# Structural Implications of the Synergistic Interactions between Steel Reinforcement and UHPC

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**Abstract**

The excellent mechanical properties of ultra-high-performance concrete (UHPC) can revolutionize the design of reinforced concrete structural members. However, UHPC material properties were typically measured without including longitudinal steel reinforcement while the synergistic interaction of UHPC and steel reinforcement can significantly alter UHPC material properties. Experimental studies indicates that UHPC's tensile ductility can be significantly enhanced due to the presence of longitudinal reinforcement. In addition, while concrete's cracking strength can hardly be considerably increased in a typical reinforced concrete member, increasing the longitudinal reinforcement ratio by a significant amount can enhance the cracking strength of UHPC up to five to eight times that of plain concrete. The reason behind this is that when the longitudinal reinforcement ratio surpasses a critical threshold, the longitudinal reinforcement can efficiently share tensile force and halt the growth of microcracks in concrete, preventing them from connecting and forming a percolation crack. Allowing a higher steel ratio also leads to smaller tensile and bond stresses in tensile reinforcement. Because crack widths in a flexural member are roughly proportional to the stress in steel reinforcement, the low stress will allow better control of the crack width and, hence, stiffness of the member. Notably, UHPC's maximum compressive strain can reach about five times higher than that of plain concrete. This high compressive strain capacity allows a concrete member to maintain tension-controlled behavior even with a high amount of steel rebars. Neglecting the greater compressive ductility of UHPC can limit the maximum permissible quantity of longitudinal reinforcement, resulting in the early rupture of longitudinal reinforcement and restricted flexural capacity.

**Keywords:** UHPC, Tension-Controlled, Maximum Compressive Strain, Cracking Strength.

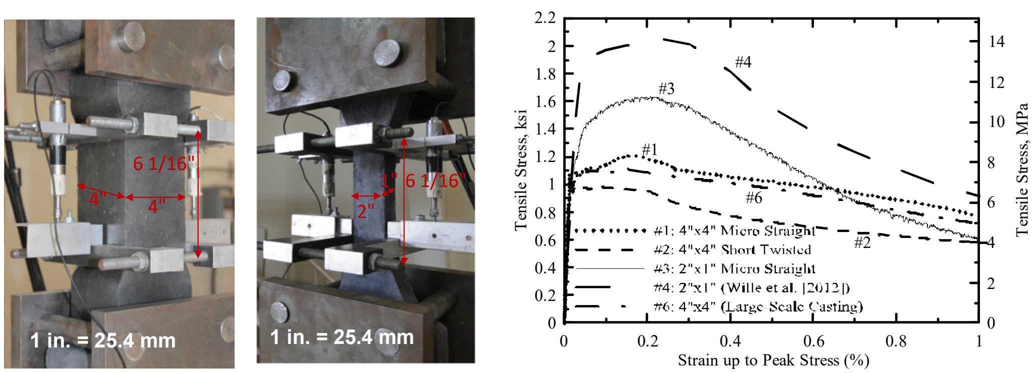
## 1. Introduction

Ultra-high-performance concrete (UHPC) possesses outstanding mechanical properties that have the potential to revolutionize the design of reinforced concrete structural members. However, the evaluation of UHPC material properties often excludes longitudinal steel reinforcement, and the deformation capacity of UHPC is typically not considered in the design of steel reinforcement for reinforced UHPC structural members. By considering the synergistic interactions between UHPC and continuous steel reinforcement, it is possible to effectively utilize the exceptional material properties of UHPC and develop a new structural design approach for reinforced UHPC members that differs from the conventional method. The paper addresses three key aspects related to the synergistic interactions between UHPC and steel reinforcement.

## 2. Size Effect and Effect of Continuous Steel Reinforcement on Tensile Response of UHPC

To determine the fundamental tensile behavior of UHPC materials, it is essential to conduct direct tensile tests as the macro-scale properties of UHPC are dependent on their stress-strain characterization under tension (Naaman, 2017). Tensile tests carried out on relatively smaller-sized specimens may not account for more realistic fiber distribution and content variability in full scale structural applications. A review of previous research on UHPC tensile tests has indicated that smaller-sized tensile specimens typically exhibit greater values of peak tensile stresses and strains (Naaman and Shah, 2022). Figure 1 shows a study comparing smaller- and larger-sized UHPC tensile specimens (Aghdasi et al., 2016). It is observed that the smaller-sized tensile specimens obtain significantly higher tensile strengths (30 to 60% higher) and moderately increased hardening response when compared to that of the larger-sized specimens. Therefore, when applying to actual structural members, caution should be used when interpreting tensile properties based on the testing of smaller-sized specimens.

