ABSTRACT

WILLIS, MEADE HANES. Experimental and Analytical Investigation of an Innovative Composite Shallow Floor Framing System. (Under the direction of Dr. Emmett Sumner).

The conventional composite floor system comprised of a concrete slab with light gauge steel deck form supported by wide flange structural steel girders has been widely used in construction in the United States since the early 1950’s. In recent years, modifications to the conventional system have been made to meet a market need for floor systems with reduced structural depth to be used in a variety of building types including office buildings, hotels, and hospitals. In 2007, Diversakore® LLC developed the Versa :T:™ beam, an innovative shallow floor composite framing system that utilizes a small structural depth and also makes improvements in the ease and speed of construction. The system includes a u-shaped steel plate which supports hollow core plank flooring during construction and serves as a stay-in-place form for a cast-in-place reinforced concrete girder. The concrete girder is cast monolithically with a topping slab to engage the hollow core planks and the u-shaped steel section into a composite t-beam. A research program to evaluate the performance of this innovative floor system has been developed and is currently ongoing at the Constructed Facilities Laboratory at North Carolina State University in Raleigh, NC. The experimental and analytical program includes full-scale tests of representative sub-assemblages and utilizes a layered sectional analysis to predict the behavior. The results of the analytical model and the experimental investigation are presented along with conclusions drawn from the initial phase of the research program.
Experimental and Analytical Investigation of an Innovative Composite Shallow Floor Framing System

by
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A thesis submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering

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Biography

Meade Hanes Willis, son of Robert and Katherine Willis, was born March 4th, 1985 in Boone, NC. Meade has two younger siblings, sister Mary Katherine and brother Robert Rutherford. He attended Mountain Pathways Montessori School until 3rd grade when he moved up to Parkway Elementary school to finish off his primary school education. At age 13 he decided to attend The McCallie School in Chattanooga, TN where his lifelong passion for learning began to mature. In 2003 he enrolled at North Carolina State University to study Civil Engineering with a concentration in structures. During the summer of 2005, Meade took an internship with the national architectural and engineering design firm HDR, Inc. in their Raleigh, NC office. He continued to work with HDR until completing his undergraduate degree in 2007. He decided to continue his education at North Carolina State University by attending graduate school to study Structural Engineering and Mechanics to obtain his Master’s of Science in Civil Engineering. Upon completion of his master’s degree in 2009, Meade plans to move to Greensboro, NC to begin working in industry as a structural engineer where he will continue his education in hopes of obtaining a professional engineering license.
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Chapter 1

Introduction

1.1 Background

Composite steel and concrete construction is widely used throughout the world in all types of structural elements including composite floor systems. As the uses and demands of structures have changed over the years, the composite structural elements used in their construction have changed to meet those needs. For example, as land prices rise and available space for construction becomes more limited, structures in cities have been pushed to make more efficient use of the space they have available. The development of composite floor systems with smaller structural depth was created mostly in response to the changing needs of such multistory urban buildings including office buildings and hotels. Low floor to floor height composite systems are designed to meet the same strength and serviceability requirements as conventional composite construction with a reduction in structural depth. The reduction in structural depth allows for more efficient use of space in a building of the same height when compared to conventional methods. Often times the innovative systems can also make improvements in the speed, ease, and safety of construction when compared to the conventional system comprised of a concrete slab with steel decking supported by wide flange girders.
The Versa :T:™ beam system developed by Divesakore® LLC is an innovative composite shallow floor framing system design to compete with conventional composite floor construction. The system is built using a steel U-section, precast hollow core slabs, mild steel reinforcing, and cast-in-place concrete. As shown in Figure 1-1 a u-shaped steel plate supports hollow core plank flooring during construction and serves as a stay-in-place form for a cast-in-place reinforced concrete girder. The concrete girder is cast monolithically with a topping slab to engage the hollow core planks and the u-shaped steel section into a composite t-beam.

Figure 1-1 – Diversakore® Shallow Floor Framing System
The construction of Diversakore’s shallow floor framing system is expedited in comparison to conventional composite floor construction. The bent plate steel U-sections arrive on site and are hoisted into position spanning between columns. Shear studs are installed on the bottom flange of the steel U-section and reinforcing cages are tied and dropped into the steel girders as well. Once the beams are in place, precast hollow core slabs are placed perpendicular to the girders creating the floor system as shown in Figure 1-2. The hollow core slabs and steel U-section create the formwork for a reinforced concrete girder. Welded wire mesh is placed on top of the precast hollow core slabs as reinforcement for the cast-in-place concrete topping slab that is cast simultaneously with the reinforced concrete girders. Once the system is set in place, an entire floor can be cast in a single pour, creating a monolithic frame.

![Figure 1-2 – Placement of Hollow Core slabs](image-url)
The innovative shallow floor framing system developed by Diversakore® LLC has many advantages over conventional composite construction. The steel U-sections and precast hollow core slabs arrive on site ready to be hoisted into place. Because the pieces can be lifted straight from the truck to their desired locations, the innovative process reduces the need for staging areas on the jobsite and can accommodate the demands of urban construction where space is limited. The offsite construction of the precast floor panels allows for greater quality assurance and less need for onsite supervision and quality control. The precast hollow core slabs also remove the need for setting up and taking down formwork and shoring that is necessary in conventional composite floor construction. Once the precast floor is set in place, it creates a safe working platform for the remainder of the construction process not typically present using conventional methods. The result is a system that not only has a smaller structural depth than traditional composite floor framing systems, but is faster, easier, and safer while meeting the same strength and serviceability requirements.

1.2 Objectives

The research program designed to investigate the composite shallow floor framing system includes three primary objectives to better understand the behavior of the innovative assembly and to validate its predicted behavior.
1. Validate and evaluate the current procedures used to design the composite shallow floor framing system for strength and serviceability requirements.

2. Evaluate the effect of the presence of shear studs on the composite action and general behavior of representative composite t-beam specimens tested in the laboratory.

3. Make recommendations for changes in the construction of the assembly and current design procedures to produce a simply yet comprehensive design method for the innovative composite shallow floor framing system.

1.3 Scope of Thesis

The design methodology and behavior of the innovative shallow floor framing system was investigated in two phases of the research program. In the first phase of the program, the existing design procedures were evaluated by comparing the results of each limit state calculation with behavior predicted by an analytical model. The analytical model was based on a layered sectional analysis of the representative composite t-beam used in the experimental investigation of the system. The analytical model was used to validate the existing design procedures as well as give a more detailed prediction of the behavior of the beams during testing.
The second phase of the program was an experimental investigation of the innovative composite shallow floor framing system where two representative composite t-beams were constructed and tested in the laboratory. The results of the experimental investigation were used to validate the predicted behavior from the analytical model, identify and evaluate additional failure mechanisms not predicted by the existing design procedures, and begin to investigate the effects of composite interaction on the system. The first specimen was designed with shear studs to represent full composite interaction while the second test specimen was constructed without shear studs, relying on the interaction generated from the natural bond formed between the cast-in-place concrete girder and steel U-section during curing. The results from the analytical and experimental investigations were used to evaluate the pertinent limit states and failure mechanisms to make recommendations for construction of the innovative composite shallow floor framing system as well as to generate a comprehensive procedure for its design.

1.4 Organization of Document

This thesis document is organized into six chapters including this introductory chapter. Brief descriptions of each chapter are presented in this section.
Chapter 2 presents a short history of composite construction and summarizes a review of relevant literature corresponding to behavior of composite construction used in buildings.

Chapter 3 describes the analytical investigation, which evaluates the current design procedures and the calculations of the predicted relevant limit states by comparing them to an analytical model developed from a layered sectional analysis. Preliminary results from the analytical investigation are also presented in this chapter.

Chapter 4 describes the experimental investigation including the design and construction of the test specimens, the test setup, instrumentation, test program, and measured properties of the materials used in the test specimens. Some general results from the experimental investigation are also presented in this chapter.

Chapter 5 presents the comparison of the results from the analytical and experimental investigations as well as some discussion about the validation of the analytical model and explanation of any deviations from the predicted behavior.

Chapter 6 summarizes the observations from the research program, presents recommendations for the design and construction of the innovative composite shallow floor framing system, and describes conclusions drawn from the analytical and experimental investigations. Suggestions for future research are also presented in this chapter.
In addition to the chapters described above, two appendices are included to provide additional details about the existing design procedures for the innovative shallow floor framing system as well as more detailed results from the analytical and experimental investigations.
Chapter 2

Background

2.1 A Brief History of Composite Beams and Floor Systems

Composite construction is an integral part of the majority of modern structures and is used worldwide in the construction of columns, beams, floor systems, and many other structural elements. Engineers have blended the relative strengths of the two most prominent structural materials, the compressive strength of concrete and the tensile strength of steel, to produce highly efficient and effective structural members. The first cases of the advantages of steel and concrete acting compositely were recognized in the late 1920’s due to the observation of increased strength in concrete encased steel beams. This led the New York City Building code to allow for an increased stress limit on concrete encased beams based on a series of full-scale tests conducted by F.A. Randall. (Viest et al 1997) Although the increase in strength was minimal by present day standards, it was recognition of the advantages of composite action nonetheless. Over the next ten years many more codes began to adopt the idea of composite action, including the American Institute of Steel Construction, but continued to permit only a conservative increase in strength. Part of the reason for not embracing the principal of composite action, was the observed inability to sustain the adhesive bond between steel and concrete
in the field for long periods of time. The need for a manner in which to ensure interaction between the steel and concrete is what opened the door for the development of mechanical shear connectors. One of the first systematic extensive studies on mechanical shear connectors was conducted by R. A. Caughey in 1929. Various styles of connectors were tested including flat bar, channel brackets, and angle brackets. The later was recommended by Caughey as the most effective but was shortly replaced by spiral shear connectors a few years later. However, its introduction in the 1950’s, the shear stud quickly became the most widely used mechanical shear connector, and continues to be the standard practice for modern construction.

As adequate shear connectors were pushing the development of effective steel-concrete composite beams, the recognition of composite action in floor systems was just being recognized. Prior to 1950, concrete was used exclusively as a filling material with no structural contribution to the overall strength. The many advantages of using conventional reinforced concrete slabs over steel decking to create composite floor systems were quickly recognized by early engineers. Not only does the decking provide a sturdy and safe platform from which to work during construction stages, but when it acts compositely with the concrete slab, the necessary amount of positive moment steel reinforcement can be significantly reduced. Sometimes the only necessary reinforcement is the steel mesh used to control temperature and shrinkage effects in the slab. Because the decking acts as stay-in-place formwork, floors can be constructed much more quickly.
and inexpensively due to the elimination of building and removing wood formwork and shoring. The use of cellular decking allows for electrical and mechanical ducts to be placed within the height of the floor inside the deck cells. These advantages are some of the reasons composite floor systems are so widespread in present day building construction. The first use of a steel deck floor system, known as a “keystone beam”, was produced by the H. H. Robertson Company in 1938. The system was used to construct multistory industrial buildings. However it was not used as a composite system. It was not until 1950 that the “Cofar” composite reinforced concrete and steel deck floor system was first produced by the Granco Steel Company. The advantages of the “Cofar” beam with its high strength steel deck and T-wire reinforcement welded to the top of the deck cells were studied and described by Friberg. Friberg’s work was the first significant study on composite slabs and provided a detailed cost comparison between the new steel deck reinforced concrete slabs and the conventional concrete slabs that were standard practice. By 1967 the American Iron and Steel Institute decided to fund a research project to develop a method of design for these composite floor systems. Similar to composite construction for beams, a means of creating interaction between the steel and concrete beyond its adhesive bond was necessary. This issue was first addressed by forming indentations and embossments in the metal decking. Push out tests would be used to determine the effectiveness of these anchors. The correlation between push out tests results and actual beam behavior was first discussed significantly in 1967
by Robinson. Soon manufacturers were designing new deck geometries to ensure full interaction between the decking and the concrete slab.

Deck geometries were not the only area of composite construction that was being pushed into innovation. In fact, increasingly specialized uses for composite construction were emerging and growing every day, pushing the need for innovative changes to conventional composite construction on almost every front. One such expanding arena was the need for composite beams with a smaller structural depth to meet the increasing demand for more efficient use of available space. The number of products boasting a reduced structural depth on the market increased rapidly coining the common descriptions of slim floor beam, shallow floor framing, low floor to floor height structural systems, etc. Competing designs began to build on the advantages of the shallow floor design concept by improving the safety, constructability and performance of the innovative floor framing systems.

To this day, many companies and engineers are creating new ways to take advantage of composite steel and concrete assemblies to increase speed and ease of construction and further benefit from the relative strengths of concrete and steel with new and innovative designs.
2.2 Horizontal Shear Demand

The magnitude of the horizontal shear demand across the interface of a composite concrete and steel beam can be calculated using one of three general methods described below. The size, number, and spacing of mechanical shear connectors for a composite beam can then be designed from the resulting computation. The three general methods include the Global Force Equilibrium Method, the Simplified Elastic Beam Behavior Method, and Classical Elastic Method. All three of the general methods are permitted to be used to calculate the horizontal shear demand in a composite beam by AASHTO LRFD Bridge Design Specifications. Variations of the equations are also included in the design guidelines presented in the AISC Steel Design Manual and similar specifications for composite design. For the sake of simplicity, each of the three methods will be explained using the example of a simply supported composite beam with a uniformly applied load as shown in Figure 2-1 and Figure 2-2.

