen was reconnected to the testing frame and external instrumentation at a later date, and the second session was conducted between the drift ratios of 2.5% and 3.5%. The first and second sessions were denoted by the ending letters “-A” and “-B”, respectively. The tabulated data of the force-displacement curves are shown in Tables 7.4 and 7.5, and the load measurements at the various drift levels were included.

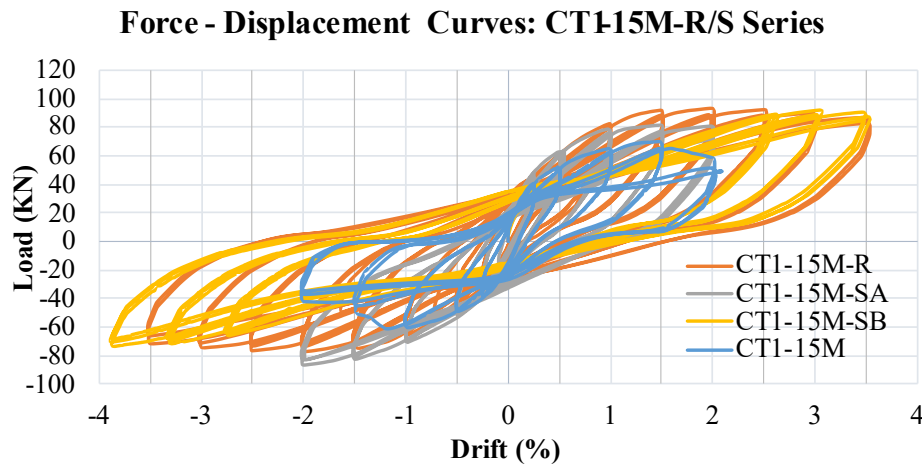


Figure 7.14. Force – Displacement curves for the CT1-15M-R/S specimen series.

In the case of the CT1-15M-R/S specimen series, the load capacity was increased by 36% and 21%, in the repair and strengthening retrofitting scenarios, respectively. In the former case, the peak load was attained at the drift of 1.5%, while in the latter case, the peak was obtained at a drift of 3%, during the second session of the experiment. Both specimens showed comparable strengths at drift levels exceeding the 2% milestone. In both cases, both directions of loading showed almost completely symmetrical responses, with the strength being reduced significantly at drift ratios

exceeding 2%. In addition, it was shown that until the final drift of 3.5%, strength reduction was not more than 10%, between successive drift magnitudes, in both UHPFRC-enhanced specimens, while in the non-retrofitted specimen the strength was reduced by approximately 25%, at the final drift ratio of 2%. Lastly, between successive cycles at the same drift magnitude, the strength reduction was on average 5% and 13%, in the UHPFRC-enhanced and non-retrofitted specimens, respectively.

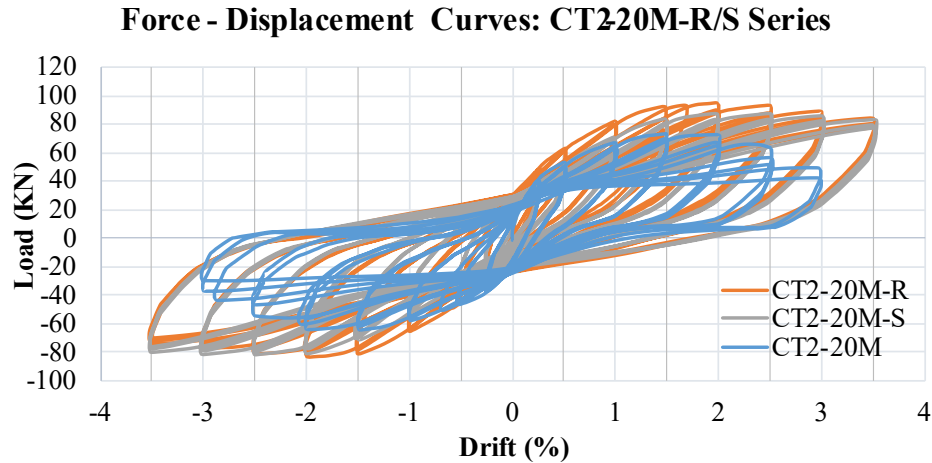


Figure 7.15. Force – Displacement curves for the CT2-20M-R/S specimen series.

The behavior described above was also in accordance with the CT2-20M-R/S specimen series. In this series, the maximum load attained for the repair and strengthening retrofit scenarios was 29% and 16% higher than the non-retrofitted specimen, respectively. For the repaired specimen, peak load was obtained at 1.5% drift, while for the strengthened case, the peak load was obtained at 2%. In both cases, the load was symmetrical in both directions of loading, even at drift magnitudes exceeding 2%. Strength reduction between successive cycles did not exceed 12% in the repair and retrofitted specimens, for up to 3.5% drift, however, the initial specimen showed loss of strength by 25% in the final drift level of 3%. Between successive cycles of the same drift magnitude, the loss of strength was 5% for the UHPFRC-enhanced specimens, while, for the non-retrofitted specimen the load dropped up to 40% prior to the termination of the experiment.

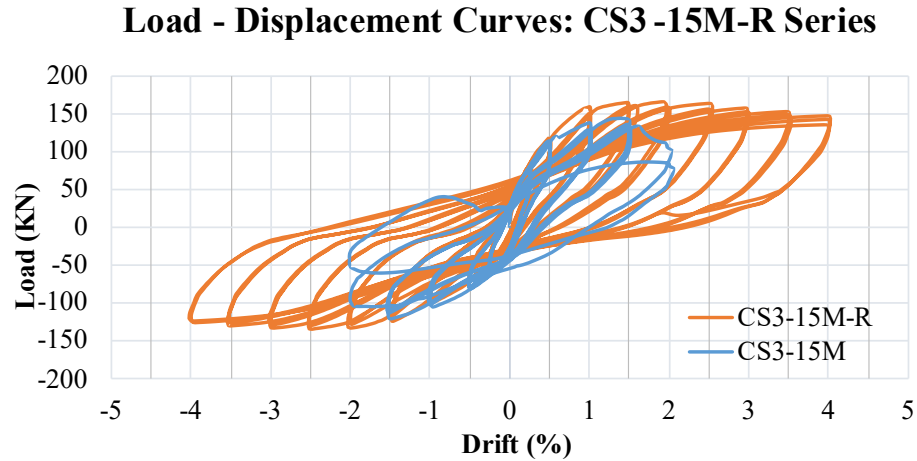


Figure 7.16. Force – Displacement curves for the CS3-15M-R specimen series.

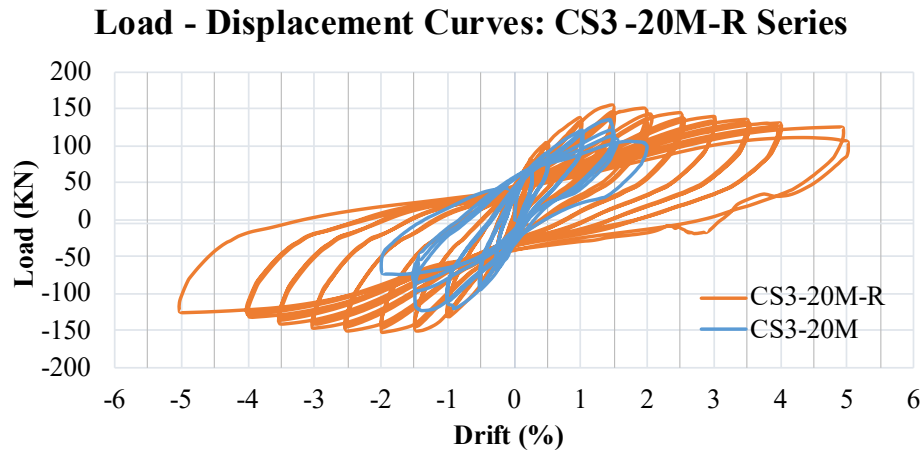


Figure 7.17. Force – Displacement curves for the CS3-20M-R specimen series.

In the CS3-15M-R specimen series, the strength was increased by 15%, with the addition of the UHPFRC jacket, after repair. In the repaired case, the peak load was observed at the drift magnitude of 1%, while in the non-retrofitted case, the peak was attained at a drift of 1.5%. While in the initial specimen the load was reduced by approximately 30% after the maximum value, the repaired specimen showed gradual strength deterioration, without exceeding more than 8% in the load reduction between successive drift magnitudes. In addition, between successive cycles of the same drift magnitude, the strength reduction of the UHPFRC-repaired specimen remained under 10%, even after reaching the drift of 4%. The non-retrofitted specimen showed abrupt reduction of strength in the post-peak range, particularly after reaching the drift of 2%, where the strength was reduced to less than 50% of the previous magnitude.

In the CS3-20M-R specimen series, the strength was increased by approximately 25%, with the addition of the UHPFRC jacket, after repair. In both cases, the peak load was observed at the drift ratio of 1.5%. Strength deterioration was gradual in the repaired specimen of this series as well, with load reduction not exceeding 7% between successive drift ratios, even after attaining the ratio of 5%. On the contrary, in the initial specimen, the strength was reduced by 25%, at the maximum drift of 2%, which was the cycle immediately after the attainment of the peak load. Between successive cycles of the same drift ratios, the UHPFRC-enhanced specimen did not exceed 5% in strength reduction, while the non-retrofitted specimen showed strength reduction up to 15%.

It was evident of the specimens described above, that the addition of the UHPFRC jacket, both in the repair and retrofitting scenarios, significantly contribute to the performance of the columns under cyclic loading. Of the specimens constructed with the normal shear span ($L_v = 1450\text{mm}$), the deformation capacity was effectively doubled, with the final drift ratios for all specimens reaching up to 3.5% (the maximum available for this experiment) and, similarly, the short shear span specimens ($L_v = 850\text{mm}$) could be safely tested until the drift of 5%. Strength improvements were also noted in both cases, with strength reduction between successive cycles being gradual and stable. Failure in all specimens was localized in the column-footing interface; in particular, the interface defined by the extended UHPFRC jacket and footing surfaces. Flexural cracks were minimal, with sparse and fine cracking being developed throughout the jacket surface, as well as within the column, at the termination of the jacket. Shear cracking was absent from all specimens.

In the following figures (Figs. 7.18-7.23) the cracking patterns throughout the experiment are shown, as captured by the dedicated DSLR camera. In specimen CT1-15M-R, in the second session of the experiment, no external instrumentation was attached on the UHPFRC jacket, on account of the existing damage at the end of the first session. The force-displacement response was obtained via the string potentiometers attached to the cap beam of the specimen.



Figure 7.18. CT1-15M-R at drift ratios: a) 0.25%; b) 0.5%; c) 1%; d) 1.5%; e) 2%; f) 2.5%; g) 3%; h) 3.5%.

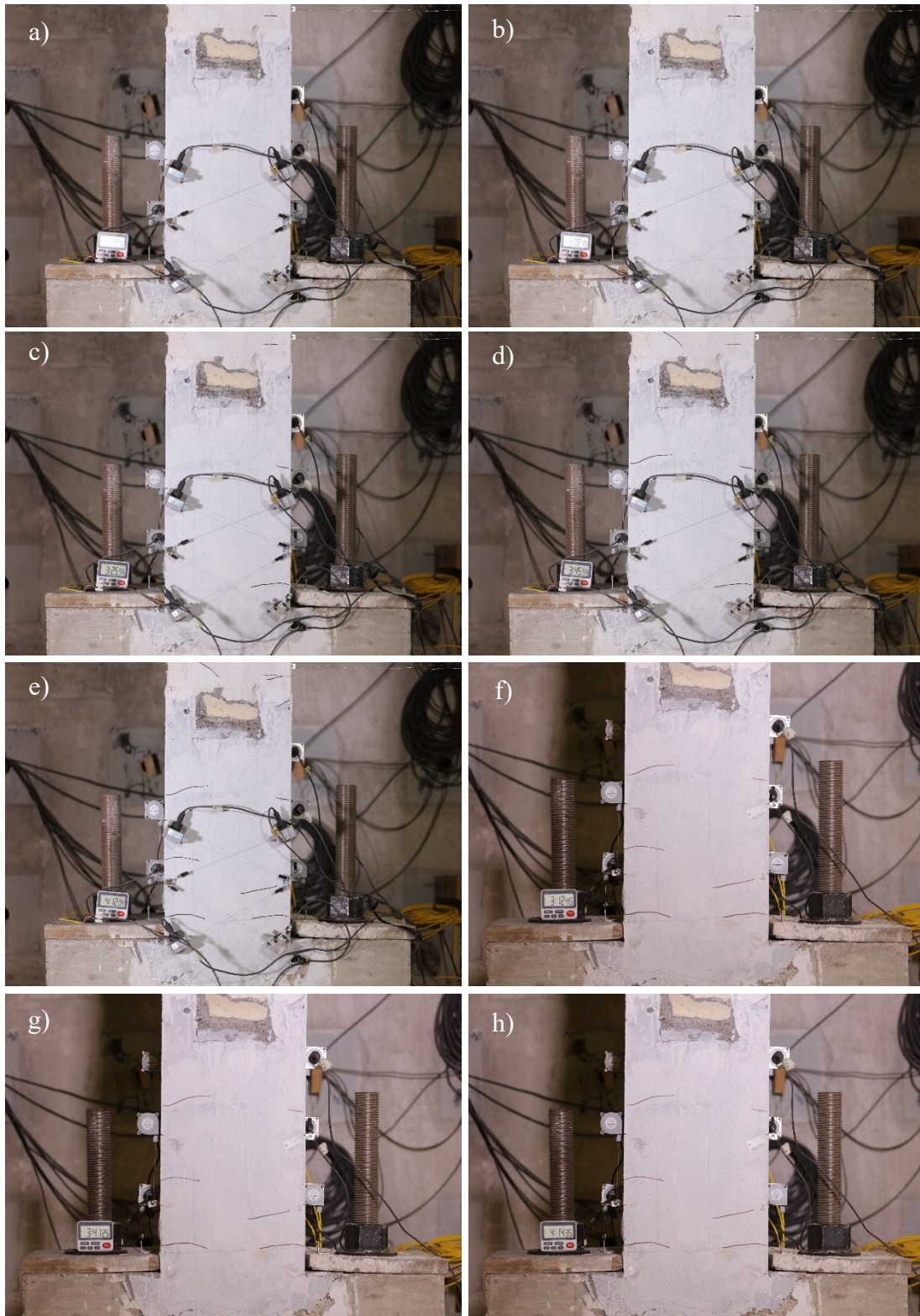


Figure 7.19. CT1-15M-S at drift ratios: a) 0.25%; b) 0.5%; c) 1%; d) 1.5%; e) 2%; f) 2.5%; g) 3%; h) 3.5%.

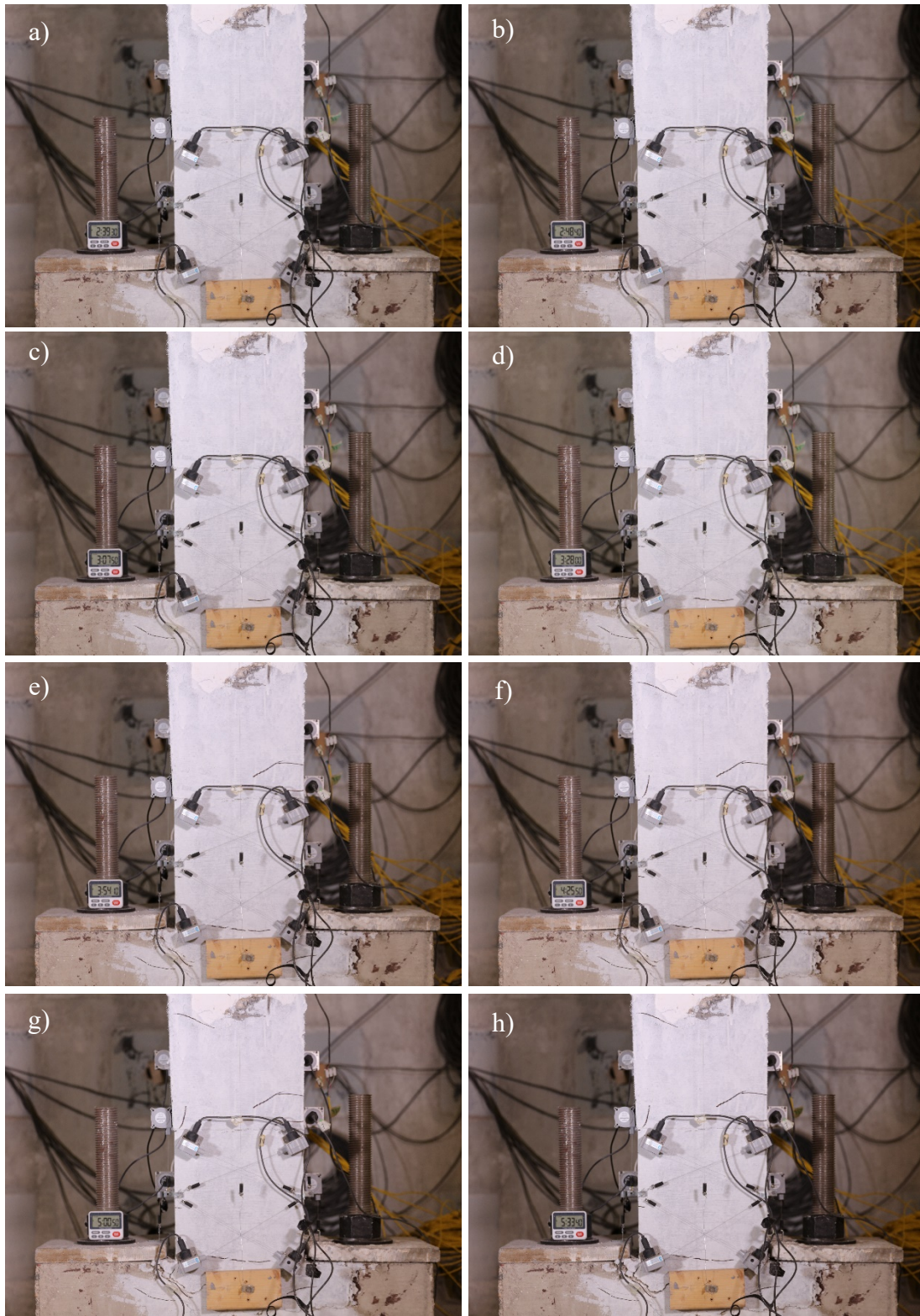


Figure 7.20. CT2-20M-R at drift ratios: a) 0.25%; b) 0.5%; c) 1%; d) 1.5%; e) 2%; f) 2.5%; g) 3%; h) 3.5%.

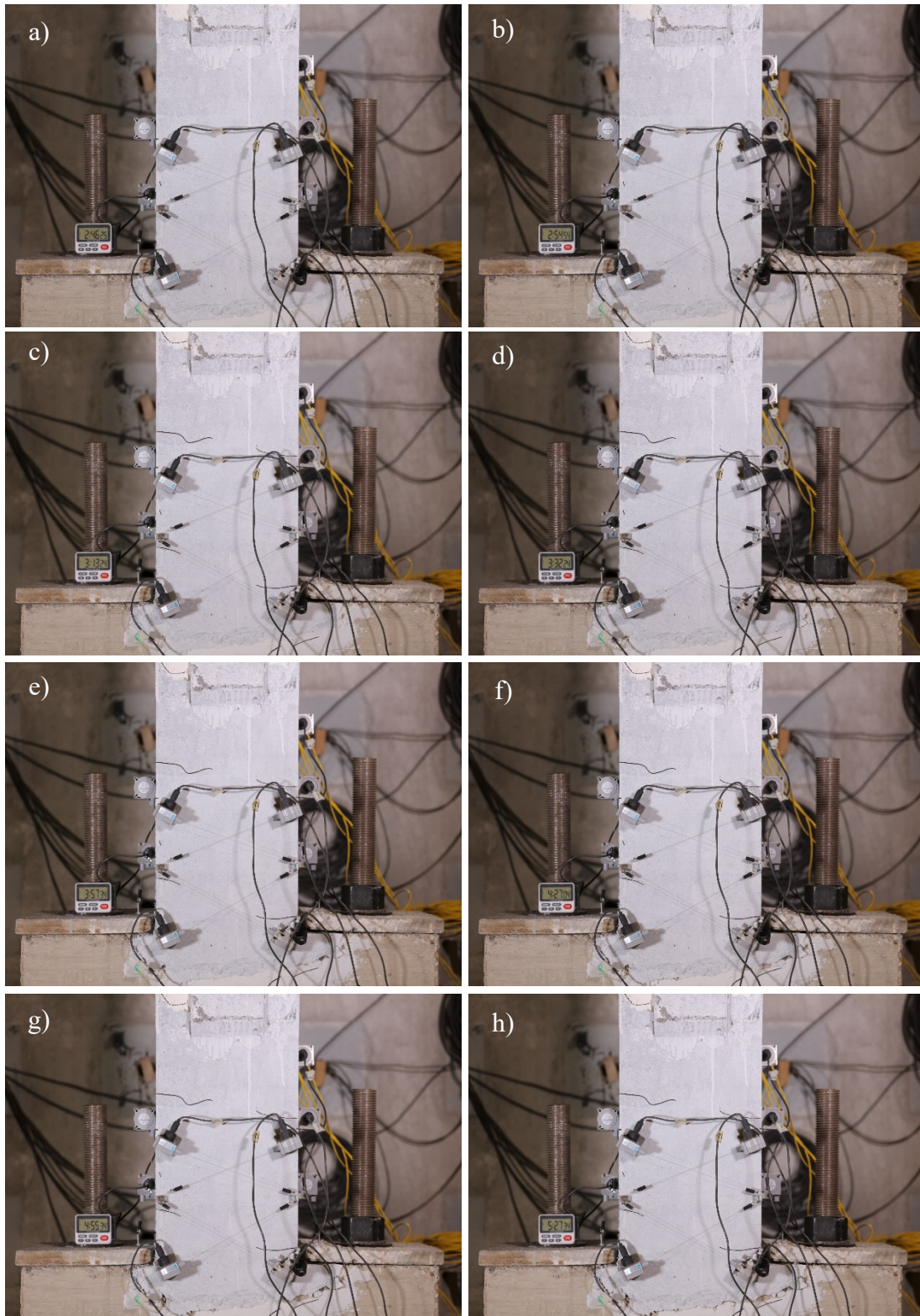


Figure 7.21. CT2-20M-S at drift ratios: a) 0.25%; b) 0.5%; c) 1%; d) 1.5%; e) 2%; f) 2.5%; g) 3%; h) 3.5%.

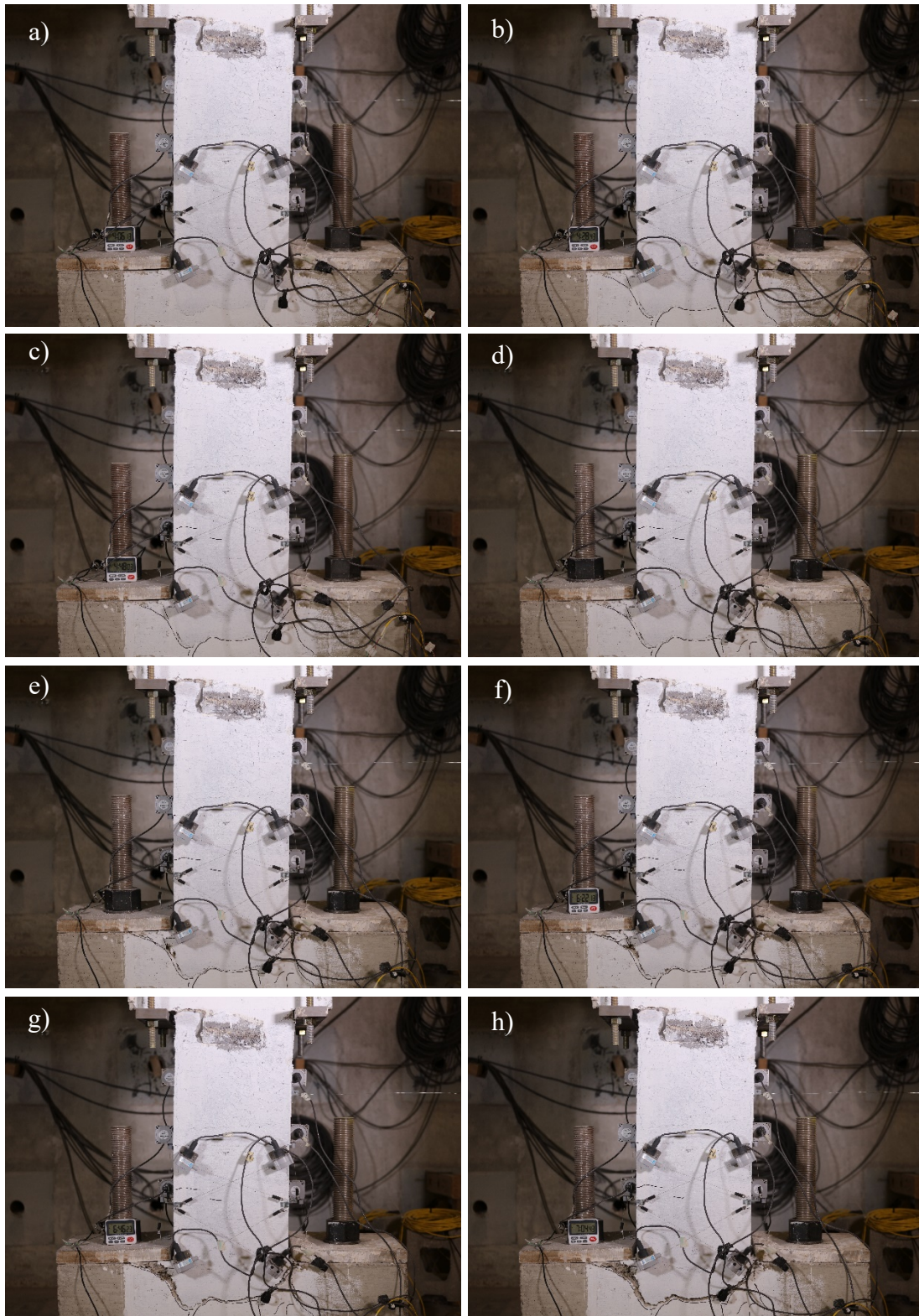


Figure 7.22. CS3-15M-R at drift ratios: a) 0.5%; b) 1%; c) 1.5%; d) 2%; e) 2.5%; f) 3%; g) 3.5%; h) 4%.

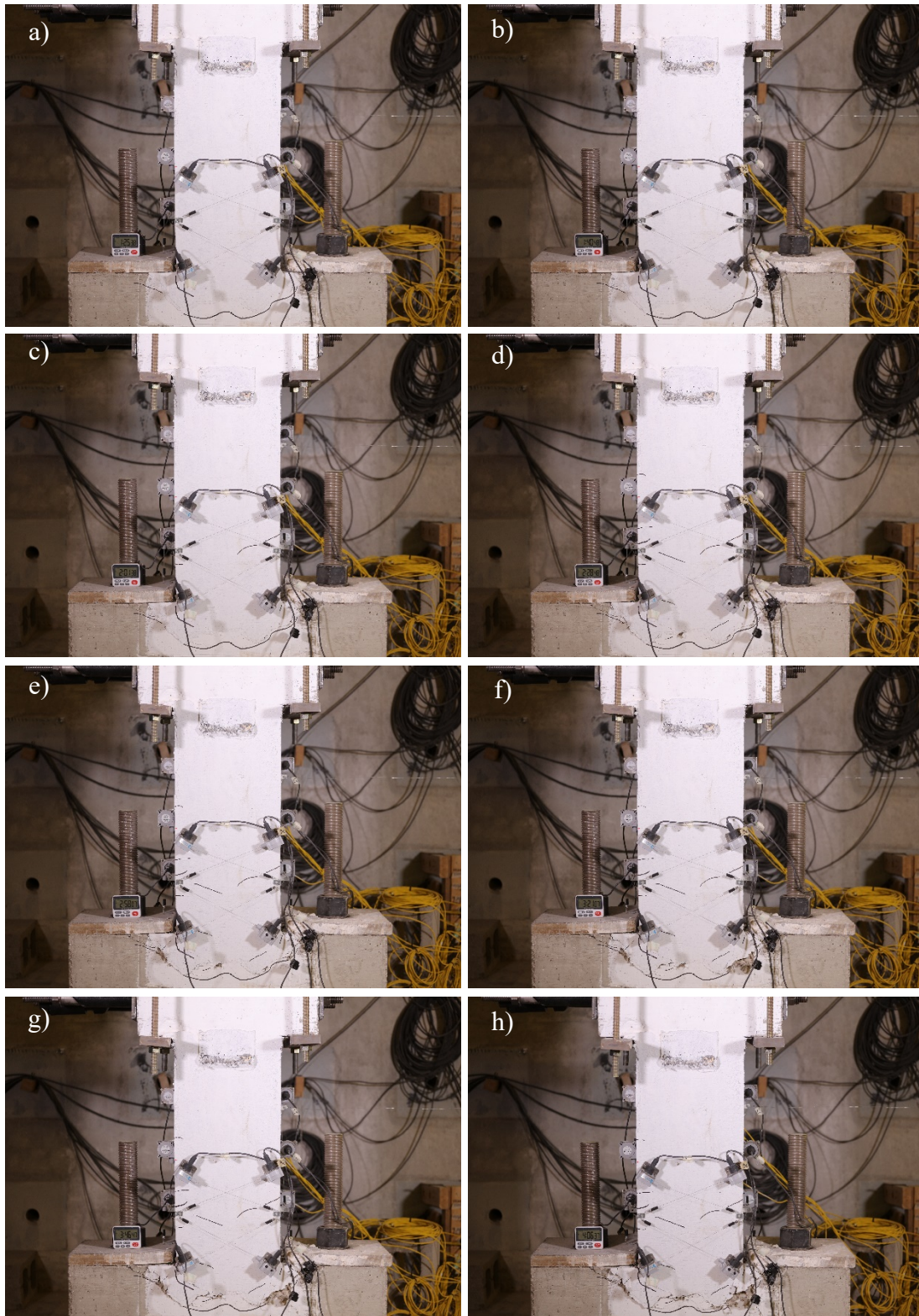


Figure 7.23. CS3-20M-R at drift ratios: a) 1%; b) 1.5%; c) 2%; d) 2.5%; e) 3%; f) 3.5%; g) 4%; h) 5%.

It is observed that, in most of the tests, the spacing between the flexural cracks was sparse, in comparison to the non-retrofitted specimens. In addition, no cracks indicating crushing of the concrete or longitudinal cracking along the lap-spliced bars were observed near the final drift ratio, as it was observed in the previous chapter. Lastly, the short shear span specimens showed virtually no other cracks, with the exception of the column-footing interface, at the end of the UHPFRC jacket. Thus, it can be safely concluded that the shear mechanism was mitigated to a great extent by the UHPFRC material.

7.5.2 Curvature Distribution

Curvatures were calculated using the method described in Chapter 6. The curvature distributions, along the column height, are shown for the CT1-15M-R, CT2-20M-R, CS3-15M-R and CS3-20M-R cases, in Figs. 7.24-7.27, respectively. Comparisons with the non-retrofitted corresponding cases are also shown in the figures on both the positive and negative sides of loading. On account of similarity of results, the strengthened cases are not shown, for clearer presentation. In Figs. 7.24-7.27, the dashed lines correspond to initial specimens and the continuous lines to the repaired counterparts.