The size effect on ultimate tensile strength of quasi-brittle materials is well-known (e.g., Carpinteri et al., 1995) and can be explained by the crack band theory using fracture mechanics in terms of energy balance between the released energy and the energy required for the crack band extension, where the released energy is proportional to the size of the quasi-brittle member (Bažant and Planas, 1997). As a result, fracture mechanics predicts that the critical stress leading to the fracture of concrete decreases as the size of geometrically similar members increases. It is believed that, although the fracture toughness of UHPC is much greater than that of plain concrete, the greater released energy in larger-sized UHPC tensile specimens also causes size effect due to the same reason.




(a) (b) (c)

**Figure 1: (a) large-scale tensile specimen (16.0 in.$^2$ [10,323 mm$^2$]), (b) small-scale tensile specimen (2.0 in.$^2$ [1,290 mm$^2$]), and (c) tensile stress-strain curves of UHPCs with different specimen sizes**

Additionally, the majority of the UHPC tensile test specimens did not consider the synergistic effects of continuous steel reinforcement, which is present in actual structural elements. Figure 2 presents a previous study that examined the tensile behavior of strain-hardening FRC using both smaller and larger-sized specimens, the latter of which included a continuous prestressing strand.

It was observed that the strain capacity of a strain-hardening FRC was considerably enhanced (Figure 2(c)). Figure 2(d) shows that the specimen with no continuous reinforcement developed a smaller number of cracks prior to the onset of damage localization. On the other hand, the larger-sized specimen with continuous reinforcement developed extensive multiple cracks, and no significant damage localization was observed up to about 0.8% strain.

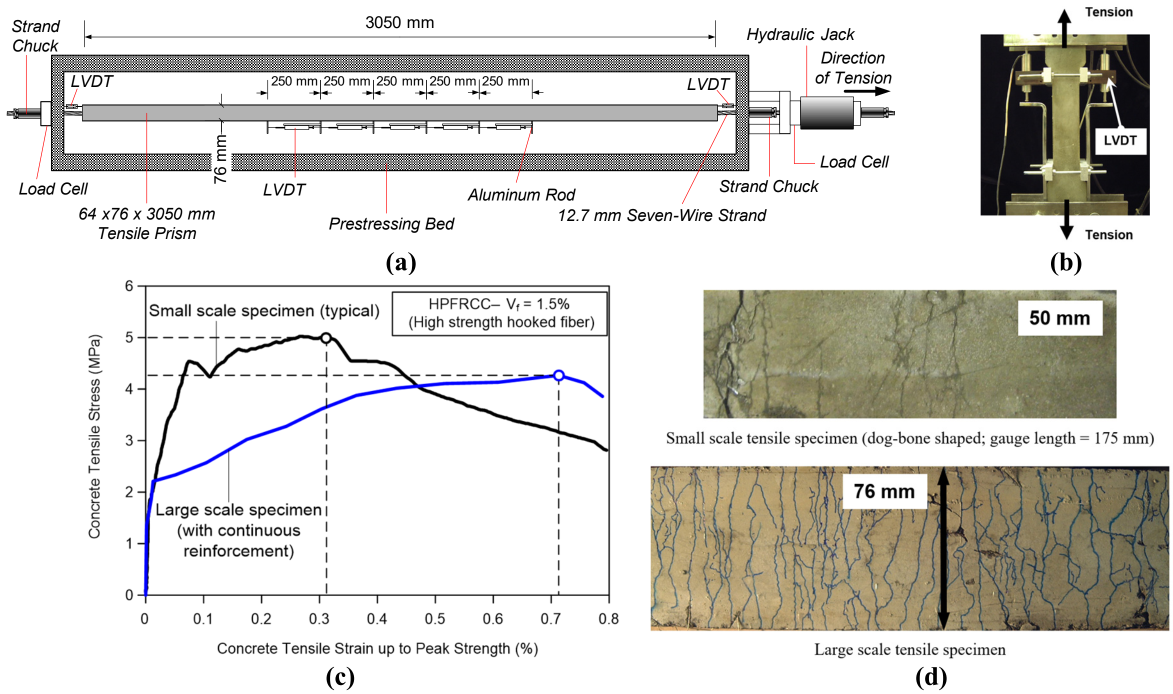



**Figure 2: (a) larger-sized tensile specimen (with continuous steel reinforcement), (b) smaller-sized tensile specimen, (c) comparison of stress-strain responses between smaller- (no continuous steel reinforcement) and larger-sized specimen, (d) crack distributions in smaller- and larger-sized tensile specimens (Chao et al., 2007)**

