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BRIDGE PIER SEISMIC STRENGTHENING USING UHPFRC COVER

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Abstract

The paper presents the results of an experimental program aimed at developing an innovative seismic strengthening technique using Ultra-high Performance Fibre Reinforced Concrete (UHPFRC) cover to eliminate splitting failure mode in splice regions with deficient reinforcement details. With the proposed strengthening technique, the concrete around the bars in lap splice regions is first removed using conventional demolition methods, and is replaced by self-levelling UHPFRC. The exceptional tensile properties of UHPFRC allow eliminating splitting cracks thereby allowing transferring lapped bar forces through the surrounding UHPFRC. The testing program included cyclic tests on full-scale rectangular bridge pier specimens with cross-sectional aspect ratios of 2:1 and 4:1. A refined 3D finite element model at rib scale was developed to provide a numerical tool for evaluating the performance of lap splice regions strengthened by UHPFRC cover. The analyses were validated from the experimental program done at local scale on uniaxial tension specimens performed on concrete prisms.

Test results led to the following conclusions for nominally confined 24 d_b long lap splices: 1) UHPFRC cover allows eliminating the splitting failure mode; 2) drift ductility ratio up to 8 was obtained without any strength reduction for bars up to 45 mm in diameter; 3) longitudinal bar buckling failure mode was eliminated for 300 mm stirrup spacing. Results of the project also suggest that this technique could be applied at other locations in columns where reinforcement details are inadequate. The technique allows keeping the column initial dimension and could thereby be used for columns of any shape. The advanced nonlinear finite element analyses illustrated their capability for explicitly expressing the bond performance of lap splices in UHPFRC according to the measured tensile properties. These models, combined with experimental investigations, could provide precious information on the behaviour of lap spliced in UHPFRC for the future development of design guidelines.

Keywords: Seismic strengthening, Rapid repair method, UHPFRC, Bridge pier, Lap splice, Bond splitting strength

1 Introduction

1.1 Vulnerability and retrofitting of existing bridge piers

Bridges are the most critical component of the transportation network. The severe damage or collapse of several bridges worldwide observed in past earthquakes have shown vulnerability at the base of the existing bridge piers to strong seismic events. Reinforced concrete bridge piers built prior to the introduction of the first seismic design provisions around the 1980's often have inadequate design of lap splice details located at the base of the piers, where large inelastic demand is needed during a seismic event. A common design practice before the 1970's, and even until the 1990's, was to use dowel bars in footings that were lapped with continuing bars at the bottom of columns (Fig.1). Moreover, U-shaped stirrups lapped in the cover only were often used while little if any lateral support was provided for longitudinal bars along the long faces of wall or rectangular bridge piers. These lap splices lack in strength and ductility to withstand major earthquakes due to a combination of too short splice length (24 to 36 d_b) and poor confinement immediately above the footing. Different research programs have highlighted that this type of piers experience premature and sudden splitting failure along spliced length under cyclic tests [1-3]. Bond failure of spliced bars, characterised by splitting cracks within the concrete cover, is one of the commonly observed failure modes. Although modern design recommendations prohibit lap splices where plastic hinges may form during severe earthquakes, many bridges incorporating such inadequate details are still on the road network.

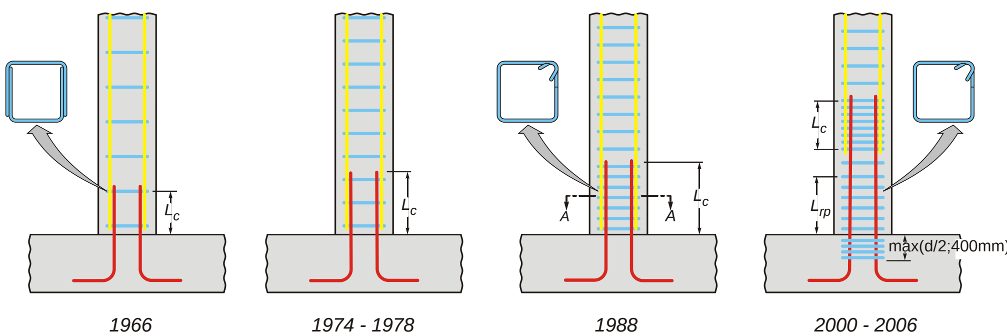
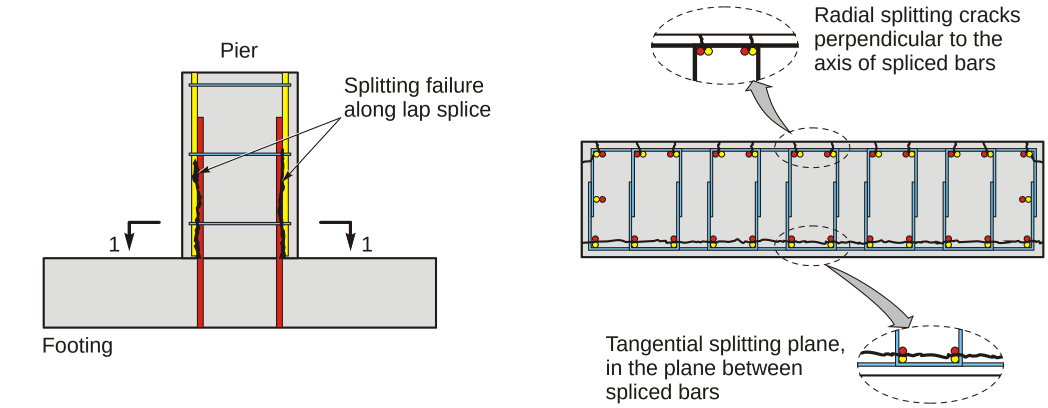


Fig. 1 – Evolution of reinforcement detailing requirements in the Canadian Bridge Code [4].

