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END-REGION EVALUATION OF HYBRID ULTRA-HIGH-PERFORMANCE
CONCRETE/CONVENTIONAL CONCRETE PRESTRESSED GIRDERS

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END-REGION EVALUATION OF HYBRID ULTRA-HIGH-PERFORMANCE
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Abstract

In prestressed concrete girders, the end-regions are often susceptible to high internal forces induced by prestressing, resulting in crack propagation. Resisting these forces necessitates a substantial amount of reinforcing steel, leading to complicated design and congested formwork. Despite this reinforcement, cracking due to prestressing still frequently occurs. Moreover, girder end-regions are susceptible to corrosive environments, posing a significant concern due to their correlation with sudden and catastrophic failure types.

Ultra-High-Performance Concrete (UHPC) exhibits exceptional mechanical strength and durability, making it a promising material for addressing issues in prestressed applications. However, UHPC is a much higher-priced material than conventional concrete, making monolithic UHPC construction substantially more expensive. An alternative approach involves implementing UHPC selectively in sections of a girder, creating a composite member rather than a fully monolithic one, offering a more cost-efficient solution for prestressed concrete girders. This innovative concept, defined as a hybrid girder, encompasses conventional prestressed concrete girder design for most of the member, except for the end-regions, where UHPC replaces both the conventional concrete and internal reinforcement. The primary concerns surrounding this concept relate to ensuring efficient construction and maintaining continuity across the interface between the different concrete types, facilitating homogenous behavior. This research investigates various interface designs for achieving adequate stress transfer in hybrid girder design while also evaluating the constructability and load-response of these hybrid girders. The research project comprises two main components: a small-scale program characterizing seven different interface methods and a large-scale hybrid girder testing program using the four top-performing interfaces.

The findings of this study indicate that the concept of hybrid girders is plausible in terms of construction, interface continuity, and predictability of load-response. The performance of the interface depends on factors such as texture shape, size, pattern, and casting method. Simultaneously cast concretes with a similar rheology and trapezoidal-shaped cold-joint textures demonstrate enhanced performance. The shear testing of hybrid girders consistently exhibited web-crushing failure, contrary to the expected failure mode based on the shear span-to-depth ratio employed in testing. An assessment of the hybrid girder testing in light of existing shear calculation methodologies reveals that accurate capacity estimation can

be achieved using established approaches. Empirical equations from the ACI 318 Detailed Method and strut-and-tie modeling with prescribed amendments provide precise lower and upper bounds for hybrid girder capacity, regardless of the selected interface design. Overall, the hybrid girder concept offers a cost-effective solution for prestressed concrete girders, combining the strength and durability of UHPC in critical end regions. The research underscores the practicality of this concept and offers a design procedure for hybrid girders.

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Chapter 1: Introduction of Research

1.1. Background and Justification

Over the past two decades, Ultra-High-Performance Concrete (UHPC) has gained considerable popularity in the United States, and codes have begun implementing design processes tailored specifically to its use (Russell & Graybeal, 2013; Muzenski et al., 2023). Beyond its superior mechanical and engineering properties, UHPC exhibits remarkable resistance to corrosive environments and a reduced rate of carbonation. Consequently, it is expected to provide a longer service life compared to conventional concrete. Research efforts continue to enhance the understanding of the properties and performance of UHPC, enabling designers and researchers to identify optimal applications.

One increasingly prevalent application of UHPC is in the strengthening of existing structures. Over 200 bridges in the U.S. have incorporated UHPC in new construction or rehabilitation projects, with most being implemented at connections rather than on primary structural members (Graybeal & El-Helou, Development of an AASHTO Guide Specification for UHPC, 2019). Several studies have evaluated UHPC's potential for enhancing bridge decks and beams (Aaleti & Sritharan, 2019; Bahraq et al., 2019; Crane, 2010; Hussein & Amleh, 2015; Ronanki et al., 2019), but there is limited large- or full-scale testing publications that involve the replacement of the end-regions of a girder with UHPC (Hamilton et al., 2020).

The design of long-spanning concrete or prestressed concrete girders typically requires a significant amount of reinforcing in the end-regions to resist shearing stresses and end-region cracking due to prestressing forces. This reinforcement can lead to congestion within the end regions and often is inadequate for mitigation of the cracking induced by prestressing. Moreover, corrosion near the supports is another typical concern during the life-span of these members and often leads to strand bond issues (Murray, 2017).

One potential solution to these challenges is the incorporation of UHPC in prestressed girders, which would reduce the required reinforcing, mitigate cracking, and improve durability. However, the high cost of UHPC often limits its use in industry, primarily associated with proprietary UHPC products. While non-proprietary UHPC options are

available and more affordable, they still surpass the cost of conventional concretes, making strategic implementation necessary to maximize the cost-benefit relationship. Girder end-regions present the highest potential for selective use of UHPC due to their exposure to higher stresses and corrosive environments.

This study aimed to evaluate the plausibility of incorporating non-proprietary UHPC in the end-regions of prestressed concrete girders to enhance their strength and durability. Such members, defined as hybrid girders, leverage multiple concrete types to create a more efficient design for prestressed concrete girders.

To design and test a hybrid girder for increased shear resistance, two fundamental parameters are essential: understanding the shear behavior of the materials to be used and assessing whether the composite member can act as a monolithic, homogeneous member (Russell & Graybeal, 2013). The failure pattern of a composite concrete member often depends on the shear capacity of the interface of the concretes (Halicka, 2011).

While there is extensive research on the shear capabilities between conventional and high-strength concretes, design provisions in codes such as ACI 318 (2019), PCI Design Manual (2014), and AASHTO LRFD Bridge Design Specifications (2020) predominantly pertain to these materials. As UHPC is a relatively new material, existing design provisions do not yet fully consider its material and engineering properties. Instead, various studies have examined the applicability of different codes for UHPC members and several more have covered UHPC's mechanical properties. Recent publications have shown efforts to develop structural design specifications for UHPC using existing AASHTO methods as a foundation (Russell & Graybeal, 2013; Graybeal, 2022).

One important aspect of composite behavior that has been previously studied is the bond behavior of UHPC to various substrates, either as an overlay or in conjunction with conventional concrete as the deck. Several of these studies are discussed in the literature review in Chapter 2. This research project serves as a continuation of those studies with a specific focus on characterizing interfaces with self-consolidating concrete (SCC) cast with UHPC, with the aim of providing design guidance, limitations, and implementation in hybrid girders.

1.2. Research Scope

The intent of this study was to explore the practicality of implementing hybrid girders with UHPC end regions. The methodology used to investigate this concept was through the implementation of a two-program research plan.

1.2.1. Small-Scale Program: Interface Development and Characterization

The primary concern in implementing a composite member such as a hybrid girder is ensuring continuity, with the potential point of discontinuity located along the interface between the two concretes. In the first program, this concern was addressed by selecting an array of interface textures and assessing the limitations of these interfaces between the two concretes. The selection of interface textures for this study drew from the findings of previous research on stress transfer across UHPC-to-conventional concrete interfaces. Additionally, the program included an interface where fresh concretes were cast simultaneously, combining chemical and physical bonding, which was an approach that was never previously attempted. The theory behind this approach was to enhance strength through combined bonding and expedite the construction process with simultaneous casting. Each interface type was assessed for their capacities based on direct shear strength (shear only), slant shear strength (axial compression plus shear), and flexural strength.

1.2.2. Large-Scale Program: Hybrid Girder Testing and Evaluation

After characterizing the interfaces, the hybrid girder concept was evaluated, which incorporated an array of interfaces to assess constructability and performance in a hybrid girder shear testing program. This program involved recording load-deflection responses and failure modes to gain a deeper understanding of hybrid girder behavior. A comprehensive analysis of existing shear models was comparatively assessed against the responses observed in the large-scale hybrid girders. The outcome of these evaluations guided the development of a design process, along with proposed recommendations, for this innovative girder type. A supplementary study was executed to discover the overall plausibility of the concept compared to existing prestressed concrete construction methods.

1.3. Research Significance

This project contributes to the existing knowledge base on hybrid girders in three significant ways. First, very little research has explored the concept of composite hybrid girders with UHPC ends or its plausibility in terms of construction. Most evaluations of this concept have been theoretical, primarily through finite element modeling, with only one study implementing a full-scale girder testing program that was unable to test the members to failure. Accordingly, there is a notable absence of literature providing data on the failure of these members. While these previous studies form a valuable foundation, this research advances the field by conducting comprehensive testing, including the exploration of hybrid girder behavior up to and including failure. Also, based on the test results, a recommended design process was formulated with specific limitations pertaining to hybrid girder behavior.

The textures evaluated in this research build upon the conclusions and recommendations from existing literature, particularly those concerning cold-joint textures with various geometries. The literature contains a wealth of information on interfacial behavior for various types of concrete, including UHPC-to-conventional concrete that is used in this project. However, given the countless number of texture options available, it is clear that an optimized interface between UHPC and conventional concrete could vary greatly depending on the intended application. This project did not aim to create a fully optimized interface texture for hybrid girders, but rather to assess an array of adequate textures for potential use in hybrid girder implementation. The primary metric for evaluation of the interfaces in this study was to enable homogeneous beam behavior across the interface. Consequently, a range of new and existing interface methods is presented with accompanying descriptions for their performance under various loading scenarios.

Two of the interface options given in this study are innovative solutions to constructing hybrid girders. These interface options are created by the simultaneous wet-casting of UHPC and conventional concrete to enhance chemical and physical bonding. Evaluation and testing results of this texture provide a new approach to casting composite concrete members simultaneously.

1.4. Dissertation Overview

The content of this dissertation is presented within seven chapters and two appendices. This chapter, Chapter 1, introduces the topic and describes the hybrid girder concept and outlines the project. Chapter 2 serves as a review of existing research publications that relate to aspects of the project or the topic itself. The subsequent chapters discuss the details of this project in chronological order. Chapter 3 covers the small-scale interface testing specimen construction and evaluation. Chapter 4 describes the construction and testing of the large-scale hybrid girders. Analysis of the results from the Small-Scale Program and Large-Scale Program are discussed in Chapters 5 and 6, respectively. Project findings, conclusions, and recommendations are given in Chapter 7. Supplementary data such as material properties, concrete mix designs, specimen design drawings, and a catalog of the small-scale testing specimens are all included in the appendices.

Chapter 2: Literature Review

2.1. Introduction

This research aims to conduct a comprehensive evaluation of the hybrid girder concept, addressing various questions concerning its feasibility as a practical structural component. Two primary questions arise in connection with this concept: the potential discontinuity near the composite section of the member and its plausibility for real-world structural applications. The hybrid girder concept involves the incorporation of a specialized concrete possessing enhanced strength and durability characteristics to ensure longevity in corrosive environments while mitigating issues commonly associated with prestressed concrete girders. This specialized concrete, known as Ultra-High-Performance Concrete (UHPC), is a high-cost material, and the intention is to use it effectively in regions with high-stress requirements, while employing conventional concrete for the remaining portions of the girder. Consequently, the interface between the two concrete types is the area most likely to experience discontinuity.

Within this review chapter, descriptions are presented of existing assessments of interfacial strength and performance, along with recommendations for the interface between UHPC and conventional concrete. Regrettably, the concept of the hybrid girder is still in its early stages, and there is very limited data discussing theoretical or actual evaluations of these members. Included herein are existing research publications relevant to this concept and an overview of existing methods for determining the capacity of prestressed concrete girders.

2.2. Existing Interfacial Strength Testing Methods

Several testing methods are used to assess bond strength of concrete interfaces. These test methods can broadly be categorized into two groups: direct shear transfer and indirect shear transfer (Valikhani et al., 2019). Slant-shear testing is the most utilized test for evaluating indirect shear transfer, while various other methods are used for evaluating direct shear transfer. More common methods for direct shear assessment are push-off testing (also known as Modified Vertical Shear Bond), push-out testing, and bi-surface shear testing (Harris et al., 2015).

These tests consider multiple parameters, including a wide range of surface textures, different casting sequences, curing conditions, and concrete types or strengths. There are two prevalent approaches to creating textures that facilitate adequate shear transfer: utilizing form liners to generate artificially textured roughness or exposing coarse aggregates through sandblasting or water-jetting, particularly in cases where conventional concrete is used as the substrate.

2.3. Interface Properties & Corresponding Effects

2.3.1. *Surface Preparation & Roughness*

The importance of proper surface preparation for achieving satisfactory bond strengths between different concretes, particularly for shallow or lightly roughened textures, cannot be overstated. Aaleti and Sritharan (2019) conducted an evaluation of the unreinforced interfacial shear strength between conventional concrete and UHPC under various curing conditions and textures, featuring amplitudes in the range of 0.05 to 0.25 inches. To maintain consistency in surface textures, each texture was created using form liners.

Results from slant shear testing revealed that texture amplitudes exceeding 0.063 inches were sufficient for effective horizontal shear transfer across the interface. Additionally, the authors emphasized that "proper surface preparation with sufficient roughness" holds more significance for interface bond strength than the individual concrete capacities. This finding is reiterated in several other studies (Crane, 2010; Jang et al., 2017; Valikhani et al., 2019). Carbonell Muñoz et al. (2014), who also examined UHPC/NSC interfacial bond strengths, also shares this finding. In that study, Carbonell Muñoz et al. conducted a program that encompassed splitting tensile testing, slant-shear testing, and pull-off testing on monolithic specimens, as well as composite specimens constructed using UHPC and conventional concrete.

All specimens containing conventional concrete were initially cast with that concrete and then subjected to a 24-hour moist curing process before being stored in a lime bath until reaching 28 days of age. Upon reaching this age, a variety of surface preparations using hand tools were implemented to create roughness within the range of 0.6 to 0.96 mm (0.02-0.04 in.) in amplitude for the splitting tensile specimens. When the UHPC portions were added, two saturation levels were assessed for the conventional concrete portions of the splitting tensile

specimens: dry and saturated-surface dry (SSD). The results of the splitting tensile testing showed a significant difference in strength between dry and SSD conventional concrete substrates. Consequently, all slant-shear and pull-off specimens in subsequent testing were cast with a UHPC overlay under SSD conditions.

It was concluded that, for lightly textured interfaces (0.02-0.04 in.), bond strength is enhanced and becomes less dependent on roughness when the substrata is properly wetted. It is worth noting that in the study by Carbonell Muñoz et al. (2014), UHPC was cast onto a conventional concrete substrate, making moisture migration from the fresh UHPC into the existing conventional concrete the primary concern. This approach for casting UHPC onto existing concretes is a common practice in many interface strength studies, including those by Aaleti & Sritharan (2019), Jang et al. (2017), and Valikhani et al. (2019). Many of these studies have observed improved bonds when the substrata were maintained at SSD conditions during additional casting (Carbonell Muñoz, Harris, & Ahlborn, 2014; Crane, 2010; Russell & Graybeal, 2013).

2.3.2. Texture Amplitude, Orientation, & Shape

Achieving significant increases in interface bond strength can be accomplished by incorporating textures that facilitate stress transfer through coarse aggregates in the conventional concrete (Valikhani et al., 2019). Jang et al. (2017) conducted an evaluation of a series of composite push-off specimens, implementing five different types of interface textures between conventional concrete and UHPC with and without reinforcement to assess shear performance. The unreinforced interfaces featured textured surfaces that included a regular construction joint with no surface preparation, a water-jet surface treatment, and two-dimensional square-grooved textures created from form liners with amplitudes of 10, 20, and 30 mm. Test results revealed that for the composite interfaces, larger square-grooved textures and exposed coarse aggregate interfaces yielded the better results. The failure planes for both texture types occurred in the conventional concrete stratum near the interface. The authors concluded that higher shear strengths depend on the presence of coarse aggregate in the conventional concrete and its ability to fit into the grooved textures. Similar results were observed in the water jet surface treatment for UHPC-to-UHPC specimens, which was shown to be less effective due to the natural absence of coarse aggregate in UHPC. Grooved textures were better able to distribute stresses across the interface and exhibited a linear increase in

shear strength as the groove size also increased. This is attributed to the increased presence of coarse aggregate in the conventional concrete at the interface, which allowed the grooved textures to more effectively distribute stresses across the interface. Among the two better-performing textures, the exposed aggregate interface demonstrated higher shear strength which led the authors to theorize that variations in texture in multiple directions along the interfacial plane could yield optimized results, preventing the localization of stresses on a single plane.

This idea finds further confirmation in a study by Yang et al. (2022), which focused on evaluating two-dimensional grooved (fluted) textures with different groove widths, shapes, and spacing. They concluded that mechanical interlocking of UHPC and conventional concrete provided superior strength and delayed cracking compared to homogeneous conventional concrete specimens. Grooves with trapezoidal shapes exhibited higher strengths and stiffness than square grooves, primarily due to the transfer of shear forces from UHPC into the conventional concrete stratum rather than along the interface. Through finite-element modeling of additional interface variations, the authors found that 25-mm (1-inch) amplitudes yielded optimal performance.

Many studies evaluating a range of smaller amplitude textures (0 to 0.25 inch) have also reached the conclusion that increased amplitudes correspond with increased shear capacities. Crane (2010) conducted a comparative assessment of monolithic and composite UHPC/high-strength concrete push-off specimens with various interface types. Monolithic specimens were tested both with and without a precracked interface, while composite specimens were constructed with a smooth cold joint, a "mildly rough" burlap-textured surface, and a grooved 0.25-inch form liner texture. Monolithic UHPC specimens withstood the highest loads, regardless of whether they were precracked; however, precracked specimens exhibited slippage at lower loads than the uncracked specimens. Results of the composite push-off specimens indicated that the form liner texture offered peak overall performance for the composite specimens.

In a study by Valikhani et al. (2019), thirty composite UHPC to conventional concrete bi-surface shear specimens were evaluated for direct shear strength. These 6-inch cube specimens were constructed with one-third of the cube cast with UHPC and the remaining two-thirds cast as conventional concrete. The conventional concrete portions were initially cast and stored in ambient lab conditions for two months before the UHPC was added. At testing, the

cubes were loaded through 2-inch-wide steel plates, arranged for three-point loading, ensuring direct shear along the interface and at the midspan of the conventional concrete section as illustrated in Figure 2.1. Three surface textures were analyzed for the composite specimens: smooth, sandblasted, and "normal" surfaces with an applied bonding agent. Each surface texture was also evaluated with and without interface reinforcement. Test results showed that the sandblasted surface increased bond strength by approximately 125% compared to the smooth surface. Bonding agents only achieved a 34% increase in interfacial bond strength compared to the smooth interface specimens. The study concluded that adhesive failure at the interface could be mitigated by increasing surface roughness. Furthermore, when adding interface reinforcement, the failure mechanism shifted from adhesion in the interface to a cohesive failure in the conventional concrete (Valikhani et al., 2019).

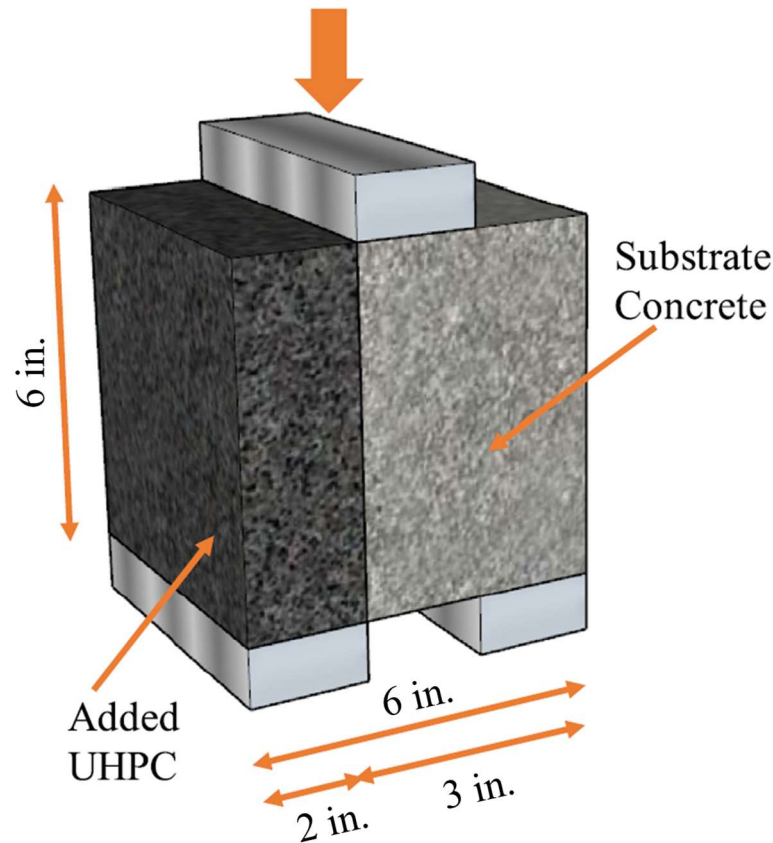


Figure 2.1: Bi-surface shear specimen (Valikhani et al., 2019)

2.3.3. *Interface Reinforcement*

Numerous studies have demonstrated that bond strength can be significantly enhanced by introducing reinforcement across the interface. Some even reveal a linear relationship between shear strength and the interfacial reinforcement ratio, irrespective of the various surface preparation methods (Crane, 2010). In the previously introduced study by Valikhani et al. (2019), direct shear testing revealed that the incorporation of reinforcement across the interface increased the strength by 17% compared to specimens without reinforcement.

The linear relationship between reinforcement and interfacial shear capacity was further substantiated in a publication by Ronanki et al. (2019), focusing on hybrid girders. In that study, composite push-off specimens with UHPC and conventional concrete were examined. The specimens were tested to develop parameters for Finite Element Analysis (FEA) models of a hybrid girder with varying amounts of interfacial reinforcement and two-dimensional, grooved form liners to texture the conventional concrete surface. Three configurations of interfacial reinforcement were tested, all with the same surface area and surface treatment: no reinforcement, two No. 3 bars, and four No. 3 bars. Figure 2.2 illustrates the reinforcement diagram and texture pattern employed on the interface. The results indicated that the reinforced specimens exhibited a near-linear increase in shear stress as the reinforcement increased and higher stiffness compared to unreinforced specimens. This effect was attributed to the combined contributions of mechanical interlock at the interface and tensile resistance from the interfacial reinforcement.

Stage B-C (Loss of Shear Resistance): As the reinforcement continues to yield, and the adhesion and cohesion of the concrete surface is lost, there is a corresponding loss in shear resistance. However, shear is substantiated by the coupling of the clamping force and shear friction.

Stage C-D (Dowel Action and Rupture of Steel): In this stage, dowel action of the reinforcement ensues, leading up to the rupture of the reinforcement.

Stage D-E (Rapid Load Decrease with Minimal Displacement Increase): The final stage is characterized by a rapid decrease in load, accompanied by minimal additional displacement.

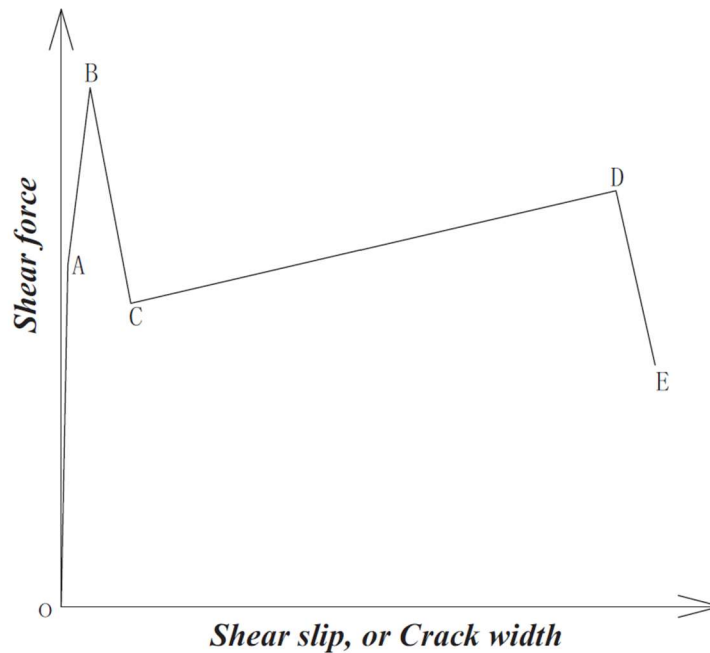


Figure 2.3: Idealized load-deflection curve for reinforced, cold-joint push-off testing (Liu et al., 2019)

2.4. Large- & Full-Scale Interface Evaluations

Only a limited number of studies have implemented large- or full-scale beam tests with the intent of investigating shear transfer between concrete sections. These tests typically involve precast beams and decks or portions of a deck. One such study, conducted by Patnaik (2001), focused on horizontal shear transfer between precast girders and decks. The beams were cast to resemble the construction process of a precast web section and a cast-in-place slab or flange. The interfacial surface on the web sections were troweled smooth and cured for seven days before adding the top flanges. The beam reinforcement was designed to resist shearing forces in the web before exhibiting horizontal shear failure, with varying amounts of ties crossing the interface. Each beam was loaded with a single point load at the midspan of the simply supported member. As the load increased, crack propagation began at the interface, causing the flange section to slip relative to the web. With increasing slip, flexural cracking developed throughout the web and flange, ultimately leading to horizontal shear failure. This caused the member to behave as two independent beams stacked atop one another. While it is recommended to calculate horizontal shear strength as a function of the lesser compressive strength of the two composite concretes, Patnaik did not specify a direct relationship between horizontal shear strength and the compressive strengths of either stratum. The study did find a linear relationship between the total amount of reinforcement crossing the interface and shear stress. Although, it was also concluded that the depth-to-interfacial reinforcement spacing ratio did not influence horizontal shear strength between the deck and girder, signifying that the importance is placed on the inclusion of reinforcement, not the location.

The second phase of the study conducted by Aaleti and Sritharan (2019) included a large-scale, flexural evaluation program involving composite slabs with a UHPC overlay subjected to three-point loading to further assess interface bond strength. All composite specimens were either cast with the UHPC or the conventional concrete section first and allowed to cure under ambient conditions for 28 days before the next portion was cast onto the existing section. The results indicated that interfacial bond strength between UHPC and conventional concrete primarily depended on interfacial roughness rather than the individual material strengths, aligning with the studies in Sections 2.3.1 and 2.3.2. From this testing, it was recommended that an interface roughness of at least 0.12 inches be used for adequate deck shear transfer between UHPC and conventional concrete.

Another section of the report by Crane (2010) assessed five large-scale composite T-beams and three full-scale prestressed UHPC girders with conventional concrete decks. For the large-scale beams, the UHPC was textured using form liners or burlap pressed against the interfacial surface. Each beam underwent three-point loading and was instrumented to record relative slip between the web and the flange, as well as vertical deflection at midspan. The large-scale testing revealed that beams with form-liner-textured interfaces resisted higher loads before slipping and exhibited greater toughness compared to beams with smooth or burlap-textured surfaces. Beams with the latter two textures and a reduced quantity of stirrups behaved more like beams without flanges. The results indicated that smooth interfaces failed to fully enable the composite members to act monolithically, and using form liners was recommended for UHPC to conventional concrete cold-joints. Additionally, push-off tests were found to accurately predict the behavior of these composite beams.

In the full-scale girder evaluation by Crane (2010), specimens were tested to observe diagonal tension and interfacial shear capacities, as well as flexural strength. The prestressing of the beams aimed to replicate that of a 90-foot-long girder with a 7.5-foot girder spacing, based on AASHTO LRFD design. Interface reinforcement, surface area, and roughness were varied to analyze their effect on interfacial shear capacity. The composite girder configuration with the form liner-textured surface and minimal interfacial reinforcement effectively mitigated horizontal shear failure at the interface, whereas the smooth surface with minimal reinforcement could not. When comparing push-off specimens and small-scale beam testing results against the full-scale program, the small-scale beams provided a conservative prediction of full-scale behavior, and push-off testing accurately predicted small and full-scale behaviors. However, push-off testing for the smooth surface was an exception and was not recommended for estimating behavior due to high variability exhibited by testing strengths.

2.5. Hybrid Girder Analysis

The literature on the evaluation of the hybrid girder concept presented in this study is currently limited. However, a report by Hamilton et al. (2020) addressed the full-scale performance of prestressed concrete girders with UHPC end-regions. The study included full-scale Florida-I beam 72 (FIB 72) and Florida-I beam 54 (FIB 54) girders, both with straight strand profiles. In the case of the FIB 72 girders, five partial-length girder replicas were analyzed for end-region cracking due to prestress release. The results indicated that UHPC ends exhibited significantly less cracking compared to traditional conventional concrete ends, even with reduced quantities of reinforcing steel. The two full-length FIB 54 hybrid girders were tested to determine shear capacity and monitor performance. However, limitations relating to load capacity of the testing setup prevented the capture of the hybrid girder's failure loads (i.e., the girders could not be tested to failure). Nevertheless, the authors were able to monitor service load behavior and performance up to the capacity of the testing equipment. Three key takeaways were observed: (1) an exposed aggregate interface texture effectively transferred stresses without sustaining damage, (2) extending the length of the UHPC end-region increased girder stiffness, and (3) the UHPC end-regions resisted 25% higher shear loads (not failure loads) than equivalent girders with traditional end-region construction. The authors also developed FEA models from the testing data and theorized that the hybrid girder approach can facilitate increased maximum span lengths, potentially up to 10%, depending on girder shape. The FEA model was directly correlated with individual material parameters and validated against full-scale girders. The FEA modeling accurately predicted girder performance at the interface and cracking behavior during strand release and superimposed loading. However, Hamilton et al. found that the FEA model tended to overestimate maximum crack widths in the UHPC but not in the self-consolidating concrete (SCC). The authors emphasized the need for failure strengths of hybrid girders to further validate their models.

The limited number of full-scale testing studies can be attributed, in part, to the higher cost and specialized equipment required for such testing. Conversely, some studies have explored the concept through FEA modeling. Ronanki et al. (2019) developed an FEA model of two 205-foot-long Precast Concrete Economical Fabrication Committee standard bulb-tee girders. One of the two girder models was designed as a monolithic section of conventional concrete to establish a baseline for comparison to the other girder. This model was validated

against a set of four 54-foot-long, actual versions of the same girder, constructed of high-strength concrete with varying amounts of end-region reinforcement. These girders were instrumented to capture behavior during strand release and when subjected to loading to induce shear cracking. The second FEA girder model represented a UHPC/high-strength concrete hybrid girder with UHPC end-regions extending twice the girder height from each end. The reinforcement design for shear and end-region stresses expected in the high-strength concrete section adhered to AASHTO LRFD design specifications. The hybrid girder utilized the AASHTO specified minimum reinforcing in the UHPC section: double-leg, No. 4 stirrups at 12 inches on center. The interface texture matched that shown in Figure 2.2. Results of the full-scale FEA modeling indicated that the grooved interface texture assessed for the push-off specimens was adequate for stress transfer across the modeled interface. Additionally, the composite girder was able to successfully mitigate prestress release cracking that had occurred in the end-regions of the conventional concrete model and constructed girders. Under maximum moment and shear loading, the modeled stresses at the interface in the hybrid girder remained below the maximum stresses for both shear and rupture, affirming the plausibility of the hybrid UHPC/conventional concrete girder concept.

2.6. Shear Behavior and Analysis of Prestressed Concrete

A fundamental component of a prestressed member, the axial compressive force resulting from prestressing, leads to different behavioral responses and failure modes in prestressed concrete beams compared to reinforced concrete. This axial force substantially reduces shear stresses and principal stresses in prestressed beams. Given that concrete's tensile strength is considerably lower than its compression strength, the principal tensile stress tends to be the limiting factor for shear capacity. Induced shear stresses can generate principal tensile stresses surpassing concrete's tensile strength, causing cracks. These cracks tend to form along inclined angles due to the combined effects of internal flexural and shearing forces from applied loads. Prestressing increases the compressive force within the member, altering the tension field within a beam, resulting in crack angles that are much shallower compared to the typical 45-degree angle observed in reinforced concrete members.

Shear cracking in prestressed members can be categorized into either flexure-shear or web-shear cracking. These classifications depend largely on the location of the load along the member's length with respect to the overall member height. Flexure-shear cracking initiates as vertical flexure cracks at the bottom of the section, and as it extends, the crack path is directed toward the applied load. This type of cracking and potential failure mode typically occurs further into the span of the member, coinciding with where the maximum principal tensile stress occurs in prestressed members. Therefore, when failure occurs due to flexure-shear, it is often referred to as diagonal tension failure. In contrast, web-shear cracks initially form in the web, closer to the beam ends than flexure-shear cracks and extend concurrently toward the tensile reinforcement and compression zones with increasing load (Nawy, 2010).

Despite concrete design typically assuming homogeneity for the individual materials used in the member (e.g., concrete, steel, etc.), the reality is that concrete is not homogeneous, and its construction rarely achieves a high level of precision. As a result, the regions for flexure-shear or web-shear behavior are not always well-defined or consistently accurate for all beams. There may be cases where the regions for flexure-shear and web-shear behavior overlap. While the behavior in each zone is generally agreed upon by many codes and authors, the boundaries of each zone are more subjective (Ibraheem & Rasheed, 2023; Murray, 2017; Nawy, 2010; Park & Choi, 2017; Pimanmas, 2010; Wight & MacGregor, 2012). Typically, web-shear behavior is located closer to the ends of prestressed girders, around the discontinuity regions (D-regions), while flexure-shear behavior is more prevalent in an "intermediate" range between the web-shear-controlled and flexure-controlled regions (Nawy, 2010).

2.6.1. Prestressed Concrete Shear Modeling Methodologies

The fundamental equations for calculating shear in prestressed concrete beams largely resemble those used in reinforced concrete. To estimate the shear capacity of prestressed concrete, the summation of the individual shear-resisting components in a beam is taken, including shear resistance provided by the concrete, mild-steel reinforcement, and the vertical component from prestressing (in cases of harped-strand applications). The primary distinction between prestressed and reinforced concrete shear calculations lies in the method for determining the shear resistance of the concrete and the prestressing component.

In the context of the hybrid girder concept, the process for flexure-shear design remains the same as for a traditional, homogenous prestressed concrete girder, as it features matching designs. However, the end-regions of hybrid girders, with the incorporation of UHPC ends, have the potential to exhibit different behavior. Therefore, it is essential to consider existing methods for estimating behavior in the end-regions, predominantly in the regions controlled by web-shear.

Various methods can be employed to model girder capacity based on web-shear behavior. The standard shear design procedure for bridge girders is given by AASHTO LRFD (2020), which is based on the Modified Compression Field Theory (MCFT). The MCFT assumes that cracking forms due to principal tension stress, and compression is subsequently carried by a truss-like system of compressive struts, also called a compression field (Wight & MacGregor, 2012). The MCFT method is notoriously complicated, and as an alternative, AASHTO also offers a simplified approach for calculating concrete shear strength. The AASHTO Simplified Method determines shear capacity through calculation of both web-shear and flexure-shear, where the lesser of the two is the controlling capacity. ACI 318 (2019) uses a similar approach for estimating the shear contribution of concrete in its Detailed Method. Empirical equations, such as those found in the ACI 318 Detailed Method or the AASHTO LRFD Simplified Method, provide a more straightforward approach for determining shear capacity. Additionally, both AASHTO LRFD (2020) and ACI 318 (2019) allow for shear design of end-regions through strut-and-tie modeling. In contrast to the MCFT, strut-and-tie modeling is intended to be used for estimating capacity in D-regions. Due to the complicated geometry of the internalized stress fields in these regions, strut-and-tie models offer a simplified approach to quantify these areas through a system of compressive struts and tension ties. Both codes have integrated their own approaches for modeling shear design and their own individual component limitations based on the strut-and-tie method (Pimanmas, 2010). All the previously mentioned techniques are accepted for design practice; however, research has shown that the ACI Detailed Method provides more accurate approximations for shear strength compared to the AASHTO methodologies (Murray, 2017).

In the specific context of calculating shear strength for girder end-regions, commonly used and recommended methods involve strut-and-tie modeling, or the application of empirical equations based on sectional design. For this research, the primary approaches for analyzing end-region behavior include strut-and-tie modeling and the empirical equations outlined in the Detailed Method of ACI 318 (2019).

2.6.2. *Prestressed Shear Design using ACI 318 Code*

The nominal shear capacity for a prestressed concrete beam, as per ACI 318 (2019), is determined by calculating the separate contributing factors that resist shear and summing them together, as expressed in Equation 2-1. In this equation, the shear contribution from the concrete is computed using either the Simplified Method (Equation 2-2) or the Detailed Method (Equations 2-3 and 2-5). In the Detailed Method, the concrete shear resistance is considered as the lesser of the two capacities for web-shear or flexure-shear. Other factors contributing to member shear capacity in Equation 2-1 include the resistance provided by mild steel reinforcement and the vertical component of the effective prestressing. The calculation for the steel reinforcement component follows the same procedure applied to reinforced concrete, while the prestressing component depends on whether harped strands are employed and the corresponding prestressing force and strand orientation.

$$V_n = V_c + V_s + V_p \quad \text{Equation 2-1}$$

where:

V_n = nominal shear strength of the member (lbs)

V_c = nominal shear strength provided by the concrete (lbs)

V_s = nominal shear strength provided by the steel reinforcement (lbs)

V_p = vertical component of effective prestress force at the section (lbs)

$$V_c = \left(0.6\lambda\sqrt{f'_c} + 700 \frac{V_u d_p}{M_u} \right) b_w d_p \quad \text{Equation 2-2}$$

where:

- V_c = nominal shear strength provided by the concrete (lbs)
- λ = lightweight concrete reduction factor
- f'_c = concrete compressive strength (psi)
- V_u = factored shear force at section (lbs)
- d_p = effective depth of prestressing, taken as distance from extreme compression fiber in girder to the centroid of prestressing (in.)
- M_u = factored moment at the section (lb-in.)
- b_w = web width (in.)

