ABSTRACT

KENNEDY, KURTIS JAMES. Investigation and Design of Innovative Composite Shallow Floor Framing System. (Under the direction of Dr. Emmett A. Sumner III.)

To meet the needs of an ever changing design and construction industry Diversakore® has developed an innovative cast in place composite shallow floor framing system, the Versa :T:® which offers improved overall ease and speed of design and construction. The system is comprised of a U-shaped steel plate, hollow core planks, and a cast-in-place concrete beam and topping slab to tie the system together. The sides of the girder above the U-shaped steel plate are formed to create the inverted T-beam section and provide an end bearing surface for the hollow core planks. The U-shaped steel plate supports the hollow core planks during construction, and provides stay in place formwork for the cast in place beam and slab. The cast-in-place beam and topping slab is monolithically cast, engaging the planks and forming a composite T-beam. Shoring is required to support the U-shaped steel plate while hollowcores are placed before the concrete beam and slab are poured. A research program is currently ongoing at the Constructed Facilities Laboratory at North Carolina State University to evaluate the ultimate strength and serviceability performance of the composite floor system. The experimental and analytical investigation includes full-scale tests of representative sub-assemblages and utilizes a layered sectional analysis to predict the behavior of the system. Limit state analysis is conducted and compared with experimental results to evaluate the behavior of the system. Provisional design procedures have been developed for the design and construction of the shallow floor framing system.
Investigation and Design of Innovative Composite Shallow Floor Framing System

by
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DEDICATION

This thesis is dedicated to those in my life who continually challenge me to be the best that I can in all that I do.
BIOGRAPHY

Kurtis James Kennedy was born March 15th, 1987 in Goldsboro, NC. After living 13 years in North Carolina he moved to Germany and then to California. After graduating from high school he moved back to North Carolina to attend North Carolina State University pursuing an undergraduate degree in Civil Engineering. He decided to pursue a Master’s of Science in Civil Engineering upon finishing his undergraduate studies. He plans to pursue a career as a professional engineer upon graduation from the Master’s program.
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1 INTRODUCTION

1.1 Overview

Composite framing members have been developed and used in industry for more than 100 years in the United States and around the world. Concrete encased steel beam box members were the earliest forms of composite construction in the U.S. utilizing concrete as fire protection for steel frame members. A significant increase in strength of such members was recognized throughout the industry prompting continued research into the behavior of composite framing members. As a result more conventional composite framing members were developed, pushing research further into exploring horizontal shear forces that develop at interfaces between steel and concrete. The mechanical shear connector, commonly known as the shear stud, was developed in the 1950’s to enhance the horizontal shear capacity of such composite members. Ribbed metal decking was invented also, as a stay in place formwork for casting concrete. Over time as the use of mechanical shear connectors and ribbed metal decking became part of mainstream design and construction conventional composite framing systems were developed, utilizing standard hot rolled wide flange section with cast-in-place concrete poured over ribbed metal decking connector using shear stud connectors. As the need for efficiency increased within the civil engineering industry slim floor systems were developed, designed with alternative steel sections and stay-in-place formwork type components, such as hollowcore slabs. As limitations were discovered
different structural configurations were developed eventually leading into the development of precast shallow floor framing systems.

The Diversakore® slim floor framing system is one example of such a slim floor framing system. Utilizing the Versa :T:® beam, the system maintains a slim profile while providing adequate strength required for typical frame structures. The Versa :T:® beam consists of a hot rolled steel section bent into a u-shape upon which hollowcore slabs are placed, resting on the return flanges of the steel plate section. Mechanical shear connectors are welded to the interior of the steel section. A monolithic cast-in-place concrete beam and slab is poured tying the composite system together. The cross section detail of this framing system is shown in Figure 1.1.

![Figure 1.1 - Diverskore Slim Floor Framing System Cross Section Detail](image_url)
1.2 Purpose

The civil engineering industry is in constant need of more efficient and cost effective designs that can be rapidly constructed. Nearly all modern structures are constructed utilizing composite framing members creating demand for lighter, stronger framing members. The Diversakore® framing system proves advantageous as designs allow for shallow member depths which reduce overall height, weight, and essentially cost of structures. The configuration of the Diversakore® framing system inherently lends itself as a clean, safe working surface enhancing the rapid construction process as shown in Figure 1.2. Although the advantages of utilizing this system can be seen and much research has been done, more research is needed. Various design limit states such as flexural and vertical shear strength of the slim floor system under construction and ultimate loading must be analyzed along with a performance and design evaluation of the system.

![Figure 1.2 - Diversakore® Slim Floor System Field Photo](image)
To fulfill the need for this research North Carolina State University has undertaken a three phase research program to evaluate the performance and behavior of the slim floor system. The first phase of the program was conducted by Meade Willis (2009) during which he conducted an extensive literature review and initial investigation of the flexural design limit state of the Diversakore® system. Adam Amortnont (2010) continued the second phase by investigating a precast Diversakore® framing system. He studied the effects of camber and prestressing on the performance and behavior of the composite Diversakore® system. The final phase of the program consists of further investigation of the flexural design limit state with special consideration on observed failure modes from previous tests such as horizontal shear strength and longitudinal shear strength as well as investigation of the vertical shear design limit state. Included within the final phase of the research program is the development of an analysis and design procedure to be used for future design reference or further research of the slim floor framing system.

1.3 Objectives and Scope

1.3.1 Objectives

The overall objective of the research undertaken was to conclude the research program on the behavior of the Diversakore® composite slim floor framing system. The evaluation of performance of the system was planned to be achieved by setting the following objectives:
1. Investigate the design limit states of vertical, horizontal, and longitudinal shear through experimental testing of sections.

2. Develop analytical models to predict member responses and evaluate design limit states investigated to compare with experimental data.

3. Develop design and construction guidelines for the framing system based upon comparisons of experimental behavior and analytical models.

1.3.2 Scope

A three phase program has been undertaken at NC State University. The first phase of the program consisted of an extensive literature review and initial investigation of the flexure design limit state of the Diversakore® system. The second phase was focused on the effects of camber and prestressing on the performance of a precast Diversakore® framing system. The final phase of the program consists of further investigation of the flexural design limit state with special consideration on observed failure modes from previous tests such as horizontal shear strength and longitudinal shear strength as well as investigation of the vertical shear design limit state. To complete the research program, analysis and design provisions are developed and proposed for use by the design engineers. To accomplish the objectives set forth for this research program all objectives of this study were carried out.
Experimental testing was completed and compared to analytical models. Design guidelines based upon accepted codified specifications were developed.

1.4 Thesis Content

The content of this thesis is separated into six chapters. Brief descriptions of each chapter and extra content are given below:

Chapter 2 is a review of the current state of knowledge regarding composite framing. The literature review mainly focuses on the overall behavior of shallow floor framing systems and also addresses the limit states of horizontal and longitudinal shear. The need for future research is included at the end of the chapter.

Chapter 3 contains detailed descriptions of the experimental portion of the research. This chapter details the specimen design, construction, and testing of the two DK specimens. Experimental data collected during testing is presented while observed behavior and failure modes are also described within this chapter. To conclude the chapter a summary of results is presented.

Chapter 4 presents the analytical models used for comparison to experimental data collected during the testing of the specimens. Layered sectional analysis is described for the two specimens tested along with design limit state evaluations. Experimental results are
compared with analytical models to aid in the development of design and construction guidelines.

Chapter 5 includes the unabridged design procedure for the Diversakore® composite shallow floor framing system.

Chapter 6 is a summary of the research project which includes conclusions and recommendations drawn from the completion of the experimental and analytical investigation of the composite framing system.

Appendices are included at the end of the thesis which contain a full design example following the proposed design procedure and results from experimental testing of two composite sections.
2 LITERATURE REVIEW

2.1 Introduction

This chapter is a brief overview of the current state of knowledge of slim floor composite steel and concrete framing systems. Included in this chapter are the sections titled Background, Slim Floor Framing Systems, Limit States, and Need for Research. The Background section details the scope of Diversakore® beam studies conducted with regard to slim floor framing. Following in the Slim Floor Framing Systems section are overviews of previous studies on the Diversakore® composite framing system conducted by Lindsey et al. (2002), Willis (2009), and Amortnont (2010). Next the Limit States section will detail specific limit states that are considered during the design of conventional composite or composite slim floor framing systems, particularly the Diversakore® composite floor system. Finally the Need for Research section addresses specific areas where future research should be focused in order to better understand slim floor framing systems to a higher degree.

2.2 Background

Composite framing has become a mainstream choice for cost effective rapid design and construction of new structures around the world. (ASCE 2002) Recent developments in composite framing have been pushing the industry toward shallow floor composite framing systems in which efficiency is increased, floor heights and overall structure heights are reduced, and design and construction costs are decreased. Previous studies conducted by
Lindsey et al. (2002), Willis (2009), and Amortnont (2010) focused on an innovative slim floor framing system developed by Diversakore®. Lindsey et al. (2002) conducted the pilot study for this framing system at the Georgia Institute of Technology. Following Lindsey et al. (2002), Willis (2009) began the research program at North Carolina State University studying the Diversakore® slim floor framing system. Amortnont (2010) conducted subsequent research intended to expand upon the work and findings reported by Willis (2009). Both Willis (2009) and Amortnont (2010) performed analytical and experimental investigations as part of their research projects. Referenced from Willis (2009) and Amortnont (2010), who both conducted extensive literature reviews, the earliest applications of composite framing in the United States were concrete encased steel beams where concrete encasement served as fire protection for the steel section. Design specifications were appended to allow higher stresses within composite members in 1936 as a result of observed increases in strength and stiffness of composite members compared to bare steel members. (ASCE 2002) During the next 40 years two major developments were introduced into the composite design and construction industry which included the invention of the welded headed shear stud connector and light gauge stay in place steel decking. Design and construction of composite structures has subsequently become more efficient and cost effective due to the composite interaction between steel components and concrete. Over time conventional composite framing geometries, hot rolled steel wide flange sections with stay in place metal decking and cast-in-place concrete slab connected on top with mechanical shear studs, have evolved into lighter, slimmer, and more efficient composite configurations.
2.3 Slim Floor Framing Systems

Diversakore® has developed one such example of a slim floor composite beam configuration, from which three experimental programs have been conducted. The first research program, conducted by Lindsey et al. (2002) from the Georgia Institute of Technology was done with the objectives of developing and verifying a design methodology and observing the fire resistance capabilities of the composite system. Follow-up research was conducted by Willis (2009) with specific focus on validating prior test results and examining various design limit states. Amortnont (2010) then conducted further research with precast composite beams with specific focus on the design limit states of horizontal shear and longitudinal shear. Slim floor framing systems behavior has been previously researched by Leskalä (1993), Fontana and Borgogno (1997), Leskalä (1997), and Ma and Mäkeläinen (2006). Research was focused on structural behavior and design criteria of the slim floor framing systems.

The first testing of the Diversakore® slim floor framing system was done by Lindsey et al. (2002) from the Georgia Institute of Technology. Three flexural tests were conducted and analyzed. The cross section of each flexural test consisted of a steel u-shaped beam with hollowcore units resting on the return flanges bound together by a monolithically cast concrete beam and slab. Figure 2.1 depicts the general view of the composite structure tested. Several numerical studies predicting the behavioral response when test specimens were subjected to ASTM E 119 fire loads were completed using ABAQUS. For all three tests it was found that experimental and analytical ultimate flexural strength values were
similar. Calculated deflections and flexural capacities were similar to those measured during the testing of each flexural specimen. However, it was stated by Lindsey, Leon, and Kim that additional amounts of embedded transverse and longitudinal reinforcing bars may be needed to ensure that fire resistance of the structure is to specification. Recommended research included additional full scale tests of composite beams with the objective of verifying the performance and behavior of the system.