Bond action of deformed bars generates an outward radial pressure induced by the ribs. When the force generated by bond exceeds the concrete tensile stress in the concrete cover around the lap splice, longitudinal splitting cracks start to form in the concrete along the reinforcing bars (Fig.2a), leading to an abrupt failure of the existing bond [5]. The role played by splitting cracks in bond failure emphasises the importance of both the tensile properties of the concrete cover and the presence of transverse confinement in lap splice regions. Splitting cracks may form either perpendicular or in the plane of the spliced reinforcing bars according to the relative position of bars as illustrated in Fig.2b.



(a) Splitting failure along the splice length (b) Orientations of splitting cracks

Fig. 2 – Splitting failure mode in inadequately detailed lap splice region.

Several column retrofit techniques have been developed over the past decades. Most of them use passive confinement provided by external jackets (steel, reinforced concrete, precast segment or composite materials) around existing piers and are more appropriate and efficient for circular or square columns [6]. As illustrated in Fig.3, conventional jacketing solutions, applicable to circular, square or slightly rectangular ($b/h \leq 2$) columns, aim at increasing the concrete core confinement for eliminating failure modes in lap splice regions, and at providing sufficient shear capacity. However, as the aspect ratio of the section increases for wall piers, the efficiency of external jacketing decreases due to the inability to provide sufficient and uniform lateral confinement. Consequently, retrofit solutions require laborious installation of additional transverse reinforcement on existing wall piers such as prestressing strands [7] or FRP anchors [8], to eliminate the formation of splitting cracks by applying confining compressive stresses to the concrete. In this context, simpler and more cost-effective and efficient methods for retrofitting rectangular bridge piers with deficient lap splices are needed.

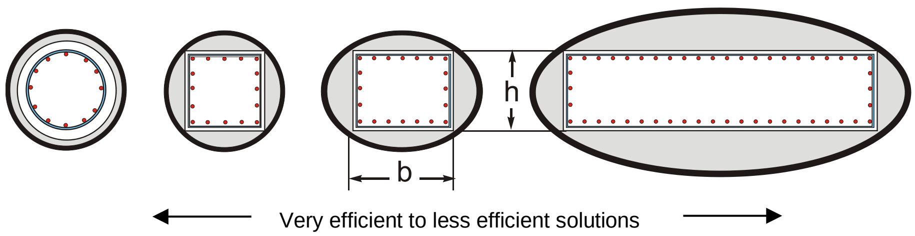


Fig. 3 – Conventional retrofitting technique with steel or FRP jackets.

1.2 Scope of the paper

The paper presents the basic principles of the proposed strengthening technique and the experimental program that has been carried out on large scale columns and locally in lapped splices with internal strain measurement inside lapped bars in UHPFRC for developing this new technique. It also introduces results of an experimental and numerical investigation aimed at studying the bond transfer mechanism in UHPFRC at the rib level. Preliminary design recommendations based on these studies are suggested.

2 Strengthening technique principle

2.1 Elimination of splitting cracks

In order to get an effective seismic strengthening technique, it is of primary importance to eliminate the brittle splitting mode of failure, thereby enabling bars to yield and provide the desired ductility. Typical seismic retrofit approaches consist of providing additional confinement at the footing and pier junction through external systems. The proposed approach consists to tackle the problem internally, directly at its source, by replacing the brittle concrete in the lap zone by a ductile material. Fig.4 illustrates the confining action around spliced bars provided by the concrete cover. Replacing ordinary concrete with brittle tensile behaviour (Fig.4b) by a high strength and ductile material in tension around spliced bars (Fig.4c) leads to a significant improvement of the confining performance and thus to an enhanced bond behaviour between lapped bars.

2.2 Proposed retrofitting technique with UHPFRC

A research project was initiated at Polytechnique Montreal in 2002 to develop this retrofitting technique for rectangular bridge piers with high aspect ratio. In the first attempt, Vachon and Massicotte [9] used conventional steel fibre reinforced concrete with 30 mm long hooked-end fibres at 80 kg/m^3. Very satisfactory results were obtained. However, the fibre size required thick cover and deep demolition of the existing concrete to expose the lapped bars. The availability of self-compacting Ultra-high Performance Fibre Reinforced Concrete (UHPFRC) by Braike [10] and Habel et al. [11] presenting exceptional tensile characteristics allowed further improving the strengthening technique.