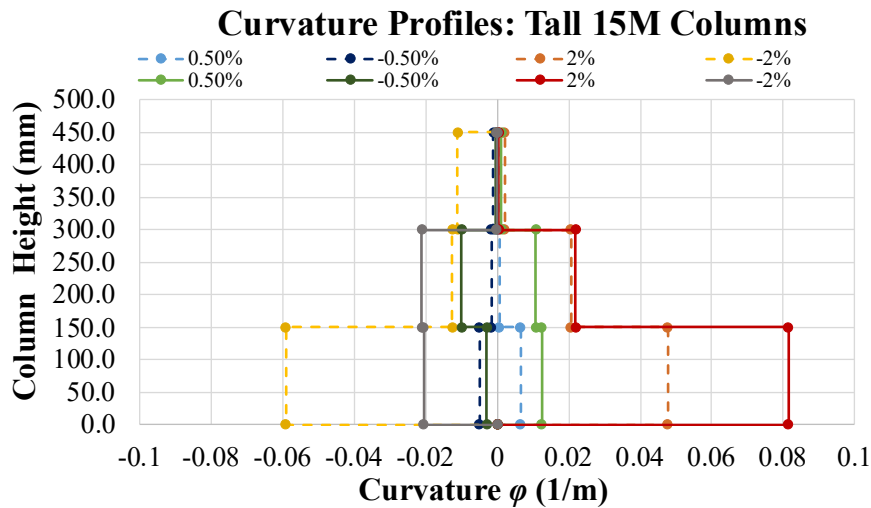


Figure 7.24. Curvature distribution along column height for the CT1-15M-R specimen series.

In the CT1-15M-R specimen series, at 0.5% drift, curvatures on the bottom segment of the column (0 mm – 150 mm) were shown to be approximately double than those of the initial case, on the positive side, 0.012 rad/m. For the upper segments of the column, a similar pattern was observed, however, with smaller magnitudes of curvature. On the negative side, throughout the column

height, both the repaired and initial cases showed similar values, with the curvature in the bottom segments obtaining values up to -0.005 rad/m. For 2% drift, in the bottom segment of the column, the positive side showed larger curvatures for the repaired specimen case, exceeding the non-retrofitted specimen by 70%. On the negative side, the curvature of the repaired specimen was less than the non-retrofitted case. This was attributed to the localization of failure under the jacketed region, on the column footing interface. For the upper segments of the column (150mm and higher) the curvature on both sides of the repaired specimen were similar, with maximum curvature values of 0.02 rad/m, and reducing for higher column height, which exceeded the values of the non-retrofitted case.

For specimen CT2-20M-R, the curvature response was virtually symmetrical, in the positive and negative sides at both the 0.5% and 2% drift ratios. In the bottom segments, the curvature of the repaired specimen exceeded the non-retrofitted case on the positive side but showed lower curvature on the negative side. It was also shown that, for 2% drift, the repaired specimen showed 76% of the curvature observed in the non-retrofitted specimen, indicating that the repaired specimen showed a prolonged flexural response.

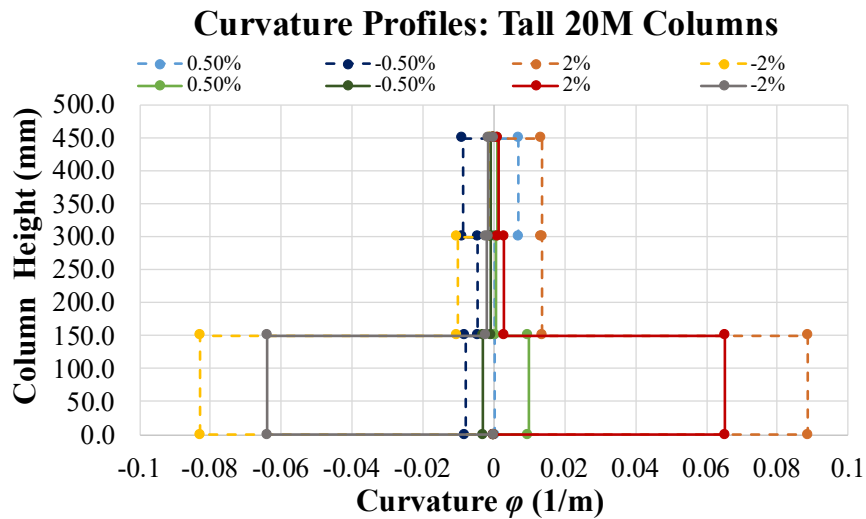


Figure 7.25. Curvature distribution along column height for the CT2-20M-R specimen series.

For specimens CS3-15M-R and CS3-20M-R, the curvatures in the bottom segment (0mm – 150mm) were larger on both the positive and negative sides, than the remainder of the column height. At 2% drift, the maximum curvature was 0.031 rad/m and 0.042 rad/m, for the former and latter case, respectively. This pattern shows that the failure mode in the short retrofitted columns

was similar to that of the tall columns (flexural), as the shear resistance provided by the UHPFRC jacket eliminated shear failure completely – thus the distribution of curvature was not affected by cracking patterns characteristic of shear failure. This is a critical finding, as the UHPFRC jacket provided substantial resistance that compensates for the sparse transverse reinforcement of the original intact specimen.

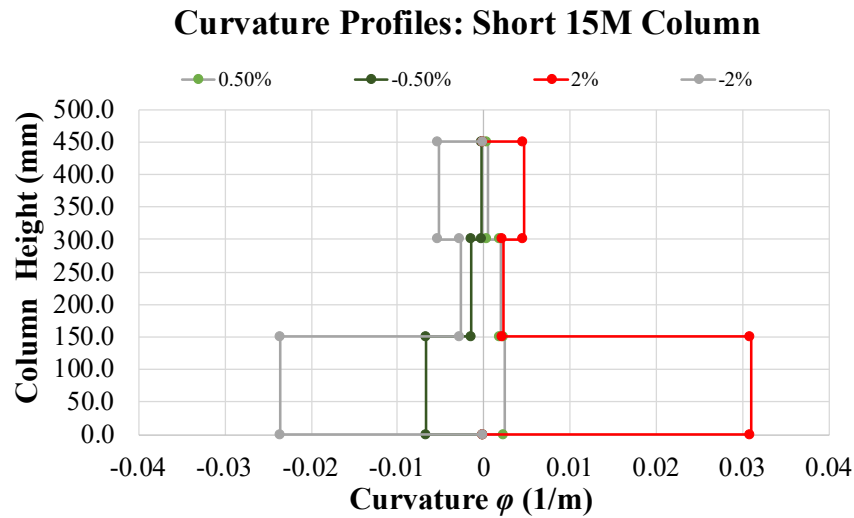


Figure 7.26. Curvature distribution along column height for the CS3-15M-R specimen series.

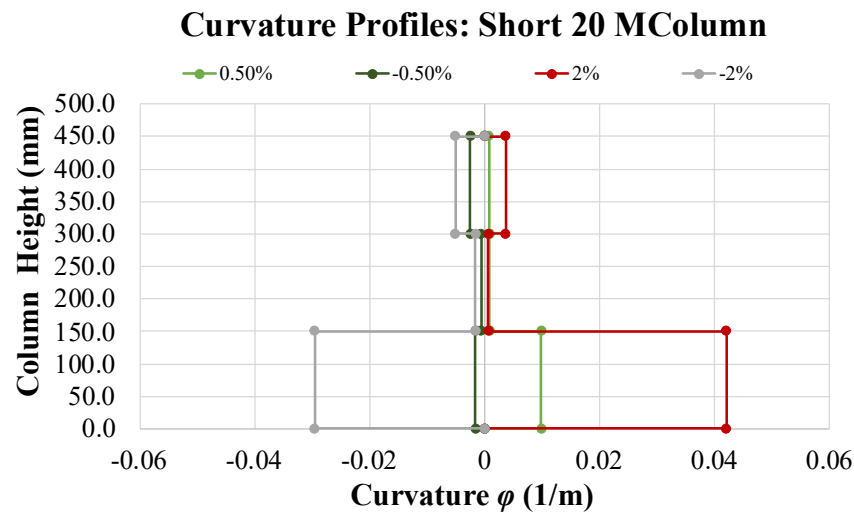


Figure 7.27. Curvature distribution along column height for the CS3-20M-R specimen series.

7.5.3 Strain Gauge Measurements

For the strain gauge measurements, the same instrumentation on the embedded reinforcement as shown in Fig. 6.12 was used. The majority of the gauges were damaged, during the pre-damaging

phase of the experiment, while in the strengthened specimens, some of the strain gauges were removed during the concrete removal process. Based on the available data, some conclusions were drawn regarding the strain development on the longitudinal and transverse reinforcement.

For the normal shear span specimens ($L_v = 1450\text{mm}$), higher strains were observed on the corner longitudinal column bars. Strains did not exceed the value of $1000 \mu\epsilon$ on the top end of the lap-splice, whereas in the middle of the lap splice, higher strains were developed on the column bar, reaching up to $1500 \mu\epsilon$. On the starter bars, the strains towards the portion of the bar above the unbonded zone were negligible. On the contrary, on the portion of the bar under the unbonded zone, the strains exceeding yielding values ($2000 \mu\epsilon$), even at the drift ratio of 2%; for even higher values of drift, the strain gauges reached their maximum capacity. On the transverse reinforcement, it was shown that strains did not exceed the values of $1000 \mu\epsilon$ throughout the duration of the experiments.

For the short shear span specimens ($L_v = 850\text{mm}$), the same pattern of behavior was observed, with strains on the longitudinal bars above the unbonded zone not exceeding $500 \mu\epsilon$. Under the unbonded zone, the strain gauges exceeded yielding limits, and exhausted their capacity towards the end of the experiment.

7.5.4 Shear Strains and Drift

The configuration to obtain the shear strains was the one shown in Fig. 6.12, and the final values for the shear strains and shear drift were obtained from the measurements based on Eqs. 6.12-6.13. The shear drift versus the lateral drift diagram is provided in Fig. 7.28.

Based on Fig. 7.28, it is evident that the shear drift is dramatically reduced in the repaired and strengthened specimens, thanks to the presence of the UHPFRC jacket. All cases of retrofitted specimens do not exceed the shear drift of 0.01%, while the largest value of shear drift is obtained from the CT1-15M-R specimen, at 0.0084%. Compared to the non-retrofitted specimen, the CT1-15M-R and CT1-15M-S showed a reduction in shear drift of 62% and 68%, respectively, at 2% lateral drift. In similar fashion, the reduction in shear drift at 2% was 32% and 72%, for the CT2-20M-R and CT2-20M-S, respectively, and for specimen CS3-20M-R the reduction was 96%. It is also seen that the strengthened specimens showed greater reduction in shear drift than the repaired specimens. As such, the presence of the UHPFRC jacket greatly reduced overall shear drift in both the repair and retrofitting scenarios.

Shear Drift: All Columns

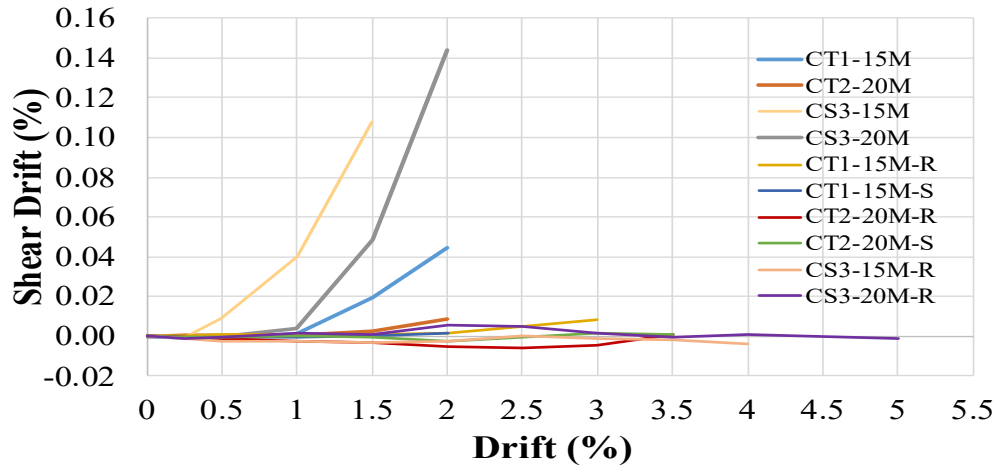


Figure 7.28. Shear drift versus lateral drift ratio for all column specimens.

7.5.5 Axial Elongation of the Columns

The axial displacement was measured using the vertical string potentiometer that was placed on the cap beam and along the column axis, as it is shown in Fig. 6.12. Plots of the displacement in respect to the drift ratios, for the repair and strengthened specimens are shown in Figs. 7.29-7.30.

For the tall columns, the repaired specimen of the CT1-15M-R series, reached an axial shortening value of 0.6mm. Throughout the experiment, the axial shortening measurements increased gradually, up to the maximum value at the drift ratio of 3.5%. This was contrary to the non-retrofitted specimen, which showed an elongation of 1.5mm, at the maximum drift of 2%. The specimen of the CT2-20M-R series reached a value of 4.4 mm, on the negative side, in the third cycle of the 3.5% drift ratio. In the initial case, the specimen had shown only axial shortening behavior.

The strengthened specimens, with shear span $L_v=1450$, showed axial elongation throughout the experiment, with maximum values reaching 0.85 mm and 0.25 mm, for the CT1-15M-S and CT2-20M-S series respectively. It is worth noting that for the CT1-15M-S specimen series, this value was measured for the drift ratio of 2%, in the first session of the experiment. Therefore, it was shown that in the strengthening scenario, both columns increase in length, until the termination of the experiment; however, this elongation is very small as compared with the repaired counterparts

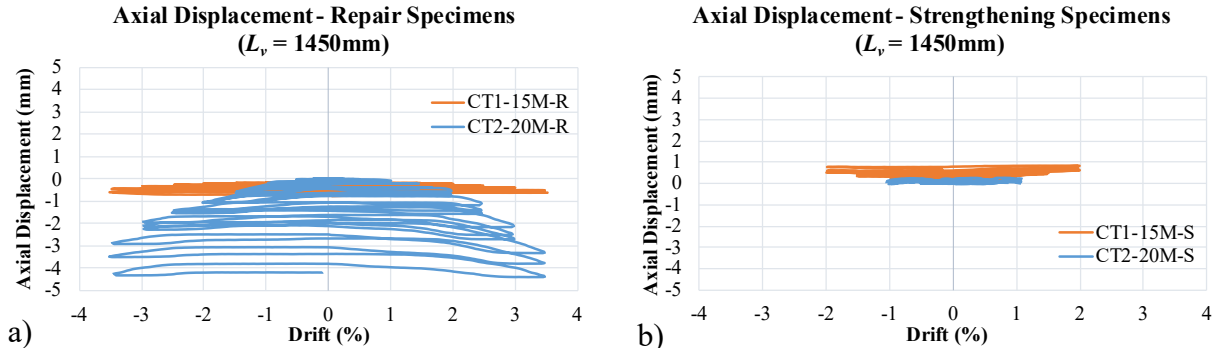


Figure 7.29. Axial Displacement – Drift curves for CT1-15M-R/S and CT2-20M-R/S specimens.

For the short shear span specimens, axial shortening was observed, with the CS3-15M-R and CS3-20M-R specimens series reaching up to 1.22 mm and 1.88 mm respectively. Compared to the initial columns, which reached corresponding values of 6.07mm and 2.86mm, the presence of the UHPFRC jacket substantially reduced the axial shortening of the columns. In both the UHPFRC-enhanced and non-retrofitted specimens, the columns of the short shear span geometry show reduction of length in the axial direction during the cyclic tests.

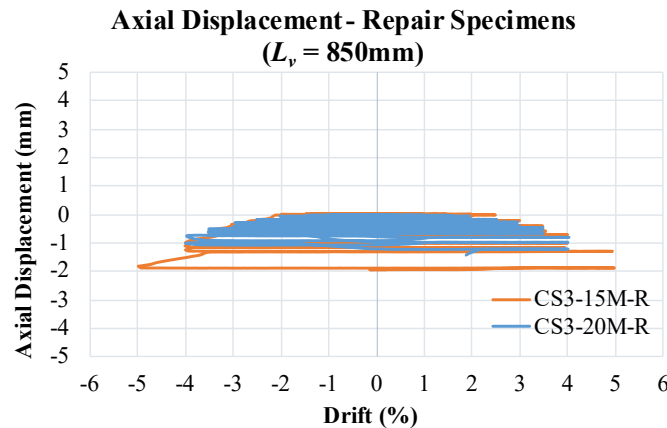


Figure 7.30. Axial Displacement – Drift curves for CS3-15M-R and CS3-20M-R specimens.

7.6 Summary

In this chapter, the second part of the structural scale experimental program was discussed, including the combined axial and cyclic lateral loading tests, of six NSC columns reinforced with thin UHPFRC jacket at the lap-splice locations. The tests were conducted using the same structural arrangement as in the first part of the experiment, utilizing the Strong Wall – Strong Floor

structural facility and two high-capacity hydraulic actuators. The specimens comprised the four columns included in the previous chapter, in addition to two column specimens having shear span ($L_v = 1450\text{mm}$). Two retrofitting scenarios were investigated: one where the pre-damaged specimens of the previous chapter were repaired using the UHPFRC jacket, to replace the critically damaged concrete at the column base, and one where two intact specimens were strengthened by adding the UHPFRC jacket around the lap-spliced zone, while preserving the undamaged NSC core. Mechanical tools were used to remove the disposable concrete, with additional openings created for the jacket to expand to the footing. Bar segments of the footing bars in the lap-spliced zone were covered to eliminate the bond with the added concrete and reduce the concentration of strains closer to the footing. Casting was performed on-site, using the commercial material Densit for the UHPFRC jackets, which were poured through custom plywood formworks, mechanically attached on the columns. External instrumentation included string potentiometers and remaining strain gauges from the previous experimental phase, and the arrangement used was identical to the pre-damage tests. The following observations were made from the analysis of the acquired data:

8. The presence of thin UHPFRC jackets in the repaired specimens significantly improved the overall performance the columns, contributing to both strength and deformation capacity. The load at peak was increased by 36% and 29% for the CT1-15M-R and CT2-20M-R specimens, respectively, while for the short shear span specimens, the strength was increased by 15% and 25%, for series CS3-15M-R and CS3-20M-R, respectively. In all cases of repair scenarios, the ductility of the UHPFRC-enhanced specimens exceeded that of the initial specimens, with the taller shear span columns reaching up to 3.5% drift ratio (owing to geometrical limitations of the testing frame) and the short shear span columns reaching up to 5%. Strength degradation beyond the 2% drift, which corresponded to the upper limit of the pre-damaging phase, was gradual and stable, with reduction in strength between successive drift increments not exceeding 10%. The reduction in strength between successive cycles of the same load increment were less than 5%.
9. In the strengthening scenario, the addition of the UHPFRC showed similar improvement in performance as in the repair case. In comparison with the non-retrofitted specimens, the strengthened specimens reached maximum loads that were 21% and 16% higher, for the CT1-15M-S and CT2-20M-S specimen series, respectively. The strengthened specimens were also tested up to the maximum drift ratio of 3.5%. Strength degradation was shown

to be gradual and stable, with reduction of strength between successive drift ratios up to 12%. Similar to the repaired specimens, the reduction in strength between successive cycles of the same load increment were less than 5%.

10. Failure was localized below the UHPFRC jacket, at the column-footing connection, in all specimens of both geometries and shear spans. The taller columns showed sparse flexural cracks throughout the jacket surface, while cracks also formed at the top end of the jacket. No evidence of shear cracking was observed or crushing and longitudinal cracking along the lap-splice bars. In the short shear span specimens, virtually no cracks appeared except for the bottom major crack at the column-footing connection.
11. Curvature distributions also showed similar patterns among the UHPFRC-retrofitted specimens, with the greater magnitudes of curvature located in the lower segments along the column height, in the proximity of the column-footing interface. In the shorter columns, the curvature distribution was altered from the one obtained in the non-retrofitted specimens, illustrating the elimination of the shear failure mode in both the CS3-15M-R and CS3-20M-R specimens.
12. Strain gauge measurements showed higher strains on the lower end of the lap-spliced connection and under the unbonded segment. Within the UHPFRC jacket length, none of the bars reached yielding, as per the measurements obtained in this experiment. Strains on the transverse reinforcement within the jacket reached up to 1500me.
13. The fraction of the rotation capacity of the columns that was owing to shear deformation was drastically reduced in the case of the UHPFRC-retrofitted columns. Reductions among all cases were on average 66%, while for drift ratios exceeding 2% the shear distortions in the retrofitted specimens showed no further increase. It was concluded that the presence of the UHPFRC jacket virtually eliminated the development of shear distortions in the retrofitted columns.
14. Of the taller specimens, all cases showed axial elongation throughout the tests, with the exception of specimen CT1-15M-R. The values of the axial deformation did not exceed 5 mm, with specimen CT2-20M-R reaching the higher value at the maximum drift of 3.5%. Of the short shear span specimens, both cases were shortened, even at drift ratios reaching the value of 5%, but did not exceed 2 mm, among the two specimens.

8 Numerical Analysis of UHPFRC-retrofitted Columns using FEA Modeling

In this chapter, the experimental responses are investigated further using advanced nonlinear finite element simulation, with the objective to obtain insights regarding the actual behavior of the retrofitting scheme. The chapter begins with a detailed description of modelling the intact and the UHPFRC-retrofitted column specimens. The specimens are modeled in the commercial software package ATENA, in combination with the general-purpose modeler GiD[®]. For the simulation of concrete, solid three-dimensional elements were used, with the properties of the UHPFRC material model being based on experimental values. The reinforcement was modeled with one-dimensional (truss) elements and the bond properties were accounted for using contact springs throughout the reinforcement. Various parameters were considered in the analysis, including the column cross-section size, the thickness of the UHPFRC jacket, the depth of the jacket-footing interface and the presence of unbonded segments. Results are compared with the experimental values obtained from the previous chapters.

8.1 Material Models

For the concrete elements, the “SOLID Concrete” material option was selected, using the models with codenames *CC3DNonLinCementitious2* and *CC3DNonLinCementitious2SHCC*, for NSC and UHPFRC, respectively. These materials were applied directly to the volume elements representing the concrete. For the reinforcement, the material model “1D Reinforcement” was used, which was applied to the line elements representing the bars. Lastly, the steel loading plates were modeled using the “SOLID Steel” option, with the 3DSteelVonMises option selected.

8.1.1 Concrete: *CC3DNonLinCementitious2*

For conventional concrete, the mechanical properties were defined based on the compressive strength of the cylinder tests given in Table 6.3. The modulus of elasticity and tensile strength were defined based on Eq. 8.1 and 8.2, below and Poisson’s ratio was set to the value of 0.2.

$$E = 4500\sqrt{f'_c} \quad (8.1)$$

$$f_t = 0.1 \cdot f'_c \quad (8.2)$$

An example of the compressive and tensile functions used by the model is shown in Fig. 8.1. The ascending branch on the compression side follows a curved path until the peak stress f'_c , while softening is represented by a linear branch until complete loss of strength. On the tension side, the ascending branch is assumed to be linear elastic, until the tensile strength f_t , where the softening branch commences (idealized by the corresponding points on the curve).

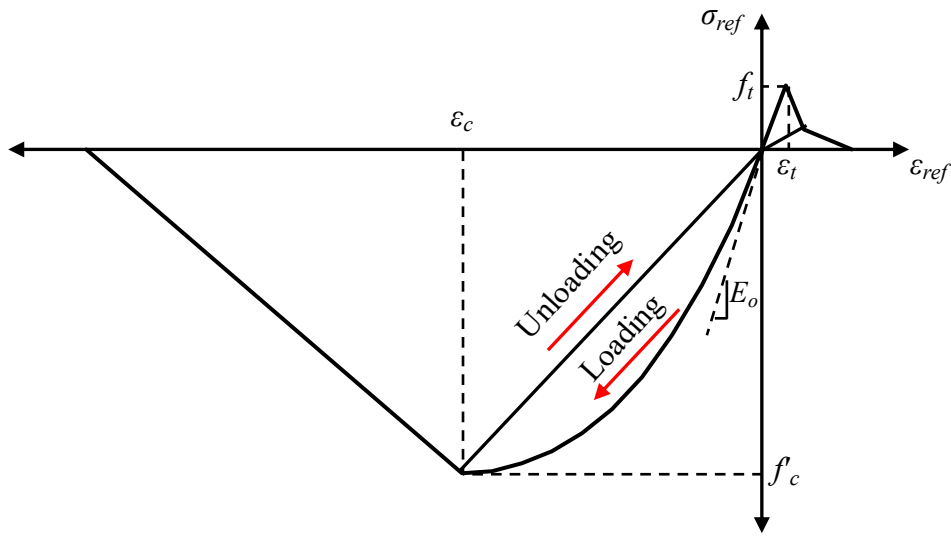


Figure 8.1. Compression and tension qualitative functions for concrete model (ATENA Program Documentation 2019).

In the “Tensile” properties tab, the fracture energy (G_f) was calculated based on Eq. 8.3, as shown in Model Code 2010:

$$G_f = 73 \cdot (f'_c)^{0.18} \quad (8.3)$$

For the analysis of the pre-damage tests, the fixed crack factor was set to the value of 0.7, which enables the rotating crack option to take over, after the post-peak reduction of the stress to 70% of the peak. Aggregate interlock was enabled, with the maximum size of the aggregate set to 0.015m. The unloading factor was also enabled, with the value of 0.1; this controls the reduction of unloading stiffness in the hysteretic cycles relative to the elastic value, and the amount of residual compressive strain (unloading path shown in Fig. 8.1). In the “Compressive” properties tab, the

default properties were used, and the crush band minimum value was activated, at 0.05m. All other material model properties were maintained at default values.

8.1.2 UHPFRC: *CC3DNonLinCementitious2SHCC*

For the UHPFRC material, a variation of the material model that was originally developed for conventional concrete case was used; in this modified version, the model is adjusted for the presence of fiber reinforcement. In addition, this model enables the definition of the post-fracturing and plastic constitutive relationships, with user-defined stress-strain values.

The mechanical properties were defined based on the cylinder tests described in Chapter 7, with the compressive strengths of Table 7.4. Similar to the conventional concrete model, the modulus of Elasticity and tensile strength were defined by Eqs. 8.1 and 8.2. Poisson's ratio was set to the value of 0.2.

In the "Tensile" properties tab, the tensile function, in the post-elastic range, the coordinates were defined as shown in Table 8.1, which are based on the work shown in Chapter 3. The tension characteristic size was set to be 0.05m and the tension localization onset was selected to be 0; the former parameter was selected to be equal with the finite element size and the latter denotes the strain where the softening branch commences (the value of zero corresponds to the end of the ascending, or hardening, branches). Aggregate interlock was not used in this model on account of the absence of coarse aggregates. The unloading factor was also activated, with the value of 0.1. In the "Compressive" properties tab, the compression, in the post-elastic range, the coordinates of the curve are the ones shown in Table 3.4, of Chapter 3. The compression characteristic size was selected to be 0.05m and the compression localization onset was set to 0, as described above.

Table 8.1. Tension function for UHPFRC material model.

Coordinate ID	ε_f	σ_t/f_t
1	0	1
2	0.0005	0.125
3	0.3	0.0

For the fiber reinforcement, the volume fraction value was set to 0.02 since this was used in the actual tests. The fiber modulus of Elasticity and diameter were 200GPa and 0.2mm, respectively. The cross-section factor was selected to be 0.9.

8.1.3 Reinforcement: *CCReinforcement*

For the embedded reinforcement used during the experiments of Chapter 6 and 7, bar segments from the 15M and 20M bars were tested at the MTS Universal Testing Machine, to obtain the material properties (one segment for each, due to absence of bars). The tests were performed by placing the bar segment between the machine anchors and attaching an MTS extensometer on the coupon tested. For this extensometer model, the gauge length was 50mm. The resulting constitutive stress – strain laws, obtained for each type of bar are shown in Fig. 8.2, whereas a multilinear material law was used for the model input (as shown in the same figure). A similar multilinear material model was assumed for the stirrups, as the material were provided from the same quarry. In all cases, the modulus of Elasticity was 200GPa.

The bar elements were defined based on the topological and geometrical properties of the reinforcement. Bar properties such as cyclic parameters and bond stress – slip relationships are discussed in the following sections. To account for model symmetry, bars that were placed on the plane of symmetry were reduced to half the bar profile by changing the corresponding factor to 0.5.

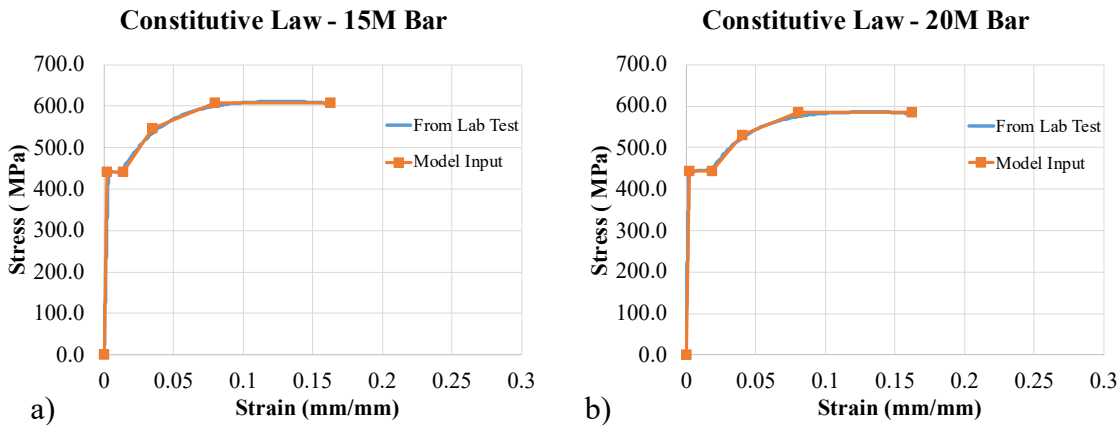


Figure 8.2. Non-linear functions for HPFRC: a) Tension; b) Compression.

8.1.3.1 Cyclic Reinforcement

To consider the effects of the cyclic loading regime, the model subcategory *CCCyclingReinforcement* was activated, which utilizes the Menegotto (1973) model for the constitutive properties of reinforcement. In the “Menegotto-Pinto” tab, for the column specimens described in Chapter 6, the Bauschinger factor R_o was set to the value of 20, and factors c_1 and c_2

were selected to be 0.925 and 0.15, respectively – these are the default optimal values of the parameters as defined by the ATENA program manual (ATENA Program Documentation 2019). For the column specimens described in Chapter 7, factors R_o , c_1 , and c_2 were altered to values 22, 0.8 and 0.08, respectively; the selection of these values was based on the work of Carreno et al. (2019). The above factors determine the curvature of the unloading and reloading paths defined by parameter R , as per Eq. 8.4. An example of the influence of this parameter is shown in Fig. 8.3, where the default values are the ones provided by the ATENA recommendations, described above.

$$R = R_o - \frac{c_1 \cdot \xi}{c_2 + \xi} \quad (8.4)$$

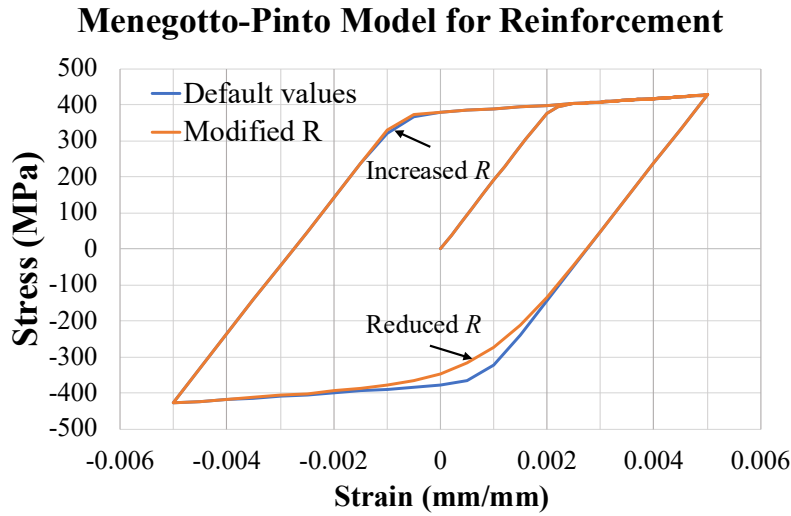


Figure 8.3. Influence of curvature parameter R for the Menegotto-Pinto model for cyclic reinforcement.