Figure 2.1 - General View of Georgia Institute of Technology Composite Beams (Lindsey et al. 2002)

Willis (2009) initiated the ongoing research program by investigating the flexural behavior of the Diversakore® shallow floor framing system at the Constructed Facilities Laboratory at North Carolina State University. Willis (2009) performed an analytical study in order to validate the current design procedures and assumptions used to design the composite floor system under investigation. The accompanying experimental program
consisted of 2 full scale flexure tests of representative sub-assemblages comprised of steel u-shaped stay in place girders, precast hollowcore planks, and a cast-in-place concrete beam and topping slab. Sudden and brittle failures occurred at a reduced ultimate capacity of the specimen due primarily to reduction in effective compression flange and eventual crushing of concrete in extreme compression fibers. An unforeseen failure mechanism, longitudinal shear cracking, was observed and is unaccounted for in the current design procedures. Although the assumptions of full interaction and a linear strain profile are verified the longitudinal shear cracking mechanism changes the section properties thus inherently nullifying the previous assumptions. Recommendations were presented by Willis (2009) to develop an analysis and design procedure to account for the unforeseen limit state of longitudinal shear cracking. The current design procedure was found to be unable to correctly design the mechanical shear connection required to resist the maximum horizontal shear demand. Willis (2009) presented various methods to properly account for horizontal shear demand and horizontal shear design. Two suggested methods were: Ultimate Strength Method and Simplified Elastic Beam Behavior Method.

Amortnont (2010) continued research work on the Diversakore® shallow floor framing system. Amortnont (2010) evaluated the ultimate strength and serviceability performance by examining four specimens comprising of composite precast prestressed concrete-steel girders, precast hollowcore planks, and a cast-in-place concrete topping slab. Layered sectional analyses were performed for each of the test specimens in order to predict behavior and compare with results from experimental testing. Generated load deflection
models were compared to actual behavior of the composite members while individual failure modes were analyzed in an attempt to develop guidelines for design and construction of the system under investigation. Brittle failures were observed in both precast girders and both composite floor sub-assemblages. Ultimate moment capacity was not achieved for any of the specimens under investigation. All specimens failed due to a combination of failure modes which included steel yielding, concrete crushing, flexural shear cracking, shear connection failure, and longitudinal shear cracking. Primary failure behaviors observed were concrete crushing and shear connection failure. Following analysis and comparison of results with model predictions it was determined that the horizontal shear connection between the steel u-shaped plate and precast concrete core and longitudinal shear connection between hollowcore planks and in-situ concrete fill between hollowcores and precast core to be inadequate to achieve predicted strength. Further investigation was recommended to analyze the design limits states of longitudinal and horizontal shear failures and to conduct a parametric study to consider changes in different design parameters.

2.4 Limit States

Individual design limit states have been researched in an attempt to understand the interaction between composite components and improve the performance of the composite framing system. Horizontal shear strength and longitudinal shear strength limit states are the primary focus of this research.
Hofbeck et al. (1969) conducted a study on shear transfer in reinforced concrete structures. There were six objectives of the study: determine if cracking prior to testing affected the shear transfer strength, determine the effects of reinforcement size, arrangement, and yield strength on shear transfer strength, determine concrete strength effects on shear transfer length, investigate “dowel action” influence on shear transfer strength, examine the validity of “shear friction” model in cases when cracks pre-exist along the shear plane, and “attempt to relate the shear transfer strength measured in push-off tests to the compressive and tensile strengths of concrete and steel.” Thirty-eight push-off tests were completed, with some specimens containing a pre-existing crack while others did not contain a pre-existing crack along the shear plane. Failures were characterized by compression spalling and diagonal tension cracking in the vicinity of the longitudinal shear plane. Reinforcement amount and longitudinal shear strength were not found to have a directly proportional relationship for the specimens containing a crack prior to testing. It was found that the relationship between longitudinal shear strength and the reinforcement parameter, $\rho f_y$, was not affected by the way that the reinforcement ratio was varied. It was observed that reinforcement in initially uncracked specimens was subjected to direct tension while reinforcement in initially cracked specimen was subject to more shearing forces. Being subjected to shearing forces caused a “dowel action” relationship between longitudinal shear reinforcement and the concrete. General conclusions developed include that ultimate shear transfer strength decreases and slip increases at all load levels when a pre-existing crack is present along the shear plane. The reinforcement parameter, $\rho f_y$, is a critical parameter for determining shear transfer strength. “Dowel action” played a vital part in reinforcement of...
specimens containing a pre-existing crack while it does not contribute significantly to the reinforcement of specimens which were initially uncracked.

Johnson (1970) presents design equations for longitudinal shear in composite beams. Neither AISC nor ACI 318-63 addressed longitudinal shear stress present in flanges of concrete slabs of composite T-beams. Johnson developed an ultimate strength design method for transverse reinforcement for beams tested by Hofbeck, Ibrahim, and Mattock and 60 beams tested at Cambridge University. Two conclusions underlie the design methodology. The first conclusion states that all transverse reinforcement contributes to the longitudinal shear strength regardless of vertical location of reinforcement in the concrete slab. The second conclusion states that when determining longitudinal shear strength of a composite beam longitudinal bending does not need to be accounted for. According to Johnson the amount of transverse reinforcement needed is determined using Equation 2.1 and Equation 2.2.

\[ \rho f_y \geq 1.26 v_u - 3.8 (f_c)^{1/2} \]  \hspace{1cm} \text{Equation 2.1}

\[ \rho f_y \geq 80 \]  \hspace{1cm} \text{Equation 2.2}

Where:

\[ \rho = \text{total transverse reinforcement per unit area intersecting the shear plane, } \text{in}^2/\text{in}^2 \]

\[ f_y = \text{yield strength of transverse reinforcement, psi} \]
\[ v_u = \text{mean ultimate longitudinal shear stress on a possible plane of longitudinal shear failure, psi} \]
\[ f'_c = \text{ultimate concrete compressive strength, psi} \]

Another requirement stated by Johnson is that bottom reinforcement within the concrete slab should be greater than a minimum given by Equation 2.3 and Equation 2.4.

\[ \rho f_y \geq 0.63v_u - 1.9(f'_c)^{1/2} \quad \text{Equation 2.3} \]
\[ \rho f_y \geq 40 \quad \text{Equation 2.4} \]

Stress concentrations develop at the lower portions of the face of the concrete slab requiring excessive transverse reinforcement in those areas. The transverse compression associated with the longitudinal shear reinforcement was shown to enhance the slab shear strength through experimental testing.

El-Ghazzi et al. (1976) developed a method to calculate the amount of transverse longitudinal shear reinforcement needed to cause longitudinal shear failure to occur at the same time as ultimate flexural failure within conventional steel concrete composite beam floor systems. He presented a design chart based on estimates of transverse normal stresses that require reinforcement. El-Ghazzi et al. (1976) determined that slab width and shear span of the composite beam are primary parameters affecting the longitudinal shear capacity of conventional composite beam floor systems. El-Ghazzi developed the ultimate strength design method for longitudinal shear based on the estimation of \( \rho f_y \) which allows longitudinal cracks to develop at the same time as ultimate flexural capacity is achieved. Utilizing
Mohr’s circle and Cowan’s criterion of failure a relationship between $\rho f_y$, $f'_c$, $L_v$, and $b$ was constructed given in Equation 2.5.

$$ (\rho f_y)_u = \frac{f'_c}{1 + \left[2L_v/b\right]^2} $$  

Equation 2.5

Where:

- $(\rho f_y)_u$ = transverse stress produced by transverse slab reinforcement for the case of full flexural capacity, psi
- $f'_c$ = concrete compressive cylinder strength at 28 days, psi
- $L_v$ = shear span, in.
- $b$ = width of concrete slab, in.

The proposed method is applicable to the case where the P.N.A. is located within the steel beam. The shear force within the shear connection is given by Equation 2.6 at the flexural failure of the slab.

$$ \Sigma Q = 0.85f'_c bt $$  

Equation 2.6

Where:

- $\Sigma Q$ = shear force in a shear connection, lbs
- $f'_c$ = concrete compressive cylinder strength at 28 days, psi
\[ b = \text{width of concrete slab, in.} \]

\[ t = \text{effective thickness of slab, in.} \]

An equation was also developed for a partial flexural interaction. The equation was verified after being compared to tests carried out by Davies in 1969. The theoretical method developed was in good agreement with the values of shear connector force at the onset of longitudinal cracking. Estimated percentages of transverse reinforcement also agreed with the design of the beams Davies tested.

Buckner et al. (1981) characterized longitudinal shear failure as splitting of the concrete slab parallel to the span of the member. Stub girder tests were carried out with the objectives of providing description of the experimental tests, bringing attention to the longitudinal shear failure mechanism in composite stub girders, and recommending the use of transverse slab reinforcement to increase the longitudinal shear strength of the composite girders. The authors present the following design recommendations for longitudinal shear design.

1. Longitudinal shear strength must be great enough to allow the shear connector capacity to be reached. The strength must be designed neglecting any benefit from transverse forces caused by membrane action.
2. The ultimate longitudinal shear force can be approximated using Equation 2.7. The nominal ultimate longitudinal shear stress is determined is then determined by Equation 2.8.

\[ V_u = Q_u - 0.85f_c b_s t - A_{sl} f_y \]  \hspace{1cm} \text{Equation 2.7}

\[ \nu_u = \frac{V_u}{2tl_s} \]  \hspace{1cm} \text{Equation 2.8}

Where:

\[ V_u \] = ultimate longitudinal shear force, lbs

\[ Q_u \] = ultimate horizontal shear capacity of the connectors, lbs

\[ f_c' \] = specified compressive strength of concrete, psi

\[ b_s \] = transverse distance between shear planes, in.

\[ t \] = slab thickness, in.

\[ A_{sl} \] = area of longitudinal slab reinforcement between shear planes, in.\(^2\)

\[ f_y \] = specified yield strength of reinforcing steel, psi

\[ l_s \] = length of the stub under consideration, in.

3. Computed shear stress should not exceed the following limits which can be calculated using either imperial or metric units:
a. Normal weight concrete: \( \nu_u \leq [0.8f_y + 400 \ psi \ (2,760 \ kPa)], (0.3f'_c) \)

b. Sand-lightweight concrete: \( \nu_u \leq [0.8f_y + 250 \ psi \ (1,720 \ kPa)], (0.2f'_c) \)

c. All lightweight concrete: \( \nu_u \leq [0.8f_y + 200 \ psi \ (1,380 \ kPa)], (0.2f'_c) \)

Where:

\[
\rho = \text{ratio of transverse reinforcement, in}^2/\text{in}^2 \text{ or m}^2/\text{m}^2
\]

\[
f_y = \text{specified yield strength of reinforcing steel, psi or kPa}
\]

\[
f'_c = \text{specified compressive strength of concrete, psi or kPa}
\]

4. Buckner et al. stated that half of the reinforcement required must be placed in the bottom half of the slab.

5. Longitudinal shear stress is assumed to vary linearly up to the critical value.

Buckner et al. stated that longitudinal shear capacity of composite beams should be investigated further as part of the generation of a more complete design procedure.

Johnson and Oehler (1981) analyzed 125 push tests, conducted 101 of their own push tests, tested four composite t-beams, and completed a parametric study to determine parameters that affect horizontal and longitudinal shear strength. The authors stated that width of concrete slabs in composite beams greatly affect the push strength of the 125 push tests. One conclusion about the 125 push tests was that some of the scatter in the results of
the push tests could be attributed to experimental error. Johnson and Oehlers also concluded “lateral restraint from reinforcement and friction at the base of the slabs increases with the diameter of the studs.” A method was developed to predict longitudinal splitting near shear connectors in the concrete slab of composite beams.

Oehlers and Park (1992) conducted research to determine the dowel strength of shear connectors after longitudinal cracking had occurred in conventional composite beams. It was found in previous research done by Johnson and Oehlers in 1981 that the compressive strength and stiffness of shear stud material and concrete in the area directly in front of the stud greatly affect the dowel strength of the horizontal shear connection in composite beams. Along with the dependence on these two parameters from each material the triaxial restraint of the concrete directly in front of the stud affects the dowel strength of shear connectors. Oehlers and Park studied the interaction between dowel strength and longitudinal crack formation. A method is presented for determining dowel strength of connectors related to the amount of transverse reinforcement. It is noted that dowel strength has been found to be controlled by stiffness and not strength of transverse reinforcement. It was observed by the researchers that longitudinal cracking weakened a single line of shear connectors while it appeared that a double line of connectors were unaffected by longitudinal cracking. Oehlers and Park concluded that stiffness rather than strength of transverse reinforcement affected the dowel strength in longitudinally cracked slabs. They also stated that failure of connectors can occur before yielding of transverse reinforcement because crack openings diminish the triaxial restraint on the compression zone of concrete. Single line connectors with high
spacing were dominated by dowel failure while shear plane failure was the characteristic failure of specimens with smaller longitudinal spacing and increased number of lines of connectors.