2.2.1 Global Force Equilibrium Method

The Global Force Equilibrium Method resolves the horizontal shear demand from the change in compression (or tension) forces that act along the composite interface at two points along the beam due to the applied load. A section of the beam is taken along the length and the resulting shears and moments are computed as shown in Figure 2-1.
The shear force across the interface \((V_h)\) is taken as the difference in compression forces \(C_1\) and \(C_2\) as shown in Equation 2-1.

\[
V_h = (C_1 - C_2)
\]

Equation 2-1

Where:

\(V_h = \) horizontal shear force between points 1 and 2

\(C_1 = \) compressive force in concrete slab at point 1

\(C_2 = \) compressive force in concrete slab at point 2

The horizontal shear stress is then calculated by dividing the change in compressive forces by the area over which the difference is transferred as shown in Equation 2-2.

\[
\nu_h = \frac{(C_1 - C_2)}{(l^*b_0)}
\]

Equation 2-2

Where:

\(\nu_h = \) horizontal shear stress along composite interface between points 1 and 2
1 = length of interface between points 1 and 2

\(b_v\) = width of composite interface

2.2.2 Simplified Elastic Beam Behavior Method

The simplified elastic beam behavior method equates the vertical shear acting on the section to the horizontal shear demand using flexural beam theory. A relationship between the vertical shear acting on the section to the horizontal shear stress that the applied load induces is determined by applying force equilibrium over a small segment of

Figure 2-1 – Global Force Equilibrium Method
the beam as shown in Figure 2-2. From static force equilibrium applied to the section shown in the figure the following equations can be determined.

Summing moments about the vertical face at point two, Equation 2-3 and Equation 2-4 are derived.

\[ \Delta M = V \Delta x \]  
Equation 2-3

\[ \Delta M = d_e \Delta C \]  
Equation 2-4

Summing forces in the horizontal direction in the concrete slab Equation 2-5 is derived.

\[ \Delta C = V_h \]  
Equation 2-5

\[ V_h = \frac{V \Delta x}{d_e} \]  
Equation 2-6

\[ v_h = \frac{V_h}{b_v d_e} \]  
Equation 2-7

Substituting the previous equations into each other and solving for the horizontal shear stress along the composite interface between points 1 and 2 Equation 2-8 is derived.

\[ v_h = \frac{V}{b_v d_e} \]  
Equation 2-8
Where:

\[ \Delta M = \text{change in moment between points 1 and 2} \]

\[ \Delta C = \text{change in compressive force between points 1 and 2} \]

\[ V = \text{shear on vertical face at point 1} \]

\[ \Delta x = \text{change in length between points 1 and 2} \]

\[ d_e = \text{distance between centroids of compressive and tensile forces in section} \]

\[ b_v = \text{width of composite interface} \]

\[ V_h = \text{horizontal shear force between points 1 and 2} \]

\[ v_h = \text{horizontal shear stress on composite interface between points 1 and 2} \]
2.2.3 Classical Elastic Method

The Classical Elastic Method has been comprehensively used in research for numerous years to calculate the horizontal shear demand of a composite section. The recognizable formula is given in Equation 2-9. The Classical Elastic Method is based on the elastic response of composite beams. Therefore, the equation is not valid for cracked section.

\[ v_h = \frac{V}{f_{Pv}} \]  

Equation 2-9
Where:

\[ v_h = \text{horizontal shear stress across composite interface} \]

\[ Q = \text{first moment of area with respect to the neutral axis of the slab} \]

\[ I = \text{moment of inertia of the composite section} \]

\[ b_c = \text{width of the composite interface} \]

2.3 **Strength of Shear Stud Connectors**

The first attempt to quantify the useful capacity of mechanical shear connectors of varying types was developed in the 1957 AASHO Standard Specifications for Highway Bridges. The formulas, based partially on engineering mechanics and partially on empirical data, are still the basis for the accepted design procedures for bridge members today. The first step towards developing shear connector requirements for building members was established jointly between ASCE and ACI committees on composite construction in 1960. Shortly thereafter, design recommendations for shear studs in composite construction were included in the American Institute of Steel Construction (AISC) specifications in 1961. Although the specifications were sufficient to ensure structural safety, the efficiency of the design procedures to produce the most economical
result had yet to be determined. In the decades to follow, the effectiveness of the specifications to produce an economical result was the focus of many research investigations.

The specifications were later modified to include a strength reduction factor to adjust the available strength of a shear stud based on several parameters studied by Ollegaard et al (1971). However, studies by Hawkins and Mitchell (1984) as well as Jayas and Hosain (1987) put the AISC specifications under questioning as to their reliability to conservatively predict the reduction in strength of shear studs used with various configurations of metal decking. Since 1987 modifications to the code have been the topic of discussion and research for many engineers around the world. Jayas and Hosain (1988-89), Gibbings, Easterling, and Murray (1992), Lawson (1992) and Rambo-Roddenberry, Lyons, Easterling, and Murray (2000) have conducted push out and full scale beam tests in an attempt to provide sufficient comparable data along with a single design methodology to modify or replace the current AISC specifications. Research is currently ongoing to formulate provisions for the design of shear studs to replace the existing AISC standards.

The specifications that were originally developed to determine the useful capacity of mechanical shear connectors did not recognize the relationship between the strength of the shear connection and the potential ultimate flexural capacity of the member as we do today. It was not until Slutter and Driscoll (1965) that the ultimate flexural strength of
the composite member was used as part of the approach to design the minimum shear connection for composite members.

Ideally, the shear connection would be stiff enough to ensure full composite interaction by completely preventing any slip along the interface. This is not possible however, because it necessitates infinitely stiff mechanical connectors. The current AISC code limits the nominal shear transfer strength to the minimum of the values determined from the equations below.

$$ C_{max} = 0.85f_c b_w t_s $$  \hspace{1cm} \text{Equation 2-10} \\

Where:

$$ b_w = \text{effective width of the slab} \hspace{1cm} t_s = \text{thickness of the slab} $$

$$ T_{max} = A_s F_y $$  \hspace{1cm} \text{Equation 2-11} \\

Where:

$$ A_s = \text{cross sectional area of steel section} $$

The strength of an individual shear stud connector is defined in the expression given in Equation 2-12 below. The determination of the analytically complex connector
capacity was developed through numerous research programs summarized by Viest (1974) in a rational approach for computing the strength of various types of connectors.

\[ Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq A_{sc} F_u \]  

Equation 2-12

Where:

- \( Q_n \) = nominal strength of one stud
- \( A_s \) = cross sectional area of one stud
- \( F_u \) = minimum specified ultimate tensile strength of stud

The number of shear studs required between points of maximum and zero bending moment for full composite interaction is given by dividing the minimum of \( C_{\text{max}} \) and \( T_{\text{max}} \) by the nominal strength of an individual stud \( Q_n \). Slutter and Driscoll (1965) stated that the magnitude of the ultimate moment resistance is not affected by interfacial slip as long as the equilibrium condition is satisfied (described by the expressions above) and the slip is not great enough to cause failure of an individual connector.
2.4 Flexural Strength of Composite Beams

The nominal moment resistance of a composite concrete-steel section is based on the allowable concrete compressive force and the yield strength of steel used in the section. The nonlinear stress strain relationship for concrete in compression is traditionally simplified to an equivalent rectangular stress block as developed by C. S. Whitney in the 1930’s.

![Figure 2-3 – Whitney Rectangular Stress Distribution](image)

The concrete stress taken as $0.85f'_c$ is assumed to be uniformly distributed over an equivalent compression zone to a depth $a = \beta_1 c$ from the extreme compression fiber. The factor $\beta_1$ is equal to 0.85 for concrete with compressive strength less than 4000 psi.
The factor is reduced by 0.05 for each 1000 psi in excess of 4000 psi compressive strength. It is common practice to assume failure of concrete due to crushing when the extreme compression fiber in the concrete reaches a strain value of 0.003.

The maximum allowable tensile force in the steel portion of the composite section is taken as the product of the area of steel in tension and the yield stress of the structural steel. Steel is assumed to reach yield stress at a strain of 0.002. If the composite section was designed with sufficient shear connection to ensure full interaction, a linear strain profile is assumed throughout the depth of the section.

The average compressive and tensile forces create a couple moment as shown in Figure 2-3 above. From force equilibrium the compressive and tensile forces must have equal magnitudes. Therefore, the resulting nominal moment resistance of the composite section is described in Equation 2-13 where $d$ is the distance from the compression face to the centroid of the tension steel.

$$M_n = A_s F_y (d - a/2)$$  \hspace{1cm} \text{Equation 2-13}

More accurate computations for the nominal moment resistance can be achieved by using constitutive models that more precisely predict that actual stress strain relationship of the materials used in the composite section. Discretization techniques such as a layered sectional analysis are also used to increase the accuracy of predictions by
developing the moment-curvature relationship at a specific section of the composite member.

2.5 Shear Strength of Composite Beams

In typical concrete and steel composite design, the presence of composite interaction is not considered to improve the shear resistance of the concrete or steel sections acting alone. Traditionally the shear strength of the steel section is checked for adequacy with the predicted applied loads and if it is insufficient to resist the ultimate applied shear, the concrete section must be able to resist the full ultimate applied shear without the aid of any portion of the shear resistance of the steel section. Therefore, the shear strength of the steel beam falls out to the familiar equation for the nominal shear resistance as seen in the AISC LRFD Design Manual shown in Equation 2-14.

\[ V_n = 0.6F_y A_w \]  

Equation 2-14

The factors influencing the shear strength of reinforced concrete sections and the formations of inclined cracks are significantly more complex than those associated with a homogeneous material such as structural steel. It is difficult to declare a definitive conclusion regarding the correct mechanism for predicting inclined cracking and resistance to vertical shear of reinforced concrete sections. The work of Bresler and
MacGregor in 1967 introduced an excellent systematic correlation of the basic concepts upon which ACI and ASCE committees have built the current design specifications for shear in reinforced concrete members.

The traditional ACI approach for shear strength design of a reinforced concrete member considers the sum of the shear strength of the concrete and steel reinforcing separately as shown in Equation 2-15.

\[
V_n = V_c + V_s \quad \text{Equation 2-15}
\]

The simplified equation developed by ACI to predict the shear strength of concrete is generally taken as shown in Equation 2-16. It has been shown that because of the wide range of scatter of test results that this equation can be over conservative in some cases. A more accurate and complex method is given in Equation 2-17.

\[
V_c = 2\sqrt{f_c'}b_w d \quad \text{Equation 2-16}
\]

\[
V_c = \left(1.9\sqrt{f_c'} + 2500\rho_w \frac{V_{ud}}{M_{ud}}\right)b_w d \leq 3.5\sqrt{f_c'}b_w d \quad \text{Equation 2-17}
\]

Where:

\( f_c' = \text{concrete compressive strength} \)

\( \rho_w = \text{reinforcement ratio} \)
\(V_u = \text{factored shear force}\)

\(M_u = \text{factored moment}\)

\(b_w = \text{width of web}\)

\(d = \text{distance from compression face to centroid of tension steel}\)

The expression for shear resistance of the mild steel reinforcement is given in Equation 2-18. The formula was developed from a truss analogy with the formation of cracks inclined at a 45 degree angle creating compression struts that are balanced by steel reinforcing acting as tension ties.

\[V_s = \frac{A_vf_yd}{s}\]  

Equation 2-18

Where:

\(A_v = \text{shear reinforcement area}\)

\(s = \text{centerline spacing of shear reinforcing}\)
2.6 Composite Deflection

Accurate predictions for actual deflections in composite beams can be a laborious and complicated task even for the simplest of structures. Deflection is effected by complex time dependent phenomena such as creep, shrinkage, and temperature effects. However, the real complications arise from the unpredictability of the load changes throughout the life of the structure, the inability of structural models to account for three dimensional effects and unintentional continuities in the structure, and the fact that the effect of the nonlinear shear connections due to shear stud behavior is ignored in most deflection calculations. Despite the difficulty in computing precise deflections due to the aforementioned complications, the effects of known loading conditions, shoring, and even time dependent factors such as creep, shrinkage, and temperature changes are relatively well understood and can be accurately calculated.

More simplified calculations of deflections for composite beams involve the computation of the elastic cracked transformed section moment of inertia \( I_e \). The section can be converted to a homogeneous beam of either steel or concrete by using the modular ratio. Equation 2-19 shows the modular ratio used to transform the steel portion of the composite section into an equivalent area of concrete.

\[
n = \frac{E_c}{E_s}
\]

Equation 2-19
The gross moment of inertia of the transformed section is determined from the familiar expression shown in Equation 2-20. The cracked transformed moment of inertia \( I_{cr} \) is computed using the same formula, but ignoring the concrete in tension.

\[
I_g = I_o + \sum A \bar{y}^2 \quad \text{Equation 2-20}
\]

The elastic cracked transformed section moment of inertia is determined using the expression shown in Equation 2-21. This effective moment of inertia can be used in traditional expressions for deflections corresponding to the applied loading and support conditions.

\[
l_e = I_{cr} + (I_g - I_{cr}) \left( \frac{M_{cr}}{M_a} \right)^3 \quad \text{Equation 2-21}
\]

Where:

- \( M_{cr} \) = cracking moment
- \( M_a \) = applied moment

The use of an appropriate modulus of elasticity for the time frame of a sustained load can be used to determine approximate creep deflection values. AASHTO and the ACI-ASCE joint committee recommend reducing the elastic modulus for concrete to one
half or one third of its typical value. Although, these arbitrary specifications only approximate the effects of creep on the composite system on the order of ± 30%.