8.1.3.2 Bar With Bond

For the longitudinal reinforcement of the columns, the model subcategory “Bar With Memory Bond” was activated, which enables the introduction of bond springs along the embedded bars; bond stress – slip relationships including the unloading path are specified in this model. These were defined based on the guidelines of Model Code 2010, as follows: for the bars within the shear span of the column specimen, the curves based on Splitting Failure type, with good bond conditions and accounting for the presence of stirrups were considered; for the bars within the footing, the curves based on Pullout Failure, with good bond conditions and presence of stirrups, were considered. An example of the bond stress – slip curves for the concrete strength of $f'_c = 25.5$

MPa is shown in Fig. 8.4, whereas the bond strength values for the various column specimens are listed in Table 8.2.

To account for the presence of the UHPFRC jacket, the location dependency parameter was activated, which enables modification of the bond strength of the embedded reinforcement at different locations of the bar. Within the UHPFRC jacket height, the bond strength was modified was set equal to $1.3\sqrt{f_{c,UHPC}}$, as per the findings of the DTP bond tests shown in Chapter 3 of this study. In addition, at the locations of the bond breakers, as shown in Chapter 7, the bond strength modifier was set to 0, to account for the unbonded segment. The bond strength values in the presence of the UHPFRC jacket, for the various column specimens, are listed in Table 8.3. To account for the cyclic loading, the unloading/reloading paths defined by the Bond with Memory model were activated, as shown in Fig. 8.5. In this model, the unloading path follows a vertical drop until the bond stress value equal to $\tau = \tau_1$, where τ_1 is the bond stress on the residual branch of the constitutive bond stress – slip law.

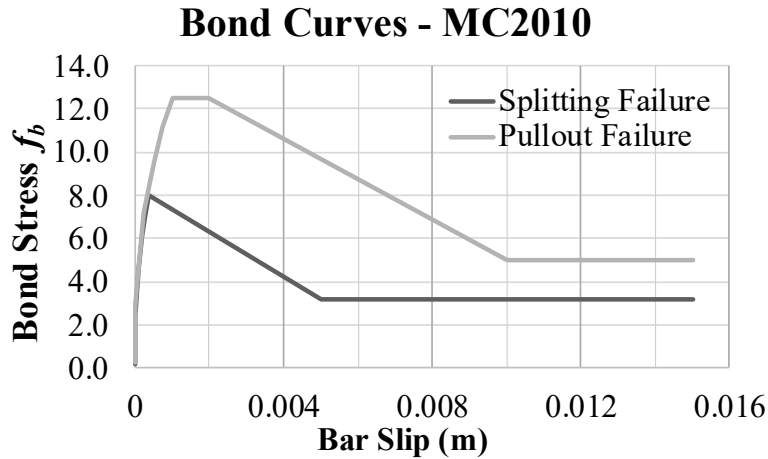


Figure 8.4. Constitutive bond stress – slip relationships for splitting and pullout failures.

Lastly, to consider the presence of the hooks in the footings and cap-beam of the specimens, the options “FIXED START” and “FIXED END” were selected, for the corresponding points of the bars.

Table 8.2. Bond strength values for FEA models.

Specimen Series	$f_{b,max}$ – Splitting (MPa)	$f_{b,max}$ – Pullout (MPa)	$f_{b,max}$ – UHPFRC (MPa)
CT1-15M-R/S	7.6	11.3	17.1
CT2-20M-R/S	8.04	12.62	16.9
CS3-15M-R & CS3-20M-R	8.18	13.1	15.5 & 16.1

Bond with Memory Model

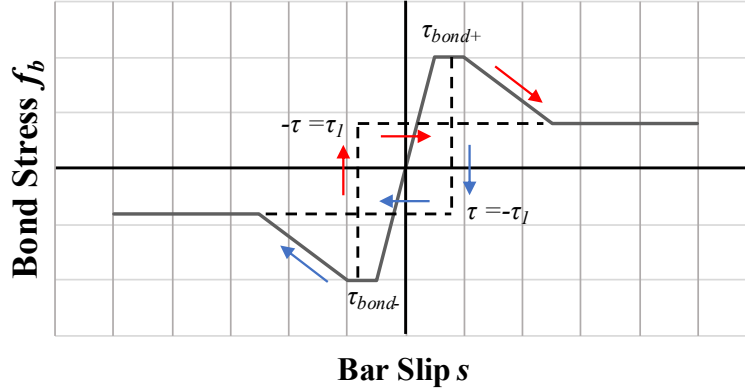


Figure 8.5. Bond with Memory model for cyclic loading on reinforcement (ATENA Program Documentation 2019).

8.1.4 Steel: *3DSteelVonMises*

For the supporting steel plates, a bilinear material law was used, with the yield strength and modulus of Elasticity to be 800MPa and 200GPa, respectively. Poisson's ratio was selected to be 0.3. The remaining parameters were maintained at default values.

8.2 Geometry of the Model

The column models were initially generated in the GiD[®] modeler. The coordinates of the geometrical entities were defined in accordance with the models shown in the companion study conducted by El-Joukhadar (2022). The columns were modelled at half their original thickness, to account for symmetry. All concrete and loading plate elements were modelled with 3D solid elements, whereas the reinforcement was modelled with 1D truss elements, using bond-links. A schematic of the modelling interface between element blocks is shown in Fig. 8.6a.

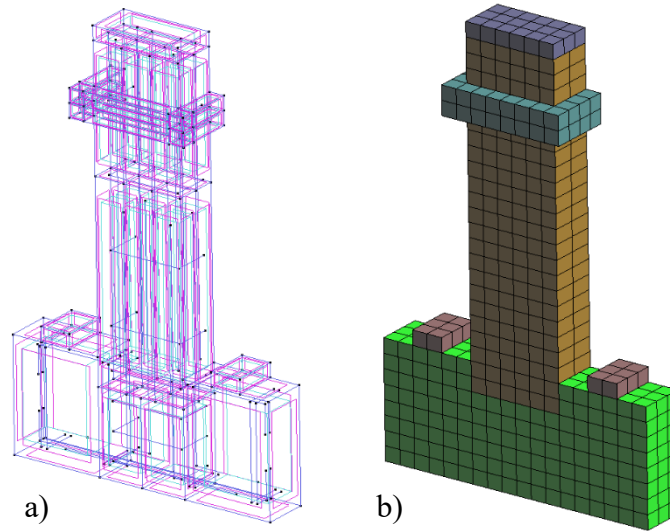


Figure 8.6. Schematic of C-15M column: (a) Definition of Element blocks; (b) Overall Geometry of the Mesh.

The models were meshed with hexahedral elements, of 0.05 m size. This size was selected as an optimum value for accuracy and computational load. Meshing of the reinforcement elements was performed automatically by the program. An overview of the mesh is shown in Fig. 8.6b. This modelling approach has been calibrated through comparison of benchmark tests by Tsiotsias and Pantazopoulou (2021) and by Dameh and Pantazopoulou (2022); successful correlation is established by comparing the calculated and experimental load displacement hysteretic responses of the modelled specimens.

8.2.1 Interval Properties

To apply the loading sequence and boundary conditions, the model was subjected to separate intervals for the axial loading and the subsequent lateral displacement cyclic reversals. All intervals contained the same support conditions and restrictions due to symmetry. The first interval contained the application of the axial load, whereas each subsequent interval contained 1/4 of the lateral displacement cycles, which were applied until all cycles were completed.

8.3 Results

The results of the analyses are obtained by application of monitor points for force and displacement at the loading point. Force-displacement curves are compared with the ones obtained from the experimental program and the longitudinal strains on the concrete and reinforcement are discussed.

The force-displacement responses of the models, in comparison with the responses obtained from the experimental program, are illustrated in Figs. 8.7-8.12, whereas the cracking distribution and longitudinal strains on the surfaces of the models are shown in Fig. 8.13-8.18. To account for P-Delta effects, the lateral resistance obtained from the finite element models was adjusted, as shown in Eq. 8.5. This equation corrects the lateral resistance by adding the term $P \cdot \theta$, where P is the axial load and θ is the corresponding drift ratio. For the column models, the axial load was simulated with a constant external force acting on the cap beam, in the Z (vertical) direction; the correction accounts for the second order moment generated by the constant force parallel to the column axis.

$$V_{res} = M/L_v + P \cdot \theta \quad (8.5)$$

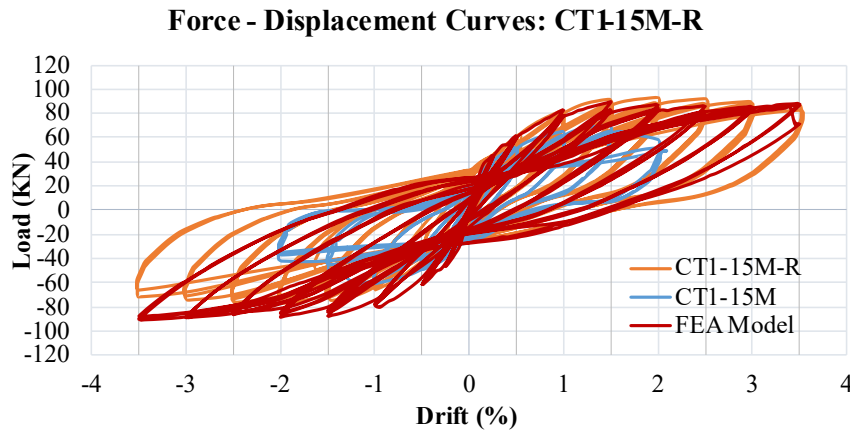


Figure 8.7. Force – Displacement curves for CT1-15M-R model.

It was observed that, in all cases, the models showed similar responses to the experimental curves. Both the models of the UHPFRC-repaired and -strengthened specimens show similar responses with the ones obtained experimentally, with the differences between peak lateral loads never exceeding 10%. From Figs. 8.13 and 8.14, it was seen that in both the repaired and strengthened specimens, flexural cracks develop in the shear span, at peak load, whereas further cracking activity is shown in the footing. Throughout the jacketed section, higher strains are observed in the proximity of the column-footing interface, whereas mild horizontal cracking developed in the remainder of the jacket length. Flexural cracks were also shown at the end of the jacketed section, within the shear span, which was also observed in the experimental response. At the end of the analysis, both column models showed significant cracking in the proximity of the column-footing interface and on the edges of the footing around the shear span. Overall, the cracking patterns in

the models were similar with the experimental responses, with localization occurring at the column-footing interface and sparse flexural cracking on the shear span.

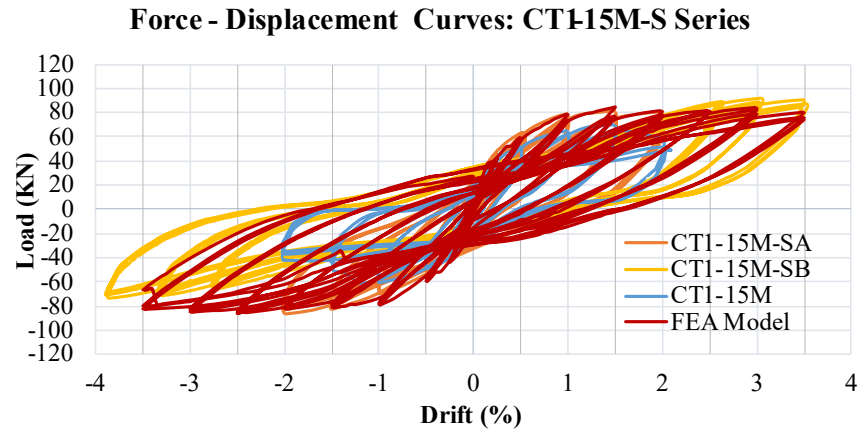


Figure 8.8. Force – Displacement curves for CT1-15M-S model.

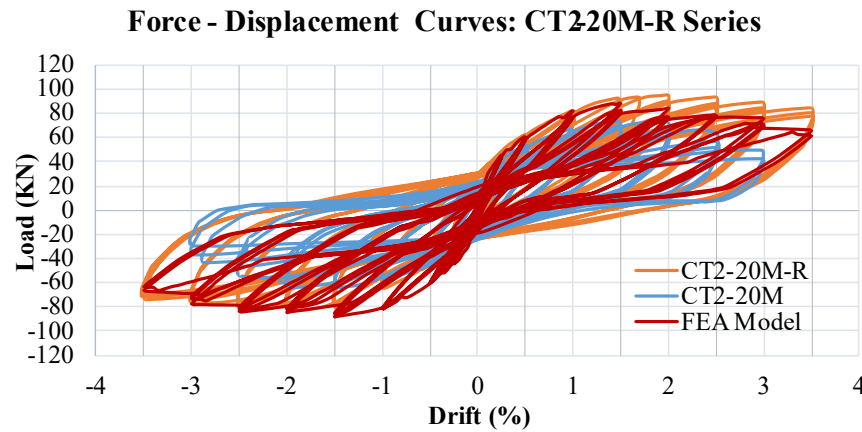


Figure 8.9. Force – Displacement curves for CT2-20M-R model.

From Figs. 8.15 and 8.16, the distributions of longitudinal strains for the repaired and strengthened specimens of the CT2-20M-R/S series are shown. For these specimens, flexural cracks appeared throughout the shear span, as it was also shown in the experimental response, while cracking was more pronounced in the proximity of the column-footing interface. Within the jacket length, flexural cracks were observed closer to the footing, whereas further horizontal (flexural) cracks appeared in the shear span, above the jacketed section. At the end of the analysis, the cracking was more pronounced at the bottom end of the shear span, with significant cracking at the column-footing interface and on the footing sides, around the shear span length. The response and cracking

patterns of these specimens was very similar to the CT1-15M-R/S series, which is in accordance with the experimental response.

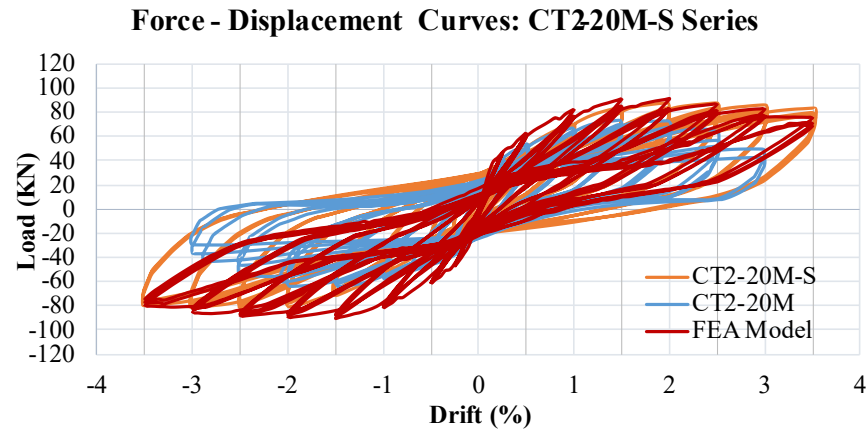


Figure 8.10. Force – Displacement curves for CT2-20M-S model.

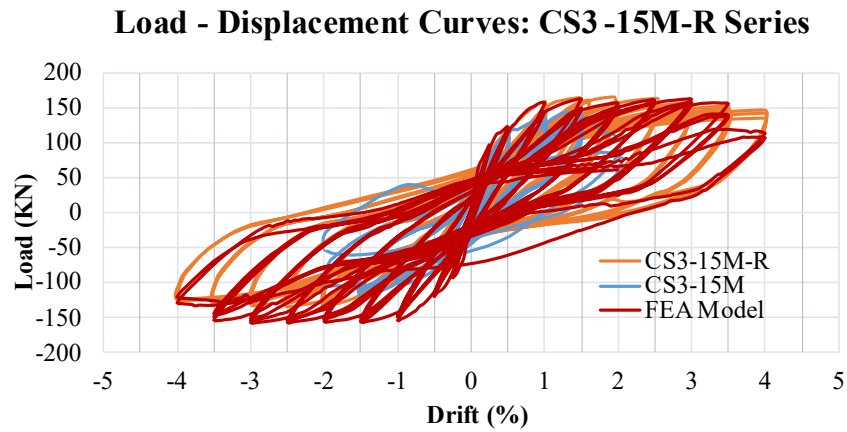


Figure 8.11. Force – Displacement curves for CS3-15M-R model.

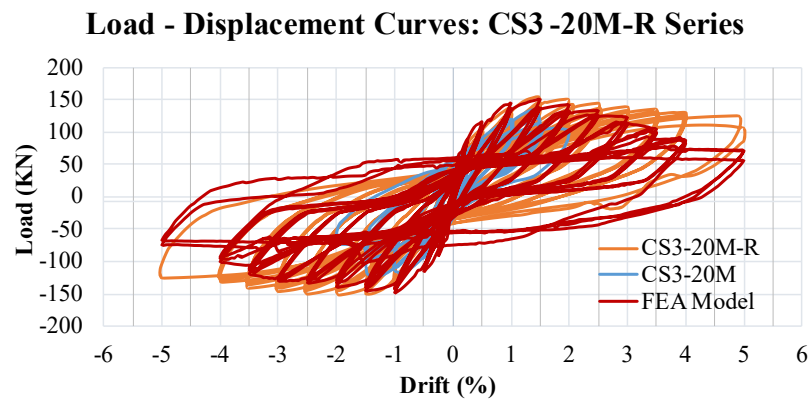


Figure 8.12. Force – Displacement curves for CS3-20M-R model.

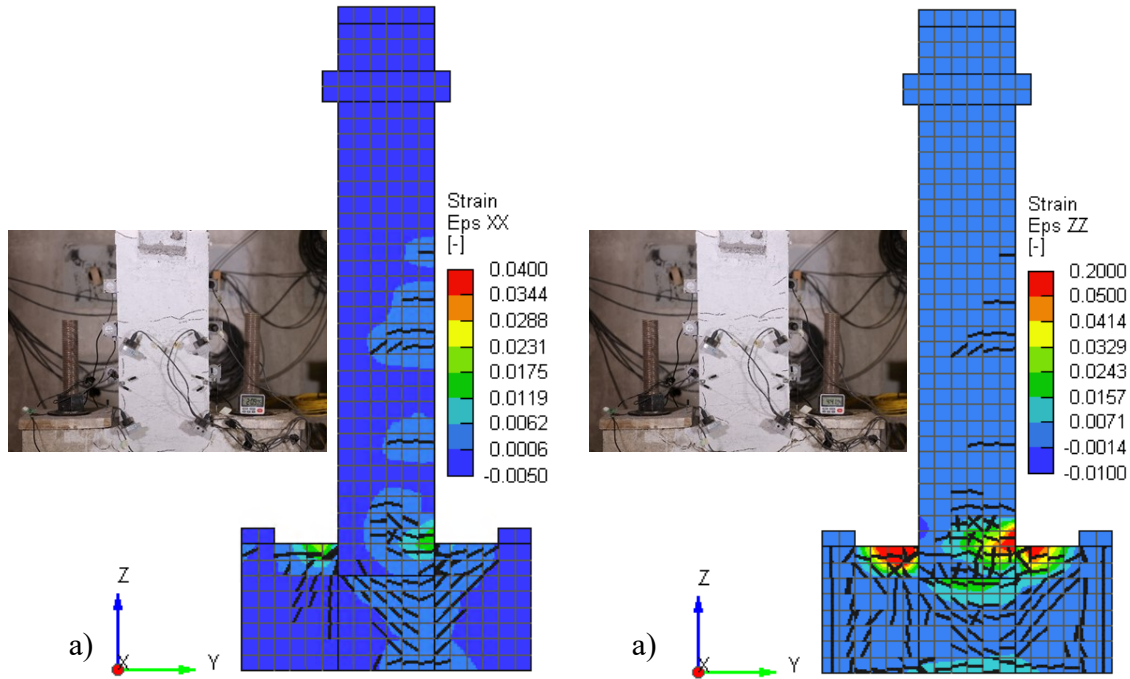


Figure 8.13. Distribution of longitudinal strains for column CT1-15M-R: (a) Drift 1%, at peak; (b) Drift 3.5%, at end.

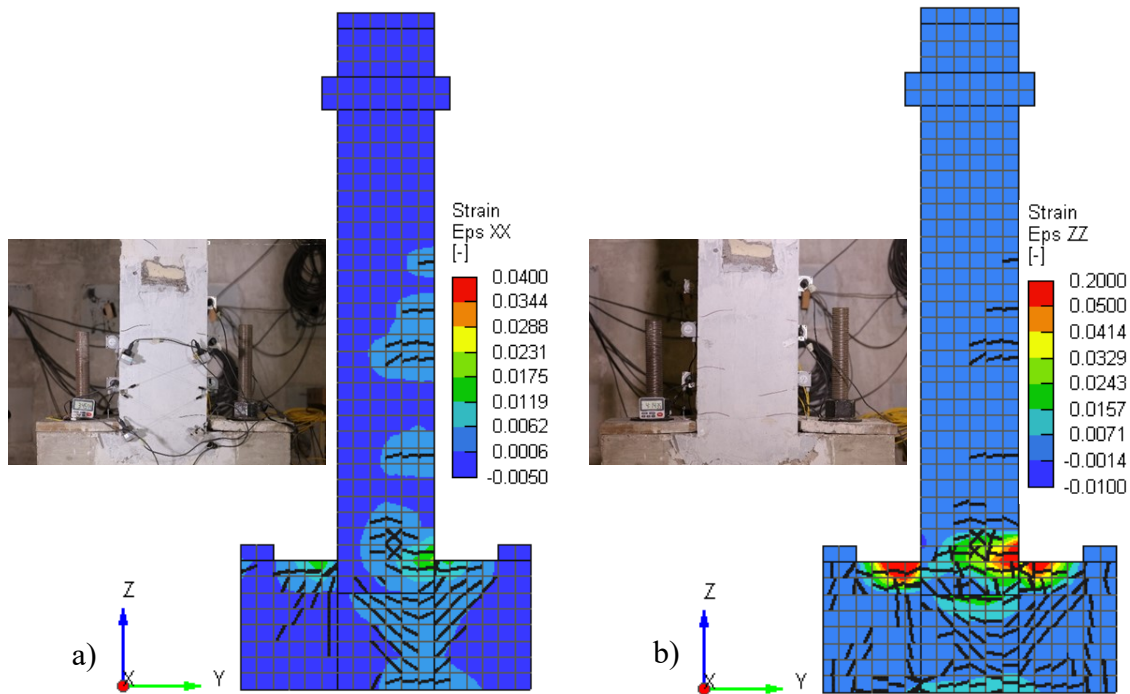


Figure 8.14. Distribution of longitudinal strains for column CT1-15M-S: (a) Drift 1%, at peak; (b) Drift 3.5%, at end.

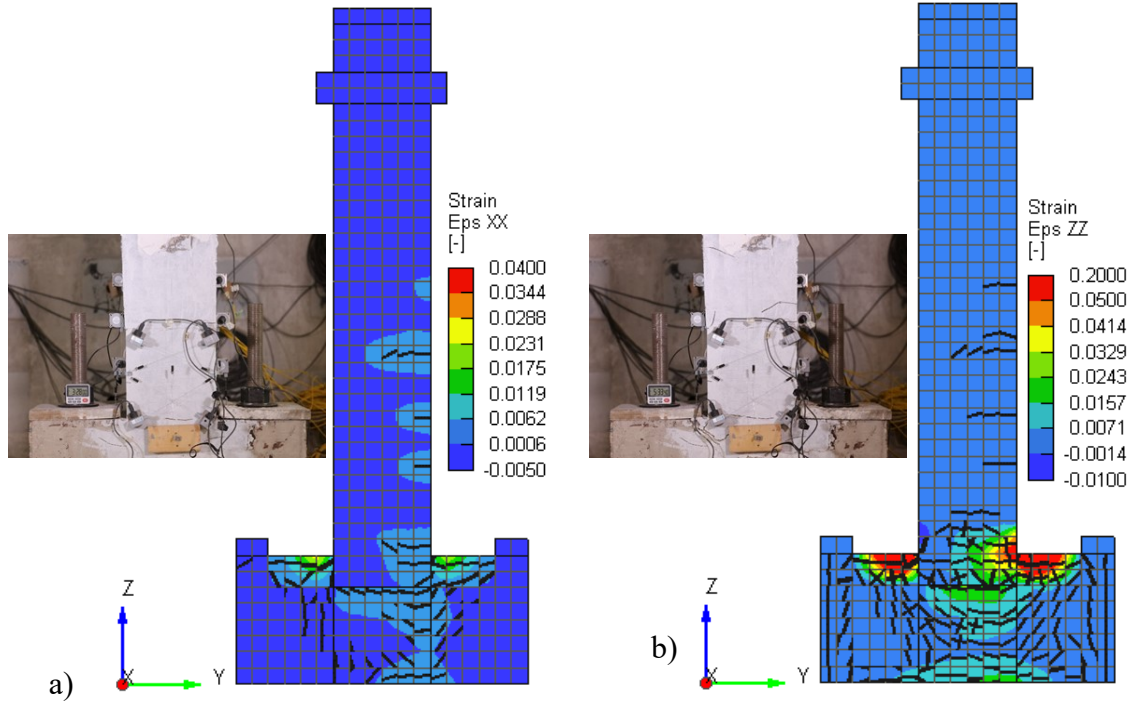


Figure 8.15. Distribution of longitudinal strains for column CT2-20M-R: (a) Drift 1%, at peak; (b) Drift 3.5%, at end.

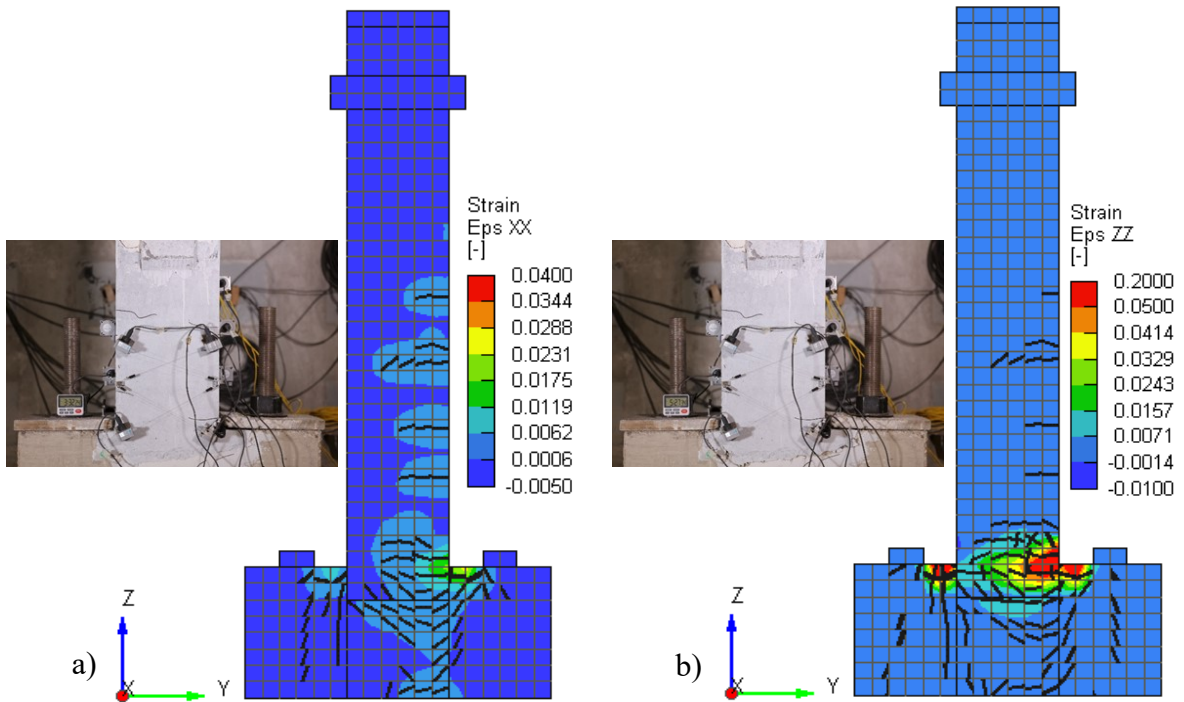


Figure 8.16. Distribution of longitudinal strains for column CT2-20M-S: (a) Drift 1%, at peak; (b) Drift 3.5%, at end.

From Figs. 8.17 and 8.18, the distributions of longitudinal strains for the repaired specimens of the CS3-15M-R and CS3-20M-R series are shown. In these specimens, the flexural cracks are less pronounced. Higher longitudinal strains can be observed in the proximity of the column-footing interface, within the jacketed section, whereas cracking was less pronounced in the shear span beyond the jacket length. At the end of the analysis, cracking was more pronounced at the column-footing interface and at the footing sides, around the shear span.

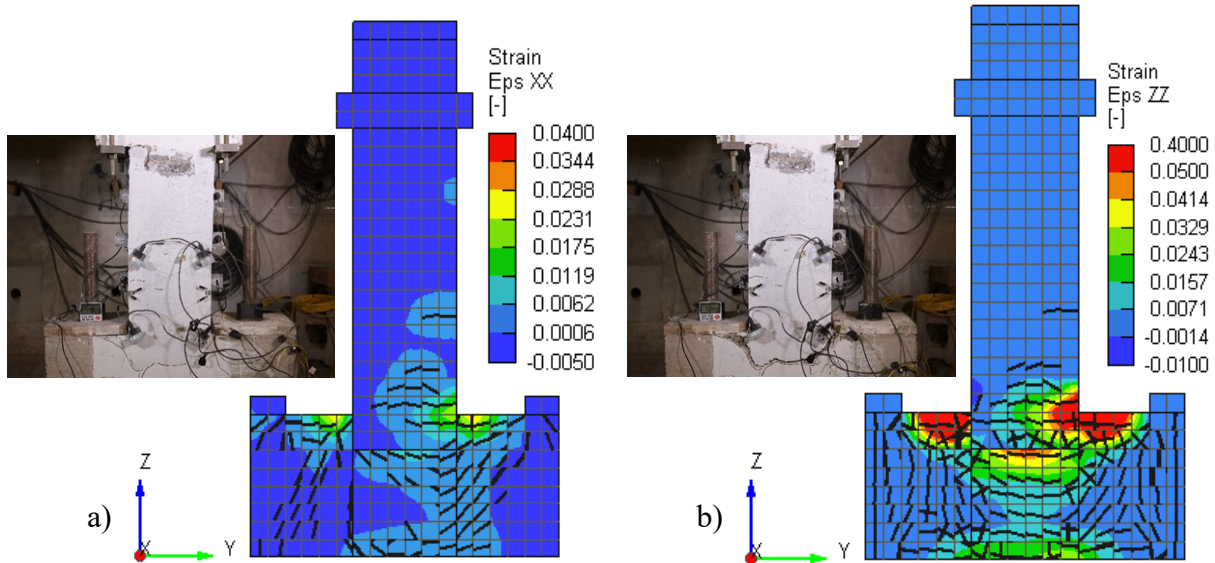


Figure 8.17. Distribution of longitudinal strains for column CS3-15M-R: (a) Drift 1%, at peak; (b) Drift 4%, at end.

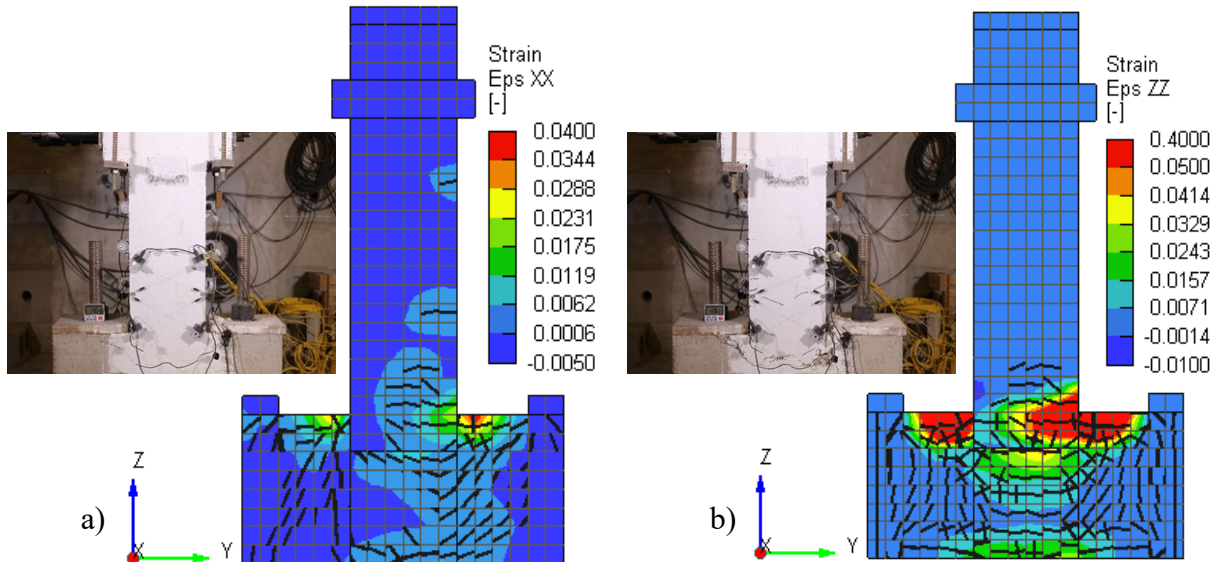


Figure 8.18. Distribution of longitudinal strains for column CS3-20M-R: (a) Drift 1%, at peak; (b) Drift 5%, at end.