The contribution from web-shear, denoted as V_{cw} , is determined by Equation 2-3. This calculation assumes that web-shear cracking is initiated by the principal tensile stress (f'_t) accumulating a value as determined by Equation 2-4, and this occurs at the level of the centroid in the section.

$$V_{cw} = \left(3.5\lambda\sqrt{f'_c} + 0.3f_{pc} \right) b_w d_p + V_p \quad \text{Equation 2-3}$$

where:

- V_{cw} = nominal web-shear strength of the concrete (lbs)
- λ = lightweight concrete reduction factor
- f'_c = concrete compressive strength (psi)
- f_{pc} = compressive stress provided by the prestressing (psi)
- b_w = web width (in.)
- d_p = effective depth of prestressing, taken as distance from extreme compression fiber in girder to the centroid of prestressing (in.)
- V_p = vertical component of the prestressing force that resists shear (lbs)

$$f'_t = 4\lambda\sqrt{f'_c} \quad \text{Equation 2-4}$$

where:

f'_t = concrete tensile stress (psi)

λ = lightweight concrete reduction factor

f'_c = concrete compressive strength (psi)

Flexure-shear capacity, defined by Equation 2-5, also requires an additional calculation of the moment causing flexural cracking within the section due to externally applied loads. In this method, ACI (2019) assumes that the precast girder section primarily resists shear, and the calculations exclusively consider the properties of the girder, excluding those of the deck.

$$V_{ci} = 0.6\lambda\sqrt{f'_c}b_wd_p + V_d + \frac{V_iM_{cre}}{M_{max}} \quad \text{Equation 2-5}$$

where:

V_{ci} = nominal flexure-shear strength of the concrete (lbs)

λ = lightweight concrete reduction factor

f'_c = concrete compressive strength (psi)

b_w = web width (in.)

d_p = effective depth of prestressing, taken as distance from extreme compression fiber in girder to the centroid of prestressing (in.)

V_d = shear force at the section due to unfactored dead load (lbs)

V_i = factored shear force at the section due to externally applied load occurring simultaneously with M_{cre} (lbs)

M_{cre} = moment causing flexural cracking at the section due to externally applied load, calculated using estimated section stresses and section properties (lb-in.)

M_{max} = maximum factored moment due to externally applied load (lb-in.)

2.6.3. *Prestressed Shear Design using Strut-and-Tie Modeling*

Alternatively, AASHTO (2020) and ACI (2019) also permit the use of strut-and-tie modeling for design and analysis of the end-regions of concrete beams. These models serve to idealize the behavior of D-regions, primarily due to the inapplicability of the plane-sections assumption in the Bernoulli beam theory (Murray, 2017; Nawy, 2010). As previously mentioned, the strut-and-tie approach conceptualizes the internal stresses within a member as a truss system, comprising struts, ties, and the connecting joints referred to as nodes. Nodes connect at least three forces and are categorized based on whether these forces are in tension, compression, or a combination of the two. The arrangement of the model is typically not strictly defined for design and is often left to the engineering judgment of the designer. Codes such as ACI (2019) have simplified the design process by incorporating limits that facilitate model development.

Once the internal truss layout is established, a truss analysis is performed to determine the forces that make up the truss. The widths of the struts are proportioned based on the force from the applied load and effective compressive stress. Similarly, ties are reinforced to ensure the required strength is met. Nodal stresses are subsequently examined, and if necessary, confinement reinforcement is designed.

2.7. Literature Review Summary

This literature review comprehensively examines information pertaining to interfacial properties between Ultra-High-Performance Concrete (UHPC) and conventional concrete. It also delves into the existing literature surrounding the hybrid girder concept and shear calculation methods. From these extensive studies, it is evident that enhancing interface performance between UHPC and conventional concrete is achievable through proper surface preparation, consideration of texture shape and size, and the incorporation of reinforcement, all of which contribute to more effective stress transfer.

Furthermore, in the realm of hybrid girders, both theoretical modeling and prior full-scale evaluations have yielded promising results, indicating the plausibility of this innovative concept. This dissertation contributes to the collective body of knowledge by building upon these foundational studies and rigorously evaluating the accrued insights.

Chapter 3: Small-Scale Program – Interface Testing

3.1 Introduction

The focal point of the Small-Scale Program revolves around the comprehensive characterization of seven distinct interface types. These specific interfaces were chosen based on insights drawn from the data presented in Chapter 2. The range of interfaces includes two approaches for casting Ultra-High-Performance Concrete (UHPC) and Self-Consolidating Concrete (SCC) simultaneously, with and without a divider at the interface. For this research, these two interfaces are referred to as “BiFluid” and “BiFluid with Divider”. Additionally, a spectrum of five cold-joint textured interfaces, visually depicted in Figure 3.1, were selected for analysis, providing a total of seven distinct interface types. Full detail and description of each interface is provided in Appendix A.



Figure 3.1: *Interfaces from Middle UHPC Section of Direct Shear Specimens¹*

¹ Specimens pictured are: (left to right) 2D Trapezoidal, 3D Trapezoidal, Exposed Aggregate, Smooth Interface, & Sine Wave

Before the characterization testing of the selected interfaces for the small-scale program, a preliminary study was undertaken to assess the plausibility of BiFluid casting of UHPC and SCC. This investigation was conducted both with and without a divider to create a straight or irregular vertical interface. See Figure 3.2 for an illustration of the divider before adding concrete. The primary objective was to gauge the feasibility of creating interfaces of these casting types and their corresponding strengths.



Figure 3.2: *BiFluid with Divider assembly for slant shear specimen*

Five separate cold-joint interfaces were evaluated for interfacial characteristics in addition to the BiFluid methods. Of these five, a smooth interface was included with the intention to serve as a benchmark for assessing cohesion among the various cold-joint interfaces. Recognizable and widely used interfaces, such as the exposed coarse aggregate interface and the fluted, 1/4-inch sine wave textures, were also integrated into the study. Furthermore, two trapezoidal textures were incorporated: a fluted, 2-dimensional trapezoidal pattern, and a 3-dimensional trapezoidal pattern resembling a checkerboard design (fluted in two directions). The trapezoidal pattern design was created from previous research discussing the influence of various texture types on interfacial strength.

For this research, the UHPC and SCC mixes were carefully chosen. The selection criteria included existing mix designs endorsed by the Oklahoma Department of Transportation (ODOT), as well as prior investigations into non-proprietary UHPC mix designs tailored for the state of Oklahoma. In the case of SCC, the chosen mix design aligns with ODOT's Class AA structural concrete mix design. This specific composition is tailored to meet the State's requirements for superstructure components, including beams and girders. As for the UHPC, the selected design is the "J3" mix design, a product of prior research at the University of Oklahoma (Crone, 2017).

The Small-Scale Program follows a structured progression comprising testing methodologies that were carefully chosen from existing literature, starting with direct shear testing, subsequent slant shear testing, and concluding with flexural testing for the evaluation of the modulus of rupture. These testing protocols effectively replicate the anticipated stress scenarios occurring at interfaces proximate to the end regions of prestressed concrete girders.

As the program advanced, a strategic approach was employed wherein the interface textures that exhibit higher performance in each test method are integrated into the subsequent method. This iterative progression ensured that the most resilient and effective interface textures are identified and carried forward. The ensuing chapter elaborates on the specific methods employed in this interface characterization process.

3.2 Interface Strength Assessment of BiFluid Casting

3.2.1 *BiFluid Concept*

The inception of this concept originated with the goal of casting hybrid girders using UHPC and SCC in parallel, as opposed to sequential casting. This idea gained further traction through review of previous literature and was reinforced by Hamilton et al.'s suggestion to explore a "wet cast joint" at the interface (2020). This approach involves casting both types of concrete simultaneously, fostering enhanced interfacial strength through the synergy of chemical and physical bonding mechanisms.

However, the implementation of this concept presents challenges in maintaining a clean and distinct vertical interface. This challenge is addressed by controlling the rates at which UHPC and SCC are added based on their respective rheology. Nevertheless, there are still concerns regarding the practicality of this approach in precasting facilities. To overcome this,

an additional concept was introduced: the insertion of a removable vertical barrier between the concretes during casting. This barrier allows for controlled differential casting rates, further refining the casting process. Within this research, these two interface configurations differ from each other in the context of BiFluid casting in that one is without a vertical barrier, referred to as the BiFluid interface, while the other with the casting barrier is referred to as the BiFluid with Divider interface.

The objective is to gauge the viability of this simultaneous casting methodology for construction applications, particularly focusing on the strength of the resultant interface. While a comprehensive characterization of the interface was not deemed essential, an answer to what the added strength would be from chemical bonding was considered sufficient for assessing the feasibility of the BiFluid interfaces. The most suitable approach to answer this question was through splitting-tensile testing of the interfaces. For comparative purposes, the same splitting-tensile testing procedure was also applied to homogeneous specimens of SCC and UHPC. The results of these evaluations would provide insight into the potential of the BiFluid casting approach, informing decisions on whether to proceed with these interfaces in hybrid girder construction.

3.2.2 Construction

The casting process involved the utilization of multiple concrete mixers for producing the BiFluid specimens. The UHPC was mixed using a vertical axel mortar mixer, while the SCC was mixed using a traditional drum mixer. The detailed mix designs for both UHPC and SCC are provided in Appendix A. It should be noted that the UHPC in this portion of the study omitted the incorporation of fibers for the mix design. This was decided on the basis that the introduction of fibers would only increase the tensile capacity of the UHPC and the goal was to obtain a minimum tensile capacity of the interface alone.

To ensure synchronized mixing procedures, the UHPC mixing commenced before the SCC materials. Dry materials for UHPC were pre-mixed for the designated 10 minutes, after which the fluid materials were introduced to initiate the UHPC mixing process. Concurrently, the cementitious materials for SCC were added, and the standard 3-3-2 mixing protocol from ASTM C192 was initiated (2015). The two mixing processes were closely monitored and executed in tandem to ensure simultaneous completion and casting.

To document the workability characteristics of the mixes, the specimens were cast as the flow rates for each mix were being recorded. The casting of BiFluid specimens involved simultaneously introducing both UHPC and SCC at a matched rate. Human error during this process led to a non-uniform interface. Despite the differing workability, the rheology of each concrete was similar and allowed for each material to retain its position after flowing together. Conversely, when the addition rate of one concrete was increased, this shifted the position of the interface due to material pushing towards the opposite side. To rectify this, the addition rates for each were adjusted and compensated to maintain a relatively vertical, average interface. Figure 3.3 shows each of the resulting BiFluid specimens.

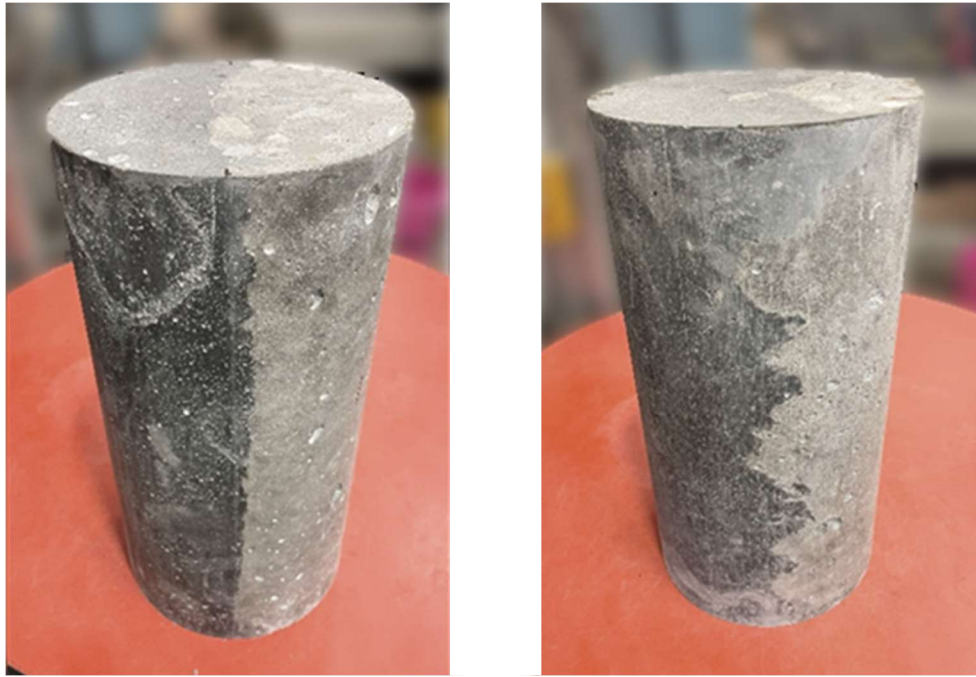


Figure 3.3: *Splitting tensile specimens for BiFluid with Divider (left) and BiFluid (right)*

Subsequent casting involved the BiFluid with Divider specimens. These were cast by adding the two concretes at comparable rates, followed by the removal of the divider. To account for volume loss due to divider removal, small amounts of concrete were added. The presence of the divider facilitated the creation of a more precisely defined vertical interface, as shown in Figure 3.3, alleviating the need for perfect addition rates for each of the concretes.

The recorded slump flows for UHPC and SCC mixes were approximately 10 inches and 20 inches, respectively. As noted previously, in spite of differences in viscosity and composition, the two concretes displayed the ability to flow together and maintain their shape, post contact. Minor instances of intermixing were observed at the interface, indicating further fluid interaction between the concretes. Both sets of BiFluid specimens used 6-inch by 12-inch cylinder molds, while monolithic SCC specimens utilized 4-inch by 8-inch molds, and monolithic UHPC specimens were cast in 3-inch by 6-inch molds. A set of monolithic, compressive strength cylinders were cast for each concrete, using the same specimen dimensions. All specimens underwent moist curing for the initial 3 days, followed by transfer to ambient laboratory conditions until attaining a 28-day age.

3.2.3 Testing & Results

The splitting tensile testing protocol closely followed the guidelines specified in ASTM Standard C496 (2017). For the assessment of compressive strength specimens, the procedure outlined in ASTM Standard C39 (2021) was adopted. Subsequent failure loads, dimensions, and failure planes were documented.

The results from the splitting tensile testing of both sets of BiFluid specimens closely mirrored the splitting tensile strengths exhibited by the monolithic SCC specimens. In the case of all the non-homogenous specimens, failure planes exhibited a consistent trend along the interface of the concretes, positioned just within the SCC section, and occasionally traversing through small segments of UHPC (see Figure 3.4). It was observed that both BiFluid specimens, irrespective of having a linear interface originating from the divider or an irregular interface, showcased comparable strength values. This observation appears to validate the hypothesis that tensile capacity is predominantly influenced by the relatively weaker stratum, in this case the SCC.



Figure 3.4: Two halves of tested BiFluid splitting-tensile specimen

Furthermore, it is believed that the introduction of fibers into the UHPC matrix, alongside the chemical bonding, could potentially lead to the migration of the failure plane deeper into the weaker substrate. The summarized strength values collected from the splitting tensile testing are presented in Table 3.1. These results fell within the acceptable standard deviation listed in ASTM C882 (2020) and provided confirmation that the exploration of both BiFluid interfaces warrants continued investigation within this study. These outcomes lend confidence to the validity and potential efficacy of the BiFluid approach, encouraging its further scrutiny and advancement.

Table 3.1: BiFluid Splitting-Tensile Test Results

Specimen Type	Avg. Splitting-Tensile Strength (psi)	Standard Deviation (psi)
SCC (Monolithic ODOT AA) ($f'_c = 5920$ psi)	468	69
UHPC (Monolithic J3) ($f'_c = 15,590$ psi)	1067	206
BiFluid	522	41
BiFluid with Divider	482	18

3.3 Direct Shear Specimens

In the assessment of shear strength at the interface between two concrete materials, various methodologies are available. For this study, the direct shear method, also known as push-out shear, was chosen. This selection is based on its advantages in terms of specimen fabrication and materials required for the testing arrangement, which are relatively straightforward in nature. Alternative shear testing methods, such as push-off shear testing (referred to as vertical shear bond or guillotine), call for a more intricate layout for specimens along with internal reinforcement to resist internal moments (e.g., Harris et al., 2015; Valikhani et al., 2019).

Each specimen for direct shear testing was divided into three distinct sections: two outer sections consisting of SCC, and a central section composed of UHPC. This configuration enables the presence of two distinct planes of pure shear, one at each interface between the two concretes. To ensure statistical significance, a total of three specimens were subjected to testing for each type of interface.

3.3.1 Construction of Direct Shear Specimens

The design of the direct shear specimens was influenced by typical AASHTO bridge girder sections as well as engineering judgment. Each specimen was intended to mimic the web width of girders, resulting in a depth of 6 inches (measured from the front face to the rear face). To ensure accurate strength evaluation without oversizing the specimens, an overall height of 8 inches (measured from the top surface to the bottom surface) was chosen. This height facilitated sufficient interface texture length.

Each shear plane was configured to measure 8 inches by 6 inches, producing a total area of 96 in² for the two planes in each specimen. While the widths of the outer sections of SCC varied with each interface texture, minimum dimensions were set to enable proper stress transfer and effective aggregate distribution. Notably, the minimum dimension of the SCC was set at 2.25 inches from the outer surface to the nearest peak of texture amplitude. Considering the amplitude of 0.75 inches from both 2D and 3D textures, the median interfacial plane location for the SCC was determined to be 2.625 inches from the outer surface. The UHPC section kept a minimum width of 2 inches, and the distance between the median locations of the interfaces measured 2.75 inches. The conceptual model of a typical direct shear specimen is visually depicted in Figure 3.5.

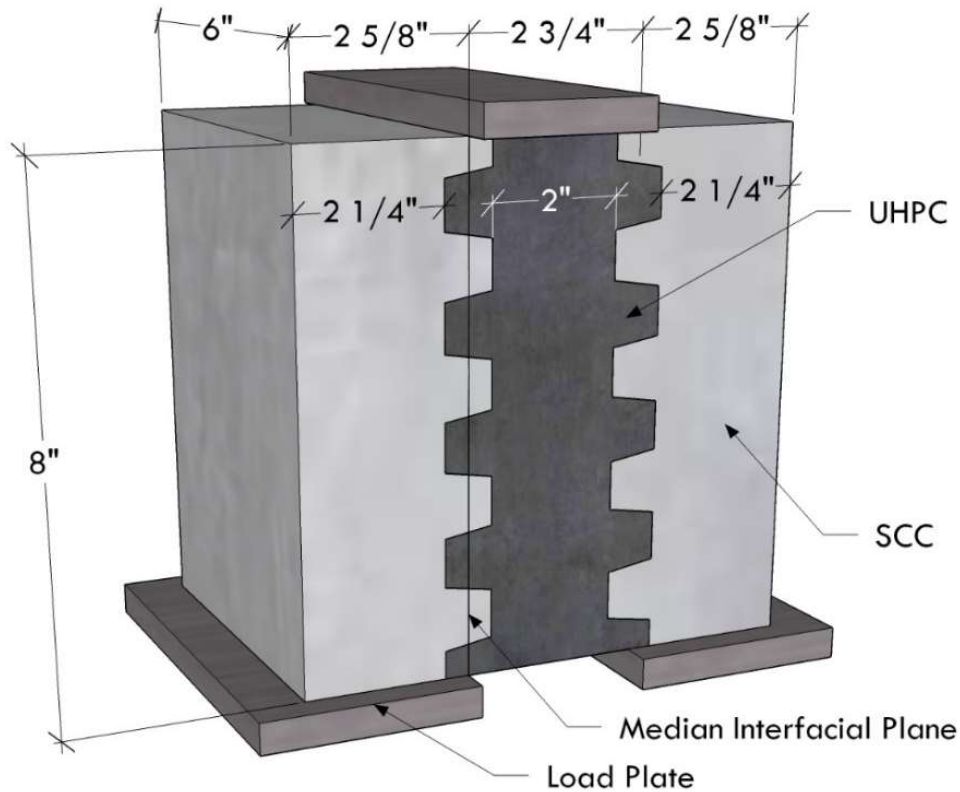


Figure 3.5: *Dimensional model of direct shear specimens*

All specimens were cast on their sides with smooth top and bottom testing surfaces created from concrete form-boards. In instances of cold joint interface specimens, the central UHPC section was initially cast with texture molds placed at the interface locations. This approach avoids potential moisture migration issues and the subsequent weakening of UHPC at interface locations. Curing procedures followed a standard procedure, including a minimum of 3 days of moist curing before removing the interface molds. At a later date, the outer SCC sections of cold-joint specimens were cast, along with both sets of BiFluid specimens. With UHPC being cast first, the Exposed Aggregate interface is impossible to achieve through traditional exposure methods due to the absence of coarse aggregate in the UHPC matrix. Instead, crushable foam with partially-depressed pieces of coarse aggregate in the foam were used to simulate an exposed aggregate interface, shown in Figure 3.6. All cold-joint interface textures were placed such that the amplitude of the texture was centered at 2.75 inches from each other (2.625 inches from the outer edges).



Figure 3.6: *Exposed aggregate interface molds for direct shear specimens*

BiFluid and BiFluid with Divider specimens adhered to the methodology outlined in Section 3.2.2, with the UHPC and SCC mixed simultaneously. Both sets of BiFluid specimens were cast before adding SCC to the cold-joint specimens. BiFluid forms were marked along the interfacial planes to assist with alignment of adding the concretes. Still, casting the BiFluid specimens without dividers was observed to be challenging, leading to difficulties in maintaining reasonable interface amplitudes. Conversely, specimens with dividers presented no such casting challenges. This was due to the implementation of the dividers which were mounted such that they could resist pressure from the fresh concrete but were easily removable.

During the initial 2 hours of the setting period, specimens were stored in an ambient lab space before being transported to a controlled temperature and humidity storage area for further curing. Over the initial 7 days of curing, the specimens were consistently kept in a fully moist condition within their molds. After a total of 21 days, the specimens were demolded and prepared for testing by marking alignment points for the loading plates.

Due to issues encountered in achieving the prescribed workability of the SCC, the first set of specimens displayed signs of segregation during casting. As a remedy, a second full set of specimens was cast to replace the first set. In the second set, some BiFluid specimens exhibited partial consolidation issues in the outer corners of the SCC portions. This was attributed to the avoidance of normal consolidation measures to maintain interface integrity. To address the under-consolidated sections, the outer SCC sections of the bottom loading surfaces were set in Hydro-Stone, creating a smooth contact surface for loading.

3.3.2 *Direct Shear Testing & Results*

Direct shear testing occurred once the SCC portions and BiFluid specimens reached a curing age of 28 days. Tests for the monolithic, compressive strength cylinders and direct shear specimens were tested on the same day. The testing arrangement for direct shear specimens included two steel plates beneath the SCC sections. The inner edge of these two plates were aligned at 2.625 inches from the outside faces of the SCC (at the median interface location). A single plate, machined to a 2.75-inch width, was placed above the UHPC section and centered (with each edge aligned at both median interfaces).

A preload of 500 lbs was applied to the direct shear specimens and increased at a rate of 288 lb/s until failure. The preload magnitude was established using the minimum possible value allowed by the testing machine. This was also chosen due to limited knowledge from the influence of preloading on interface behavior and what an anticipated preload should be used. The testing load rate was adapted from prior research conducted by Valikhani et al. (2019), who used a load rate of 210 lb/s for 6-inch cube bi-surface specimens.

Failures during testing were characterized by sudden and often explosive occurrences. Failure planes predominately occurred along the interface between concretes in the cold-joint specimens and just inside the SCC for the BiFluid sets. Failure planes only traversed across the UHPC in the BiFluid (no divider) and the 3D Trapezoidal textures where a protruding section of UHPC was in contact with a load plate (see Figure 3.7). Notably, both BiFluid sets produced the highest interfacial strengths, followed by the 3D then 2D Trapezoidal textures, the Sine Wave texture, Exposed Coarse Aggregate, and lastly, the Smooth Interface. A comprehensive summary of the testing results is provided in Table 3.2. Strengths were calculated using the maximum recorded load and the area of the failed interfacial planes.



Figure 3.7: Bottom bearing surface of BiFluid specimen No. 3²

Table 3.2: Direct shear test results

Interface Type	Avg. Shear Strength (psi)	Coefficient of Variation
BiFluid	1992	13.7%
BiFluid with Divider	1511	17.4%
Exposed Aggregate	759	14.3%
Sine Wave	929	15.8%
Smooth Interface	195	85.7%
2D Trapezoidal	1154	9.1%
3D Trapezoidal	1382	5.4%

Visual documentation of the specimens was taken before and after testing to capture the location of the failure plane and the extent of damage. This examination revealed that the chosen specimen dimensions effectively produced shear planes that operated independently without mutual influence. Observations were also made in the BiFluid (no divider) specimens, indicating large amounts of UHPC crossing the loading plane. It is possible that this observation might explain the strength differential between the BiFluid sets with and without dividers. Post-test analysis of the Smooth Interface set revealed trace amounts of UHPC fibers

² Figure 3.7 depicts failure across the UHPC protrusion on the right side of the image. The front of specimen is facing up.

bonded to the SCC along the interface edges, illustrated in Figure 3.8. Consistently lower strength values were still observed in this set, although one specimen tested at a relatively higher capacity. Consequentially, this specimen also showed higher amounts of fibers along the edges when compared to the others in this set. This suggests that the capacities are the result of imperfections from fibers bridging the interface of the smooth interface, and not from cohesion. A documentation catalog of the specimens can be found in Appendix B.

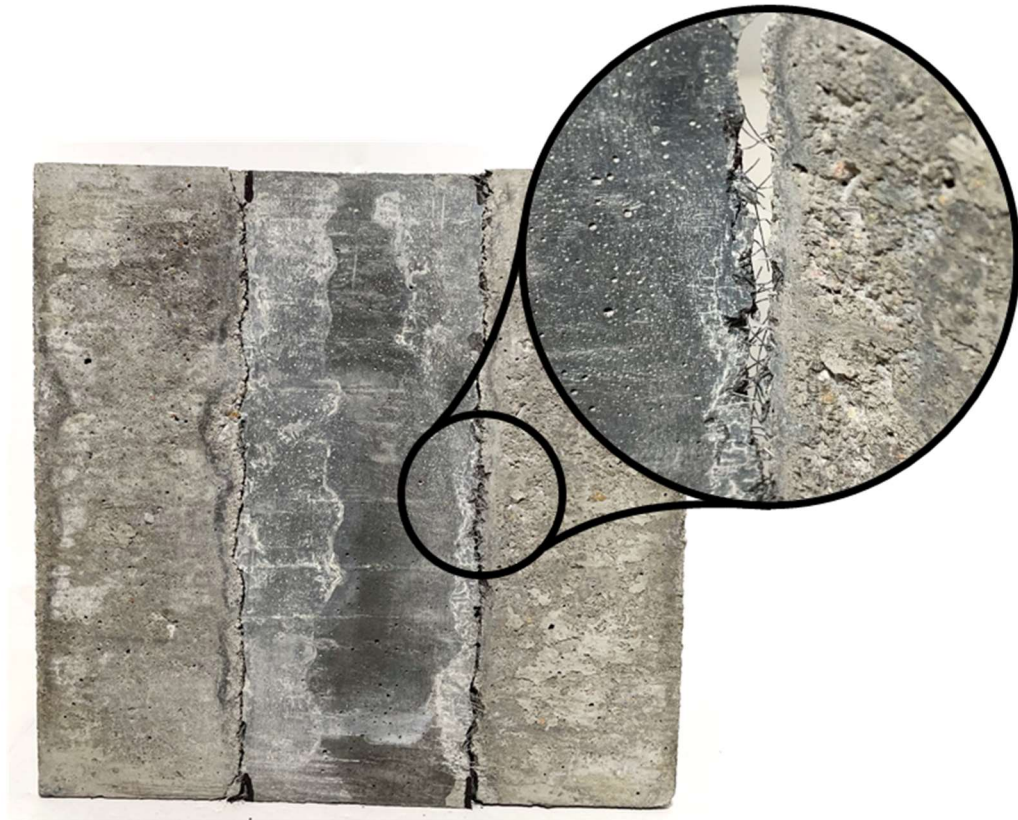


Figure 3.8: *Fibers at interface of Smooth Interface direct shear specimen*

For three specimens (one BiFluid with Divider, one Sine Wave, and one 3D Trapezoidal specimen), an initial loading occurred in each test and the machine falsely detected failure and stopped the test. These specimens were loaded up to, but not exceeding, 10 kips. An audible ‘pop’ sound was noted during the conclusion of these initial tests, possibly indicating the presence of deleterious material between the specimen and a loading plate. This imperfection appeared to correct itself upon reaching its capacity without damaging the specimen. Retesting was carried out due to lack of any clear signs of failure along the

interfaces. Among these specimens, two exhibited the highest loads within their respective texture set (BiFluid with Divider and Sine Wave). The 3D Trapezoidal specimen fell between the high and low values of its texture set. There was no clear relation drawn between this attempted preloading and the final test results.

3.4 Slant Shear Testing

Slant shear testing was selected for the small-scale program due to its stress-state similarity to the forces present in beams. The combination of compression and shear forces within slant shear specimens can provide greater insight into the performance of the interface between two concretes, crucial for the characterization of hybrid girders. The design of the slant shear specimens was parallel to the prescribed layout listed in ASTM C882 (2020). Unlike direct shear, slant shear specimens incorporate only one interfacial plane. This plane is centrally positioned within the specimen and is inclined at a 30-degree angle from the vertical axis. Prestressed beams are known to exhibit shear planes shallower than non-prestressed beams (typically 45 degrees), supporting the 30-degree orientation used in these specimens. Again, to ensure statistical significance, a total of three specimens were subjected to analysis for each type of interface.

Continuing into the subsequent phases of the small-scale program, modifications were introduced to the SCC mix design. Specifically, fly ash was incorporated at a ratio of 25% relative to the total cementitious materials. The primary objective of this adjustment was to enhance the SCC's flowability while mitigating the risk of segregation. This mix design is listed in Appendix A.

3.4.1 Construction of Slant Shear Specimens

The design, layout, and dimensions of the slant shear specimens were guided by standard AASHTO bridge girder web widths, parallel to the direct shear testing. Furthermore, an appropriate length was selected to ensure accurate quantification of interface strength. To facilitate simpler construction of interfaces, the specimens were crafted as square prisms rather than cylinders, with an overall height of 13 inches and a cross-sectional area of 6 inches by 6 inches. This design choice facilitated the assembly of interfaces for the cold-joint specimens. The cumulative, median interfacial plane area summed up to 72 in².

Following the same casting procedure employed for the direct shear specimens, the cold-joint specimens were initially cast with the UHPC sides first. After a minimum curing time of 24 hours, the interfaces were removed, preparing the specimens for casting the SCC. Both BiFluid and BiFluid with Divider sets were cast concurrently with the SCC sections of the cold-joint specimens. As in the direct shear specimens, the inside of the formwork for the BiFluid set was again marked to establish the location of the interfacial plane. This marking proved essential for aligning the concretes accurately with the predetermined interface position. The BiFluid with Divider specimens incorporated similar mounting hardware to facilitate the secure attachment of the removable dividers. All specimens were cast with formboards against the loading surfaces to ensure a smooth, flat surface. The curing process extended until the concretes had obtained an age of 28 days, mirroring the curing conditions applied to the direct shear specimens. Figure 3.9 shows a well-formed slant shear specimen after casting.



Figure 3.9: *Slant shear specimen with Sine Wave interface*

A subset of the cold-joint specimens exhibited a challenge during the SCC casting process within the acute angle of the formwork. This difficulty caused an area of incomplete consolidation in the Exposed Aggregate and 3D Trapezoidal sets despite a 22-inch flow for the SCC, depicted in Figure 3.10. Consequently, it was deduced that the textural protrusions impeded the SCC from achieving full consolidation within this specific section. Additional procedures were taken after testing to document and account for this defect, see Section 3.4.2.



Figure 3.10: *Unconsolidated areas of slant shear specimens*

3.4.2 Slant Shear Testing & Results

The testing protocol adhered to the guidelines specified in ASTM C39 (2021), following the instructions outlined in ASTM C882 (2020) for slant shear testing. As per the standard practice, compressive loading was applied until either the interface or the specimen itself failed. The preload values were determined based on shear strengths obtained from prior direct shear testing. Loading rates were conducted in accordance with ASTM specifications until failure was reached. In instances where SCC sections exhibited incomplete consolidation, the area of the failure plane in the SCC was measured using calipers. Shear strengths were calculated by using trigonometric relationships between the total load and a load acting parallel to the interfacial plane. This resulting magnitude was divided by the total area of the failure plane; the equation used to calculate shear stress is given as Equation 3-1. In this slant shear stress equation f_{ss} is shear stress, P is total applied load, θ_{ss} is the angle of the failure plane from vertical, and $A_{failure}$ is the area of the failure plane.

$$f_{ss} = P \cdot \cos(\theta_{ss}) / A_{failure} \quad \text{Equation 3-1}$$

To ensure the use of preloads permitted by the ASTM, the magnitudes were derived from taking the maximum shear stress values obtained from direct shear testing and applying half that stress to the slant shear interfacial plane. Employing this methodology, the more conservative shear strength from the BiFluid with Divider interface type was used for calculating the preload for both BiFluid sets, resulting in a preload of 50 kips. Likewise, the preload for the cold-joint specimens was set at 18 kips, a value obtained from the direct shear strengths of the Exposed Aggregate and Sine Wave textures. The only exception to this approach was observed in the case of the Smooth Interface set, where the preload was limited to 500 lbs.

The results from testing showed higher overall shear strengths as compared to the direct shear testing, most likely due to the clamping (normal) component of the applied load (i.e., the component of the applied load perpendicular to the interfacial plane). Both BiFluid and BiFluid with Divider sets exhibited vertical cracking throughout the specimens, with failure patterns in the SCC region indicative of compressive-type failures (see Figure 3.11). The BiFluid with Divider specimens demonstrated more violent failures in comparison to the BiFluid specimens, with conical failures in the SCC sides. In the cold-joint interface specimens, failure predominantly occurred along the interface, just inside the SCC region. Among the cold-joint specimens, the 2D Trapezoidal texture exhibited the most violent failure behavior, followed by the Sine Wave texture; both demonstrated higher intensity failures than the BiFluid with Divider specimens. Interestingly, these three interface types yielded the highest average shear strengths, with the order being BiFluid with Divider, Sine Wave, and 2D Trapezoidal. Subsequent in the ranking were BiFluid (no divider), 3D Trapezoidal, Exposed Aggregate, and the Smooth Interface recording the lowest shear strength. The average shear stress values for each interface type are summarized in Table 3.3. A distinctive characteristic of the Smooth Interface specimens was their failure located almost exclusively at the interface, exhibiting only minimal SCC cracking. The Smooth Interface specimens also gave a high coefficient of variation for shear strengths, but all tested strengths were below 105 psi.

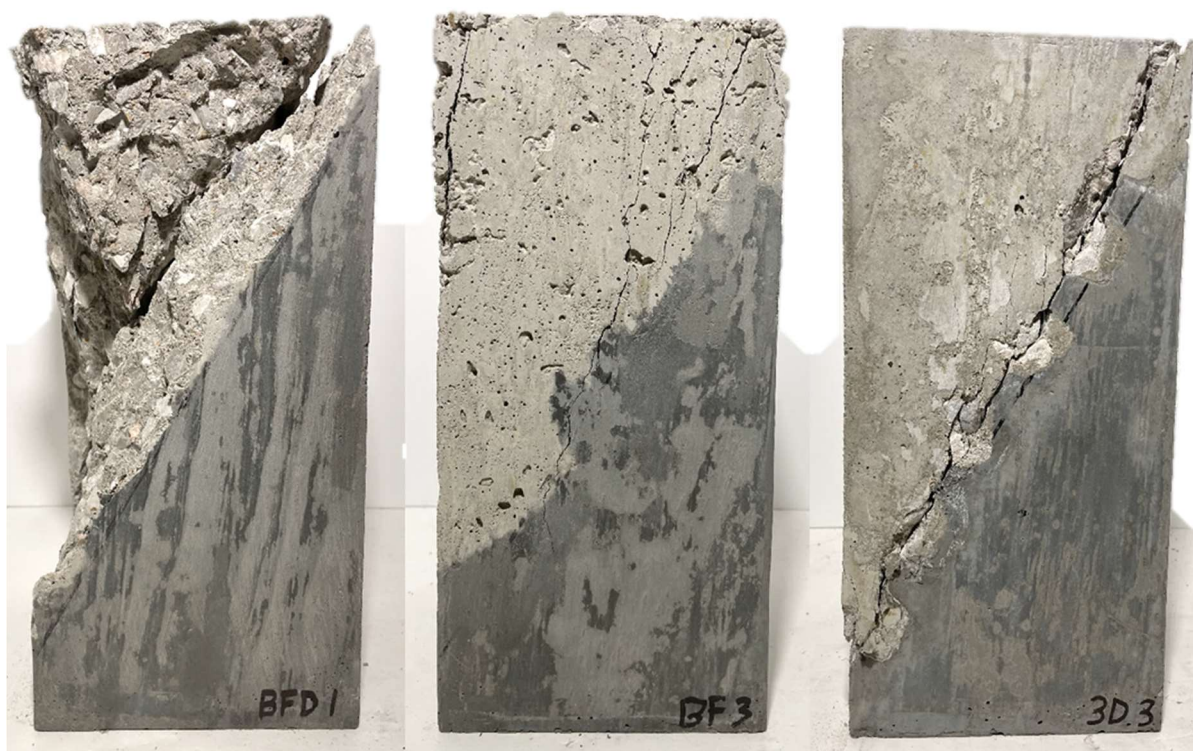


Figure 3.11: Failure planes of BiFluid with Divider (left), BiFluid (center), and 3D Trapezoidal (right, included for comparison) slant shear specimens

Table 3.3: Slant shear test results

Interface Type	Avg. Shear Strength (psi)	Coefficient of Variation
BiFluid	2827	5.6%
BiFluid with Divider	3488	6.1%
Exposed Aggregate	2562	8.3%
Sine Wave	3227	3.3%
Smooth Interface	54	66.4%
2D Trapezoidal	2910	5.2%
3D Trapezoidal	2751	4.3%

3.5 Modulus of Rupture Testing

The determination of rupture capacity through flexural testing holds significant importance for the hybrid girder design, particularly when the interface is located within a zone susceptible to flexure-shear capacity concerns. Given the current lack of comprehensive data pertaining to failure modes and performance capacities of hybrid girders, this parameter stands as an appropriate metric justifying further investigation.

Incorporating results from the preceding phases, the top-performing textures – namely, both BiFluid interfaces, 3D and 2D Trapezoidal, and Sine Wave textures – were integrated into the modulus of rupture (MOR) testing phase. The inclusion of these textures was aimed at further evaluating their performance characteristics with respect to different stress scenarios. In the interest of streamlining the process and facilitating focused classification of the interfaces, the Smooth Interface and Exposed Aggregate textures were omitted, effectively narrowing down the scope to include the more viable interfaces. Again, to ensure statistical significance, a total of three specimens were subjected to analysis for each type of interface.