Aghdasi et al. (2016) also reported the same behavior in UHPC reinforced with a rebar, as shown in Figure 3(a). Figure 3 (b) depicts the tensile stress-strain curves for reinforced UHPC (blue) and unreinforced UHPC (green). Despite both having a comparable peak tensile stress, the inclusion of a rebar significantly increased the tensile ductility of UHPC, maintaining its strain-hardening behavior up to a strain of 1.3%. Compared to the specimen without a rebar, the specimen with a rebar demonstrated an approximately 7.5 times higher strain capacity.

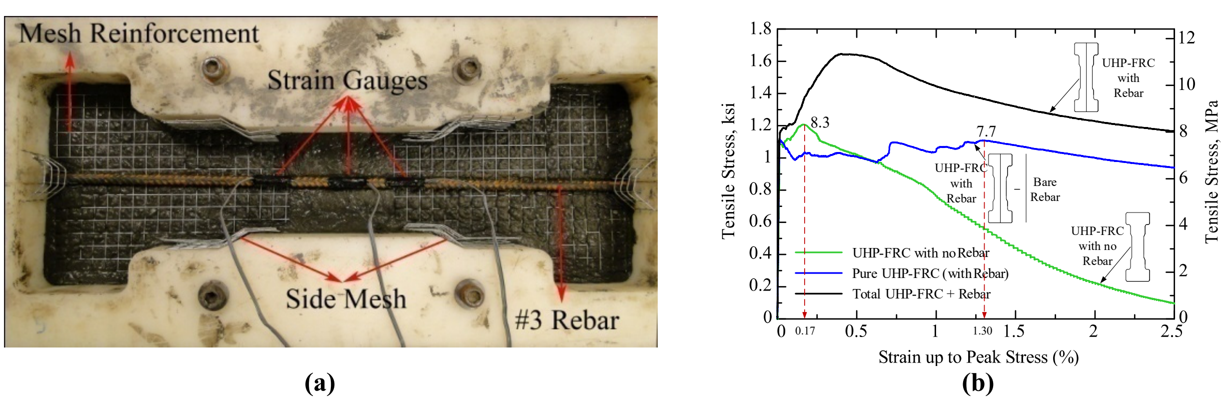
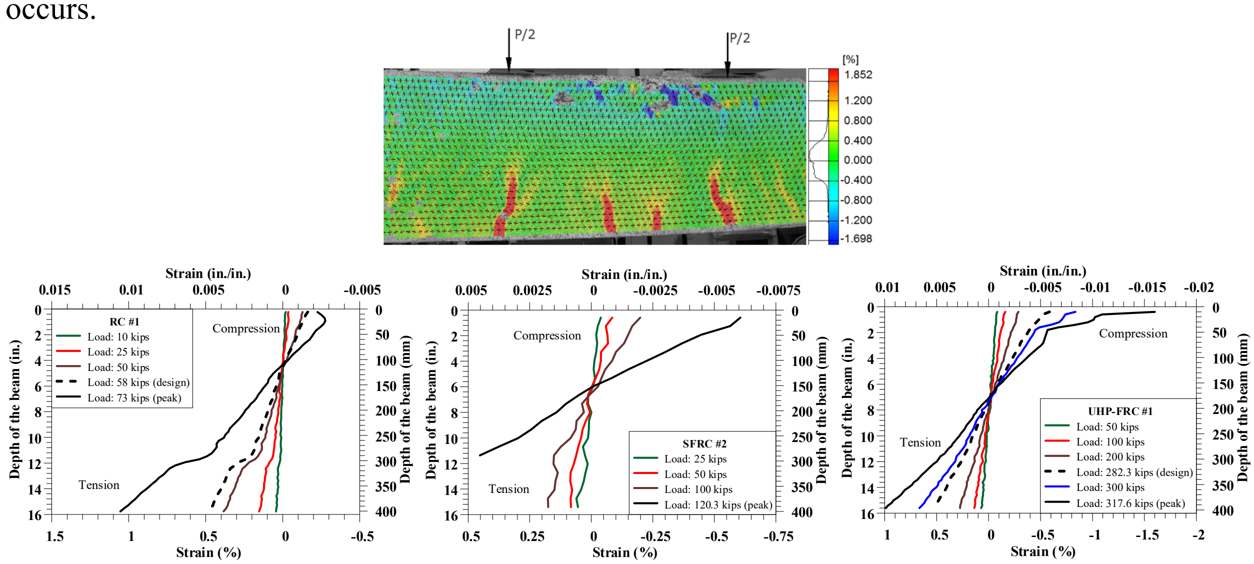