8.3.1 Reinforcement strains

In this section, the reinforcement strains of all the specimen series are shown. Comparisons between the original and retrofitted specimens are made and the results are discussed.

8.3.1.1 CT1-15M-R/S Series

From Figs. 8.19-8.21, the longitudinal strains of the reinforcement for the original and retrofitted specimens of series CT1-15M are shown. In the original column (Fig. 8.19), it can be observed that up to the 0.5% drift, the reinforcement remains below yielding limits. For drifts from 1% and higher, the starter bars in the lap-splice connection have exceeded the yielding strain of 0.002, at the location of the column-footing interface. The length of the post-yield region was approximately 110mm and remained virtually the same for larger drift ratios. As the lateral displacement increases, the longitudinal strains were accumulated at the location of the column-footing interface, whereas the bars within the shear span showed reduced contribution to the lateral response; no stirrup was yielding (which underscores the fact that bond degradation probably occurs too early leading to reduced resistance in the column. At this point, it was shown that the plastic hinge has formed at this location.

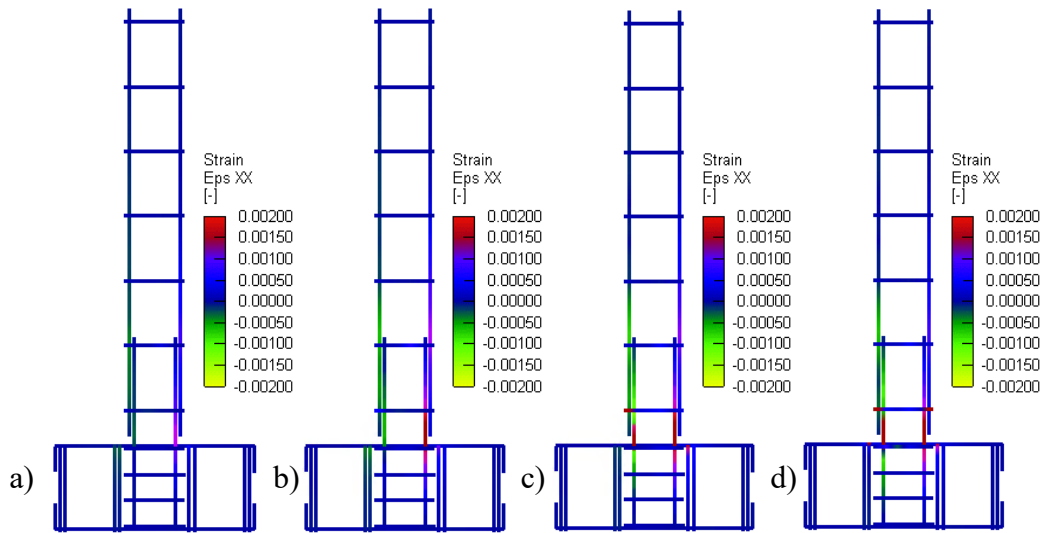


Figure 8.19. Distribution of longitudinal strains of reinforcement for column CT1-15M: a) 0.5%; b) 1%; c) 1.5%; d) 2.0%.

For the repaired CT1-15M-R (Figs. 8.20-8.21), it is observed that at 1% drift, the greater portion of the shear span longitudinal reinforcement is engaged in strains even in compression. While the

bars within the jacketed section, at the column-footing interface, have reached or exceeded the yielding strain of 0.002, at a length of approximately 110mm, the main bars develop longitudinal strains of approximately 0.0015, beyond the length of the jacket. For larger drift levels, up to magnitude of 2%, it is observed that a greater portion of the starter and main bars have developed strains greater than 0.0015, while the length of the yielded portion on the starter bars, at 2% drift, was increased to 240mm. At this point, a small region of the main bars has reached yielding, at a length of 50mm, which is located approximately 600mm away from the column-footing interface. It was also observed that the footing stirrups and longitudinal reinforcement also develop longitudinal strains that exceed the yielding limit. Lastly, at the termination of the analysis, at a drift ratio of 3.5%, the starter bars and footing bars have exceeded the yielding strain, whereas the strain on the shear span longitudinal bars remained at the same strain magnitudes as previously. The length of the yielded region on the starter bars increased to approximately 320mm. At this point, the failure was localized at the column-footing interface, with both the longitudinal and stirrups bars of the footing being activated. It should be noted that, even at the 3.5% drift, the columns maintained substantial flexural resistance, as was observed in the experimental response curves as well.

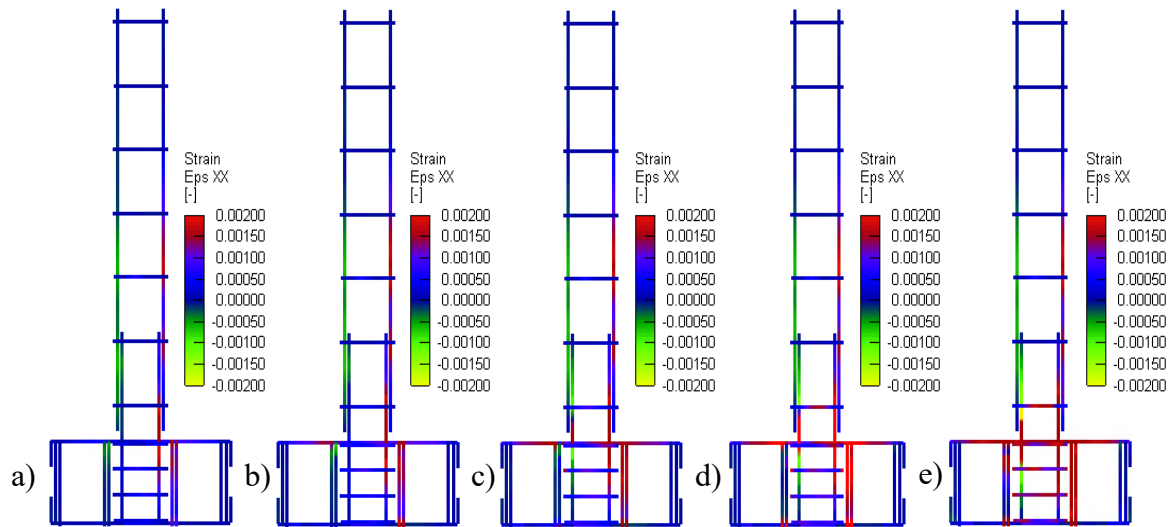


Figure 8.20. Distribution of longitudinal strains of reinforcement for column CT1-15M-R: a) 1.0%; b) 1.5%; c) 2.0%; d) 2.5%; E) 3.5%.

For the strengthened CT1-15M-S column, the longitudinal strain distributions were similar to the repaired specimen case, however, the lengths of the post-yield regions on the reinforcement were substantially reduced. At 1% drift, the start bars exceeded the yielding strain for a length of 60mm,

within the footing, starting at a distance of 50mm. At 2% drift, the main bars attained yielding over a small length of 60mm, located at approximately 600mm from the column-footing interface. Finally, at the end of the analysis, at 3.5% drift, the majority of the starter and main bar regions are below the yielding limits. This shows that in the strengthened case, the reinforcement sustained overall lower magnitudes of longitudinal strain, even through the pattern of strain distribution was similar to the repaired specimen case.

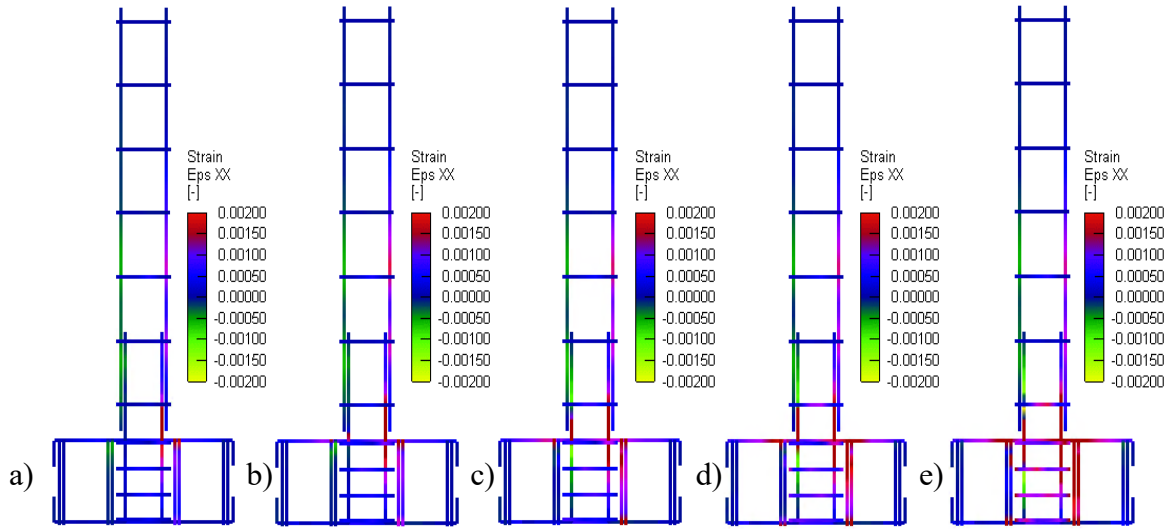


Figure 8.21. Distribution of longitudinal strains of reinforcement for column CT1-15M-S: a) 1.0%; b) 1.5%; c) 2.0%; d) 2.5%; E) 3.5%.

8.3.1.2 CT2-20M-R/S Series

The axial strains of the reinforcement for the original and retrofitted column specimens of the CT2-20M series are shown in Figs. 8.22-8.24. In the original case (Fig. 8.22), it is observed that the starter bars, at the column-footing connection, reach the yielding limit at 1% lateral drift, over length of approximately 110mm. For larger magnitudes of lateral displacement, at a 2% drift ratio, the yielded region increased to a length of 140mm, illustrating the formation of the plastic hinge at the column-footing connection.

For the retrofitted columns (Figs. 8.23 and 8.24), it was seen that in the repaired column, the starter bars in the column-footing connection reach longitudinal strain values up to 0.0015, at 1% lateral drift ratio, whereas the starter bars of the strengthened column reached yielding over region of 140mm, at the same location. For greater magnitudes of the lateral displacement, up to 2.0% drift, it is shown that the main bars were activated to a larger extend (approximately half the shear span

height), indicating the greater participation of the reinforcement at locations away from the column-footing connection. It can be also seen that, in both the repaired and strengthened columns the starter bars yielded over a region of approximately 250mm. Lastly, towards the end of the analysis, at the 3.5% drift, greater values of the longitudinal strain are shown in the footing reinforcement and stirrups, where the bars have exceeded the yielding limit; this was observed on both the repaired and the strengthened columns, however, the extent of yielding in the footing bars was less in the case of the strengthened specimen. At the same time, the starter bars on the compression side attained yielding over a length of 130mm and the main bars, showed reduced values of the longitudinal strain, to approximately 0.001; this was true for both the repaired and strengthened cases. In this stage, the longitudinal strain had localized in the footing and dowel bars.

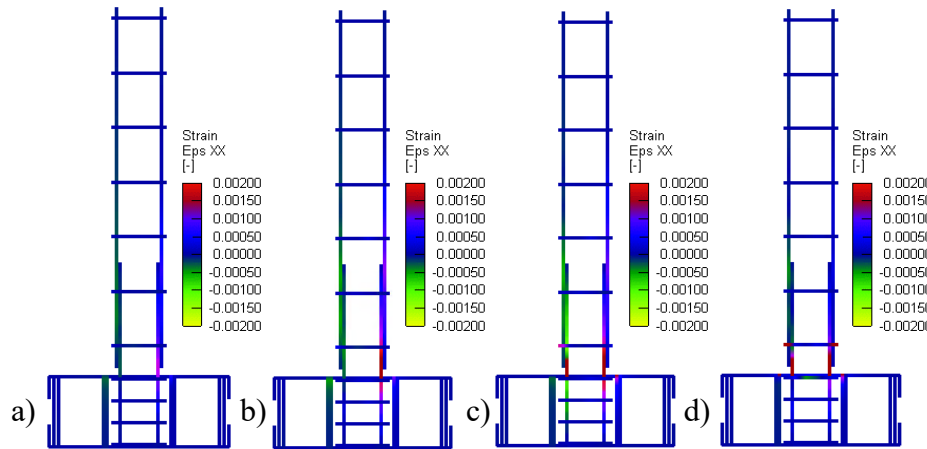


Figure 8.22. Distribution of longitudinal strains of reinforcement for column CT2-20M: a) 0.5%; b) 1%; c) 1.5%; d) 2.0%.

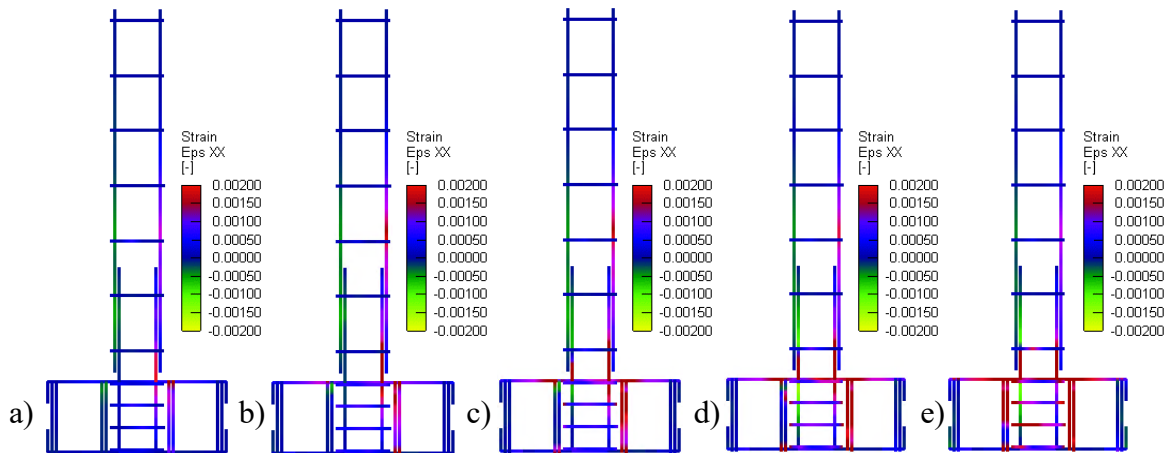


Figure 8.23. Distribution of longitudinal strains of reinforcement for column CT2-20M-R: a) 1.0%; b) 1.5%; c) 2.0%; d) 2.5%; E) 3.5%.

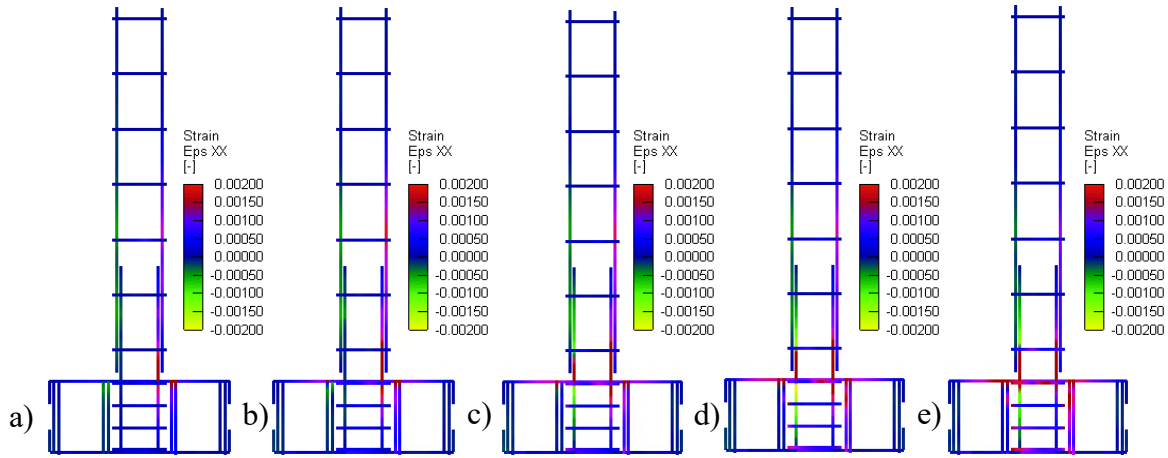


Figure 8.24. Distribution of longitudinal strains of reinforcement for column CT2-20M-S: a) 1.0%; b) 1.5%; c) 2.0%; d) 2.5%; E) 3.5%.

8.3.1.3 CS3-15M-R & CS3-20M-S Series

The axial strains on the reinforcement for the original and retrofitted column specimens of the CS3-15M-R series are illustrated in Figs. 8.25-8.26. In the original case (Fig. 8.25), it was found that the starter bars reached the yielding strain 0.002, at 1% drift, over a length of 100mm, at the column footing interface. At this stage, the lap-splicing extension bars had not developed strains greater than 0.001. For greater lateral displacements, up to 2% drift ratio, the longitudinal strains localized at the starter bars, in the proximity of the column-footing connection, while the stirrups within the lap-splicing zone had developed strains greater than 0.0015. At this stage, the plastic hinge had formed at the column-footing connection.

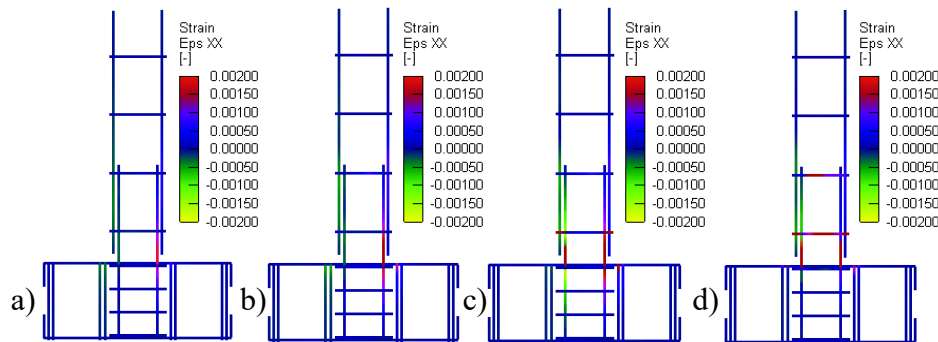


Figure 8.25. Distribution of longitudinal strains of reinforcement for column CS3-15M: a) 0.5%; b) 1%; c) 1.5%; d) 2.0%.

In the retrofitted case (Fig. 8.26), it was seen that at 1% drift, the starter bars had exceeded the yielding limit over length of 190mm. It was also observed that a greater portion of the bars had exceeded the longitudinal strain of 0.0015, in comparison to the original column, while the footing bars and stirrups had also exceeded the yielding limit. For greater lateral displacements, up to 2.0% drift, the yielding strain was exceeded in a greater portion of the footing bars, while strains exceeding 0.002 had developed in the starter bars, over a length of 260mm. At this stage, the extension bars developed longitudinal strains up to 0.001. At the end of the analysis, at 4% drift, the majority of the footing bars, including the longitudinal reinforcement and stirrups, had exceeded the yielding strain, as well as the starter bars over the lap-length in the shear span, on the tension and compression sides; the length of the post-yield region on the starter bars was approximately 280mm. For this column, greater damage was observed in the footing.

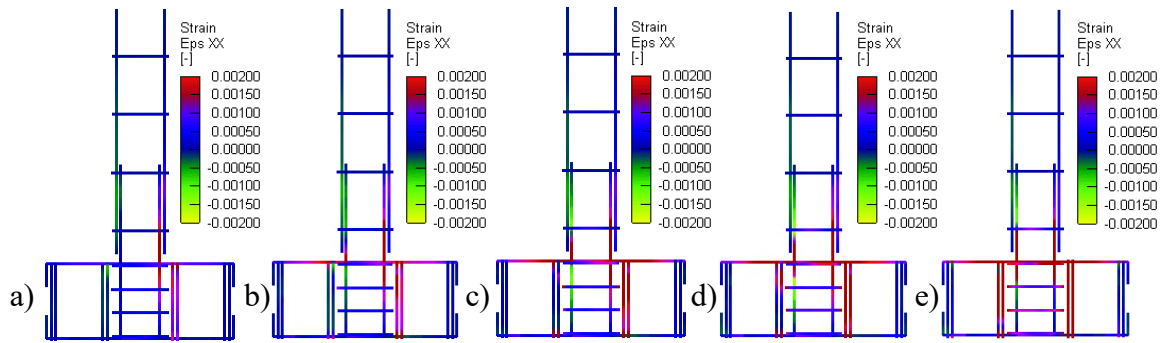


Figure 8.26. Distribution of longitudinal strains of reinforcement for column CS3-15M-R: a) 1.0%; b) 1.5%; c) 2.0%; d) 2.5%; E) 4.0%.

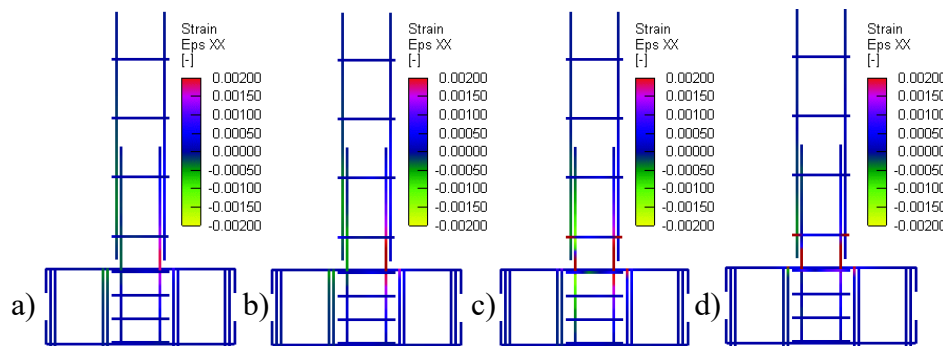


Figure 8.27. Distribution of longitudinal strains of reinforcement for column CS3-20M: a) 0.5%; b) 1%; c) 1.5%; d) 2.0%.

Axial strains for all the bars of the series CS3-20M (both initial and retrofitted) are plotted in Fig. 8.27-8.28. In the initial case (Fig. 8.27), it can be seen that the starter bars reached the yielding strain of 0.002, at 1% drift, over a length of 110mm. For larger lateral displacements, up to 2%

drift, the longitudinal strains were localized on the starter bars, on the column-footing connection; the inelastic strains were distributed over a length of 130mm.

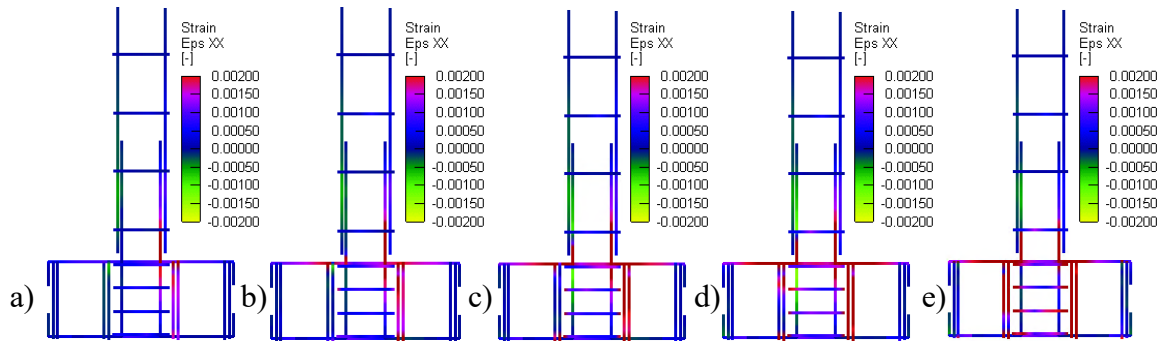


Figure 8.28. Distribution of longitudinal strains of reinforcement for column CS3-20M-R: a) 1.0%; b) 1.5%; c) 2.0%; d) 2.5%; E) 5.0%.

In the retrofitted case (Fig. 8.28), at 1% drift, the longitudinal strain in the dowel bars had exceeded the yielding limit, over a length of 140mm, whereas the strain in the footing bars, had reached values up to 0.0015. For larger lateral displacements, up to 2.0% drift ratio, a greater portion of the starter bars had exceeded the yielding strain of 0.002, over a length of 240mm. At this stage, the footing bars, including longitudinal reinforcement and stirrups, had exceeded the yielding limit. At the end of the analysis, at 5% drift, a greater portion of the footing bars had reached yielding, whereas, on the starter bars, the region where the longitudinal strain had exceeded the yielding limit was reduced to a length of 180mm, on both the tension and compression sides. No stirrup yielding was observed in the column shear span.

8.4 Summary

Results of detailed FEA of the original and UHPFRC-retrofitted column specimens of Chapters 6 and 7 are presented. The models were assembled using the combined GiD/ATENA Studio software package, using 3D elements for the concrete and 1D elements for the reinforcement. The concrete materials were based on a fracture-plasticity model, while for the UHPFRC the same model was used with custom non-linear functions for tension and compression. Explicit consideration of the bond conditions using the Model Code 2010 provisions were used to model the interaction of the reinforcement with the surrounding concrete cover. Loading and boundary conditions were applied such that the exact experimental response was simulated.

1. The UHPFRC-retrofitted column models adequately simulated the response obtained from the experimental program. It was shown that significant damage developed under the jacketed section and on the footing sides, around the shear span column, where a sliding “cradle” was formed and governed the failure of the specimens; this was observed in the experimental response. Flexural cracks within the shear span were sparse, whereas no sign of shear failure was observed. At the end of the analyses, greater damage was observed on the footing and localization of longitudinal reinforcement strains at the column-footing interface.
2. The distribution of the longitudinal strains showed greater activation of the main reinforcement, particularly above the UHPFRC-jacketed sections, showing that the presence of the jacket improves the performance of the embedded reinforcement. This observation was made only for the large shear span ($L_v = 1450$ mm) columns. However, in all cases, shear failure was suppressed entirely, whereas both repaired and strengthened columns with identical details showed no differentiation in their response. This suggests that the shear strength provided by the jacket sufficed to resist the increased demands resulting from the enhanced flexural resistance of the columns.
3. Reinforcement yielding of the footing bars and corresponding damage was observed in all UHPFRC-retrofitted models. However, the column models showed substantial increase in lateral resistance and ductility, as was observed in the experiments. That was because with the replacement of cover after retrofitting, the entire layer of UHPFRC material participates in sectional equilibrium, by developing significant tensile and compressive forces over the entire area – whereas in the conventional cover concrete case, the region of the cover in tension would be neglected and would not contribute to the equilibrium of sectional forces.
4. Based on the above results, it can be concluded that the cyclic performance of UHPFRC-retrofitted columns can be adequately simulated using the combined GiD/ATENA Studio platform. The assumptions of the model include the bond stress – slip constitutive properties; here the Model Code 2010 modelling approach was used. Adjustment of the bond strength for reinforcement embedded in the UHPFRC jacket was sufficient to reproduce the desired effects. Consideration of the location dependency of the bond properties included the unbonded segments on the starter reinforcement.

9 Conclusions

The goal of this study was to evaluate the cyclic performance of UHPFRC material for application in seismic retrofitting of conventional concrete columns in the form of cover replacing jackets. A series of experimental investigations were conducted, including a) small-scale experiments with UHPFRC-confined concrete cores subjected to cyclic axial loading, and b) large-scale experiments on UHPFRC-retrofitted reinforced concrete columns, designed based on detailing preceding modern seismic codes and tested under combined axial and cyclic lateral load. The contribution of the UHPFRC material on the bond behavior of embedded reinforcement was thoroughly investigated by assembling a database of experiments from the literature and conducting detailed FEA on the various bond test setups. The critical outcomes of this study included: i) the verification of the effectiveness of UHPFRC jackets, in the repair and strengthening repair methodologies, where both approaches resulted in significant improvement of the seismic performance of impaired and damaged RC columns, ii) the quantification of the bond strength between reinforcement and HPFRC matrices, which was confirmed through extensive investigation of previous experimental studies and computational simulation, and iii) the determination of the constitutive properties of UHPFRC-confined concrete under cyclic compression, where the contribution to deformation capacity, imparted from the UHPFRC jacket, was illustrated. In more detail, the outcomes of this study were the following:

1. Based on the bond test database, a pattern of overestimation of the measured bond strength was observed in some experimental studies, depending on the presence of unaccounted for lateral stresses on the bar generated spuriously by test boundary conditions. Conventional test setups such as the Standard Pullout (SP) or Beam Test (BE) showed contribution of the confining stresses and flexural response of the test bar on the obtained bond strength. Test setups that eliminate, or reduce, the magnitude of compressive lateral stresses around the reinforcement, such as the Direct Tension Pullout (DTP), Contact Lap-Splice DTP (CLP), and Non-Contact Lap-Splice DTP (NCLP), resulted in the lowest bounds of bond strength measurements.

2. The database revealed the same sensitivity of UHPFRC bond test specimens to parameters such as test setup, anchorage length and cover thickness. It was shown from the database that, at the benchmark anchorage length value of $5D_b$, the average value for local bond strength from all specimens was $1.8\sqrt{f_c}$ whereas the corresponding magnitude in the case of DTP was only $1.3\sqrt{f_c}$. The same pattern was observed in the case of the ECC materials, where the average value from the SP and BE tests was $1.55\sqrt{f_c}$, whereas for the DTP test, this value was $0.7\sqrt{f_c}$. This discrepancy in measured local bond strength was attributed to the different boundary conditions between the various test setups and sensitivity of bond behavior to spurious confining stresses.
3. Experimental findings on the bond behavior of reinforcement embedded in UHPFRC, conducted in previous studies, using both DTP (Tsotsias 2019) and anchored four-point beam (AFPB) specimens (Saikali 2019), were compared to obtain the values of bond strength. Both tests correlate with the lower bound for short anchorage lengths ($5D_b$), and two cover thicknesses of $1D_b$ and $2D_b$, respectively, where the local bond can be averaged without significant error across the anchorage length. The two tests resulted in comparable values, with the former and latter being in the range of $1.3\sqrt{f_c}$ and $1.45\sqrt{f_c}$, respectively, and across all materials. The AFBP specimens showed higher tensile resilience, particularly in the post-peak range, compared to the DTP specimens, which was attributed to the curvature of the beam at large levels of deflection. This is one of the most critical findings obtained in this study.
4. The parametric FEA analysis of the various bond test setups, included in the database, investigated the effect of spurious stresses generated by boundary conditions on the specimen model. The specimen models included were for the SP, BE, CLP, NCLP, restrained NCLP (R-NCLP), and DTP tests. Models that were restrained laterally, due to the presence of restrictive boundary conditions, such as the SP and R-NCLP, resulted in the least conservative bond strength values. The BE test was also shown to be less conservative than the CLP, NCLP and DTP tests, which was attributed to the contribution of the flexural effects and arching action of the compressive strut that may confine the bonded bar perimeter over the support point. Based on the relationship between lateral stress and bond stress, it was shown from the FEA simulations that the CLP test showed the least sensitivity to the change of anchorage length and cover thickness, whereas the

NCLP test showed more sensitivity to the cover thickness increase. The DTP test was, on average, on the lowest bound of bond strength values for all cases. It is noted here that owing to the very high value of f_c' , the bond value calculated from this lower bound is significant – suggesting that anchorage lengths as low as $8D_b$ suffice to yield standard GR 60 (400 MPa) bars. (Note that $L_b/D_b = (1/4) \cdot (f_y/1.3\sqrt{f_c'}) = 7.6$ for $f_c' = 100$ MPa).