3.5.1 *Construction of Modulus of Rupture Specimens*

The MOR testing specimens were designed to conform to the criteria outlined in ASTM C78 (2022). Continuing with the methodology employed in the earlier phases of small-scale testing, the choice of standard 6-inch by 6-inch MOR specimens was retained. Interfaces were strategically positioned at the midspan of each specimen. The configuration of third-point loading was implemented to provide a uniform moment section overlapping the interface.

The casting and curing procedures of the two preceding phases were adopted for the MOR testing. In summary, the sequence of casting included the UHPC sections of the cold-joint specimens, followed by the SCC portions and both the UHPC and SCC portions for the BiFluid and BiFluid with Divider sets. Throughout the casting process of the MOR specimens, no difficulties were encountered.

After a curing duration of 28 days for both the SCC and BiFluid castings, the flexural testing was initiated. This timeframe allowed for the attainment of previously adopted curing conditions before subjecting the specimens to testing.

3.5.2 Modulus of Rupture Testing & Results

The testing procedure for the MOR specimens adhered to the specifications outlined in ASTM C78. This included the ASTM designated loading arrangement, load rate, and calculation of the failure stress using specimen dimensions and peak load. Keeping with the same comprehensive documentation, photographic records were obtained both before and after the testing phase.

In the context of the BiFluid and BiFluid with Divider sets, the typical failure patterns were characterized by rupture planes within the SCC, near the interface, at approximately one inch from the UHPC section. Conversely, for the cold-joint specimens, failure occurred predominantly at the interface, with small amounts of residual SCC bonded to the UHPC. It was observed that the bonding between the SCC and UHPC predominantly occurred in the recesses or between protrusions of the UHPC surface. Figure 3.12 depicts these details and the representative failure modes for each interface set.



Figure 3.12: Typical MOR failures for each interface type³

³ Specimens pictured in the figure are: 2D Trapezoidal (top left), 3D Trapezoidal (top right), Sine Wave (center), BiFluid (bottom left), and BiFluid with Divider (bottom right)

The results of the rupture strength testing yielded a ranking that closely paralleled the findings of the direct shear testing. However, the exception to this trend was evident in the BiFluid and BiFluid with Divider specimens, which did not possess the highest strengths. Instead, the order of rupture strengths starting with the highest is as follows: 3D Trapezoidal, 2D Trapezoidal, BiFluid with Divider, BiFluid, and, finally, the Sine Wave texture registering as the lowest strength. Table 3.4 provides an overview of the calculated average values for the modulus of rupture corresponding to each interface type. The interpretation drawn from these results suggest that the greater amplitudes exhibited by the two trapezoidal textures contribute to a more gradual stiffness transition through the interface, thereby correlating with higher rupture strengths. Conversely, the BiFluid sets exhibited a shallower transition zone at the interface, regardless of chemical intermingling. Similarly, the Sine Wave texture's lower rupture strength could be attributed to the lower texture amplitude plus the absence of chemical bonding at the interface, creating a discontinuity at the cold-joint interface that gave a rupture strength lower than that of the SCC alone.

Table 3.4: Modulus of rupture test results

Interface Type	Avg. MOR (psi)	Coefficient of Variation
BiFluid	455	2.6%
BiFluid with Divider	495	5.2%
Sine Wave	417	1.5%
2D Trapezoidal	512	5.3%
3D Trapezoidal	608	4.3%

3.6 Discussion of Small-Scale Program Results

The testing for this program encompassed three phases involving several interface types, adapted from prior research, to evaluate various stress scenarios imposed on the interfaces. The results, summarized in Table 3.5, revealed that the strength of each interface is contingent upon the existing stress conditions at the interface and its ability to transmit stiffness between concretes.

Table 3.5: Summary of the average interfacial strengths from small-scale program testing

Interface Type	Direct Shear Strength (psi)	Slant Shear Strength (psi)	MOR (psi)
BiFluid	1992	2827	455
BiFluid with Divider	1511	3488	495
Exposed Aggregate	759	2562	--
Sine Wave	929	3227	417
Smooth Interface	195	54	--
2D Trapezoidal	1154	2910	512
3D Trapezoidal	1382	2751	608

Under pure shear loading conditions, the strength was observed to be linked with bond type, amplitude, and texture. Interfaces with greater physical variation perpendicular to the interfacial plane exhibited higher strengths. This effect generated a zone parallel to the interface, characterized by a stiffness gradient transitioning between the two concrete sections, effectively causing the failure plane to shift into the weaker stratum. Furthermore, the findings validated Jang et al.'s theory regarding enhanced shear strengths due to multi-directional interface texture variations (2017). However, this was still contingent on the material strength crossing the interface, especially when comparing the Exposed Aggregate interface to the 3D Trapezoidal texture. The former utilized coarse aggregate as the load-carrying mechanism, whereas the latter used protruding UHPC knobs. Improved performance also stemmed from mitigating discontinuities present in cold-joint interfaces through chemical bonding at the interface, as seen in the BiFluid approaches. Among the three lowest amplitude, two-dimensional textures (BiFluid with Divider, Sine Wave, and Smooth Interface), the BiFluid with Divider set resulted in significantly higher strengths. It is deduced that of the three characteristics, chemical bonding exerts the most influence on pure shear interface loading.

Conversely, under the combined loading of compression and shear through slant shear testing, different interface characteristics led to increased strengths. Notably, the strengths from combined loading exhibited a nearly inverse ranking compared to pure shear loading. Higher amplitude textures (2D and 3D Trapezoidal) were less effective in contrast to the Sine Wave texture. This discrepancy could potentially arise from the orientation of the angled sides

of trapezoidal protrusions and their efficiency in transmitting stress into the weaker strata. The alignment of the normal force resulting from these side angles and the orientation of the interfacial plane could be such that it creates a shear failure plane through the SCC, along the face of the UHPC protrusions. This concept is illustrated in Figure 3.13. Further exploration of this theory might also clarify the lack of discernible benefit from utilizing the 3D Trapezoidal texture over the 2D variant for this loading scenario. Closer investigation of the interfacial areas in each of the two concretes revealed that the 3D Trapezoidal texture encompassed a larger area of UHPC at the median interface location, measuring 23.63 in², compared to the 22.50 in² of the 2D Trapezoidal interface. Consequently, the 2D interface featured more SCC crossing the interface, lowering the total stress exhibited on the SCC, and possibly contributing to its greater shear resistance. The two BiFluid sets demonstrated failure patterns more consistent with monolithically cast specimens, displaying compressive failures rather than interfacial failures. This suggests that the load required to induce bond failure in the BiFluid sets surpassed the load needed to fail the specimen in compression.

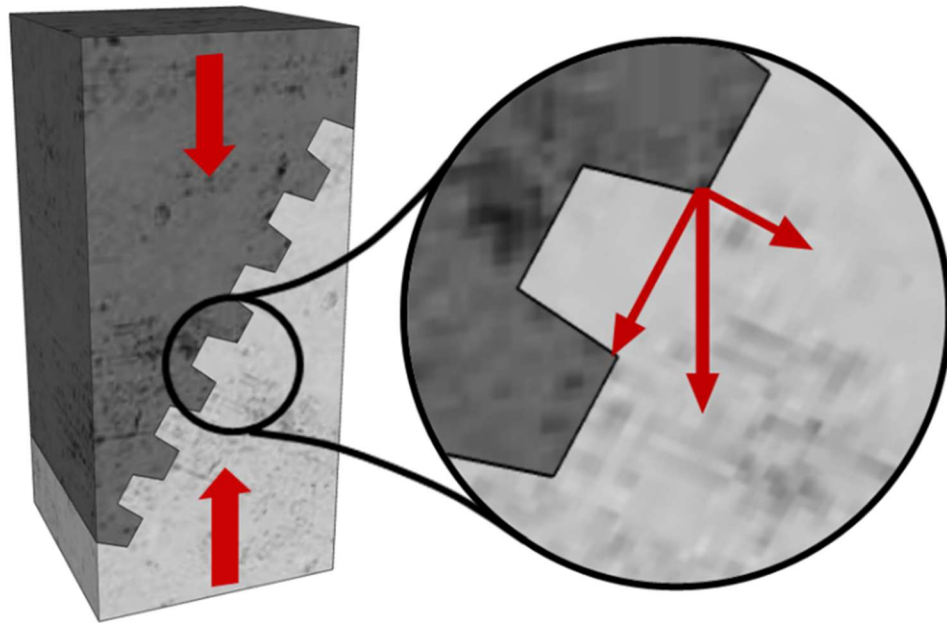


Figure 3.13: Force diagram of Trapezoidal textures of slant shear specimens

The outcomes of the flexural strength testing closely mirrored the pure shear results, with interface performance primarily contingent on the width of the interface transition from one concrete to the other. As detailed in Section 3.3.2, the 2D and 3D Trapezoidal textures resulted in a larger zone of gradual stiffness change from UHPC to SCC. In contrast, the two BiFluid methods exhibited shallower transition zones, yielding lower rupture strengths, with the failure plane diverted into the SCC. The Sine Wave texture displayed the weakest strength, attributed to the lack of chemical bonding at the interface and a shallower texture amplitude. All specimens yielded values either at or below the calculated rupture strength as per ACI 318 (2019).

Considering the results outlined in this chapter, it can be concluded that both BiFluid sets and both Trapezoidal texture sets delivered the best overall performance. The BiFluid sets, both with and without Divider, exhibited failures indicative of homogeneous concrete with no interface. Conversely, the 2D and 3D Trapezoidal textures did not demonstrate this behavior, yet their strengths were comparable to those of the BiFluid methods. Across all three testing methodologies, the failure planes were consistently located within the SCC.

Chapter 4: Large-Scale Program – Hybrid Girder Testing

4.1. Introduction

The primary goal of the large-scale program was to take the testing data obtained from the small-scale program and apply it to hybrid UHPC/conventional concrete prestressed girders and evaluate their performance. The rationale behind utilizing hybrid girders, as opposed to traditional ones, stemmed from the need to address common issues associated with the end regions of prestressed beams and judiciously employ the higher cost Ultra-High-Performance Concrete (UHPC) material. These issues include highly congested formwork resulting from internal reinforcement, cracking induced by prestress release, and post-installation deterioration. Hybrid girders that incorporate UHPC, a material recognized for its high strength and durability properties, in the end regions may mitigate these concerns. However, the continuity in composite members like hybrid girders and their response to loading was a question of interest. The most likely location for discontinuity resides at the interface between the two concrete materials. To address this issue, the large-scale program conducted an analysis of the load-deflection response of hybrid girders containing the top performing interfaces evaluated in the small-scale program.

The hybrid girders in this program were designed and constructed using the same ODOT Class AA Self-Consolidating Concrete (SCC) mix design and employed in the small-scale program. To account for the smaller member cross sections in the large-scale program, a reduction in coarse aggregate size was implemented, reducing it from 3/4-inch to 5/8-inch nominal. Furthermore, the J3 UHPC mix design, previously developed at the University of Oklahoma, was utilized once again. However, a reduced fiber content was employed in the large-scale program, as explained in Section 4.2.1.

The objective of this program was to assess the performance of the interface under failure conditions. To induce these conditions, the girders were subjected to three-point loading, with the applied load at one-third of the overall span and corresponding to a distance of at least three times the effective girder depth. The interface was positioned at the midpoint between the support and the applied load within this one-third span, as shown in Figure 4.1.

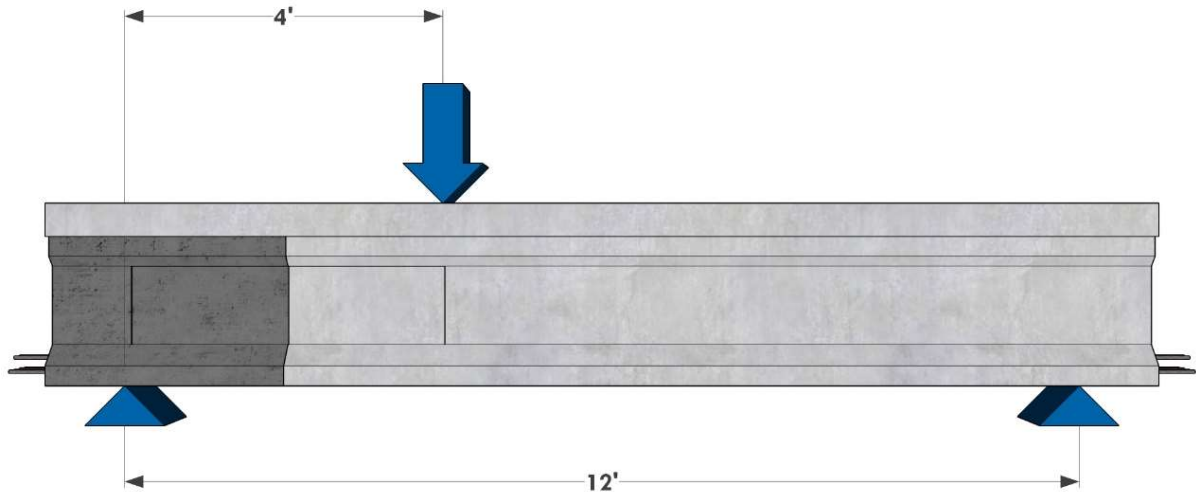


Figure 4.1: *Three-point load arrangement for hybrid girder testing*

4.2. Hybrid Girder Design & Construction

4.2.1 Section & Reinforcement Design

The chosen section for the hybrid girders was a half-scale AASHTO Type II girder with an increased web height, as depicted in Figure 4.2. This section had demonstrated effective sizing in several prior studies conducted at the University of Oklahoma and was readily accessible for research purposes. Given the physical configuration of the hybrid girders, with UHPC and the interface located at the end regions, the primary focus was on evaluating shear failure near or at the interface.

The design process began with the known section size and material properties of the proposed concretes. Taking these values, iterative determination of the flexural capacity was achieved by calculating shear capacities until the desired failure mode could be realized. It was discovered that, to obtain shear failure, the section required a high flexural capacity. Adding a deck section above the girder and a substantial amount of prestressing in the bottom bell was necessary to induce shear failure while remaining within the acceptable flexural capacity limits.

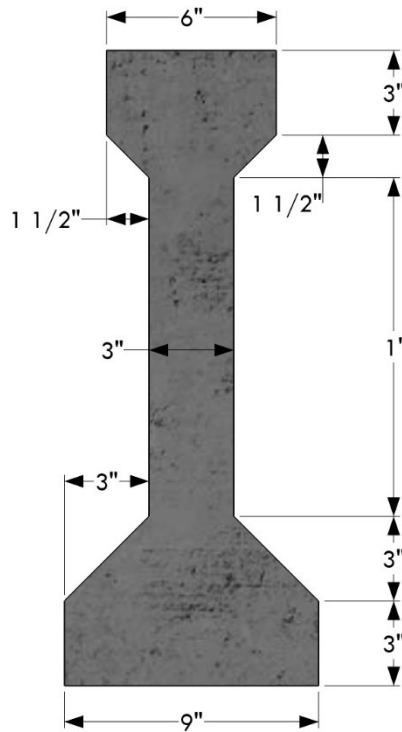


Figure 4.2: *Girder section dimensions*

Shear capacity was calculated independently for both the SCC and UHPC. The shear capacity of the UHPC was computed using average values obtained from design equations collected from Graybeal (2019), the Japanese Society of Civil Engineers (JSCE) provided in Hamilton et al. (2020), and the French Standard Institute (2016). Next, using the UHPC as the benchmark for minimum design capacity, the SCC was designed using the ACI Detailed Method to ensure that the shear capacity near the interface was comparable to the capacity of the UHPC. The minimum flexural capacity of the girder was then determined from same load required to induce shear failure in the UHPC with sufficient reserve capacity to ensure a shear failure prior to a flexural failure.

Each of the three methods used to calculate shear in the UHPC accounted for various section characteristics and material properties, including tensile strength, shear angle of inclination, and the lever arm of internal forces. The UHPC shear equations developed by the JSCE (Equation 4-1) and those from the French Standard Institute (Equation 4-2) separately calculated contributing factors for shear and combined them to obtain the total shear capacity, as shown in Equation 4-3. Graybeal's method (Equation 4-4) already provides the total shear capacity of the UHPC, accounting for these individual factors.

$$V_f = \left(\frac{f_{vd}}{\tan \theta} \right) b_w \frac{z}{1.15 \cdot \gamma_b} \quad \text{Equation 4-1}$$

$$V_f = A_{fv} \cdot \sigma_R \cdot \cot \theta \quad \text{Equation 4-2}$$

$$V = V_c + V_f + V_s + V_p \quad \text{Equation 4-3}$$

$$V_{UHPC} = \gamma_f \cdot b_w \cdot d_v \cdot f_{cr} \cdot \cot \theta \quad \text{Equation 4-4}$$

Total shear capacity of the UHPC defined in Equation 4-3 as V is a summation of the contribution from concrete (V_c), the contribution from fibers (V_f), the contribution from mild-steel reinforcement (V_s), and the contribution from prestressing (V_p). In Equation 4-1, V_f is defined as shear contribution from the UHPC fibers, f_{vd} is the tensile yield strength of the UHPC, θ is the angle of the shear crack, b_w is effective width, z is the effective depth, and γ_b is a factor with a value of 1.3. For Equation 4-2, V_f is taken as shear from the UHPC fibers, A_{fv} is the area projected onto the cross section of the area that the fibers are engaged (taken as the effective width multiplied by 90% of the effective depth), σ_R is the post-cracking strength of the UHPC, and θ is the shear angle of inclination. Graybeal's equation (4-4) for the total capacity of the UHPC (V_{UHPC}) uses a fiber orientation reduction factor (γ_f) which is undefined and taken as 1.0 for these calculations. It also uses effective width (b_w), UHPC tensile cracking strength (f_{cr}), angle of shear cracking (θ), and the moment arm between internal tension and compression forces (d_v) was taken as 90% of the effective depth to be consistent with the values used in Equation 4-2.

For the previously mentioned equations, the value used for the shear crack angle (θ) was 35 degrees, based on the contribution of prestressing. The effective width (b_w) was taken as the web width of the shear span and the effective depth (z) was taken as the depth to the centroid of prestressing. Values for tensile and rupture strength utilized in these calculations were derived from prior research at the University of Oklahoma, which characterized the J3 UHPC mix with varying fiber percentages (Dyachkova & Floyd, 2020). Due to the relatively high shear resistance offered by the previous equations, the UHPC mix design was adjusted to incorporate a 1-percent fiber volume for the hybrid girder testing. Following the same logic,

the web section within the shear span was also reduced to a width of 1.5 inches, as illustrated in Figure 4.3, with no additional vertical shear reinforcement included in the UHPC web. Beyond the shear span test region, the SCC section maintained the 3-inch web width.

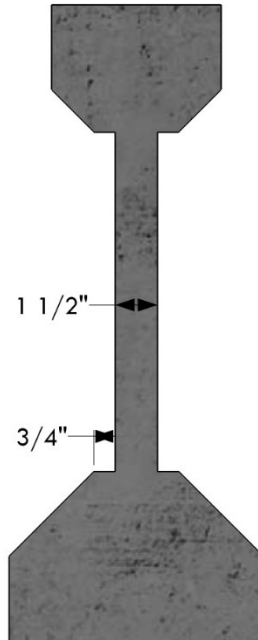


Figure 4.3: *Girder cross section near the interface*

The design of shear reinforcement for the SCC followed the ACI Detailed Method to obtain a section capacity near the interface equal to that of the UHPC. The same methodology was applied to design vertical reinforcement for the remaining SCC section, opposite to the interface. The resulting stirrup design utilized two No. 3 bars placed at 3.5 inches on center with a 2-inch clear distance from the interface and 8 inches on center for the remainder of the girder. These stirrups extended into the deck section (cast later), serving as the primary resistance to horizontal shear between the deck and the girder. Horizontal shear between the deck and the UHPC was also resisted by two No. 3 bars extending from the top bars of the girder into the deck. These shorter stirrups were spaced at 6 inches on center, starting at a clear distance of 2 inches from the interface. The final hybrid girder reinforcement design is provided in Appendix A.

The deck section had been developed in prior research at the University of Oklahoma and was adopted for this program. The section size and reinforcement design were chosen with the intent to replicate an equivalent deck section using a standard girder spacing commonly

found in Oklahoma bridges. The prestressing strands chosen were selected on the basis of availability, this coupled with the flexural design requirements resulted in a total of six 0.50-inch special seven-wire strands in the bottom bell of the girder. These grade 270 strands were pretensioned to 180 ksi. An analysis of the resulting stresses at release indicated high tensile stresses at the top of the girder, requiring the placement of two No. 5 longitudinal bars in the top bell. The design of these reinforcing bars followed ACI 318-14 R18.4.1(c) (2019) guidelines, and Figure 4.4 shows a model of the resulting girder reinforcement.



Figure 4.4: Initial prestressing and mild-steel reinforcement design for hybrid girders

4.2.2 Hybrid Girder Construction

The size of the section, the extents of the two concrete types, and the internal reinforcement followed a consistent layout for all hybrid girders, regardless of the interface method used. Preexisting formwork for the girder section utilized in this program facilitated rapid and efficient casting of girders. Initially, one side of these forms was mounted onto an elevated precasting table while the prestressing strands were placed. These strands were originally tensioned to 6.5 ksi (1000 lbs per strand) to ensure their tautness, which in turn improved the process of placing stirrups. The top longitudinal bars for the girder were suspended in place using tie wire, featuring 180-degree hooks at both ends in adherence to ACI 318 code requirements. Stirrups for the SCC sections were attached by tying them to the top longitudinal bars, securing them centrally within the web section.

Due to the dimensional constraints of the reduced web section within the shear testing region, the stirrups were paired together to maximize coverage, as shown in Figure 4.5. To prevent movement during pretensioning and casting, the stirrups were braced with 0.25-inch steel rods fastened together using tie wire. The shorter stirrups for horizontal shear in the UHPC were directly tied to the top bars of the girder.

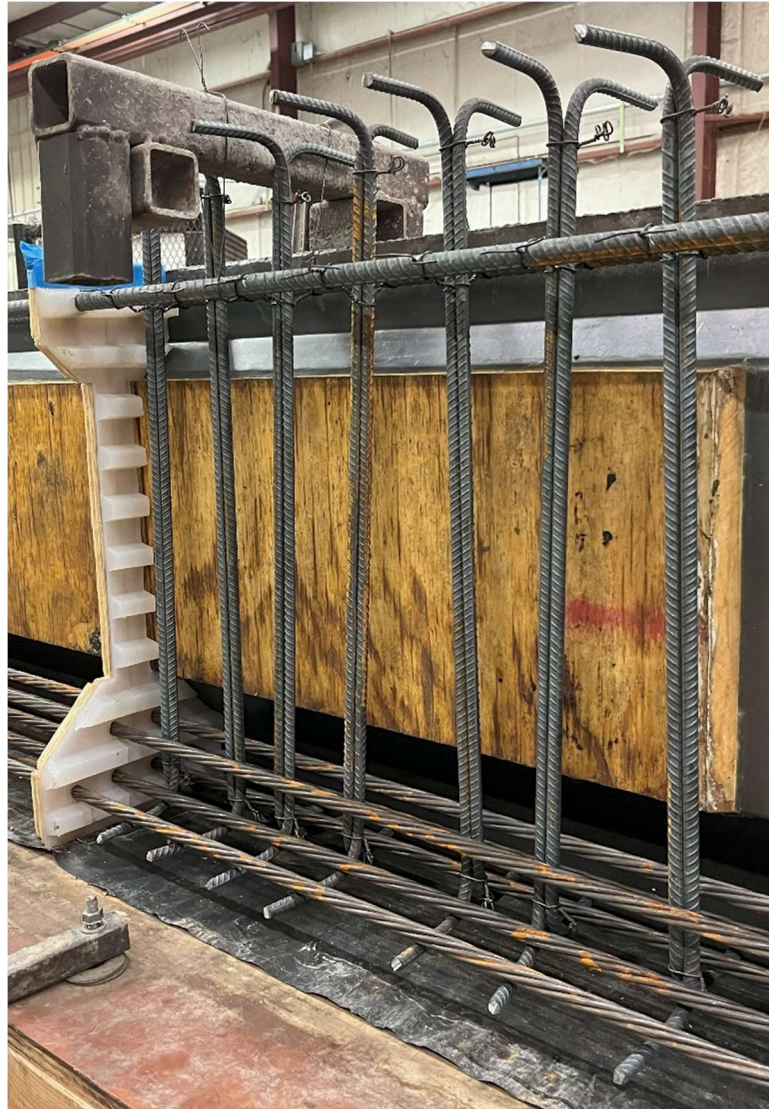


Figure 4.5: *Vertical reinforcing stirrups at the reduced web section⁴*

For visual reference, Figure 4.6 provides a comprehensive view of the reinforcement of a hybrid girder before the placement of the other form for casting. Directly before the mixing and casting of the concrete, the strands were fully tensioned to the specified design stress. Girders were then allowed to undergo moist-curing using wet burlap for three days, coinciding with the SCC reaching a minimum strength of 4500 psi. After, the prestressing was released, and the strands were cut approximately 6 inches from each end of the girder.

⁴ Stirrups in Figure 4.5 are two No. 3 bars paired together and the 2D Trapezoidal texture is pictured at the interface.



Figure 4.6: Internal reinforcing of hybrid girder before casting, (a) elevation of full hybrid girder reinforcement, (b) short stirrups between the UHPC and deck, and (c) SCC end reinforcement and top bar hooks

The initial set of hybrid girders cast utilized the BiFluid interface. The forms for this process did not include dividers or obstructions. Instead, both the UHPC and SCC were added at rates that facilitated the proper alignment of the interface. The UHPC was introduced at the girder's end, while the SCC was initially added at midspan. This approach reduced the fill rate of the SCC and allowed for better control of the UHPC flow during casting. Maintaining a vertical interface was achieved by ensuring a steady flow of both concrete types after their initial contact at the interface. It is worth noting that the higher viscosity of the UHPC initially

caused challenges in creating uniform contact with the SCC at the interface, resulting in an appearance of SCC extending beyond the interface location in the bottom bell. However, it was evident during casting that the UHPC indeed reached the interface directly beneath the web section, visible on the base of the bottom bell in Figure 4.7. Occasionally, deviations in the fill rate and interface alignment occurred when retrieving additional concrete to add to the forms. The depictions of the resulting interfaces for each of the BiFluid hybrid girders with these alignment details are visible in Figure 4.8.

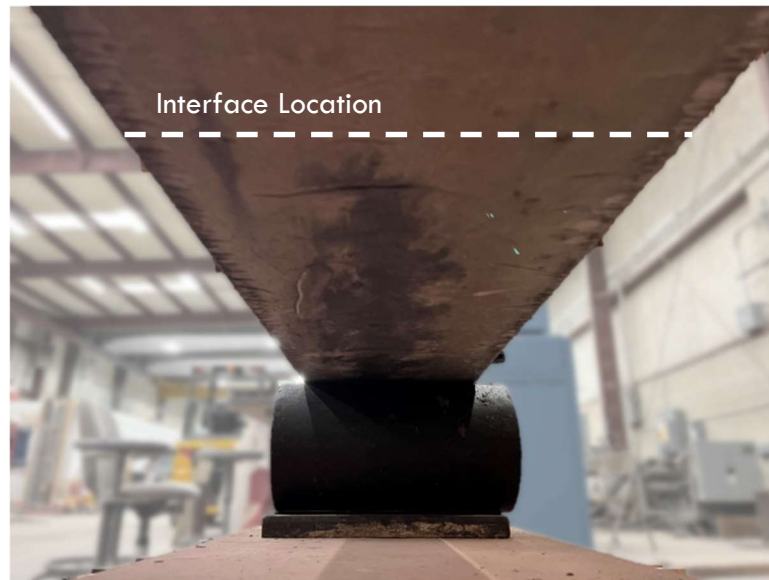


Figure 4.7: Bottom of BiFluid girder no. 2 with UHPC extending to interface location



Figure 4.8: Interface alignment of BiFluid girder no. 1 (left) and BiFluid girder no. 3 (right)

Similar issues concerning concrete passing the interface were also observed during the casting of BiFluid with Divider hybrid girders. Although the divider did facilitate improved placement and alignment of the interface, some of each concrete type passed the interface due to incomplete sealing of the section by the divider. Given the complexity of the girder's cross section and the type of formwork used in this program, a vertically extracted, transparent acrylic interface divider was implemented with a width matching the girder web. For this casting method, the same procedure used for the BiFluid casting was applied, then the divider was removed immediately by gently pulling it upward slowly and out of the concrete, as shown in Figure 4.9. The design for the divider was intended to facilitate vertical alignment of the interface at the web for the full depth of the section. However, due to non-uniformities in the formwork within the web section, openings at the bottom half of the web allowed UHPC to flow past the divider. Correspondingly, the absence of a full divider in the bottom bell gave casting results similar to the BiFluid girders, including the appearance of SCC flowing past the interface.



Figure 4.9: Removal of divider immediately after casting the second BiFluid with Divider girder

Both sets of hybrid girders with the 2D and 3D Trapezoidal interface textures were constructed with the SCC and UHPC cast at different times. Before placing the reinforcement, the interface textures were created using a silicone mold kit, identical to the approach that was described in the small-scale program detailed in Chapter 3. Due to safety concerns, the casting procedure was reorganized to add the SCC section of the hybrid girders first, followed by the UHPC. Consequently, this meant that the interface molds used for the 2D and 3D textures were negatives of the textures used in the small-scale testing. These interfaces were modified to include holes to allow for the longitudinal bars and prestressing strands to pass through. A plywood backer board was used to brace the silicone texture, preventing movement during casting, as depicted in Figure 4.10. After casting the SCC, one side of the formwork was temporarily detached, and the backer board and texture were removed. Following removal and thorough cleaning, short horizontal shear stirrups were tied to the top bars of the girder, and the side of the form was reset in place. During this time, the SCC was allowed to moist-cure, and the interface kept saturated. Saturated conditions for the SCC side of the interface continued until casting the UHPC to prevent any potential moisture migration concerns between the UHPC and SCC. After the UHPC was cast, the girder was allowed to cure for an additional 24 hours, reaching a UHPC compressive strength of at least 8000 psi before prestressing was released. The girders were then permitted to cure further, covered with wet burlap for an additional 48 hours after release.



Figure 4.10: Interface texture placement with plywood backerboard before and after casting SCC section of the first 2D Trapezoidal hybrid girder

Once each girder section was completed, it was removed from the prestressing bed, and deck reinforcement and formwork were added. Longitudinal reinforcement was tied to the stirrups protruding from the top of the girder, and the deck formwork was assembled. The deck was cast using the same SCC employed for the hybrid girders. Workability of the concrete was not an issue for deck construction; therefore, the decks were cast immediately after the hybrid girders, except for the final hybrid girder deck, which was cast separately. The same curing procedure applied to the girder sections was also implemented for the deck, involving moist curing using wet burlap for three days, as depicted in Figure 4.11.



Figure 4.11: Moist curing of deck section of hybrid girder specimen using saturated burlap

4.3. Testing Method

Once the girder sections had reached their full 28-day strength, and the compressive strength of the decks had exceeded 6500 psi, the hybrid girders were prepared for testing. The load arrangement incorporated pin-and-roller supports positioned one foot from each end, equivalent to a 12-foot span. The applied load was situated 4 feet from the support nearest to the interface and 2 feet from the interface itself. Load and deflection were monitored using a 200-kip capacity load cell and two wire potentiometers, one on each side of the girder. This instrumentation was all positioned at the location of the applied load. A schematic illustrating the testing setup is shown in Figure 4.12.

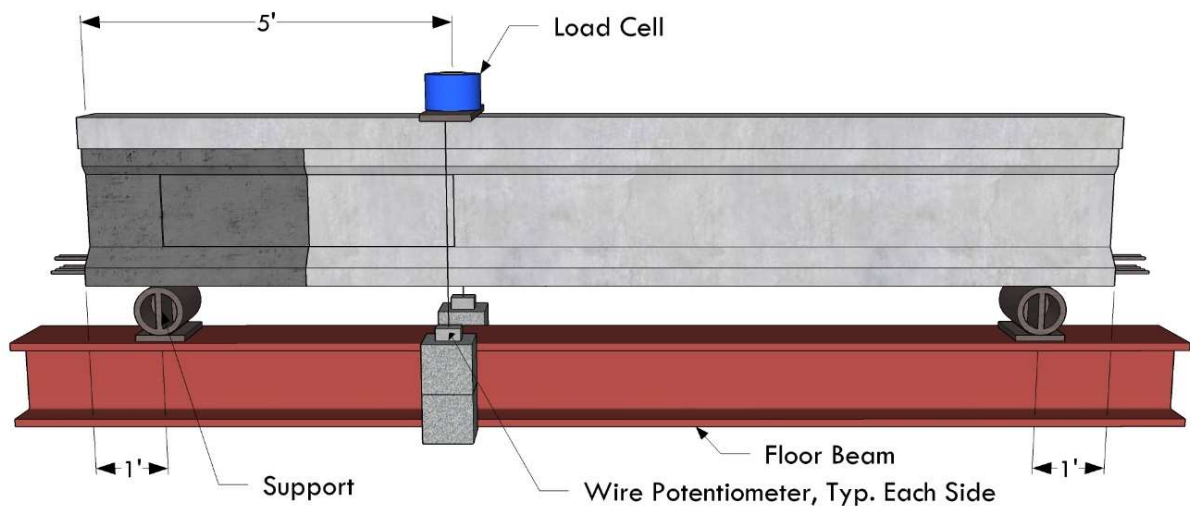


Figure 4.12: Hybrid girder testing instrumentation arrangement

Before commencing the testing, any cracks resulting from prestress release were carefully marked, and camber measurements were recorded at one-foot intervals along the span of the girder. These measurements were obtained using a taut stringline spanning between each support.

The load testing was conducted following the documentation of camber measurements. Loading was carried out incrementally, starting with 10-kip increments up to 60% (80 kips) of the anticipated failure load and subsequently proceeding in 5-kip increments until failure. However, after consistent results were obtained in terms of peak failure loads for the initial three girders, the 10-kip loading progression was adjusted to 75% (100 kips) of the anticipated failure load. From that point, the loading continued in 5-kip increments until failure.

At each loading step, crack progression was documented through photographs and by marking the extents of new and expanding cracks on the member. Once the peak failure load was achieved, the girder was photographed again and subjected to additional deflection to reveal further failure damage. Additional photographs were captured before relieving the applied load, marking the conclusion of the test.

4.4. Hybrid Girder Test Results

During the assessment of all girders, it was observed that flexural cracking had occurred in the top bell of each girder as a result of prestress release. These cracks were localized solely within the SCC sections and did not extend into the UHPC or affect the interface between the two concrete types. These cracks initiated at the base of the deck and extended vertically down into the top of the web, with none of them extending more than 4 inches into the web.

Compressive strength values for the concrete mixes used in each hybrid girder were consistent across all four sets. The average reported strength of the UHPC in all hybrid girders was approximately 18 ksi, while the SCC for both the girder and deck averaged just over 8 ksi. A record of the compressive strengths of the concrete mixes for each hybrid girder is provided in Appendix A. Additionally, Appendix A contains information on camber measurements and the corresponding specimen ages for each girder.

4.4.1 *BiFluid Hybrid Girder*

The testing results of the BiFluid hybrid girders revealed that the initial shear cracks appeared near or across the interface at around 60 kips, approximately 45% of the peak load, as depicted in Figure 4.13. Cracking continued uniformly throughout the web section in both the SCC and UHPC, regardless of the interface location. Notably, cracking was localized in regions where the SCC extended into the UHPC during the casting process. This effect is particularly evident in BiFluid girder number 1, which exhibited sharp SCC extensions into the UHPC and cracks appeared to develop through the tip of these extensions (refer to Figure 4.14). In the other two girders, cracks were evenly distributed throughout the depth of the section. This behavior is believed to occur because cracks tend to propagate through the less stiff material, and in the case of BiFluid girder number 1, they were funneled through the SCC projections.

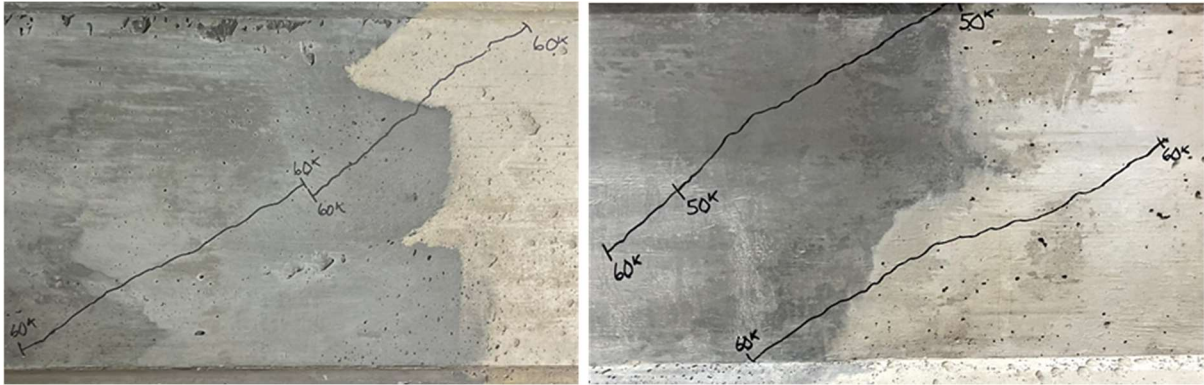


Figure 4.13: First cracks to propagate in BiFluid girder no. 1 (left) and no. 3 (right)

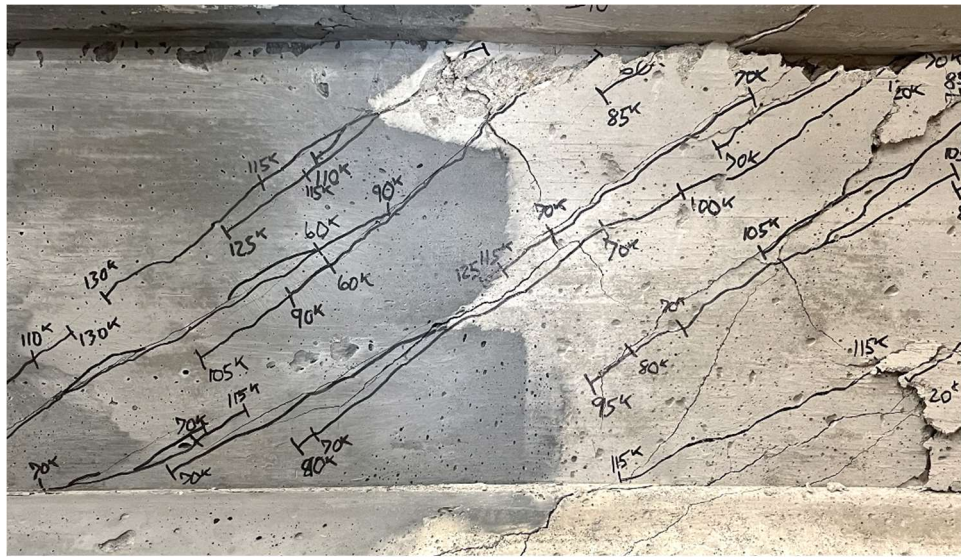


Figure 4.14: Localized cracking through SCC projections at interface in BiFluid girder no. 1

Cracks were predominantly concentrated in the reduced width web section, with few shear cracks appearing at the opposite end of the girder after an applied load of 125 kips. Flexural cracks began propagating under the load point at 120 kips. Ultimately, failure was observed in the SCC near the top of the web and in the deck. This damage was characterized by web splitting and crushing of both the deck and web, as illustrated below in Figure 4.15, Figure 4.16, and Figure 4.17. The failure behavior exhibited a sudden drop in load, corresponding to an immediate 20% reduction from the peak load, followed by continued displacement with decreasing load capacity. Consequently, it can be inferred that the failure mode for all three BiFluid hybrid girders was compression-shear failure.