Piotter (2001) conducted research focused on the appropriate prediction of and design for longitudinal shear which is present in typical composite steel and concrete beam floor systems. After an extensive review of the state of knowledge Piotter developed an analytical model and tested two types of framing configurations. The minimum amount of transverse reinforcement to prevent longitudinal shear failure was determined after acquiring the results from testing. An analytical comparison was carried out to generate and verify a design procedure for determining the amount of transverse reinforcement. Existing and new procedures were synthesized in order to create an appropriate procedure for the limit state of longitudinal shear failure. Bottom chord yielding, buckling of steel components, and longitudinal slab cracking were observed failure modes. The flexural strength model used for analysis was limited by $\Sigma Q_n$, defined as the minimum force required to be transferred between components of the composite structure. The minimum of the steel yield force of the bottom chord or tension steel, welded shear stud yield force, or concrete crushing force must be transferred at ultimate flexural strength. However, due to the limit state of longitudinal shear cracking the full force required to be transferred throughout the concrete deck is only transferred by concrete within the longitudinal cracking planes, thus limiting the flexural strength of the system. Piotter presents two methods to calculate both longitudinal shear demand and longitudinal shear strength. The first method to determine longitudinal shear
demand assumes a constant thickness of concrete slab and presents the demand as a ratio of the flexural strength, $\Sigma Q_n$. The ratio is taken as the difference between the effective width of slab and width inside longitudinal shear planes over the effective width of slab. The second method assumes that the concrete between longitudinal shear planes, $A_{cs}$, resists a portion of the force needing to be transferred. The remaining force $\Sigma Q_n - 0.85f'cA_{cs}$ is then required to be transferred across the longitudinal shear planes. The longitudinal shear demand can be calculated using method 1 in Equation 2.9 or method 2 in Equation 2.10. Figure 2.2 supplements Equation 2.9 and Equation 2.10 depicting $b$, $b'$, and $A_{cs}$.

$$V_u^T = ((b - b')/b) \Sigma Q_n$$  \hspace{1cm} \text{Equation 2.9}$$

$$V_u^T = \Sigma Q_n - 0.85f'cA_{cs}$$  \hspace{1cm} \text{Equation 2.10}$$

Where:

$V_u^T = \text{total ultimate longitudinal shear demand force, lbs}$

$b = \text{effective width of slab, in.}$

$b' = \text{portion of slab between the shear plane or between slab edge and shear plane, in.}$

$\Sigma Q_n = \text{total compressive force in the slab, taken as the minimum force of steel yielding or concrete crushing, lbs}$

$f'c = \text{concrete compressive strength, psi}$

$A_{cs} = \text{area of concrete between shear planes, in.}^2$
Poitter presents three equations used to determine the longitudinal shear strength. Equation 2.11 is adapted from ACI (1999) and is based upon the ACI shear friction model.

\[ V_r = 0.8A_s f_y + K_A_{cv} \]  

Equation 2.11

Where:

\[ V_r \] = total nominal longitudinal shear strength, lbs

\[ A_s \] = transverse reinforcement area across shear plane, \( \text{in}^2 \)
\( f_{yr} \) = yield strength of transverse reinforcement steel, psi

\( K_1 \) = 400 psi for normal weight concrete, 200 psi for all-lightweight concrete, and 250 psi for sand-lightweight concrete, psi

\( A_{cv} \) = area of concrete section resisting shear transfer, in.\(^2\)

Oehlers and Bradford (1995) developed a similar equation to represent longitudinal shear strength. Equation 2.12 is the same as that of ACI except that \( K_1 \) is substituted for a single value of 200.

\[
V_r = 0.8 A_s f_{yr} + 200 A_{cv}
\]  \hspace{1cm} \text{Equation 2.12}

An expanded form of Equation 2.12 was also developed in which the coefficient 0.8 is represented by the bracketed term in front of \( A_s f_{yr} \) in Equation 2.13.

\[
V_r = \left[ 3.4 * \left( \frac{E_c}{E_s} \right)^{0.40} * \left( \frac{f_c'}{f_{yr}} \right)^{0.35} \right] A_s f_{yr} + 200 A_{cv}
\]  \hspace{1cm} \text{Equation 2.13}

Where:

\( V_r \) = total nominal longitudinal shear strength, lbs

\( E_c \) = modulus of elasticity of concrete, psi

\( E_s \) = Modulus of elasticity of reinforcing steel, psi

\( f_c' \) = concrete compressive strength, psi

\( f_{yr} \) = yield strength of transverse reinforcement steel, psi
\[ A_{cv} = \text{area of concrete section resisting shear transfer, in.}^2 \]

Piotter states that design equations presented by Oehlers and Bradford, Equation 2.12 and Equation 2.13, should be used for design.

Fahmy and Abu-Amra (2007) conducted a research study investigating parameters that effect the development of longitudinal cracking in shallow floor composite steel concrete framing systems. Eight parameters were examined which included the type of loading, slab compressive strength, steel beam yield stress, beam span to slab width ratio, steel beam size, use of ribbed metal decking, ribbed metal deck height, and transverse reinforcement percentage. The systems under investigation were conventional composite systems composing of a steel girder, stay-in-place ribbed metal decking, and cast in place concrete topping slab. After developing the analytical model, Fahmy and Amra compared the analysis of experimental test specimens found in literature to the analytical result of the model in order to verify the model. Upon verification that the model would accurately predict behavior of composite steel beams with concrete a slab supported with ribbed metal decking eighty-four simply supported composite beams were tested in two phases. During Phase I the investigators identified the primary factors that contribute to the phenomenon of longitudinal cracking. Phase II research included the examination of the effects of percentage of connection and the compilation of parameters investigated with regard to developing design curves. In Phase II fifty-six beams were tested and analyzed. As a result of loading transverse tensile forces were developed in the concrete slab around the area of shear connectors. Uniform loading was found to cause the longitudinal cracks to initiate at
the top of the concrete slab over the steel beam while single or two point loading caused longitudinal cracking to begin at the bottom of the slab. A consistent observation was that the concrete compressive strength increased the initial cracking moment which delayed the onset of longitudinal cracking resulting in a greater ultimate moment capacity. Coupled with $f'_c$, $F_Y$ (yield stress of steel beam) was concluded to be a parameter that contributes heavily to the longitudinal cracking phenomenon observed in the composite beam tests. As the ratio $F_Y/f'_c$ increased, it was observed that $M_{LC}/M_U$ (initial cracking moment/ultimate moment) increased. An increase in the beam span to slab width ratio ($L/b$) and the presence of ribbed metal decking are two parameters that improved the longitudinal cracking behavior of the composite beam sections. Longitudinal cracking behavior was improved in cases where the ribbed metal deck height was increased. Lower compressive stresses are induced in the slab in the longitudinal direction as a result of a larger moment arm between the compressive force and tension force in the section. Lastly for all ratios of $L/b$ and $F_Y/f'_c$ increases in transverse reinforcement increase the ratio of $M_{LC}/M_U$.

2.5 Need for Research

The behavior of composite steel and concrete framing systems has been researched for many years with specific focus on the numerous design limit states associated with such systems. Although much research has been done on conventional composite framing systems there is a lack of knowledge regarding design limit states of shallow floor composite framing system, both cast-in-place and precast systems. In specific regard to the
Diversakore® shallow floor systems more research should be conducted to better understand the design limit states of horizontal shearing between the u-shaped plate and cast-in-place concrete and longitudinal shearing at the interface between the hollowcore units and the cast-in-place concrete. Additional research should be focused on the validation of these two limit states and implementing them into a proper design procedure along with flexural and shear design limit states.
3 EXPERIMENTAL INVESTIGATION

3.1 Introduction

To explore the behavior of the Diversakore® slim floor framing system and expand on the previous research of this framing system an experimental program was undertaken at the North Carolina State University Constructed Facilities Laboratory in Raleigh, North Carolina. The objective of the experimental investigation was to characterize the design limit states of vertical, horizontal, and longitudinal shear. The experimental program included one full scale flexural test and one full scale vertical shear test, both of which were cast-in-place systems. The design, construction, instrumentation, and testing of each specimen is detailed in this chapter.

3.2 Test Specimens

Two cast-in-place floor system sub assemblages were designed, constructed, and tested at the Constructed Facilities Laboratory (CFL) at North Carolina State University. One specimen, DK 3, was configured to investigate the flexural mode of failure while the other specimen, DK 4, was configured to investigate the vertical shear behavior.
3.2.1 Specimen Components

3.2.1.1 U-shaped Steel Plate

A ¼” thick A572 steel u-shaped plate served as the base of the composite slim floor framing system. The dimensions of the steel plate measured 20” in width along the base and 6” deep with 2 ½” return flanges at the top of each web as seen in Error! Reference source not found. The plate sections were manufactured to a length of 30’ for both DK 3 and DK 4 specimens by Feralloy Corporation and Pegasus Steel Products, located in South Carolina. After construction of the full specimens, the DK 4 specimen was sawcut in half for a final length of 15’.

Figure 3.1 - U-shaped Steel Plate
3.2.1.2 Shear Studs

The shear stud layout for the DK 3 specimen consisted of ¾” diameter 8” deep headed shear studs. They were placed in two rows with 8” staggered spacing with a three 3” offset from the centerline of the u-shaped plate as depicted in Figure 3.2. The DK 4 shear stud layout was identical to the DK 3 layout except that the staggered spacing was 10”.

![Figure 3.2 - Shear Stud Layout](image)

3.2.1.3 Steel Reinforcement

All positive and negative flexural reinforcement bars used in the DK3 and DK 4 specimens were Grade 60 steel rebar. Six No. 9 bars measuring 29’ 8” in length were used as
positive moment reinforcement and two No. 5 bars measuring 29’ 8” served as negative moment reinforcement. A 2” clear cover was used for the positive moment reinforcement measured from the inside bottom surface of the u-shaped steel plate. A 2” clear cover was also used for the No. 5 bars from the top surface of the composite section. Shear reinforcement was designed in accordance with ACI 318-08 to be the same for both DK 3 and DK 4. Shear reinforcement consisted of No. 4 double legged stirrups spaced 3” from both ends over a distance of 86” from the ends. Stirrups were spaced 6” on center between the 3” spacing zones. Figure 3.3 shows the longitudinal and shear reinforcement layout along the span of the members.

![Figure 3.3 - Longitudinal and Shear Reinforcement](image)
3.2.1.4  Bar Stock

3” by ¼” steel plate straps, shown in Figure 3.4, were welded across the top of the section onto the underside of the return flanges of the u-shaped steel sections to stiffen the open steel section and avoid opening or bending of the steel plate webs or return flanges.

Figure 3.4 - Welded Steel Plate Strap

3.2.1.5  Precast Hollowcore Planks

8” thick precast hollowcore plank units were placed transversely on top of the steel u-beam return flanges for both DK 3 and DK 4 as shown in Figure 3.5. The dimensions of the hollowcore units were 4’ wide by 3’ in length. Styrofoam blockouts were placed inside the
cores to allow 4” of cast-in-place concrete infill in order to increase the connection between the core beam and extending floor units.

Figure 3.5 - Placement of Hollowcores

3.2.1.6 Cast-in-place Concrete

The DK 3 and DK4 specimens utilized cast-in-place concrete to create the girder and the topping slab. The concrete beam and 2” topping slab were monolithically cast to tie the entire composite system together. Concrete filled the u-shaped steel beam and flowed 4” into the cores of the hollowcore units. The target concrete mix design strength was 5,000 psi, however actual compressive strengths found during cylinder testing are assumed throughout experimental and analytical calculations and comparisons. Shrinkage and temperature
reinforcement within the slab was provided with ASTM A185 WWF 6 x 6 x W2.9 x W2.9 with a cover of 1”.

The fully constructed specimens are pictured in Figure 3.6.