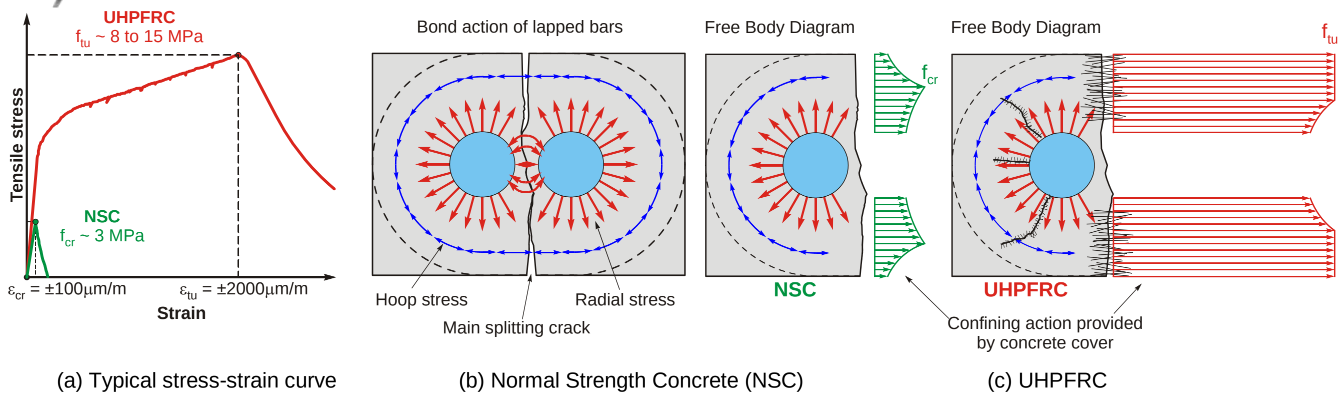


Fig. 4 – Confining action provided by concrete cover

The proposed strengthening method was applied on a large scale column with a 4:1 cross-sectional aspect ratio as presented in Fig.5 [12, 13]. With this concept, the concrete around the bars in the lap splice region is first removed using conventional demolition methods, and is replaced by self-levelling UHPFRC. This technique allows keeping the original column dimensions and can be easily applicable to any type of geometric pier section. Although the amount a demolition may appear extensive, it is a common practice for the rehabilitation of deteriorated columns. The use of UHPFRC to strengthen lap splice connections represents one of the promising applications of these materials using their outstanding tensile properties used locally in area of high stresses where the material cost versus the associated benefits is optimal.

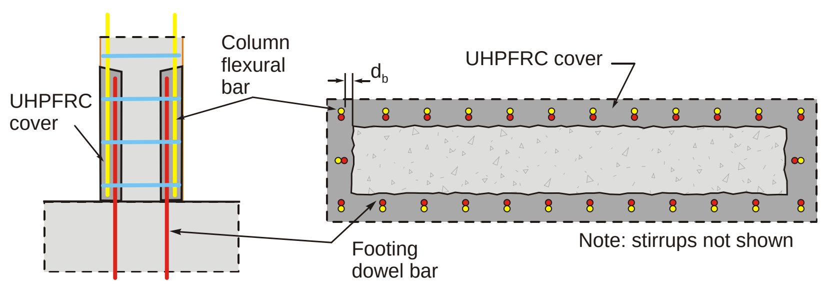


Fig. 5 – UHPFRC cover

The strengthening technique also allows adding reinforcement where needed. In existing columns, shear reinforcement is often deficient. Additional shear reinforcement can easily be added. The exceptional tensile behaviour of UHPFRC allows to use U-shape bars lapped in the cover in the long direction or adding bars through the column in the short direction. These two techniques were used efficiently in one of the test series described below. Outside the lap splice region demolition would normally be required only to expose the column longitudinal bars (half of the demolition depth required in the lap splice region in Fig. 5).

2.3 UHPFRC characteristics

Direct tensile properties measured on dog-bone specimens for different UHPFRC mixes developed at Polytechnique Montreal for this project are shown in Fig.6 [14, 15]. The tensile properties shown in Fig.6 are for well oriented fibres and represent the best expected tensile characteristics. Actual tensile properties in structural elements would necessarily be less due to fibre orientation and mix heterogeneity. Tests on beams with poorly detailed lap splices strengthened with various UHPFRC mixes carried out by Dagenais [16] showed that 2% fibre content allows bar yielding but presented moderate ductility before lap splice failure whereas the performances were not satisfactory for specimens strengthened with only 1% fibre content UHPFRC. Based on these considerations, in a highly stressed region, such as lap splice connections, it is recommended to select a high fibre dosage (at least 3%) to ensure a proper density and effectiveness of fibres regardless of the location in the element.

The selected UHPFRC mix for the piers contains 240 kg/m^3 (±3% by volume) of 10×0.20 mm straight steel fibres. This mix offers strain hardening characteristics in direct tension measured on dog-bone test reaching up to 10 MPa at strains exceeding 2000 με (Fig.6). The compressive strength typically reaches 120 MPa at 28 days and exceeds 150 MPa after a few months. This mix presents a high workability with a fresh concrete spreading exceeding 700 mm with the slump cone test, allowing casting thin elements or covers.