**Figure 3: (a) the tensile specimen with one #3 rebar, (b) tensile stress-strain curves**

This enhanced ductility is believed to occur because of the stabilizing effect the continuous reinforcement has on the microcracking process, as well as its ability to delay localization. Consequently, it is justifiable to increase the expected ductility levels for design when strain-hardening UHPC is reinforced by rebars or prestressing strands in structural applications.

## 3. Large Deformation Demand in Steel Reinforcement due to UHPC Compressive Ductility

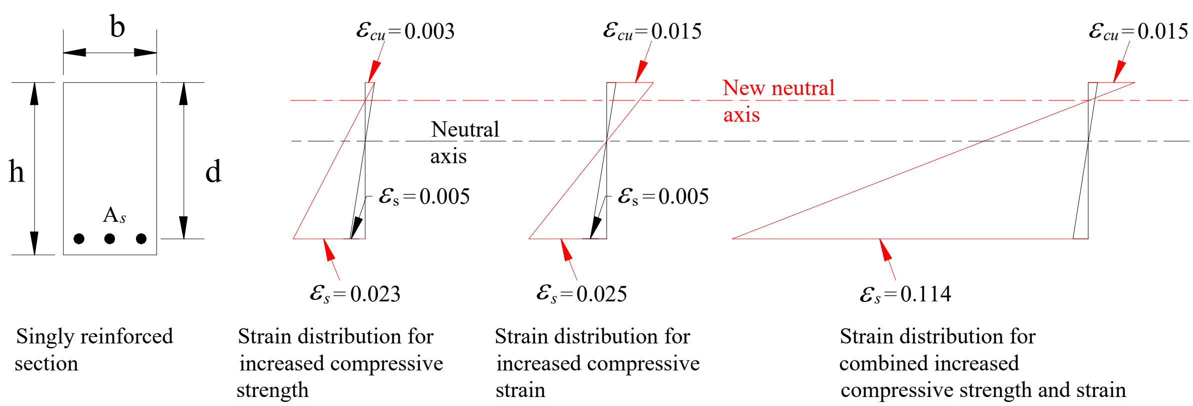
While UHPC is known for its high compressive strength compared to conventional concrete, the addition of steel fibers also considerably increases its compressive strain capacity. Figure 4 shows strain distributions of large-scale steel rebar reinforced beams made of plain concrete, conventional FRC (0.75% steel hooked fibers by volume), and UHPC (3% straight smooth steel fibers by volume). Theses strains were measured by a non-contact digit image correlation (DIC) system with a measuring strain accuracy of 0.01% (in./in.). These strains represent an average of all strains within a 10-in. (254 mm) gauge length within the constant moment region. It is evident from the data that the maximum compressive strain, $\varepsilon_{cu}$, before plain concrete crushing occurred was approximately 0.003, which is consistent with the maximum design strain recommended by both the ACI 318 code (ACI, 2019) and AASHTO LRFD Specifications (AASHTO, 2020). The addition of 0.75% steel fibers resulted in a moderate increase of $\varepsilon_{cu}$ to 0.006. On the other hand, UHPC containing 3% steel fibers achieved a maximum compressive strain of 0.0015, which is 5 times the maximum compressive strain of plain concrete. Due to the small strain capacity of plain concrete, only a small amount of longitudinal reinforcement is allowed by the codes in order to ensure that the flexural member is tension-controlled. For a tension-controlled beam section, the tensile strain in the extreme tension reinforcement (closest to the tension face) must be sufficiently large ($\geq 0.005$); therefore, the beam shows a large deflection as a warning before concrete crushing occurs.