![Figure 3.6 - DK 3 and DK 4 Specimens with Cast-In-Place Concrete](image)

3.2.1.7 *Transverse Reinforcement*

No. 4 bent bars were used as transverse reinforcement in both DK 3 and DK4 extending from the core of the concrete girder into the grouted shear keyways between hollowcore units. The purpose of these bars was to provide an additional tie between the hollowcore units and the girder. Bars measured 42” in total length and were bent to have a 6” return as shown in Figure 3.7.
3.2.1.8 Cementitious Grout

Cementitious grout was mixed at the CFL and placed within the shear keyways between hollowcore units of both composite beam specimens. The grout was used to secure the transverse bent bars and solidify the connection between the hollowcores and core beam. Compressive strength tests on 2” cementitious grout cubes indicated a compressive strength of 1,200 psi. The testing method is further discussed in the Material Properties section.

3.2.1.9 Perforated Angle

1 ½” by 3” 14 gauge slotted steel angles were attached to the bottom of the hollowcore units along the outside edges of both the DK 3 and DK 4 specimens as shown in Figure 3.8. Under normal field conditions the hollowcore units would span an entire bay and be supported by the next composite girder. The perforated angle was attached to simulate the continuity between hollowcore units along the span of the girder.
3.2.2 Material Properties

Material property testing was carried out at the CFL for the cast-in-place concrete, steel u-shaped beam, and cementitious grout. Strengths of each material were taken and used for analysis. Table 3.1 at the end of the section presents averaged material strengths of each material for each of the test specimens.

3.2.2.1 Concrete Compressive Strength

4” by 8” cylinders were cast according to ASTM C31 during the casting of the composite beams specimens. Compressive strength data was collected at 7, 21, 28, and test days. An average diameter was measured from which the cross sectional area was calculated. All cylinders were tested according to ASTM C39 with a 500 kips capacity
Forney machine. The rate of load application used was $35 \pm 7$ psi/s. An example of a concrete cylinder compressive strength test is shown in Figure 3.9.

![Concrete Compressive Strength Test](image)

**Figure 3.9 - Concrete Compressive Strength Test**

### 3.2.2.2 U-Shaped Beam Tensile Strength

The steel u-shaped beams used for both DK 3 and DK 4 specimens were manufactured from the same batch of ASTM A572 steel as the steel plates used in previous testing done on similar specimens carried out at the CFL by Willis (2009) and Amortnont (2010). Three $\frac{3}{4}''$ by 10” coupons were milled down to $\frac{1}{2}''$ width along a reduced gage length of 4”. Coupons were then measured using vernier calipers and tested in a 20 kips
MTS Q-test machine according to ASTM A370. Coupon data from these tests was used from which yield strength and ultimate strength were referenced during analytical evaluation of the specimens. The modulus of elasticity was assumed to be 29,000,000 psi for all steel components. Steel material properties are summarized in Table 3.1

3.2.2.3 Grout Compressive Strength

Three grout cubes were cast for each of DK 3 and DK 4. The cubes were cast in 2” molds. Grout cubes were measured and tested on the day of the composite beam tests to verify the compressive strength using the Forney Machine at a rate of 35 ± 7 psi/s as shown in Figure 3.10.

![Figure 3.10 - Grout Compressive Strength Test](image-url)
Table 3.1 - Material Properties of DK 3 and DK 4 Specimens

<table>
<thead>
<tr>
<th></th>
<th>DK 3</th>
<th>DK 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Compressive Strength (psi)</td>
<td>4,900</td>
<td>5,200</td>
</tr>
<tr>
<td>U-Shaped Section Yield Strength (psi)</td>
<td>62,500</td>
<td>62,500</td>
</tr>
<tr>
<td>U-Shaped Section Ultimate Strength (psi)</td>
<td>74,000</td>
<td>74,000</td>
</tr>
<tr>
<td>Grout Compressive Strength (psi)</td>
<td>1,200</td>
<td>1,200</td>
</tr>
</tbody>
</table>

3.3 Construction of Specimens

The composite beam floor specimens were constructed in multiple phases including: manufacturing of components, shear stud and steel plate welding, tying of reinforcement cages, placement of hollowcore units, and concrete casting.

3.3.1 Manufacturing of Components

The steel u-shaped plate sections were manufactured to a length of 30’ for both tests by Feralloy Corporation and Pegasus Steel Products, located in South Carolina. The plate sections were then shipped to the CFL at North Carolina State University for construction of the composite beam floor systems. Gate Precast Company provided 8” thick precast hollowcore units measuring 4’ wide by 3’ in length to the Constructed Facilities Laboratory. Shear studs and reinforcing steel including welded wire mesh were provided by Nelson Stud
Welding and Gerdau Ameristeel, respectively. Cast-in-place concrete was provided by S.T. Wooten while finishing services were provided by IQ Contracting. All construction of specimens took place at the Constructed Facilities Laboratory.

3.3.2 Shear Stud and Steel Plate Welding

Shear studs were welded onto the u-shaped steel plates of specimens DK 3 and DK 4 using a Nelson Nelweld Model 6000 stud welder. 1,200 amps and a plunge time of 0.7 seconds were the settings during the welding process. Ceramic ferrules were utilized to create a consistent weld collar at the base of the shear studs. Steel plates were installed after tying the reinforcement cage underneath spanning between the return flanges of the u-shaped plate using a wire fed MIG welder. Stud welding and straps are shown in Figure 3.11.

Figure 3.11 - (a) Shear Stud Welding and (b) Steel Plate Welding
3.3.3 **Tying of Reinforcement Cages**

The reinforcement cages were assembled directly around the shear studs and steel straps within the u-shaped steel beam as seen in Figure 3.12. Six No. 9 mild steel reinforcing bars were used as tension reinforcement while two No. 5 bars were used for negative reinforcement. Prefabricated No. 4 hat shaped stirrups were hung from the negative reinforcement and tied to the positive reinforcement to construct a rigid reinforcement cage. All longitudinal reinforcement measured 29’ 8” in length.

![Figure 3.12 - Reinforcement Cage](image)
3.3.4 Placement of Hollowcore Units

Before placement of hollowcore units 1 ½” thick Styrofoam disks were inserted into each core of the hollowcore units creating a 4” blockout for concrete to flow into. The hollowcore units were placed along the span of the u-shaped plates using the outdoor mobile crane as depicted in Figure 3.13. The hollowcore units rested on the return flanges and the temporary wood shoring while No. 4 bent bars were inserted and grouted into the shear keyways between them. Duct tape was used to inhibit the flow of grout from the keyways down into the u-shaped plate. Perforated angle was attached to the outside bottom corner of the hollowcore units creating a continuous connection between units which would be present in full bay construction.

![Figure 3.13 - Placement of Hollowcore Units](image)
3.3.5 **Concrete Casting**

Formwork was attached to the edges of the composite beam specimens via Tapcon concrete screws. A hammer drill was used to bore holes and attach 1” by 6” wood boards along the span which acted as formwork for casting a 2” topping slab. ¾” plywood was cut to the shape of the cross section and used to cover the ends of the beam. They were attached in the same manner as the 1” by 6” boards. Welded wire fabric was cut to fit within the formwork and was supported by 1” plastic chairs to be at the mid-depth of the topping slab. Duct tape was then used to cover all joints or cracks between hollowcore units, wooden formwork, and the u-shaped steel beams to avoid unnecessary concrete loss. Concrete was delivered by S.T. Wooten and placed and finished by IQ contracting as shown in Figure 3.14 (a). Fifteen 4” by 8” cylinders were cast for each beam in accordance with ASTM C31 and placed nearby their respective composite beams to cure under similar conditions. Complete specimens are shown in Figure 3.14 (b). Both beams were covered with wet burlap and plastic to inhibit moisture loss during the initial setting and curing stages.

![Figure 3.14 - (a) Concrete Placement and (b) Complete Specimens](image-url)
3.4 Test Setup

During the 28 days required for the specimens to cure the test setup was assembled. The DK 3 composite specimen test setup included a four point load application assembly and a simply supported span measuring 27’. The loading configuration was selected to simulate distributed load acting on the framing system. The composite beam was placed using a pin and roller system atop 24” high concrete blocks. Load cells were present under only the pin support to determine the vertical reaction. A 440 kips capacity MTS actuator along with an adequate strength loading tree delivered the load distributed over four points. Pin and roller supports were used intermediately between layers of the loading tree. The secondary spreader beams rested on the 8” side of HSS 8 x 6 x ½ sections positioned transversely to the surface of the composite beam. Neoprene pads were placed underneath the HSS sections at the four load application points. The DK 3 test setup is shown in Figure 3.15.

Figure 3.15 - DK 3 Flexure Test Setup
The DK 4 composite section was modified from its original length of 30’ to 15’. Utilizing a 36” diamond blade and road saw the specimen was cut to 15’. DK 4 was tested on an 8’ 3” clear span with 2’ 1” and 4’ 8” of overhang at each support. A shear failure mechanism was desired for the DK 4 section, thus necessitating a three point loading configuration in which the load was applied 28” from the left support with a shear span to depth ratio of 1.75. A single HSS 8 x 6 x ½ section was placed on top of a neoprene pad underneath the 440 kips capacity MTS actuator to deliver a transverse distributed line load to the specimen. The DK 4 test setup is shown in Figure 3.16.

![Figure 3.16 - DK 4 Shear Test Setup](image-url)
3.5 Instrumentation

Each of the test specimens were instrumented to measure the behavior and response of the specimens when subjected to the applied load. The applied load, deflection, strain, and relative slip between specimen components were monitored and recorded throughout the loading sequence. Details of the instrumentation configuration are provided herein.

3.5.1 String Potentiometers

String potentiometers (string pots) were positioned underneath the midpoint and quarter points of the DK 3 specimen to measure deflections taken from the steel u-shaped beam and the overhanging hollowcores on each side of the specimen. The deflection measurements were taken to generate a longitudinal deflection profile and transverse deflection profiles at the quarter and midpoints for each load step. Four string pots were attached at midspan of the composite section. Two string pots were offset 8” to either side of the centerline attached to the steel u-shaped plate. The other two string pots were offset 32” from the centerline attached to the hollowcore units. Three string pots were attached at each quarter point, one at the centerline while the remaining two were offset 32” attached to the hollowcore units. Figure 3.17 depicts the typical string pot layout on the cross section of the DK 3 test specimen while Figure 3.18 is a photograph of the typical midpoint string pot layout of the DK 3 specimen.
Figure 3.17 - DK 3 Midpoint (top) and Quarter Point (bottom) String Pot Layout

Figure 3.18 - DK 3 String Plot Layout
DK 4 deflection measurements were taken at the midpoint of the shear span and directly beneath the load point. Deflections were measured underneath the steel u-shaped beam and the overhanging hollowcores to develop deflection profiles in the same manner as for the DK 3 specimen. Four string pots were attached directly underneath the load point: Two string pots offset 3” to either side of the centerline and two string pots offset 32” to either side of the centerline attached to the hollowcore units. Three string pots were attached at mid shear span, one at the centerline and the remaining two offset 32” to either side of the centerline. Figure 3.19 depicts the typical string pot layout for the DK 4 test.

Figure 3.19 - DK 4 Load Point and Mid Shear Span String Pot Layout
3.5.2 Strain Gauges

Strain gauges were placed on different steel components at the midpoint and quarter points of the DK 3 and DK 4 test specimens to capture strain profiles and check for yielding in steel components at different load levels. The placement and covering of strain gauges is depicted in Figure 3.20 (a), (b), and (c). To protect interior strain gauges a protective coating was painted over each gauge followed by the application of a rubber butyl adhesive pad which prevented moisture or concrete flow from affecting the gauges.

Figure 3.20 - (a) Strain Gauges on U-Shaped Plate With Butyl Cover, (b) Strain Gauges on Reinforcing Steel, and (c) Strain Gauge with Protective Coating
A total of sixteen 120 ohm resistance strain gauges were placed at the midpoint while ten strain gauges were used at each of the quarter points of the DK 3 specimen. Twelve strain gauges were attached to the outer and inner surfaces of the u-shaped steel plate at the midpoint while eight strain gauges were attached to the steel u-shaped plate at the quarter points. The remaining strain gauges were evenly distributed and attached to the longitudinal positive and negative reinforcing bars. DK 3 midpoint and quarter point strain layout is shown in Figure 3.21.