2.7 Effective Width of Composite Sections

When composite T-beams are subjected to positive bending, the concrete slab acting as the flange of the beam resists longitudinal compression that is balanced by the longitudinal tensile force in the steel reinforcement. The longitudinal compressive stress varies over the width of the flange where it is largest directly over the beam web and declines as the distance from the web increases. The variations in stress are due to shear lag or longitudinal shearing deformations which reduce the longitudinal compressive strain in the slab with increased distance from the web. To simplify the varying stress distribution across the full width of the slab flange, a smaller width, known as the effective width, over which a uniform stress is distributed that corresponds to the same total compressive force as the varying stress distribution, is generally used for design purposes.

The effective width of a slab in a composite beam is traditionally calculated using the same procedure used for a reinforced concrete T-section as described in ACI section 8.10. For interior beams, the specifications state that the overall effective width shall not
exceed one-quarter of the beam span length. Also, the specifications limit the effective width of the overhanging flange on each side of the beam web to the least of

a. Eight times the slab thickness

b. One-half the clear distance to the next web

There are claims that the method described above used to calculate the effective width of composite beams produces results that are conservative from the observed behavior. Kuhlmann and Reig (2004) suggest that the conservatism of the current specifications is exaggerated in composite beams of reduced height. Although a proposed design procedure to calculate the effective width of composite beams with reduced structural depth was not explicitly developed as a result of the study, Kuhlmann and Reig claim that changes to the current method need to be made.
Chapter 3

Analytical Investigation

3.1 Objectives

The objectives of the analytical study are to initially analyze and understand the current procedures and accompanying assumptions used to design the composite floor system. Additionally, the behavior of the composite floor system and representative test specimens is predicted by an analytical model based on a layered sectional analysis developed on sound engineering principles. Comparative calculations are used to evaluate the validity of the current design procedures and assumptions made. Eventually, in conjunction with experimental testing, the analytical investigation is used to develop a revised design methodology for the composite floor system.

3.2 Current Design Procedures

This section describes in detail the procedures used to design the beam component of the Diversakore® floor system. The assumptions made during the design process, the
load combinations evaluated, and the specific procedures used to design for each of the limit states will be discussed.

3.2.1 Assumptions

Several simplifying and generally conservative assumptions are made during the design of Diversakore® composite beams using the current design procedures. First, it is assumed that there is full interaction between the cast-in-place concrete beam and the steel U-beam, as well as between the concrete topping and the hollow core slabs, up to the predicted failure of the beam, allowing for the development of sufficient stress in the section to be considered tension controlled failure. The assumption of full interaction between these components also allows for the use of a linear strain profile for the analysis of the predicted response. Additionally, it is assumed that there would be no sharing of vertical shear resistance between the steel U-beam and the shear reinforcement of the concrete beam. That is, if the steel plate were unable to resist the full shear force, the concrete beam must be designed to carry the full shear force on its own. It is assumed that the top flute of the hollow core slabs is included in the depth of the slab available for the effective compressive flange width of the section and that the effective width of the composite beam can be accurately predicted by the specifications set forth by current codes. Lastly, it is assumed that the structural integrity of the steel U-beam is entirely negated in the case of a fire requiring at least a 2 hour design rating.
3.2.2 Loading

The loading conditions considered in the design of Diversakore® beams follows the guidelines set out in ASCE 7-05 chapter 2. Specifically, the load combinations described in section 2.3.2 were considered in conjunction with the load combinations for extraordinary events discussed in section C2.5. The controlling load combinations for most typical scenarios the composite beams are designed for, and consequently the ones used in design of the test specimens, are show below.

1) 1.2D + 1.6L (Full composite section)
2) 1.2D + 0.5L (Reinforced concrete section only)

Where:

\[ D = \text{dead load} \]
\[ L = \text{live load} \]

The live loads considered in the design of the composite beams are generally taken from Table 4-1 of ASCE 7-05. The magnitude of the live load considered in a typical design is 100 psf over the entire area tributary to a single beam. The use of live load reduction as described in section 4.8 is administered in the current design process for
most cases. Equation 3-1 below is typically used to determine the specified reduced live load. The maximum allowable reduction in live load is 50% for members supporting one floor and 60% for members supporting two or more floors.

\[
L = L_o \left( 0.25 + \frac{15}{K_{LL}A_T} \right)
\]

Equation 3-1

Where:

\( K_{LL} \) = live load element factor (Table 4-2 ASCE 7-05)

\( A_T \) = tributary area for the member being designed

3.2.3 Limit States

The prominent limit states that are considered during the design of Diversakore\textsuperscript{®} composite beams include strength limit checks on yielding or rupture of the steel section in tension due to flexure, yielding of steel webs due to shear, crushing of concrete in compression due to flexure, allowable shear resistance of reinforced concrete, and serviceability checks on deflection limits for construction loads, service loads, and sustained loads.
The flexural strength limit states for the steel and concrete portions of the composite section are specified in the 13th Edition of the AISC Steel Construction Manual Chapter F and ACI 318-05 Chapter 10 respectively. The nominal flexural strength of the steel section is given as shown in Equation 3-2.

\[ M_n = F_y Z_x \]  
Equation 3-2

Where:

\[ Z_x = \text{plastic section modulus about the x axis} \]

The nominal flexural strength of the concrete component of the section is limited by the maximum usable strain in the extreme compression fiber of the concrete equal to 0.003 based on its distance from the neutral axis and the stress in the steel components less than or equal to their respective yield stress based on section 10.3.2 of ACI 318-05.

The shear strength limit states for the steel and concrete components of the composite section are indicated in AISC Steel Construction Manual Chapter G and ACI 318-05 Chapter 11 respectively. As mentioned, in section 3.2.1 of AISC Chapter I for design of composite members, the applied shear force must be resisted by either the steel section alone or the reinforced concrete section alone. The nominal shear strength of the steel section based on limit states of yielding and buckling is given as shown in Equation 3-3.
The nominal shear strength of the reinforced concrete component of the composite section is specified in section 11.3 and 11.5 of ACI 318-05. The overall shear strength of the reinforced concrete section is calculated as the sum of the separate resistances of the concrete and reinforcing steel as shown in Equation 3-4. The concrete component is calculated using Equation 3-5, and the steel component is determined from Equation 3-6.
\[ V_n = V_c + V_s \]  
\[ V_c = 2\sqrt{f'_c b_w d} \]  
\[ V_s = \frac{A_v f_y d}{s} \]

Where:

- \( b_w \) = section effective width
- \( d \) = depth from extreme compression fiber to the centroid of reinforcing steel
- \( A_v \) = cross sectional area of shear reinforcing
- \( s \) = centerline longitudinal spacing of reinforcing bars

Limits on the maximum allowable spacing for shear reinforcement are taken from ACI 318-05 specifications in section 11.5.6.3. The recommended equation for the minimum allowable area of steel is rearranged to solve for the maximum spacing as shown in Equation 3-7.
Additional limitations on the maximum allowable spacing are also specified in section 11.5.5 based on the geometry of the member and the shear resistance of the reinforcing steel in the section. Often times, because of the geometry of the reduced height of the composite section, the spacing of shear reinforcement is limited to one quarter of the depth to the centroid of the steel as recommended by the specifications mentioned above.

Serviceability limit states for deflection are also accounted for in the current design procedures. Maximum allowable deflections are limited by ratios with the beam span as described in Table 1604.3 of the International Building Code 2006. Figure 3-1 below shows the allowable deflection limits for beams depending on the use of their construction and various load combinations they will be subject to during the life of the structure.

<table>
<thead>
<tr>
<th>Construction</th>
<th>L</th>
<th>S or W</th>
<th>D + L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Members</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Supporting plaster ceiling</td>
<td>L / 360</td>
<td>L / 360</td>
<td>L / 240</td>
</tr>
<tr>
<td>Supporting nonplaster ceiling</td>
<td>L / 240</td>
<td>L / 240</td>
<td>L / 180</td>
</tr>
<tr>
<td>Not supporting ceiling</td>
<td>L / 180</td>
<td>L / 180</td>
<td>L / 120</td>
</tr>
<tr>
<td>Floor Members</td>
<td>L / 360</td>
<td>-</td>
<td>L / 240</td>
</tr>
</tbody>
</table>

**Figure 3-1 – Deflection Limits**
Values for the allowable shear strength of the steel U-section as well as the reinforced concrete T-section are compared to the ultimate factored applied shear for each of the controlling load cases in Figure 3-2 below. The values for the allowable moment resistance compared to the ultimate factored moment for each load case is also tabulated. The design of the test specimens was most limited by the required flexural strength necessary to resist the factored moment applied by the fire loading condition 1.2D + 0.5L where the steel U-section is not considered effective.

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Effective Section</th>
<th>( V_u ) (kips)</th>
<th>( \Phi V_{n \text{Steel}} ) (kips)</th>
<th>( \Phi V_{n \text{R.C.}} ) (kips)</th>
<th>( M_u ) (kip-ft)</th>
<th>( \Phi M_n ) (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2D + 1.6L</td>
<td>R.C. Beam + Steel U-Beam</td>
<td>89</td>
<td>75</td>
<td>102</td>
<td>600</td>
<td>715</td>
</tr>
<tr>
<td>1.2D + 0.5L</td>
<td>R.C. Beam Only</td>
<td>61</td>
<td>0</td>
<td>102</td>
<td>350</td>
<td>356</td>
</tr>
</tbody>
</table>

**Figure 3-2 – Limit States**

### 3.3 Layered Sectional Analysis

The layered sectional analysis produced predictions of the nominal moment capacity, elastic stiffness, load deflection behavior, and strain profiles for the composite beam section. The analytical model also provided the ability to input exact material
properties and loading conditions from the performed tests to increase the accuracy of the predicted behavior when compared to the observed specimen response. The details of the assumptions made within the analytical model, the specific steps taken in the computation of the nominal moment capacity and load deflection behavior, as well as the selection of the constitutive material models used will be described in this section.

3.3.1 Overview

The proposed analytical model of concrete-steel composite beams predicts the moment curvature response of the beams based on a layered sectional analysis, which satisfies force equilibrium and strain compatibility. The model integrates constitutive material models that predict the nonlinear behavior of both the concrete and steel portions of the composite section. The modified form of the equation proposed by Popovics was implemented to predict the compressive response of concrete and the Ramberg-Osgood equation was used to describe the stress-strain relationship of structural steel. The proposed analytical model assumes that there is no slip between the concrete and the steel sections. Therefore, there is full composite interaction with no strain discontinuities throughout the height of section. The tensile strength of concrete is neglected in the analytical model. Ultimate failure of the beam is assumed to be due to concrete crushing when the extreme compression fiber of the concrete reaches the ultimate strain of 0.003
in/in. Other potential modes of failure including local buckling of steel webs or flanges were not considered in the proposed analytical model.

3.3.2 Procedure

The layered sectional analysis model is based on a series of selected strain values for the extreme compression fiber of the concrete slab. From the selected top strain value ($\varepsilon_{\text{top}}$) and an assumed depth of the neutral axis (c), the strain at any depth d ($\varepsilon_d$) is determined using Equation 3-8. The corresponding linear distribution of strain is shown in Figure 3-3.

$$\varepsilon_d = \left(\frac{c-d}{c}\right)\varepsilon_{\text{top}}$$  \hspace{1cm} \text{Equation 3-8}

Figure 3-3 – Linear Strain Distribution
The composite beam is separated into several major components based on changes in geometry and material properties throughout the depth of the section. The components are themselves divided into many layers in which the strains are computed using Equation 3-8.

The stress distribution throughout the depth of a section is determined from the known strain values at each layer using the appropriate constitutive model for the material being analyzed. The force in each section is then computed by integrating the product of the width \( b \) and the calculated stress \( f(x) \) over the depth of the section \( d \) as shown in Equation 3-9. The integral can be approximated by using an accurate numerical integration technique such as Simpson’s rule.

\[
F = \int_{d_1}^{d_2} b f(x) \, dx \tag{Equation 3-9}
\]

The stress in the concrete sections, including both cast-in-place concrete as well as the precast hollow core slabs, is determined by inputting the calculated strain in each concrete layer to the modified form of Popovics' equation and solving for the concrete stress, \( f_c \), as shown in Equation 3-15 3-15. Equation 3-15 is valid only for concrete in compression.