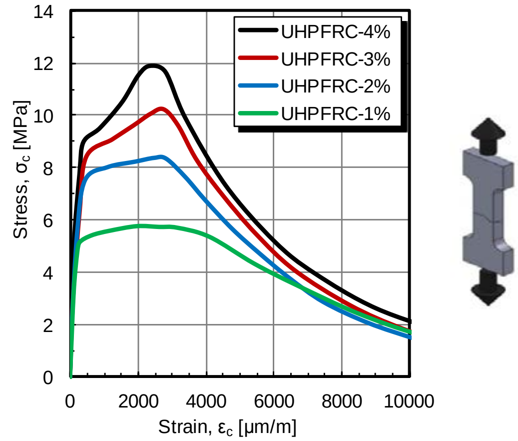


Fig. 6 – UHPFRC direct tensile strength properties

3 Experimental program

3.1 General

The experimental program included tests carried out at three scales: 1) on uniaxial tension specimens for studying the local behaviour of lap splices cast into a UHPFRC with different splice lengths, bar diameters and fiber contents in UHPFRC mixes [17], 2) on large beams subjected to monotonic or alternate cyclic loading for determining the minimum lapped length required in order to reach high ductility at bar stress beyond yielding as expected in seismic applications [16, 18], and 3) on large scale piers specimens for validating the strengthening method. Three test series were carried out on full-scale pier specimens:

- two 2 m×0.5 m×5.25 m high specimens (Fig.7a), with as built and strengthened conditions (P1 and P2 respectively), with 1.3% longitudinal reinforcement consisting of 25M bars, and loaded with respect to the weak flexural axis [12, 13];
- four 1.2 m×0.6 m×4.835 m high strengthened specimens (Fig.7b), with 1.6% longitudinal reinforcement consisting of 25M, 30M, 35M and 45M bars (S1 to S4 respectively), and loaded with respect to the weak flexural axis [14];
- two 1.2 m×0.6 m×4.150 m high strengthened specimens (Fig.7c), with 1.6% longitudinal reinforcement consisting of 35M bars, with different shear reinforcement configurations (S5 and S6), and loaded with respect to the strong flexural axis [19].

3.2 Specimen fabrication details

Specimens were fabricated according to existing bridge details commonly used before the introduction of modern seismic design requirements. Footing and column bars are lapped at the column base over a length equal to 24 bar diameter, a common value used up to the 1960's. Shear reinforcement was spaced at 300 mm in the lap splice regions and was overlapped in the concrete core of column faces for some specimens (P1, P2, S6), also a common detailing practice for rectangular columns. The specimen nominal concrete strength was 30 MPa while the nominal steel yield strength was 400 MPa to be representative of existing columns.

The concrete cover and the concrete around lapped bars of strengthened specimens were removed by using jackhammer, following conventional repair practice. Tests on beams by Dagenais and Massicotte [16, 18] indicated that the optimum demolition depth behind innermost bars must be at least one bar diameter in splice regions. If shear strengthening is required outside splice regions (Fig.7c), removing concrete just to expose the longitudinal reinforcement was judged sufficient. UHPFRC cover was placed vertically to reproduce actual field conditions in term of fibre orientations and compaction. A self-levelling UHPFRC is required. Further investigation after the tests showed that the concrete completely surrounded the bars.