Figure 3.21 - DK 3 Midpoint (top) and Quarter Point (bottom) Strain Gauge Layout

120 ohm resistance strain gauges were also attached to the DK 4 specimen prior to testing to capture strain profiles and observe if steel components yield. Three strain gauges
were attached to the bottom outer surface of the u-shaped steel plate at the midpoint of the shear span as shown in Figure 3.22.

![Figure 3.22 - DK 4 Mid Shear Span Strain Gauge Layout](image)

3.5.3 **Rosette Gauges**

Four 120 ohm resistance three wire rosette strain gauges were applied to the exterior of the u-shaped steel plate at the mid and a single quarter point of the shear span on the DK 4 specimen as shown in Figure 3.23 to examine the state of shear stress at each load step. Longitudinal, transverse, and maximum shear stress data was collected by the rosette gauges during the testing procedure. The rosette gauges were placed at the mid height of 3” on the webs of each side of the specimen. Rosette gauges, shown in Figure 3.23, were only used during the testing of the DK 4 specimen.
3.5.4 Pi Gauges

100 mm Pi gauges were used in conjunction with the strain and rosette gauges to provide a complete strain profile at the given load level and point along the span. As shown in Figure 3.21 three Pi gauges were placed at the midpoint and one Pi gauge was placed at each quarter point of the DK 3 specimen. Three Pi gauges were placed along the centerline of the composite section at the quarter and midpoints and two additional Pi gauges were placed at mid span offset 32” to either side of center to be attached directly above the string potentiometers on the underside of the hollowcores. Midpoint Pi gauges of DK 3 are shown in Figure 3.24.
Three 100 mm Pi gauges were placed at the midpoint of the shear span of the DK 4 specimen as shown in Figure 3.22. One Pi gauge was placed along the centerline of the specimen while the remaining two were offset 32” to either side.

![Figure 3.24 - DK 3 Midpoint Pi Gauge Setup](image)

3.5.5 **Linear Motion Transducers**

Relative end motion or slip between the steel, hollowcore, and concrete components was measured through the use of linear motion transducers attached to each end of the DK 3 specimen and DK 4 specimen shown in Figure 3.25. Four sensors were symmetrically placed at both ends to measure the relative slip between the hollowcore units and u-shaped steel plate and the concrete and the u-shaped steel plate as shown in Figure 3.26.

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3.5.6 **Load Cells**

Two hundred kips capacity load cells were situated as part of one of the supports during the testing of the DK 3 and DK 4 specimens. Due to the symmetric support and loading conditions of the DK 3 specimen, two load cells, shown in Figure 3.27, were used to measure applied load, compare with hand calculations and load output from the actuator, and
confirm the correct distribution of load between the two supports in each test case. The DK 4 reactions were hand calculated for each load step and compared to the load cells output and the actuator load reading during testing.

![Figure 3.27 - Two 200 Kip Load Cells](image)

3.5.7 Data Acquisition

All measurements were continuously collected, recorded, and monitored using a Vishay 5000 data acquisition system and Strain Smart software.
3.6 Loading Procedure

Both DK 3 and DK 4 were tested using a 440 kips capacity MTS actuator running in displacement control. Specimens were cyclically loaded and unloaded through a full range of load steps they would potentially be subjected to as part of a structural frame. Specimens were statically tested with a load rate of 0.0333 in/min and an unload rate of 0.0666 in/min through the full range of loads. *Error! Reference source not found.* presents the different load steps for the DK 3 and DK 4 specimen tests. DK 3 loading steps included preload, one half service load, full service load, one and one half factored service load, and failure load. The steps of loading for the DK 4 specimen included preload, factored load, factored shear load, nominal shear load, and failure load. All loads are tabulated as total load delivered by the actuator into the loading tree to be distributed to the specimens. The failure load was considered to be the maximum load delivered before concrete crushing occurred for each specimen. Applied moments were determined from the distributed point load reactions from the loading tree onto the specimens.

Table 3.2 - DK 3 and DK 4 Loading Procedure

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</table>
3.7 Observed Specimen Behavior

3.7.1 Introduction

Section 3.7 presents general results and specimen behavior observed during the experimental testing of the DK 3 and DK 4 specimens. A discussion of deflections, strains, slip, and general test behavior is given with figures and plots accompanying respective data. Experimental data is given within data packs included in Appendix B.

3.7.2 Deflection

Deflection data was measured under the steel plate and the hollowore units at the mid and quarter points of the DK 3 specimen while load was measured using a load cell installed within the actuator. The total moment applied was calculated from the load cell output for the DK 3 specimen and plotted against the measured deflection. Figure 3.28 depicts the averaged deflections measured from the bottom of the steel u-shaped plate for the entire loading protocol including both the loading and unloading cycles up to failure.

Deflection was measured for the DK 4 specimen at the point of load application and at the midpoint of the shear span. The total moment applied was calculated and plotted against the measured deflections as shown in Figure 3.29.
Figure 3.28 - Moment vs. Deflection for DK 3
Figure 3.29 - Moment vs. Deflection for DK 4
3.7.3 Strain Profiles

Strain measurements were taken throughout the cross section at both quarter points and the midpoint of the DK 3 specimen while strain measurements were taken at the first quarter point and the midpoint of the shear span of the DK 4 specimen. Strain gauges were adhered to the inner and outer surfaces of the u-shaped steel plate, mild positive longitudinal, and mild negative longitudinal steel reinforcement. Pi gauges were attached on top of the concrete deck to complement the strain gauges attached to steel components and provide an entire strain profile within the section. A plot of the DK 3 averaged strain measurements, negative strains signifying components in compression and positive strains signifying components in tension, is presented in Figure 3.30. From the collected strain data strain profiles were developed for each load step of the loading protocol to depict the changing strain profiles throughout the testing. Strain profiles are assumed to be linear throughout the cross section until the cross section changes due to longitudinal cracking and relative movement between components. The plot of strain profiles for the DK 3 specimen supports this assumption as the specimen approaches the failure load cycle it is evident that a linear strain distribution is no longer present. An example plot of the DK 3 strain profiles verse load cycles is given in Figure 3.31. Additional strain data and plots for the DK 3 and DK 4 specimen are given in Appendix B.
Figure 3.30 - Applied Moment vs. Midpoint Strain for DK 3
Figure 3.31 - Strain Profiles for DK 3

I – 0.5 Service Load Step (220 k-ft)  
II – Service Load Step (444 k-ft)  
III – 1.5 Factored Load Step (663 k-ft)  
IV – Failure Load Step (790 k-ft)
3.7.4 Longitudinal Slip

Relative slip was measured between the hollowcores and steel u-shaped plate and hollowcores and cast-in-place concrete on both the DK 3 and DK 4 specimens. Slip measurements were taken to determine when full interaction was lost and partial interaction between components began.

The maximum slip recorded for the DK 3 specimen was 0.52 inches during the failure loading cycle between the steel u-shaped plate and one side of the hollowcore units. All other measurements of slip between components were identical below 0.1 inches of slip. Slip measurements indicate that movement between the cast-in-place concrete and steel u-shaped plate begins at approximately 450 k-ft of applied moment which can be attributed to the deviation from initial stiffness of the specimen. Significant movement is depicted between the hollowcores and steel u-shaped plate beyond 700 k-ft of applied moment.

Despite significantly less slip being measured, the maximum recorded slip was approximately 0.22 inches for the DK 4 system, occurring between core concrete and the u-shaped steel plate on one end of the specimen similar to the motion observed from the DK 3 specimen.

Figure 3.32 and Figure 3.33 are plots of the slip measurements recorded for DK 3 and DK 4, respectively.
Figure 3.32 - Slip Plots for DK 3

Note: 0.52” maximum slip recorded
Figure 3.33 - Slip Plots for DK 4

Note: 0.22" maximum recorded slip
3.7.5 **Effective Flange Width**

Both the DK 3 and DK 4 specimens were constructed with 89” flange widths while the effective flange widths of the composite T-sections were calculated according to ACI 318-08. The effective flange widths were calculated to be 68” limited by eight times the slab thickness of overhang beyond the web. A 3” slab width, 2” of concrete slab and 1” of hollowcore flute, was used in the calculation. Strain measurements were taken at three points along the top of the DK 3 specimen at the midpoint and quarter points from which the strains within the effective flange were analyzed. Pi gauges were placed at the centerline and offset thirty two inches to either side at the midpoint and quarter points. Figure 3.34 depicts the strain profiles across the effective flange width throughout testing of the specimen. Majority of the strain was concentrated on the center of the specimen beyond the one half service load step. When longitudinal shear cracking occurred strain readings from PG MP I and PG MP III became unreliable due to reduction of the effective flange width and loss of load carrying capacity of overhanging material. Similar data was collected for the DK 4 specimen with Pi gauges placed in the same manner as the DK 3 specimen at the midpoint of the shear span, but was found to be erroneous.
Figure 3.34 - DK 3 Effective Flange Strain Profiles
3.7.6 Flexural-Shear Cracking

Flexural shear cracking occurred during the testing of both the DK 3 and DK 4 specimens. As the load was increased cracks propagated at the interface between the hollowcores and cast-in-place concrete out from the confinement of the steel u-shaped plate upward. As cracks approach the middle of the specimen they became more and more characteristic of flexural cracking until the point of failure. Flexural shear cracking was less pronounced during the DK 4 shear test. It was intended to induce a shear failure mode during the DK 4 test, however with the onset of longitudinal shear cracking and the loss of the effective flange a small amount of flexural shear cracking along with concrete crushing under the load led to the premature failure of the DK 4 system.

3.7.7 Hollowcore Splitting

Flexural tension splitting cracks were observed on the underside of the hollowcore units during the testing of both the DK 3 and DK 4 specimens as shown in Figure 3.35. Transverse cracking typically occurred underneath points of load application, beginning as microcracking but quickly propagating as the curvature of the specimens increased.
3.8 Observed Failure Modes

3.8.1 Concrete Crushing

Concrete crushing was the ultimate failure mode observed during the testing of the DK 3 and DK 4 specimens. Concrete crushing occurred at lower than the anticipated maximum applied moment due to cross sectional changes observed during the testing of the two specimens. Concrete crushing occurred at the midspan along the centerline right beneath the Pi gauge placed at the center of the DK 3 specimen as pictured in Figure 3.36. The maximum recorded strain before concrete crushing occurred was 0.0036 in/in which was recorded well beyond the maximum moment applied to the structure. The highest measured
strain was approximately 0.002 in/in at both quarter points. There was no sign of concrete crushing at either quarter point. The DK 4 specimen exhibited similar concrete crushing behavior; however, crushing occurred in the concrete below the point of load application. The maximum measured strain, 0.0032 in/in, was measured by PG MP II at the midpoint of the shear span before crushing occurred under the load point. It should be noted that PG MP III reported strains up to 0.005 in/in, but were determined to be erroneous readings due to longitudinal shear cracking and loss of effective flange overhangs.

![Concrete Crushing Failure of DK 3](image)

**Figure 3.36 - Concrete Crushing Failure of DK 3**
3.8.2 Steel Yielding

ACI 318-08 defines a general yielding strain of mild reinforcing steel as 0.002. However, for analysis different yield strain values were determined based upon material properties of the steel components. A yield strain value of 0.0022 and 0.002 were determined for the steel u-shaped section and the longitudinal mild reinforcement, respectively. Strain values greater than these were measured only at the midpoint in the bottom flange of the u-shaped steel plate, lower half of the webs of the u-shaped steel plate, and the longitudinal positive moment reinforcing steel of the DK 3 specimen. Similar strain readings were observed during the DK 4 testing located at the midpoint of the shear span.

Yielding strain in the DK 3 steel plate was first measured during the failure load cycle at 668 k-ft just beyond the factored moment of 663 k-ft. As the curvature of the specimen increased the plastic zone extended upward to encompass the positive moment reinforcement. Compression reinforcing steel was measured to reach the yielding strain well beyond the application of the maximum applied moment. Significant changes had occurred to the beam cross section due to loss of effective flange and yielding of tension reinforcing steel leading to the yielding of compression reinforcing steel. White wash, a lime based coating, was applied to the exterior of the steel plate in order to observe any elongation of the u-shaped plate. Despite measured yielding there were no visible signs of yielding from white wash. The maximum recorded bottom fiber strain in the steel plate before the test was concluded and unloading began was 0.0038 in/in.
Yielding strain was first observed at an applied moment of 575 k-ft beyond the maximum applied moment of 273 k-ft for the DK 4 specimen. Although the applied moment was decreasing and unloading had not been started at this point during testing, the curvature was increasing causing measured yielding of the steel u-shaped plate. The maximum recorded strain value in the steel plate was 0.00253 in/in, however there were no visible signs of yielding from the white wash coating. It should also be noted that no strain measurements were taken at either positive or negative longitudinal reinforcement levels.