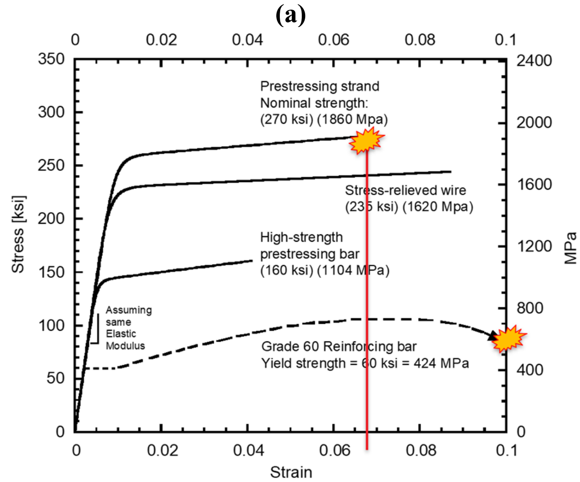

(a) (b) (c)

**Figure 4: Average measured strains at various loads for the test beams by digit image correlation (DIC) measurement: (a) plain concrete, (b) conventional FRC (0.75% $V_f$), and (c) UHPC (3% $V_f$)**

However, if the steel reinforcement for a UHPC beam is designed based on the assumption that UHPC's maximum compressive strain is 0.003, the combination of high compressive strength and strain results in a shallow neutral axis depth and an excessive demand for tensile strain in the steel reinforcement. Figure 5(a) provides an illustration, while Figure 5(b) demonstrates that the strain demand may surpass the typical prestressing strands and rebars' strain capacities by a significant margin. As a result, the steel reinforcement may rupture prematurely before the concrete reaches its crushing strain. Therefore, it is recommended to be cautious when designing reinforced UHPC using traditional assumption ($\varepsilon_{cu}$ = 0.003) for reinforced concrete structural members. Specifically, designing UHPC beams with a fiber volume fraction in the range of 2% to 3% and without a cast-in-place composite deck made of conventional concrete requires special attention. On the other hand, if a cast-in-place composite deck is constructed using conventional concrete, employing UHPC for the beam is not cost-effective because the beam cannot fully utilize UHPC's high compressive strength.




**(a)**



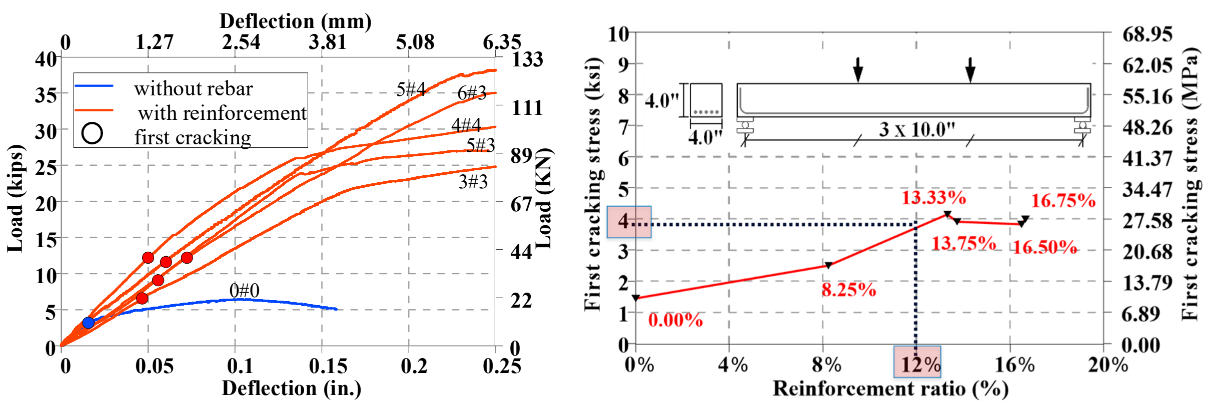

**(b)**

**Figure 5: (a) tensile strain demands in the longitudinal steel reinforcement caused by the maximum compressive strain of UHPC, (b) strain capacities of typical prestressing strands and rebars**

## 4. Enhancement of Stiffness and Resistance to Cracking, Bending, and Shear in UHPC Beams through High Longitudinal Steel Reinforcement Ratio

According to the discussion above, for UHPCs with high compressive strain capacity, a beam can be used more efficiently by including a significantly greater quantity of longitudinal reinforcement while still ensuring that the behavior remains tension-controlled. (Figure 5(a)). Moreover, prior research has shown that the presence of reinforcing bars in structural members enhances the cracking distribution and tensile ductility of strain-hardening FRCs due to the tension-stiffening effect (Chao et al 2007; Aghdasi et al 2016). Therefore, the addition of a significant quantity of longitudinal reinforcement not only considerably increases the flexural strength of a UHPC beam but also enhances the UHPC's resistance to cracking on the beam's tensile side.