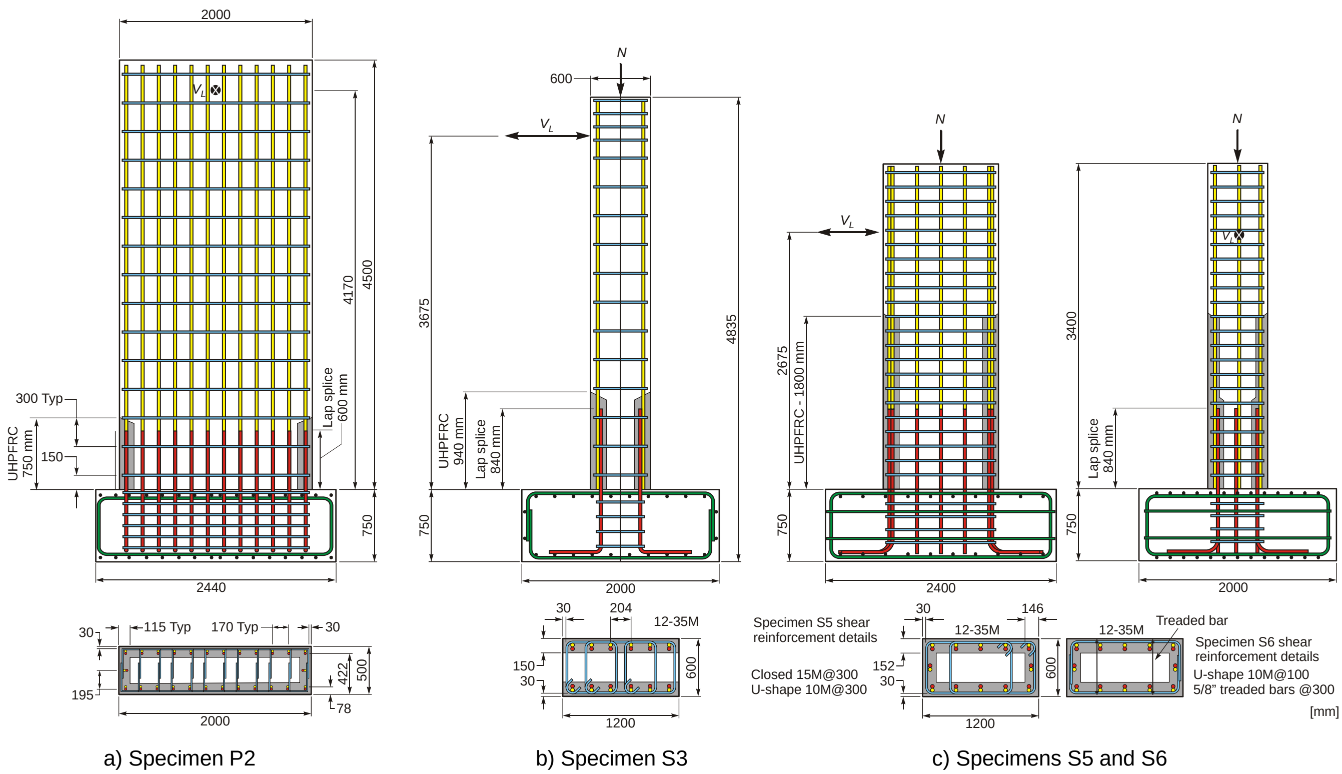
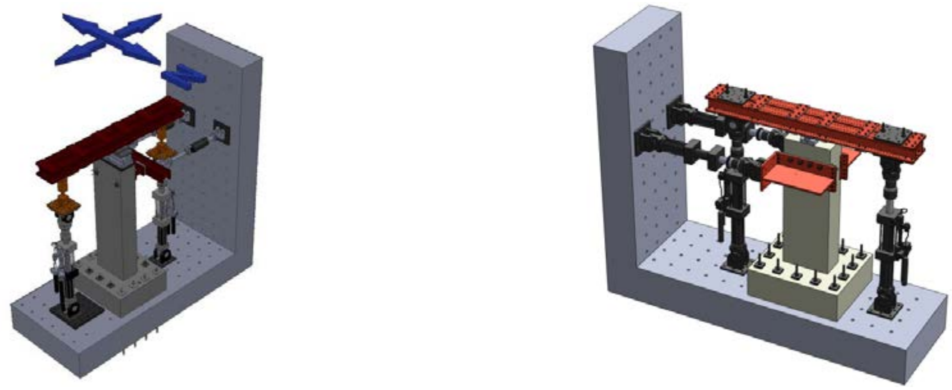


Fig. 7 – Specimen dimensions

In all cases shear reinforcement was designed to resist the shear forces associated with the flexural capacity. Due to the higher shear forces in specimens S5 and S6 loaded with respect to the strong axis (higher flexural strength and lower height of the horizontal force), additional stirrups were added compare to specimen P1, P2, and S1 to S4. In specimen S5 (Fig. 7c), two sets of closed 15M stirrups spaced at 300 mm and pairs of U-shape 10M stirrups alternately spaced at 300 mm were provided. In specimen S6, using the exceptional characteristics of the UHPFRC cover, only pairs of U-shaped stirrups spaced at 100 mm, lapped on the column short faces were used. Pairs of ties at 300 mm were added to maintain a minimum confinement.

3.3 Loading

For specimens P1 and P2, no vertical load was applied to allow isolating the flexural failure mode and measure the contribution UHPFRC cover. For the other specimens, a constant axial load of 1500 kN corresponding to 6%$A_g f'_c$ was applied using two 1000 kN vertical actuators (Fig.8). The applied axial load level typically corresponds to gravity loads acting on large rectangular bridge piers. The specimens were submitted to a quasi-static cycle loading with increasing displacement. The lateral force was applied by two actuators 500 kN actuators for specimen S1 to S4 and by two 1000 kN actuators for specimen S5 and S6, all controlled in displacement.



a) Specimens S1 to S4 – Weak flexural axis loading b) Specimens S5 and S6 – Strong flexural axis loading