3.8.3 **Horizontal Shear Connector Failure**

Three quarter inches diameter eight inches tall shear studs were intended to provide a full shear interaction between the u-shaped steel plate and cast-in-place concrete for both the DK 3 and DK 4 specimens. Acting as dowels, shear studs were to resist the tendency of concrete to slip within the confinement of the steel plate. Less than 0.022 inches of slip was recorded between the steel plate and concrete indicating relatively no loss of connection between the two components for both specimens. A full shear connection was maintained, however partial interaction was observed through measurement of slip between the hollowcore units and the u-shaped steel plate after the formation of longitudinal shear cracks and reduction of the effective flange. The maximum recorded slip for the DK 3 specimen occurred between hollowcores and the steel plate measuring 0.52 inches while the maximum slip recorded for the DK 4 specimen was 0.22 inches. Figure 3.37 shows the relative movement between the hollowcore units and the u-shaped steel plate on the DK 3 specimen.
Similar observations were made for the DK 4 specimen concerning slip. Local end cracking is present but it should be noted that no movement is visible between the steel plate and cast-in-place concrete.

![Figure 3.37 - DK 3 Relative Hollowcore Steel Plate Slip](image)

3.8.4 **Longitudinal Shear Cracking**

Longitudinal shear cracking at the interface between hollowcore units and cast-in-place concrete was observed during the testing of both the DK 3 and DK 4 specimens. Transverse reinforcement was provided by WWF, #4 rebar grouted into hollowcore shear
keyway, and four inches of core infill within the hollowcores. Styrofoam disks were inserted four inches into hollowcore cores to allow infill concrete to enhance the longitudinal shear capacity of the system. Previous testing suggested that infill concrete would provide enough strength to maintain the entire effective flange of the specimens. However, cracking occurred during both tests causing premature ultimate failure of the test specimen due to concrete crushing. Longitudinal shear cracking, shown in Figure 3.38, occurred along both hollowcore cast-in-place concrete interfaces on both beams extended along the entire specimen between supports.

Figure 3.38 - Longitudinal Shear Cracking of DK 3
3.8.5 **Summary**

Both the DK 3 and DK 4 specimens were observed to have concrete crushing as the ultimate failure mode while the reduction in cross section was caused by longitudinal shear cracking. Neither specimen retained their full effective flange throughout testing, thus never reached the design moment strength. Subsequent limit states observed after longitudinal cracking failure included compression steel yielding, tension steel yielding, and concrete crushing.
4 ANALYTICAL INVESTIGATION

4.1 Introduction

This chapter details the analysis of the Diversakore® test beams and comparison of results between the experimental and analytical investigations. Results of layered sectional analyses done on both DK 3 and DK 4 are compared to experimental test results while individual strength and serviceability design limit state criteria are compared to testing results and observations. A summary of results and comparison is also presented at the end of the chapter. Included within this section where appropriate are results and comparisons of previous experimental testing performed by Willis (2009) and Amortnont (2010).

4.2 Layered Sectional Analysis

Layered sectional analyses were carried out for both the DK 3 and DK 4 test specimens based on actual specimen geometry, reinforcement layout, material properties, and constitutive models. Layered sectional analyses of the DK 3 and DK 4 specimens consisted of the development of a Moment vs. Curvature plots and an Applied Load vs. Deflection plots to use in comparison to experimental results. Strain data collected during the experimental investigation was compared to theoretical strain values calculated during the analytical investigation of both specimens. Analysis and comparisons of data for DK 3 and DK 4 along with tests previously completed are presented separately in the following sections.
4.2.1 DK 3 - Flexural Specimen

4.2.1.1 Moment Curvature

A moment curvature analysis was employed to predict the behavior of the DK 3 specimen based on actual material properties, reinforcement layout, and constitutive models. The plot of Applied Moment vs. Curvature developed from moment curvature analysis is shown in Figure 4.1 with the neutral axis depth also plotted against the curvature of the section. The ultimate moment strength of the DK 3 specimen was calculated to be 911 k-ft. The moment curvature relationship was developed with the underlying assumption that full interaction, that is, no slip between components occurs, would be maintained through the use of mechanical shear connectors between the steel u-shaped plate and cast-in-place concrete. Additional assumptions used during the analytical investigation were the effective flange would be maintained throughout testing until failure of the specimens and linear strain profiles throughout the section during the entire test.
4.2.1.2 Load Deflection

An Applied Moment vs. Midspan Deflection plot was generated from the moment curvature analysis of DK 3 specimen and compared to experimental moment deflection data. The plot of Applied Moment vs. Midspan Deflection comparison for the DK 3 specimen is shown in Figure 4.2. An experimental moment deflection envelope was used in the plot to further illustrate the full load behavior and aid in easier comparison to predicted moment deflection behavior. The initial stiffness of the DK 3 specimen matched with the predicted stiffness while the expected strength was not achieved. Deviation from initial stiffness occurred around 450 k-ft of applied moment, which is the point where slipping began to
occur between the core concrete and steel u-shaped plate. Longitudinal shearing of the effective flange on both sides was the primary cause of the resulting softening and understrength. Subsequent failures occurred beyond the maximum applied moment until unloading of the specimen. Overall, loss of the effective flange caused an approximate ultimate strength loss of 13%, dropping from the predicted 911 k-ft to the experimental 790 k-ft.

![Figure 4.2 - DK 3 Midspan Moment Deflection Comparison](image-url)
4.2.1.3 Strains

Strain measurements were taken from various components throughout the section of the DK 3 specimen. Complete strain data sets were plotted against predicted values calculated during moment curvature analysis of the sections. Concrete top strain, compression reinforcing steel strain, tension reinforcing steel strain, and steel u-plate bottom fiber strain are plotted in Figure 4.3. Component strain values matched very closely with the predicted strains until approximately 550 k-ft when concrete slab strain values and tension reinforcing strain values began to deviate from predicted values. Longitudinal shear cracking was observed during the testing of the DK 3 specimen which can be a cause of divergence of the experimental strain values. Compression reinforcing steel and the bottom steel plate strain values matched their predicted values for each component up to the maximum applied moment of 790 k-ft. It should be noted that residual strain remained within all steel components upon unloading of the DK 3 specimen. After concrete crushing occurred the specimen was unloaded from which residual strain can be observed in the plot.
4.2.1.4 Strain Profiles

Strain profiles are plotted in Figure 4.4 against height within the cross section of the DK 3 specimen. Predicted values of strain for each load cycle were taken from the moment curvature analyses conducted for each of the specimens. Strain profiles for the DK 3 specimen matched nearly perfectly with the experimental profiles at each load cycle up to the experimental failure of the specimen. At the failure load strain values of the extreme compression fiber and tension steel components were higher in magnitude than the predicted values indicating curvature of the member was higher than the expected curvature. Although
the applied moment did not increase above 790 k-ft the DK 3 specimen continued to deform which is apparent by the rising magnitude of strains in different components until eventual unloading.

Figure 4.4 - DK 3 Experimental and Predicted Midspan Strain Profiles
4.2.1.5 Slip

Relative movement between precast hollowcore, steel u-shaped plate, and cast-in-place concrete was measured at both ends of the DK 3 specimen to observe the level of shear interaction between the specimen components. Slipping between structural components occurred as shown in a plot of Applied Moment vs. Slip in Figure 4.5. Loss of shear interaction begins at approximately 450 k-ft of applied moment with relative slip occurring between hollowcores and cast in place concrete and hollowcores and steel plate at both ends of the beam. The relative slipping between cast in place concrete and steel plate was less significant than the slip measured between the hollowcores and cast-in-place concrete. Longitudinal shear cracking, which occurred at an applied moment of approximately 700 k-ft, of the effective flange is the primary reason for greater slip between hollowcores and the steel plate.
Figure 4.5 - DK 3 Slip Plots

Note: 0.52\" maximum slip recorded
4.2.2 DK 4 - Vertical Shear Specimen

4.2.2.1 Moment Curvature

Moment curvature analysis was carried out for the DK 4 specimen in the same manner as for the DK 3 specimen from which a load deflection plot was generated and compared to experimental values. The nominal moment capacity was calculated to be 919 k-ft. Figure 4.6 depicts the moment curvature relationship for the DK 4 specimen. Also included is a plot of the neutral axis depth related to the curvature of the specimen.

Figure 4.6 - DK 4 Moment vs. Curvature Prediction
4.2.2.2  *Load Deflection*

Experimental load point deflection of the DK 4 specimen was compared to a load deflection prediction generated from moment curvature analysis. The initial stiffness of the experiment load deflection is significantly lower than the predicted stiffness. The nature of the DK 4 test was to induce a shear failure of the specimen while still observing flexural behavior if any. The predicted load deflection considers only displacement caused by bending but it is evident that shear deformation causes significantly more experimental displacement. Again the DK 4 experimental maximum deflection was significant lower than the predicted maximum deflection due to unforeseen reduction of the effective flange and premature flexural failure of the beam specimen. The predicted load deflection is plotted against the load point deflection envelope shown in Figure 4.7.
Figure 4.7 - DK 4 Load Point Moment Deflection Comparison
4.2.2.3 Strains

Various strain measurements were taken throughout the cross section of the DK 4 section. Figure 4.8 depicts the strain measurements taken at the top of the section as well as the bottom of the section. Pi gauges were attached to the concrete slab at the midpoint of the shear span and strain gauges were attached to the steel u-shaped plate at the midpoint of the shear span. Initial slopes of all strain values corresponded very well except for the values collected by PG MP III which were deemed to be erroneous. The data for PG MP III was removed from the plot for clarity. Strain values begin to deviate from the initial slope as longitudinal shear failure occurs at the interface between cast-in-place concrete and hollowcores. Because of the smaller shear span strut and tie action occurs. A diagonal strut forms between the point of load application and the support. The longitudinal reinforcement and steel plate act as the tension tie. Strain gauges readings are consistent with this phenomenon reaching yield strain before unloading of the specimen. Pi gauge readings become skewed because of premature failures and development of strut and tie action that changes the cross section of the specimen.
Figure 4.8 - DK 4 Load Point Strains
4.2.2.4 Slip

Slip measurements were also taken for the DK 4 specimen. Horizontal and longitudinal failure mechanisms were observed during the testing of DK 4 causing slip measurements to significantly increase. Much of the movement measured occurred between the hollowcores the steel u-shaped plate while relatively no movement was measured between the cast-in-place concrete and steel plate. The maximum slip 0.22” occurred between the hollowcores and steel plate at the front of the specimen. Slip measurements are plotted against applied moment in Figure 4.9.

Figure 4.9 - DK 4 Slip Plots
4.3 Limit States

In addition to observing and comparing experimental behavior of the DK 3 and DK 4 specimens analyses of various design and serviceability limit states were completed. Design limit states analyzed included the flexural strength, vertical shear strength, horizontal shear strength, and longitudinal shear strength while the serviceability limit state analyzed was the deflection. Limit states were analyzed and compared for the full composite sections. Design and service limit states of the steel u-shaped plate under construction loads were analyzed using typical methods found within AISC 13. Methods for calculating and predicting various design strengths included a detailed layered sectional moment curvature analysis, AISC 13 code provisions, and ACI 318-08 code provisions.