5. Based on experimental testing of NSC cores confined with UHPFRC jackets, the compressive performance of the encased concrete showed performance of passively confined concrete, with moderate improvements in strength but with significant enhancement of ductility. For monotonic axial loading, applied directly to the inner cores, the stress and strain at peak for cylindrical specimens increased by 30% and 100%, for jacket thicknesses at $t_j = 25$ mm and $t_j = 37.5$ mm, respectively. In the case of the prismatic section, the improvements in stress and strain at peak, for the jacket thickness of $t_j = 25$ mm were 25% and 153%, respectively. As such, the mechanical performance of NSC was significantly improved in the presence of the UHPFRC jacket.
6. When subjected to cyclic loading, the UHPFRC confined cores showed substantial improvement in ductility, compared to the unconfined NSC cores, under the same history of loading. The contribution of the UHPFRC jackets enabled the cylindrical confined specimens to reach 4.4 and 6.1 times the strains of the unconfined NSC cores, for jacket thicknesses $t_j = 25$ mm and $t_j = 37.5$ mm, respectively. The compressive strength was also improved, however, at lower magnitudes compared to the monotonic type of loading, while for the prismatic type sections, the contributions in strength and ductility were of similar values as with the cylindrical specimens. Despite the reduced contribution to the stress and strain at peak, it was shown that, in the presence of the UHPFRC jackets, deformation in the confined NSC cores was ductile at the application of cyclic loading. This outcome highlights the significance of UHPFRC in the improvement of the seismic performance of conventional concrete for retrofitting and rehabilitation scenarios.
7. Expansion measurements showed that the total circumferential expansion of the cylindrical UHPFRC-confined cores was reduced, albeit slightly, in comparison to the unconfined NSC specimens. While for monotonic loading, this observation was made for both jacket thicknesses of $t_j = 25$ mm and $t_j = 37.5$ mm, in the case of cyclic loading, the total measured expansion at peak for confined specimens exceeded the one obtained from the unconfined

specimen. For the prismatic section specimens, it was shown that, based on digital image correlation (DIC) analysis of the outer jacket surface, the total measured crack width for the jacket with $t_j = 37.5$ mm was reduced, compared to the specimen with $t_j = 25$ mm. Therefore, the increase of the jacket thickness showed reduction of the lateral deformations on the jacket surface.

8. Application of a combined analytical model, based on the works of Thorenfeldt et al. (1987) and Darwin and Pecknold (1974) sufficiently captures the axial stress – axial strain response of the monotonic and cyclically loaded UHPFRC-confined NSC cores. The model was based on the common points and turning points curves definitions to determine the coordinates of the unloading/reloading hysteretic cycles for unloading from / reloading towards the monotonic envelope curve. Parameters for the various envelope and cycle branches were defined using the data from the corresponding experiments. Pertinent equations compatible with both the cylindrical and prismatic section types were considered.
9. An experimental program comprising column specimens with conventional concrete having a uniaxial compressive strength $f_c' = 25\text{MPa}$ and mild reinforcement detailing was conducted. The specimens contained lap-spliced connections at the column-footing interface, and they were designed to be tested under combined constant axial and cyclic lateral loading. The damaged specimens were then repaired by cover replacement using thin UHPFRC jackets, while a number of intact specimens were also strengthened using the same method, and all specimens were tested with the same type of loading. Comparison of unretrofitted and retrofitted column specimens showed that the presence of the UHPFRC jacket improved the flexural performance of the columns, both in strength and ductility. The specimens prior to retrofitting, having old-type detailing that represented construction practices prior to the 1980's showed significant loss of strength and severe concrete damage at the plastic-hinge region, reaching drifts of up to 2%, and 3% for one specimen only. The retrofitted specimens, however, were tested with drifts up to 3.5%, and up to 5% for the short aspect ratio specimens, showing significantly increased lateral strength, reduced strength degradation and a considerably more ductile response.
10. In the initial, non-retrofitted specimens, failure was characterized by flexural cracking in the taller specimens and splitting along the lap splice lengths, followed by cover

delamination and crushing of the concrete at the latter stages of applied lateral drift. The longitudinal cover splitting indicated bond failure of the lap-spliced connection. In the shorter shear span specimens, severe shear cracking was observed, as well as damage to the inner concrete cores, at the end of the experiments. After application of the UHPFRC jacket by replacement of cover and any fragments of the core, flexural cracking was minimized, with sparse horizontal cracks developing on the jacket and on the concrete column above the jacketed section. Shear failure was eliminated, despite the sparsely placed transverse reinforcement, while failure was transferred under the jacketed section, at the column-footing interface. No longitudinal splitting over the lap-splice was observed during the tests thanks to the improved bond performance of the reinforcement within the UHPFRC jacket. Both the repaired and strengthened specimens showed comparable responses in the cyclic tests, with similar types of failure in all cases.

11. Overall, the outcomes of the cyclic tests of the UHPFRC-retrofitted columns demonstrate the efficiency and practicality of UHPFRC cover replacement for the improvement of seismic performance of deficient and damaged columns. All retrofitted specimens showed significant strength enhancement, almost double the ductility of the non-retrofitted specimens, as well as gradual strength deterioration and reduced concrete damage. Bond failure phenomena, such as longitudinal splitting over the shear span and cover delamination, were eliminated, whereas the improved bond strength between the UHPFRC and reinforcement reduced the effect of strain penetration within the starter bars of the lap splice, even for the repaired specimens. As such, this material can provide very effective retrofit solutions that are based on confinement on grounds of its significant tensile resilience to seismic rehabilitation of older structures.
12. UHPFRC-retrofitted columns were modeled using detailed 3D-FEA in the combined GiD/ATENA platform. Results from the analyses show adequate replication of the experimental responses, particularly at the force – lateral displacement curves. Failure phenomena were captured, with the retrofitted cases showing displacement of the failure region beyond the jacketed section, at the jacket interface with conventional concrete in the footing. Longitudinal strain profiles reveal that the starter bars reached yielding during the unretrofitted column specimen testing, while at the retrofitted column specimens,

yielding of the bars was found both for the starter bars of the lap-spliced connection and the footing reinforcement.

13. Based on the results of this study, it becomes possible to estimate the contribution of a thin UHPFRC jacket, as a Cover Replacement strategy, for the seismic retrofitting of conventional RC columns. A schematic for this calculation is shown in Fig. 9.1a. The confining stresses imparted by the UHPFRC jacket were obtained through equilibrium with the core confining stress σ_{lat} with the jacket tensile stress, as described by Eq. 9.1, where b_{core} is the width of the confined core concrete and the cover (thus the jacket thickness) was assumed to be 35mm. For the geometry of the column specimens included in this study the value of σ_{lat} was approximately 2.4MPa.

$$\sigma_{lat} = \frac{2 \cdot t_j \cdot f_t}{b_{core}} \quad (9.1)$$

14. Following the above, the shear resistance V_j of the UHPFRC jacket was also provided as shown in Eq. 9.2, which was based on the pertinent work by Ioannou et al. (2021), who described the shear resistance imparted by ECC jackets on flexural members. For the specimens of this study, the angle of the shear sliding plane with the longitudinal was estimated to be 40°, as per the works of Pardalopoulos and Pantazopoulou (2019), to account for the constant axial load of N (Fig. 9.1b). As such, the value of V_j was approximately 126 kN, which consists of a significant contribution to the total shear resistance of the RC columns specimens; note that the shear resistance of the column specimens in pristine condition was calculated to be up to 108 kN.

$$V_j = 2 \cdot t_j \cdot f_t \cdot (h - t_j) \cdot \tan\theta \quad (9.2)$$

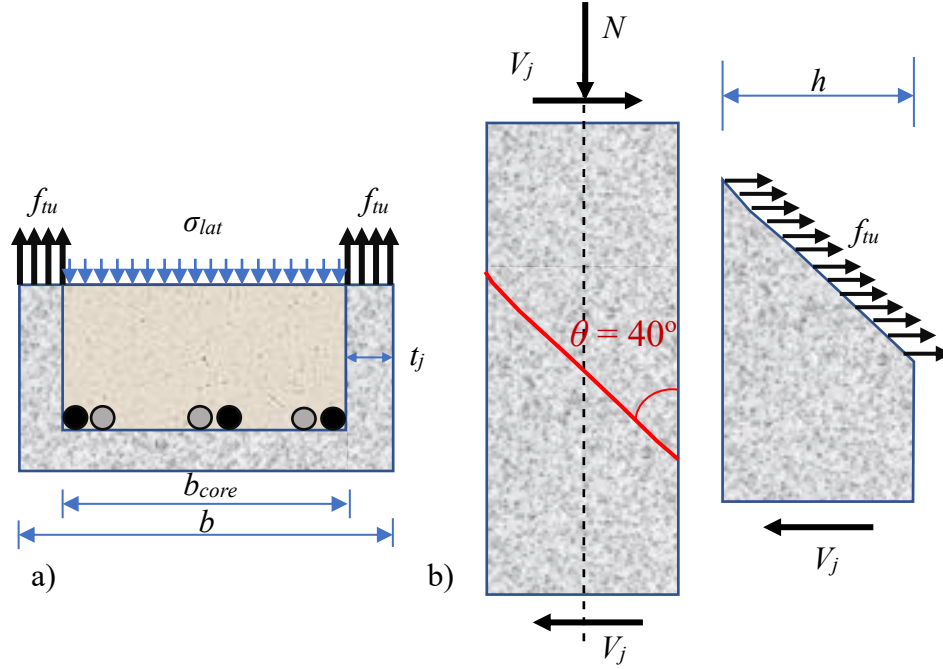


Figure 9.1. a) Confining stresses imparted by the UHPFRC jacket; b) Contribution of UHPFRC jacket to shear resistance of axially loaded column.

15. Lap splices encased in the jacketing cover develop enhanced bond strength which is estimated based on the model depicted in Eq. 9.3, proposed by Megalooikonomou et al. (2018). In this model, the value of λ is determined based on the direction of the governing splitting crack (shown schematically in Fig. 9.2), μ_{fr} is the coefficient of friction, f_{ct} is the tensile strength of the jacket, and N_b is the number of intercepted longitudinal bars in the spliced region on the splitting crack path. Based on the geometries utilized in this study, the maximum bond stress of the UHPFRC-confined lap-splices was approximately 17.4MPa and 14MPa, for sections containing 15M and 20M longitudinal reinforcement, respectively, in the arrangement depicted in Fig. 9.2.

$$f_b^{max} = \frac{2 \cdot \mu_{fr}}{\pi \cdot D_b} \cdot (\lambda \cdot t_j \cdot f_{ct} + 0.33 \cdot A_{st} \cdot f_{y,st} / (N_b \cdot S)) \quad (9.3)$$

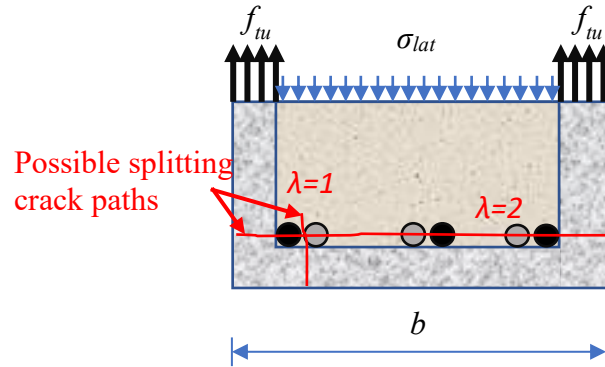


Figure 9.2. Possible splitting crack paths with corresponding number of intercepted longitudinal bars.

9.2 Recommendations for Future Work

The present research may be extended further, in order to address some of the topics of interest that emerged from the investigations conducted in this study, as follows:

1. The assembled database on bond test setups contains experimental evidence from the literature, on tests conducted using UHPFRC and ECC materials up to the date of publication of the current study. Additional tests added after the time of publication of this study should be considered in future studies. In addition, distinction between crucial parameters contributing to bond behavior is recommended, such as bar diameter, reinforcement rib profile and corrosion condition of the bar surface. Lastly, classification of the cementitious materials is recommended according to strength and tension hardening/softening types, based on newly established code definitions (e.g. see Annex 8 of CSA-S6 (2019), which will improve the accessibility of the database in the scientific community.
2. Extensive experimental investigation on bond of reinforcement in UHPFRC, considering alternative test setups in parallel (same batch), is recommended. In particular, further tests using the DTP and AFPB tests, as well as spliced pullout setups such as the CLP and NCLP tests, should be conducted, considering additional parameters such as varying anchorage lengths, different fiber orientations by means of casting direction, cover thicknesses and bar profiles. This is necessary in order to obtain more tangible evidence regarding the parametric sensitivity of bond behavior of reinforcement in UHPFRC materials, with specimens fabricated under identical laboratory conditions.

3. The effect of bond under cyclic loading conditions comprises a pivotal topic for seismic design using UHPFRC, for which the experimental evidence is limited. Pertinent bond tests, following the test setup configurations discussed above are thus recommended, which consist of the next step of the work presented in this study.
4. Further experimental tests, such as the ones conducted in this study, comprising cores of NSC confined with UHPFRC-jackets, and tested under monotonic and cyclic load, are recommended. These tests should include additional parameters, namely, the geometries of the NSC cores (i.e., dimensions and shape), jacket material types, jacket thicknesses and cyclic loading protocols. Different sizes of the inner cores should be considered to account for size effects in the material strength. Jacket thicknesses should include larger thickness, such as $t_j = 50\text{mm}$, to obtain further experimental evidence on the confinement effectiveness of the UHPFRC jackets. The protocols of cyclic loading should include unloading/reloading loops at varying increments (i.e., 0.25mm), as well as varying number of cycles per loop (i.e., 3 hysteresis loops at each unloading point) and pattern of load increase per step.
5. Modification of the experimental setup mentioned above that includes embedded reinforcement within the UHPFRC jacket, is recommended. Due to the prominence of UHPFRC in applications for retrofit and seismic rehabilitation of existing structures, the experimental investigation of the compressive behavior of the confined cores, in the concurrent presence of reinforcement, is essential towards the objective to define the constitutive properties of the material. As such, experimental tests considering the effect of reinforcement detailing are proposed as a priority in further documentation of the retrofit methodology.
6. Based on the above recommendations for the cyclic compressive testing of UHPFRC-confined NSC cores, further application of the analytical model proposed in this study should be performed using future experimental data. The model should be updated to the newest experimental parameters, such as larger jacket thicknesses, core material strength, jacket material type, as well as cyclic loading protocols, to improve accuracy and compatibility to a greater range of experimental cases.
7. Structural-scale tests involving the application of UHPFRC in the repair and strengthening of older components are critical to obtain tangible evidence of the potential of the material

in seismic design. As such, further retrofitting scenarios and structural member types are strongly recommended for such tests. Such cases include: i) the replacement of the damaged reinforcement in the plastic hinge region prior to application of the UHPFRC jackets, ii) the increase of the UHPFRC jacket length within the damaged footing to account for the strain penetration effect in the starter bars of the lap-spliced connection, iii) pre-damaging of the intact specimens under higher axial loads ($v \approx 30\%-40\%$ range), iv) application of the UHPFRC jacket without removal of the axial load, to more closely emulate realistic scenarios in practice, v) application of debonded segments in the starter bars of the lap-spliced connections at varying lengths and locations during the jacketing process, and vi) consideration of alternate geometrical properties, section types and reinforcement detailing arrangements.

8. To more reliably predict and capture the seismic behavior of UHPFRC-retrofitted components, computational tools capable of large-scale simulations and detailed modeling are necessary. These tools should include the pertinent analytical material models, which have been calibrated based on experimental evidence, such as the ones described in this study. Further preparation of computation simulations is thus recommended to validate the accuracy and reliability of such analyses.
9. In preparation for the tests, it was necessary to operate the high-capacity crane mechanism and sky-jack cart, to install the pertinent equipment and move the specimens. It was also required to make considerable use of high-power mechanical equipment and tools (i.e., power drills, cutting saws, grinders) for the preparation of test components. It is advised to include the training for such operations prior to commencing these experiments.

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Appendix A: FEA Results of Bond Pullout Tests

A.1 Model: Standard Pullout

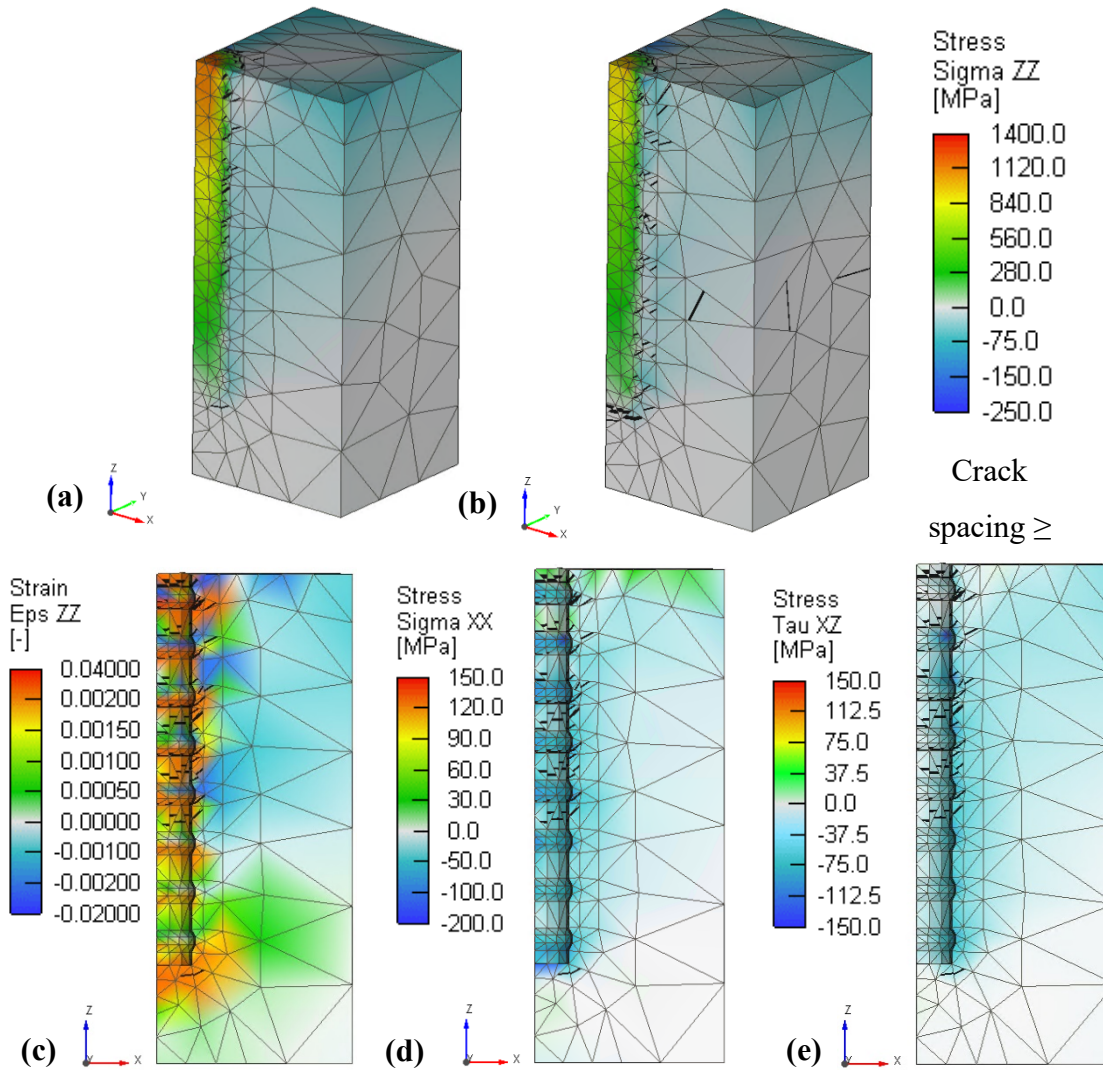


Figure A.1. Finite element model for SP-C2-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{xx} at peak; e) Concrete Stress τ_{xz} at peak.

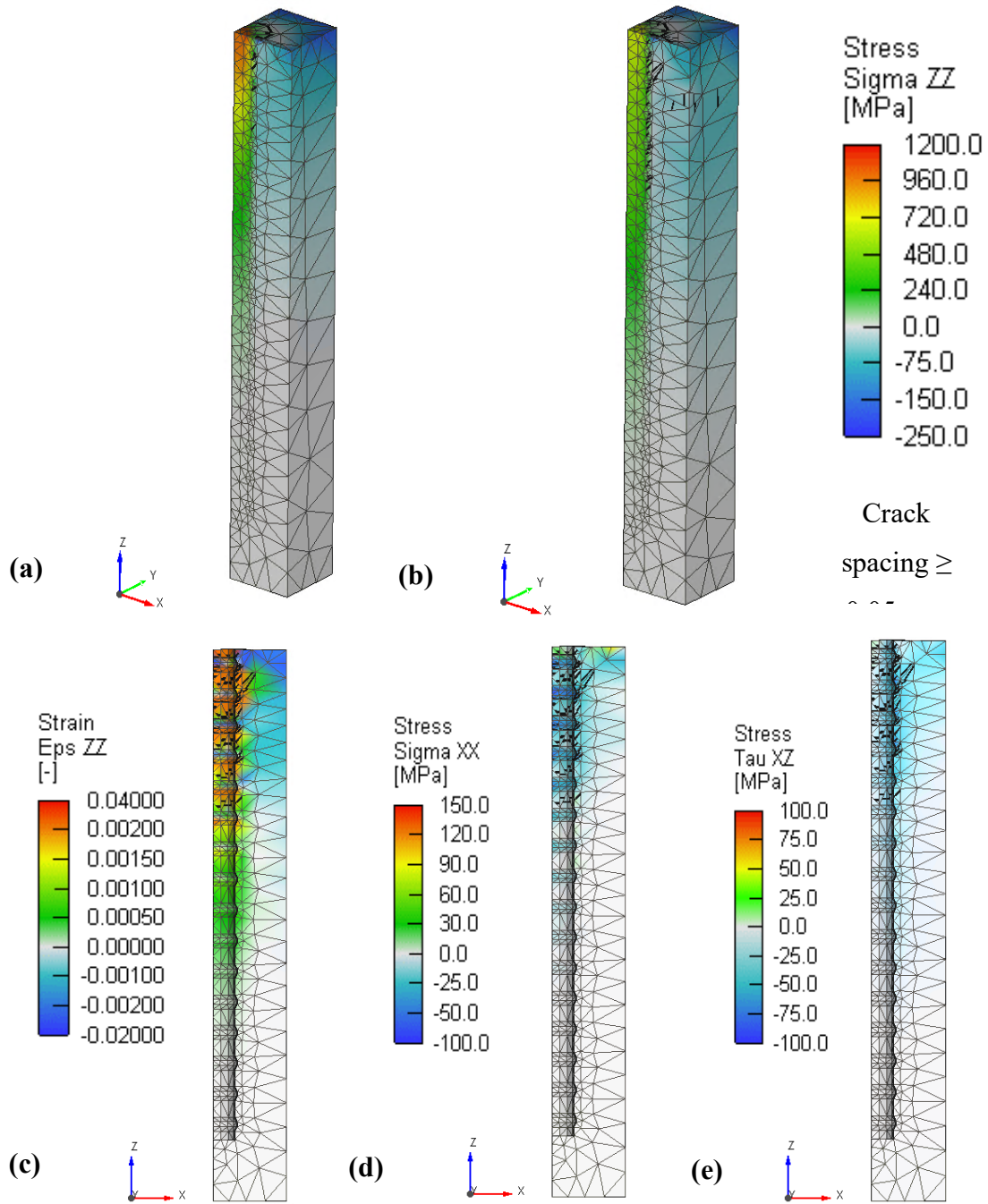


Figure A.2. Finite element model for SP-C1-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{xx} at peak; e) Concrete Stress τ_{xz} at peak.

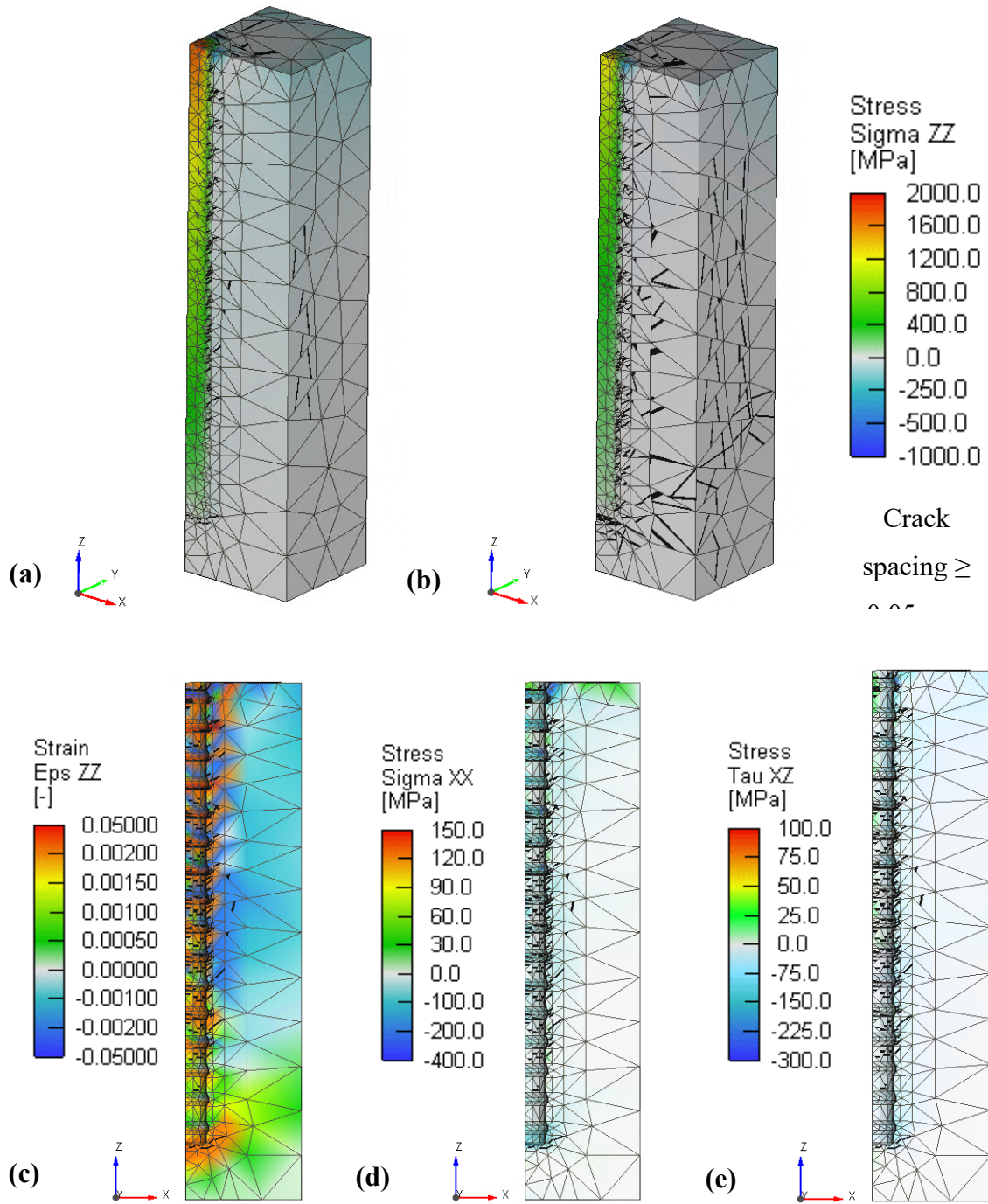


Figure A.3. Finite element model for SP-C2-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{xx} at peak; e) Concrete Stress τ_{xz} at peak.