Figure 4.15: Failure of BiFluid hybrid girder no. 1



Figure 4.16: Failure of BiFluid hybrid girder no. 2



Figure 4.17: Failure of BiFluid hybrid girder no. 3

Collectively, the average peak failure load across the three BiFluid hybrid girders was approximately 133 kips, with a coefficient of variation of 1.81%. The average deflection at these peak loads was recorded as 0.289 inches. A summary of the BiFluid girder peak failure loads and deflections is given in Table 4.1, and the load-deflection response for all three girders is graphically depicted in Figure 4.18.

Table 4.1: Failure loads and corresponding deflections of each BiFluid hybrid girder

Hybrid Girder Index	Failure Load (kips)	Corresponding Deflection (in.)
BiFluid No. 1	134.1	0.284
BiFluid No. 2	135.7	0.254
BiFluid No. 3	130.0	0.328

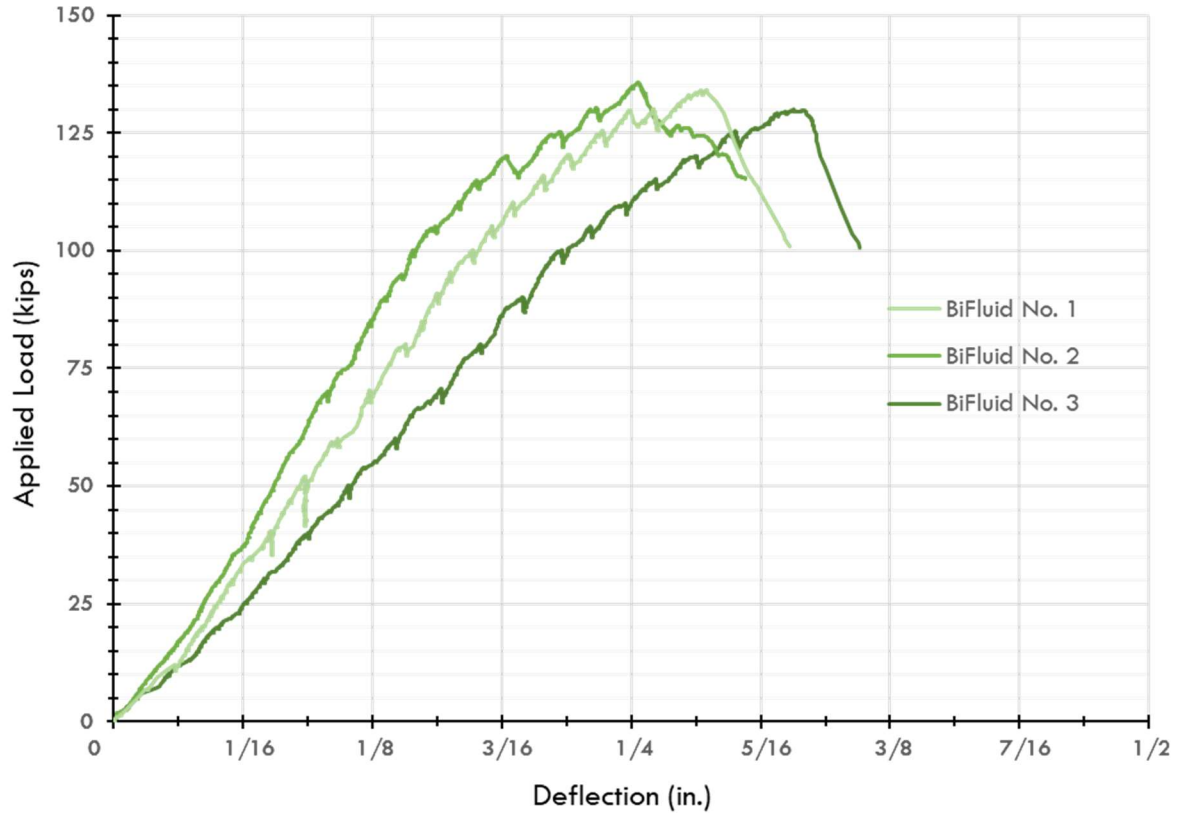


Figure 4.18: Load-deflection plot of the BiFluid hybrid girder set

4.4.2 BiFluid with Divider Hybrid Girder

The load-response and failure details of the BiFluid with Divider hybrid girders closely reflected those of the BiFluid girders. Initial cracking appeared across the interface (Figure 4.19), similar to the BiFluid set, although these first cracks occurred at lower loads than those observed in the BiFluid set (50 kips for the first girder and 40 kips for the second). Crack progression during loading was identical to that of the BiFluid girder set, including evenly dispersed cracking that extended throughout the web section, unphased by the presence of the interface. Flexural cracks emerged in the bottom bell at the load point after 120 kips of applied load. At 130 kips, shear cracks developed in the SCC on the end opposite the interface. Unlike the BiFluid girders, there were no protrusions of the SCC into the UHPC from casting, and no localization of cracking was observed. Although small amounts of UHPC did seep around the divider at the outer surfaces of the web, this did not appear to affect the cracking response.



Figure 4.19: First cracks to propagate in BiFluid with
Divider girder no. 1 (left) and no. 2 (right)

Upon reaching the failure load, the performance closely resembled that of the BiFluid hybrid girders, denoted by a sudden failure and a subsequent drop in load (20-30% decrease in load based on the peak value). Failure was confined to the top of the web on the SCC side, away from the interface. The exhibited damage in the web was a splitting-type failure, as depicted in Figure 4.20 and Figure 4.21, indicating a compressive strut failure between the shear cracks. After reaching the peak load, the girders continued to be loaded, resulting in increased deflection. During this post-failure loading phase, cracks began to open in the web, and the deck exhibited signs of compression failure.

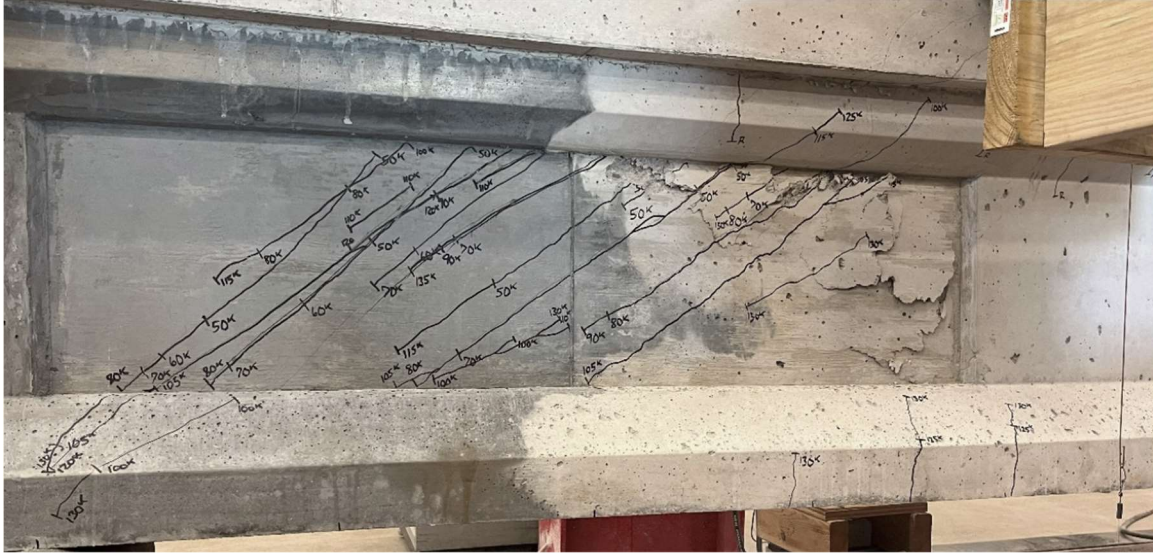


Figure 4.20: Failure of BiFluid with Divider hybrid girder no. 1



Figure 4.21: Failure of BiFluid with Divider hybrid girder no. 2

The average peak load and corresponding deflection for these compression-shear failures was measured at 137 kips and 0.264 inches, respectively. Individual values for each hybrid girder are given in Table 4.2, and the load-deflection response for the BiFluid with Divider set is illustrated in Figure 4.22.

Table 4.2: Failure loads and corresponding deflections of each
BiFluid with Divider hybrid girder

Hybrid Girder Index	Failure Load (kips)	Corresponding Deflection (in.)
BiFluid with Divider No. 1	139.29	0.304
BiFluid with Divider No. 2	135.19	0.225

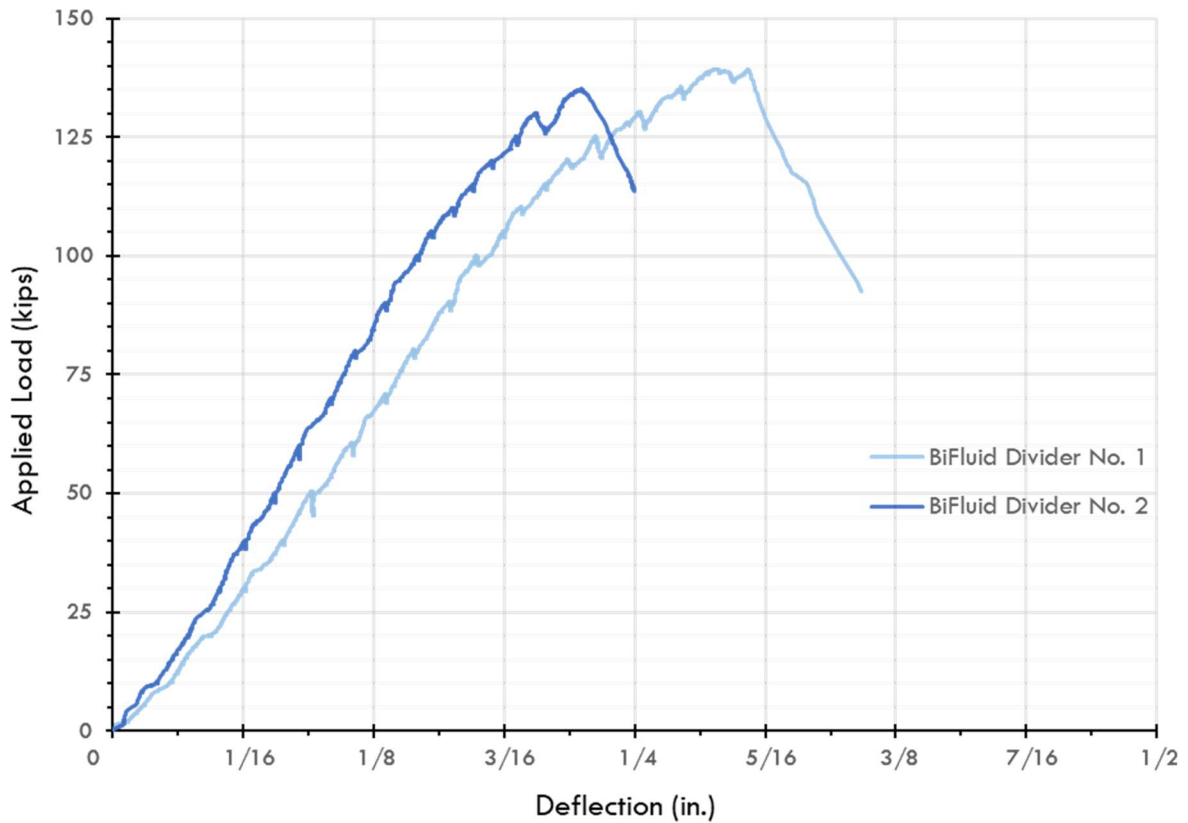


Figure 4.22: Load-deflection plot of the BiFluid with Divider hybrid girder set

4.4.3 2-Dimensional Trapezoid Interface Hybrid Girder

The testing results of the 2D Trapezoidal hybrid girders demonstrated slightly different responses when compared to the previous two BiFluid girder sets. Initial cracking was notably influenced by the presence and texture of the interface. Both hybrid girders in this set developed their first cracks on the UHPC side of the interface at an applied load of 40 kips. After increased loading, girder number 1 displayed further propagation of this initial crack on

both sides of the interface, eventually bridging across the interface at 60 kips (refer to the photo on the right in Figure 4.23). The left image in Figure 4.23 shows the crack extents at 50 kips, with the crack in the UHPC being arrested by the interface, and the crack in the SCC stopped near the location of the first stirrup. The first crack in girder number 2 followed and crossed the interface into the SCC, as illustrated in Figure 4.24. As the load continued to increase, further expansion of cracking in both girders appeared to culminate at or near the location of the initial crack. These cracks extended from the web into the bottom bell of the UHPC and into the top bell of the SCC, reaching up to the base of the deck.



Figure 4.23: First cracks to propagate in 2D Trapezoidal hybrid girder no. 1 (left); further crack progression at the interface (right), photo taken of the region outlined in red

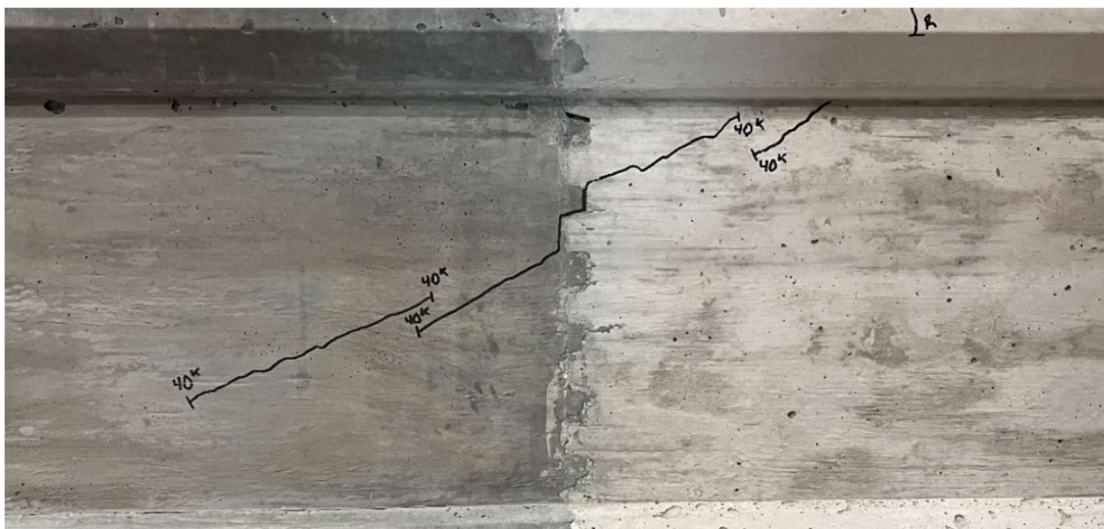


Figure 4.24: First cracks to propagate in 2D Trapezoidal hybrid girder no. 2

As each girder loading neared failure, they displayed increased deflection with a reduced increase in load. This behavior continued up to the peak load, at which point failure occurred, followed by an increase in deflection and a strong decrease in load. The resulting failure was perceived to be less sudden than in the BiFluid sets. However, as with the previously tested girders, flexural cracking initially appeared under the load in the bottom bell at 120 kips. No shear cracks were observed in the SCC section opposite the shear span, possibly due to the lower overall applied loads.

The average peak failure load for the 2D Trapezoidal hybrid girders was 120 kips, with an average deflection of 0.246 inches. Individual peak values are given in Table 4.3, and the load-deflection responses are illustrated in Figure 4.25. Ultimately, the failure mechanisms were analogous to those observed in the BiFluid with Divider set, characterized by compression-shear failure at the top of the web, away from the interface, as shown in Figure 4.26 and Figure 4.27.

Table 4.3: *Failure loads and corresponding deflections of each 2D Trapezoidal hybrid girder*

Hybrid Girder Index	Failure Load (kips)	Corresponding Deflection (in.)
2D Trapezoidal No. 1	121.31	0.241
2D Trapezoidal No. 2	118.95	0.251

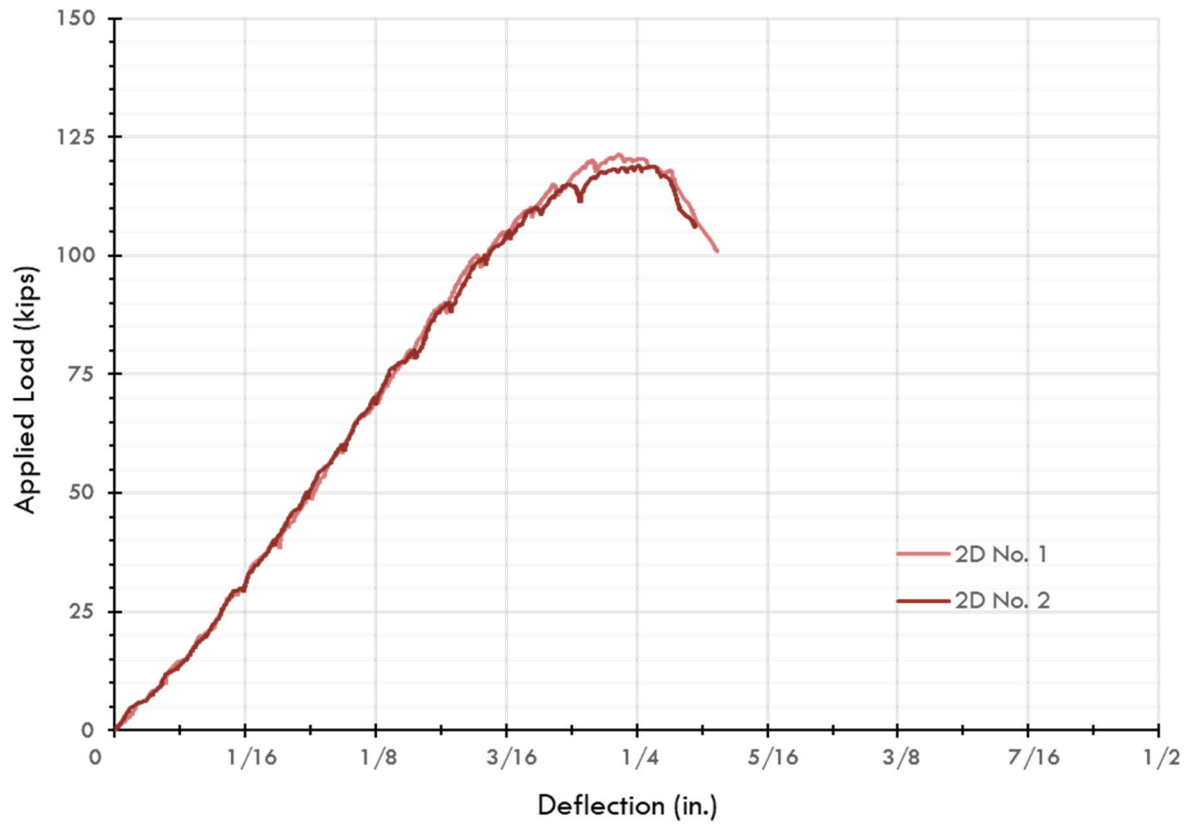


Figure 4.25: Load-deflection plot of the 2D Trapezoidal hybrid girder set



Figure 4.26: Failure of 2D textured hybrid girder no. 1



Figure 4.27: Failure of 2D textured hybrid girder no. 2

4.4.4 3-Dimensional Trapezoid Interface Hybrid Girder

The load-deflection response and cracking behavior of the 3D Trapezoidal hybrid girders closely paralleled that of the 2D Trapezoidal girders. Differences between the two sets primarily related to the degree of influence each texture held on the route of the crack across the interface and the behavior near the failure load.

Similar to the 2D girder set, the initial cracks in the 3D girders developed across the interface in the web section, occurring at applied loads of 40 and 50 kips. It became apparent that the formation of these cracks appeared at the location between the protrusions of the 3D texture, as illustrated in Figure 4.28. While the texture may have guided the path of these cracks to some extent, it did not follow along the interface to the same degree as observed with the 2D texture. This observation suggests that variations in the texture in both directions of the interfacial plane can lead to more homogeneous behavior.

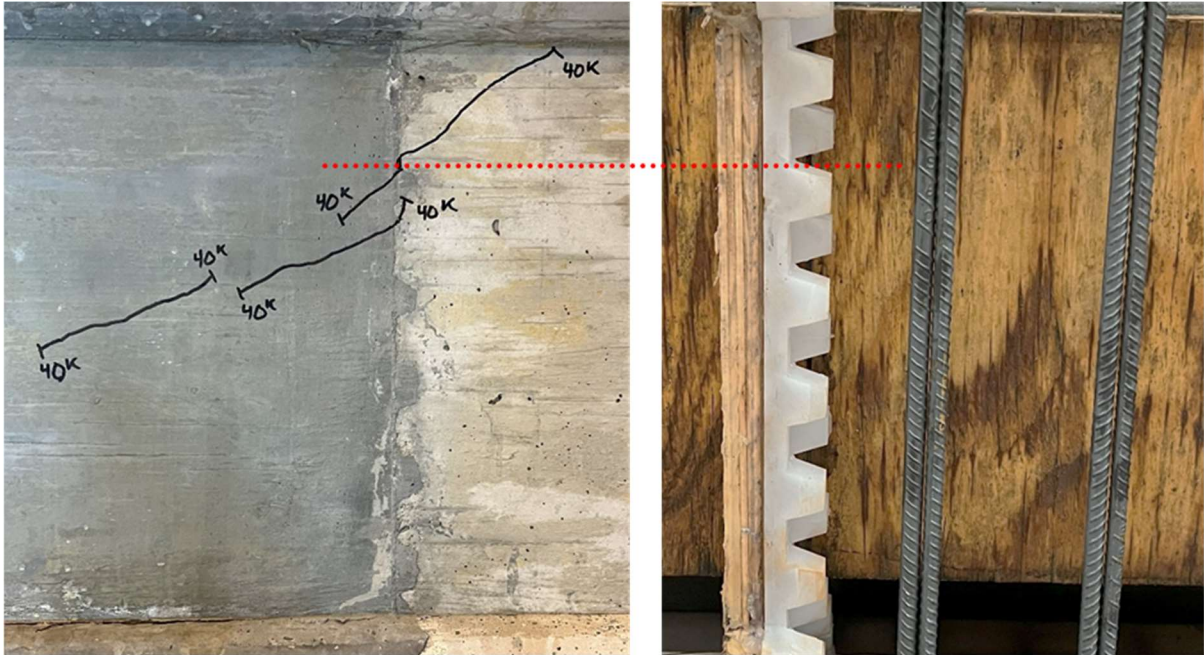


Figure 4.28: First crack for 3D Trapezoidal girder no. 1 (left) and the alignment of the gap between UHPC protrusions

The progression of cracks beyond the initial formation resembled that of the 2D Trapezoidal hybrid girders, including the expansion of the original crack and the development of new cracks along the same line. These cracks initiated in the web and extended into the top bell and bottom bell of the SCC and UHPC, respectively. Notably, the first girder of the 3D set displayed a more concentrated line of cracking, while girder number 2 exhibited more evenly distributed cracking across the web, with the width of the first crack noticeably spreading with increased loading.

The behavior of these girders near failure aligned more with the behavior of the BiFluid sets, in contrast to the 2D Trapezoidal set. As the load approached failure, the cracking behavior mirrored that of the 2D Trapezoidal set, but the load-deflection response was more like the BiFluid girders. The latter was evident immediately before failure, when the deflection increased linearly with load, followed by a sudden, violent failure accompanied by an 18% drop in load (based on the peak load) with increased deflection.

The exhibited failure mechanism remained consistent with all previous hybrid girders, representing a compression-shear failure in the SCC at the top of the web. However, the 3D hybrid girder failure damage extended back to the interface along the top of the web section.

This damage was stopped by the UHPC at the interface (see Figure 4.29) in the first girder. After the initial failure of girder number 2, during the induced additional deflection, the failure plane extended violently from the SCC, along the top of the web, back into the UHPC through the end of the girder at the mid-height of the web (refer to Figure 4.30).



Figure 4.29: Failure of 3D textured hybrid girder no. 1



Figure 4.30: Failure of 3D textured hybrid girder no. 2

The average peak failure load for the 3D Trapezoidal hybrid girders was approximately 131 kips, with a corresponding average deflection of 0.260 inches. Individual peak values are given in Table 4.4, and the load-deflection response is shown in Figure 4.31.

Table 4.4: Failure loads and corresponding deflections of each 3D Trapezoidal hybrid girder

Hybrid Girder Index	Failure Load (kips)	Corresponding Deflection (in.)
3D Trapezoidal No. 1	127.83	0.262
3D Trapezoidal No. 2	133.65	0.259

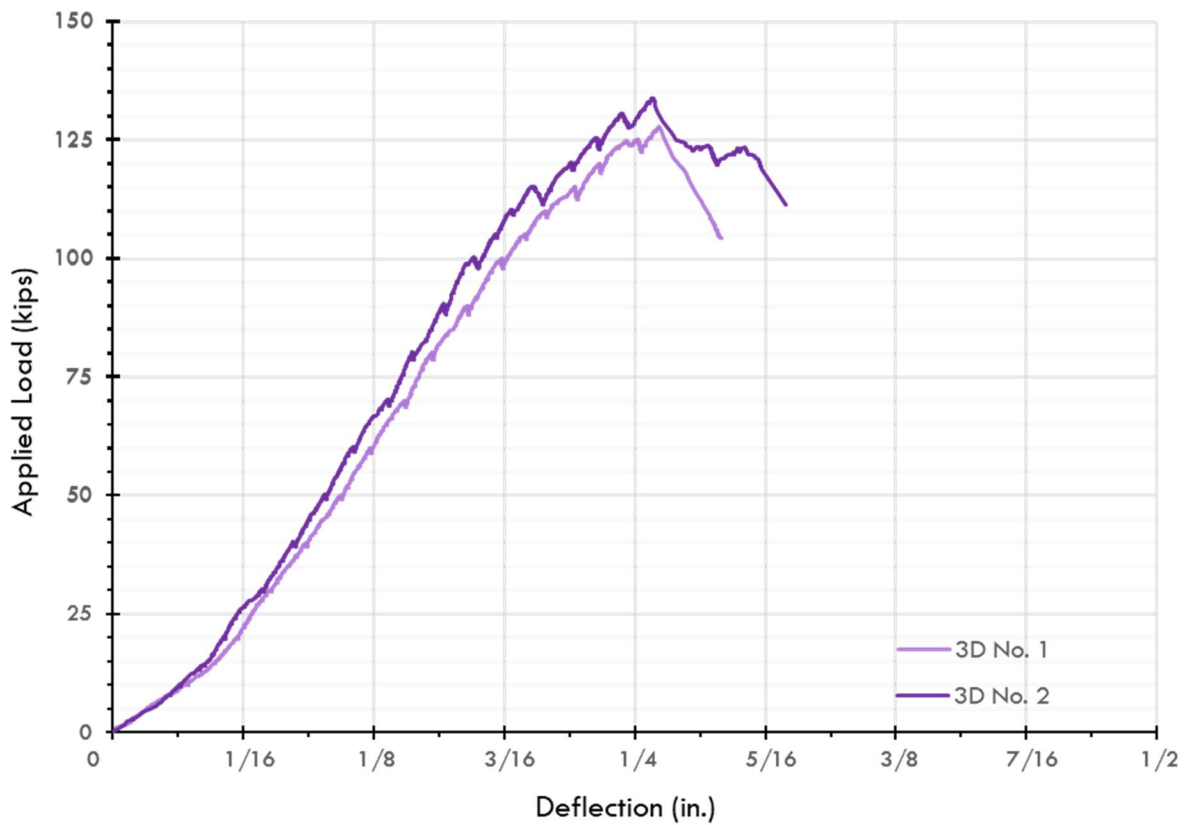


Figure 4.31: Load-deflection plot of the 3D Trapezoidal hybrid girder set

4.5. Discussion of Large-Scale Program Results

This program demonstrated the feasibility of constructing hybrid girders with various interfaces. Cold-joint interfaces, although simpler to construct, require additional time for sequential casting of the two concretes. Casting BiFluid members presents a greater level of complexity, but the introduction of dividers can assist the process. In the case of the BiFluid with Divider method, dividers can provide greater aid for matching fill rates for each concrete, especially when the dividers provide complete, face-to-face separation between concretes. While the scope of this study did not allow for a comprehensive exploration of various divider designs for casting BiFluid specimens, it is worth noting that dividers could be fabricated to obstruct more of the section, further limiting concrete crossflow at the interface. One such method could be realized using a divider extracted from the side of the formwork, placed between form sections.

Overall, the hybrid girders yielded relatively consistent results for maximum load, with an average failure load of approximately 130.67 kips and a coefficient of variation of 4.93%. Most of the failure damage was confined to the reduced web section of the SCC, and no failures originated along the interfaces. Crack development appeared to be more evenly distributed throughout the webs of the BiFluid and BiFluid with Divider sets. Conversely, cracking in the 2D and 3D Trapezoidal hybrid girders appeared to be localized near the location of the first cracks within the section. This may be attributed to the natural discontinuity of cold-joint interfaces, allowing for imperceivable amounts of separation along the interface. In contrast, BiFluid interfaces are bonded both physically and chemically, making cracking less susceptible to being guided by physical discontinuities in either concrete.

The behavior of hybrid girders near failure is influenced by the type of interface. BiFluid methods produced sudden, dramatic failures at peak loads, whereas the 2D texture exhibited a more gradual failure with lower loads at failure. Conversely, the 3D texture behaved more comparably to the BiFluid sets, characterized by higher capacities with a sharp drop in load after a sudden, intense failure. This suggests that varying the cold-joint texture in multiple directions along the interfacial plane can enhance overall performance. Additionally, upon analyzing the cracking behavior of the 3D texture, it may reflect the behavior of a more homogeneous section with added texture modifications. This could potentially be achieved by altering the texture to ensure that the rows of UHPC protrusions overlap more, eliminating any

horizontal gaps between UHPC protrusions along the interfacial plane (refer to Figure 4.28 in Section 4.4.4).

From the load-deflection plots, combined in Figure 4.32, it can be inferred that the hybrid girder stiffness and toughness were relatively consistent across all girders. Considering this and the consistent failure loads, which closely approached the design failure load, it appears that the shear performance of hybrid girders is more predictable than traditional shear data for concrete members. This predictability is believed to be a result of the specific mode of failure exhibited by these girders, as they enable compression-shear failures that have been shown in this program to be both predictable and consistent.

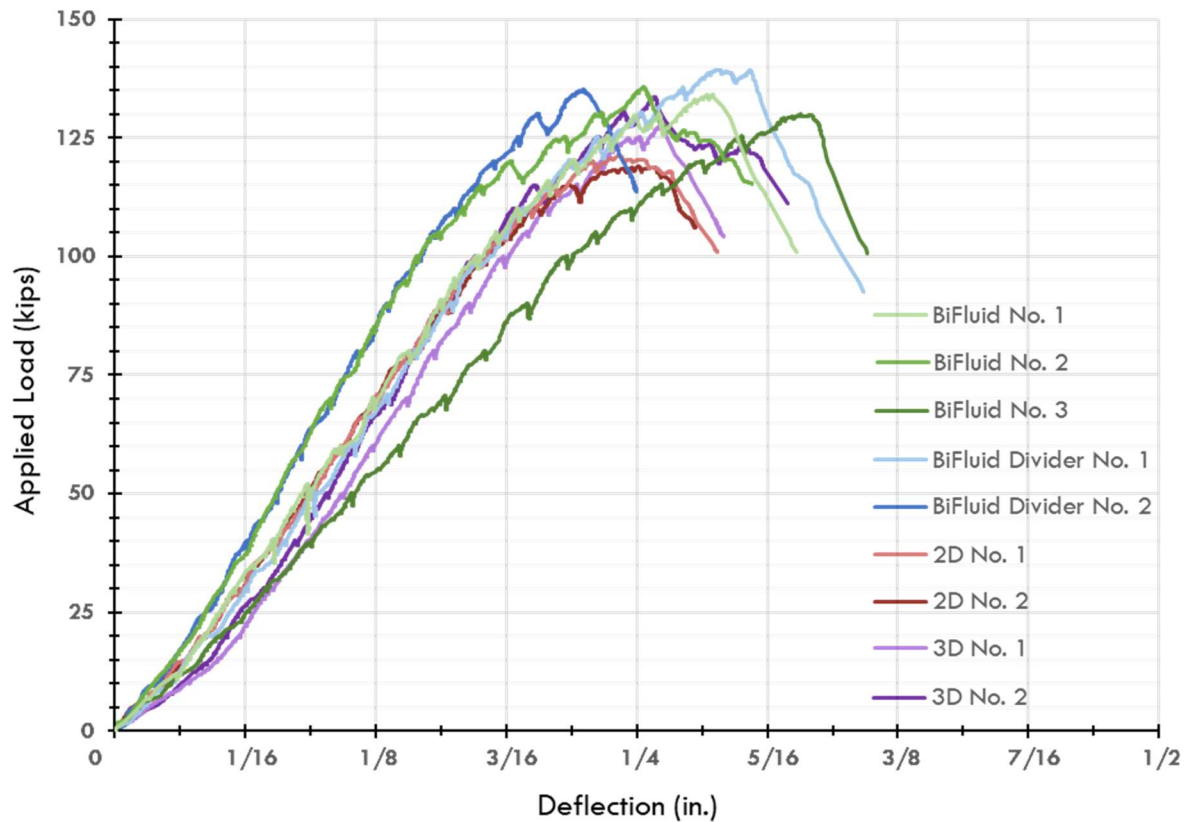


Figure 4.32: Combined load-deflection plot for all hybrid girders

Chapter 5: Analysis of Small-Scale Program

5.1. Introduction

The small-scale program of this research is a comprehensive interfacial characterization study, with an emphasis on evaluating the bond between Ultra-High-Performance Concrete (UHPC) and conventional concrete, specifically for implementation in hybrid girders. This investigation encompassed seven distinct interface types, each intentionally selected based on insights obtained from existing literature concerning UHPC-to-conventional concrete interfaces. These interfaces include two BiFluid methods, one with a divider and one without, that involved simultaneous casting of UHPC and Self-Consolidating Concrete (SCC). Additionally, two interfaces featuring trapezoidal-shaped grooved textures were evaluated, one in a 2-dimensional (2D) fluted pattern and the other in a 3-dimensional (3D) checkerboard pattern. The remaining three interfaces were chosen to represent commonly used textures in interface characterization studies: an exposed coarse aggregate texture, a 2-dimensional quarter-inch sine wave texture, and a smooth, untextured interface.

For a comprehensive understanding of each interface type, a detailed description is provided in Appendix A. The selected characterization methods for this program were guided by existing literature and focused primarily on assessing the shear-carrying capacity of each interface. As such, the shear capacities were measured through direct shear testing, also known as push-out shear testing, and slant shear testing. Flexural capacity was also included in the assessment by modulus of rupture (MOR) testing. Detailed information regarding these tests and their outcomes is provided in Chapter 3.

An in-depth discussion and analysis of the results obtained from the small-scale program is given in this chapter. Through this analysis, a comparison of the strengths of each interface type is contrasted with values obtained in existing literature. A discussion of different existing approaches to interfacial design and capacity calculations is provided. Ultimately, a design recommendation is offered for addressing shear capacity of different interfaces.

5.2. Interfacial Strength Evaluation

Under closer evaluation of the results from the small-scale program presented in Chapter 3 with regard to the existing literature in Chapter 2, many of the previously mentioned theories and testing results were confirmed in the results of this program. Direct shear testing results of this study reinforced the theory that with increased texture amplitudes, there is a corresponding increase in bond behavior (Jang et al., 2017; Valikhani et al., 2019; Yang, et al., 2022). This theory is due to the ability of coarse aggregates in the conventional concrete to fit between the UHPC protrusions of the texture and support the load further into the conventional concrete. In the direct shear testing, the 2D Trapezoidal results showed a 25% increase in shear strength over the Sine Wave results, supporting this theory. However, the Exposed Aggregate interface gave lower overall results compared to the Sine Wave texture, which is inverse to the results detailed by Jang et al. (2017). When analyzing the most similar textures of that study in comparison to this program, Jang et al. (2017) reported direct shear strengths of an exposed aggregate texture (through water-jet treatment) with comparable strength to that of the Exposed Aggregate texture used herein. The 10-mm and 20-mm box-grooved textures from that study are more comparable to the Sine Wave and 2D Trapezoidal textures in this program, but the reported strengths are not of equal magnitudes. The Sine Wave and 2D Trapezoidal textures possessed much higher strengths. This is indicative that other texture characteristics, such as shape, can provide increased bonding strength and can account for the discrepancy between Jang et al.'s (2017) reported strengths. Table 5.1 lists the average direct shear strengths of comparable cold-joint interface textures given by Jang et al. (2017), Yang et al. (2022), and this study for contrast.