4.3.1 Design Limit States

Design limit states must be satisfied to ensure life safety and inhibit structural collapse of a system. The design limit states of flexural strength, vertical shear strength, horizontal shear strength, and longitudinal shear strength for the composite sections were studied within this and previous thesis completed by Willis (2009) and Amortnont (2010). The following sections will provide detail for each limit state along with calculated values of respective strength for each specimen analyzed. Analytical design limit strengths for each specimen are compared with experimental results and observations in the closing section of this chapter.
4.3.1.1 Flexural Strength

Adequate flexural design strength is essential to maintain structural integrity and ensure life safety of inhabitants. Safety factors are multiplied against the nominal strength while load factors are multiplied against applied loads to protect from possible understrength or overloading of the member. Nominal flexural strengths of DK 3 and DK 4 were calculated in accordance with AISC 13 and ACI 318-08. Stress block analysis and moment curvature analysis were the two methods for calculating nominal flexural strengths. Important assumptions made for the calculation of flexural strength was that full shear interaction between the steel u-shaped plate, shear connectors, and cast-in-place concrete was maintained, and linear strain profiles remain throughout the depth of the sections. The moment curvature analyses utilized the Popovics model for concrete and hollowcores, the Ramberg-Osgood model for steel, and also considered tension strength of concrete during stress and force calculations. A detailed description of the moment curvature analysis procedure developed for analysis of DK specimens can be referenced from Appendix A of Amortnont (2010).

The method used for calculating the nominal flexural moment of the composite sections followed that of the AISC plastic stress distribution method (AISC 2005). An equivalent rectangular stress block, with depth $a=\beta_{1c}$ from the top of the section, is taken to model concrete within the compression zone as shown in Figure 4.10. Force equilibrium is done to determine the distance $a$ by summing concrete compressive force, given by Equation
4.1 and steel component compression or tension forces given by Equation 4.2, Equation 4.3, and Equation 4.4.

\[ C_c = 0.85 f'_c \beta_1 c b_{eff} \]  \hspace{1cm} \text{Equation 4.1}

Where:

- \( f'_c \) = concrete compressive strength, psi
- \( \beta_1 \) = factor relating depth of stress block to neutral axis depth, 0.65-0.85
- \( c \) = neutral axis depth, in.
- \( b_{eff} \) = effective width of section, in.

\[ C_{si} = A_{sci} f_y (\varepsilon_{si} \geq \varepsilon_y) \]  \hspace{1cm} \text{Equation 4.2}

\[ T_{zi} = A_{st} f_y (\varepsilon_{zi} \geq \varepsilon_y) \]  \hspace{1cm} \text{Equation 4.3}

\[ T_{pt_i} = A_{pt} f_y (\varepsilon_{pt_i} \geq \varepsilon_y) \]  \hspace{1cm} \text{Equation 4.4}

Figure 4.10 - Stress Distribution and Force Equilibrium
Where:

\[ A_{sc_i} = \text{compression steel rebar cross-sectional area, in}^2 \]
\[ A_{st_i} = \text{tension steel rebar cross-sectional area, in}^2 \]
\[ A_{pl_i} = \text{steel plate cross-sectional area, in}^2 \]
\[ f_y = \text{steel rebar yield strength, psi} \]
\[ F_y = \text{steel U-shaped plate yield strength, psi} \]
\[ \varepsilon_{si} = \text{strain in the steel rebar at the level, } d_{si} \]
\[ \varepsilon_{pl_i} = \text{strain in the steel U-shaped plate at the level, } d_{pl_i} \]
\[ \varepsilon_y = \text{yield strain of steel} \]

By equating compression and tension forces the equivalent stress block depth, \( a \), can be found using either Equation 4.5 or Equation 4.6 depending on the depth of \( a \).

\[ a = \beta_1 c = \frac{T - C_s}{0.85f'_c b_{eff}} \text{ for } (a \leq t_{slab}) \quad \text{Equation 4.5} \]
\[ a = \beta_1 c = \frac{1}{b_w} \left( \frac{T - C_s}{0.85f'_c} - b_{eff} t_{slab} \right) + t_{slab} \text{ for } (a > t_{slab}) \quad \text{Equation 4.6} \]

Where:

\[ T = \text{total tension force in steel rebar and U-shaped steel plate} = \Sigma T_{si} + \Sigma T_{pl_i}, \text{ lb} \]
\[ C_s = \text{total compression force in steel rebar} = \Sigma C_{si}, \text{ lb} \]
\[ b_w = \text{width of concrete web between hollow core plank faces, in.} \]
\[ a = \text{depth of equivalent compression zone, in} \]
The nominal moment capacities were calculated by summing moments at the top of the stress block, at the extreme compression fiber of the sections using Equation 4.7.

\[ M_n = -C_c \left( \frac{a}{2} \right) - C_{s_i} \left( d_{c_{s_i}} \right) + T_{s_i} \left( d_{T_{s_i}} \right) + T_{pl_i} \left( d_{T_{pl_i}} \right) \]  

Equation 4.7

Where:

- \( C_c \) = concrete compressive force, lb
- \( C_{s_i} \) = compression steel rebar force, lb
- \( T_{s_i} \) = tension steel rebar force, lb
- \( T_{pl_i} \) = U-shaped steel plate tension force, lb
- \( a \) = depth of compression zone stress block, in.
- \( d_{s_i} \) = distance from top compression fiber to centroid of reinforcement, in.
- \( d_{pl_i} \) = distance from top compression fiber to centroid of u-shape plate steel, in.

The design flexural strengths were calculated using the equivalent stress block method and moment curvature analysis, presented, and compared within Table 4.1.

**Table 4.1 - Experimental and Predicted Flexural Capacity**

<table>
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<th>Test ID</th>
<th>( M_{exp} ) (k-ft)</th>
<th>( M_n ) (k-ft)</th>
<th>( M_{M-\phi} ) (k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DK 1</td>
<td>765</td>
<td>799</td>
<td>864</td>
</tr>
<tr>
<td>DK 2</td>
<td>666</td>
<td>885</td>
<td>881</td>
</tr>
<tr>
<td>DK 3</td>
<td>790</td>
<td>950</td>
<td>911</td>
</tr>
<tr>
<td>DK 4</td>
<td>637</td>
<td>957</td>
<td>919</td>
</tr>
</tbody>
</table>
4.3.1.2 Vertical Shear Strength

AISC 13 Section I3 details the conservative method used to design members for vertical shear strength. The code states shear strength of composite beams utilizing shear connectors shall be determined according to AISC 13 Chapter G based on either the properties of the steel section alone or the concrete and shear reinforcing steel properties. It is noted by AISC that if the concrete and shear steel reinforcement properties are used that shear strength may be determined following ACI 318-08 Chapter 11. AISC Section G5 for rectangular HSS and box members states that vertical shear strength shall be calculated using provisions of Section G2.1 with \( A_w = 2ht \). The vertical shear strength is calculated for the steel section alone by Equation 4.8 given in AISC Section G2.

\[
V_h = 0.6 f_y A_w C_v 
\]

Equation 4.8

Where:

\[
A_w = \text{area of the web} = 2ht, \text{ in.}^2
\]

\[
h = \text{clear distance between flanges less the inside corner radius on top and bottom, in.}
\]

\[
t = \text{thickness of steel plate, in.}
\]

\[
C_v = \text{web shear coefficient}
\]

The alternative method for determining vertical shear strength of a composite beam is given by ACI Chapter 11, by Equation 4.9.

\[
V_h = V_c + V_s
\]

Equation 4.9
Where:

\( V_c = \) nominal shear strength provided by concrete, lb
\( V_s = \) nominal shear strength provided by shear reinforcement, lb

The respective strengths of the concrete and reinforcement steel components are given by Equation 4.10a, Equation 4.10ab, and Equation 4.11.

\[
V_c = 2\lambda \sqrt{f'_c} b_w d \quad \text{Equation 4.10a}
\]

\[
V_c = \left(1.9 \sqrt{f'_c} + 2500 \rho_w \frac{V_{u_d}}{M_{u_d}} \right) b_w d \leq 3.5 \lambda \sqrt{f'_c} b_w d \quad \text{Equation 4.10ab}
\]

\[
V_s = \frac{A_v f_{yt} d}{s} \quad \text{Equation 4.11}
\]

Where:

\( b_w = \) concrete web width, in.
\( d = \) distance from extreme compression fiber to centroid of longitudinal tension steel reinforcement, in
\( \lambda = \) modification factor reflecting the reduced mechanical properties of lightweight concrete
\( V_{u_d} = \) factored shear force at different sections of beam, lbs
\( M_{u_d} = \) factored moment at different sections of beam, lb-in
\( \rho_w = \) ratio of \( A_s \), longitudinal tension steel, to \( b_w d \)
\( A_v = \) area of shear reinforcement within spacing \( s \), in\(^2\)
\( f_{yt} = \) specified yield strength of shear reinforcement, psi
\( s = \) center-to-center spacing of shear reinforcement, in
Predicted shear capacities for the DK 3 and DK 4 sections calculated using both AISC and ACI methods are given along with experimental values within Table 4.2. Predicted and experimental shear values are also presented for previous beams tested under Willis (2009) and Amortnont (2010).

<table>
<thead>
<tr>
<th>Test ID</th>
<th>$V_{\text{exp}}$ (k)</th>
<th>AISC $V_n$ (k)</th>
<th>ACI $V_n$ (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DK 1</td>
<td>94.5</td>
<td></td>
<td>136.2</td>
</tr>
<tr>
<td>DK 2</td>
<td>82</td>
<td>89.1</td>
<td>138.4</td>
</tr>
<tr>
<td>DK 3</td>
<td>97.5</td>
<td>89.1</td>
<td>141.4</td>
</tr>
<tr>
<td>DK 4</td>
<td>274</td>
<td></td>
<td>142.3</td>
</tr>
</tbody>
</table>

4.3.1.3 **Horizontal Shear Strength**

AISC 13 Section I3.2d provide detailed design requirements to satisfy horizontal shear strength and load transfer at the interface between the steel u-shaped beam and cast-in-place concrete by mechanical shear connectors. Because the specimens under investigation are not fully concrete-encased beams the horizontal shear forces at the interface are assumed to be transferred by the shear connectors. The horizontal shear strength shall be determined as per AISC (Sect. I3.2d) to provide the full shear connection strength to the section. Equation 4.12 is the basic design equation for the limit state of horizontal shear strength.
stating the shear connector strength must be greater than or equal to the nominal horizontal shear demand.

\[
\Sigma Q_n \geq V'
\]

Equation 4.12

Where:

\[\Sigma Q_n = \text{sum of nominal strengths of shear connectors between the points of maximum positive moment and the point of zero moment, lb}\]

\[V' = \text{nominal horizontal shear demand, lb}\]

The total horizontal shear force, \(V'\), between the point of maximum positive moment and the point of zero moment is given as the lowest value for the limit states of concrete crushing or steel tensile yielding from Equation 4.13.

\[
V' = \min \left| \frac{0.85 f'c A_c + A_{st} f_y + A_{sc} f_y}{F_y A_{pl}} \right|
\]

Equation 4.13

Where:

\[A_c = \text{area of slab within concrete effective width, in.}^2\]

\[A_{pl} = \text{area of U-shaped steel plate cross section, in.}^2\]

The nominal strength of one stud shear connector embedded in solid concrete or in a composite slab is calculated by Equation 4.14.

\[
Q_n = 0.5 A_{stud} \sqrt{f'c E_c} \leq R_g R_p A_{stud} F_u
\]

Equation 4.14
Where:

\[ A_{stud} = \text{cross sectional area of stud shear connector, in.}^2 \]

\[ E_c = \text{modulus of elasticity of concrete} = w_c^{1.5} \times 3.3 \sqrt{f_c'}, \text{psi} \]

\[ F_u = \text{specified minimum tensile strength of a shear stud connector, psi} \]

\[ R_g = \text{coefficient to account for group effect} = 1.0 \text{ for any number of studs welded in a row directly to the steel shape} \]

\[ R_p = \text{position effect factor for shear studs} = 1.0 \text{ for studs welded directly to the steel shape} \]

\[ w_c = \text{weight of concrete per unit volume} (90 \leq w_c \leq 155), \text{lb/ft}^3 \]

The required number and longitudinal spacing of shear studs shall be determined by Equation 4.15 and Equation 4.16, respectively.

\[ n = \frac{V'}{Q_n} \]

Equation 4.15

\[ s = \frac{l}{n} \]

Equation 4.16

Where:

\[ n = \text{required number of shear studs between the point of maximum positive moment and the point of zero moment} \]

\[ s = \text{longitudinal spacing of shear studs, in.} \]

\[ l = \text{span between the point of maximum positive moment and the point of zero moment, in.} \]
The horizontal shear demands and capacities for each of the tests conducted in this research project are presented in Table 4.3 along with previous tests completed by Willis (2009) and Amortnont (2010).