A.2 Model: Beam-End

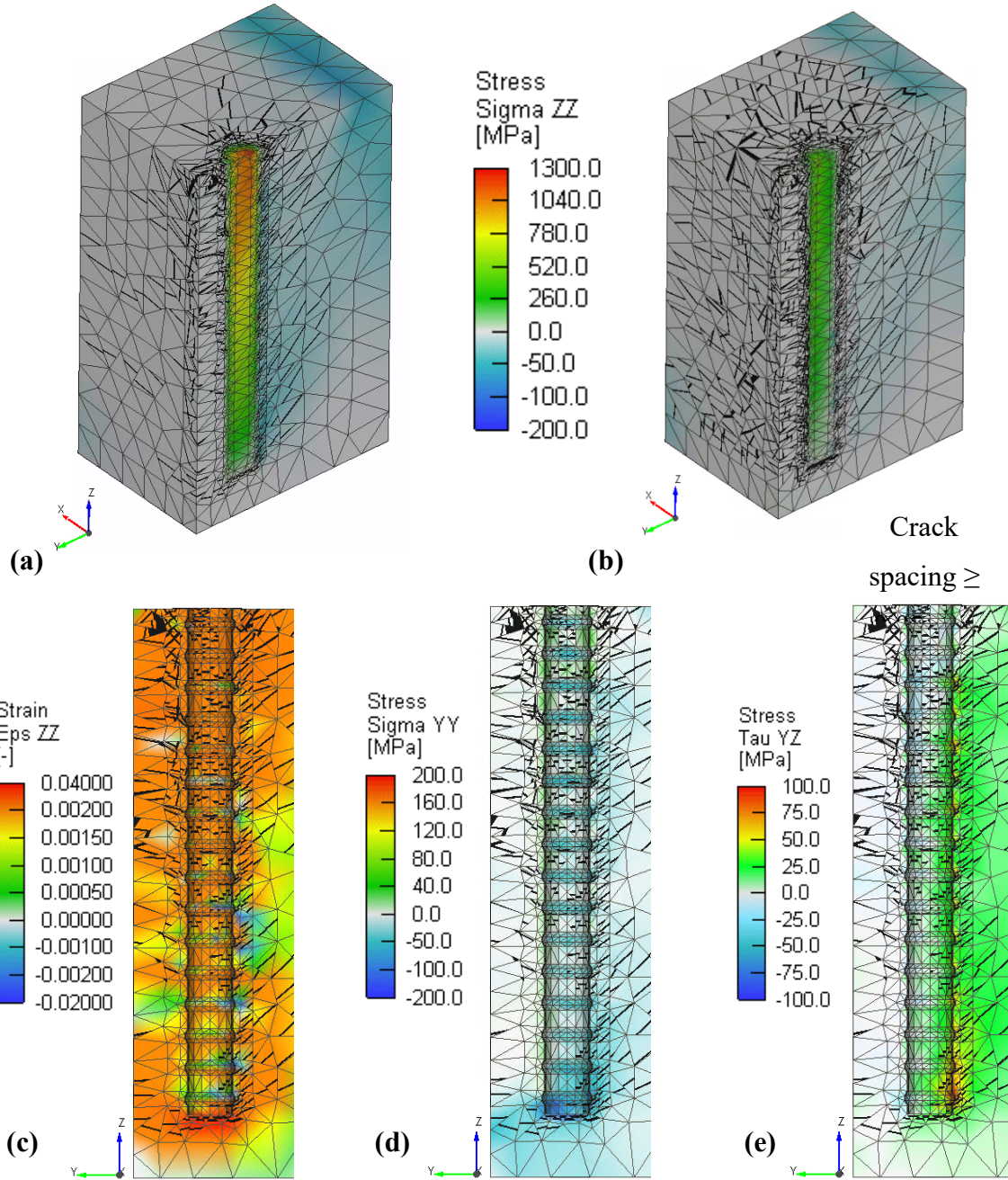


Figure A.4. Finite element model for BE-C1-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

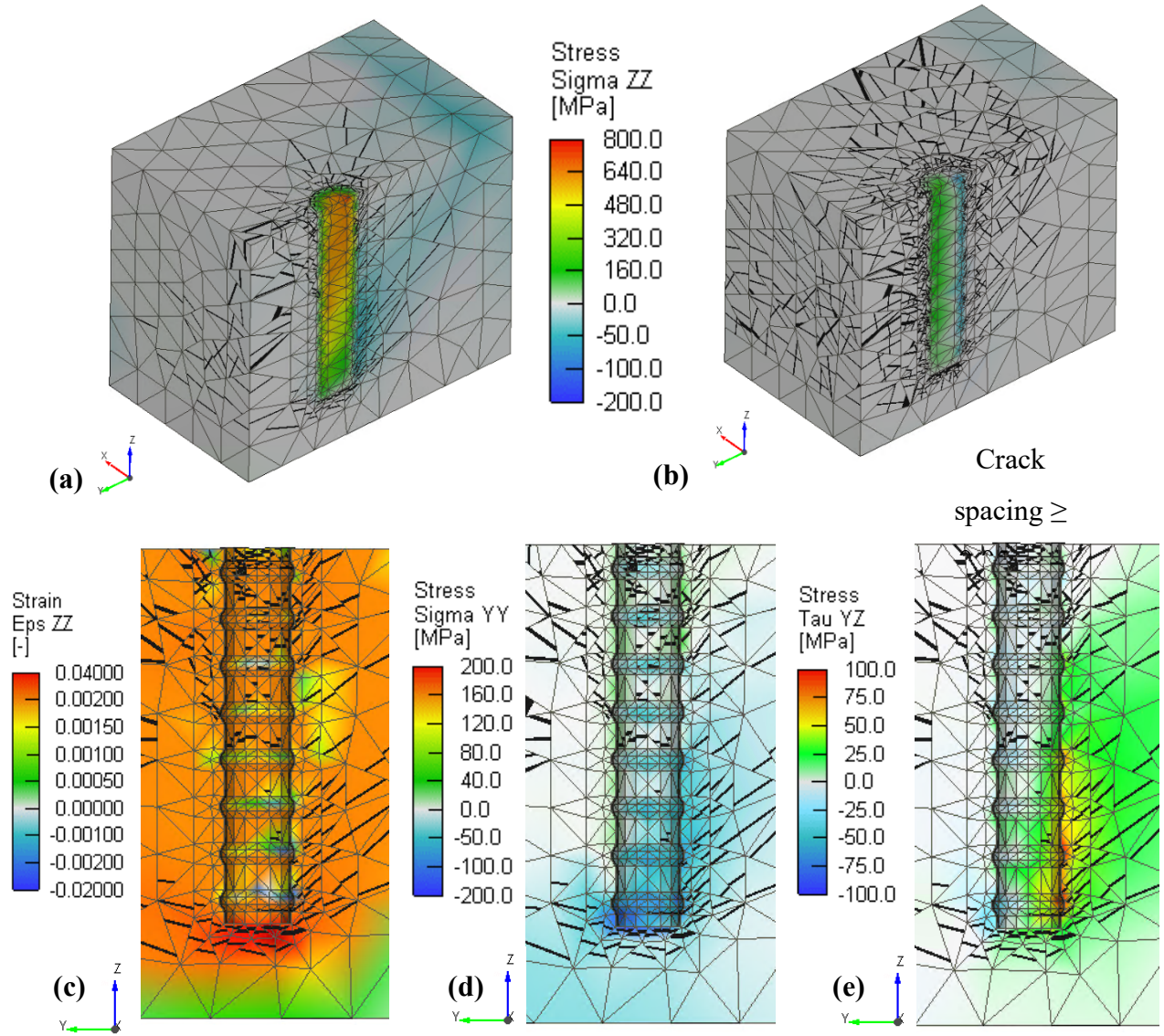


Figure A.5. Finite element model for BE-C2-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

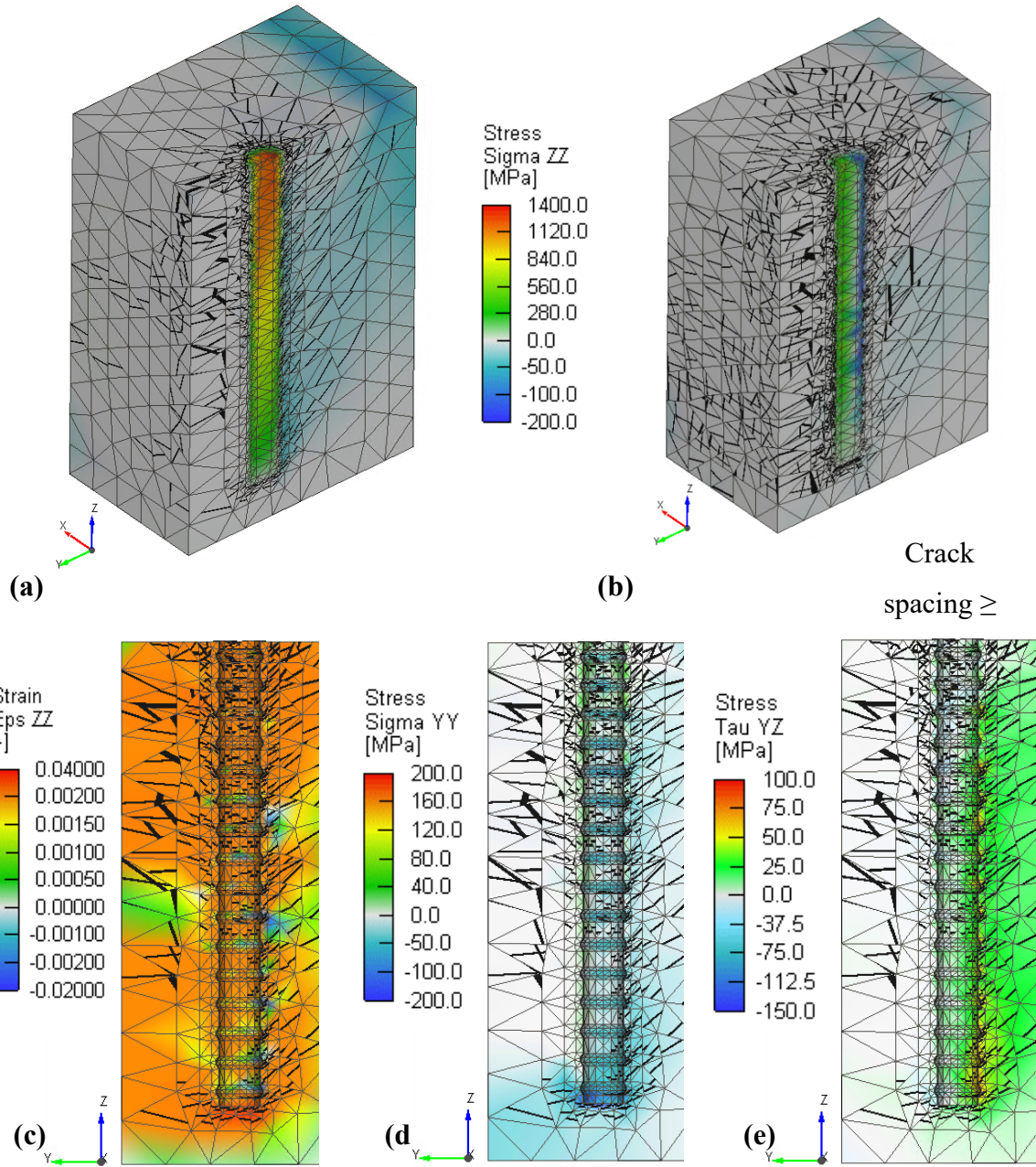


Figure A.6. Finite element model for BE-C5-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

A.3 Model: Direct Tension Pullout

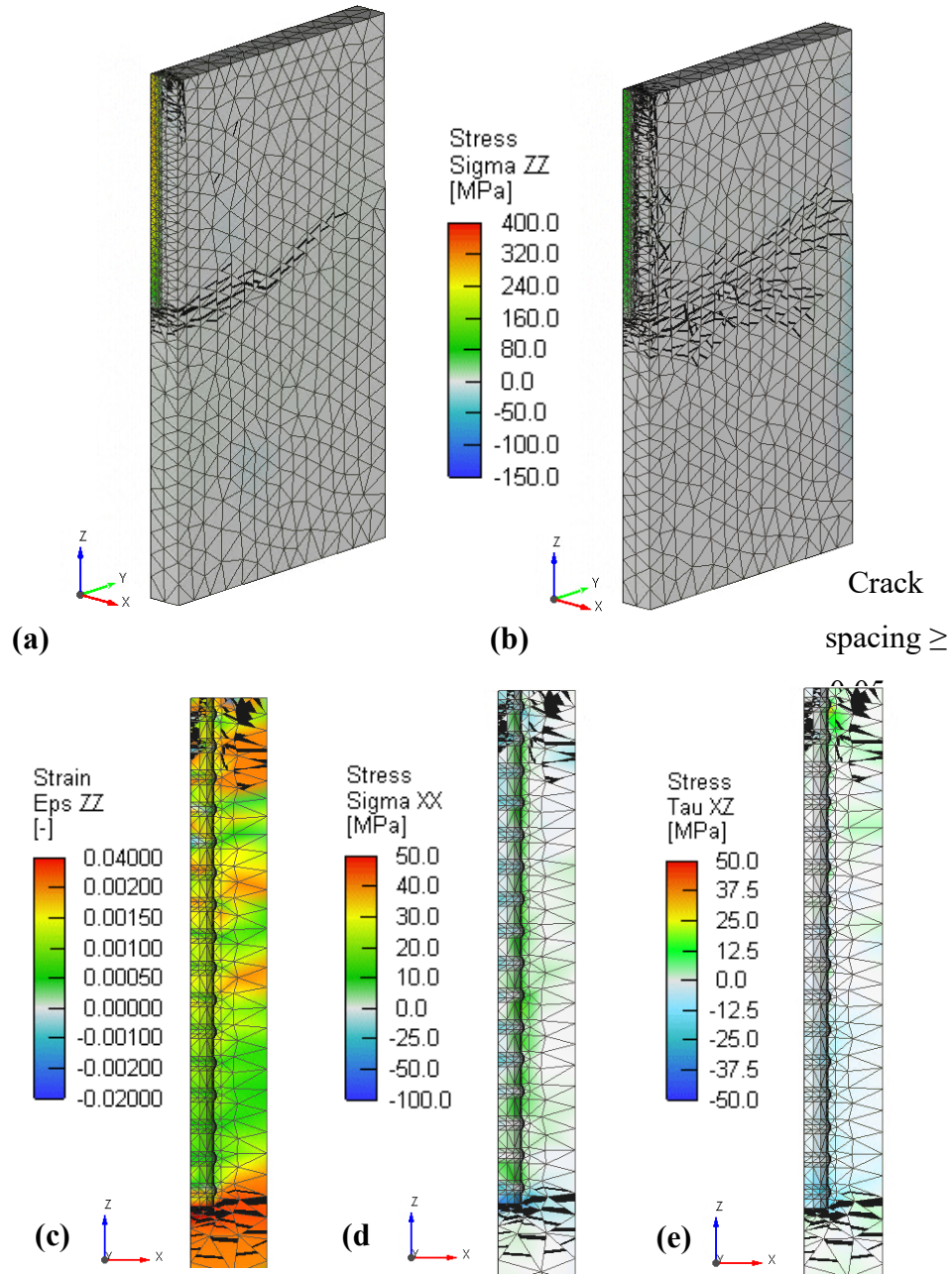


Figure A.7. Finite element model for DTP-C1-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{xx} at peak; e) Concrete Stress τ_{xz} at peak.

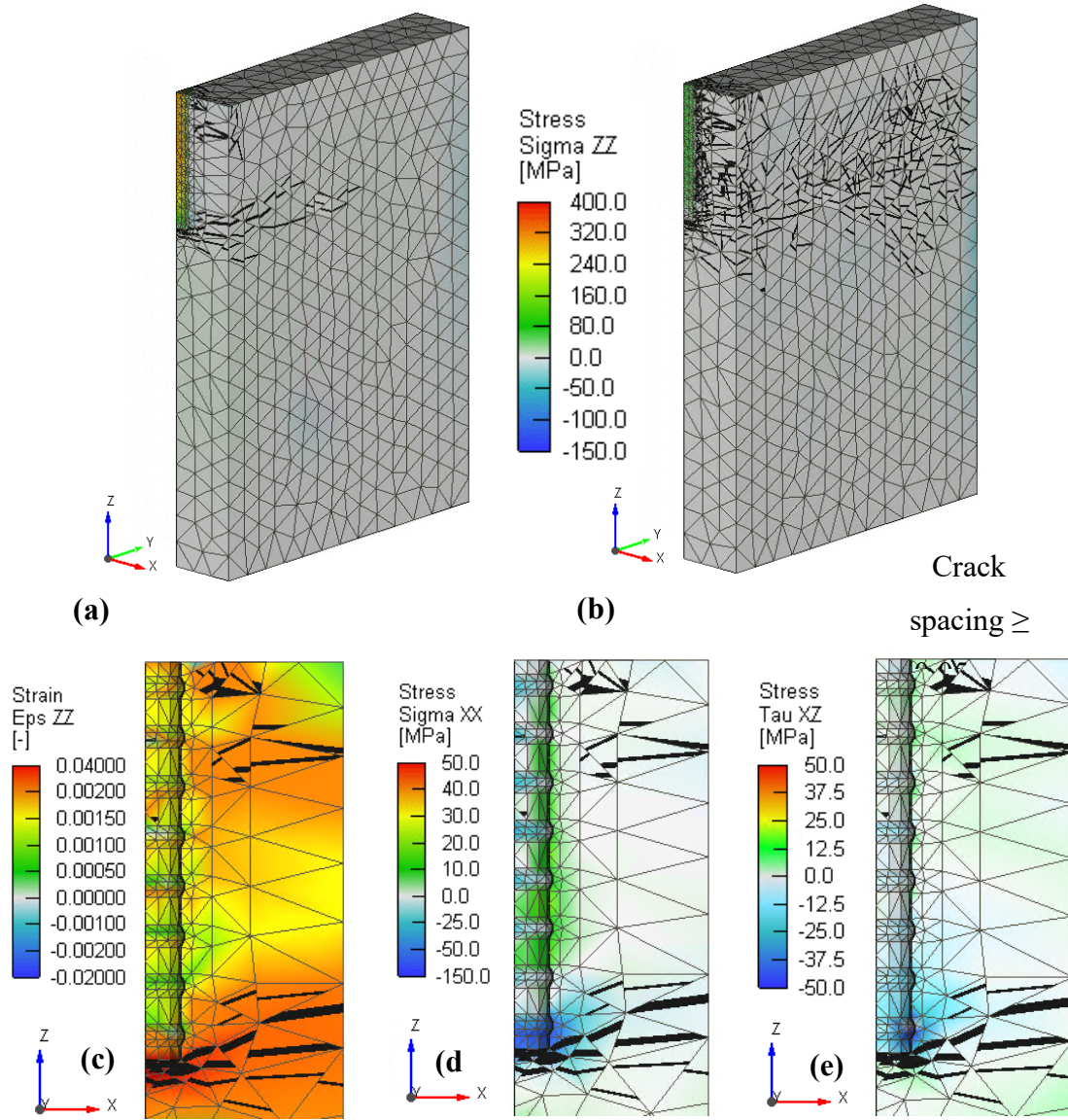


Figure A.8. Finite element model for DTP-C2-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{xx} at peak; e) Concrete Stress τ_{xz} at peak.

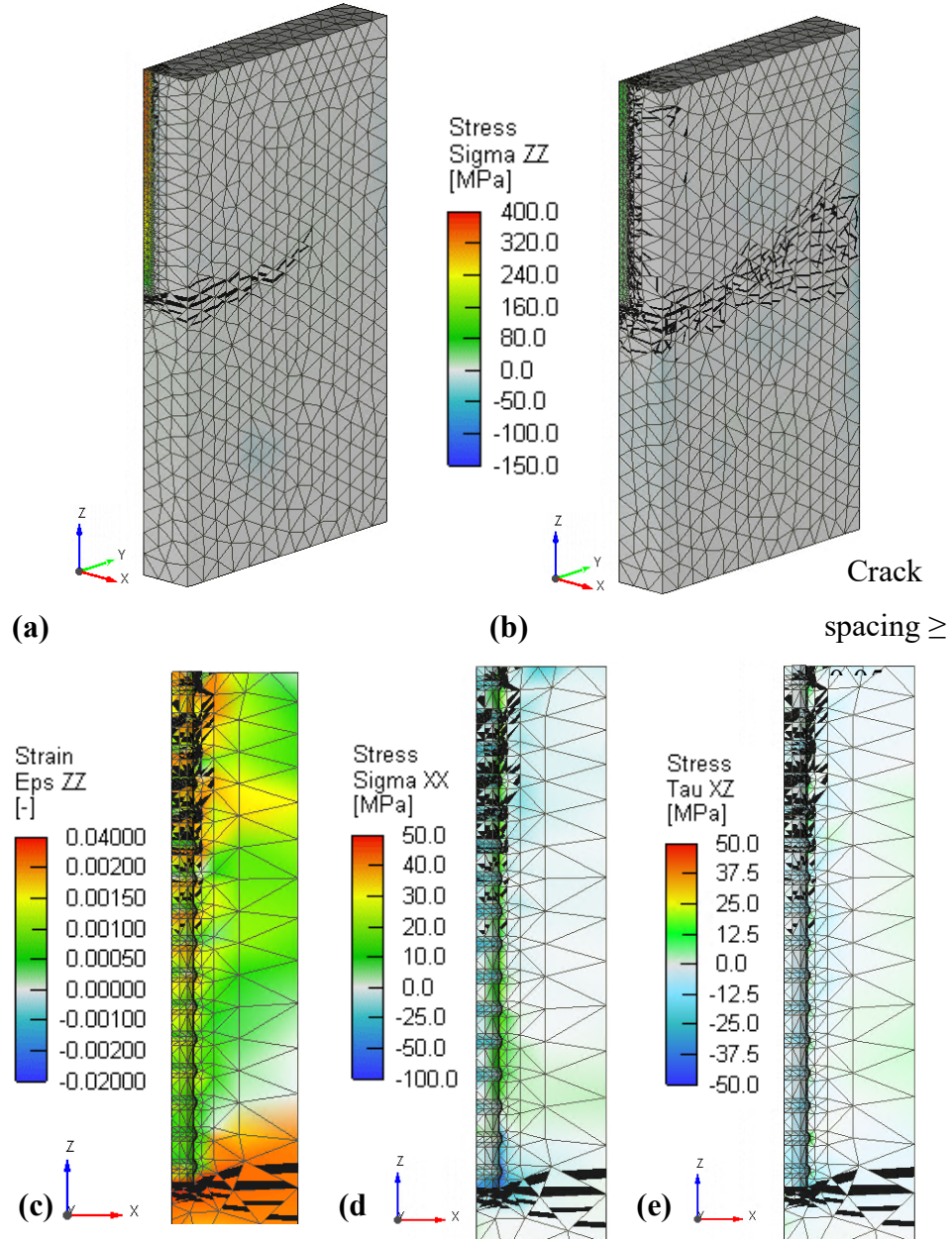


Figure A.9. Finite element model for DTP-C2-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{xx} at peak; e) Concrete Stress τ_{xz} at peak.

A.4 Model: Contact Lap-Splice Direct Tension Pullout

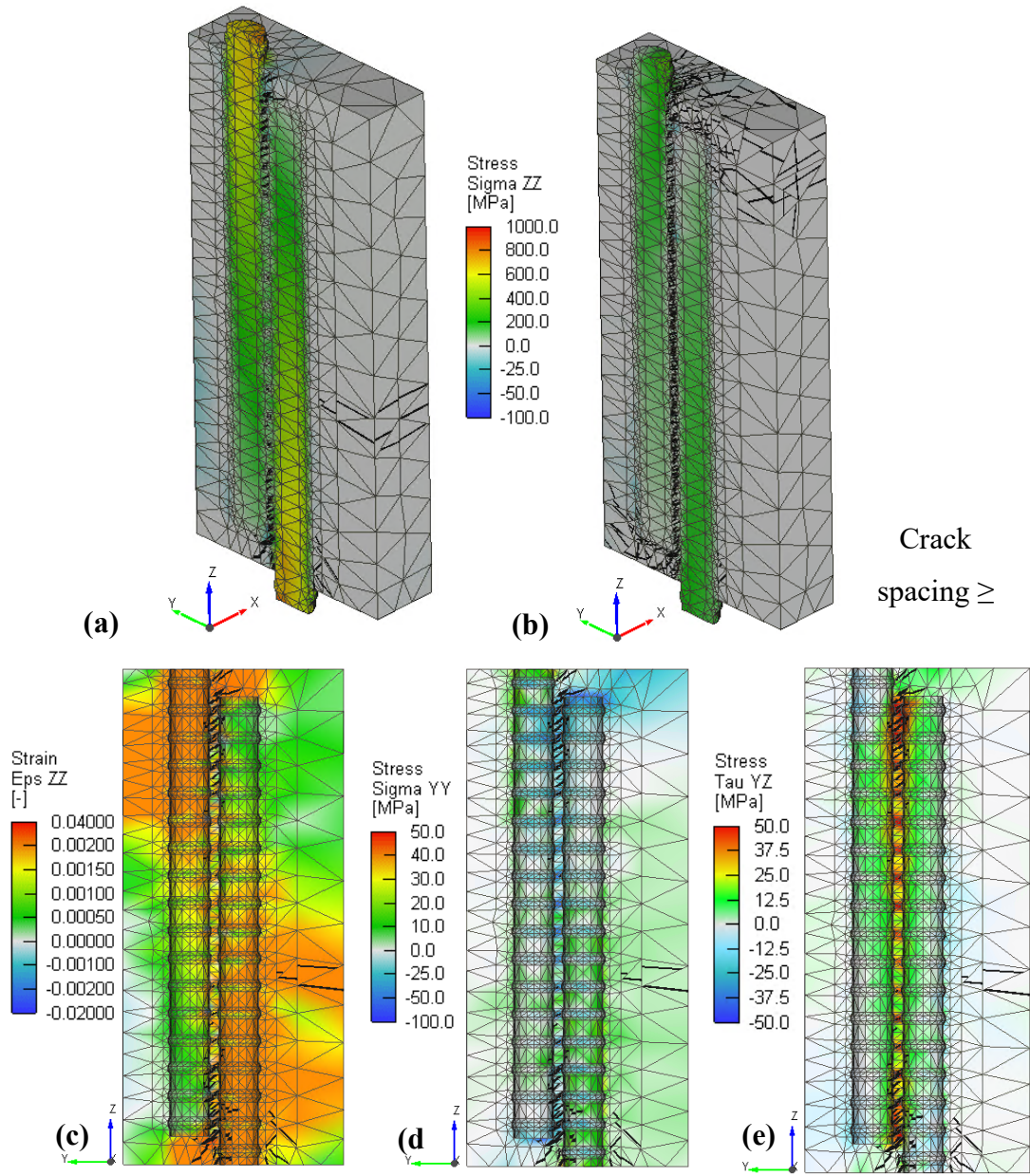


Figure A.10. Finite element model for CLP-C1-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

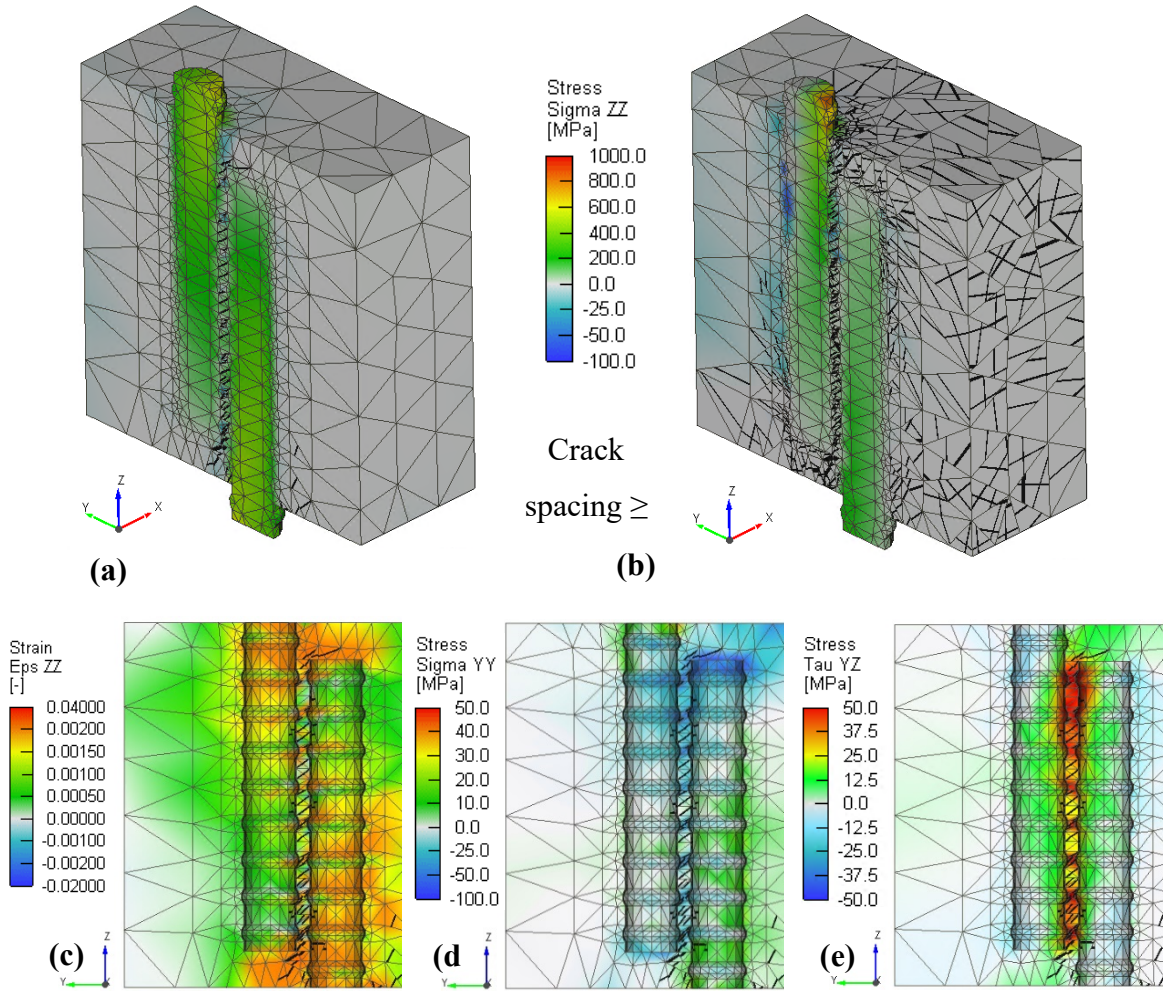


Figure A.11. Finite element model for CLP-C2-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

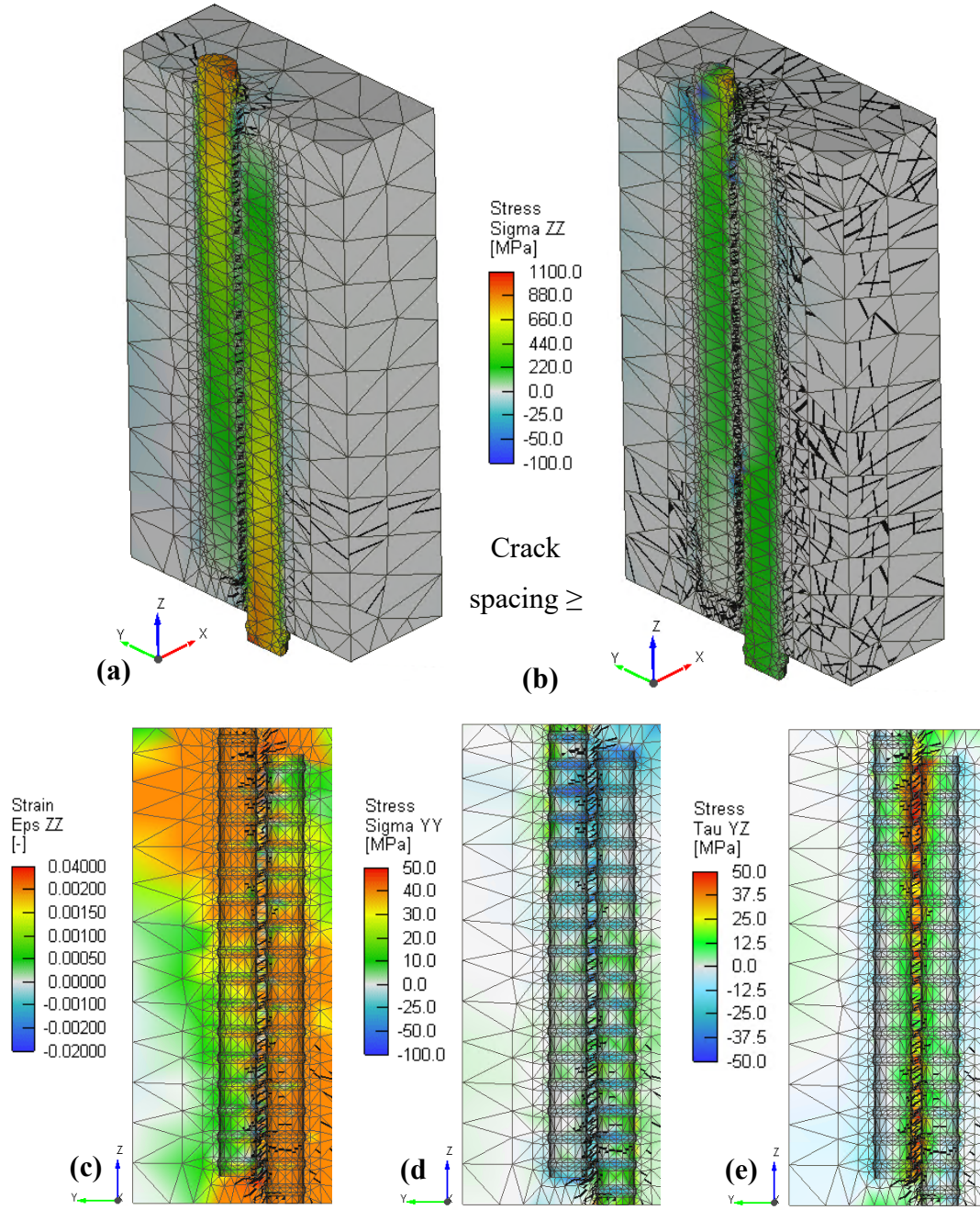


Figure A.12. Finite element model for CLP-C2-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