Table 5.1: Direct Shear Strengths of cold-joint textures from the small-scale program, Jang et al. (2017), and Yang et al. (2022)⁵

Interface Type (Source)	Direct Shear Strength	Texture Description
Water Jet Treatment (Jang et al., 2017)	873 psi	Exposed aggregate.
Exposed Aggregate (Small-Scale Program)	759 psi	Exposed aggregate (see Appendix).
Sine Wave (Small-Scale Program)	929 psi	0.25-inch sine wave, fluted.
Box-Grooved 10mm (Jang et al., 2017)	548 psi	10x10mm square, fluted.
10x10mm Box, 75mm Spacing (Yang et al., 2022)	302 psi	10x10mm square, fluted with 85mm texture period.
20x10mm Box, 75mm Spacing (Yang et al., 2022)	450 psi	20mm width & 10mm tall rectangular, fluted with 85mm texture period.
10x10mm Trapezoid, 100mm Spacing (Yang et al., 2022)	577 psi	10mm tall trapezoid, 10mm top width, 14mm base width, fluted with 110mm texture period.
20x10mm Trapezoid, 100mm Spacing (Yang et al., 2022)	541 psi	10mm tall trapezoid, 20mm top width, 24mm base width, fluted with 110mm texture period.
2D Trapezoidal (Small-Scale Program)	1154 psi	0.75-inch trapezoid, fluted (see Appendix).
20mm Groove (G1) (Jiang et al., 2021)	544 psi	20x20mm square, fluted.
Box-Grooved 20mm (Jang et al., 2017)	599 psi	20x20mm square, fluted.

In the small-scale program, one of the key shape characteristics examined is the profile of texture protrusions. The notable strength discrepancy observed between the 2D Trapezoidal texture and the 20mm Box-Grooved texture from Jang et al. (2017) could potentially be attributed to differences in texture shape. Yang et al. (2022) reported a potential 25% increase in shear capacity by altering the shape to redirect the path of the normal force exerted by the UHPC texture protrusions. To further explore this, the average stress for the 20mm Box-Grooved texture was normalized for differences in compressive strengths and Yang et al.'s

⁵ Interfaces in Table 5.1 are ranked in texture amplitude from lowest to highest and shape.

multiplier for increased capacity was applied. This yielded a shear capacity much closer to that of the 2D Trapezoidal texture. A similar calculation can be applied to compare the strength of the 10-mm Box-Grooved texture from Jang et al. with the Sine Wave texture from the small-scale program outlined in Chapter 3. Yang et al.'s report only considered texture amplitudes of 10 mm and angles of 11 degrees for the trapezoidal shape, either in an obtuse or acute orientation. If larger amplitudes and increased angles are incorporated, it may be possible to achieve capacity gains exceeding the reported 25% when switching from box shapes to trapezoidal shaped texture patterns, emphasizing the significance of both amplitude and shape in texture patterns. This reaffirms Yang et al.'s deduction on the substantial influence of texture shape on shear capacity.

Another validated method to enhance bond behavior involves incorporating texture variation in multiple directions along the interfacial plane (Jang et al., 2017). The small-scale direct shear testing results indicated a 20% increase in shear capacity of the 3D Trapezoidal texture compared to the 2D Trapezoidal texture. Variability of the texture in both directions provided the 3D Trapezoidal texture with an increased number of UHPC protrusions crossing the interface location, which resulted in the increased capacity. For slant shear specimens, the difference in area of UHPC crossing the median interfacial plane between the 2D and 3D Trapezoidal textures was approximately 5%. Interestingly, the average strength of the 3D Trapezoidal texture was about 5% lower than that of the 2D Trapezoidal texture. It is inferred that this difference may be due to the reduced area of the SCC at the interface, resulting in limited stress-carrying capacity and the creation of a shear plane in the SCC along the face of the UHPC protrusions.

These findings are consistent with observations from Jang et al.'s study, where a “smooth cut-off” at the face of UHPC grooves on the conventional concrete side was observed. Jang et al. (2017) also reported an increase in capacity with groove spacing large enough to accommodate coarse aggregate, aligning the strength with that of the aggregates. This emphasizes the importance of having an adequate amount of the weaker substrate to facilitate shear transfer.

5.3. Existing Interface Design Approaches

Current design provisions for determining the shear capacity of interfaces primarily rely on shear-friction modeling and modified versions of this theory (such as horizontal shear) (Nawy, 2010). Both AASHTO LRFD (2017) and ACI 318 (2019) offer similar approaches for the shear-friction model. This model calculates shear capacity based on the frictional resistance provided by the clamping force exerted on the interface by the tension component of the reinforcement. Intrinsically, this model requires the presence of reinforcement at the interface for capacity calculation. AASHTO LRFD goes a step further by incorporating a cohesion component in the equation, which is independent of the clamping force and provides a minimum stress value. In contrast, the ACI method does not include the cohesion element due to potential negative effects stemming from cracking. Consequently, the ACI model tends to yield more conservative estimates of capacity for non-reinforced interfaces when compared to the AASHTO methodology.

Researchers studying the interfacial effects of UHPC and conventional concrete have proposed alternative methods for calculating interfacial strength. Jiang et al. (2021) introduced an amended version of a model originally presented by the Association Francaise de Genie Civil (AFGC), as detailed in Equation 5-1. The modified model aimed to predict shear stress in the context of grooved or fluted, two-dimensional textures. The calculation of shear stress (v) in this model involves several parameters, including the groove shape factor (k), density correction factor (λ), elastic limit under tension ($f_{ctk,el}$), clamping stress (ρf_y), AFGC interface roughness factors (c and μ), maximum post-cracking stress (f_{ctfk}), duration effect coefficient (α_{cc}), and compressive strength (f_{ck}). The groove shape factor is calculated as the square root of the cross-sectional area of the groove divided by 20.

$$v = k\lambda(c f_{ctk,el} + \rho f_y + (0.35\mu + 0.3)f_{ctfk}) \leq 1.14\alpha_{cc}f_{ck}^{2/3} \quad \text{Equation 5-1}$$

However, it's important to note that due to a lack of available information regarding this calculation or any limitations associated with it, the shear strengths were unable to be attained using this method.

A simpler alternative to calculating shear stress at the interface was proposed by Yang et al. (2022), which centers on the contribution of individual components present at the interface. This method involves determining a weighted average of the individual shear strengths provided by each material based on the respective area of each component at the interface. However, this approach is primarily used for analysis purposes rather than for design considerations. This is because it takes into account the dimensions of the areas that have experienced failure (Yang, et al., 2022).

5.4. Proposed Interfacial Shear Capacity Model

To quantitatively assess the shear data gathered in the small-scale program, it is worth considering the development of a new equation to estimate the shear capacity of interfaces between UHPC and conventional concrete. Observations from the small-scale program, specifically the contrast between the 2D and 3D Trapezoidal textures, suggests a correlation between shear strength and the area of the weaker concrete at the interface. Integrating this empirical relationship into an existing interfacial shear capacity model has the potential to provide more accurate predictions for capacity. However, it's important to note that a recurring observation in this study was that the better-performing textures often failed just beyond the interface, primarily within the weaker, conventional concrete. This observation implies that in cases with properly textured interfaces, or interfaces featuring a BiFluid-type design, the limiting factor shifts to be the weaker concrete just beyond the interface. As a result, the interface itself becomes a zone of gradual transition of capacity from one concrete type to the other. Consequently, it is critical that the interfacial capacity need not exceed the monolithic capacity of the weaker concrete. Further evaluation of these effects is necessary to develop an interfacial shear capacity equation with accurate estimations for strength between UHPC and conventional concrete. It is important to note that it was never the intention of this study to derive an interfacial shear capacity equation. Rather, the focus was developing textures that would prevent shear failure at the interface and result in girder behavior that resembled monolithic concrete.

Chapter 6: Analysis of Large-Scale Program

6.1. Introduction

The primary objective of the large-scale program was to take the data derived from the small-scale program and apply the findings to the analysis of hybrid girders. Hybrid girders are designed with the intention of addressing several inherent issues associated with traditional prestressed girder designs. Primarily, these issues revolve around the development of cracks resulting from high prestressing forces in the ends of the members and durability concerns in the end regions during in-situ applications. Ultra-High-Performance Concrete (UHPC) presents itself as an ideal material to address these concerns, however, this solution has a substantial cost implication. As an alternative, hybrid girders offer a more cost-effective solution by incorporating UHPC exclusively in the end regions while retaining conventional concrete for the remaining portion of the section.

In the subsequent chapter, a discussion is included of the findings from the large-scale hybrid girder testing program, as outlined in Chapter 4. This analysis provides an intricate examination of the damage progression observed during testing, with an emphasis on the shear behavior at the interface and localized behavior response within the shear span. The examination extends beyond failure, offering a comprehensive understanding of the failure modes presented. Furthermore, this chapter presents an exhaustive exploration of existing methods for estimating the ultimate capacity of hybrid girders and offers a recommended approach for hybrid girder design.

6.2. Hybrid Girder Failure Progression

A detailed discussion of the progression and behavior of each hybrid girder during testing is detailed in the following subsections. Locations and applied load values are given for each notably observed behavior up to and beyond the point of primary failure. A full account of the loading procedure is described in Chapter 4.

6.2.1 *BiFluid Hybrid Girder No. 1*

Before loading the girder, an inspection was conducted to identify existing cracks that had developed due to the release of prestressing. These cracks were primarily located along the Self-Consolidating Concrete (SCC) section of the girder, concentrated in the top bell. While some cracks extended into the web, no prestressing-related cracks were found in the reduced width web section or UHPC.

The first signs of cracking during loading occurred when the applied load reached 60 kips. These initial cracks originated from the top of the web in the SCC, approximately a few inches away from the interface between the SCC and UHPC. They extended down to the base of the web in the UHPC, closer to the support. These cracks exhibited an orientation trending at approximately a 30-degree angle from horizontal, crossing the interface about 3 inches below the top bell.

As loading continued, additional cracks formed along the same angle, spanning across the web near the interface and within the SCC section. The second major crack crossing the interface occurred at 70 kips. This crack propagated through a protruding section of SCC into the UHPC, just below mid-height of the web along the interface. It extended from the base of the web in the UHPC, traversing at a 30-degree incline to the top of the web in the SCC. More cracks emerged at the top of the web in the SCC section, closer to the load point, and followed the same orientation. Initially, these cracks appeared in the top quarter of the web and continued to extend as the load increased, mirroring the trajectory of the previous cracks.

Between 90 and 110 kips of loading, existing cracks continued to extend into the bottom bell of the UHPC, moving towards the support. During this phase, existing cracks continued to extend with fewer new cracks being formed (as shown in Figure 6.1). Another set of cracks, parallel to the initial ones, formed at 110 kips. These cracks appeared approximately 3 inches above the original ones, crossing the interface at another protruding section of SCC near the top of the web. They extended through several different cracks down to the support and started to merge as the load increased.



Figure 6.1: BiFluid Girder No. 1 cracking in the shear span up to failure

Web-shear cracking became visible at 120 kips in the top of the web within the homogenous SCC section, on the side opposite to the interface. Flexural cracks began to propagate in the bottom bell at 125 kips. Simultaneously, the web-shear cracks in the SCC near the interface extended into the top bell, reaching up to approximately one inch below the deck. Crack extension continued past the last documented load increment of 130 kips.

At failure, there was a sudden opening of the top of the web, resembling a splitting action, with the surfaces of the web section pushed outward. The deck and top bell exhibited fracture consistent with compression-type failure, along with detachment of a large section of the SCC below the strands directly under the web damage. Additional cracking also occurred across the interface in the bottom bell. At this location, another portion of SCC crossed the interface, and the crack followed the outline of the UHPC until the transition between concretes curved back, at which point the crack continued into the UHPC beyond the interface.

Following the initial failure, additional deflection was induced to highlight the exhibited failures. The resulting damage showcased a full-depth fracture in which the web damage expanded down to connect with the fracture section below the strands (see Figure 6.2). Loose concrete was removed from the girder, revealing a "V"-shaped vertical fracture in the web, coming to a point between the stirrups (visible in Figure 6.3), and detachment of the concrete in the bottom bell around the strands. Furthermore, cracks in the top bell were notably large, with an approximate 1/8-inch gap, and the deck exhibited behavior analogous to a double hinge.



Figure 6.2: Post-failure damage observed in BiFluid Girder No. 1



Figure 6.3: *BiFluid Girder No. 1 failure after additional displacement featuring a vertical, “V-shaped” failure along the vertical reinforcing in the web.*

6.2.2 *BiFluid Hybrid Girder No. 2*

Before subjecting the girder to loading, cracks that had developed due to prestress release were marked. These cracks were primarily distributed along the length of the SCC section in the top bell of the girder. Most of these initial cracks were confined to the top bell, and only a few extended into the web. None of these prestress-induced cracks were found in the reduced width web section or in the UHPC.

The first cracks observed during load testing appeared at an applied load of 70 kips. These cracks appeared to be evenly distributed throughout the web, extending from just under the load point to approximately 12 inches into the UHPC. They were spaced at intervals of about 3 inches and exhibited an approximate 35-degree angle along the top of the web.

The cracks that were closer to the load point extended only a quarter of the web's depth, while those closer to the interface propagated further down the web until reaching the UHPC. Cracks forming in the UHPC extended the full height of the web in the same 35-degree orientation. As loading continued, the initial cracks elongated, and a few new cracks formed throughout the web following the same angle. Beyond 100 kips of loading, the spacing between cracks decreased due to the formation of new diagonal cracks, which formed parallel to the existing ones. Further propagation continued primarily as extensions of the existing cracks with few newly formed cracks. Web-shear cracks on the homogeneous SCC side began to propagate at the top of the web, beginning at 120 kips. At 130 kips, additional cracking formed that was localized near the top of the reduced width web, particularly around the interface.

As the girder approached failure, cracks began to bridge along the top portion of the thinner web in the SCC as shown in Figure 6.4. At the point of failure, a sudden vertical splitting occurred, approximately halfway between the interface and the load point within the SCC portion of the web. This fracture followed an existing crack back and down the web to another vertical split that connected it to the bridged cracks at the top of the web. Figure 6.5 illustrates this failure path.

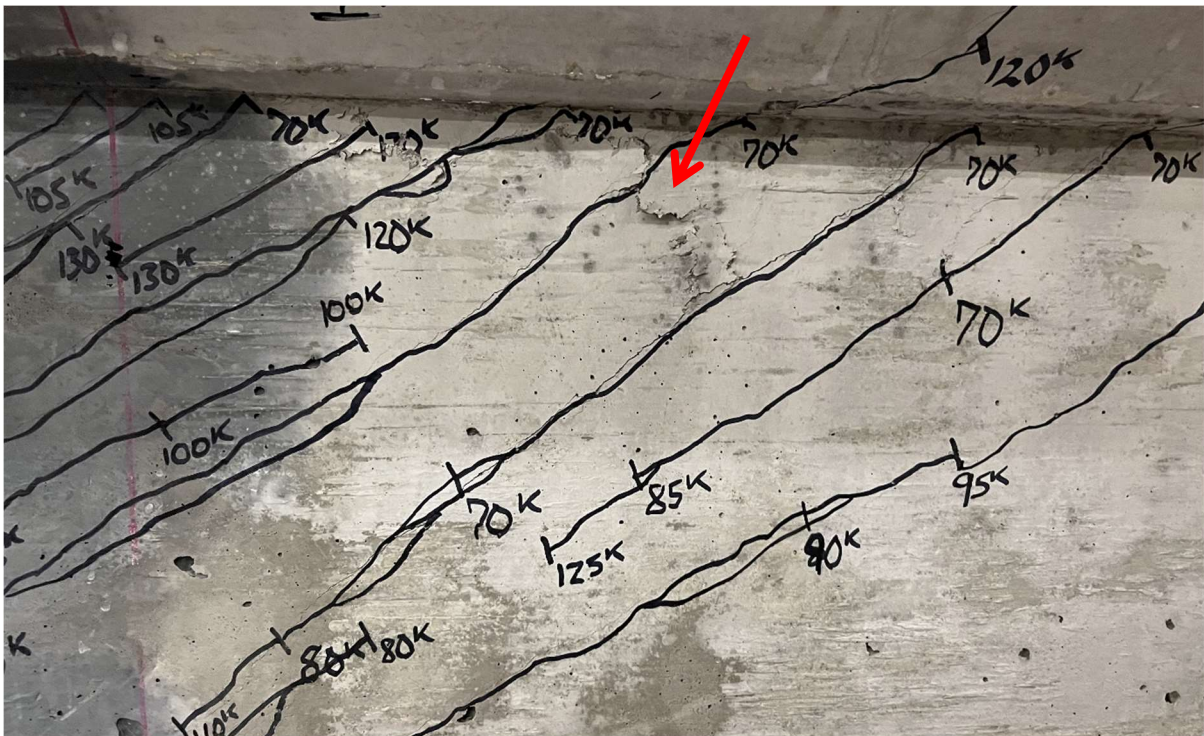


Figure 6.4: Initial bulging failure in the web of BiFluid Girder No. 2



Figure 6.5: Highlighted failure path of web failure in BiFluid Girder No. 2

After the initial failure, the girder was subjected to additional deflection to highlight the failure modes present. The resulting damage showcased a full-depth fracture, with the web damage expanding downward to connect with the fractured section below the strands. Detached pieces of the split web fell away, revealing a wedge-shaped vertical fracture plane in the web, converging at a point between the stirrups, as visible in Figure 6.6. The deck also exhibited fracture directly above the failed SCC and was consistent with a compression-type failure. Sections of the SCC that were under the strands, near the interface, had split away from the strands.



Figure 6.6: *Observed, post-testing failure in BiFluid Girder No. 2 on the unmarked face (backside of girder)*

6.2.3 BiFluid Hybrid Girder No. 3

Before subjecting the girder to loading, the existing cracks resulting from prestress release were marked. These cracks were distributed along the length of the girder, primarily concentrated in the top bell. Notably, two vertical cracks were also identified in the top bell of the UHPC. The majority of these initial cracks were confined to the top bell, with only a few extending into the upper part of the web. None of the prestress-induced cracks were found in the reduced web section. Additionally, some longitudinal cracking was observed at the SCC end of the girder, along the bottom row of strands (likely due to splitting stresses within the transfer region), and another at approximately one-third of the web's height, as illustrated in Figure 6.7.



Figure 6.7: *Observed cracking due to prestressing forces in BiFluid Girder No. 3*

The first cracks observed during load testing appeared at applied loads of 50 and 60 kips. These cracks formed near the interface, with one crack traversing through both concretes. The first crack initiated at the interface at the top of the web and extended halfway down the UHPC portion of the web at an angle of approximately 30 degrees. The second crack developed across the interface at 60 kips, located in the bottom half of the web. It extended from the base of the web a few inches inside the UHPC, continuing up just beyond half the web's height within the SCC. These cracks were parallel and spaced approximately 6 inches apart.

During the loading up to 90 kips, the first crack continued to elongate along the same orientation, reaching down into the bottom bell above the support. A few steeper cracks also formed in the UHPC within the web above the original crack, extending through most of the web at 70 kips. At 80 kips, another crack formed at the same angle, 2 inches below the first crack in the UHPC near the interface. During this phase multiple cracks propagated in the top of the SCC web, paralleling the original cracks.

As the load increased up to 110 kips, the primary response was characterized by the extension of existing cracks, with few new formations. Beyond 100 kips, new cracks that did appear tended to form approximately halfway between two existing cracks, especially in the

Subsequently, the girder was subjected to additional deflection to highlight the failure modes present. Small amounts of concrete along the web fracture fell away, revealing a vertical fracture in the web that came to a point along a stirrup, visible in Figure 6.9. Concurrently, the deck exhibited fractures that extended down into the top bell, consistent with a compression-type failure. Fracturing of the SCC along the prestressing strands created large bulging of the concrete in the bottom bell, which extended up to the damage in the web. For this test no web cracking was observed on the homogeneous SCC side of the girder.



Figure 6.9: Observed, post-testing failure in BiFluid Girder No. 3

6.2.4 BiFluid with Divider Hybrid Girder No. 1

Before loading the girder, existing cracks resulting from prestress release were marked. These cracks were predominantly distributed along the length of the SCC portion in the top bell of the girder. Most of these initial cracks were limited to the top bell, with only a few extending into the web. None of the prestress-induced cracks were observed in the UHPC or the reduced width web section.

The first cracks observed during load testing occurred at an applied load of 50 kips. These cracks were primarily located in the top half of the web and manifested as three cracks. One of these crossed the interface roughly 4 inches from the top of the web, while the other two were within the UHPC, approximately 12 inches from the interface. These cracks exhibited varying orientations, with the two in the UHPC angled at approximately 45 degrees, compared to the one crossing the interface at about 30 degrees (as illustrated in Figure 6.10).

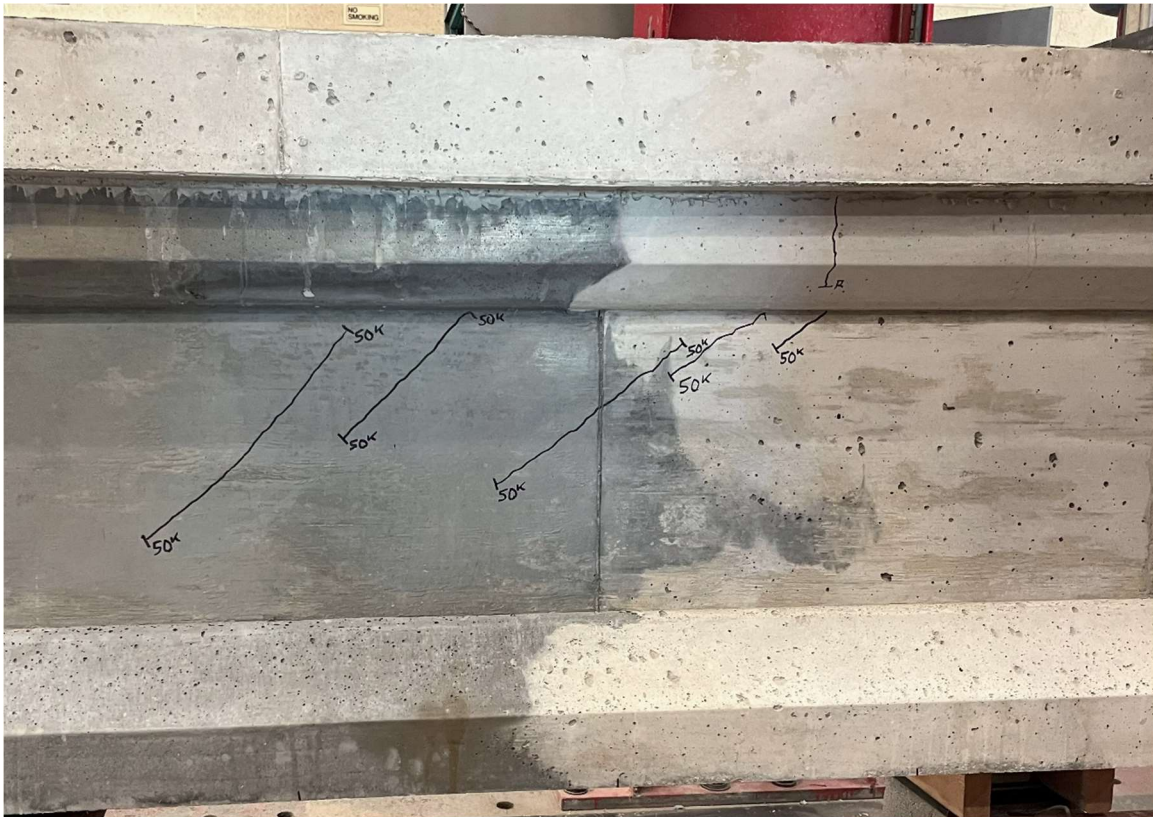


Figure 6.10: Initial cracks observed in BiFluid with Divider Girder No. 1

As the load increased up to 100 kips, several new cracks formed in the UHPC, extending the full height of the web at an approximate 30-degree orientation. The two initial cracks oriented at 45 degrees extended with increasing load, reaching down to the base of the web. In the SCC section, three new cracks appeared, each covering a substantial portion of the web, and propagated at load levels of 70, 80, and 105 kips, respectively. These cracks emerged in sequential order, approximately 3 inches below the previous one.

The first cracks penetrating the top and bottom bells occurred at 100 kips, near the load point and the support. Subsequent cracking post 100-kip loading primarily involved extensions of existing cracks, with only one new crack appearing at the mid-height of the web in the SCC. Shear cracking in the homogeneous SCC side of the girder appeared at 130 kips.

At the point of failure, a sudden fracture occurred across the top of the web, bridging between the existing shear cracks, and extended into the SCC from the interface for approximately 16 inches. This fracture then veered straight down to the base of the web, stopping directly above a flexure crack in the bottom bell. The resultant failure caused pieces of concrete in the web to push outward, creating an appearance on the surface resembling scales (as shown in Figure 6.11).

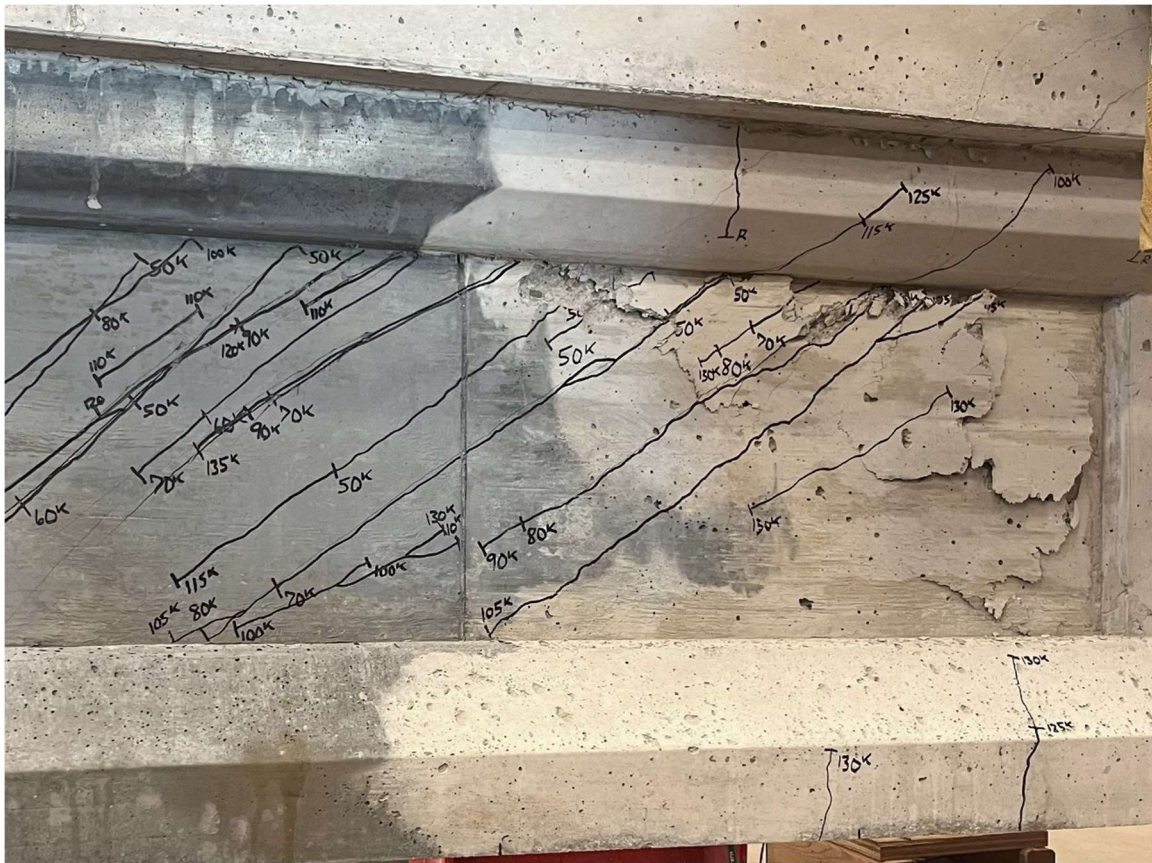


Figure 6.11: Initial bulging failure in the web of BiFluid with Divider Girder No. 1

Following the failure, the girder was subjected to additional deflection to highlight the failure modes present. Damage that resulted is captured in Figure 6.12 and shows fracture in the deck that continue into the top bell, consistent with a compression-type failure.



Figure 6.12: Observed, post-testing failure in BiFluid with Divider Girder No. 1

6.2.5 BiFluid with Divider Hybrid Girder No. 2

Existing cracks resulting from prestress release were marked before loading. These cracks were primarily distributed along the length of the SCC section in the top bell of the girder. Most of the initial cracks were confined to the top bell, with only a few extending into the web. None of the prestress-induced cracks were witnessed in the UHPC or the reduced width web section. Additionally, longitudinal cracking was observed at the SCC end of the girder, extending horizontally at about 4 inches above the base of the web.

The first cracks observed during the load test appeared at an applied load of 40 kips as two cracks in the top half of the web. One of these crossed the interface about 2 inches from the top of the web, while the other was approximately 6 inches from the interface in the SCC. The latter extended a quarter of the way down the web. These cracks exhibited an orientation of approximately 30 degrees, as illustrated in Figure 6.13.



Figure 6.13: Initial cracks observed in BiFluid with Divider Girder No. 2

As the load increased, the initial cracks continued to elongate, and new cracks developed across the interface, spanning the full height of the web in the same 30-degree orientation. Between 60 and 70 kips, another full web depth crack formed in the UHPC, approximately 8 inches above the extension of the first crack crossing the interface. More cracks propagated in the top of the web of the SCC as loading continued past 80 kips. These cracks were all parallel to the existing cracks and appeared to be evenly distributed throughout the web of the SCC, with a spacing of about 3 inches. In contrast, the spacing of the cracks in the UHPC was closer to 4 inches, and there were fewer cracks compared to the SCC.

Starting at 90 kips, cracking entered the top bell near the load point, and extensions grew into the bottom bell at 105 kips in the UHPC near the support. At the same time, the first flexural cracks appeared in the bottom bell at 120 kips below the load point. From 105 to 130 kips, there were numerous extensions and some new cracks that formed halfway between existing cracks in the web following the same orientation. Shear cracking in the homogeneous SCC side of the girder propagated at 130 kips.

At the point of failure, a sudden fracture occurred across the top edge of the web, bridging between the existing cracks, and extending approximately 18 inches into the SCC from the interface. This fracture then veered straight down the web, stopping at the mid-height directly above an existing flexure crack in the bottom bell.

Following the initial failure, the girder was subjected to additional deflection to highlight the failure modes present. The deck then exhibited failure that extended from the load point down into the top bell at the interface which was consistent with a compression-type failure, as depicted in Figure 6.14. The web failure continued to spread to the base of the web, and an existing crack crossing the interface widened to about a 1/8-inch gap. The fibers of the UHPC were clearly visible in this area and still bridged the gap, as illustrated in Figure 6.15.



Figure 6.14: Observed, post-testing failure in BiFluid with Divider Girder No. 2

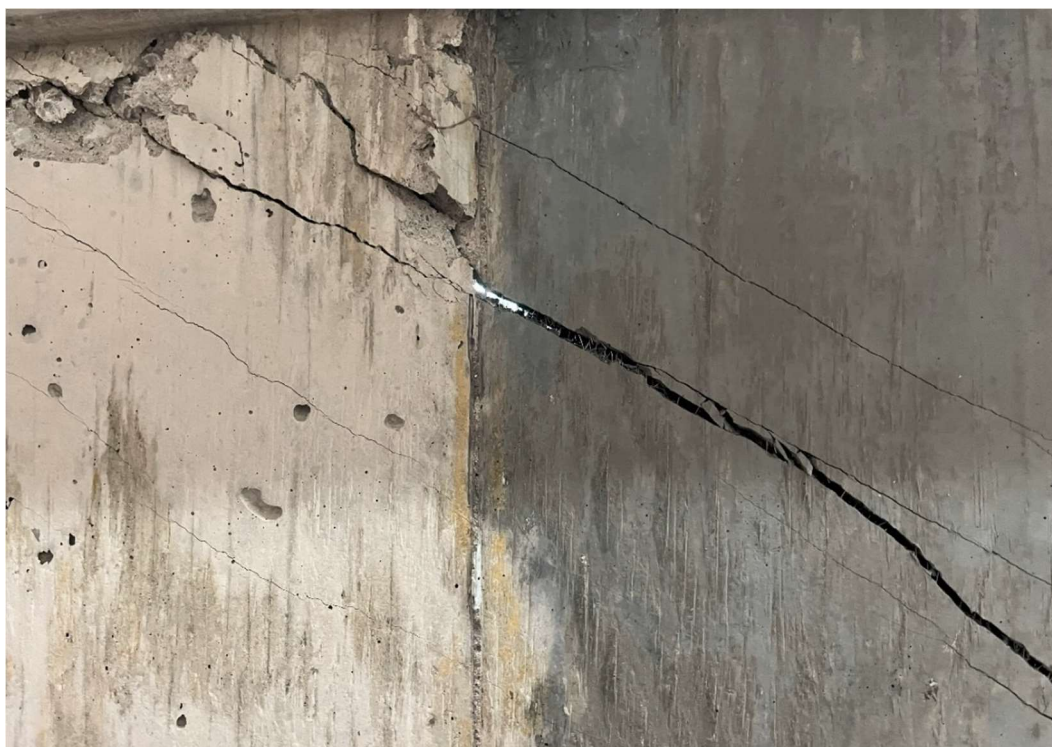


Figure 6.15: *Observed, post-testing crack at the interface in BiFluid with Divider Girder No. 2*

6.2.6 2D Trapezoidal Hybrid Girder No. 1

Prestress-induced cracking was observed along the length of the SCC section of the girder in the top bell prior to testing. Many of these cracks were confined to the top bell, with only a few extending down into the web. None of the prestress-induced cracks were present in the UHPC or the reduced width web section. Some longitudinal cracking was noted at the SCC end of the girder, primarily along the bottom row of strands and approximately 4 inches above the bottom of the web.

The first cracks observed during load testing appeared at an applied load of 40 kips in the UHPC section. These two cracks formed in a single vein at the mid-height of the web and extended from the interface, down within 3 inches of the base of the web, following a 30-degree angle. At this load, it was not apparent that the crack had crossed the interface. At 50 kips the cracks continued to the base of the web. Concurrently, the first cracks in the web of the SCC propagated, aligning with the existing cracking in the UHPC (Figure 6.16). At 60 kips, these cracks appeared to connect at the interface, although this connection was not fully evident due to casting imperfections at the surface (see Figure 6.17).



Figure 6.16: Initial cracks observed in 2D Trapezoidal Girder No. 1

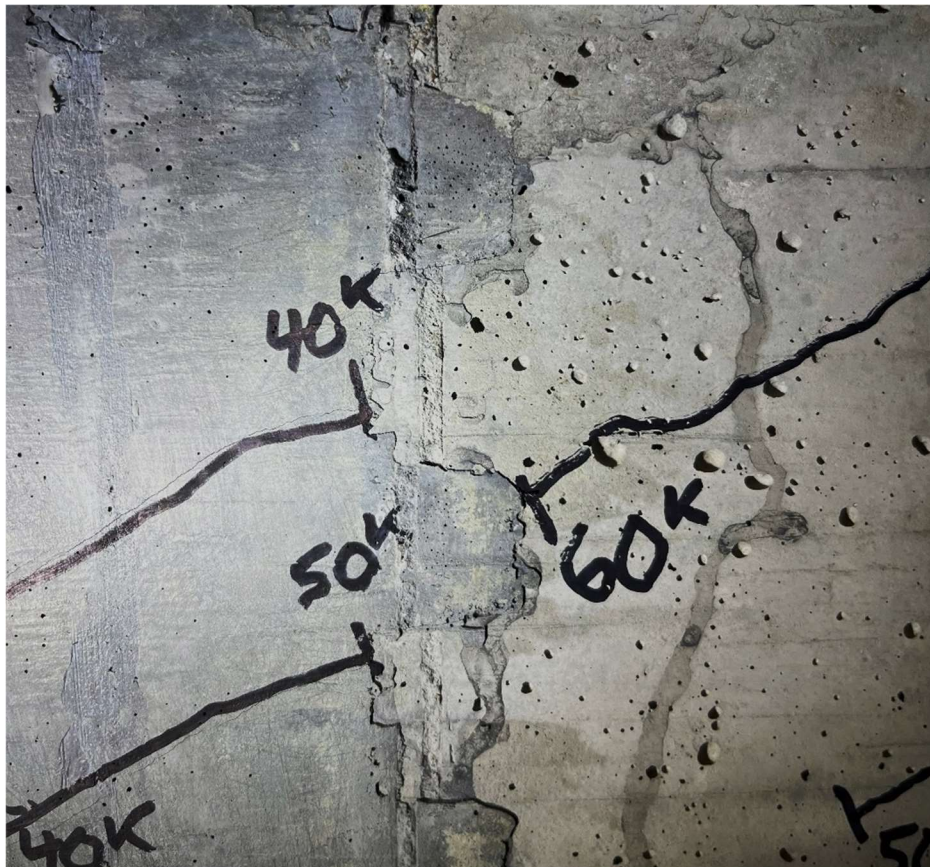


Figure 6.17: Initial cracks observed near the interface in 2D Trapezoidal Girder No. 1

The propagation of further cracking up to 105 kips in the SCC was primarily confined to the area at the top of the web near the interface. Parallel lines of cracks formed in this region, extending along the length of the reduced web section with increasing load. The first crack that penetrated the top bell occurred at 80 kips. In the UHPC, cracking was concentrated along the original vein of cracking and continued to expand into the bottom bell at 90 kips. Beyond 105 kips of loading, new cracks began to form in the bottom half of the web across the interface. A few of these cracks traversed the interface, and the cracks on the UHPC side extended into the bottom bell between the interface and support. In the SCC at this load, existing cracks extended down from the top of the web to the interface, and a new one formed at the middle of the web and extended down to the bottom of the web. The first flexural cracks appeared in the bottom bell at 120 kips below the load.

At the point of failure, a sudden fracture occurred that extended from the original vein of cracking at the interface, across the top edge of the web, bridging between existing cracks, and extending approximately 16 inches into the SCC. This fracture then veered downward in the web, stopping at mid-height, where it connected with an existing shear crack in the web. The extent of the fracture is pictured in Figure 6.18.