The stress in the steel components, including the structural steel U-beam and the mild reinforcing steel, is determined by inputting the calculated strain in each steel layer to the Ramberg-Osgood as shown in Equation 3-21.
Once the forces in each of the sections have been determined, the sum of the forces is computed to establish equilibrium. Equation 3-10 shows the sum of the forces in the cast-in-place concrete top flange \( F_{ct} \), precast concrete hollow core top fluted middle flange \( F_{cm} \), cast-in-place concrete bottom flange \( F_{cb} \), cast-in-place concrete web \( F_{cw} \), steel U-beam top flange \( F_{st} \), steel U-beam web \( F_{sw} \), steel U-beam bottom flange \( F_{sb} \), positive moment reinforcing steel \( F_s^+ \), and negative moment reinforcing steel \( F_s^- \). Force equilibrium is established by iterating the depth of the neutral axis until Equation 3-10 equals zero.

\[
F_{tot} = F_{ct} + F_{cm} + F_{cb} + F_{cw} + F_{st} + F_{sw} + F_{sb} + F_s^+ + F_s^- \quad \text{Equation 3-10}
\]

Once force equilibrium is established, the contribution to moment resistance of each section can be determined by inserting the distance from the neutral axis multiplier \((y)\) within the integral for the calculation of force as shown in Equation 3-11.

\[
M = \int_{d_1}^{d_2} y b f(x) dx \quad \text{Equation 3-11}
\]

The nominal moment capacity can be calculated by summing the moment contributions of the cast-in-place concrete top flange \( M_{ct} \), precast concrete hollow core top fluted middle flange \( M_{cm} \), cast-in-place concrete bottom flange \( M_{cb} \), cast-in-place concrete web \( M_{cw} \), steel U-beam top flange \( M_{st} \), steel U-beam web \( M_{sw} \), steel U-
beam bottom flange ($M_{sb}$), positive moment reinforcing steel ($M_{s^+}$), and negative moment reinforcing steel ($M_{s^-}$) as shown in Equation 3-12.

$$M_n = M_{ct} + M_{cm} + M_{cb} + M_{cw} + M_{st} + M_{sw} + M_{sb} + M_{s^+} + M_{s^-}$$  \hspace{1cm} \text{Equation 3-12}$$

Once the nominal moment capacity has been determined, the corresponding curvature is calculated from the depth of the neutral axis needed to establish force equilibrium as shown in Equation 3-13. The moment and corresponding curvature establishes one point on the desired diagram predicting the relationship between these two variables.

$$\Phi = \frac{\epsilon_{top}}{c}$$  \hspace{1cm} \text{Equation 3-13}$$

The value of the strain at the top of the concrete ($\epsilon_{top}$) can be incrementally increased and the procedure described above repeated for each chosen value of strain until the full moment-curvature relationship is developed. Since the predicted failure mode for this specimen was concrete crushing at the extreme compression fiber, the value of $\epsilon_{top}$ is increased until it reaches the maximum allowable value of 0.003.

The predicted load-deflection response of the composite beams can be calculated using the predicted moment-curvature relationship in conjunction with the second moment of area theorem. To determine the deflection of the beam at a desired location along the beam line, the beam must be broken into various segments along its length.
The applied moment due to the support conditions of the beam and a particular configuration of applied loads must be calculated at each of the various locations along the span of the beam from one support to desired location. Each calculated moment can then be related to a corresponding curvature based on the predetermined relationship. Then, knowing the distance from the support, the calculated applied moment, and corresponding curvature of each segment up to the desired location, the deflection due to the configuration of applied loads can be determined using Equation 3-13. A graphical representation of the process used to determine the deflection at midspan under a uniformly distributed load by calculating the applied moment and relating it to the beam’s curvature is shown below in Figure 3-4 using Equation 3-14.
3.3.3 Constitutive Models

The constitutive material models used in this analysis were chosen based on accuracy as well as versatility. Since the composite floor system incorporates precast hollow core slabs in addition to cast-in-place concrete, the representation of the compressive behavior of normal concrete and the potentially higher strength precast slabs
within a single constitutive model is desirable. By the same theory, a more adaptable steel constitutive model was chosen to represent the bent steel plate U-beam, mild reinforcement, and potential for prestressing steel to be added to the system.

To accurately represent the potential range of concrete compressive strengths in the layered sectional analysis, a modified form of Popovics’ equation was chosen over common concrete stress strain relationships. Figure 3-5 is a graphical comparison of a conventional bilinear model and Hognestad’s equation with Popovics’ equation. Popovics’ equation was chosen because bilinear models were not considered accurate enough for research applications; and Hognestad’s equation is only valid for concrete compressive strengths up to 6000 psi. Equation 3-15 shows Popovics equation modified by Thornfeldt, Tomaszewicz, and Jensen (Collins 1997)

![Figure 3-5 – Concrete Constitutive Model Comparison](image_url)
\[ \frac{f_c}{f'_c} = \frac{n \left( \frac{\epsilon_{cf}}{\epsilon'_c} \right)}{n-1 + \left( \frac{\epsilon_{cf}}{\epsilon'_c} \right)^{nk}} \]

Equation 3-15

Where:

- \( f_c \) = stress caused by strain \( \epsilon_{cf} \)
- \( f'_c \) = peak stress observed in cylinder test
- \( \epsilon'_c \) = strain when \( f_c \) reaches \( f'_c \)
- \( n \) = curve-fitting factor equal to \( \frac{E_c}{(E_c - E'_c)} \)
- \( E_c \) = tangent stiffness when \( \epsilon_{cf} \) equals zero
- \( E'_c = f'_c / \epsilon'_c \)
- \( k \) = factor to increase the postpeak decay in stress.

For \( \left( \frac{\epsilon_{cf}}{\epsilon'_c} \right) < 1 \), \( k = 1.0 \).

Ideally, the constants used in the model are derived directly from an experimentally determined stress-strain curve, so that the constitutive model and the experimental data match exactly. This solution works best in research applications. For most design conditions, only the suggested compressive strength \( f'_c \) and unit weight \( w_c \).
are known. It is therefore necessary to estimate the remaining parameters based on the known parameters.

![Stress Strain Relationship of Concrete and Components](image)

**Figure 3-6 – Stress Strain Relationship of Concrete and Components**

As shown in Figure 3-6, the initial stiffness of concrete is somewhere between the aggregate stiffness and the stiffness of the hardened cement paste. The concrete initial stiffness can be approximated using one of several material modeling laws. Included in the material modeling laws is the following expression presented in the ACI Code based on the work of Pauw (ACI 2005).

\[ E_c = w_c^{1.5} 33 \sqrt{f_c'} \]  

*Equation 3-16*
Where:

\[ w_c = \text{unit weight of concrete lb/ft}^3 \]

This equation should only be used to estimate the initial stiffness for normal strength concrete. Pauw’s work in developing these equations was based on concrete with compressive strength up to 5500 psi. It was later discovered by Carrasquillo, et al that Pauw’s equations overestimate the stiffness of concrete for compressive strengths higher than 6000 psi. Carrasquillo et al recommended using the following expressions in lieu of Pauw’s equations for concrete with high compressive strength.

\[ E_c = 40,000\sqrt{f_c'} + 1,000,000 \quad \text{Equation 3-17} \]

The remaining parameters needed to evaluate Popovics equation can be determined from Equation 3-18, Equation 3-19, and Equation 3-20. For low strength concrete, the values of n and k are typically around 4.0 and 1.0 respectively. For high strength concrete, n and k rise to 1.3 and 2.0 respectively.

\[ n = 0.8 + \frac{f_c'}{2,500} \quad \text{Equation 3-18} \]

\[ k = 0.67 + \frac{f_c'}{9,000} \quad \text{Equation 3-19} \]
\[ \epsilon' = \frac{\epsilon_c^t}{E_c} \frac{n}{n-1} \]

Unlike concrete, steel is a homogeneous material. This is significant because it allows for more accurate predictions of the stress-strain relationship due to the absence of material inconsistencies or defects. Thus, the level of accuracy is high even for simplified constitutive models. Although bilinear or trilinear relationships provide accurate stress-strain response for steel, the Ramberg-Osgood model was chosen because of its ability to utilize specific data input from experimental testing of samples to match the response of each steel component of the composite system. Figure 3-7 depicts steel stress-strain behavior as predicted by a bilinear, trilinear, and Ramberg-Osgood model.
The Ramberg-Osgood relationship is expressed in Equation 3-21 below. Values for the various constants associated with the Ramberg-Osgood model are typically determined from experimental tests but may be approximated based on commonly used material properties of the particular steel used.

\[
f = E_p \varepsilon_{pf} \left\{ A + \frac{1-A}{[1+(B \varepsilon_{pf})^c]^{1/c}} \right\}
\]

Equation 3-21
Where:

\[ f = \text{stressed caused by strain } \epsilon_{pf} \]

\[ E_p = \text{tangent stiffness when } \epsilon_{pf} \text{ equals zero} \]

\[ A = \text{factor associated with the slope of the post yield response} \]

\[ B = \text{factor associated with the location of first yield} \]

\[ C = \text{transition curve factor from elastic behavior to post yield response} \]

### 3.4 Preliminary Results

A comparison of several analytical stiffness calculations is depicted graphically in Figure 3-8 below. As seen in the figure, the elastic stiffness, or tangent stiffness, computed from transformed section properties with no cracking compares favorably to the initial stiffness of the load-deflection response derived from the moment-curvature relationship of a section of the test beam cut at midspan. The secant stiffness, calculated by the current design procedures using a level of cracking pertaining to the predicted yield moment of the composite section, is less stiff than the initial elastic stiffness predicted by the analytical model as shown in the figure.
The predicted moment curvature relationship of a section of the representative test beams cut at midspan is shown in Figure 3-9 below. The change in the depth of the neutral axis as it relates to the change in curvature is also shown in the figure.

Figure 3-8 – Analytical Stiffness Comparison
Figure 3-9 – Moment Curvature Relationship and Neutral Axis Depth
Chapter 4

Experimental Investigation

4.1 Introduction

This chapter presents the details of the experimental program investigating the Diversakore® composite slim-floor system conducted in the Constructed Facilities Laboratory at North Carolina State University. The objective of the experimental program was to validate the proposed behavior of the composite system as well as to evaluate the effect of composite action on the system’s performance. The program included the design, construction, and laboratory testing of two full-scale composite beams. The details of the design of the beams, construction methods and sequence, test setup, instrumentation of specimens, test load cycles, and properties of the materials used in the innovative composite system will be discussed in this section.

4.2 Design of Specimens

The design of the innovative composite beams tested in the experimental program was a collaborative effort between the engineers at Diversakore® LLC and North Carolina State University. Two full scale specimens were designed and constructed for
laboratory testing. One of the beams was designed to represent full composite interaction, while the second would represent the inherent partial interaction achieved from the bond formed between the cast-in-place concrete and bent plate steel beam.

Both beams were designed to be consistent with their most prominent use as a girder for an office building type structure. The beams were designed based on a typical 30 ft. by 30 ft. bay with characteristic loads and design methods as described in Chapter 3. Full composite interaction for test specimen DK1 was ensured by installing two rows of ¾ in. diameter 8 in. tall shear studs spaced 6 in. apart with a 10 in. stagger. No additional mechanical shear connectors were used in the construction of the second beam as to limit the composite interaction to the natural bond between the concrete and steel U-beam components. Steel plates ¼ in. thick and 3 in. wide were installed 4 ft. on center along the length of the beam to resist the U-beam’s tendency to buckle outwards during the placement of the hollow core slabs.

The concrete beam was reinforced with longitudinal compression and tension mild steel reinforcing bars to meet the requirements for flexural design specified in ACI 318-05. The concrete beam was also reinforced with mild steel hat-shaped stirrups in accordance with the ACI requirements for shear design. The flexural and shear reinforcement along with the welded straps are shown in Figure 4-1 below.
Precast hollow core slabs 4 ft. long, 3 ft. wide, and 8 in. thick, were used as the major components of the floor system. The width of the hollow core slabs was chosen to fully encompass the allowable effective flange width of the composite section calculated in accordance with ACI section 8.10.2. #4 mild steel reinforcing bars were grouted into the hollow core keyways and hooked into the concrete beam web to increase continuity between the slabs and the beam core. Two inch thick styrofoam block-outs were installed in the slab cores at the beam-slab interface to prevent flow of cast-in-place concrete into the cores. The block-outs were installed at the beam-slab interface to create the worst case scenario for beam construction by providing the least amount of continuity between the concrete beam and hollow core slabs via concrete seepage. Perforated angle 1 ½ in. x 3 in. x ⅛ in. was mounted along the outside bottom edge of the hollow core slabs to increase the continuity between slab sections. At high deformations, the slabs
may attempt to separate along the outside bottom edge. However, in field conditions, the slabs would span between two girders and would resist the potential separation. The perforated angle was installed to better simulate such field conditions. The 2 in. topping slab cast above of the concrete beam web and hollow core slabs was reinforced at midheight with $6 \times 6 \times W2.9 \times W2.9$ welded wire fabric to prevent cracking due to shrinkage or temperature effects and enhance the effectiveness of composite action.

### 4.3 Construction of Specimens

The construction of the two test specimens was a collaborative effort between Pegasus Steel Products, Gate Precast Company, S.T. Wooten, IQ Contracting, and the Constructed Facilities Laboratory at North Carolina State University. The innovative composite floor system was constructed in several phases including production of the steel U-beam, shear stud and steel strap welding, tying and placement of reinforcing cage, construction, preparation, and placement of hollow core slabs, beam shoring and installation of formwork, and concrete casting and finishing.