Fig. 8 – Column test set up

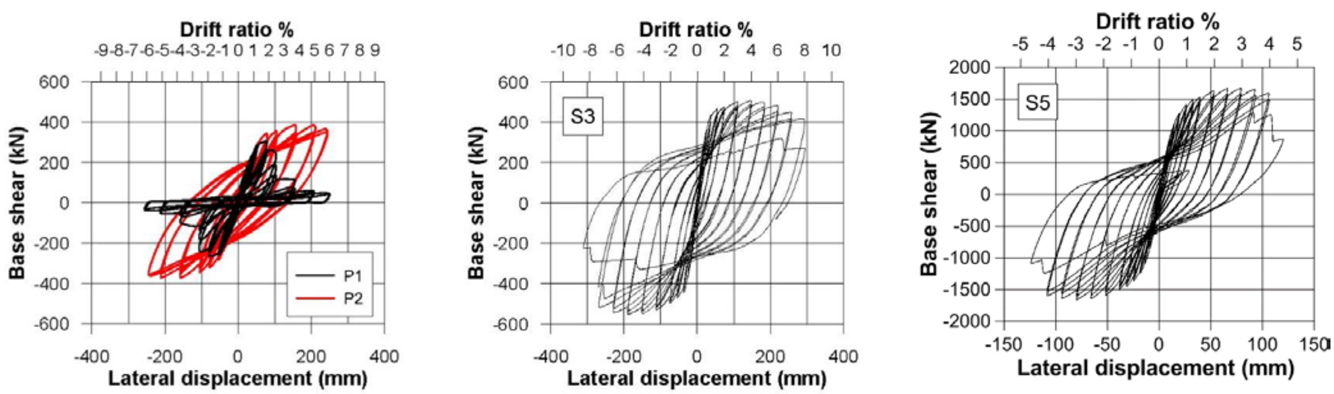
4 Results and discussions

4.1 As built specimen

The load displacement response of specimen P1 is shown in Fig.9a. At a displacement corresponding to a displacement ductility ratio μ_Δ of 2 (where μ_Δ is defined as the ratio of the lateral deflection to the deflection at yielding, $\mu_\Delta = \Delta/\Delta_y$), the specimen showed a rapid loss of strength. Locally, it exhibited the formation of splitting cracks in the lap splice region that led to a drastic loss of bond between reinforcing bars and concrete followed by sliding of the lapped bars, leading to a brittle failure without any sign of ductility. This behaviour is typical of bridge piers designed mainly for gravity loads built before the introduction of modern seismic requirements [20].

4.2 Lateral load-displacement hysteretic response of strengthened specimens

Lateral load-displacement hysteresis responses of strengthened specimens P2, S3 and S5 are shown in Figure 9. All retrofitted specimens exhibited an outstanding hysteretic response with high energy dissipation. For specimens S3 and S5, corrections were made to the applied horizontal load to remove the contribution of the vertical actuators in order to obtain the actual base shear values.



a) Specimens P1 and P2 – No axial load b) Specimen S3 – 1500 kN axial load b) Specimen S5 – 1500 kN axial load

Fig. 9 – Behaviour of specimens loaded with respect to the weak flexural axis

For specimen P2 no failure was observed. As shown in Fig.9a, the specimen did not exhibit any sign of deterioration up to a displacement ductility ratio μ_Δ of 5 at which the test was interrupted due to limitation of the test set up used at that occasion. Modifications to the loading configuration allowed carrying out a pushover test.

The applied load reached 367 kN at a displacement of 365 mm, which corresponds to $\mu_\Delta > 7$. The actuators used in the test set up for specimens S1 to S6 allowed pushing the specimens to failure.

All specimens displayed approximately a linear response until first yielding, which was then followed by stable nonlinear symmetrical loops. The lateral force displacement responses of all strengthened specimens are characterised by wide and stable loops, indicating the ability of the columns to efficiently dissipate energy. All specimens were able to sustain peak load for several cycles.

4.3 General behavior and failure mode

The first flexural crack in all strengthened specimens developed at the column-footing interface. As the lateral load increased, some flexural cracks developed along the height of the column in the normal concrete above the repaired zone (P2 and S1 to S4) whereas no flexural cracks were observed in specimen S5 which had UHPFRC up to the bottom of the loading plates (Fig.7c and Fig.8b). Only localised and very fine flexural cracks (<0.1 mm) were present in the UHPFRC zone. They were due to the restrained shrinkage of UHPFRC and did not progress further during the entire duration of the tests. For all specimens, most of the visible plastic deformations were located at the base of the column with one major opening forming at the footing-column interface.

The failure of all the specimens was progressive and ductile as they all failed due to the progressive rupture of the dowel bars in the footing. Specimens were able to sustain a large number of cycles, up to a displacement ductility ratio μ_Δ of 8 and, most importantly, the splitting mode of failure was eliminated. Although transverse reinforcements with proper seismic detailing were spaced at 300 mm, no longitudinal bar bucking was observed. Moreover, there was no concrete cover spalling and the column integrity was maintained intact throughout the test. UHPFRC was effective for eliminating or substantially diminishing the apparition of splitting cracks. Eliminating these failure mechanisms that occur at high ductility ratio allowed the strengthened specimens to perform better than similar specimens designed according to modern seismic requirements tested by Khaled et al. [21].

After testing, two columns were cut 50 mm above the footing to observed the extent of damage in the UHPFRC around the spliced bars at the location where the highest strains develop in the dowel bars and where UHPFRC is the most highly stressed. Although some cracking was observed, the visual inspection confirmed that the cover was still efficient for transferring forces between lapped bars. It can be concluded that the action of a UHPFRC containing an appropriate volume of fibres is such that lapped bars behave as if they were welded with little, if any, degradation of the concrete on the column side. Hence, most if not all, plastic deformations occurred in the starter bars mostly into the footing. Although visible information indicated that plastic deformation occurred at the column to footing cold joint, further analysis of the experimental data indicated that little deformation occurred in the bars columns whereas the plastic hinge region extended within the footing [14], suggesting that this zone must be adequately designed. For all specimens tested in this research program, footing reinforcement details were those required by the Canadian Bridge Code [4] for new constructions.