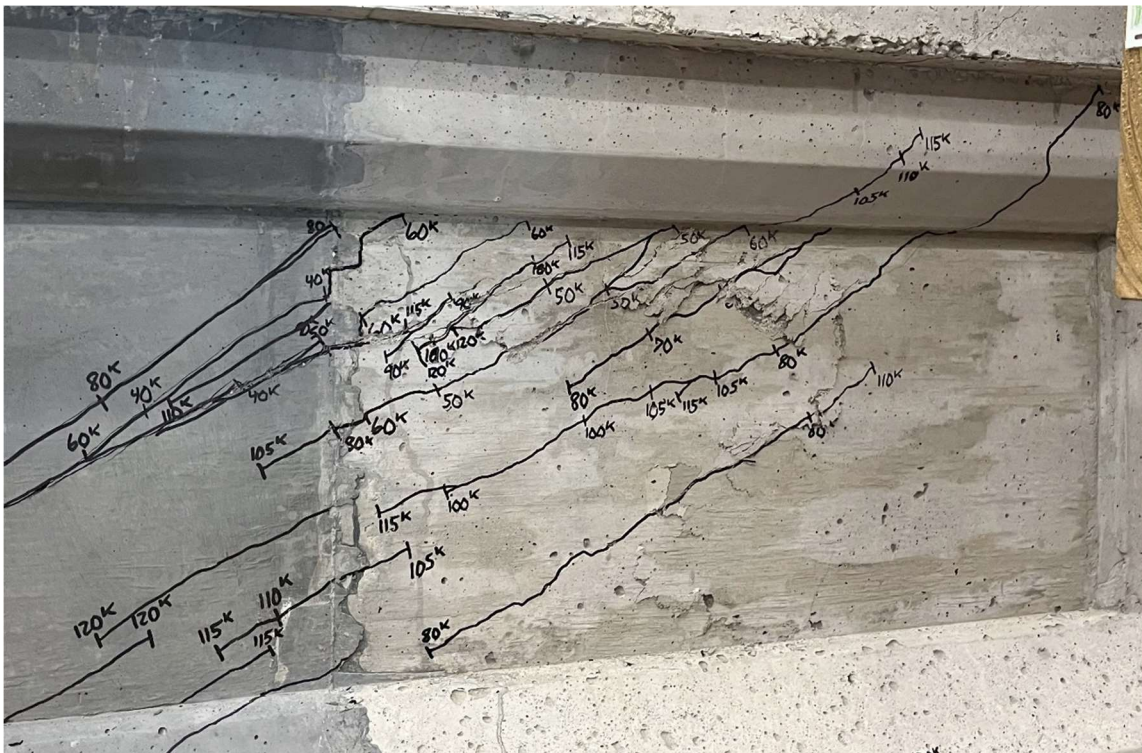


Figure 6.18: Initial web failure in 2D Trapezoidal Girder No. 1

Subsequently, additional deflection was applied to the girder to increase exposure of the failure modes. The deck exhibited failure that extended from the load point down to the bottom of the deck at the interface, consistent with a compression-type failure. At the interface location at the top of the girder, cracking followed the interface and then continued into the UHPC near the height of the top bars of the girder. The web failure continued to expand to the base of the web, causing large pieces of concrete in the web to be forced outward and fall away. This resulted in exposed internal reinforcing in the web, as depicted in Figure 6.19.



Figure 6.19: Observed, post-testing damage in 2D Trapezoidal Girder No. 1

6.2.7 2D Trapezoidal Hybrid Girder No. 2

Prior to testing, cracking resulting from prestress release was observed along the top of the SCC segment of the girder. Many of these cracks were concentrated in the top bell, with only a few extending into the web. None of the prestress-induced cracks were discovered in the UHPC or the reduced width web section. Additionally, there was evidence of longitudinal cracking at the SCC end, primarily along the bottom row of strands and another at approximately 4 inches from the bottom of the web.

The first cracks observed during load testing occurred at 40 kips across the interface. These cracks formed in a single vein at a distance of 3-4 inches from the top of the web, oriented at a 30-degree angle. This line of cracks extended from the top of the web in the SCC down to about 4 inches from the base of the web. Figure 6.20 shows the crack location and that the crack follows the texture of the interface for almost 2 inches before continuing into the adjacent concrete.



Figure 6.20: Initial cracks observed in 2D Trapezoidal Girder No. 2

The propagation of further cracking up to 105 kips in the SCC was mainly confined to the top region of the web. Parallel, 30-degree lines of cracks continued to form during this phase in this area. At 80 kips, one of these cracks penetrated the top bell. In the UHPC, cracking was more concentrated along the original line of cracking. As this line approached the bottom bell, shorter, individual cracks formed at angles increasing up to 45 degrees. However, this grouping continued to maintain a 30-degree slope through the web section and developed into the bottom bell at 80 kips.

Beyond 105 kips of loading, new cracks began to form in the bottom half of the web in the SCC and in the top half of the web in the UHPC. None of these cracks traversed the interface. Longitudinal cracks in the bottom bell of the UHPC formed at 115 kips and extended from the support to the base of the web at the interface. The first flexural cracks also appeared in the bottom bell at 115 kips near the load point.

At the point of failure, a sudden fracture occurred that extended from the original vein of cracking at the interface, across the top of the web in the SCC, bridging between existing cracks, and extended away from the interface approximately 18 inches. This fracture then veered downward at a steep 60-degree angle back toward shear cracking that led back to the interface, as shown in Figure 6.21. No web cracking was observed in the SCC in the long span of the girder during loading.

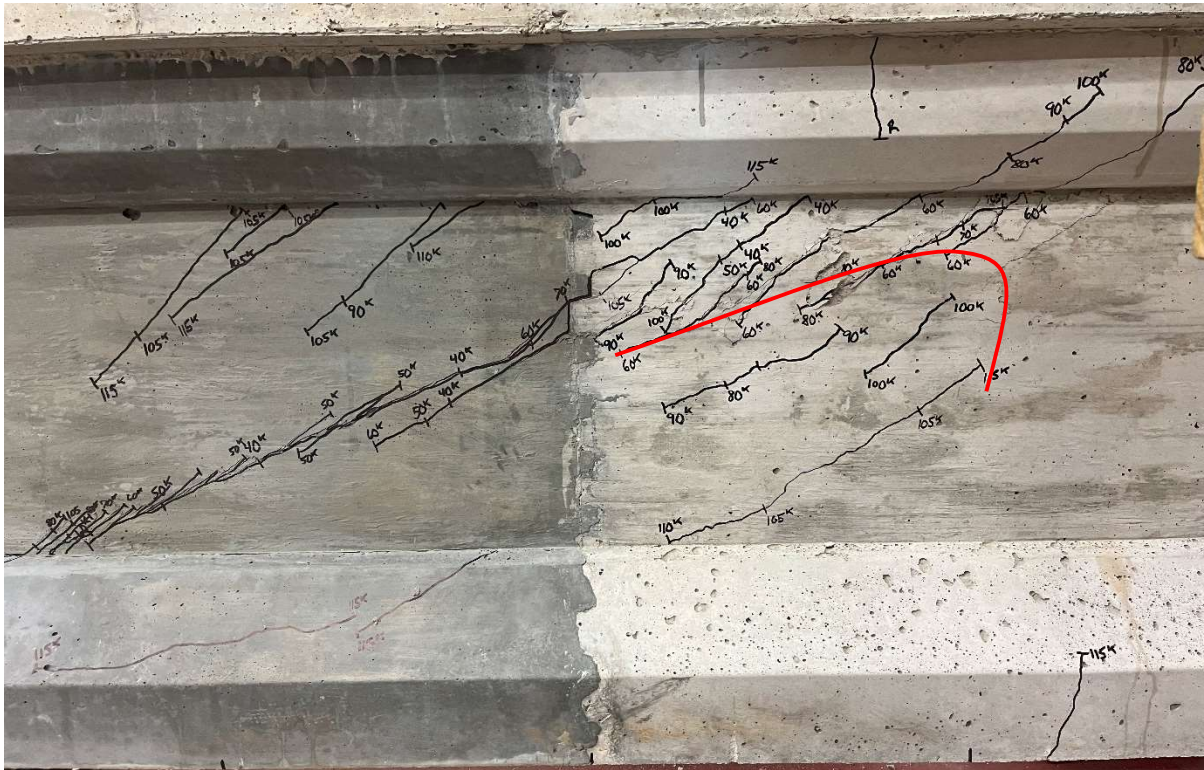


Figure 6.21: Initial web failure (path is highlighted in red) in 2D Trapezoidal Girder No. 2

Afterwards, the girder was subjected to additional deflection to highlight the failure modes. This induced additional crack extensions from the web of the SCC up through the top bell into the deck. Previous fracture in the top of the web continued to expand, forcing large pieces of concrete in the web outward, then followed existing inclined cracks to the base of the

web. The deck also exhibited failure that extended from the load point down to the bottom of the deck at the interface location and exhibited fracture consistent with a compression-type failure.

6.2.8 3D Trapezoidal Hybrid Girder No. 1

Prior to testing, cracks resulting from prestress release were observed along the length of the SCC section of the girder in the top bell. Most of these cracks were confined to the top bell, with only a few extending down into the web. None of the prestress-induced cracks were discovered in the reduced width web section. However, a single longitudinal crack was observed in the UHPC at the mid-height of the web and two additional longitudinal cracks were in the SCC end of the girder. The longitudinal SCC cracks were located along the bottom row of strands and roughly 4 inches above the bottom of the web. (See Figure 6.22 for visual reference.)



Figure 6.22: Observed cracking due to prestressing forces in 3D Trapezoidal Girder No. 1 (UHPC end (left) and SCC end (right))

The first cracks observed during load testing appeared at 40 kips across the interface, approximately 3-4 inches from the top of the web. These cracks formed in parallel, offset by less than an inch, and were oriented at a 30-degree angle. They extended from the top of the web in the SCC down to the middle of the web within the UHPC. (Refer to Figure 6.23.) From 50 to 60 kips of applied load, this line of cracks propagated more small, repeating cracks in the UHPC, with increasing angles for each crack that propagated closer to the base of the web above the support. At the same time, a new crack formed in the SCC, approximately 4 inches below the previous one, and extended from the top to the middle of the web.

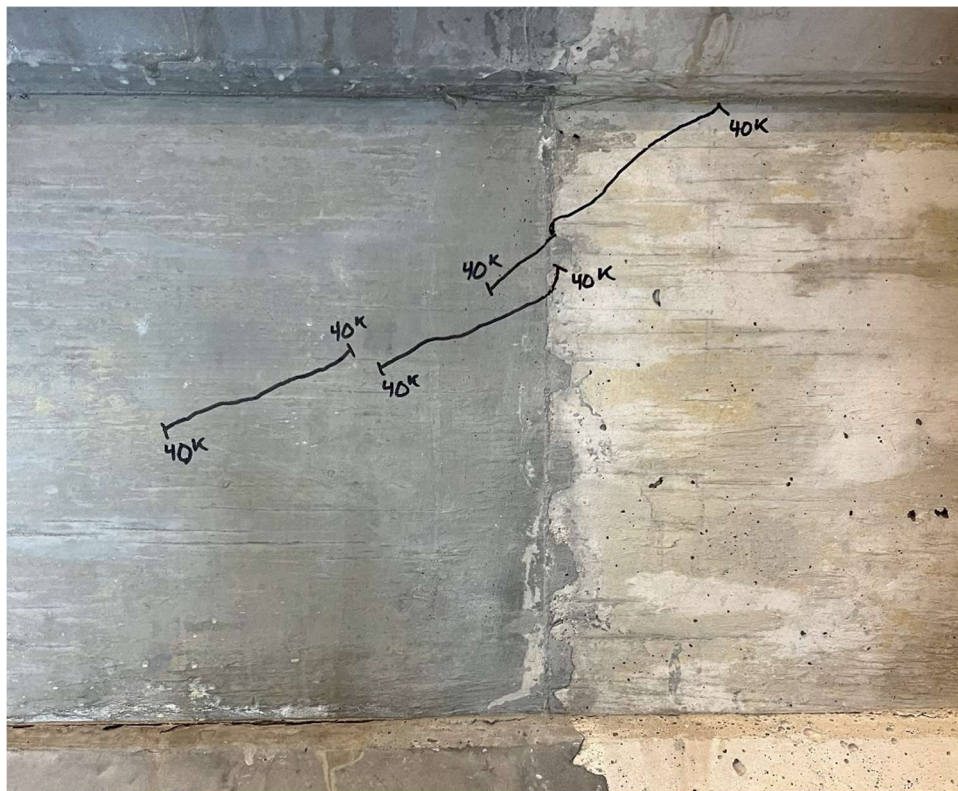


Figure 6.23: Initial cracks observed in 3D Trapezoidal Girder No. 1

As loading continued up to 80 kips, the primary line of cracks extended into the bottom bell at the support within the UHPC, and new parallel cracks formed in the SCC, gradually closer to the load point. Under an 80-kip load, one of these cracks in the SCC penetrated the top bell, extending to within an inch of the deck near the load point. Between 80 and 125 kips, the progression of cracking primarily involved extensions of existing cracks that were centralized in the top of the web of the SCC and along the original vein of cracking in the

UHPC. The spacing of the cracking in the SCC decreased with increasing load by the development of new cracks or extensions that propagated halfway between existing ones. Few new cracks formed during this phase, and those that did develop were in the web below the original line of cracking. One of the new cracks that formed across the interface at 100 kips was located 4 inches from the bottom of the web (visible in Figure 6.24). At 115 kips, the first flexural cracks appeared in the bottom bell between the interface and the load point. Shear cracking in the homogeneous SCC side of the girder appeared at 125 kips and extended through the full depth of the web.



Figure 6.24: Crack propagation at 125 kips applied load for 3D Trapezoidal Girder No. 1

At the point of failure, a sudden fracture occurred, starting from the location where the original vein of cracking met the top of the web. It then traversed across the top of the web for about 12 inches, bridging between existing cracks in the SCC. This fracture then veered downward toward the middle of the web, halfway between the interface and the load point.

Post failure, the girder was subjected to additional deflection to highlight the failure modes. This induced a new longitudinal crack at the end of the girder in the web of the UHPC, which extended to the main vein of cracks in the web. This occurrence was accompanied by a loud crack, indicating the release of significant energy. The fracture in the top of the web continued to grow downward to the base of the web and joined a new crack that followed the

base of the web back to the interface. Small amounts of spalling were observed along the fracture in the SCC and at the location where it crosses the interface (depicted in Figure 6.25).



Figure 6.25: Observed, post-testing damage in 3D Trapezoidal Girder No. 1

6.2.9 3D Trapezoidal Hybrid Girder No. 2

Prior to testing, cracking resulting from prestress release was observed to be predominantly distributed along the length of the SCC segment of the girder's top bell. Many of these cracks were localized within the top bell, with only a few extending down into the web. None of the prestress-induced cracks were observed in the reduced width web section. Additionally, a single longitudinal crack was found in the UHPC in the middle of the web, and two more were found in the SCC. These SCC cracks were situated along the bottom row of strands and at approximately 4 inches from the bottom of the web.

The first cracks observed during load testing occurred at 50 kips, crossing the interface about 3 inches from the top of the web, along the side of the UHPC protrusion of the texture. These cracks were aligned with slight offsets of less than an inch between them and were oriented at a 30-degree angle. They extended from the top of the web in the SCC down to approximately 4 inches from the bottom of the web within the UHPC.

At 60 kips, two new cracks emerged in the SCC, following the same orientation. These cracks were equally spaced 3 inches apart, with one extending from the top of the web to the interface, terminating on the SCC side. Throughout the loading up to 90 kips, the predominant behavior of the cracking was through extensions, with minimal new crack formation. Many of the cracks on the UHPC side had extended into the bottom bell by the 90-kip increment.

At 100 kips, the cracks in the top of the web of the SCC had extended into the top bell, and a new crack had formed across the interface in the bottom of the web. During the loading phase from 90 to 120 kips, few new cracks appeared, and many of the existing cracks continued to extend. At 115 and 120 kips, two cracks formed in the UHPC web above the initially formed cracks, spaced 3 inches apart from the first cracks in the section (as shown in Figure 6.26).

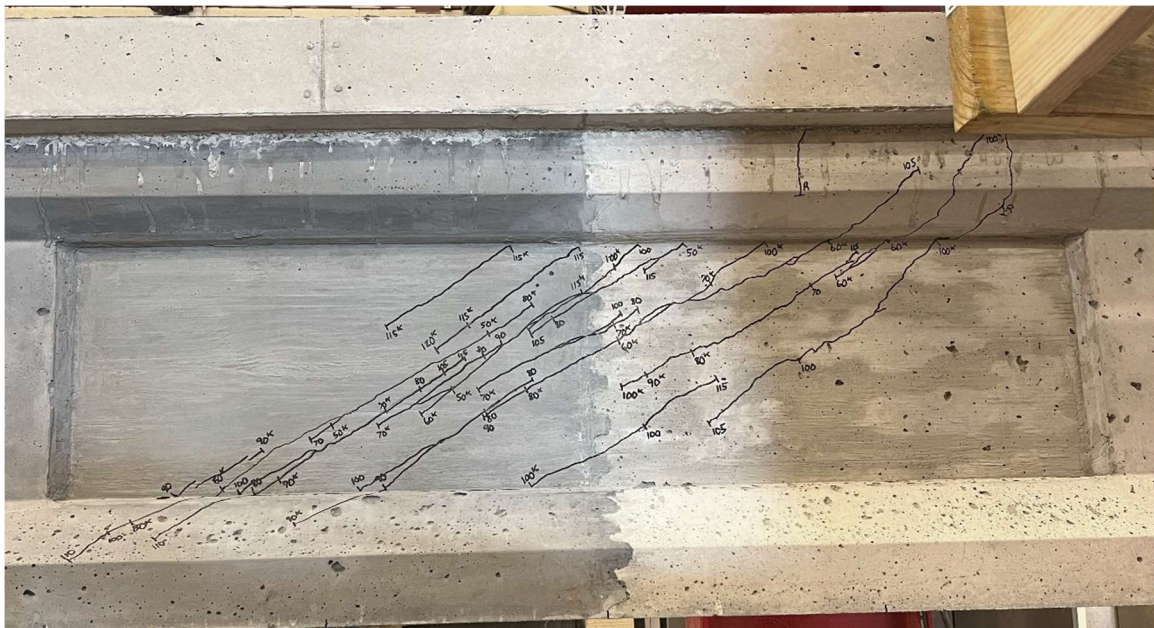


Figure 6.26: Crack propagation at 115 kips applied load for 3D Trapezoidal Girder No. 2

At 125 kips, the first flexural cracks appeared in the bottom bell between the interface and the load point. Simultaneously, shear cracks formed in the homogeneous SCC side of the girder, with many extending from the top of the web to the middle of the web, and a few extending the full web depth.

The point of failure gave a sudden fracture that initiated at the point where the original vein of cracking met the top of the web. This fracture extended approximately 14 inches across the top of the web of the SCC, bridging across existing cracks. After failure, as the girder was subjected to additional deflection, the web failure followed two paths: one through the interface into the UHPC and the other descending just beyond the middle of the web in the SCC. The resulting failure in the SCC led to spalling of concrete pieces in the web, which were forced outward along the fracture path. In the UHPC, the fracture path extended 12 inches beyond the interface before veering down to the support at a 45-degree angle (as depicted in Figure 6.27).



Figure 6.27: Observed, post-testing damage in 3D Trapezoidal Girder No. 2

6.3. Analysis of Hybrid Girder Failure Mechanism

The following section investigates the shear failure mechanism observed in the large-scale hybrid girder testing program and proposes a theoretical model for predicting those failures. Section 6.2 described the failure progression of each individual hybrid girder specimen with respect to the implemented interface construction method (i.e., BiFluid, 2D Trapezoidal, etc.). In general, all nine large-scale hybrid girder specimens failed in a very similar manner, with increasing formation and extension of diagonal web cracking followed by crushing of the web in the SCC portion of the shear span. There were slight variations in

the propagation of diagonal web cracking between specimens, but these primarily involved variations during the early loading stages and only occasionally involved slight discontinuities for a few diagonal cracks as they traversed the interface. Those discontinuities or slight jogs formed at the interface only in the cold-joint interface specimens (i.e., the non-BiFluid specimen interfaces). However, none of the interfaces showed any signs of failure in terms of either unusual cracking or any vertical or horizontal slippage. The large-scale specimen shear failures were remarkably consistent compared to the typical high degree of variability observed in shear testing of reinforced and prestressed concrete, which indicates a very specific type of failure mode for hybrid girders.

6.3.1 Shear Behavior as a Function of the Shear Span-to-Effective Depth Ratio

The shear span-to-effective depth ratio (a/d) is one of the primary factors used to quantify the shear behavior of reinforced and prestressed concrete. There are varying terminologies and values used for the different ranges of a/d and the attributed shear behaviors and corresponding failure modes. In terms of shear behavior, beams or spans of beams can be classified into three categories: deep beams, short beams, and slender beams, or alternatively, deep beams, short beams, and normal beams. For each of the three categories, the terminology and limits for a/d vary slightly between sources. Different researchers and design codes specify differing ranges with their own terminology, but the prescribed behavior for each respective category is consistent.

Deep beams are typically considered to be in the lower a/d range, with some authors specifying a range from 0 to 1.0 (Ibraheem & Rasheed, 2023; Wight & MacGregor, 2012). Alternatively, ACI 318 (2019), a consensus document, quantifies deep beams as members having concentrated loads within a distance from the support equal to twice the height (h) of the member. In terms of a/d values, this sets the range for deep beams from 0 to just over 2.0, depending on the depth of the flexural reinforcement centroid (d). In a study by Pimanmas (2010), deep beams were classified as having a more flexible upper bound for a/d , ranging from 0 to 2.0 or 2.5. The response of deep beams is generally characterized as having steeper cracks (60 degrees or more, from horizontal) that develop and extend from the load to the region near the support, with post-cracking load being resisted through compressive struts and tension ties in the form of an internalized truss system until failure (Pimanmas, 2010; Wight & MacGregor, 2012).

The shear behavior of short beams also includes the development of inclined cracks between the load and support, but with a subsequent redistribution of internal stresses from arching action to a more Bernoulli-type beam behavior. This enables a further increase in load resistance before exhibiting failure. Short beams are considered to have a/d values between 1.0 and 2.5 (Ibraheem & Rasheed, 2023; Wight & MacGregor, 2012). Although not explicitly indicated as short beams, Nawy (2010) describes a very similar shear behavior for beams with a/d values less than 2.5.

Beams with a/d values between 2.5 and 6.0 have differing definitions and connotations. Wight and MacGregor (2012) and Pimanmas (2010) classify members with these a/d values as slender beams. Wight and MacGregor further classify beams with ratios greater than 6.0 as very slender beams. They go on to describe the shear behavior of slender members as the propagation of inclined, flexure-shear cracks that continue to expand up to failure; noting that, the more slender the member, the more dominant flexural failure becomes with very slender members failing primarily in flexure. Similarly, Ibraheem & Rasheed (2023) refer to members exhibiting identical behavior as normal beams and quantify them as having a/d values greater than 2.5. Nawy (2010) classifies slender beams with a different upper limit, stating that the a/d range falls within 2.5 to 5.5. Nawy then describes the behavior of slender beams as an initial formation of flexure cracking in the bottom of the member at midspan, followed by a loss of bond and the formation of inclined cracking at a distance from the support of $1.5d - 2.0d$. Following a redistribution of stress, typically one crack will expand and extend to the compression zone.

6.3.2 Shear Failure Mode as it Relates to Shear Span-to-Effective Depth Ratio

Contrary to the classification system for shear spans of beams according to a/d values, there is generally good agreement in the literature for what failure modes correspond to deep beams, short beams, and slender (or normal) beams. In concrete beams without transverse reinforcement, deep beam failure results from the disruption of horizontal shear flow by inclined cracking, creating arching action. In this situation, the longitudinal tension reinforcement acts as a tie for the arch and failure is typically related to anchorage failure of the reinforcement (Wight & MacGregor, 2012). For concrete beams with transverse reinforcement, the post-cracking, tied-arch behavior of a deep beam is characterized by an “inclined thrust” between the support and load point and is referred to as splitting failure (or a

“true failure” by Ibraheem & Rasheed (2023)). For this mode, the ultimate failure mechanism becomes a splitting failure along the tension steel or, in members with large amounts of transverse reinforcing, compression failure in the web (Ibraheem & Rasheed, 2023; Pimanmas, 2010).

Common failure modes for short beams include the same failure modes as deep beams, plus shear tension and shear compression failure modes. Shear tension failure in this member class can occur through the failure of the tension reinforcement by bond or dowel action, which is typically observed by the appearance of cracking along the tension reinforcement. Alternatively, shear compression failure is described as crushing of the concrete directly above the inclined crack extension. Short beams are more likely to exhibit behavior that is closer to Bernoulli-type beam theory with more typical and brittle shear tension or shear compression failures (Ibraheem & Rasheed, 2023; Nawy, 2010; Wight & MacGregor, 2012).

Slender beams typically experience failure through diagonal tension or flexure. Diagonal tension failure occurs when the primary tensile crack forms and the tensile capacity of the section exceeds the flexural capacity. As beams experience higher a/d values, flexural failure becomes the increasingly more dominant failure mode.

6.3.3 *Hybrid Girder Shear Span-to-Effective Depth Ratio*

Classification and Observed Shear Failure Mode

The previous discussions of shear failure modes and a/d classifications are primarily applicable to rectangular cross-sections typical of reinforced concrete and post-tensioned concrete. However, precast, pre-tensioned concrete beams commonly utilize more economic, I-shaped cross-sections. Although the specific shapes vary considerably, from AASHTO standard sections to bulb-tee sections, they all share the commonality of a considerably reduced web thickness.

This reduced web thickness gives rise to another type of shear failure mode, referred to as web-shear failure, which is characterized by the formation of inclined cracks that initiate within the web (Ibraheem & Rasheed, 2023; Nawy, 2010; Wight & MacGregor, 2012). This behavior is due to the fact that the principal tension stresses at or near the neutral axis exceed those in the bottom flange, leading to the formation of inclined web-shear cracks prior to the formation of any inclined flexure-shear cracks. For slender beams, the inclined web-shear cracks typically form at an angle of 45 degrees for reinforced concrete and 30 to 35 degrees

6.3.4 Hybrid Girder Shear Failure Characterization and Proposed Prediction Methods

Using the analysis presented in Section 6.2, there is a clear indication that the failures observed across all nine hybrid girders involved crushing of the web, as shown in Figure 6.29 for example. This crushing failure is located within the upper third of the reduced web section. Web-shear compression failures, on the other hand, typically develop as concrete crushing directly above the propagated web-shear cracking in the section (Wight & MacGregor, 2012), which is not the mode observed in the hybrid girders.



Figure 6.29: Representative extents and location of web crushing behavior in hybrid girder specimens⁶

⁶ BiFluid Hybrid Girder No. 3 at failure.

Web crushing failure is typically encountered in heavily reinforced deep beams after diagonal tension cracks develop in the web. For this failure type, the beam's ability to resist higher shear loads primarily relies on the formation of compressive struts within the concrete, which develop between the diagonal cracks. As such, this failure mechanism depends largely on the compressive strength of the concrete within the strut (Pimanmas, 2010). Through the analysis of the failure response for each specimen described in Section 6.2, it is evident that compressive strut failure is the most accurate description of the hybrid girder failure response.

6.3.5 *Shear Capacity Using Strut-and-Tie Modeling*

The following subsection investigates the use of strut-and-tie modeling to predict failure response of the hybrid girders evaluated in this study. The observed failure mechanism clearly indicates web crushing behavior, which can be modeled through the strut-and-tie approach. Strut-and-tie modeling characterizes the compressive forces within the web as struts, which are typically described as bottle-shaped or idealized as prismatic. Failure of these struts are more sudden and brittle, which is less desirable compared to the yielding of tension ties (Wight & MacGregor, 2012). Therefore, the overall capacity in the strut-and-tie model relies on the capacity of the individual components, including struts, ties, and the intersections (nodes) that connect them. Codes such as ACI 318 (2019) or AASHTO LRFD (2020) dictate the strength limitations for each of these components.

Determination of the strut-and-tie model configuration is largely subjective and is often considered to be “more an art than an engineering science” (Nawy, 2010). However, given the observed consistent failure pattern for the hybrid girders, it is evident that the compressive strut should be oriented parallel to and in-line with the web-shear cracking. The prestressing strand within the bottom bell serves as the tie in the strut-and-tie model in the shear span of the girder. Node A is located at the support, and node B is located at the load point. The strut-and-tie model is shown graphically in Figure 6.30 and is overlaid on a photo of one of the tested hybrid girder specimens.

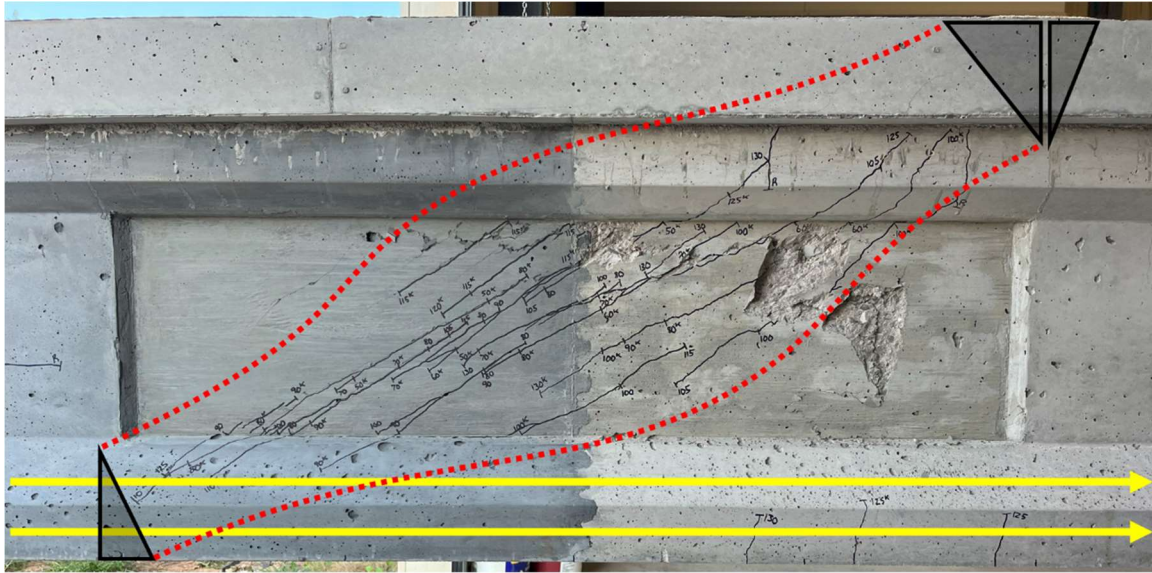


Figure 6.30: Overlaid diagram of strut-and-tie model on Figure 6.27⁷

The primary focus of the strut-and-tie modeling was to determine the capacity of the compressive strut A-B within the shear span. Also shown in Figure 6.30 is a Compression-Compression-Tension (C-C-T) node at the reaction point, node A, and a Compression-Compression-Compression (C-C-C) node at the load point, node B. Node B was split into two C-C-C nodes with the top load-bearing surface equal to the width of the load plate (8 inches). The horizontal location of the sub-node connecting strut A-B was determined using structural analysis of the loading arrangement, providing a sub-node width equal to two-thirds the width of the load plate, or approximately 5.33 inches. Both sub-nodes were vertically centered at half the depth of the compression block determined by flexural capacity calculation, approximately 2.65 inches below the deck top surface. Node A at the support was aligned with the centroidal location of the prestressing strands at 2.67 inches from the base. From these coordinates, the dimensions and orientation of strut A-B can be determined using simple trigonometric identities.

Two approaches were taken to determine the size of compressive strut A-B. Either approach involves estimating the width of the strut using research-based recommendations. The first approach uses Equation 6-1 given by Wight & MacGregor (2012) to determine the

⁷ In the strut-and-tie model of Figure 6.30, nodes are shown in black (node A on the left, node B on the right), tension ties are shown in yellow in place of the prestressing strands, and struts are shown by red-dotted lines.

width of a bottle-shaped strut (b_{ef}) based on the width of the connecting bearing area at the node (b_{ns}) and the strut length (l). Alternatively, the second approach calculates the strut width as a third of the total strut length (Wight & MacGregor, 2012). Both of these dimensions are taken at the mid-strut and are limited to a measurement no smaller than the node width and no larger than the area available in the member. The area of the mid-strut can be calculated by multiplying these effective strut widths by the web thickness to determine the area of the strut. The corresponding areas according to the effective width using Equation 6-1 and by using one-third the strut length are 21.02 in² and 25.15 in², respectively.

$$b_{ef} = b_{ns} + l/6 \quad \text{Equation 6-1}$$

The capacity of the strut can then be calculated by multiplying the mid-strut area by the effective compressive strength of the strut. Effective compressive strength (f_{cs}) is calculated in accordance with the selected design method, such as the ACI 318 code, and is generally taken as the concrete compressive strength multiplied by an efficiency factor. There are many sources that define the efficiency factor as having different values in the range from 0 to 1.0. Most of the values take into consideration the strength of the concrete, load duration effects, transverse tensile stress, and cracking within the strut (Wight & MacGregor, 2012).

ACI 318 (2019) calculates effective compressive strength using Equation 6-2, where f'_c is the compressive strength and β_s is the strut coefficient. According to the ACI 318 code, the strut coefficient ranges from 0.4 to 1.0 depending on the conditions relating to the strut. For a bottle-shaped strut acting as a boundary strut, the strut coefficient is taken as 1.0. Taking the compressive strength as being equal to the average compressive strength of the SCC recorded in the hybrid girder testing, the effective compressive strength of the strut is calculated as 7260 psi. However, as mentioned previously, the intent for this analysis is to uncover an accurate approach to estimate the failure of hybrid girders. For this reason, two additional values for effective strength were included in the analysis. The first value was calculated by taking the strut coefficient in Equation 6-2 as equal to 0.75 in accordance with an internal strut with adequate reinforcement to provide lateral restraint of the strut. The second value sets the effective compressive strength equal to the full compressive strength of the SCC. The resulting strengths of these variations give effective compressive strengths of 5440 psi and 8540 psi, respectively.

$$f_{cs} = 0.85f'_c\beta_s \quad \text{Equation 6-2}$$

Based on simple truss analysis, the ultimate applied load to cause failure can be determined from the strut capacity. A summary of the different ultimate applied loads calculated from the strut capacities using the different effective widths and effective compressive strengths is given in Table 6.1. The results of these calculations show that the most accurate method, according to the capacities of the hybrid girders, utilizes an effective strut width equal to one-third the strut length with the full compressive strength of the SCC as the effective compressive strength. It is proposed that this approach be used as the potential prediction method for hybrid girder shear capacity. All other evaluated methods produce conservative or overly conservative capacities compared to the average hybrid girder ultimate load.

Table 6.1: *Ultimate applied load capacity estimations using different effective strut widths*

Method for Effective Compressive Strength, f_{cs}	Load Capacity using b_{ef} in Eqn. 6-1 (kips)	Load Capacity using $b_{ef} = 1/3$ Strut Length (kips)
$0.85*f'_c*\beta_s$	74.40	89.05
$0.85*f'_c$	99.19	118.73
f'_c	116.70	139.69

The average normalized capacity for the strut-and-tie model using the recommended effective width and effective compressive strength with respect to the experimental capacities is 1.074. The average normalized capacity for the strut-and-tie model using the ACI 318 limits for restrained movement of an internal strut is 0.913. This affirms the reported conservative estimation given by code-guided approaches of the strut-and-tie method. A buffer is integrated into the code, which accounts for the inherent variability in the layouts of the internalized truss systems. Other factors also accounted for in the conservatism of the effective strength are the effects from load duration, transverse tensile strains, and pre-existing cracks crossing the strut. The result of these factors of the design model gives a less accurate representation following crack propagation (Nawy, 2010). Table 6.2 provides the capacities for the strut-and-tie models

using the full compressive strength of the SCC and the ACI-calculated effective compressive strength of a boundary strut, normalized to the average capacity of each hybrid girder set. The normalized capacities are expressed as unitless numbers, with values less than 1.00 indicating a conservative estimation of the actual capacity and values greater than 1.00 showing an overestimation.

Table 6.2: Summary of experimental capacities normalized to the strut-and-tie methods⁸

Hybrid Girder Set	S&T f'_c Capacity/ Experimental Capacity	ACI S&T Capacity/ Experimental Capacity
BiFluid	1.048	0.891
BiFluid with Divider	1.018	0.865
2D Trapezoidal	1.163	0.988
3D Trapezoidal	1.068	0.908

6.3.6 Shear Capacity Using ACI Detailed Method

When applying the equations in the ACI 318 Detailed Method to determine shear capacity for prestressed members, the calculation of the concrete contribution in the hybrid girders is controlled by web-shear instead of flexure-shear. Based on this approach, the calculated shear capacity of the SCC section in the shear span was determined to be 88.6 kips. With the loading arrangement employed for the testing of the hybrid girders, this corresponds to an applied load of 132.4 kips. A comparison between this value and the average load capacities obtained from hybrid girder testing validates the use of the ACI Method as an accurate predictor of shear capacity for hybrid girders of the type investigated herein.

The results of this comparison are presented in Table 6.3, where the ACI-calculated shear capacity is normalized to the average experimental capacities obtained during testing. The average normalized capacity for the ACI Detailed Method with respect to the experimental capacities was determined to be 1.02, with a coefficient of variation of 5.04%. This represents

⁸ The methods in Table 6.2 use the full compressive strength of the SCC (second column) and the strut-and-tie with the ACI 318 strength limit for boundary struts (third column)

excellent agreement between the code prescribed capacities and those found through testing even though the failure mechanism was not indicative of a shear-compression failure in the web, which is the basis of the code equation for web-shear failure. Furthermore, utilizing this same equation to estimate initial cracking in the SCC predicts the propagation of web-shear cracks at an applied load of 39.5 kips. Section 6.2 establishes this calculation to be accurate or slightly conservative regarding initial crack formation in the hybrid girders. Although, many of these initial cracks also formed in the UHPC or exclusively within that side of the interface, which suggests the possibility of strain equilibrium occurring across the interface. This observation warrants further investigation.

Table 6.3: *Summary of hybrid girder load capacities and comparison to the ACI 318 design capacity*

Hybrid Girder Set	Avg. Load Capacity (kips)	Design Capacity/ Experimental Capacity
BiFluid	133.26	0.994
BiFluid with Divider	137.24	0.965
2D Trapezoidal	120.13	1.102
3D Trapezoidal	130.74	1.013

6.4. Proposed Hybrid Girder Design Process

Based on the insights presented in this chapter, it is recommended that the design process for hybrid girders should follow the approach specified below.