The ¼ in. A572 Grade 50 steel plate used for the U-beam was hot rolled and cut to length by Feralloy Corporation in Charleston, South Carolina. The plate was then shipped to Goose Creek, South Carolina to Pegasus Steel Products. There the plate was
cut to width and bent using a large scale hydraulic press break designed by Pacific Press Technologies before being transported to North Carolina State University. Once the beams were unloaded at the Constructed Facilities Laboratory, they were moved into place to continue preparation prior to testing.

Shear studs were installed on one beam using a Nelweld 6000 stud welder donated by Nelson Stud Welding. The gun was set to discharge 650 amps of electricity for 0.35 seconds to melt the stud tip and then plunge the remaining portion of the stud into the molten steel that was contained by the ceramic ferrule. The stud multiprocess welder, accessories, typical weld collar, and installation process are shown in Figure 4-2 (a) - (d).

(a) Nelweld 6000                     (b) Stud welding gun, shear stud, and ferrule
The flat plate steel straps were welded into place using a Miller XMT 350 series welder. The wire fed MIG process was used to weld the straps into place on the underside of the U-beam top flange. The welder used and typical fillet weld applied to the steel straps are depicted in Figure 4-3 (a) & (b).

Figure 4-2 – Stud Welding Equipment and Installation
The mild steel reinforcement was fabricated by Gerdau Ameristeel in Raleigh, N.C. and delivered to the Constructed Facilities Laboratory. The reinforcing cages were assembled and tied in place in the beams. The compression reinforcing bars were temporarily supported using wood shoring. The tension reinforcing steel was supported by 1 ½ in. beam bolsters resting on the steel beam and the stirrups. Stirrups were hung from the top bars and tied into place producing a rigid cage as shown in Figure 4-4.
The hollow core slabs were manufactured at Gate Precast Company in Oxford, N.C. The slabs were cut to length prior to being delivered to the Constructed Facilities Laboratory. Styrofoam block-outs were cut and fitted into the cores at the interface between the concrete beam and hollow core slabs. The slabs were set in place resting on the U-beam top flange and temporary wood shoring using a mobile crane. The manufacturing of the block-outs and placement of hollow core slabs are shown in Figure 4-5 (a) & (b).
Once the hollow core slabs were set in place, the hooked bars were grouted into the keyways between each slab piece as shown in Figure 4-6 (a) & (b).

**Figure 4-5 – Styrofoam Block-outs in Hollow Core Slabs**

**Figure 4-6 – Hooked Bars in Grouted Joints**
Welded wire fabric was cut and placed on top of the hollow core slabs supported by 1 in. beam bolsters to reach midheight of the topping slab.

The beams were supported at the ends and shored at the third points using steel columns that were leveled from the uneven asphalt surface. The columns were also shored underneath each beam to minimize settlement during the casting and curing processes. Wood formwork was built around the outside edges and beam ends to support the concrete for the beam web and topping slab during casting and the initial stages of curing.

The concrete for the specimens was produced and delivered by S.T. Wooten based in Wilson, N.C. Assistance with the placement and finishing of the concrete was provided by IQ contracting. In each case, the beam section was poured and vibrated first, then the topping slab was poured, vibrated, screeded, floated, and trowel finished. Twelve 4 x 6 in. cylinders were taken the day of casting from the truck after half of the concrete had been cast. The cylinders were then set to field cure with the test specimens. Figure 4-7 shows an overhead view of the casting process.
Once the beams were cast, they were covered with wet burlap and plastic to contain the moisture during the initial stages of curing. After three days of curing, the formwork was removed from the beams. The beams were allowed to cure for 21 days before the first beam was moved into the lab to begin setup for testing.

4.4 Test Setup

Both composite beams were tested in a simply supported configuration with supports spaced 27 ft. on center as shown in Figure 4-9. Four equal line loads were
applied at the fifth points between supports. The load was applied via a two tier loading tree as shown in Figure 4-8. A 440 kips MTS hydraulic actuator was used to apply load to the specimen. The actuator hung from a braced steel frame tied to the strong floor at the base. A built up double channel shape strengthened with transverse stiffeners to prevent buckling was attached to the actuator and used as a primary spreader beam. The primary spreader beam rested on pin and roller supports used to transfer load to two W 14 x 176 beam sections. The wide flange sections acted as secondary spreader beams supported by 6 x 8 x ½ in. HSS members. Teflon pads between the secondary spreader beams and the HSS members were used to ensure proper mobility between the components at large deflections. The HSS members spanned the full width of the specimen and were placed on top of ½ in. thick neoprene pads to provide an evenly distributed line load across the entire width of the specimen. The specimens themselves rested on top of pin and roller systems that were supported by concrete blocks secured to the strong floor.
Figure 4-9 shows a schematic of the full setup used for the experimental testing of the innovative composite floor system.
4.5 Instrumentation

The composite beam specimens were instrumented with devices used to measure deflection, concrete strain, steel strain, interfacial slip, as well as applied load. The instrumentation was installed at several sections along the beam span not only to understand the behavior of that section, but to more accurately predict the global behavior of the system as a whole.
250 ohm ¼ in. strain gauges were used to measure the strain in the steel section and mild steel reinforcement. Prior to placing the concrete, strain gauges were installed at several locations on the inside of the steel section and reinforcing bars at the beam quarterpoints and midspan. The gauges were affixed to the steel surface, protected from water ingress and mechanical impact using the Vishay products, M Bond 200, M Coat A, and Barrier E. An example of the process used to protect the gauges is shown in Figure 4-10 (a) & (b).

![Strain gauge installation and protection](image)

(a) Strain gauge with M Coat A       (b) Strain gauges protected with Barrier E

**Figure 4-10 – Strain Gauge Installation and Protection**

The strain gauge layout was chosen to use redundant gauges to provide more accurate readings as well as to ensure the collection of pertinent data in a gauge was damaged.
during construction. The resulting data provided a full strain profile through the depth of
the section at midspan and quarterspan locations.

100 mm pi gauges, as shown in Figure 4-11, were used to measure the strain in the
concrete at the top face of the slab. Pi gauges measure strain by monitoring the change in
resistance along the gauge as it either compressed or stretched. The gauges were
mounted ¾ in. above the concrete surface on screws that were glued to the topping slab.

![Figure 4-11 – Typical Pi Gauge Installation](image)

Deflections of the beam and hollow core slabs were measured using string
potentiometers resting on the strong floor as shown in Figure 4-12. The devices were
attached to the specimen using garden wire and either magnets or eye hooks in pieces of
wood glued to the underside of the specimen. Figure 4-13 shows the locations of all of the instrumentation placed on the specimens at a typical quarter point and at midspan.

![String Potentiometer Installation](image)

**Figure 4-12 – String Potentiometer Installation**

Where:

- **SP** = String Potentiometer
- **PG** = Pi Gauge

The interfacial slip between the steel U-beam, concrete beam, and hollow core slab components was measured using linear potentiometers. The potentiometers contacted the concrete beam on small aluminum squares that were glued to the specimen to ensure more accurate readings by providing a smooth surface to measure from. A small piece of steel angle was used on the hollow core slabs so the measurements could be taken from a single base point and the slip along the third composite interface between
the concrete beam and hollow core slab could be calculated accurately. The potentiometers were held into place using three pronged grips fixed to magnetic bases attached to the steel beam. Measurements were taken on both sides of the beam as well as on both ends as shown in Figure 4-14.

(a) Typical quarterpoint instrumentation

(b) Midpoint instrumentation

Figure 4-13 – Schematic Instrumentation Diagrams
(a) Typical linear potentiometer setup

Where:

DP = Duncan Pot (linear potentiometer)

S = Steel

HC = Hollow Core

(b) Typical end point instrumentation

Figure 4-14 – Linear Potentiometer Installation
Two 200 kips load cells were placed above the concrete block and below the pin support on one side of the beam. The values measured from the load cells were compared to the load values measured from the actuator to validate the even distribution of load applied to the specimen. By taking measurement from two load cells the presence of any torsional effects could also be monitored. Figure 4-15 shows an example of the load cell used during testing and a schematic of the end cross section with the support and load cells below.
(a) 200 kips load cell under pin support

(b) Front end support cross section

Figure 4-15 – Load Cell Installation
All of the instrumentation was wired into a series of Vishay SYS 5000 scanners that relayed the information into a personal computer to record the information. The system was set to record data every 5 seconds throughout the tests. The data acquisition system was controlled by a personal computer using Vishay Strain Smart software. The software provided real time displays of the information being recorded so that the measurements and select relationships could be monitored during the testing process. In addition to the instrumentation applied to the specimens, the values for load and stroke from the MTS hydraulic actuator were recorded throughout the testing. Figure 4-16 shows the data acquisition system and wired inputs from the instrumentation.

Figure 4-16 – Data Acquisition and Wiring
4.6 Test Program

Two composite beams were tested in the experimental phase of the program. Specimen DK1 was tested first and was constructed with shear studs to ensure full composite action. Specimen DK2 was tested second and was constructed without any mechanical shear connectors. A loading scheme was devised for the tests based on the proposed ultimate moment capacity predicted by the moment curvature analysis described in section 3.3. The various load steps used for each test are shown in Figure 4-17. The service load was approximated from the calculated factored load by dividing by the average load factor of 1.5.

<table>
<thead>
<tr>
<th>Load Cycle</th>
<th>Applied Load (kips)</th>
<th>Equivalent Midspan Moment (k-ft)</th>
<th>Load Rate (in/min)</th>
<th>Unload Rate (in/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preload</td>
<td>16</td>
<td>65</td>
<td>0.033</td>
<td>0.033</td>
</tr>
<tr>
<td>1/2 Service Load</td>
<td>55</td>
<td>223</td>
<td>0.033</td>
<td>0.033</td>
</tr>
<tr>
<td>Service Load</td>
<td>110</td>
<td>446</td>
<td>0.033</td>
<td>0.033</td>
</tr>
<tr>
<td>Factored Load</td>
<td>164</td>
<td>664</td>
<td>0.033</td>
<td>0.033</td>
</tr>
<tr>
<td>Failure Load</td>
<td>185</td>
<td>750</td>
<td>0.033</td>
<td>0.033</td>
</tr>
</tbody>
</table>

Figure 4-17 – Load Cycle Table

The MTS hydraulic actuator was set in displacement control mode and the load rate of 0.033 in./min was input into the Testware controller software. For each load step, the load was monotonically increased until the target load was reached. The target load was held for a few minutes while the beam was checked for signs of distress. The applied
load was then incrementally decreased until only the dead load of the test setup was applied to the specimen before beginning the next load cycle.

4.7 Material Properties

The steel U-beams used in the experimental testing program were both manufactured from the same batch of ASTM grade A572 Grade 50 steel. Three coupons were cut from specimen DK1 to validate the material properties. The coupons were fabricated and tested in accordance with ASTM A370 to determine the yield and ultimate strengths of the material.

Figure 4-18 – Sample Coupon Stress-Strain Diagrams
<table>
<thead>
<tr>
<th>ID</th>
<th>Fy (ksi)</th>
<th>Fu (ksi)</th>
<th>% elong</th>
<th>E (ksi) x 10³</th>
</tr>
</thead>
<tbody>
<tr>
<td>DK 1</td>
<td>62.3</td>
<td>72.9</td>
<td>27.6</td>
<td>21.7</td>
</tr>
<tr>
<td>DK 2</td>
<td>61.9</td>
<td>72.7</td>
<td>34.8</td>
<td>25.42</td>
</tr>
<tr>
<td>DK 3</td>
<td>62.3</td>
<td>73.2</td>
<td>28.9</td>
<td>24.75</td>
</tr>
<tr>
<td>Averages</td>
<td>62.2</td>
<td>72.9</td>
<td>30.4</td>
<td>24.0</td>
</tr>
</tbody>
</table>

Figure 4-19 – Sample Coupon Data Table

The concrete for the casting of the test specimens was mixed and delivered by S.T. Wooten. The mix contained ¾ in. pea gravel to ensure proper distribution of concrete throughout the beam and topping slab. Cylinders were taken the day of the casting and were tested in accordance with ASTM C39. The results of the cylinder tests and corresponding compressive strengths can be found in Figure 4-20.
4.8 Experimental Results

Preliminary results from the experimental investigation including deflections, strain profiles, and measure interfacial slip are presented in this section. Discussion of data collected and comparisons of the results to values predicted by the existing design philosophy as well as to values predicted by the layered sectional analysis will be discussed in chapter 5.