4.4 Weak vs strong flexural axis behavior

Specimens P2 and S1 to S4 were designed to study the flexural behaviour in conditions requiring high ductility demand, and large height to width ratio were selected (H/D = 8.34 and 6.13 respectively – See Fig. 7). Specimens S5 and S6 were designed to further develop the strengthening technique in conditions dominated by shear with an H/D ratio of 2.23, more typical of wall type bridge piers. Ductility demand in this case is expected to be less than for higher H/D values. As shown in Figures 9b and 9c, the deflection ductility ratio μ_Δ reached values of 8 for specimens S1 to S6 (S3 and S5 being typical) before any visible or measurable damage occurred whereas the drift ratio exceeded 6% for specimens S1 to S4 loaded with respect to the weak axis with high H/D ratio, and exceed 4% for specimens S5 and S6 loaded with respect to the strong axis with lower H/D ratio.

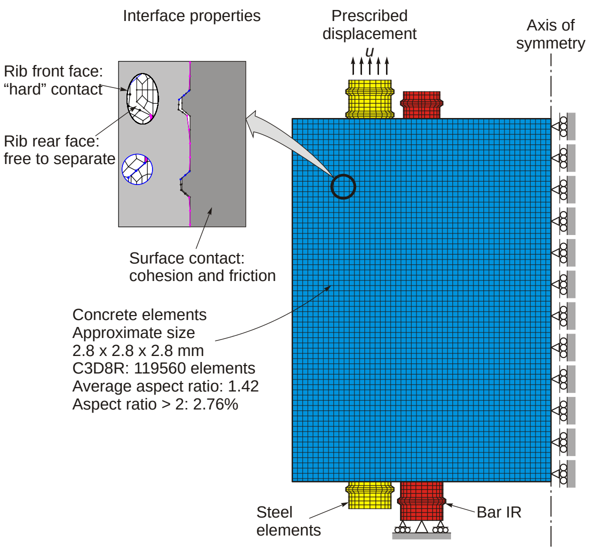
In all cases no shear crack were observed. For specimen S6 with an inappropriate stirrup configuration, UHPFRC prevented the transverse bars to open. No failure was observed in the UHPFRC cover except at the very final loading stage of the specimens loaded in the strong flexural axis (S5 and S6) where localised cracking occurred at the cross section corners without, however, producing any reduction of the shear forces.

5 Numerical evaluation of bond strength of lap splice in UHPFRC

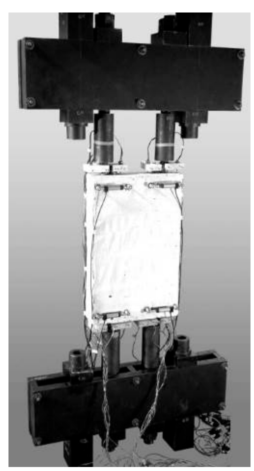
As part of this research project, a refined 3D finite element model at rib scale was developed to provide a numerical tool for evaluating the performance of lap splice regions strengthened by UHPFRC cover [22] (Fig. 10a). Numerical simulations were carried out using the 3D concrete constitutive model EPM3D [23] implemented in Abaqus/*Explicit* [24]. The numerical results were validated from the experimental program done at local scale on uniaxial tension specimens [25]. These experimental tests were performed on concrete prisms, representing an isolated portion of UHPFRC cover of a retrofitted bridge piers in which concrete and steel reinforcing bars are jointly subjected to tension (Fig. 10b). With stress distributions along splice length obtained from internal measurement in reinforcing bars (Fig. 10c), this experimental program constitutes a database reference for nonlinear FE modeling of bond with UHPFRC.

The rib scale model shows very good agreement between test results and numerical analysis in terms of the steel stress distribution along the spliced bars (Fig. 11a). The splitting failure mode are well captured by the numerical model, and the cracking patterns at failure load are accurately depicted by the numerical analyses (Fig. 11b). The analyses indicated that with the modelling developed methodology, the bar slip response, concrete surface transverse strain and splitting crack opening can be accurately predicted [22]. This demonstrates that advanced nonlinear finite element analyses are capable of explicitly expressing the bond performance of lap splices in UHPFRC according to their measured tensile properties.

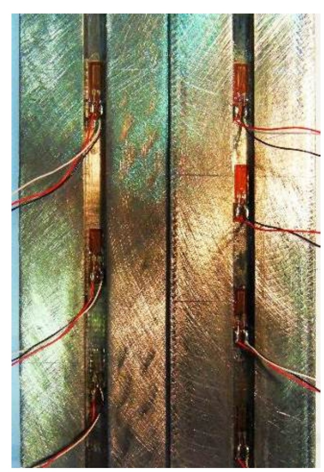
However, the numerical results [22] highlighted one of the most important challenges in UHPFRC design; the selection of the actual tensile properties in the structures in terms of both strength and deformability, accounting for fibre orientation and dispersion. Comparison between experimental and numerical results showed that using the tensile properties directly taken from material characterisation specimens strongly overestimates the lap splice strength. With reduced tensile properties, the rib scale model accurately described the steel stress distribution and was capable of explicitly expressing the bond performance of lap splices in UHPFRC.