1) Sizing UHPC End Regions:

First, designers shall select the overall layout of the girder and extents of the UHPC. These sections should extend at least from the support to the edge of the discontinuity region, or beyond it. This extension provides reinforcement and strengthening that is critical to the hybrid girder concept.

2) *Design Conventional Concrete Segment of Hybrid Girder:*

The design of the conventional concrete segment of the girder should generally align with traditional design methods used for prestressed girders. However, there is an exception for the shear design of the conventional concrete for the end regions of the girder.

A lower bound of the nominal shear capacity of the conventional concrete segment for the end regions of the girder should be determined through the ACI Detailed Method, where web-shear failure will often control for members with reduced web thickness. However, the design method must also include an upper bound check based on the strut-and-tie model of the end-region at the location of the UHPC/conventional concrete interface. In this model, the strut strength would be based on the compressive strength of the conventional concrete.

3) *Upper Bound Shear Capacity Check for Conventional Concrete in the End Regions:*

The compressive strut should have vertical extents equivalent to the internal moment arm in the full section and be oriented parallel to and in-line with web-shear cracking, typically at a 25- to 35-degree angle. Initial calculations for the angle of cracking can be estimated but should also be verified by calculating the orientation of the principal tension stress at the level of the centroid using Mohr's Circle. Alternatively, AASHTO (2020) provides a calculation for the angle of cracking based on the MCFT. The crack angle (θ) is calculated using the net longitudinal strain in the tensile reinforcement (ϵ_s) as per Equation 6-4. Calculation of the net strain on the tensile reinforcement is given by Equation 6-5 in AASHTO LRFD Article 5.7.3.4.2.

$$\theta = 29 + 3500\epsilon_s \quad \text{Equation 6-4}$$

$$\epsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{E_sA_s + E_pA_{ps}} \quad \text{Equation 6-5}$$

where:

ϵ_s = net longitudinal stain in the tensile reinforcement

$|M_u|$ = absolute value of the factored moment at the crack location (kip-in.)

d_v = effective shear depth, taken as the length of the internal moment arm between resultant tensile and compressive forces due to flexure (in.)

N_u = factored axial force (taken as negative value for compression) (kip)

- V_u = factored shear force (kip)
 V_p = shear force due to prestressing (taken as positive if resisting applied shear) (kip)
 A_{ps} = area of prestressing steel in the tensile zone (in²)
 f_{po} = effective prestress in the strands, taken as 70% of ultimate tensile strength of prestressing steel for “usual levels of prestressing” (ksi)
 E_s = elastic modulus of mild steel reinforcement (ksi)
 A_s = area of mild steel reinforcement in the tensile zone (in²)
 E_{ps} = elastic modulus of prestressing steel (ksi)

The calculation of the strut's capacity shall be determined from the area of the mid-strut and the compressive strength of the conventional concrete. The area of the mid-strut can be computed using the member's web thickness and the effective strut width, which is taken as one-third of the total strut length. Equation 6-6 defines the proposed nominal shear capacity check for the conventional concrete portion near the end regions of a hybrid girder.

$$V_n \leq f'_c \left(t_w \cdot l_s / 3 \right) \sin (\theta) \quad \text{Equation 6-6}$$

where:

- V_n = nominal shear capacity (kips)
 f'_c = compressive strength of the mid-strut, taken as the full compressive strength of the conventional concrete (ksi)
 t_w = thickness of the web section at the mid-strut location (in.)
 l_s = length of the strut (determined by the distance between the internal compressive and tension forces and θ) (in.)
 θ = angle of web-shear cracking (degrees)

This capacity check serves as an upper bound to mitigate web crushing behavior, a brittle failure type that is generally undesirable compared to yielding of reinforcement. It is important to note that this check pertains to nominal capacity, and for design purposes, load resistance and strength reduction factors must still be applied.

4) *UHPC Capacity Check:*

The UHPC ends should be verified for sufficient capacity against bearing and prestress release stresses. In cases where high stresses are present, it may be necessary to include reinforcement. However, the research conducted has indicated that the standard, minimum amount of vertical reinforcing is not required for shear in the girder due to the significant contribution of the fibers in the UHPC matrix. Shear capacity of the UHPC can be determined with conservative results by employing any of the three previously discussed approximations for UHPC shear capacity in Chapter 4.

5) *Interface Design Considerations:*

To satisfy the proposed design process, the interface between the UHPC and conventional concrete should be designed in a manner such that the controlling shear capacity is governed by the weaker conventional concrete, not the interface itself. Achieving this involves implementing the recommendations outlined in the results and analysis of the small-scale program listed in Chapter 5. Otherwise, shear capacity of the section should be based on the shear-friction model of the interface for textures with shear capacities less than the conventional concrete.

6) *Design Connection Between Deck and Girder:*

Horizontal shear capacity between the UHPC girder and the deck should be resolved by including vertical reinforcing elements, following conventional design methods. It is anticipated that only minimal reinforcement would be needed to satisfy horizontal shear requirements at these locations. Stirrups that connect the longitudinal reinforcement of the deck with the top bars of the girder are typically adequate for horizontal shear transfer. Therefore, full-depth vertical reinforcement in the UHPC is generally unnecessary if the shear resistance provided by the UHPC is sufficient for design.

Chapter 7: Findings, Conclusions, & Recommendations

7.1. Introduction

The contents of this dissertation involved two research programs, each with specific objectives, and an overall emphasis on hybrid girder construction and evaluation. The small-scale program conducted a comparative assessment of various interfaces for characterization and their applicability in hybrid girders. The interface textures were ranked based on their strength and overall performance, and the top-performing interfaces were incorporated into a larger-scale program that evaluated the load response of the hybrid girder end-region under shear loading. Limited information is available regarding best practices for the interface methods selected for this study; the same is true for the potential failure modes and limiting factors of hybrid girders.

The small-scale program in this research focused on comprehensive interfacial characterization, with a specific emphasis on assessing the bond between Ultra-High-Performance Concrete (UHPC) and conventional concrete (or Self-Consolidating Concrete, SCC). The primary goal was to develop an interface that could resist failure and provide a load response similar to that of monolithic concrete girders. Various textures were selected for the program, referenced from existing literature. Seven different interface types were implemented, including three commonly used cold-joint textures and four innovative approaches to interface construction (two BiFluid types and two Trapezoidal textures). All interfaces underwent testing for shear capacity under pure shear loading and combined shear/compression loading. The top five interfaces were further subjected to flexural strength analysis using modulus of rupture testing.

Building on the results of the small-scale program, the large-scale hybrid girder testing program sought to evaluate the shear capacity of the end-regions and continuity across the interface. These aspects were the primary concerns for the hybrid girder concept at the onset of this research. The means to address these concerns was accomplished through shear loading of half-scale AASHTO type II hybrid girders. The girders were subjected to a simply-supported, single-point load arrangement applied at one-third of the total span. The interface

between the UHPC and SCC was located at the midpoint of this shear span, between the load and the nearest support. Four of the top-performing interfaces from the small-scale program were carried over into this program, namely, the BiFluid (with and without dividers) and both variations of the Trapezoidal cold-joint textures (2D and 3D).

Findings from this research, along with conclusions and recommendations are given in this chapter.

7.2. Research Findings

7.2.1. *Findings from the Small-Scale Interface Testing Program*

The findings from the Small-Scale Interface Program revealed that the strength of each interface is dependent on the existing stress conditions along the interface and the texture's dimensional and geometric properties. Under direct shear loading, the interfacial strength was observed to be linked with bond type, amplitude, and texture. Interfaces with greater physical variation perpendicular to the interfacial plane exhibited higher strengths. This effect generated a zone parallel to the interface, characterized as a stiffness gradient transitioning between the two concrete sections, effectively causing the failure plane to shift into the weaker stratum. The 2D Trapezoidal results showed a 25% increase in shear strength over the Sine Wave results. However, interfacial strength was still contingent on the amount of material and its strength across the interface, especially when comparing the Exposed Aggregate interface to the 3D Trapezoidal. Texture variation in multiple directions also gave different strengths as indicated in the 20% increase in shear capacity from the 2D Trapezoidal texture to the 3D Trapezoidal texture. Improved performance also stemmed from chemical bonding at the interface, as with the BiFluid approaches. Among the three lowest amplitude textures (BiFluid with Divider, Sine Wave, and Smooth Interface), the BiFluid with Divider set resulted in significantly higher interfacial strengths.

Overall, the slant shear testing strengths were all relatively close and exhibited a nearly inverse ranking compared to pure shear loading. Higher amplitude 2D and 3D Trapezoidal textures were less effective than lower amplitude Sine Wave and BiFluid with Divider textures. It was theorized that this discrepancy may originate in the areas of each concrete at the median interface location. Analysis of the higher strength 2D Trapezoidal texture compared to the 3D Trapezoidal showed that the 2D Trapezoidal interface featured 5% more SCC crossing the

interface than the 3D Trapezoidal. Interestingly, the average strength of the 3D Trapezoidal texture was about 5% lower than that of the 2D Trapezoidal texture.

Flexural testing of the textures showed that the Sine Wave texture displayed the weakest strength, attributed to the lack of chemical bonding at the interface and a shallower texture amplitude. Both Trapezoidal textures outperformed the BiFluid textures, and all specimens yielded values less than or equal to the calculated modulus of rupture strength of the SCC, as per ACI 318 (2019).

Across all three testing methodologies, the failure planes were consistently located within the SCC. The BiFluid sets, both with and without Divider, exhibited failures indicative of homogeneous concrete with no interface. Conversely, the 2D and 3D Trapezoidal textures did not demonstrate this behavior, yet their strengths were comparable to those of the BiFluid methods. The two BiFluid and the two Trapezoidal interface textures exhibited the highest overall performance across the different testing methods of this program. The strengths of these four textures also outperformed most of the results reported in existing literature. This brings into question the validity of the interface testing results; however it is affirmed through comparing equivalent textures in this research with those in existing literature. The Exposed Aggregate interface in this study provided analogous shear strengths to the capacities obtained from other investigations. Further comparative analysis of the cold-joint textures in this study with textures with equivalent amplitudes from other studies showed shapes that facilitated a normal force directed into the opposing strata performed better than ones that did not. In fact, the reported 25% increase in shear strength due to shape by Yang et al. (2022) was confirmed in this texture analysis.

7.2.2. Findings from the Large-Scale Hybrid Girder Testing Program

The findings from the Large-Scale Hybrid Girder Program showed promising results for the plausibility of hybrid girder construction and load response. Existing studies reported that the time component of casting hybrid girders was not ideal for precasting plants. It was theorized that alternate interfaces, other than an exposed aggregate interface through water-jetting, could be implemented and still provide adequate shear-carrying capacity. This study found that cold-joint interfaces, although simpler to construct, still require additional time for sequential casting of the two concretes. On the other hand, BiFluid casting requires no additional time, but there is a greater level of complexity centric to casting two concretes

simultaneously. However, BiFluid casting is less difficult if both concretes possess a comparable rheology. Introducing a divider at the interface, as in the BiFluid with Divider specimens, can further assist the casting process by providing a safety margin for the addition rate of each concrete while still maintaining a distinct vertical interface.

Overall, the hybrid girders yielded relatively consistent results for maximum load, with an average failure load of approximately 130.67 kips and a coefficient of variation of 4.93%. Most of the crack propagation up to failure was confined to the reduced web section, and no failures originating along the interfaces. Crack development appeared to be more evenly distributed throughout the webs of the BiFluid and BiFluid with Divider sets. Whereas cracking in the 2D and 3D Trapezoidal hybrid girders appeared to be localized near the location of the first developed cracks within the section.

In general, all nine large-scale hybrid girder specimens failed in a very similar manner, with increasing formation and extension of diagonal web cracking followed by crushing of the web in the SCC portion of the shear span. The load-deflection plots of the hybrid girders showed relatively consistent stiffness, toughness, and failure loads. However, some variability was observed between different interface textures. BiFluid methods produced sudden, dramatic failures at peak loads, whereas the 2D texture exhibited a more gradual failure with slightly lower failure loads. Conversely, the girders with the 3D texture exhibited higher capacities with a sharp drop in load after a sudden, intense failure, similar to the BiFluid sets. Failure modes across all nine girders were remarkably consistent compared to the typical high degree of variability observed in shear testing of reinforced and prestressed concrete, indicating a very specific type of failure mode for hybrid girders. Based on the effective shear span-to-depth ratio, a/d , calculated as 4.0 for the hybrid girders, the observed web-crushing failure mode contradicts what the expected failure modes should be for this classification. According to literature, the predominant mode should be flexure-shear (diagonal tension) or compression-shear failure for members with thin webs.

Analysis of the hybrid girders was conducted using the empirical equations from ACI 318 (2019) and strut-and-tie modeling, both methods were recommended in existing literature for the end-region evaluation. The ACI Detailed method provided results with excellent agreement between the code prescribed capacities and the observed testing capacities, despite having a failure mechanism different to that which the code is based on. Comparing the

nominal capacity calculated by the ACI Detailed Method to the hybrid girder capacities gave slightly conservative results for both types of BiFluid interfaces, while marginally overestimating the capacity of the two Trapezoidal-textured interfaces. Conversely, the accuracy of the strut-and-tie modeling depended on which set of recommendations were followed with regard to strut dimensions and the effective concrete compressive strength. Capacity estimates varied from either overly conservative or to extremely accurate depending on what assumptions were implemented in the design. The strut-and-tie calculations revealed that the most accurate approach was given by taking the effective strut width equal to one-third the strut length and the full compressive strength of the SCC as the effective compressive strength. This method gave very minor overestimations of the capacity with respect to the tested capacities and will represent an upper bound shear capacity approach for hybrid girders.

7.3. Research Conclusions

7.3.1. *Conclusions from the Small-Scale Interface Testing Program*

The objective for the Small-Scale Interface Program was to develop an array of interface textures for hybrid girder implementation that enabled behavior resembling that of a homogenous member. From the research findings listed in Section 7.2.1, the following conclusions for UHPC-to-conventional concrete interfaces are presented:

- Increased interfacial shear strength can be achieved through chemical interaction of concretes, as demonstrated by both BiFluid methods discussed in Chapter 3. Failures in this interface type were displaced into the weaker concrete.
- Larger interfacial transition zones provide increased shear strengths, i.e., those with increased intermingling of concretes in the zone parallel to the interface for BiFluid interfaces or larger amplitude cold-joint interfaces.
- For cold-joint interfaces, higher strengths are attainable by designing UHPC protrusions with a sufficient amplitude and a shape that facilitates a normal force directed into the opposing strata, such as a trapezoidal shape or sine wave.
- Variation of a cold-joint interface texture in multiple directions enhances shear transfer and flexural capacity across the interface.

- The area of each concrete at the interface strongly influences shear-carrying capacity. When load is not applied directly at the location of the median interface of a cold-joint specimen, the shear capacity is limited to that of the corresponding area of the conventional concrete at the face of the UHPC texture projections. That is, the plane, parallel to the interface, with the least amount of conventional concrete controls, resulting in limited stress-carrying capacity and the creation of a shear plane in the SCC along the face of the UHPC protrusions.
- Enhanced shear transfer can be achieved through various interface design techniques. Although interface textures will continue to be optimized, the predominant constraint remains to be the strength of the conventional concrete. That is, when the interfacial strength exceeds the monolithic strength of the conventional concrete, failure occurs in the conventional concrete.

7.3.2. *Conclusions from the Large-Scale Hybrid Girder Testing Program*

The large-scale hybrid girder testing program sought to evaluate the shear capacity of the end-regions and continuity across the interface. Overall, the results from the hybrid girder testing were unexpectedly consistent despite the inherently variable nature of shear testing of concrete beams. The conclusions drawn from the findings of the large-scale hybrid girder testing program that were outlined in Section 7.2.2 are as follows:

- BiFluid with Divider interface systems permit increased control of the interface placement and limit the protruding sections of either concrete past the interface. These protrusions represent weak points in the interface and tend to guide the path of shear cracking to follow the extents of the weaker concrete.
- The load response of hybrid girders, with adequate interface texturing, closely resembles that of monolithic concrete girders up to the point of failure. The presence of the interface does not introduce any discontinuity if designed using recommended interface texturing.
- Chemical bonding at the interface via BiFluid casting as well as variability of a cold-joint interface texture in multiple directions, such as the 3D Trapezoidal texture, caused increased monolithic behavior compared to the 2D Trapezoidal texture.

- Interface selection for hybrid girders will affect the performance and capacity. The difference in crack propagation between the two BiFluid sets and two Trapezoidal sets is believed to be attributed to the natural discontinuity of cold-joint interfaces, allowing for an imperceptible amount of separation along the interface texture. BiFluid interfaces are bonded both physically and chemically, making cracking less susceptible to being guided by physical discontinuities between the concretes. BiFluid interfaces enable marginally higher shear capacity compared to the cold-joint interfaces, which is believed to be due to the interfacial stiffness gained from chemical bonding at the interface.
- The strut-and-tie modeling approach using an effective strut width equal to one-third the strut length with the full compressive strength of the conventional concrete as the effective compressive strength provides an accurate upper bound for hybrid girder end-region shear capacity.
- The failure mechanism for the end-regions of prestressed hybrid girders is shear failure by web crushing. This failure type is consistent in hybrid girders and can be accurately predicted using existing shear capacity calculation methods, such as the ACI 318 (2019) Detailed Method and strut-and-tie modeling.

7.4. Overall Conclusions

In assessment of the overall concept of hybrid girders, the concept emerges as a viable method for addressing common issues in traditional prestressed concrete girders. With the implementation of appropriate interface methods or textures, the performance of hybrid girders closely parallels that of monolithic members, with added advantages resulting from the presence of UHPC in the end-regions. In alignment with these considerations, hybrid girders do not exhibit discontinuities at the interface under the loading conditions featured in this research. In fact, failure does not occur at the interface; instead, it redirects further into the conventional concrete, highlighting that the weakest link is the weaker material, not the connection. There is no immediate need for an exclusive design approach for hybrid girders, as traditional web-shear design methods provide excellent capacity estimations. However, to prevent brittle failure due to web crushing, a limitation on the shear contribution of

conventional concrete is presented in Section 6.4 of the previous chapter. Furthermore, while UHPC is found to provide sufficient shear resistance despite having a reduced fiber quantity, it is still advisable to include vertical tie reinforcement between the longitudinal bars to facilitate horizontal shear transfer between the deck and girder in the UHPC end-regions.

This dissertation has presented two research programs that contribute new data and ideas to their respective bodies of knowledge. The simultaneous casting of UHPC and conventional concrete and the resulting interfacial strength properties were not found in existing literature. The BiFluid methods introduced in this research, along with the results of cold-joint interface textures, offer further insights into optimizing interfaces between different concretes. An array of interface techniques has been presented for potential implementation in hybrid girder design. Currently, no literature addresses the failure modes or limitations of hybrid girders; this research extends the study of the hybrid girder concept and provides data, theory, and design recommendations relating to this lack of information.

7.5. Research Recommendations

7.5.1. *Recommendations for Future Research*

Several questions related to the performance and construction of hybrid girders have been addressed in this study, but many remain unanswered, and new questions have arisen. All four of the BiFluid and Trapezoidal interface methods have proven sufficient for stress transfer across the interface of a hybrid girder. Numerous other cold-joint texture variations or alternate approaches to BiFluid casting may produce more effective or simpler solutions. As a result, more texture variations need to be evaluated in order to create a more optimized approach to interface design. Further evaluations should also be made to develop a model that estimates the shear capacity of a UHPC/conventional concrete interface. Only through the analysis of various options can there be an identification of the true controlling factors for interfacial strength.

Based on the findings of this program, the 3D Trapezoidal texture has emerged as the best performing cold-joint interface texture due to its variation of grooves in two directions along the interfacial plane. It is recommended that an evaluation be performed to assess the potential enhancement of this texture by increasing the overlap of the protruding sections of UHPC. Modification of this texture may result in more homogeneous behavior across the

interface. Two recommended potential methods for achieving this enhancement are by adjusting the locations of the protrusions on the texture grid or by rotating the current texture to orient the grid at a 45-degree angle.

It is recommended that continued research be conducted to explore additional variations of the Trapezoidal textures from the small-scale program, with the aim of also developing an equation for estimating shear transfer between UHPC and conventional concrete. The analysis and results of these textures suggest a correlation between the areas of each concrete crossing the interface and the corresponding shear capacity. Therefore, the proposed shear equation should build on existing shear friction models and be derived from the contribution of the area of each material at the interface.

Regarding hybrid girders and the overall concept, this study provides a foundational understanding, but further investigations should be conducted on hybrid girders with longer spans, girder sections with full web thicknesses, increased amounts of prestressing, higher UHPC fiber contents, and the evaluation of hybrid girders loaded at various distances from the interface. Additionally, exploring an alternate approach for a more effective BiFluid with Divider system is recommended. Under conditions where a divider system is a viable design implementation, the BiFluid with Divider interface is recommended for hybrid girder design. This recommendation is based on the increased control it offers over interface geometry during the casting process and its resemblance of load behavior to a homogenous member.

7.5.2. Recommendations for Hybrid Girder Design

Based on the insights presented in this research, it is recommended that the design process for hybrid girders should follow the approach specified below.

1) Sizing UHPC End Regions:

First, designers shall select the overall layout of the girder and extents of the UHPC. These sections should extend at least from the support to the edge of the discontinuity region, or beyond it. This extension provides reinforcement and strengthening that is critical to the hybrid girder concept.

2) *Design Conventional Concrete Segment of Hybrid Girder:*

The design of the conventional concrete segment of the girder should generally align with traditional design methods used for prestressed girders. However, there is an exception for the shear design of the conventional concrete for the end regions of the girder.

A lower bound of the nominal shear capacity of the conventional concrete segment for the end regions of the girder should be determined through the ACI Detailed Method, where web-shear failure will often control for members with reduced web thickness. However, the design method must also include an upper bound check based on the strut-and-tie model of the end-region at the location of the UHPC/conventional concrete interface. In this model, the strut strength would be based on the compressive strength of the conventional concrete.

3) *Upper Bound Shear Capacity Check for Conventional Concrete in the End Regions:*

The compressive strut should have vertical extents equivalent to the internal moment arm in the full section and be oriented parallel to and in-line with web-shear cracking, typically at a 25- to 35-degree angle. Initial calculations for the angle of cracking can be estimated but should also be verified by calculating the orientation of the principal tension stress at the level of the centroid using Mohr's Circle. Alternatively, AASHTO (2020) provides a calculation for the angle of cracking based on the MCFT. The crack angle (θ) is calculated using the net longitudinal strain in the tensile reinforcement (ϵ_s) as per Equation 6-4. Calculation of the net strain on the tensile reinforcement is given by Equation 6-5 in AASHTO LRFD Article 5.7.3.4.2.

$$\theta = 29 + 3500\epsilon_s \quad \text{Equation 6-4}$$

$$\epsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{E_sA_s + E_pA_{ps}} \quad \text{Equation 6-5}$$

where:

ϵ_s = net longitudinal stain in the tensile reinforcement

$|M_u|$ = absolute value of the factored moment at the crack location (kip-in.)

d_v = effective shear depth, taken as the length of the internal moment arm
between resultant tensile and compressive forces due to flexure (in.)

N_u = factored axial force (taken as negative value for compression) (kip)

V_u = factored shear force (kip)

V_p = shear force due to prestressing (taken as positive if resisting applied shear) (kip)

A_{ps} = area of prestressing steel in the tensile zone (in²)

f_{po} = effective prestress in the strands, taken as 70% of ultimate tensile strength of prestressing steel for “usual levels of prestressing” (ksi)

E_s = elastic modulus of mild steel reinforcement (ksi)

A_s = area of mild steel reinforcement in the tensile zone (in²)

E_{ps} = elastic modulus of prestressing steel (ksi)

The calculation of the strut's capacity shall be determined from the area of the mid-strut and the compressive strength of the conventional concrete. The area of the mid-strut can be computed using the member's web thickness and the effective strut width, which is taken as one-third of the total strut length. Equation 6-6 defines the proposed nominal shear capacity check for the conventional concrete portion near the end regions of a hybrid girder.

$$V_n \leq f'_c \left(t_w \cdot l_s / 3 \right) \sin (\theta) \quad \text{Equation 6-6}$$

where:

V_n = nominal shear capacity (kips)

f'_c = compressive strength of the mid-strut, taken as the full compressive strength of the conventional concrete (ksi)

t_w = thickness of the web section at the mid-strut location (in.)

l_s = length of the strut (determined by the distance between the internal compressive and tension forces and θ) (in.)

θ = angle of web-shear cracking (degrees)

This capacity check serves as an upper bound to mitigate web crushing behavior, a brittle failure type that is generally undesirable compared to yielding of reinforcement. It is important to note that this check pertains to nominal capacity, and for design purposes, load resistance and strength reduction factors must still be applied.

4) *UHPC Capacity Check:*

The UHPC ends should be verified for sufficient capacity against bearing and prestress release stresses. In cases where high stresses are present, it may be necessary to include reinforcement. However, the research conducted has indicated that the standard, minimum amount of vertical reinforcing is not required for shear in the girder due to the significant contribution of the fibers in the UHPC matrix. Shear capacity of the UHPC can be determined with conservative results by employing any of the three previously discussed approximations for UHPC shear capacity in Chapter 4.

5) *Interface Design Considerations:*

To satisfy the proposed design process, the interface between the UHPC and conventional concrete should be designed in a manner such that the controlling shear capacity is governed by the weaker conventional concrete, not the interface itself. Achieving this involves implementing the recommendations outlined in the results and analysis of the small-scale program listed in Chapter 5. Otherwise, shear capacity of the section should be based on the shear-friction model of the interface for textures with shear capacities less than the conventional concrete.

6) *Design Connection Between Deck and Girder:*

Horizontal shear capacity between the UHPC girder and the deck should be resolved by including vertical reinforcing elements, following conventional design methods. It is anticipated that only minimal reinforcement would be needed to satisfy horizontal shear requirements at these locations. Stirrups that connect the longitudinal reinforcement of the deck with the top bars of the girder are typically adequate for horizontal shear transfer. Therefore, full-depth vertical reinforcement in the UHPC is generally unnecessary if the shear resistance provided by the UHPC is sufficient for design.

Appendices

Appendix A

A.1 Concrete Mix Designs

Table A.1: Ultra-High-Performance Concrete Mix Designs⁹

Constituent Material	Amount Small-Scale Program	Amount Large-Scale Program
Cement, Type I (lbs/yd ³)	1180	1192
Slag Cement (lbs/yd ³)	590	596
Silica Fume (lbs/yd ³)	197	199
Masonry Sand (lbs/yd ³)	1966	1986
Fiber Amount in lbs/yd ³ (Percentage by Volume)	264.5 (2.0%)	132.3 (1.0%)
Water (lbs/yd ³) (W/C Ratio: 0.20)	393	397
High-Range Water Reducer (MasterGlenium 7920; oz/cwt)	26	24

⁹ The mix designs listed in this table are based on the non-proprietary UHPC, referred to as “J3”, that was previously developed at the University of Oklahoma.

Table A.2: Self-Consolidating Concrete Mix Design¹⁰

Constituent Material	Amount	Mix Design Notes
Cement, Type I (lbs/yd ³)	525	Total Combined Cementitious Material is 700 lbs/yd ³
Fly Ash, Class C (lbs/yd ³)	175	25% Replacement of Total Cementitious Material (by Mass)
Water (lbs/yd ³)	250	0.37 W/C Ratio
Fine Aggregate (lbs/yd ³)	1446	50% of Total Aggregate (by Volume)
Coarse Aggregate (lbs/yd ³)	1479	--
Air Entrainment (Master Builders AE-90; fl. oz/cwt)	0.15	Design Air Content: 6%
High-Range Water Reducer (MasterGlenium 7920; fl. oz/cwt)	8.50	Design Slump Flow: 22 ± 2 in.

¹⁰ The mix design listed in Table A.2 was selected to adhere to the “AA” structural concrete mix design guidelines issued by the Oklahoma Dept. of Transportation.

A.2 Interface Descriptions

BiFluid Interface

An interface method that enables both chemical and physical bonding with a fluid-like interlacing at the interface between concretes. Creation of this interface is through simultaneous casting of two concretes with a similar rheology. This method requires self-consolidating concretes with higher flowabilities to achieve enhanced intermixing at the interface. Each material is poured into the formwork such that the first contact between the concretes occurs at the interface. After initial contact, both concretes are continuously added such that the fill-rates are equal. Some variation of the lateral position of the interface will occur, especially with very flowable concretes. However, the extents of lateral variation of the interface should not exceed the thickness of the section and should be gradually changing, avoiding sharp protrusions of either concrete into the other. A visual representation of a standard BiFluid interface is illustrated in Figure A.1.

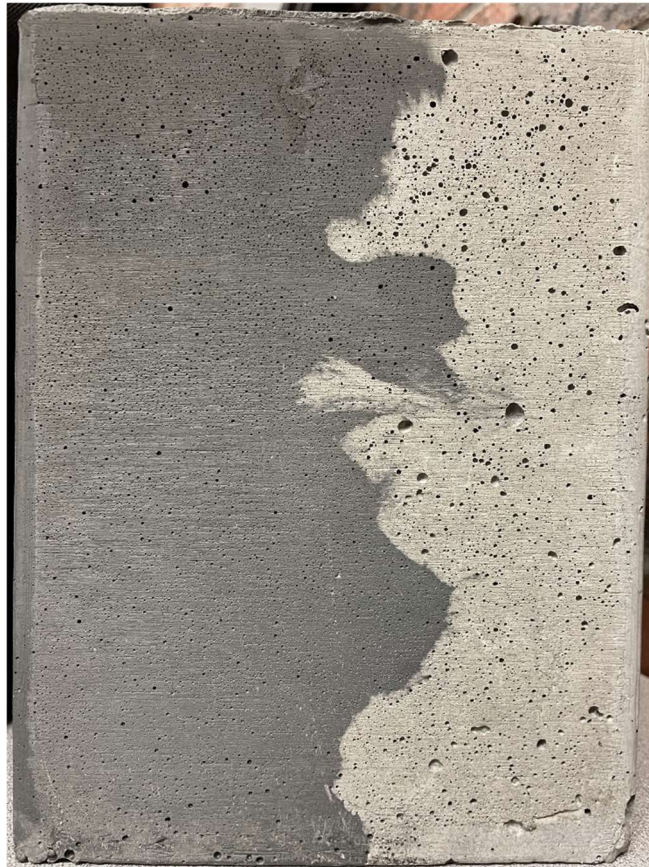


Figure A.1: BiFluid Interface Example

BiFluid with Divider Interface

An interface method that enables both chemical and physical bonding with a straight interface between concretes. Creation of this interface is through simultaneous casting of two concretes with a similar rheology. This method requires self-consolidating to achieve enhanced bonding at the interface. A divider is placed at the interface location to retain each concrete from protruding across the interface. Modifications of the divider are required to account for reinforcement crossing the interface. Each material is poured into the formwork such that the first contact of each concrete with the divider occurs at relatively the same time. The required timing for this action depends on the strength of the divider material and its modifications. After initial contact, both concretes are continuously added such that the fill-rates are similar. Again, the exact rates of addition for each concrete will depend on the divider to minimize the passing of concrete across the interface. A visual representation of a standard BiFluid with Divider interface is illustrated in Figure A.2.



Figure A.2: BiFluid with Divider Interface Example

Exposed Aggregate Interface

This commonly used interface method utilizes coarse aggregate as the primary mechanism for interface bond. In this study, this texture was used in small-scale testing where the Ultra-High-Performance Concrete (UHPC) was cast first. Consequently, coarse aggregates needed to be included in the UHPC section first. Achieving this was performed by partially depressing individual aggregates into crushable foam boards in a pattern that resembles the distribution of aggregates in conventional concrete. Figure A.3 illustrates the foam boards with aggregates used in the direct shear testing. The foam boards were aligned with the surface at the interface location such that the aggregates extended beyond the interface. Aggregates were kept moist during the UHPC casting procedure. After casting the UHPC, the specimens were cured for 24 hours before removing the foam with a commercial grade pressure washer, as in Figure A.4. The resulting UHPC sections with exposed coarse aggregates were aligned in the formwork and the Self-Consolidating Concrete (SCC) was cast subsequently.



Figure A.3: *Exposed Aggregates in Foam Boards*



Figure A.4: *Exposed Aggregate foam removal (left) and resulting texture pattern (right)*

Sine Wave Texture

The ACI 318 (2019) minimal requirement for an intentionally roughened interface texture with a 0.25-inch amplitude. The corresponding depiction in the code is a texture with a sinusoidal pattern. The Sine Wave texture used in this research is the same pattern with a 0.25-inch amplitude and 0.5-inch period. The texture is created by placing a form liner at the interface location. The Sine Wave form liner used in this study was obtained from *Spec Formliners, Inc.* and is manufactured from a plastic material intended for multiple uses. The form liner utilized for the Sine Wave texture is depicted in Figure A.5.



Figure A.5: *Sine Wave interface form liner*

Smooth Interface

The Smooth Interface is the baseline texture and establishes a standard metric for other textures. Concrete form boards, manufactured from plywood panels with hydrophobic surfaces, were used to create the texture surface. The resulting concrete surface has a smoothness comparable to glass. Figure A.6 below shows a typical interface outfitted with this texture, note the reflection on the diagonal surface.



Figure A.6: *Slant-Shear Specimen with a Smooth Interface texture*

2D Trapezoidal Texture

The 2D Trapezoidal texture was developed from the results of existing literature and the recommendations of the authors. This texture incorporates a grooved, two-dimensional texture with a pattern that is oriented with the fluted shape perpendicular to the resisting load. The pattern is a 0.75-inch amplitude, trapezoidal shape with 0.75- and 1.0-inch short- and long-side dimensions, respectively. Figure A.7 illustrates the pattern dimensions.

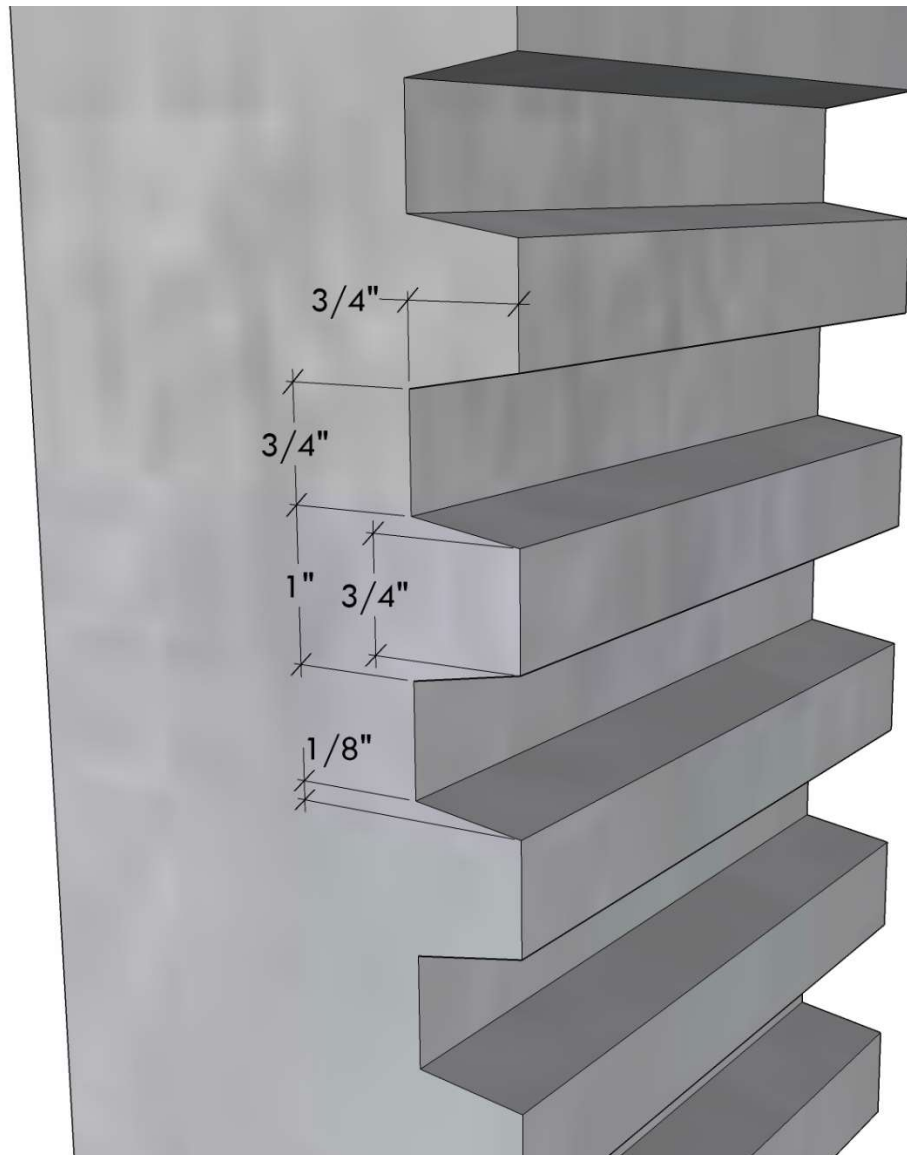


Figure A.7: 2D Trapezoidal texture dimensions

3D Trapezoidal Texture

The 3D Trapezoidal texture was developed from the results of existing literature, the recommendations of the authors, and is the multi-directional variant of the 2D Trapezoidal texture. This texture incorporates the same trapezoidal pattern as the 2D Trapezoidal texture with a repeating pattern in two directions along the interfacial plane. The orientation of the texture pattern is both parallel and perpendicular to the resisting load. The pattern incorporates the same 0.75-inch amplitude, trapezoidal shape with 0.75- and 1.0-inch short- and long-side dimensions, respectively. Figure A.7 illustrates the pattern dimensions.