To investigate the impact of reinforcement ratio on the flexural cracking strength, $f_r$, a series of small UHPC beams were subjected to a four-point loading test, as demonstrated in Figure 6(a). The constant bending moment region between the two loading points is 10 in. (250 mm). One of the specimens, 0#0, had no reinforcement and rest of them were reinforced. The longitudinal reinforcement ratio, $\rho_{TA}$, is defined as the area of the longitudinal reinforcement to the concrete tributary area which is the product of the width of the beam by twice the cover (bottom concrete fiber to the centroid of the reinforcement). In all specimens, the flexural cracking occurred prior to shear cracks. Conservatively, the flexural cracking strength was calculated based on the minimum of 1) the loading when the first crack was observed and 2) the first deviation point in the load-deflection diagram. Figure 6(b) summarizes the first cracking load for each specimen. Test results show that when $\rho_{TA}$ approaches to approximately 12%, the first flexural cracking strength increases by 285%, from 1.4 ksi (9.9 MPa) to 4 ksi (27.6 MPa). The substantial resistance to cracking results in an outcome comparable to that achieved with conventional prestressing (Naaman and Chao, 2022), elevating the first cracking strength of plain concrete (which typically falls within the range of 0.5 ksi to 0.75 ksi [3.4 MPa to 5.2 MPa]).




**(a)** **(b)**

**Figure 6: (a) test results, (b) first cracking stress versus reinforcement ration ($\rho_{TA}$ = rebar area divided by the "tributary area of concrete" = 2 × cover thickness × width of beam)**

While plain concrete's cracking strength can hardly be considerably increased in a typical RC member, this study shows that a high amount of reinforcement ratio can significantly increase the cracking strength of UHPC. A similar finding was reported by Shah (1992) who showed that a very high cracking strength of concrete can be obtained if a high volume fraction of fibers (about 15%) is used. This is because when the fiber amount reaches a certain critical threshold, they can effectively carry the force and prevent concrete's microcracks from growing and interconnecting to form a percolation crack. It is believed that a high reinforcing bar ratio provides the same effect.

Incorporating a greater quantity of steel reinforcement results in lower tensile stress levels in the steel reinforcement, even at higher loads. As crack widths in concrete beams are typically proportional to the tensile stress in steel reinforcement, reduced stress levels can enable better control of crack width and, as a result, enhance the member's stiffness. When the steel stresses are kept low under the service load, the accompanying low strains in the concrete and steel will produce only small rotations of the cross sections along the member, which translates into a small deflection. In addition, lower tensile strain in the steel reinforcement will maintain the concrete's aggregate interlock and shear capacity. Additionally, a high ratio of tensile steel reinforcement results in a higher ultimate moment capacity.

To validate the new structural design concept for UHPC beams described above, an experimental test was carried out (Shamshiri et al., 2019), and the results are presented in Figure 7. In the RC beam designed using traditional ACI and AASHTO tension-controlled design, the first flexural crack was observed on the tension side at a stress level that was roughly equivalent to the concrete's modulus of rupture (load: 12 kips or 53 kN). However, in UHPC beam, the first visible flexural crack was not traced until a load of 120 kips (535 kN). The load versus deflection curve in Figure 7(a) shows that the slope changed very slightly at about 60 kips (267 kN), which is conservatively considered the first cracking load. Nevertheless, UHPC beam exhibited a nearly linear uncracked behavior up to 250 kips (1112 kN), thereby maintaining a very high stiffness up to 80% of the peak load. As shown in Figure 4, the average concrete's compressive strains in the RC and UHPC beams at their peak strength were measured by a DIC system as 0.003 and 0.015, respectively. However, the maximum measured compressive strains were 0.006 and 0.025, respectively, for RC and UHPC beams. This indicates that using a strain $\varepsilon_{cu}$ = 0.015 to design a UHPC beam provides a sufficient safety margin.