Figure 4-21 depicts the average moment-deflection behavior observed during testing at the quarter points and at midspan of specimen DK1. The moment-deflection response illustrates that specimen DK1 did not exhibit a significant amount of ductility

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Days of Curing</th>
<th>Peak Load (lbs)</th>
<th>f'c (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DK1</td>
<td>14</td>
<td>44221</td>
<td>3520</td>
</tr>
<tr>
<td>DK2</td>
<td>14</td>
<td>44107</td>
<td>3510</td>
</tr>
<tr>
<td>DK3</td>
<td>14</td>
<td>47161</td>
<td>3750</td>
</tr>
<tr>
<td>DK4</td>
<td>28</td>
<td>43570</td>
<td>3470</td>
</tr>
<tr>
<td>DK5</td>
<td>28</td>
<td>41082</td>
<td>3260</td>
</tr>
<tr>
<td>DK6</td>
<td>28</td>
<td>41026</td>
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<td>51204</td>
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<td>48038</td>
<td>3820</td>
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<tr>
<td>DK9</td>
<td>35</td>
<td>48405</td>
<td>3850</td>
</tr>
<tr>
<td>DK10</td>
<td>36</td>
<td>48914</td>
<td>3890</td>
</tr>
<tr>
<td>DK11</td>
<td>36</td>
<td>50243</td>
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</tr>
<tr>
<td>DK12</td>
<td>36</td>
<td>50667</td>
<td>4030</td>
</tr>
</tbody>
</table>
prior to failure. As shown in Figure 4-21, the composite beam behaved in a nearly linear fashion up to the factored moment of 664 k-ft. Shortly beyond the application of the factored load specimen DK1 began exhibiting non linear behavior but had an abbreviated yield plateau prior to failure. Also seen in the figure, there was residual deflection measured after each load cycle up to ½ in. after the application of the factored load. As the beam was reloaded, the stiffness did not appear to be significantly affected, and the specimen rejoined the path of the previous loading step at essentially the same applied moment and deflection.
Figure 4-21 – DK1 Applied Moment vs. Average Deflection
Figure 4-22 depicts the average moment-deflection behavior observed during testing at the quarter points and at midspan of specimen DK2. It can be seen in the figure that the specimen behaves in an essentially linear fashion until a change of stiffness was observed at an applied load of 490 k-ft. After the linear behavior, the specimen stiffness continued to decrease as additional load was applied up to the point of failure. The figure shows that although there was a noticeable amount of deflection after the linear region, a yield plateau was never reached prior to failure. The changing stiffness of specimen DK2 corresponds to measured interfacial slip between the composite components in the system. After the application of the service load cycle, residual deflections up to ½ in. were noticed at midspan of the composite section. Although, the stiffness of specimen DK2 appeared unaffected, as the specimen response rejoined the load path of the previous cycle at the same moment and deflection.
Figure 4-22 – DK2 Applied Moment vs. Average Deflection
Average strain profiles for test specimen DK1 were developed from the strain data collected and are shown in Figure 4-23 below. The plotted values were calculated by averaging the strain data collected from all of the instruments at an individual depth throughout the section as shown in the figure. Strain profiles were developed for each quarter point as well as at midspan of the composite beam and were developed at the application of each of the previously determined load cycles including the maximum load applied during the failure load step. It can be seen from the figure that test specimen DK1 behaved in a relatively linear fashion up to the point of tension steel yielding. The elastic behavior is exhibited in the data collected from each of the three cross sections for the earlier loading cycles. Slight nonlinear behavior in the section is shown in the later load cycles applied to specimen DK1.

The strain profiles developed from data collected during the testing of specimen DK2 exhibited some interesting behavior. During the early load cycles a relatively linear behavior was observed. However, the onset of interfacial slip created strain discontinuities along the interfaces between the reinforced concrete beam and the steel U-section. The interfacial slip caused redistribution of the forces from the steel U-section to the longitudinal reinforcing as shown in Figure 4-24. It can also be seen in the figure that the magnitude of the discontinuity increases with increased slip due to the increasing applied load.
Figure 4-23 – DK1 Strain Profiles

I - 1/2 Service Load Step (223 k-ft)
II - Service Load Step (446 k-ft)
III - Factored Load Step (664 k-ft)
IV - Failure Load Step (750 k-ft)

- Concrete Strain
- Steel Strain
Figure 4-24 – DK2 Strain Profiles

I - 1/2 Service Load Step (223 k-ft)
II - Service Load Step (446 k-ft)
III - Factored Load Step (664 k-ft)

* Specimen Failed at Factored Load

- Concrete Strain
----- Steel Strain

- Strain (με)
Figure 4-25 depicts the slip measured at all three interfaces for each end of the composite specimen DK1. As shown in the figure, there were minimal amounts of slip observed in the specimen prior to the application of the failure moment of 750 k-ft. Even under the application of the failure moment, the maximum recorded slip was less than one tenth of an inch prior to failure. After failure of the specimen due to concrete crushing, relevant slip between the hollow core slabs and the steel beam was observed on the beam end nearest to the crushing failure. Although significant slip was not observed at any other location, the slip between the hollow core slabs and steel beam at the back end of the beam reached a maximum value of 0.58 in. at an applied moment of 125 k-ft during the application of the post failure loading cycle. The lack of slip between the three interfaces throughout the pertinent behavior of the specimen suggests that there was full composite interaction between each of the components up to failure.
Figure 4-25 – DK1 Applied Moment vs. Average Slip at Beam Ends
Figure 4-26 depicts the slip measured at all three interfaces for each end of the composite specimen DK2. The figure shows that initial signs of slip were noticed in the service load step at an applied moment of 446 k-ft on the order of 0.01 in. Although the bond was not entirely lost at this point, the residual slip was maintained during unloading. The natural bond between the concrete and steel began degrading at an applied moment of 490 k-ft and continued to slip throughout the duration of the factored load cycle up to failure of the specimen. The slip started on the back half of the beam containing quarter point Q3, eventually making its way to the front half of the beam at approximately 540 k-ft. As described in the previous sections, the interfacial slip significantly impacts the deflection and strain responses of the specimen due to the loss of composite interaction. The impacts can be seen in the recorded data as the specimen exhibited a significant change in stiffness and drastic changes in measured strain in conjunction with the continuous slip that initiated at an applied moment of 490 k-ft.
Figure 4-26 – DK2 Applied Moment vs. Average Slip at Beam Ends

Note: 1.12" maximum slip recorded
Chapter 5

Results and Discussion

This chapter presents the observed and recorded results from the experimental laboratory testing of composite specimens DK1 and DK2 as described in chapter 3. The results from the analytical layered sectional analysis as described in chapter 2 are presented along with the results from the evaluation of the initial design procedures as they pertain to the observed behavior of the two test specimens. A summary of the pertinent collected data from the laboratory testing, a comparison of the predicted response from the analytical study to the observed behavior, and the evaluation of the existing design procedure will be discussed.

5.1 Experimental Results

Composite test specimens DK1 and DK2 were tested in a simply supported configuration with four equal line loads spaced evenly at the 1/5 points along the beam span as described in test setup. The specimen was loaded and unloaded in several cycles corresponding to service, factored, and predicted failure loads. The results of the laboratory testing are described in the following sections discussing the deflection
behavior, the measured strain response, the interfacial composite interaction, and the observed failure mechanisms.

5.1.1 Deflection

Deflection of the composite test specimens was measured at the quarter points as well as at midspan using string potentiometers as described in Chapter 4. The results from the deflection measurements described the physical response of the composite sections to the applied load and allowed for the calculation of the actual stiffness of each section to be compared to the analytical predictions. In addition to the displacement profile along the line of the beam, the general deflected shape of a cross sectional plane could be represented. The recorded data allowed for a well defined representation of the overall deformation of the composite test specimens corresponding to the applied test loading cycles.

DK1 Deflection

The values recorded for the deflection of the steel section and each of the hollow core slab flanges displayed consistent similarity throughout the load cycles. Such analogous values suggested that there were minimal torsional stresses induced in the beam and that the composite section deflected equally across each cross section. The
supporting data is best seen in Figure 5-1. It can also be seen in the figure that beginning at an applied moment of 750 k-ft and 2 ½ in. of deflection, there is an inconsistency between the readings from quarter point 1 and quarter point 3. This inconsistency is due to the fact that the crushing failure occurred on the side of the beam closest to quarter point 3. The effective cross section near this location was less stiff due to the initiation of failure. Therefore, the deflection values recorded from quarter point 3 were subsequently higher than those measured at quarter point 1 for the same applied moment.
Figure 5-1 – DK1 Applied Moment vs. Deflection
DK2 Deflection

The values recorded for the deflection of the steel section and each of the hollow core slab flanges displayed consistent similarity throughout the load cycles with the exception of a shift observed in gauge Q3 I. The shift can most likely be attributed to a malfunction in the string potentiometer isolated to a specific section of the overall range. Outside of that problematic range, the behavior matches exactly with the behavior recorded from all of the other gauges. Also, the disturbed behavior region is repeatable and recoverable through different loading cycles and is bound by matching behavior before and after the disturbance.

The behavior of specimen DK2, as well as that observed in specimen DK1, displays a reserve capacity of the system after failure of the system under the application of the maximum load. As shown in Figure 5-2, and Figure 5-1 for specimen DK1, a plateau in the load vs. deflection plot can been seen depicting the reserve capacity of the system even after apparent ultimate failure of the system and nearly complete debonding of the steel U-section and concrete T-section. In the case of specimen DK1 the reserve strength was enough to resist the service load conditions of the system.
Figure 5-2 – DK2 Applied Moment vs. Deflection
5.1.2 Strain

Strain measurements for the composite test specimens were taken at numerous locations throughout quarter point and midspan cross sections as described in Chapter 4. Strain gauges were used to measure the strain in the u-shaped steel section as well as in the reinforcing bars. Pi gauges were installed to measure the strain in the top surface of the concrete slab. The recorded strain data supported predictions about the degree of interaction between the various components within the composite system by creating strain profiles through the depth of a cross section. The installation of multiple gauges throughout the depth of the cross section allowed for the computation of the location and magnitude of strain discontinuities. The recorded strain data was used to make judgments about the utilization of the concrete and steel as structural materials as well as validate the design assumptions made about their material properties.

DK1 Strain

The overall behavior of the average measured strain at each of the various locations throughout the midspan cross section of specimen DK1 is depicted in Figure 5-3. It can be seen from the figure that the topping slab and top reinforcing bars were in compression throughout each of the loading cycles, as shown with negative values for strain. The general behavior of the gauges in compression was as expected, with the magnitude of strain in the concrete being higher than that of the top reinforcing because
of its greater distance from the neutral axis. The values for the average tension strain groupings from the steel and bottom reinforcement behaved in a similarly expected manner, with the magnitudes of strain increasing as the distance from the neutral axis increases. The figure also demonstrates the presence of residual strain in the system after the application of the factored load cycle.

For each load cycle, the acquired strain data was used to develop strain profiles for the cross sections at each quarter point and midspan of specimen DK1 as shown in Figure 5-4. The plotted value is an average strain for all of the gauges located at that specific height in the cross section. The figure shows that prior to yielding of the steel, the midspan strain profiles behave in an essentially linear fashion as predicted by the assumption of full composite interaction. The strain profiles for quarter point 3 (Q3) behave similarly to the midspan profiles taking into account the reduced flexural stresses induced at that section. However, the behavior at quarter point 1 (Q1) exhibits an uncharacteristic behavior. Increased strain in the tension rebar was observed beginning at the service load step and amplifying as the load was increased to the failure load. Upon closer examination of the strain profiles, there are indications of increased strain at the level of the tension rebar at higher loads in the midspan and Q3 cross sections. It is possible that beam was beginning to lose its composite interaction but had not yet fully slipped. In that case, there would be no measured slip at the beam ends, but signs of the loss of interaction would be seen in the strain distribution of the sections experiencing the
interfacial slip. The reinforced concrete beam and the steel U-section would begin to bend about their respective neutral axes. The loss of the composite behavior of the steel U-section would transfer additional strain to the tension reinforcement of the concrete beam as the two components began to bend independently.
Figure 5-3 – DK1 Applied Moment vs. Average Midspan Strain
Figure 5-4 – DK1 Strain Profiles
DK2 Strain

The overall behavior of the average measured strain at each of the various locations throughout the midspan cross section of specimen DK2 is depicted in Figure 5-5. The strain behavior of DK2 is complicated by the presence of interfacial slip between the reinforced concrete girder and the steel U-section causing the loss of composite interaction between components. The effect of the loss of interaction is most noticeable in the strain level containing gauges BB-B-F-FF. It can be seen in the figure that above an applied moment of 490 k-ft the strain at this level reverses direction and begins to move into compression as the steel U-section begins to bend about its own neutral axis.

For each load cycle, the acquired strain data was used to develop strain profiles for the cross sections at each quarter point and midspan of specimen DK2 as shown in Figure 5-6. The plotted value is an average strain for all of the gauges located at that specific height in the cross section. The figure shows that the strain profile is essentially linear for the ½ service load cycle and for the full service load cycle at midspan. However, as the applied load increases, the horizontal shear force increases, creating a loss of composite interaction due to interfacial slip. As the magnitude of the slip increases, the steel U-section bends progressively more independently of the concrete T-section. The effects are first noticed in the quarter points where the shear is higher and are eventually seen throughout the section at the maximum applied load equal to the factored load.
Figure 5-5 – DK2 Applied Moment vs. Average Midspan Strain
Figure 5-6 – DK2 Strain Profiles

I - 1/2 Service Load Step (223 k-ft)  
II - Service Load Step (446 k-ft)  
III - Factored Load Step (664 k-ft)  
* Specimen Failed at Factored Load  

- Concrete Strain  
- Steel Strain
5.1.3 Slip

The slip between the reinforced concrete beam to steel interface as well as the slip between the precast hollow core slab to steel interface was measured at each side of the beam and at each end as described in Chapter 4. The values for the slip at the third interface between the reinforced concrete beam and the precast hollow core slabs was taken as the difference between the slip measured at the other two interfaces. The data recorded from the slip measurements generated predictions about the degree of interaction between the various components of the composite system at specific interfaces. The observed slip measurements were also effective in the evaluation of the effectiveness and strength of the mechanical shear connection in specimen DK1.