a) FE Model

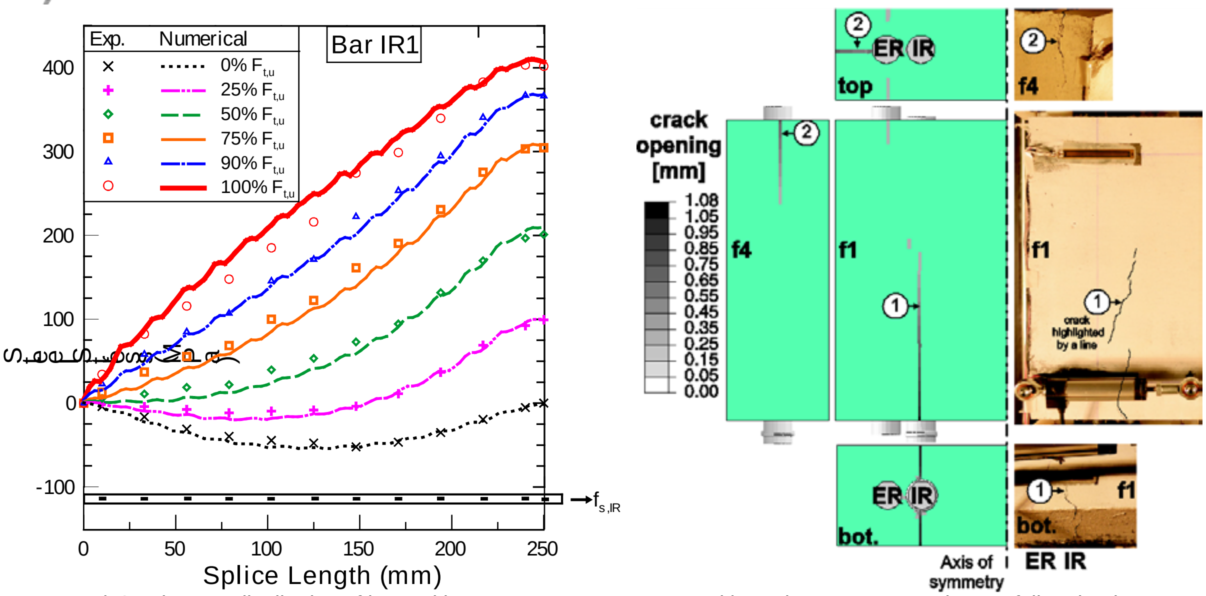


b) Test set-up



c) Internal instrumentation

Fig. 10 – Numerical investigation and experimental validation



a) Steel stress distribution of internal bar IR1 b) crack pattern comparison at failure load

Fig. 11 – Comparison between numerical and experimental results, UHPFRC-4%, d_b = 25 mm, l_s = 10 d_b

It is known that bond is a complex phenomenon influenced by many parameters and geometrical conditions. Using a validated 3D nonlinear finite element model, the influence of splice length and UHPFRC cover thickness can be evaluated. The numerical study illustrated the interest in the future for using validated nonlinear 3D finite element models at the rib scale level to provide a strong basis for establishing rational design criteria for anchorages and lap splices in UHPFRC connections.

6 Conclusions

The behaviour of large-scale bridge pier specimens having deficient lap splices strengthened with UHPFRC covers was investigated. The specimens were built according to the code requirements used prior to the 1970's. All the specimens were tested under constant axial load and incrementally increasing lateral displacement cycles. Based on the testing program results (failure modes, hysteresis behaviour, curvature distribution), the following conclusions can be drawn.

- UHPFRC can be used to strengthen deficiently detailed lap-splice zones.
- Bond failure was successfully eliminated on 24 d_b deficient lap splices using 3% fibre content UHPFRC for bar diameters ranging from 25 to 45 mm.
- Specimen behaviour was ductile and the progressive failure was caused by the dowel bars tensile rupture in the footing.
- Although the transverse reinforcement with seismic details was spaced at 300 mm, no longitudinal bar bucking was observed.
- There was no concrete cover spalling and the column integrity was maintained intact throughout the test.
- Most of the plastic deformations associated with the formation of a plastic hinge were concentrated at the base of the column and extended within the footing.
- All specimens were able to sustain several cycles and reached displacement ductility levels exceeding 7, for H/D ratios ranging from 2.2 to 8.3.

- With the proposed strengthening technique, the brittle ordinary concrete is substituted by a ductile material which eliminates, or significantly diminishes, the mechanical weaknesses of concrete associated with bond development.
- Contrary to column jacketing, the proposed method allows to keep the original column shape.
- UHPFRC tensile strength and tensile ductility are the most important material characteristics for lap splice strengthening.
- Surrounding lapped bars with appropriately designed UHPFRC can be considered equivalent to bar welding where in well-designed lap splice, bars can reach their yield strength or even be stressed up to their ultimate strength and break outside the UHPFRC connection as if the bars were welded.
- The proposed retrofit technique was effective on rectangular cross section with high aspect ratio but could also be an interesting alternative for columns of any shape.
- The results also suggest that this technique could be applied at other locations along bridge columns where the existing reinforcement is inadequately detailed.

This research project demonstrated clearly the potential of using UHPFRC to efficiently retrofit deficient lap splice connections in existing structures The findings of the project are limited to UHPFRC with tensile properties similar those of the material used. A generalization to material with other characteristics is needed to broaden the scope of applicability of the technique. Finally, the enhancement of the confinement provided by the UHPFRC cover and existing transverse reinforcement, and their effects of the shear strength, still remains to be studied.

Acknowledgements

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