A.5 Model: Non-Contact Lap-Splice Direct Tension Pullout

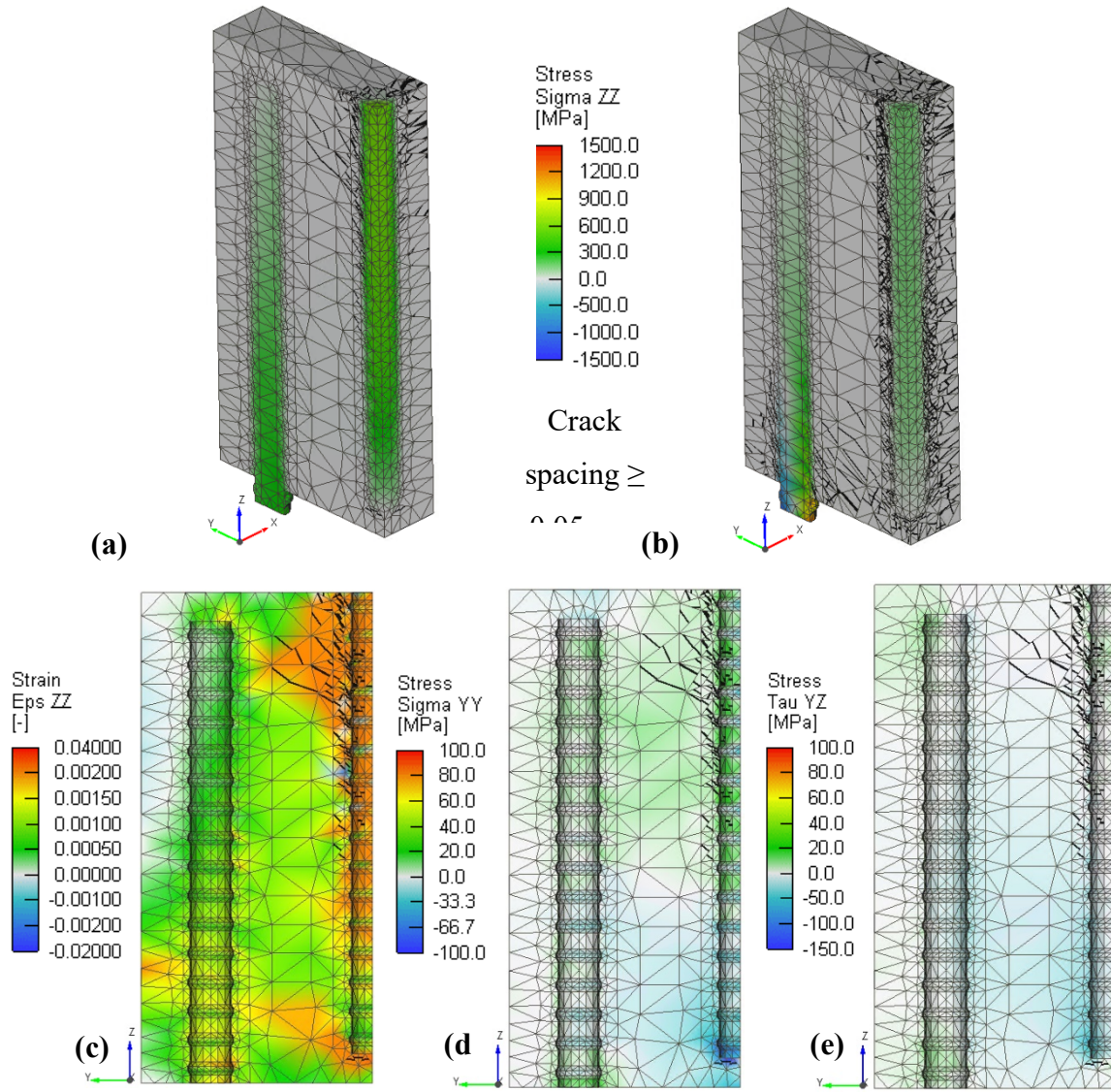


Figure A.13. Finite element model for NCLP-C1-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

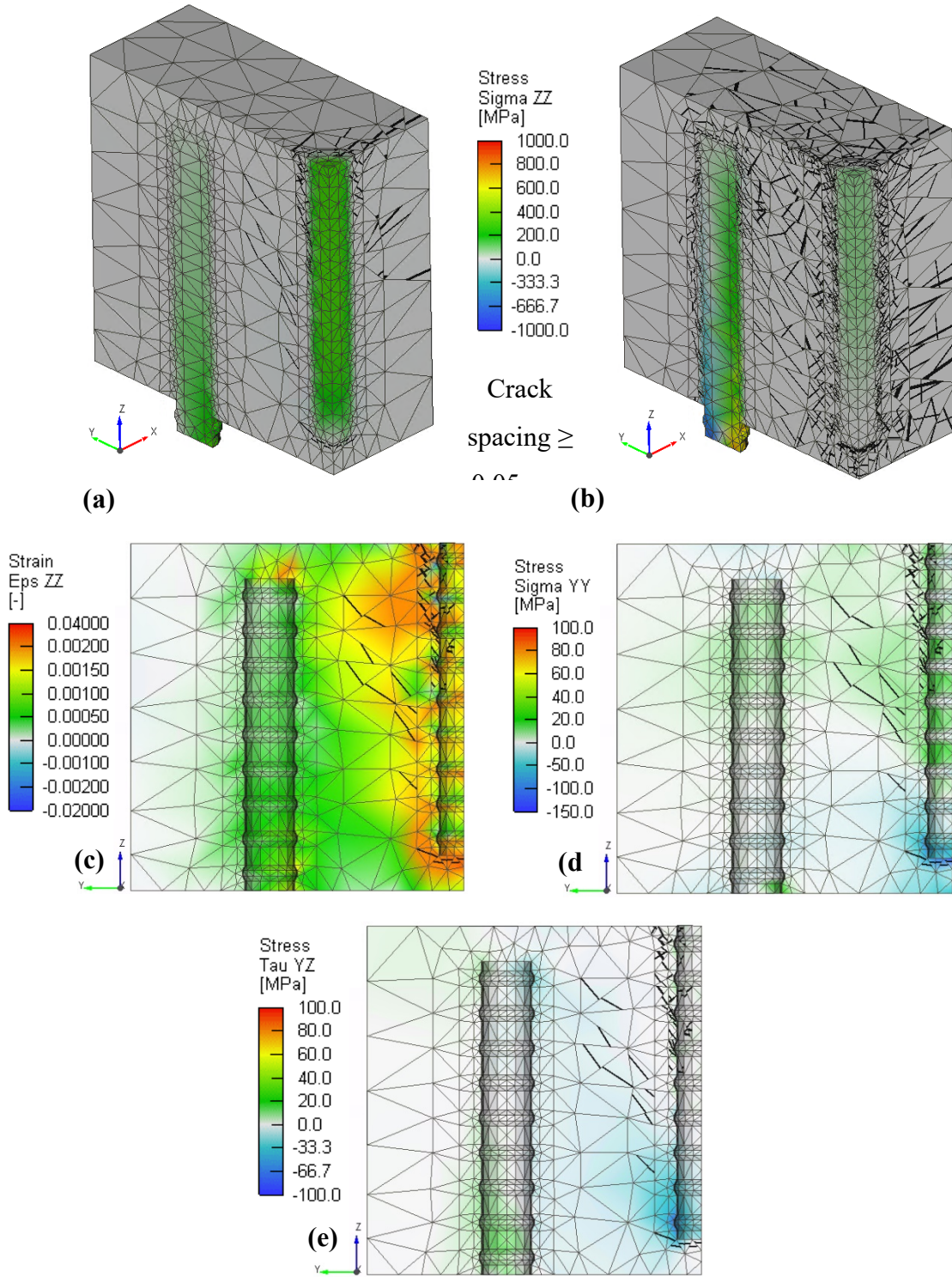


Figure A.14. Finite element model for NCLP-C2-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

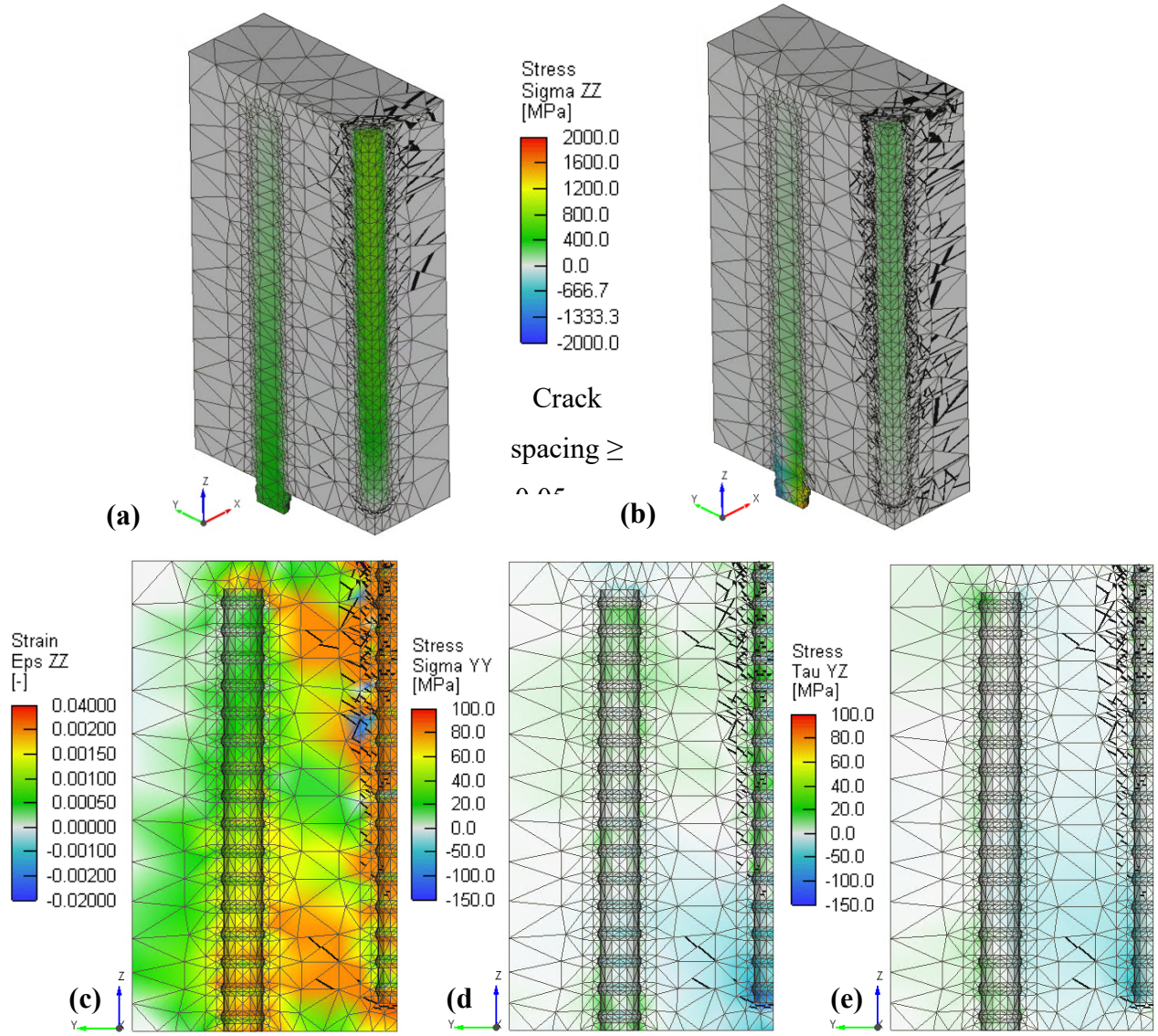


Figure A.15. Finite element model for NCLP-C2-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

A.6 Model: Restrained Non-Contact Lap-Splice Direct Tension Pullout

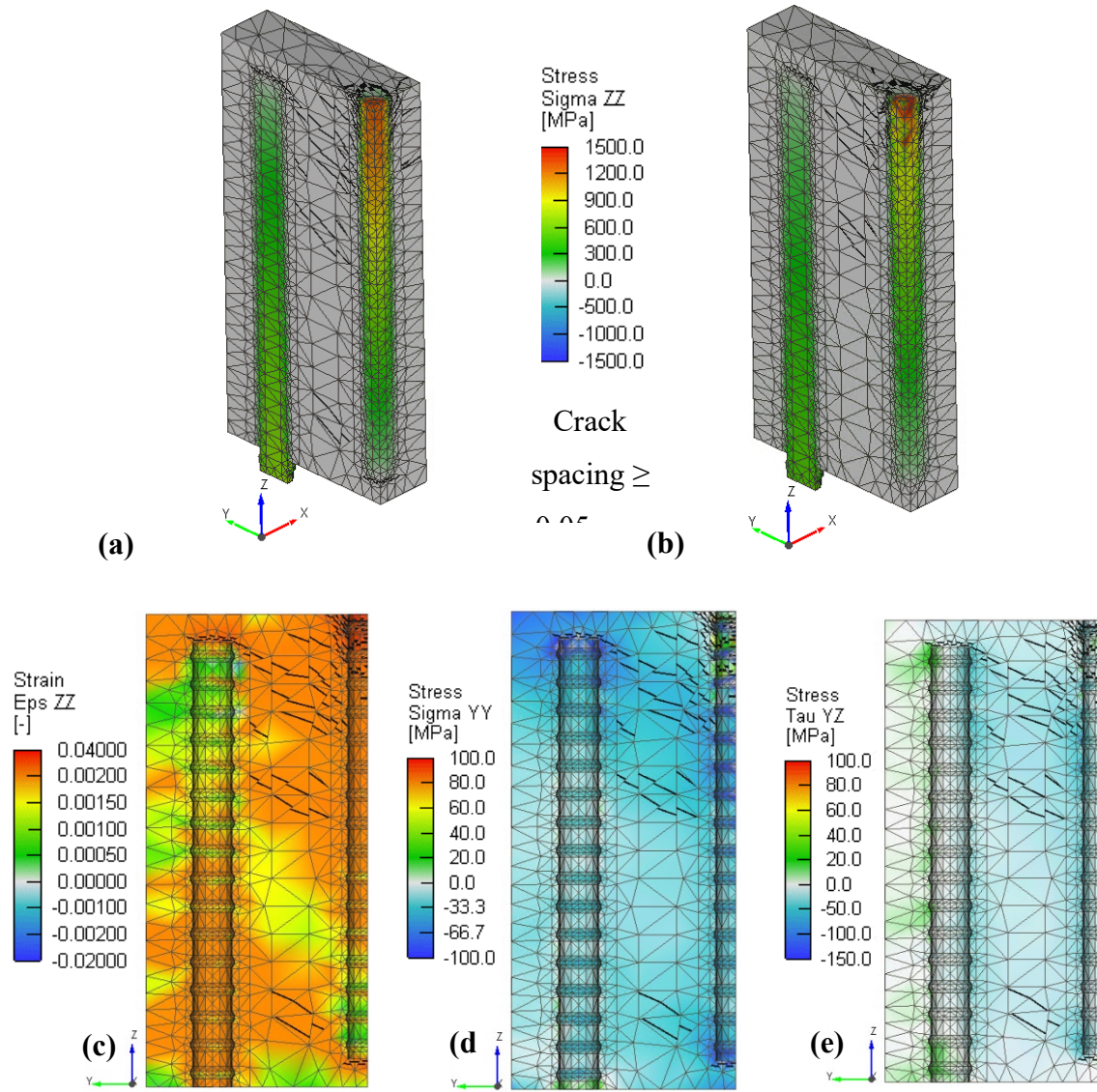


Figure A.16. Finite element model for R-NCLP-C1-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

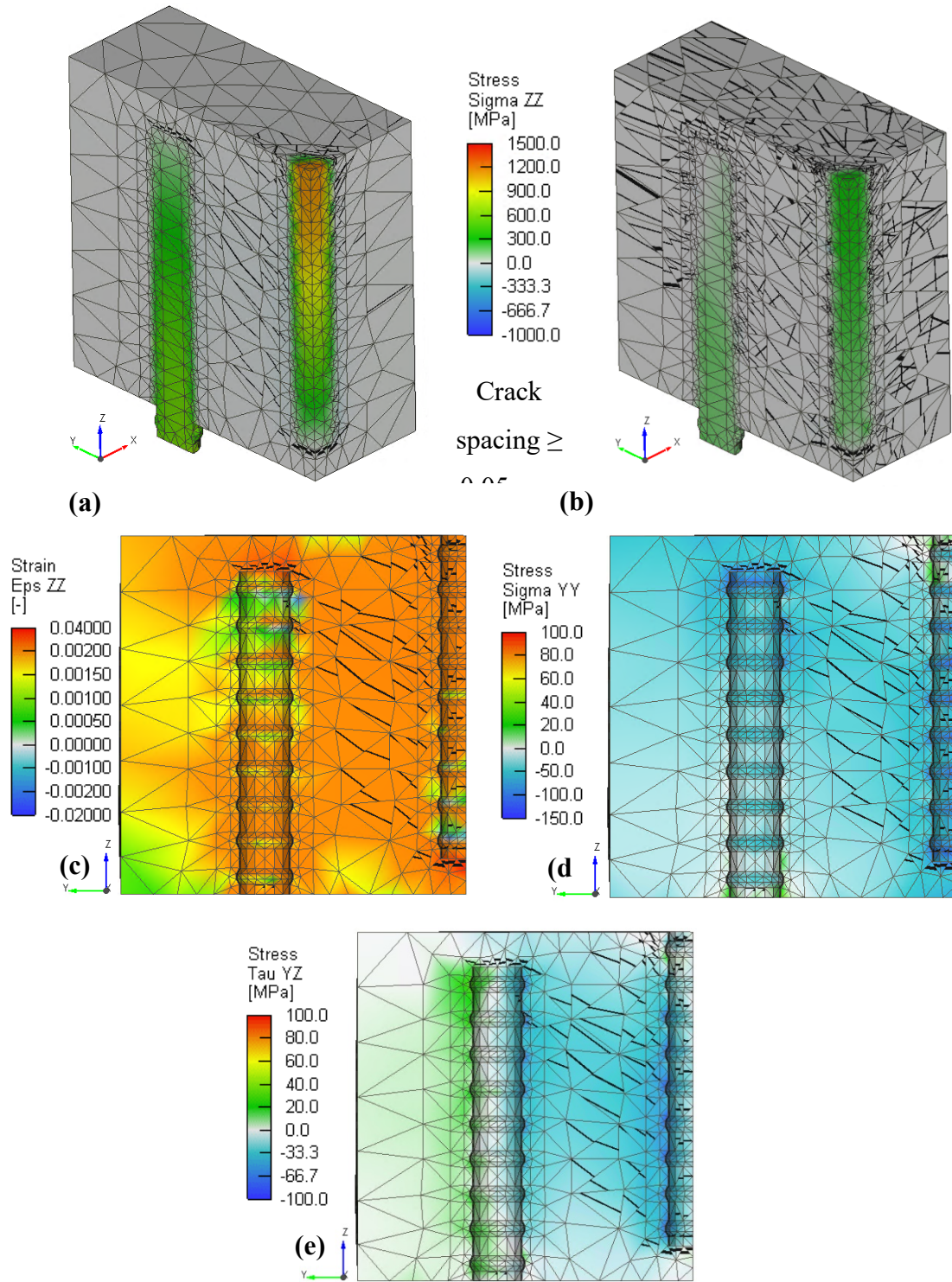


Figure A.17. Finite element model for R-NCLP-C2-L5 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

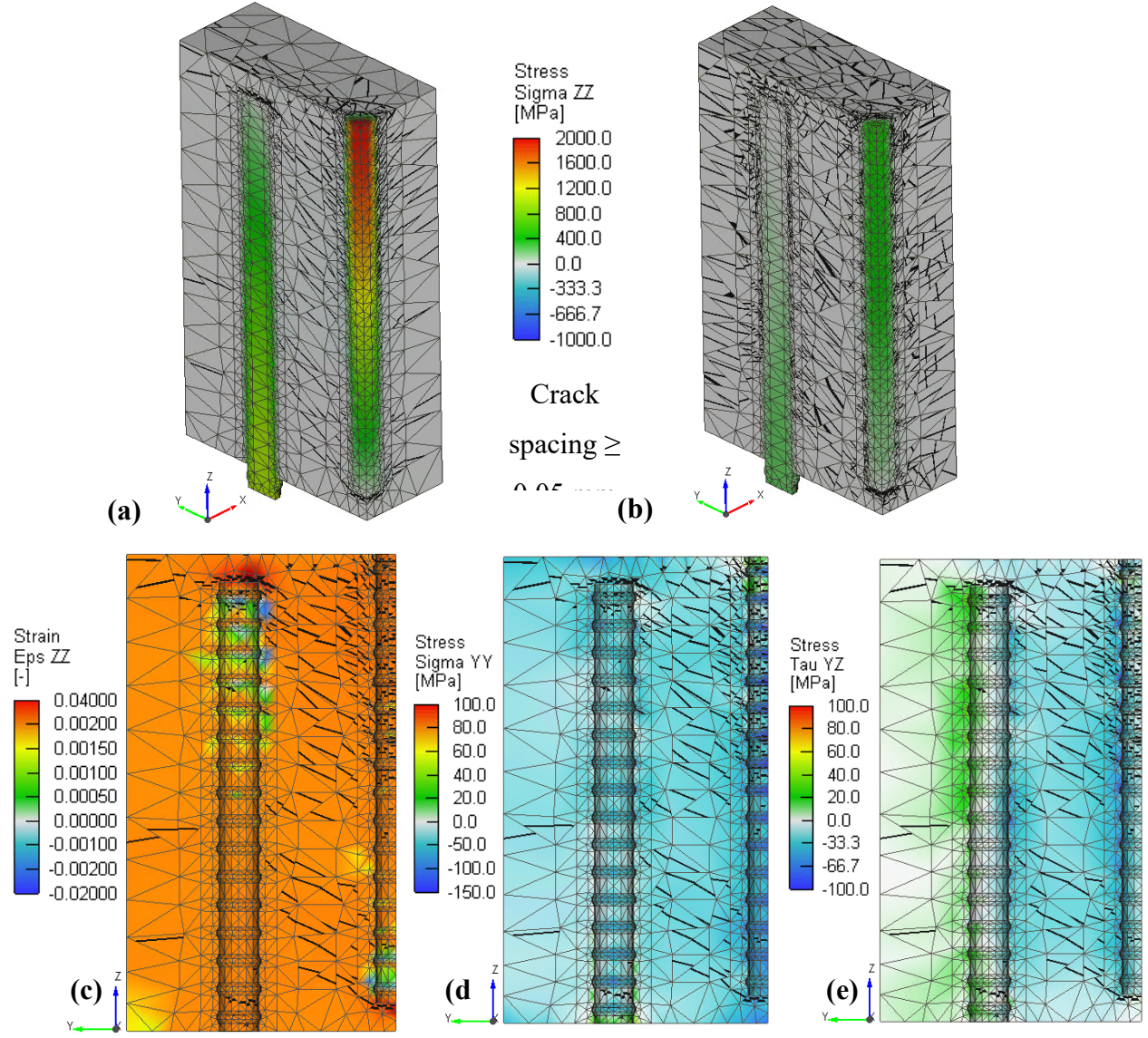


Figure A.18. Finite element model for R-NCLP-C2-L10 test: a) Bar stress σ_{zz} at peak and b) analysis end; c) Concrete Strain ϵ_{zz} at peak; d) Concrete Stress σ_{yy} at peak; e) Concrete Stress τ_{yz} at peak.

Appendix B: Material-Scale Testing and Analysis Results

Table B.1. Material Mix Design for UHPFRC Jackets.

Material Type	Proportions by Mass (kg)	Density (kg/m ³)
Prepackaged Dry-Mix (Densit)	0.84	2860
Steel Fibers ($d_f = 0.2\text{mm}$; $l_f = 13\text{mm}$)	0.11	
Water	0.055	

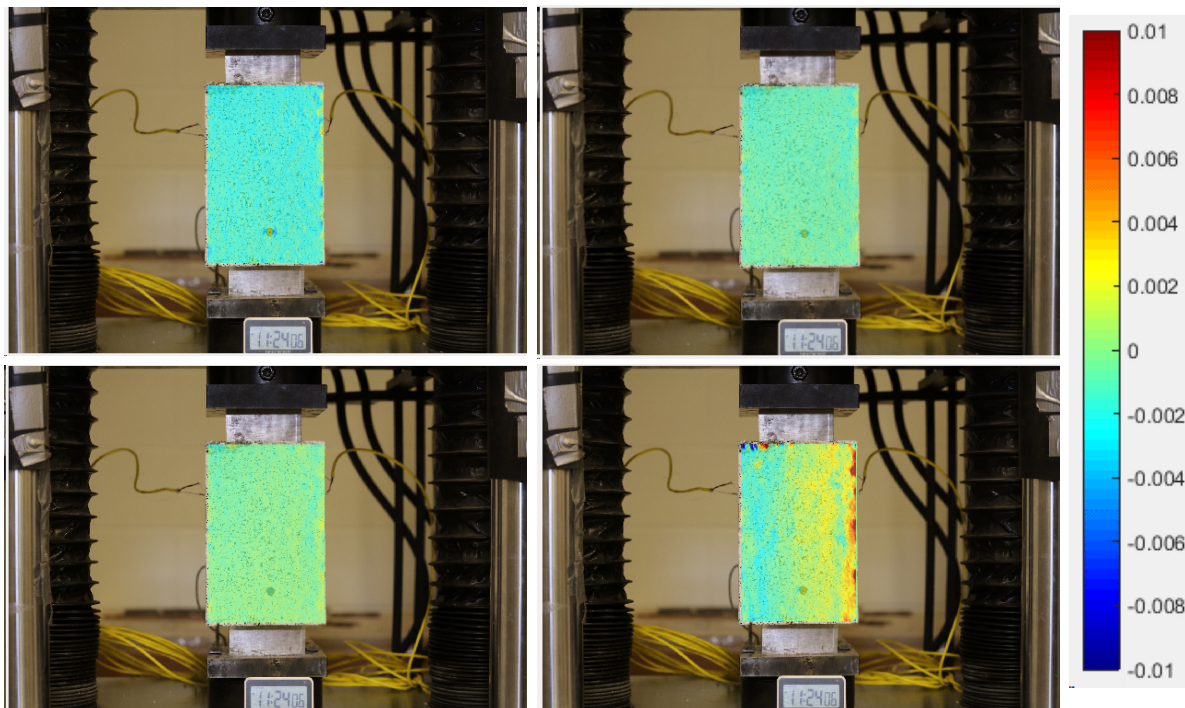


Figure B.1. Expansion responses for the CP25-M1 specimen series.

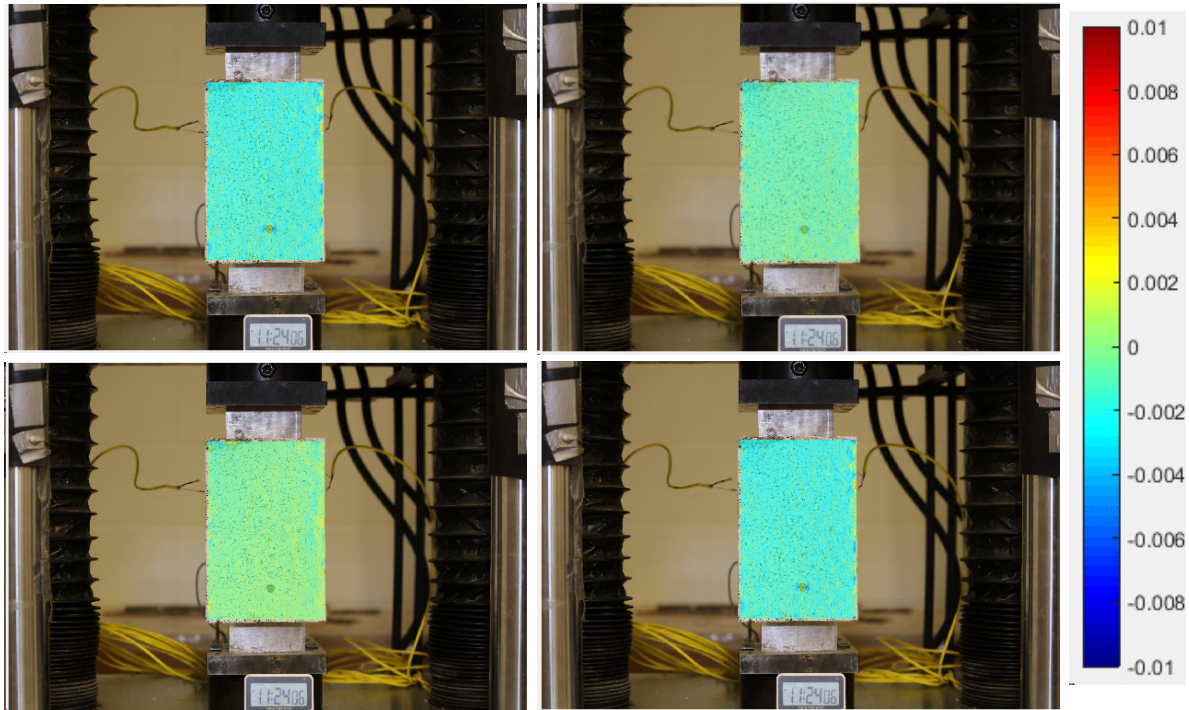


Figure B.2. Expansion responses for the CP25-M2 specimen series.

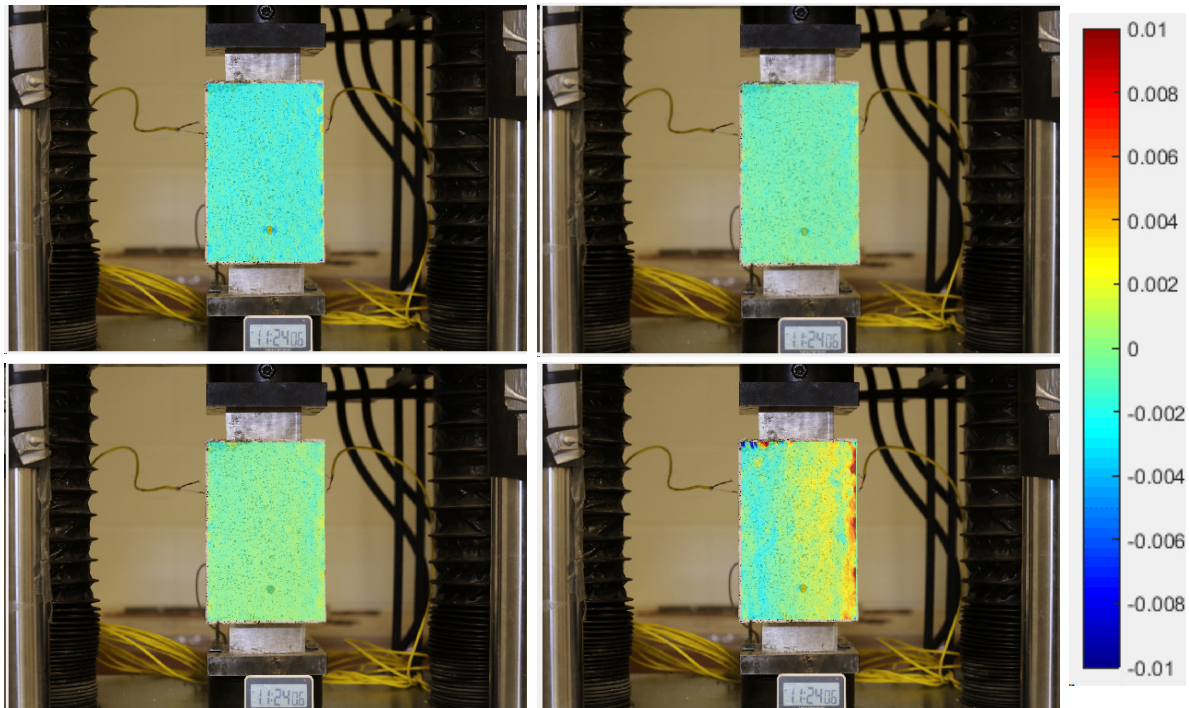


Figure B.3. Expansion responses for the CP25-CA2 specimen series.

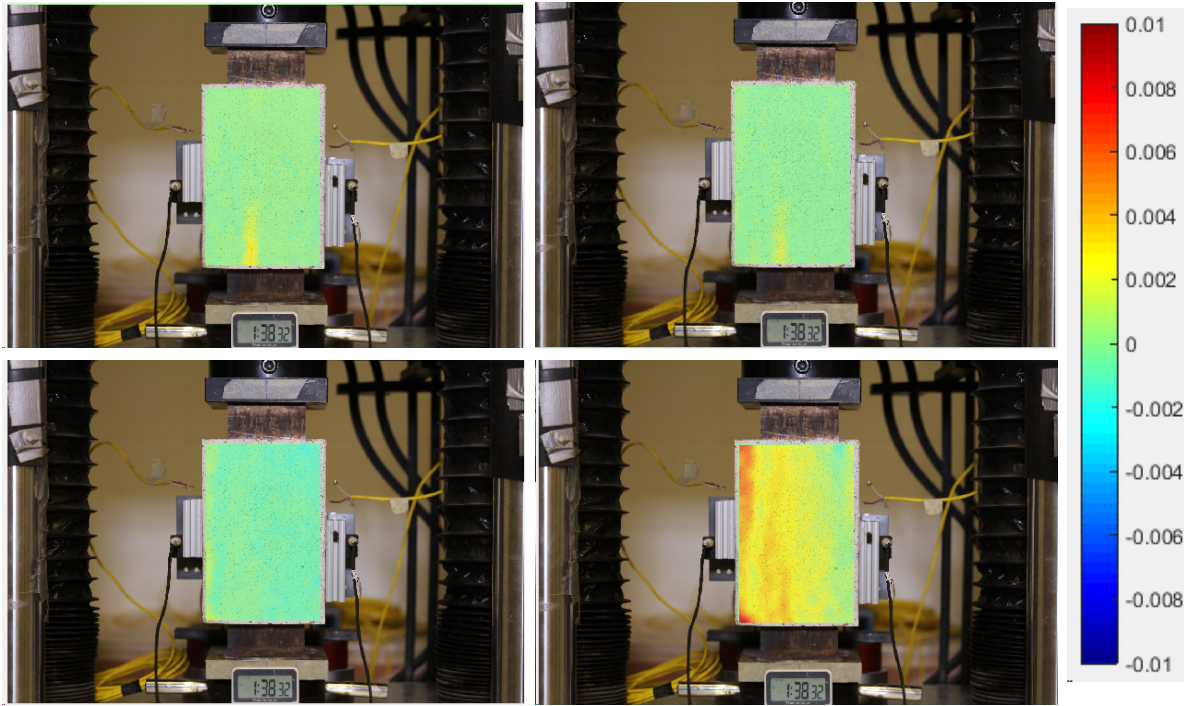


Figure B.4. Expansion responses for the CP25-CA3 specimen series.