Specimen DK1 did not experience any observable interfacial slip throughout the loading sequence until the point of specimen failure. The lack of independent movement between the reinforced concrete T-section and the steel U-section suggests that the shear stud size and pattern was sufficient to resist the longitudinal shear induced by the experimental loading and provide full composite action up to the point of specimen failure.

Specimen DK2 was not constructed with any time of mechanical shear connection between the concrete T-section and the steel U-section. The resistance to interfacial slip due to longitudinal shear was developed entirely from the natural bond and friction between the concrete and steel formed during casting. That inherent resistance was
overcome during the factored load cycle after reaching the service load mark but before reaching the full calculated factored load. The interfacial slip began in the back half of the beam containing quarter point Q3 and eventually progressed across midspan to the front portion of the beam. The increasing slip and consequential discontinuity in the strain profile further increased the strain in the top of the concrete T-section beyond that predicted for a fully composite section under the same applied load. The increased strain led to a premature failure due to concrete crushing prior to the mark for the calculated factored load for which the specimens were originally designed. The results from the experimental testing of specimen DK2 indicate that the natural bond between the concrete T-section and the steel U-section formed during casting is not sufficient to provide full composite interaction up to the design factored load. Although specimen DK2 did reach the service load mark without observable slip, the specimen was only loaded monotonically at a relatively slow rate for a limited number of cycles and only once to the service load mark. The results from the experimental testing suggest that the installation of mechanical connection is necessary to reach the factored load predicted for a fully composite section and is most likely necessary to reach service load under a substantial cycles.
5.1.4 Failure Mechanisms

The predicted failure mechanisms for the composite test specimens included yielding of the steel U-section and tension reinforcing and eventual crushing of the concrete in compression. Interfacial slip was considered negligible in specimen DK1 because of the mechanical shear connection but was predicted to occur in specimen DK2 constructed without shear studs. The emergence of several additional failure mechanisms was observed during the testing of the composite specimens as described in the following section.

DK1 Failure Mechanisms

The failure of specimen DK1 was due to a combination of several effects including flexural and shear cracking of the hollow core slab, longitudinal shear cracking, yielding of the steel U-section and tension reinforcing, and concrete crushing. Although there were significant observable signs of distress in the composite section prior to the application of maximum load, the failure of specimen DK1 was sudden and practically brittle. The process and order in which each of the failure mechanisms emerged are discussed.

Prior to testing specimen DK1, the presence of transverse cracking of the topping slab initiating from the grouted hollow core joints and propagating towards the reinforced
concrete beam core was observed. The cracks were typically 0.005 in. wide and were most likely formed because of settlement of the precast hollow core slabs after the temporary shoring was removed. The first signs of distress due to the application of load were not observed until the completion of the service load cycle for specimen DK1. After completion of the service load step, transverse cracks on the underside of the hollow core slabs were noticed. These cracks typically initiated from the outside bottom edge of the slabs and propagated towards the beam core as shown in Figure 5-7. A few of the cracks extended through the depth of the bottom flute and into the core of the precast slabs. The cracks ranged from .005-0.01 in. wide and were most likely the effect of imposed curvature in the composite beam due to the applied load.

Figure 5-7 – Typical transverse cracks on underside of hollow core slabs
Under the application of the factored load cycle, additional flexural cracking was observed on the underside of the hollow core slabs in addition to the initiation of a longitudinal shear crack along the hollow core slab to concrete beam interface. The first visible sign of the longitudinal shear crack emerged towards the end of the beam near quarter point Q3 at an applied moment of approximately 570 k-ft and propagated along the interface parallel to the beam line towards midspan. The crack width was measured to be 0.025-0.030 in. under an applied moment of 664 k-ft. A second longitudinal shear crack formed on the opposite side of the concrete core, again at the hollow core slab to concrete beam interface, under an applied moment of approximately 650 k-ft during the failure load cycle. Both cracks propagated parallel to the beam line along the entire length of the simply supported span. Near the location of the supports on either end, the cracks deviated from the hollow core slab to concrete beam interface at approximately a 45 degree angle and continued around the outside of the support in a semicircular fashion. The typical pattern is shown in Figure 5-8.
During the application of the failure load cycle, just beyond the factored load limit of 664 k-ft, the strain in the bottom flange of the U-beam at midspan reached the yield strain for structural steel. As the applied load was increased, yielding of the bottom flange was observed at the quarter points as well as in the tension reinforcement at the application of the failure moment of 750 k-ft.

As the moment applied to the composite beam DK1 approached the maximum value of 750 k-ft, several significant mechanisms contributed to the overall failure of the specimen. Several of the hollow core slabs failed in shear as large cracks propagated though the cores of the slab up to the interface between the hollow core slabs and the
cast-in-place topping. Figure 5-9 depicts a typical shear failure in the hollow core slabs. As cracks continued to propagate along the hollow core slab to concrete topping interface the interfacial bond was degraded and the topping slab began to delaminate from the hollow core sections. An example of the delamination observed at the location of a shear crack close to failure can be seen in Figure 5-10.

Figure 5-9 – Typical shear failure of hollow core slab
After the formation of the longitudinal shear cracks on either side of the reinforced concrete beam core, the effective flange width of the beam was significantly decreased and the compression strain in the concrete above the beam core was increased considerably. Eventually, the concrete above the beam core and in between the longitudinal shear cracks crushed due to excessive compression strain. The crushing failure initiated near the outer most applied line load on the back half of the beam containing quarter point Q3 and continued as far as the support on the far end of the beam. The longitudinal shear cracks and crushing failure can be seen in Figure 5-11.

Figure 5-10 – Delamination of the topping slab from hollow core section
The failure of specimen DK1 produced a sudden and significant loss of strength with a minimal yield plateau prior to concrete crushing. It was recorded that the specimen lost more than a one quarter of its moment resistance immediately after failure and lost two thirds of its strength before settling at its reserve strength limit. It was shown however in Figure 4-21, that the apparent reserve strength of the specimen was greater than the calculated service load of 446 k-ft even after the application of an additional load sequence following the unloading of the failure load cycle.
DK2 Failure Mechanisms

The failure mechanisms observed in the testing of specimen DK2 were similar to those that emerged in specimen DK1. Although, due to the lack of mechanical shear connectors, the addition of interfacial slip was added to the observed mechanisms of flexural and shear cracking of the hollow core slab, longitudinal shear cracking, yielding of the tension reinforcing including the steel U-section, and concrete crushing that were witnessed in the first test. The presence of slip within the composite system created a more ductile failure than that which was observed in the testing of specimen DK1.

Small transverse cracks along the top side of the concrete topping prior to the beam being loaded were observed in specimen DK2. As described in the previous section, these cracks were most likely formed because of post-cast settlement in the system after the shoring was removed. The cracks due to settlement were the only observable damage noticed in the specimen after the application of the ½ service loading cycle. As the loading was increased to the full service load, transverse cracks along the underside of the hollow core slabs were observed in a majority of the precast sections. These cracks were similar to those noticed in the previous test and were most likely due to increased curvature in the beam under the applied load.

Just beyond the application of the service load, significant and consistent slip between the concrete girder and the steel U-section was observed at an approximate load of 490 k-ft. The slip first occurred on one end of the beam only. Once the natural bond
between the concrete and steel was broken, shear sliding occurred as the concrete T-beam, still composite with the hollow core slabs, moved as a rigid body separately from the steel U-section. The initial separation can be seen in Figure 5-12.

![Figure 5-12 – Interfacial Slip between Concrete Girder and Steel U-section](image)

As the interfacial slip amplified with the increased applied load on the composite section, the composite concrete T-beam and steel U-sections began to behave independently, bending about their respective neutral axes. The resulting section was less stiff without full composite action and experienced larger deformations and a reduced ultimate capacity than the previous test. Under the larger deformations, more significant cracking on the underside of the hollow core slabs was observed as shown in Figure 5-13.
As cracks propagated through the hollow core slab to the interface with the cast-in-place topping, delamination of the topping slab was also observed.

![Figure 5-13 – Large Transverse Crack on Underside of Hollow Core Slab](image)

The variance in stiffness between the steel U-section and the concrete T-beam created the initiation of an additional failure mechanism. As the steel U-section was less stiff, it began to deflect more than the reinforced concrete T-section. The U-section began to separate from the T-beam but was resisted by the embedment of the U-section’s top return in the concrete girder. An example of this separation can be seen in Figure 5-14. As the slip increased further and the independent bending amplified, the U-section began to open up. The vertical webs rotated about their connection with the bottom
flange. The motion was resisted by the steel straps welded to the top returns but was still visible as shown in Figure 5-15.

Figure 5-14 – Separation between the Hollow Core Slab and Steel U-section
The formation of longitudinal shear cracks was observed simultaneously with the initiation of interfacial slip. Similar to the failure of specimen DK1 a single crack was observed along one side of the beam with a second crack forming later along the opposite side along the hollow core slab to concrete beam interface. The shear cracks isolated an effective section width along the entire length of the beam. The ultimate failure of composite specimen DK2 was due to concrete crushing above the concrete girder core in between the longitudinal shear cracks.
5.2 Moment Curvature Comparison

The results of the analytical study were used to predict the response of the composite test specimens based on several fundamental assumptions as described in Chapter 3. A layered sectional analysis was used to generate the graph in Figure 5-16, plotting applied moment vs. average observed midspan deflection. The predicted results shown in the figure are the calculated nominal moment resistance of the composite system without the application of a reduction factor phi. The ratios of the observed strain to the predicted strain for the top of the concrete topping and the bottom of the steel U-section are calculated in Table 5-1. As seen in the table, up to the application of the factored load cycle the ratios have a consistent and average value of 0.9. This would suggest that a typical reduction factor of 0.9 applied to the calculated nominal moment resistance would match well with the observed behavior of the system up to the maximum factored load applied to the section. However, beyond the factored load, the predicted response becomes inconsistent. The inconsistency is most likely due to the formation of longitudinal shear cracks that were not considered as a potential failure mechanism in the analytical study. The longitudinal shear cracking significantly reduced the effective flange width of the section changing the section properties and therefore shifting the observed behavior away from the response predicted by the analytical model.
Figure 5-16 – Observed Midspan Deflection vs. Predicted M-Φ Response
A graph comparing the predicted strains in the top of the concrete and bottom of the tension reinforcing steel with the observed strains from the experimental testing of specimen DK1 in the same locations is shown in Figure 5-17. It can be seen from the figure that the strain predictions match closely until the initiation of the first longitudinal shear crack at approximately 570 k-ft. As the longitudinal cracks change the section properties of the composite beam, the validity of the analytical model diminishes. It is important to note however that the prediction of the first yield of the section as well as the ultimate capacity of the section are relatively accurate despite the inconsistencies in the predicted strain in the bottom steel at failure. The strain in the bottom steel is greatly over predicted at failure when compared to the observed behavior. This is most likely due to a downward shift in the neutral axis occurring when the effective flange width is lost after the formation of the longitudinal shear cracks. As the neutral axis moves downward the strain in the steel is greatly reduced if full composite action and a linear strain profile are still assumed.
Figure 5-17 – Applied Moment vs. Strain Comparison
5.3 Design Procedures

The initial design procedures implemented in the design of the two test specimens are described in Chapter 3. The results of the experimental testing were used to evaluate the current design procedures and the assumptions made in the initial analytical study. The assumption that a linear strain profile throughout the depth of the section could be achieved with proper mechanical shear connection to develop full composite action was validated through the experimental results. The strain profiles presented in chapter 4 show that during initial stages of loading, test specimen DK1 exhibited full composite behavior with a linear strain profile. However, the onset of an unpredicted failure mechanism prior to reaching the full factored load changed the specimen section properties, and consequently the strain profile, despite the adherence to full composite action. Additionally, the assumption that the top flute of the hollow core slab can be considered effective in the compression flange of the concrete T-section is confirmed by the experimental results through accurate predictions of the initial stiffness of the section.

It can be inferred from the observed strain profiles that the assumption of full composite interaction is accurate up to the application of the full factored load. The experimental results showed an essentially linear strain profile across the steel U-section to concrete girder interface as well as along the hollow core slab to concrete topping interface. The experimental results confirmed a second assumption of the inclusion of
hollow core top flute in the effective width of the compression flange of the composite section by showing accurate predictions of the section’s initial stiffness. The accurate computation of the dimensions of the effective section by means set forth in present day codes as described in Chapter 3 was also confirmed by the results showing strong correlation of the predicted initial stiffness with the recorded experimental stiffness as well as similar compressive strain values recorded along the width of the top of the composite section until the formation of unpredicted longitudinal shear cracking.