The ultimate strength of the UHPC beam is 318 kips (1415 kN), which is 4.4 times that of an RC beam (72 kips or 320 kN). Figure 7(a) also shows that the UHPC beam had ample ductility, even with a reinforcement ratio five times greater than that of the RC beam. This indicates that using UHPC in flexural members can largely increase the strength and stiffness while maintaining a small self-weight. In fact, due to the UHPC beam's significant overstrength beyond the design load (around 60 kips or 267 kN), the importance of ductility capacity diminishes. Figures 7(b) and 7(c) show that the visible crack widths in the UHPC beam are very small and cracks are widely spaced even at a very high load of 300 kips (1,334 kN).

Figure 8 demonstrates why a UHPC beam with a high reinforcement ratio displays significantly wider crack spacing. The exceptional cracking strength of the concrete (Figure 6) and lower stress in the steel necessitate a greater spacing for the tensile stress in the concrete to reach its cracking strength.

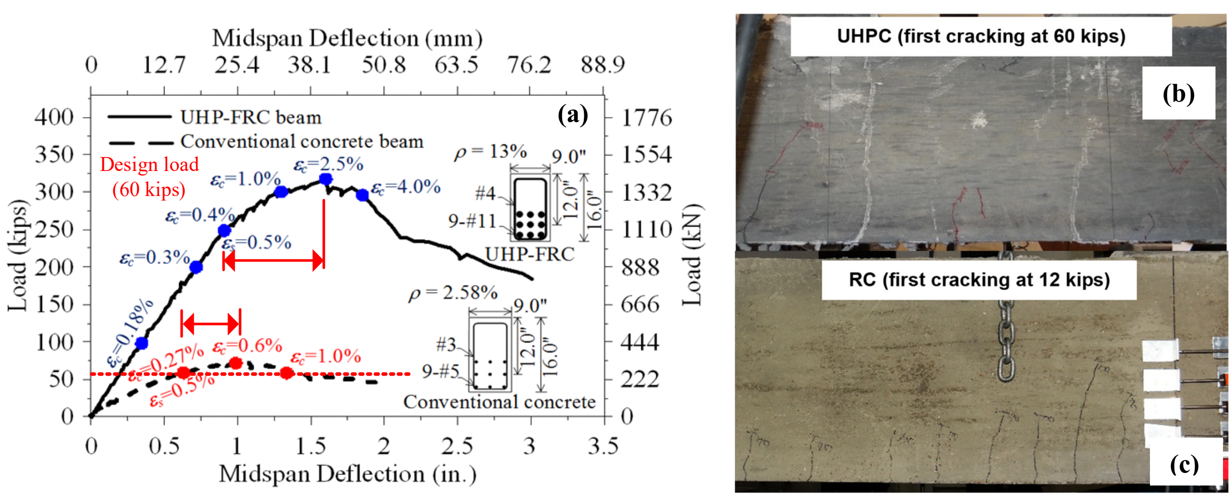



**Figure 7: (a) load versus mid-span deflection responses of RC and UHPC beams, (b) observed cracks in UHPC beam at 300-kip (1334 kN) load, and(c) observed cracks in RC beam at 70-kip (311 kN) load**

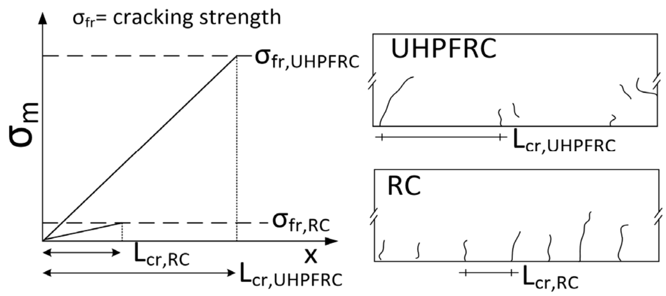



**Figure 8: Crack spacing in RC beam and UHPC beam designed with high steel ratio**

## 5. Summary and Conclusions

UHPC offers a novel approach to designing reinforced concrete flexural members by its superior mechanical characteristics. Maximizing the use of these properties necessitates a thorough comprehension of the synergistic interactions between continuous steel reinforcement and UHPC. The paper outlines three aspects and their implications for structural design utilizing UHPC. Innovative structural application that utilize these synergistic interactions are available in prior studies (Shamshiri et al., 2019).

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