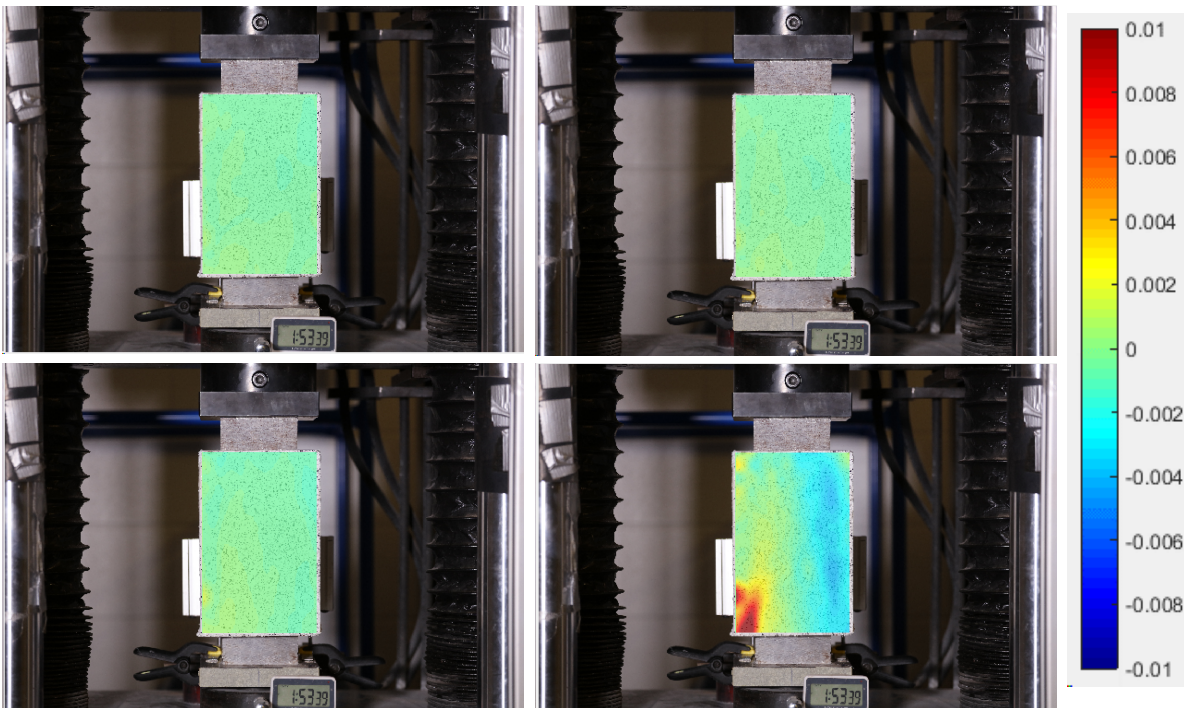


Figure B.5. Expansion responses for the CP25-CB1 specimen series.

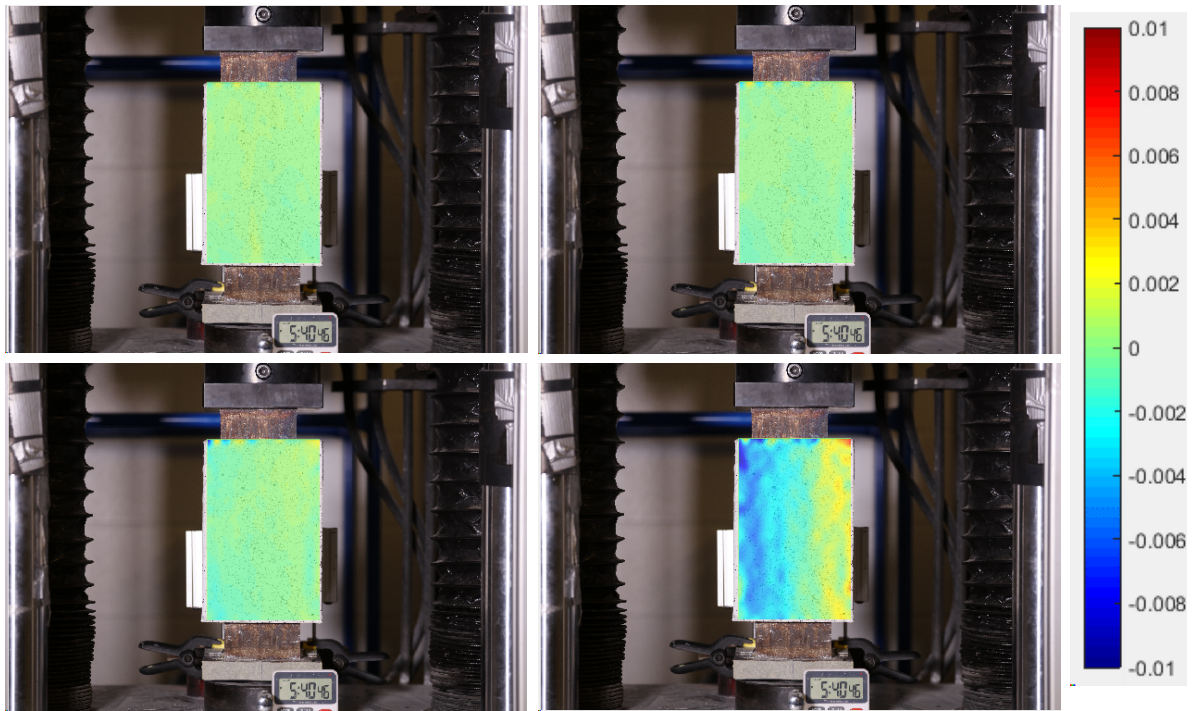


Figure B.6. Expansion responses for the CP25-CB2 specimen series.

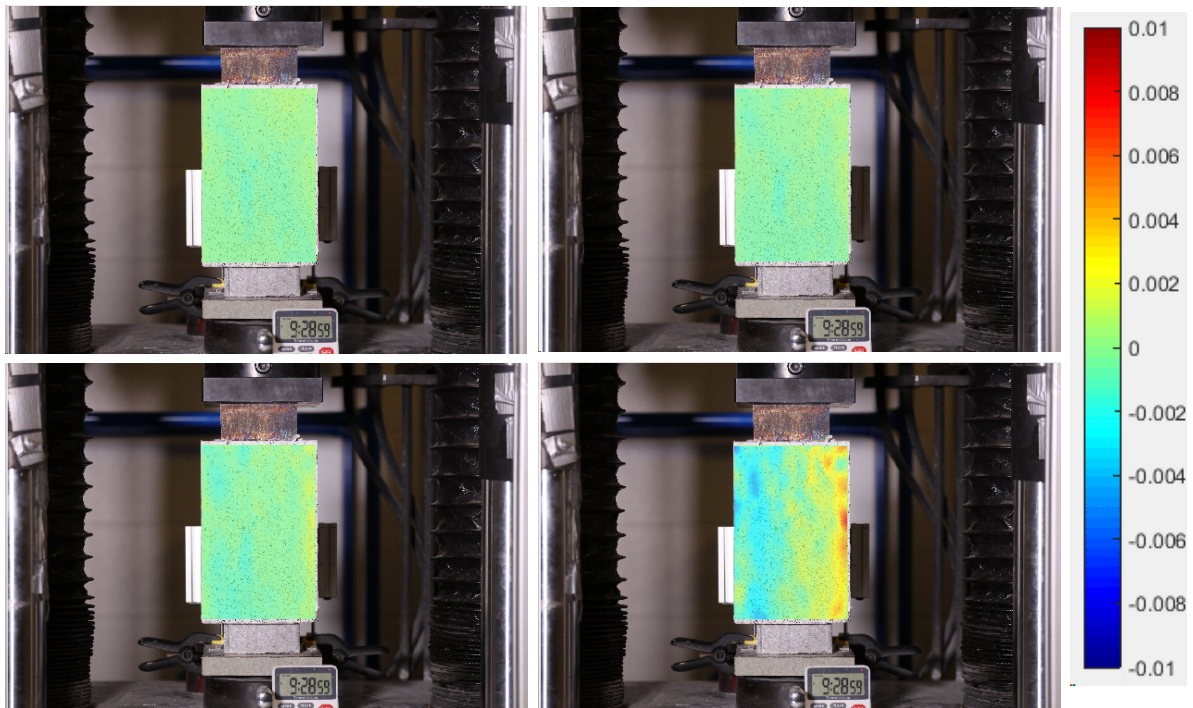


Figure B.7. Expansion responses for the CP25-CB3 specimen series.

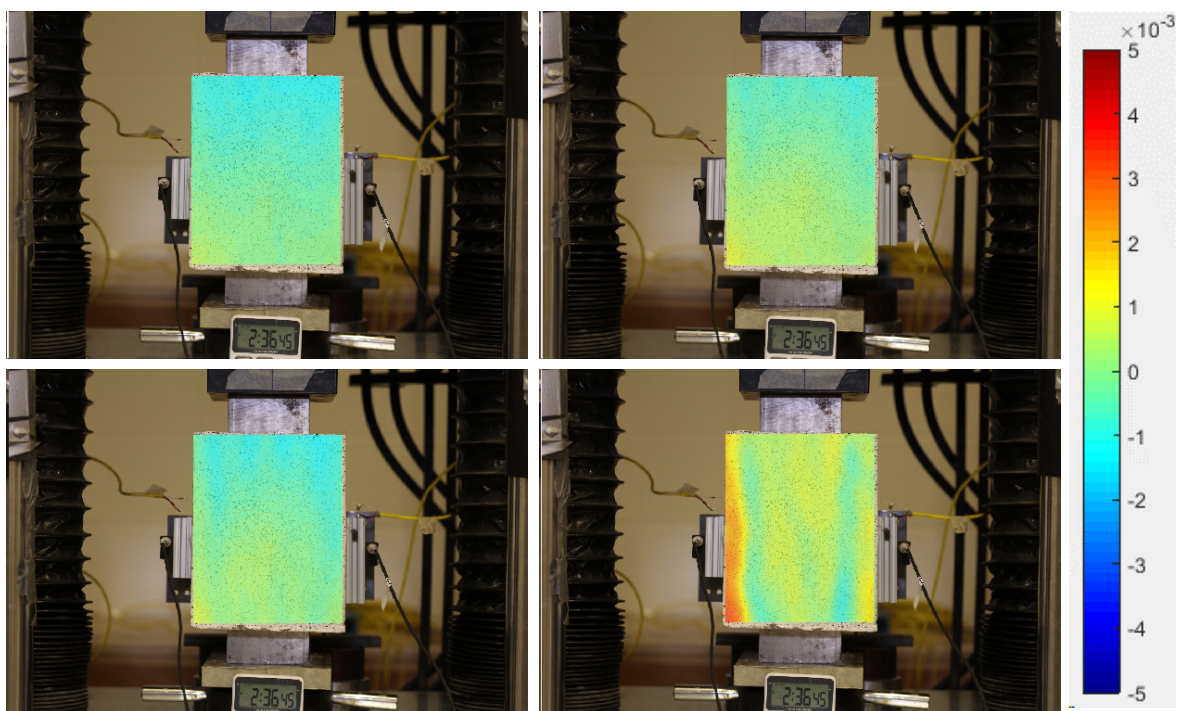


Figure B.8. Expansion responses for the CP375-C1 specimen series.

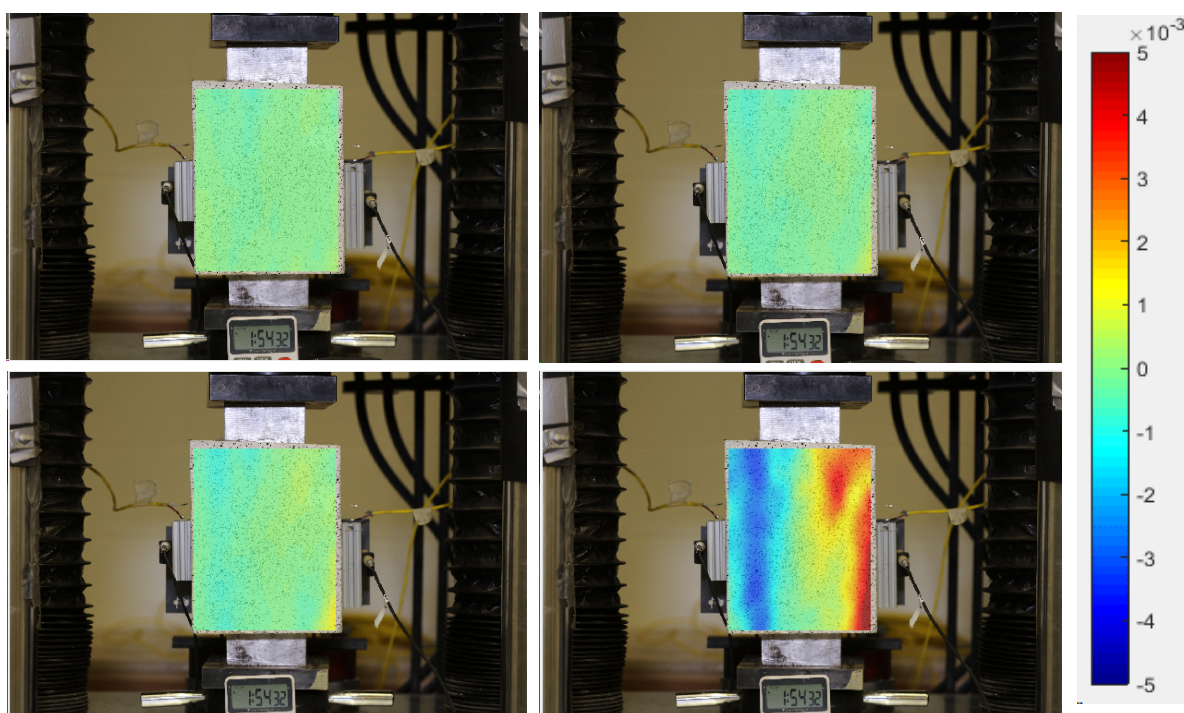


Figure B.9. Expansion responses for the CP375-C3 specimen series.

Appendix C: Structural-Scale Testing Results

C.1 Lab Operations

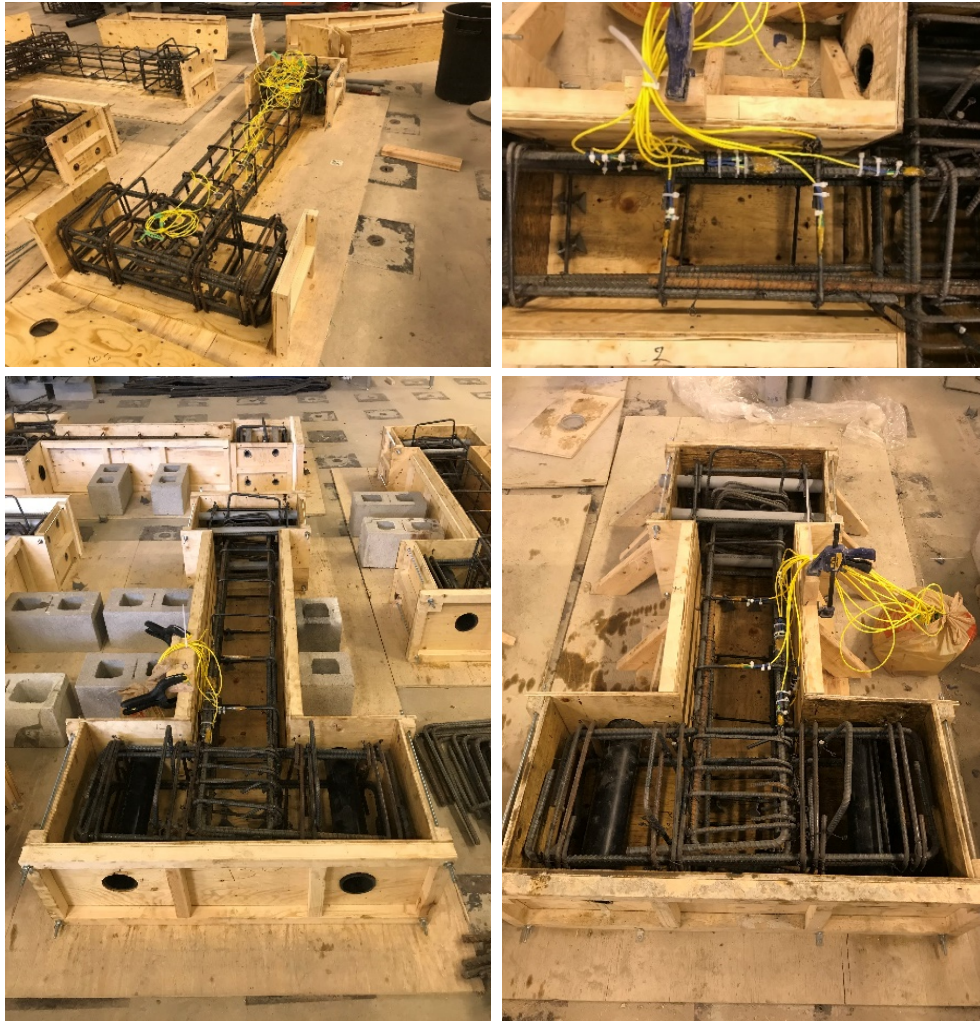


Figure C.1. Preparation of specimen formwork and reinforcement cages.



Figure C.2. Specimens after casting and prior to covering of the top surface.



Figure C.3. Casting of specimens; total number of specimens completed.

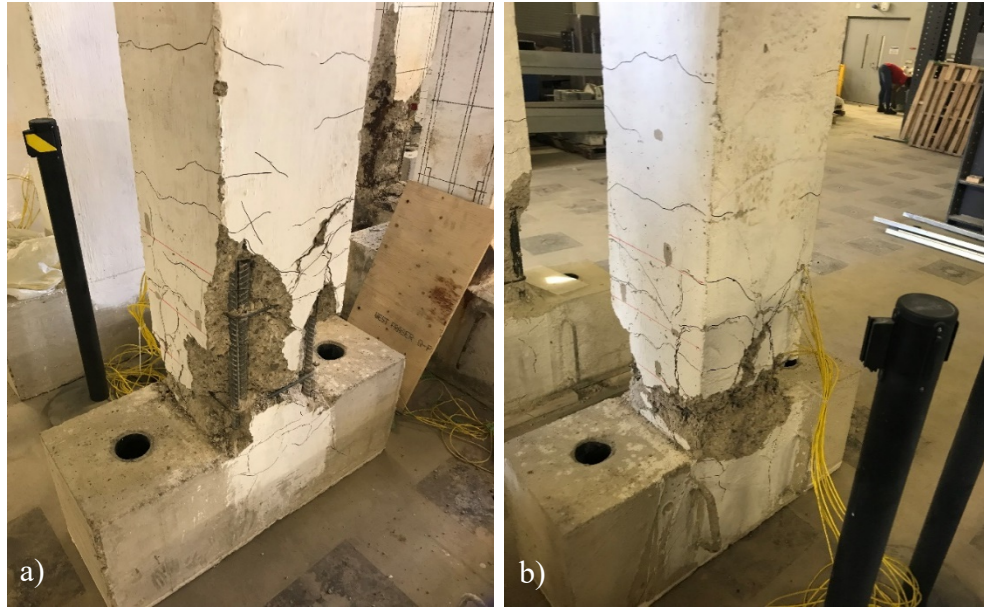


Figure C.4. Removal of damaged concrete from specimens: a) CT1-15M-R; b) CT2-20M-R.



Figure C.5. Removal of damaged core from specimen CS3-15M-R: a-b) Front view; c) Back view.



Figure C.6. Process of removal of damage cover and core concrete for specimen CT1-15M-R.



Figure C.7. Process of removal of damage cover and core concrete for specimen CT2-20M-R.



Figure C.8. Process of removal of damage cover and core concrete for specimen CS3-15M-R.



Figure C.9. Process of removal of damage cover and core concrete for specimen CS3-20M-R.

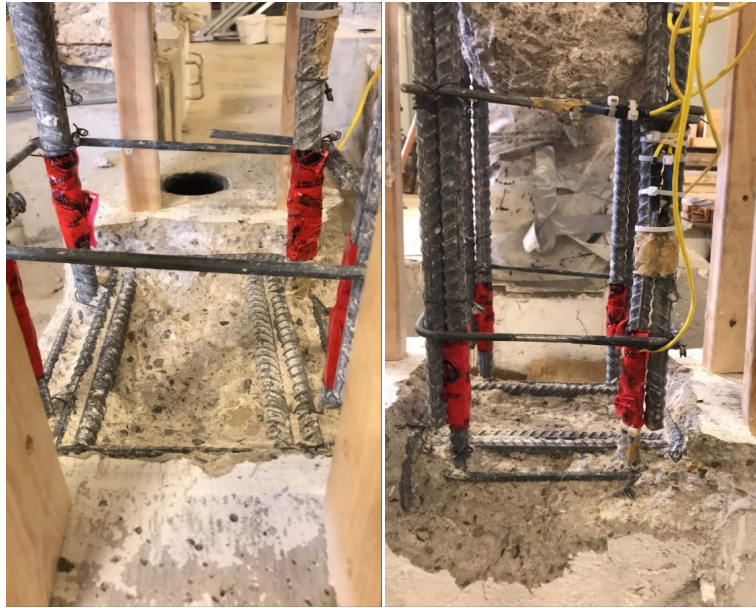


Figure C.10. Addition of tape for unbonded segments in the lap-spliced zone (Shown: CS3-20M-R).



Figure C.11. Preparation of strengthened specimens for concrete removal.



Figure C.12. Process of removal of cover and part of the core concrete for specimen CT1-15M-S.



Figure C.13. Process of removal of cover and part of the core concrete for specimen CT1-20M-S.

C.2 Specimen CT1-15M



Figure C.14. Sides of specimen CT1-15M after termination of test.

C.3 Specimen CT2-20M

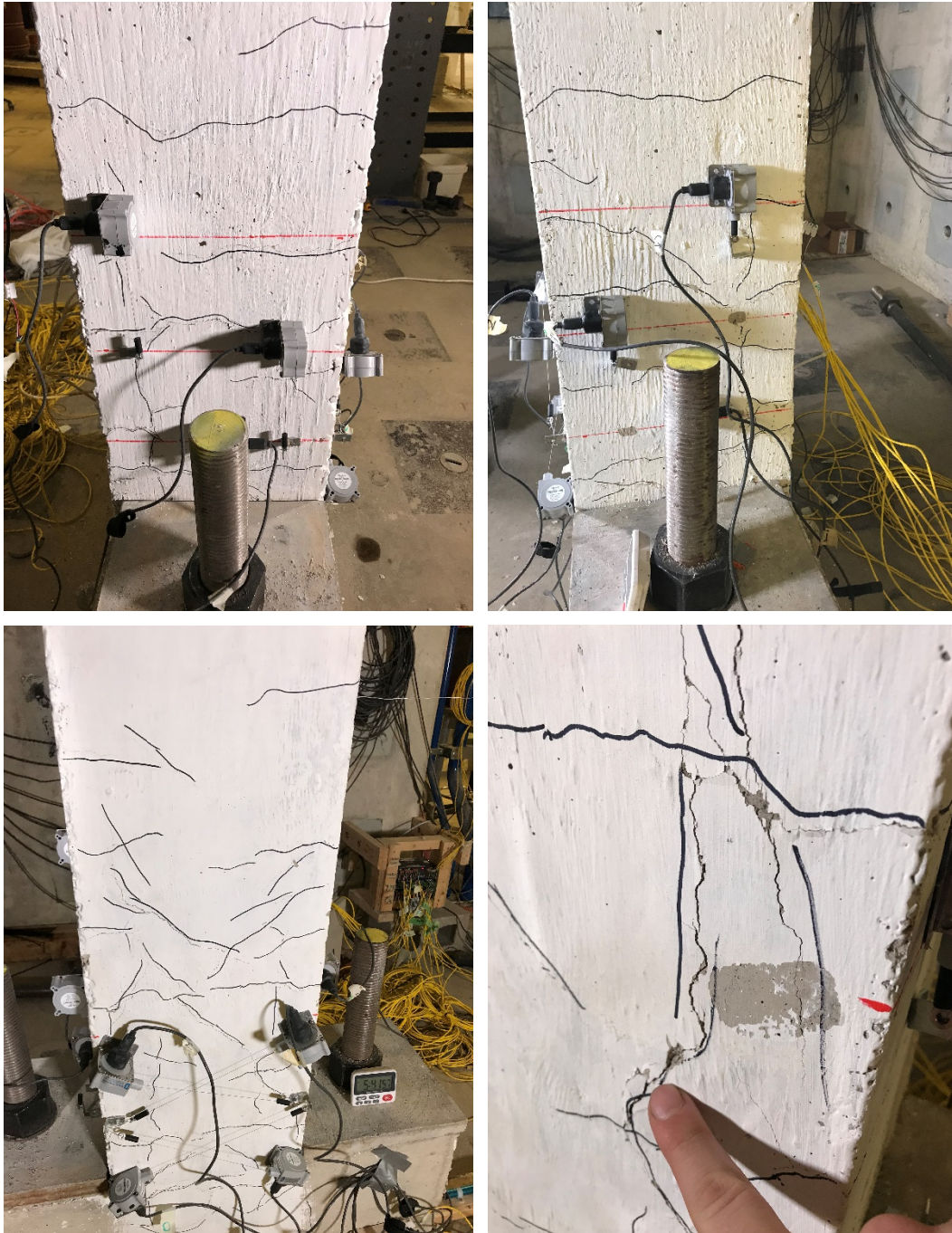


Figure C.15. Sides of specimen CT2-20M after termination of test.

C.4 Specimen CS3-15M



Figure C.16. Sides of specimen CS3-15M after termination of test.

C.5 Specimen CS3-20M



Figure C.17. Sides of specimen CS3-20M after termination of test.

C.6 Specimen CT1-15M-R



Figure C.18. Sides of specimen CT1-15M-R after termination of test.

C.7 Specimen CT1-15M-S



Figure C.19. Sides of specimen CT1-15M-SA after termination of test.



Figure C.20. Sides of specimen CT1-15M-SB after termination of test.

C.8 Specimen CT2-20M-R



Figure C.21. Sides of specimen CT2-20M-R after termination of test.

C.9 Specimen CT2-20M-S



Figure C.22. Sides of specimen CT2-20M-S after termination of test.

C.10 Specimen CS3-15M-R



Figure C.23. Sides of specimen CS3-15M-R after termination of test.

C.11 Specimen CS3-20M-R



Figure C.24. Sides of specimen CS3-20M-R after termination of test.

C.12 Tables for Column Specimens

Table C.1. Parameters for the calculation of web shear capacity (ACI 318-19).

<i>Specimen Series</i>	<i>CT1- 15M</i>	<i>CT2- 20M</i>	<i>CS3- 15M</i>	<i>CS3- 20M</i>
f_c (MPa)	25	25	25	25
λ	1	1	1	1
$\rho_{w,tension}$	0.77%	0.77%	0.77%	0.77%
D_b (mm)	16	20	16	20
<i>Spacing</i> (mm)	250	250	250	250
A_v (mm ²)	50	50	50	50
N_u (kN)	300	300	300	300
f_y (MPa)	400	400	400	400
b (mm)	300	300	300	300
d (mm)	259	257	259	257
λ_s	0.27	0.27	0.27	0.27
$A_{v,min}$ (mm ²)	65.63	65.63	65.63	65.63
V_c (kN)	56.97	56.58	56.97	56.58
V_s (kN)	41.44	41.12	41.44	41.12
V_v (kN)	98.41	97.70	98.41	97.70

Table C.2. Parameters for the calculation of web shear capacity (EN1998-3).

<i>Specimen Series</i>	<i>CT1- 15M</i>	<i>CT2- 20M</i>	<i>CS3- 15M</i>	<i>CS3- 20M</i>
f_c (MPa)	25	25	25	25
γ_{el}	1.15	1.15	1.15	1.15
ρ_{tot}	1.54%	1.54%	1.54%	1.54%
ρ_w	0.133%	0.133%	0.133%	0.133%
h_{col} (mm)	1450	1450	850	850
D_b (mm)	16	20	16	20

<i>Spacing (mm)</i>	250	250	250	250
<i>A_v (mm²)</i>	50	50	50	50
<i>N_u (kN)</i>	300	300	300	300
<i>f_y (MPa)</i>	400	400	400	400
<i>b (mm)</i>	300	300	300	300
<i>d (mm)</i>	259	257	259	257
<i>V_w (kN)</i>	18	17	18	17
<i>V_v (kN)</i>	112.84	105.73	104.62	100

Table C.3. Parameters for the calculation of web shear capacity (Pardalopoulos and Pantazopoulou 2019).

<i>Specimen Series</i>	<i>CT1- 15M</i>	<i>CT2- 20M</i>	<i>CS3- 15M</i>	<i>CS3- 20M</i>
ξ	0.265	0.265	0.265	0.265
<i>f_c (Mpa)</i>	25	25	25	25
<i>H_{cl} (mm)</i>	1450	1450	850	850
<i>D_b (mm)</i>	16	20	16	20
<i>Spacing (mm)</i>	250	250	250	250
<i>A_{tr} (mm²)</i>	50	50	50	50
ν	0.15	0.15	0.15	0.15
<i>f_y (MPa)</i>	400	400	400	400
<i>b (mm)</i>	300	300	300	300
<i>d (mm)</i>	259	257	259	257
<i>θ_v (Degrees)</i>	79.9	79.9	73.1	73.2
<i>cotθ_v</i>	0.179	0.177	0.305	0.302
<i>tana</i>	0.17	0.17	0.29	0.29
<i>V_{web} (kN)</i>	44.84	44.52	76.49	75.95

Table C.4. Parameters for the calculation of lateral load capacity of lap-spliced connection
(Pardalopoulos and Pantazopoulou 2019).

<i>Specimen Series</i>	<i>CT1- 15M</i>	<i>CT2- 20M</i>	<i>CS3- 15M</i>	<i>CS3- 20M</i>
ξ	0.265	0.265	0.265	0.265
f_c (Mpa)	25	25	25	25
μ_{fr}	1.5	1.5	1.5	1.5
L_{lap} (mm)	384	480	384	480
a_b	1	1	1	1
N_b	3	2	3	2
D_b (mm)	16	20	16	20
A_b (mm ²)	200	300	200	300
Spacing (mm)	250	250	250	250
A_{tr} (mm ²)	100	100	100	100
v	15%	15%	15%	15%
h_{col} (mm)	1450	1450	850	850
$f_{st} = f_y$ (MPa)	400	400	400	400
f_t (MPa)	2.56	2.56	2.56	2.56
f_b (MPa)	4.02	4.02	4.02	4.02
b (mm)	300	300	300	300
d (mm)	259	257	259	257
C_{lap} (kN)	464	595	464	595
C_{min} (kN)	240	240	240	240
V_{lap} (kN)	77	76	131	130