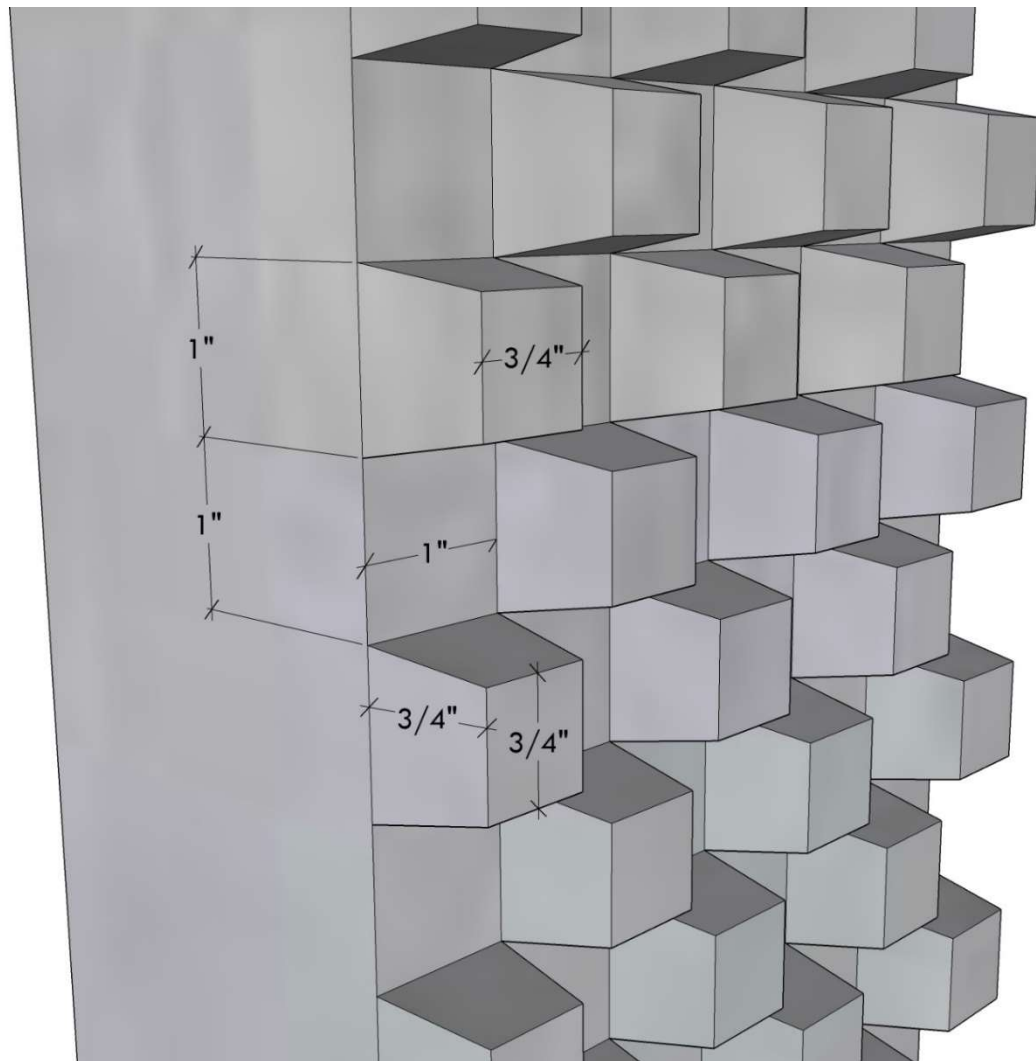


Figure A.8: 3D Trapezoidal texture dimension¹¹

¹¹ The side of the texture depicted in this illustration is intended to be the UHPC material.

A.3 Hybrid Girder Reinforcement Design

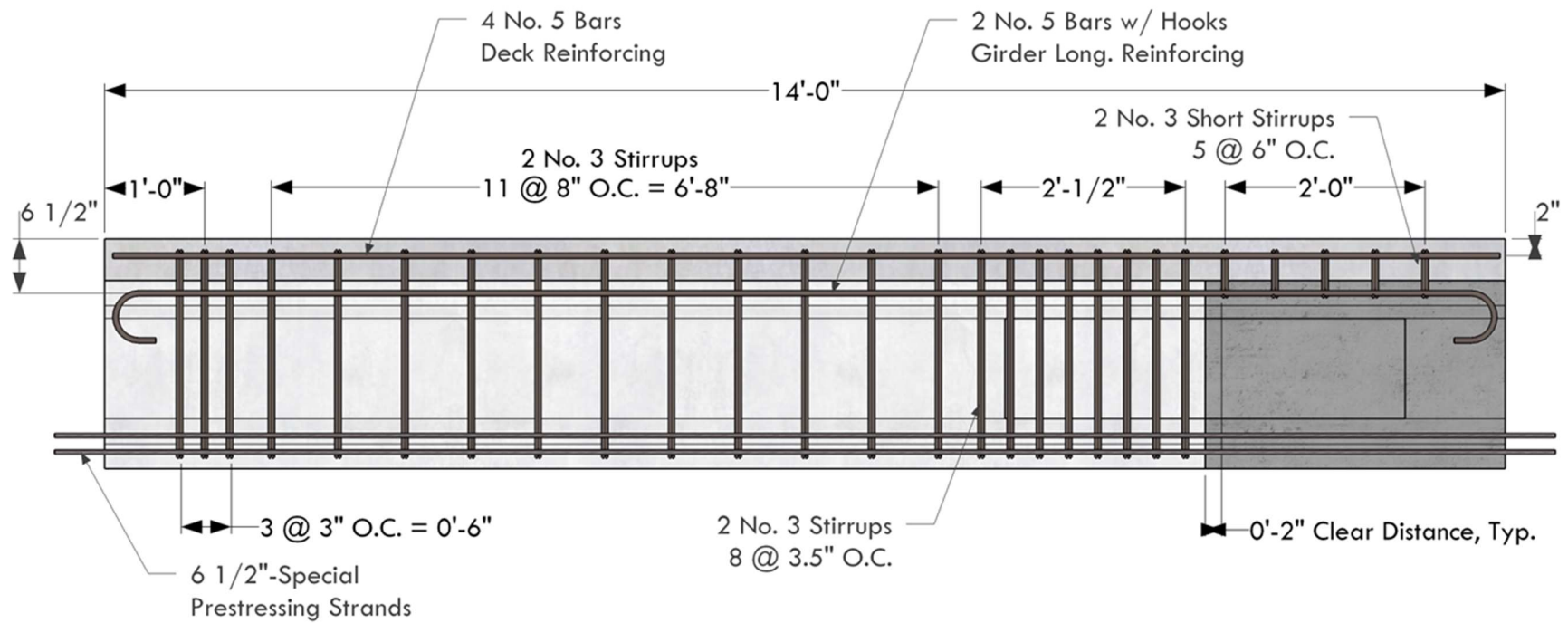


Figure A.9: Girder Reinforcement Diagram

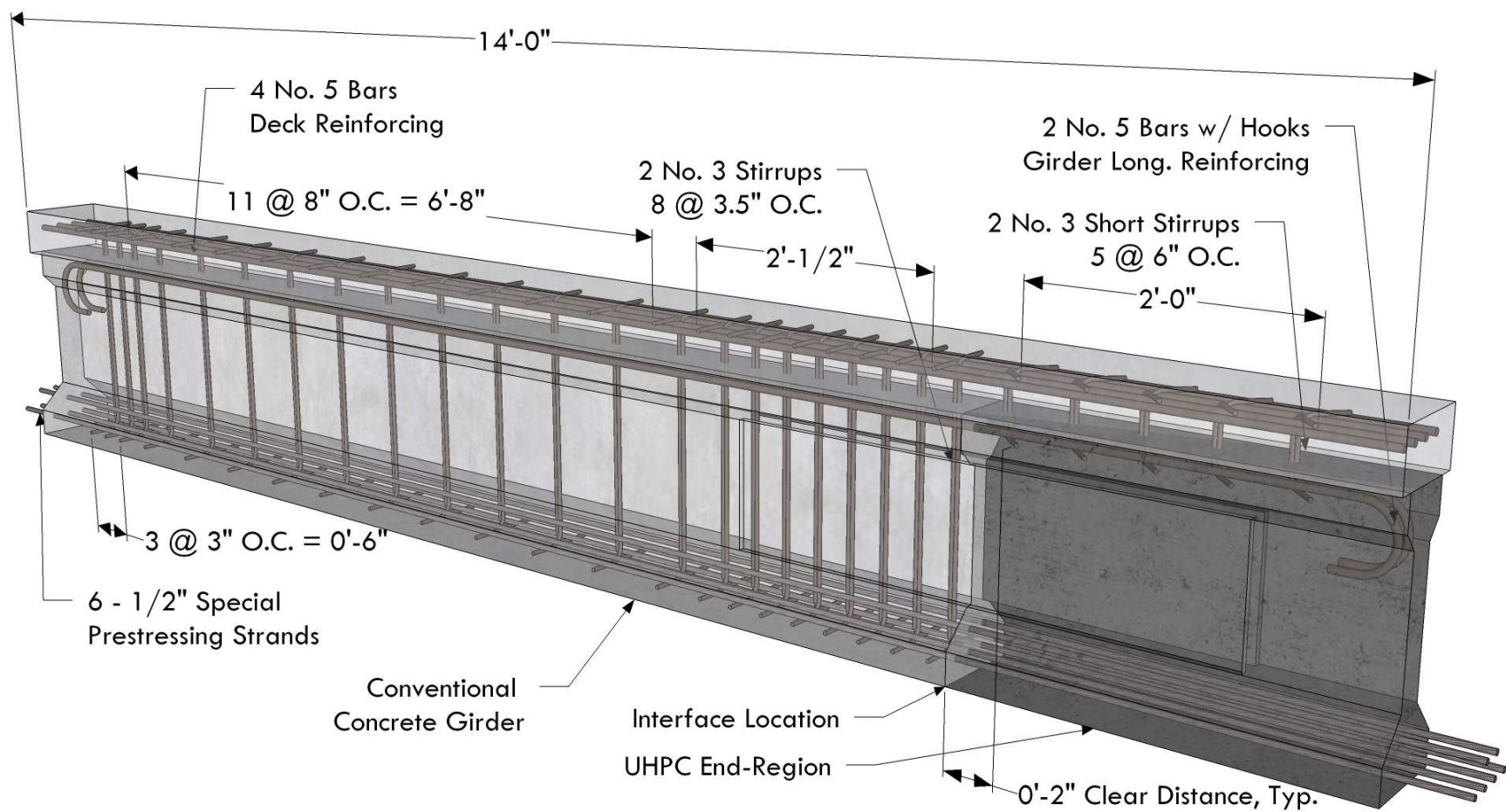


Figure A.10: Girder Reinforcement Illustration Model

A.4 Hybrid Girder Compressive Strengths & Camber Measurements

Table A.3: Hybrid Girder Test-Day Compressive Strength

Hybrid Girder	Material	Location	Test Age (days)	Average Compressive Strength (psi)	COV
BiFluid No. 1	UHPC	Girder	32	17,870	4.87%
	SCC	Girder	32	8,164	1.00%
	SCC	Deck	19	8,790	2.00%
BiFluid No. 2	UHPC	Girder	33	15,997	2.90%
	SCC	Girder	33	8,710	2.22%
	SCC	Deck	22	8,145	3.53%
BiFluid No. 3	UHPC	Girder	35	17,456	8.39%
	SCC	Girder	35	8,470	2.02%
	SCC	Deck	27	7,767	2.95%
BiFluid w/ Divider No. 1	UHPC	Girder	35	17,481	2.32%
	SCC	Girder	35	8,884	2.01%
	SCC	Deck	28	8,165	0.56%
BiFluid w/ Divider No. 2	UHPC	Girder	28	15,641	3.55%
	SCC	Girder	28	8,165	0.56%
	SCC	Deck	12	9,373	0.47%
2D Trapezoidal No. 1	UHPC	Girder	63	18,378	0.65%
	SCC	Girder	65	9,870	2.76%
	SCC	Deck	38	8,116	0.34%
2D Trapezoidal No. 2	UHPC	Girder	36	17,967	0.66%
	SCC	Girder	38	8,084	0.71%
	SCC	Deck	31	8,004	1.74%
3D Trapezoidal No. 1	UHPC	Girder	40	17,688	3.42%
	SCC	Girder	43	8,117	2.78%
	SCC	Deck	35	8,383	4.05%
3D Trapezoidal No. 2	UHPC	Girder	33	17,445	3.55%
	SCC	Girder	35	8,383	4.05%
	SCC	Deck	28	9,071	0.65%

Table A.4: Hybrid Girder Camber Measurements¹²

Location/ Material Arrangement Detail	Camber Measurements for Each Hybrid Girder at Testing (in.)								
	BiFluid No. 1	BiFluid No. 2	BiFluid No. 3	BiFluid w/ Divider No. 1	BiFluid w/ Divider No. 2	2D Trapezoidal No. 1	2D Trapezoidal No. 2	3D Trapezoidal No. 1	3D Trapezoidal No. 2
0 ft. UHPC	--	--	--	--	--	--	--	--	--
1 ft. Support	0	0	0	0	0	0	0	0	0
2 ft. UHPC	1/16	1/8	1/16	1/16	1/16	1/8	1/16	1/16	1/8
3 ft. Interface	1/8	5/16	1/8	1/8	1/8	1/4	1/8	1/8	3/16
4 ft. SCC	5/32	3/8	3/16	3/16	3/16	3/8	3/16	3/16	1/4
5 ft. Load Pt.	3/16	1/2	7/32	1/4	7/32	7/16	1/4	1/4	1/4
6 ft. SCC	7/32	9/16	1/4	1/4	1/4	1/2	5/16	1/4	5/16
7 ft. SCC	7/32	17/32	1/4	9/32	1/4	1/2	3/8	5/16	5/16
8 ft. SCC	1/4	5/8	9/32	5/16	9/32	1/2	7/16	3/8	11/32
9 ft. SCC	1/4	9/16	1/4	5/16	1/4	7/16	3/8	5/16	5/16
10 ft. SCC	3/16	7/16	3/16	7/32	3/16	3/8	1/4	1/4	1/4
11 ft. SCC	1/8	5/16	1/8	3/16	1/8	1/4	3/16	3/16	3/16
12 ft. SCC	1/16	3/16	1/16	1/16	1/32	1/8	1/16	1/8	1/8
13 ft. Support	0	0	0	0	0	0	0	0	0
14 ft. SCC	--	--	--	--	--	--	--	--	--

¹² Camber measurement values are formatted to show increasing camber shaded in red.

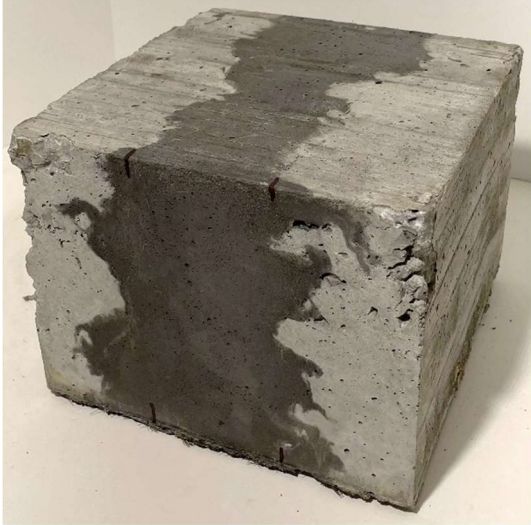
Appendix B

B.1 Direct Shear Specimen Photo Catalog

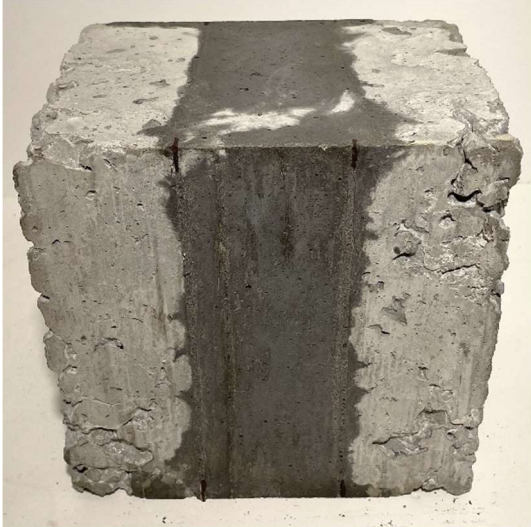
- BiFluid Interface
- BiFluid with Divider Interface
- Coarse Aggregate Interface
- Sine Wave Interface
- Smooth Interface
- 2D Trapezoidal Interface
- 3D Trapezoidal Interface

Note: Black markings demarcate the interface locations.

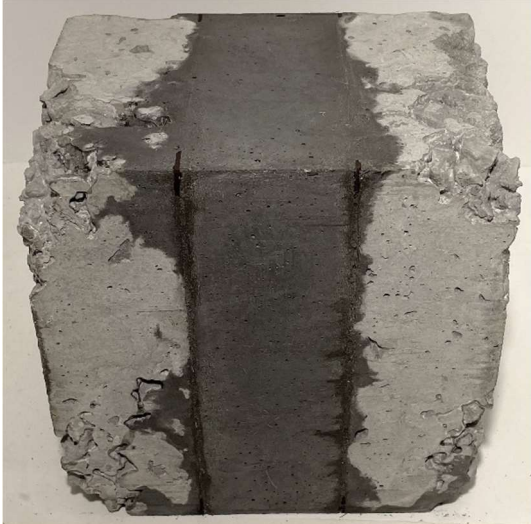
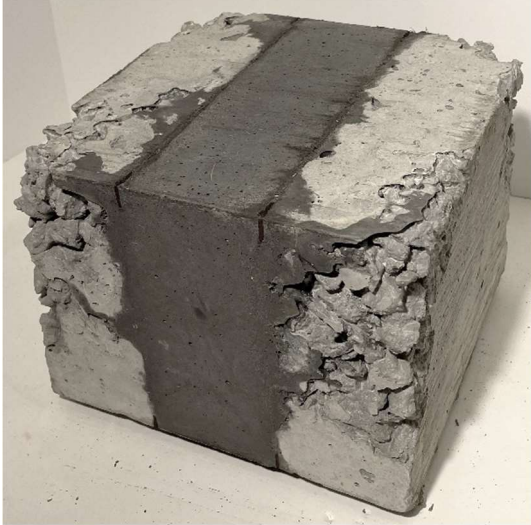
BiFluid Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

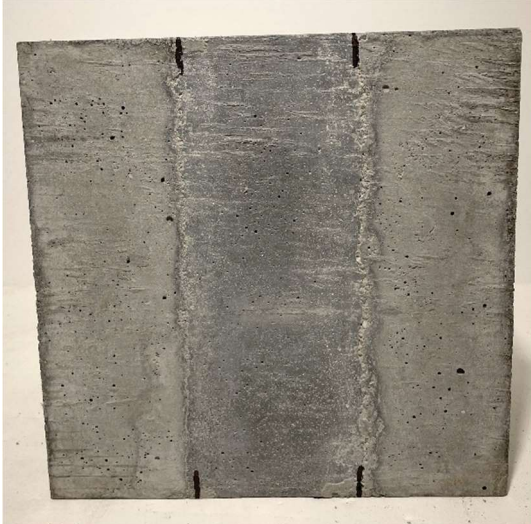
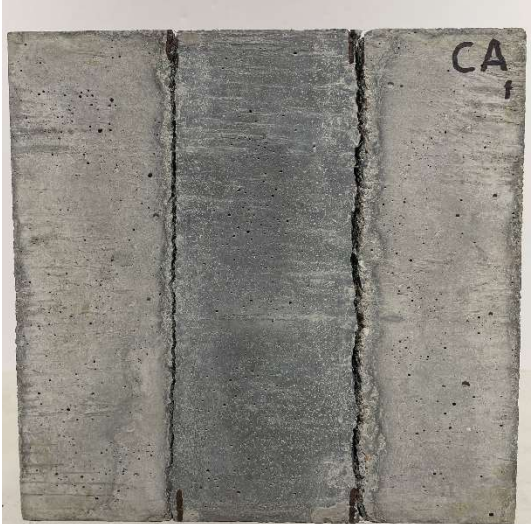
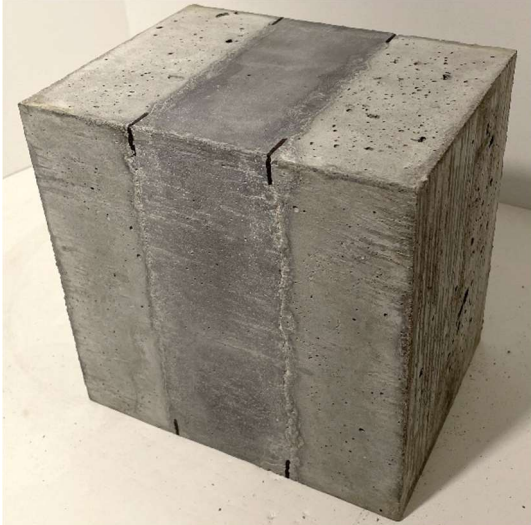
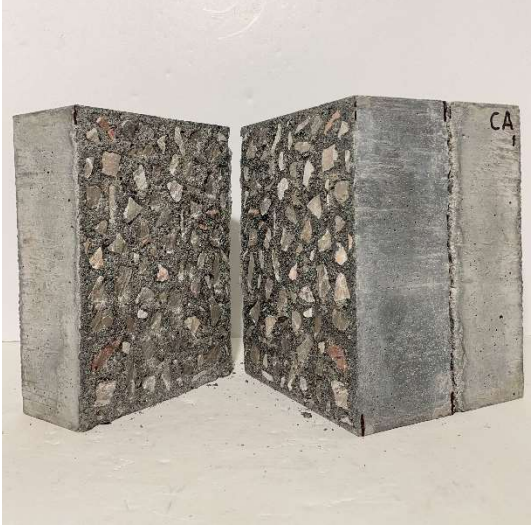
BiFluid Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

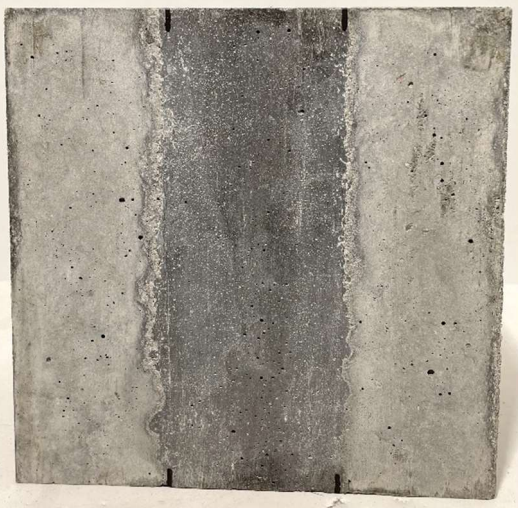
BiFluid Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

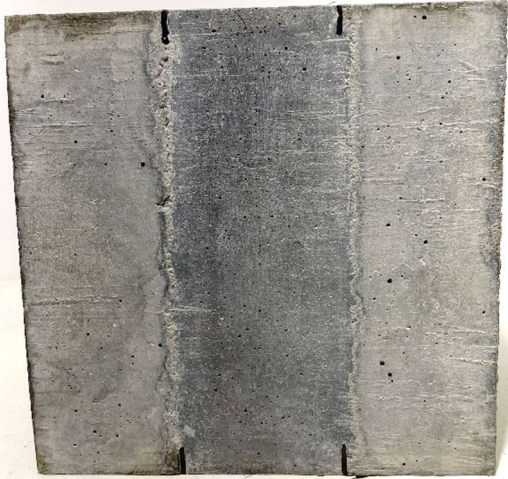
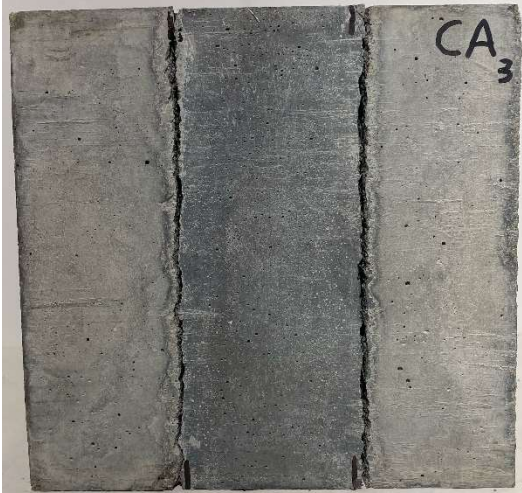
BiFluid w/ Divider Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

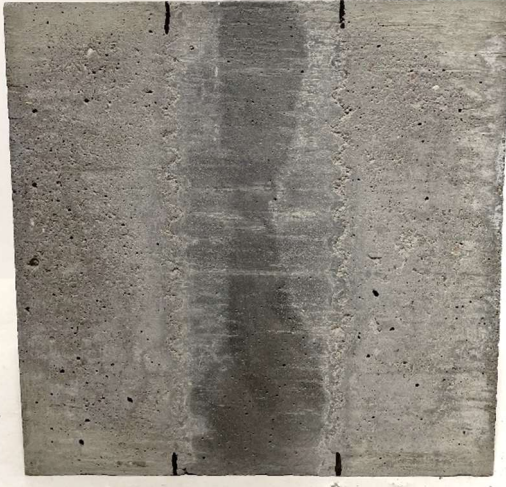
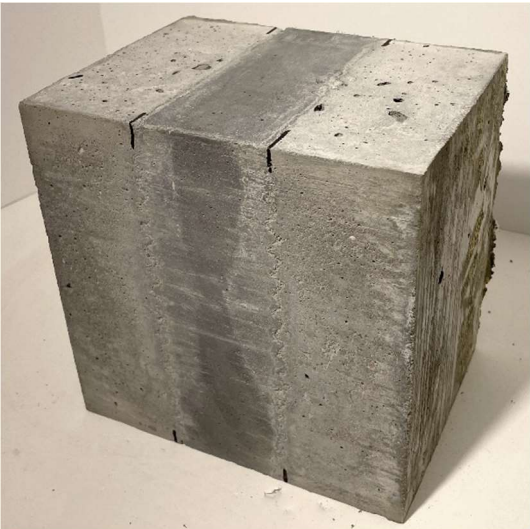
BiFluid w/ Divider Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

BiFluid w/ Divider Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

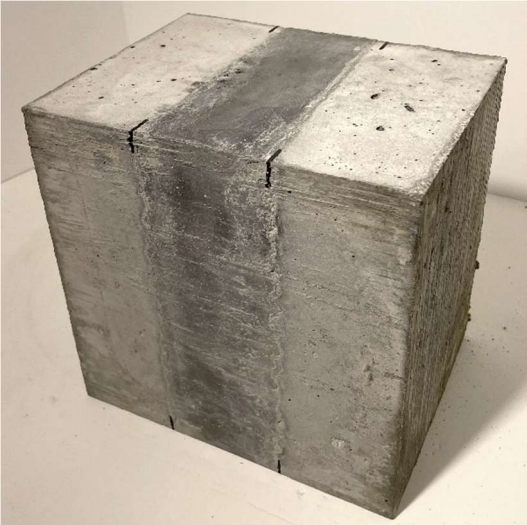
Coarse Aggregate Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

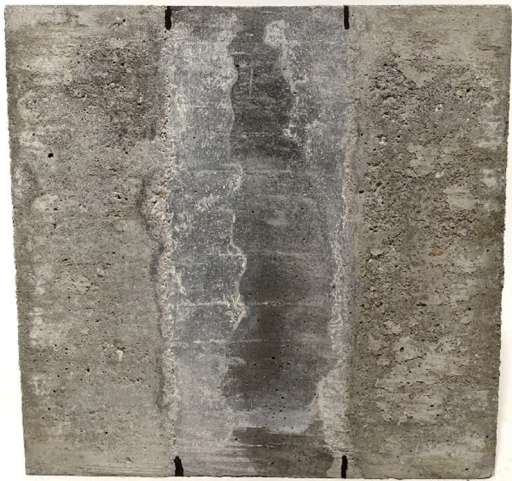
Coarse Aggregate Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

Coarse Aggregate Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

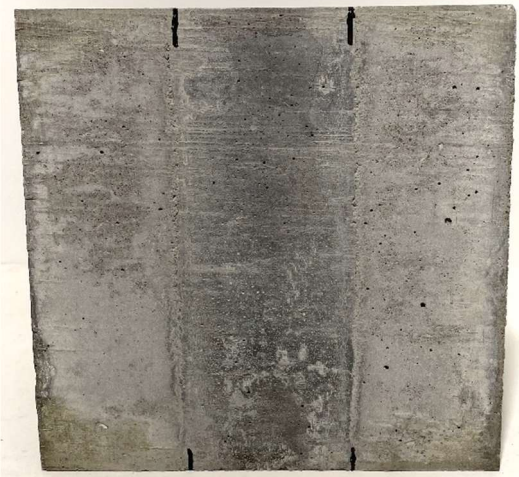
Sine Wave Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

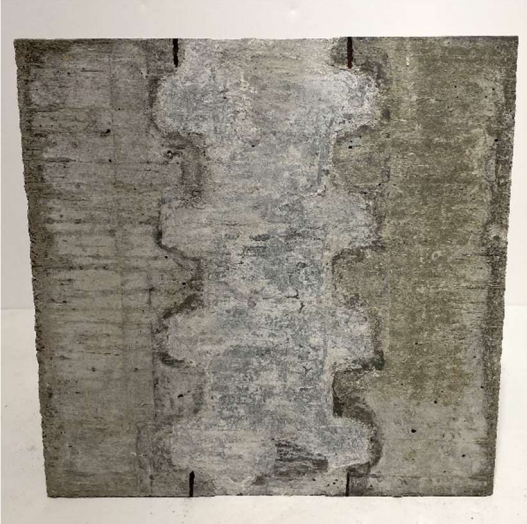
Sine Wave Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

Sine Wave Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

Smooth Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

Smooth Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

Smooth Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

2D Trapezoidal Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

2D Trapezoidal Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

2D Trapezoidal Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

3D Trapezoidal Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

3D Trapezoidal Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

3D Trapezoidal Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

B.2 Slant Shear Specimen Photo Catalog

- BiFluid Interface
- BiFluid with Divider Interface
- Coarse Aggregate Interface
- Sine Wave Interface
- Smooth Interface
- 2D Trapezoidal Interface
- 3D Trapezoidal Interface

BiFluid Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

BiFluid Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

BiFluid Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
 A vertical rectangular concrete specimen, labeled 'BF3' at the bottom right, showing a dark, irregular stain on its front face. The specimen is placed on a light-colored surface against a plain white background.	 The same concrete specimen, labeled 'BF3' at the top left, is shown after testing. It is positioned between two blue metal mesh panels. The specimen shows significant surface damage, including cracking and spalling, particularly on the right side.
 A left-side view of the concrete specimen, labeled 'BF3' at the bottom left. The dark stain is visible on the front face, and the top edge appears slightly irregular.	 A left-side view of the specimen after testing. The specimen shows extensive cracking and surface degradation, especially along the top and right edges.
 A right-side view of the concrete specimen, labeled 'BF3' at the bottom right. The dark stain is visible on the front face, and the specimen appears relatively intact.	 A right-side view of the specimen after testing. The specimen shows significant surface damage, including cracking and spalling, particularly on the right side.

BiFluid with Divider Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

BiFluid with Divider Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

BiFluid with Divider Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

Coarse Aggregate Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

Coarse Aggregate Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

Coarse Aggregate Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
 A vertical concrete specimen labeled 'CA3' at the bottom right. It shows a vertical crack running down the center and a diagonal crack near the top right corner. The surface is rough and weathered.	 The same specimen 'CA3' after testing, showing extensive cracking and fragmentation. The concrete is broken into several pieces, with a large diagonal crack and a vertical crack. The specimen is placed against a blue background.
 A vertical concrete specimen labeled 'CA3' at the bottom right. It shows a vertical crack running down the center and a diagonal crack near the top right corner. The surface is rough and weathered.	 The same specimen 'CA3' after testing, showing extensive cracking and fragmentation. The concrete is broken into several pieces, with a large diagonal crack and a vertical crack. The specimen is placed against a blue background.
 A vertical concrete specimen labeled 'CA3' at the bottom right. It shows a vertical crack running down the center and a diagonal crack near the top right corner. The surface is rough and weathered.	 The same specimen 'CA3' after testing, showing extensive cracking and fragmentation. The concrete is broken into several pieces, with a large diagonal crack and a vertical crack. The specimen is placed against a blue background.

Sine Wave Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

Sine Wave Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

Sine Wave Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

Smooth Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

Smooth Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

Smooth Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

2D Trapezoidal Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
 A vertical rectangular concrete specimen with a rough, weathered surface. The bottom right corner has the number '201' printed on it.	 The same specimen after testing, showing a prominent diagonal crack running from the top right towards the bottom left. The number '201' is visible at the bottom right.
 A side view of the specimen, showing its rectangular shape and rough texture. The number '201' is at the bottom left.	 The same side view after testing, with a visible crack along the top edge and down the side. The number '201' is at the bottom left.
 A side view of the specimen, showing its rectangular shape and rough texture. The number '201' is at the bottom right.	 The same side view after testing, showing a crack along the bottom edge. The number '201' is at the bottom right.

2D Trapezoidal Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

2D Trapezoidal Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	
	
	

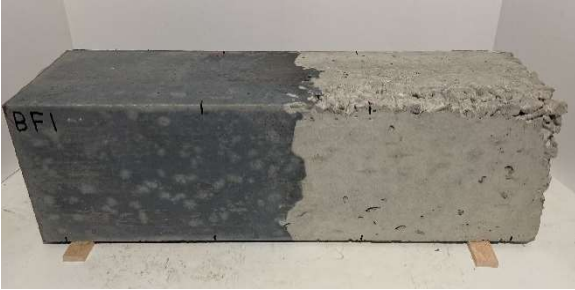
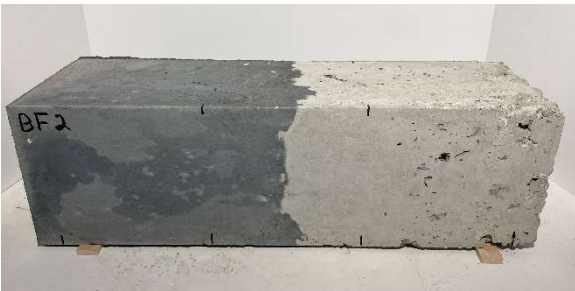
3D Trapezoidal Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
	
	

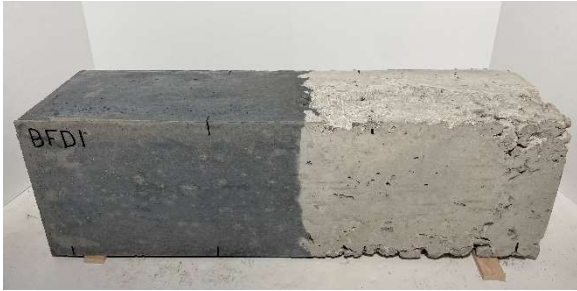
3D Trapezoidal Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
	
	

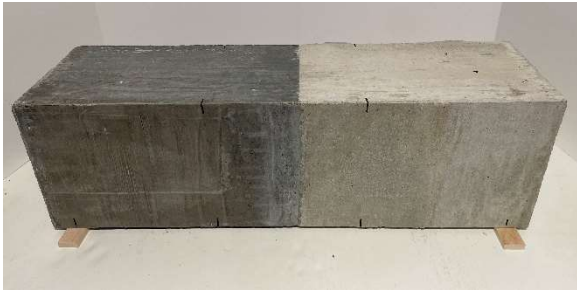
3D Trapezoidal Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
 A vertical rectangular concrete specimen with a dark, textured surface. A diagonal crack is visible on the left side. The label "3D3" is at the bottom right.	 The same specimen after testing, showing significant diagonal cracking and spalling along the left edge. The label "3D3" is at the bottom right.
 A side view of the specimen showing its trapezoidal shape. The label "3D3" is at the bottom left.	 The same side view after testing, showing extensive cracking and material loss at the base and along the side. The label "3D3" is at the bottom right.
 A side view of the specimen from the opposite side. The label "3D3" is at the bottom right.	 The specimen is shown in two pieces after testing, with significant fragmentation and exposed aggregate. The label "3D3" is at the bottom right.

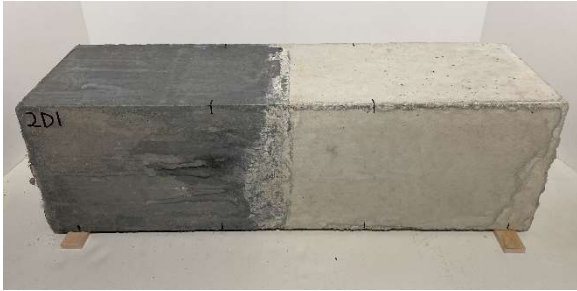
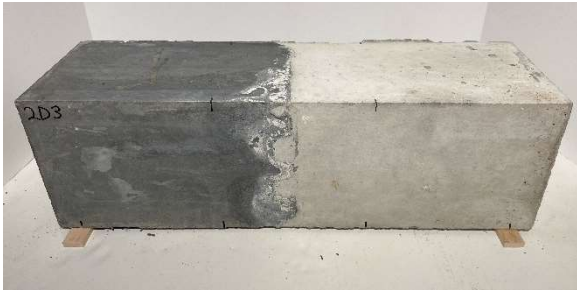
B.3 *Modulus of Rupture Photo Catalog*

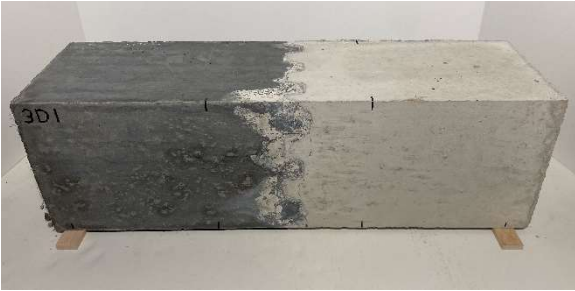
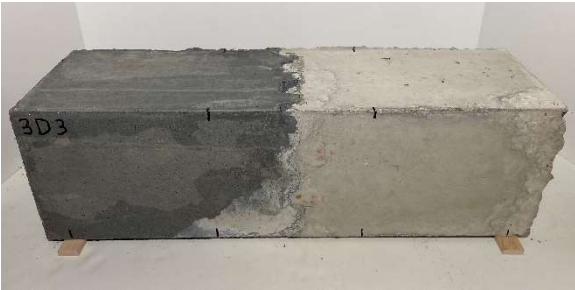
- BiFluid Interface
- BiFluid with Divider Interface
- Sine Wave Interface
- 2D Trapezoidal Interface
- 3D Trapezoidal Interface

BiFluid Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
BiFluid Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
BiFluid Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	

BiFluid with Divider Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
BiFluid with Divider Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
BiFluid with Divider Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	

Sine Wave Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
Sine Wave Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
Sine Wave Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	

2D Trapezoidal Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
2D Trapezoidal Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
2D Trapezoidal Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	

3D Trapezoidal Interface	Specimen No.1
Pre-Testing Photos	Post-Testing Photos
	
3D Trapezoidal Interface	Specimen No.2
Pre-Testing Photos	Post-Testing Photos
	
3D Trapezoidal Interface	Specimen No.3
Pre-Testing Photos	Post-Testing Photos
	

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