The experimental results also exposed some shortcomings of the current design procedures. It was observed during the laboratory testing that an unforeseen failure mechanism occurred prior to the predicted ultimate failure mechanism of concrete crushing. Longitudinal shear cracking along the hollow core slab to concrete beam interface formed during the factored load cycle. The longitudinal shear cracking was observed in both test specimens but was not predicted or designed for in the current design procedures. The onset of longitudinal shear cracking changed the section properties of the composite beam by reducing the effective flange width. The altered section properties shifted the strain profile such that the ultimate failure of the specimen was considered a compression controlled failure as opposed to the tension controlled failure of the original design. The current design procedures also did not include an explicit calculation for the size and layout of the mechanical shear connection. The shear stud layout used in the construction of specimen DK1 was designed based on empirical
knowledge of the designer. A consistent design procedure for the computation of the horizontal shear demand and desired mechanical connection to resist that demand would be beneficial, particularly for situations outside of the typical design scenarios.
Chapter 6

Recommendations and Conclusions

On the basis of the results from the analytical and experimental analyses as well as the comparisons made between the predictions of the layered section analysis and the current design procedures with the observed results from the experimental testing, several conclusions can be drawn. In addition to those conclusions, several recommendations involving the current design procedure and construction processes are made in order to establish a more complete design methodology incorporating the findings of this experimental program.

As discussed in Chapter 5, the current design procedure does not account longitudinal shear cracking of the concrete T-section as a potential failure mechanism. Longitudinal shear cracking was observed during the testing of both specimens. The longitudinal shear cracking failure mechanism has a significant impact on the ultimate behavior of the section as the neutral axis shifts in such a way to produce a compression controlled failure of the specimen. According to the design methodology laid out by ACI, compression controlled failures are penalized by larger reductions in ultimate allowable load. To maintain a tension controlled failure and avoid any unnecessary penalties in ultimate allowable design strength, a means of reinforcing the section to prevent such longitudinal shear cracking should be implemented in the construction
process. It is possible that the conservative measures taken during construction of the test specimens in blocking the cores of the hollow core slab to prevent the flow of concrete during casting created an unrealistic weak point in the specimen that aided in the formation of the longitudinal shear crack mechanism. The actual field construction of the innovative composite floor system allows some volume of concrete to flow into the cores during construction that may help prevent the formation of longitudinal shear cracking. Supplementary resistance can be achieved by adding transverse reinforcing bars across the vertical plane created by the interface between the webs of the steel U-section and the concrete T-section.

In addition to the changes made to acknowledge the potential for a longitudinal shear cracking failure mechanism, the current design procedures need to be modified to include an explicit design procedure for the mechanical shear connection required to resist the maximum horizontal shear demand due to the applied loads. Two general methods used to determine the horizontal shear demand on a composite section are described below.

The first method is an ultimate strength method based on the maximum allowable force to be transferred by the steel section to the concrete beam. The ultimate strength method is similar to the suggested design procedures for shear connectors described in AASHTO LFRD Bridge Design Specifications. The second method is based on simplified elastic beam behavior which utilizes flexural beam theory to relate the vertical
shear acting on a section to the horizontal shear demand. The second approach is more labor intensive, but is used to determine variable spacing of shear connectors along the length of the beam if desired.

The ultimate strength method can be compared to the AASHTO LRFD Bridge Design Specifications equation 6.10.10.4.2-3. The maximum allowable force that can be transferred by the steel U-section is calculated as the yield stress of the U-section multiplied by the total area of the section as shown in Equation 6-1. The region under consideration in this equation is taken as the length of the beam between the point of maximum positive moment and the point of zero moment. The number of shear studs needed to resist the computed horizontal shear demand in the region under consideration should be distributed evenly over that region.

\[ P' = F_y \times A_{s\, u-section} \]

Equation 6-1

The simplified elastic beam behavior method of calculating the horizontal shear demand relates the vertical shear applied to a given differential length to the horizontal shear across an interface along that differential length as shown in Figure 6-1. From statics of the section of beam shown in the figure, Equation 6-2 is developed to determine the shear stress across the interface where the shear studs will connect the steel.
U-section to the concrete beam. The horizontal shear demand of the beam is then determined by integrating the product of the shear stress and the area over which it acts along the entire length of the beam as described in Equation 6-3. The integral is generally approximated by summing the individual shear demands over small sections taken from the overall length of the beam for the region under consideration. Because the horizontal shear demand changes with the applied vertical shear at a given section, it is sometimes advantageous to vary the distribution of shear studs. The simplified elastic beam theory method allows for the design of variable shear stud spacing based on the changing shear demand along the length of the composite beam.

Figure 6-1 – Simplified Elastic Beam Behavior
The maximum allowable resistance to the calculated horizontal shear demand of an individual shear stud can be calculated from the expression given in Equation 6-4. Often a desired shear stud diameter is chosen based on material and installation costs. The required number of shear studs to resist the computed horizontal shear demand is then calculated by simply dividing the total horizontal shear demand for the region under consideration by the maximum allowable resistance of a single shear stud. Typically the studs are spaced evenly along the length of the beam in the region under consideration but can be variably spaced if desired.

\[ Q_n = 0.5A_{sc}\sqrt{f'_cE_c} \leq A_{sc}F_u \]  
Equation 6-4

Table 6-1 gives the results of both of the horizontal shear demand methods for the composite beams used in the experimental testing of this phase of the program. It can be seen from the results in the table that the shear stud layout used in specimen DK1 did not meet the requirements calculated by either one of the two methods described. The 10 in
spacing used in specimen DK1 was only marginally larger than the requirements calculated by the ultimate strength method and slightly more so than the spacing determined from the elastic beam theory method. It is probable then, that there were some failures of shear studs in regions of higher shear demand, as discussed previously in the discontinuities in the strain profiles near quarter point Q1. Potential overstrength of the shear studs due to the conservatism built in to the equation for the allowable shear resistance of the single stud could have prevented a global failure that would have resulted in observable slip between the composite sections.

<table>
<thead>
<tr>
<th>Method</th>
<th>Horizontal Shear Demand (kips)</th>
<th>Number of Studs Required</th>
<th>Stud Spacing (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Strength</td>
<td>900</td>
<td>32</td>
<td>9.5</td>
</tr>
<tr>
<td>Elastic Beam</td>
<td>1040</td>
<td>38</td>
<td>8.5</td>
</tr>
</tbody>
</table>

Either method of calculating horizontal shear demand is sufficient to use in determining the required size, number, and layout of shear stud connectors for the innovative composite floor system. As discussed previously, the use of proper shear connection in the composite beam is necessary to achieve consistent predictable behavior and desired ultimate moment resistance. The experimental results did not exhibit observable behavior suggesting the empirical methods used to design the mechanical shear connection were inadequate. However, after working through both horizontal shear
demand methods and calculating the allowable mechanical connection for design, the shear stud layout used in test specimen DK1 was determined to be unsatisfactory.

An area for potential improvement in the construction of the innovative composite system involves the amount of tensile reinforcement used section. Even if the longitudinal shear crack failure can be prevented or sufficiently resisted, the section is still over reinforced for efficient use of the structural steel. The presence of substantial reserve capacity is comforting for most scenarios, but it is also an indicator that there is likely room for improvement in the system’s design. The most probably location to reduce the area of tensile reinforcement is the steel U-section, especially because the strength of the exposed steel U-section is compromised in design fire conditions. By reducing the thickness of the steel U-section, the typical system geometries can be maintained, while improving the overall efficiency.

The addition of analytical methods to determine the horizontal shear demand on the section based on the applied loading conditions, coupled with design equations for the allowable horizontal shear resistance for welded stud connections will help provide a more complete design methodology for the innovative composite floor system. Furthermore, the addition of transverse reinforcement to resist the longitudinal shear cracking mechanism observed in the experimental testing will provide a more favorable and predictable failure mechanism, while the efficiency of the system can be optimized.
by reducing the thickness of the steel U-section to provide a more beneficial reinforcement ratio. The recommendations made in this chapter based on the results of analytical and experimental analyses will help further develop the innovative composite floor system and provide an improved design resisting previously unforeseen failure mechanisms. The result is a safer, more efficient system constructed on the basis of a more comprehensive design methodology.
Chapter 7

References


American Concrete Institute. Building Code Requirements for Structural Concrete. 318-05.


Firberg, B. F. "Combined Form and Reinforcement for Concrete Slabs." Journal of the American Concrete Institute (1954): 697-716.


Flexural Test Summary Sheet

Test ID : DK 1
Test Date : 5/6/2009
Test Location : Constructed Facilities Laboratory (CFL), North Carolina State University
Tested By : Meade Willis, E.I. and Adam Amortnont, E.I.
Sponsor : RB 3°C

Test Parameters
Configuration : Simple supports with four equal loads at the 1/5 points.
Span : 27 ft.
Loading Rate: 0.033 in/min
Load Cycles: 1/2 Service (223 k-ft), Service (446 k-ft), Factored (664 k-ft), Failure(750 k-ft)

Steel U-Beam
Height : 4.625 in. (nominal)  Steel Grade : ASTM A572
Thickness : 0.25 in. (nominal)  Yield Strength : 50 ksi (nominal) (measured)
Shear Studs: 0.75 in. dia. at 10 in. spacing  Tensile Strength : 65 ksi (nominal) (measured)

Hollow Core Slab
Depth : 8 in.  Concrete Compressive Strength : 6000 psi
Width : 48 in.  Reinforcement : 7 wire low relaxation strand
Length : 36 in.  Reinforcement Grade : ASTM A 416
Min. Cover : 1.5"

Concrete Topping Slab/ Beam Fill
Depth : 2.0 in  Concrete Compressive Strength : 3900 psi
Width : 87 in.  Reinforcement : 6 x 6 - W2.9 x W2.9
Length : 30 ft.  Reinforcement Grade : ASTM A185
Top Cover : 0.75-1.0 (in.)

Test Results
Maximum Applied Moment : 750 k-ft
Maximum Deflection Values: 1/2 Service (0.75 in.), Service (1.5 in.),
Factored (2.4 in.), Failure(3.75 in.)
Failure Description : Longitudinal Shear Cracking at the Hollow Core Interface,
Concrete Crushing, and Yielding of the Steel U-beam and Positive Moment Reinforcing

Test Observations
• No visible signs of distress under 1/2 service load step
• Cracking observed on underside of hollow core slabs under service load
• Formation of first longitudinal shear crack at hollow core interface at 570 k-ft
• Formation of second longitudinal shear crack at hollow core interface 650 k-ft
• Failure occurred on the half of the beam coinciding with quarter point 3
• Longitudinal shear crack widths ranged from 0.025-0.030 in at maximum load
Figure 0-1 – Applied Moment vs. Deflection
Figure 0-2 – DK1 Applied Moment vs. Average Deflection
Figure 0-3 – DK1 Applied Moment vs. Strain
Figure 0-4 – DK1 Applied Moment vs. Midspan Strain

Section A-A
Figure 0-5 – DK1 Applied Moment vs. Slip

Note: 0.58” maximum slip recorded
Figure 0-6 – DK1 Applied Moment vs. Average Slip

Note: 0.58" maximum slip recorded
Flexural Test Summary Sheet

Test ID : DK 2
Test Date : 5/12/2009
Test Location : Constructed Facilities Laboratory (CFL), North Carolina State University
Tested By : Meade Willis, E.I. and Adam Amortnont, E.I.
Sponsor : RB2C

Test Parameters
Configuration : Simple supports with four equal loads at the 1/5 points.
Span : 27 ft.
Loading Rate: 0.033 in/min
Load Cycles: 1/2 Service (223 k-ft), Service (446 k-ft), Factored/Failure (664 k-ft)

Steel U-Beam
Height : 4.625 in. (nominal)  
Thickness : 0.25 in. (nominal)  
Shear Studs: None  
Yield Strength : 50 ksi (nominal)  
Tensile Strength : 65 ksi (nominal)

Hollow Core Slab
Depth : 8 in.  
Width : 48 in.  
Length : 36 in.  
Concrete Compressive Strength : 6000 psi  
Reinforcement : 7 wire low relaxation strand  
Reinforcement Grade : ASTM A 416  
Min. Cover : 1.5”

Concrete Topping Slab/ Beam Fill
Depth : 2.0 in.  
Width : 87 in.  
Length : 30 ft.  
Concrete Compressive Strength : 3900 psi  
Reinforcement : 6 x 6 - W2.9 x W2.9  
Reinforcement Grade : ASTM A185  
Top Cover : 0.75-1.0 (in.)

Test Results
Maximum Applied Moment : 666 k-ft
Maximum Deflection Values: 1/2 Service (0.75 in.), Service (1.5 in.),  
Factored (4.3 in.), Failure(4.5 in.)
Failure Description : Longitudinal Shear Cracking at the Hollow Core Interface,  
Concrete Crushing, Slip Between U-Section and Concrete Beam,  
and Yielding of the Steel U-section and Positive Moment Reinforcing

Test Observations
- No visible signs of distress under 1/2 service load step
- Cracking observed on underside of hollow core slabs under service load
- Initiation of slip between u-section and concrete beam at 490 k-ft
- Formation of first longitudinal shear crack at hollow core interface at 490 k-ft
- Failure occurred on the half of the beam coinciding with quarter point 3
- Significant slip observed between u-sectoin and concrete beam up to 1.1 in.
Figure 0-7 – DK2 Applied Moment vs. Average Deflection
Figure 0-8 – DK2 Applied Moment vs. Average Deflection
Figure 0-9 – Applied Moment vs. Strain
Figure 0-10 – Applied Moment vs. Average Strain
Figure 0-11 – Applied Moment vs. Slip

Note: 1.12" maximum slip recorded
Figure 0-12 – Applied Moment vs. Average Slip

Note: 1.12" maximum slip recorded