### Table 4.3 - Predicted Horizontal Shear Capacity and Horizontal Shear Demand

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Capacity $\Sigma Q_n$ (k)</th>
<th>Actual Spacing</th>
<th>Demand $V'$ (k)</th>
<th>Required Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>DK 1</td>
<td>405.8</td>
<td>10&quot; Stagger Spacing</td>
<td>562.5</td>
<td>Double Line at 13.5&quot; spacing</td>
</tr>
<tr>
<td>DK 2</td>
<td>0</td>
<td>No Shear Studs</td>
<td>562.5</td>
<td>8&quot; Stagger or Double Line at 14&quot; Spacing</td>
</tr>
<tr>
<td>DK 3</td>
<td>593.1</td>
<td>8&quot; Stagger Spacing</td>
<td>562.5</td>
<td>8&quot; Stagger or Double Line at 14&quot; Spacing</td>
</tr>
<tr>
<td>DK 4</td>
<td>161.8</td>
<td>10&quot; Stagger Spacing</td>
<td>562.5</td>
<td>Double Line at 2.5&quot; Spacing</td>
</tr>
</tbody>
</table>

It should be noted that the purpose of the DK 4 test was to induce a vertical shear failure of the specimen. The reduced shear span requires an exceptional amount of shear stud connectors with a very small spacing needed which is not a typical design detail. The DK 1, 2, and 3 shear stud spacing requirement would be a typical design spacing for large open bay framing configurations.
The shear stud connectors for each beam were required to transfer the force generated by yielding of the steel u-shaped plate, which is the force in the steel plate at the point the sections each reach their nominal moment capacity.

4.3.1.4 Longitudinal Shear Strength

Longitudinal shear strength was found to be an additional limit state that must be designed for but was initially not. ACI 318-08 Section 11.6.3 states that the required shear friction reinforcement, $A_vf$, across the shear plane shall be designed using either Section 11.6.4 or any other shear transfer design method that result in prediction of strength in substantial agreement with results of comprehensive tests (ACI 2008). The total nominal longitudinal shear friction strength is given by Equation 4.17, which is the addition of the strength provided by steel reinforcement and the strength of concrete located in the compression zone between longitudinal shear planes.

$$V_{ln} = \mu A_v f_y$$

Equation 4.17

Where:

- $A_v$ = area of shear friction reinforcement, in.\(^2\)
- $f_y$ = specified yield strength of shear friction reinforcement, not to exceed 60,000 psi
- $\mu$ = coefficient of friction
To determine the amount of transverse steel the longitudinal shear strength equation is rearranged as shown in Equation 4.18.

\[ A_{vf} = \frac{(V_{lt})}{f_y \mu} \]  

Equation 4.18

It is assumed that cracking will occurred along the shear planes present within the system along which shear forces are to be resisted by dowel action of transverse steel, friction between crack faces, and resistance to shearing off of protrusions on the crack faces.

The longitudinal shear demand per unit length of a single plane of the member was determined as per AASHTO LRFD Bridge Design Specification 2004 and ACI 318-99 and multiplied by the longitudinal length of the shear plane. This provision is presented in the Alternative Design Method in Appendix A of ACI Codes prior to 2002. The demand equation has been modified such that the amount of force carried by concrete between shear planes, Figure 4.11, is subtracted from the total demand. The remaining force is then divided by two to obtain the longitudinal shear demand for one plane as shown in Equation 4.19.

![Figure 4.11 - Concrete Between Longitudinal Shear Planes](image-url)
\[ V_{tu} = \frac{\left( \frac{V_u}{b_d} \right) (l \times b) - 0.85 f'_c A_{cs}}{2} \]

Equation 4.19

Where:

\[ V_u = \text{maximum factored vertical shear in span of interest, lbs} \]

\[ d_e = \text{distance between the centroid of the tension force to the centroid of the compression force in the transformed cracked section, in.} \]

\[ l = \text{longitudinal length of shear plane, in.} \]

\[ b = \text{width of the web at the level under consideration, in.} \]

\[ A_{cs} = \text{area of equivalent stress block within longitudinal shear planes, in}^2 \]

It should be noted that the distance, \( d_e \), is calculated assuming cracked moment of inertia section properties. The experimental longitudinal shear demands, \( V_{lu,exp} \), and predicted longitudinal shear strengths, \( V_{ln} \), for DK 1, 2, 3, and 4 are given in Table 4.4.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>( V_{ln} ) (k)</th>
<th>( V_{lu,exp} ) (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DK 1</td>
<td>95</td>
<td>262</td>
</tr>
<tr>
<td>DK 2</td>
<td>95</td>
<td>213</td>
</tr>
<tr>
<td>DK 3</td>
<td>95</td>
<td>284</td>
</tr>
<tr>
<td>DK 4</td>
<td>66</td>
<td>207</td>
</tr>
</tbody>
</table>
4.3.2 Serviceability Limit States

Dead load deflection of the steel u-shaped beam and live load deflection of the complete composite section were serviceability limit states that need to be designed for. The dead load deflection of the u-shaped steel plate can be determined at any stage of construction for various loading and support conditions as in AISC Table 3-23. The calculated deflection shall be lower than an acceptable value determined by appropriate standards or through the engineer’s discretion.

To satisfy the live load deflection serviceability limit state, deflections at service load levels must not exceed limits set by building codes. The 2009 IBC limits the unfactored live load deflection to L/360 for floor systems (IBC 2009). Minimum live loads for occupancy or use are given in Table 4-1 in Chapter 4 of ASCE 7-05. A live load reduction can be used in accordance with the provisions of Section 4.8 of ASCE 7-05 and is given by Equation 4.20. A reduced live load must satisfy the minimum requirement of $K_{LL}A_T$ greater than 400 ft$^2$ and shall not be used for live loads greater than 100 psf or for live loads less than or equal to 100 psf in public assembly occupancies.

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

Equation 4.20

Where:

$$L = \text{ reduced design live load per square foot of area supported by the member, psf}$$
\[ L_o = \text{unreduced design live load per square foot of area supported by the member, psf} \]

\[ K_{LL} = \text{live load element factor, based on Table 4-2 of ASCE (2006)} \]

\[ A_T = \text{tributary area, ft}^2 \]

The unreduced service live loading of the DK specimens was determined to be 100 psf after consulting Table 4-1 of ASCE 7-05. 100 psf unreduced live loading was the minimum uniform live loading for lobbies and first floor corridors of office buildings and public rooms and corridors serving them in multi residential buildings. The \( K_{LL} \) factor was taken to be 2 for an interior beam as per Table 4-2 in ASCE 7-05 while the tributary area, \( A_T \), was calculated to be 810 ft\(^2\) (ASCE 2006). The resulting reduced live load was calculated to be 62.3 psf for each of the DK specimens. The corresponding moment was determined from the reduced live load from which the deflection was measured and compared to predicted values determined using the moment area method and transformed cracked section analysis. The reduced live load moments were calculated to be 340.45 k-ft for the DK 1, 2, and 3 specimens and 39.67 k-ft for the DK 4 specimen.

Deflections were calculated using the transformed effective moment of inertia of the composite member. Steel area was transformed into equivalent an area of concrete using the modular ratio, \( n \), and the transformed area equation given in Equation 4.21.

\[ A_t = A_c + nA_e \]

Equation 4.21
Where:

\[
A_t = \text{transformed area, in.}^2 \\
A_c = \text{concrete area, in.}^2 \\
A_s = \text{steel area, in.}^2 \\
A_g = \text{gross area, in.}^2
\]

The effective transformed moment of inertia, \( I_e \), is given in Equation 4.22.

\[
I_e = \left( \frac{M_{cr}}{M_a} \right)^3 I_g + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \leq I_g
\]  
Equation 4.22

Where:

\[
M_{cr} = \text{cracking moment, lb-in.} \\
M_a = \text{maximum moment in member, due to service load, lb-in.} \\
I_g = \text{moment of inertia of gross transformed section, in.}^4 \\
I_{cr} = \text{moment of inertia of cracked transformed section, in.}^4
\]

The cracking moment, the gross moment of inertia, and the cracked moment of inertia were all found in the process of determining the effective moment of inertia of each of the specimens. The effective transformed moment of inertia was then used to calculate the deflection at any stage of loading. The deflections of DK 1, 2, 3, and 4 are presented in
Table 4.5 alongside predicted deflection values found using transformed cracked section analysis and moment curvature procedures.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Experimental (in.)</th>
<th>Transformed Cracked Section (in.)</th>
<th>Moment Curvature (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DK 1</td>
<td>0.94</td>
<td>0.99</td>
<td>1.00</td>
</tr>
<tr>
<td>DK 2</td>
<td>1.11</td>
<td>0.96</td>
<td>0.96</td>
</tr>
<tr>
<td>DK 3</td>
<td>0.94</td>
<td>0.93</td>
<td>0.91</td>
</tr>
<tr>
<td>DK 4</td>
<td>0.01</td>
<td>0.0014</td>
<td>0.0014</td>
</tr>
</tbody>
</table>

4.3.3 Comparison of Results

Comparisons are made between experimental results and individual design limit states in Table 4.6. Design ratios are presented comparing observed limits state values to predicted values for respective design limit states. All specimens tested were unable to achieve nominal flexural strength due to premature ultimate failure due to concrete crushing. In the case of DK 2 a combination of longitudinal and horizontal interface shear cracking occurred which led to a premature flexural failure of the specimen. Vertical shear failure was not observed during any of the DK tests due to the conservative design method. The design vertical shear strength presented by AISC was surpassed experimentally during all the tests except for that of DK 2 and DK 4 while the ACI vertical shear strength was conservative. However, the vertical shear failure mode was not observed during any of the tests conducted. It should be noted that the experimental vertical shear of the DK 4 test was greater than the
summation of the AISC and ACI calculated nominal strengths, indicating that both approaches produce conservative design values even when combined. Longitudinal shear cracking was observed to be the primary cause for reducing the ultimate moment capacity of all four specimens through reduction of the effective compression flange. For the cases of DK 1, 2, and 3 the longitudinal shear strength was approximately one third of the experimental longitudinal shear demand. In the case of DK 1 and 2 this phenomenon was unexpected and unaccounted for in the initial design procedure. The available longitudinal shear strength present in DK 1 and 2 was due to transverse steel placed during construction between shear keyways of hollowcores. The DK 3 and DK 4 design included 4” concrete infill into hollowcore cores to increase the longitudinal shear capacity. After completion of testing it was determined that a conservative design procedure for longitudinal shear must be developed as longitudinal shear cracking was again observed in both the DK 3 and DK 4 testing. Loss of horizontal shear interaction occurred during the DK 2 test at approximately the same load level as longitudinal shear cracking due to the absence of horizontal shear connectors. The combination of behaviors caused the DK 2 specimen to fail prematurely in flexure. It should be noted that although the horizontal shear design ratio for DK 1 was less than one horizontal shear interaction loss was not an observed. It should also be noted that the DK 4 vertical shear, horizontal shear, and longitudinal shear design ratios are less than one. Because DK 4 was intended to fail in vertical shear experimental values of horizontal and longitudinal shear were extreme, and would not be found within a typical design of the composite sections. However, it is evident that the vertical shear design is highly
conservative as the experiment vertical shear was more than double the AISC (2005) or ACI (2008) predicted vertical shear strength.

Table 4.6 - Limit State Comparative Design Ratios

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Test ID</th>
<th>DK 1</th>
<th>DK 2</th>
<th>DK 3</th>
<th>DK 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure M-φ</td>
<td>M_{M-φ}/M_{exp}</td>
<td>1.13</td>
<td>1.32</td>
<td>1.15</td>
<td>-</td>
</tr>
<tr>
<td>Flexure ACI</td>
<td>M_{n}/M_{exp}</td>
<td>1.04</td>
<td>1.33</td>
<td>1.20</td>
<td>-</td>
</tr>
<tr>
<td>Horizontal Shear</td>
<td>ΣQ_{n}/V'</td>
<td>-</td>
<td>1.05</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Longitudinal Shear</td>
<td>V_{ln}/V_{lu,exp}</td>
<td>0.36</td>
<td>0.45</td>
<td>0.33</td>
<td>-</td>
</tr>
<tr>
<td>Vertical Shear AISC</td>
<td>V_{n,AISC}/V_{exp}</td>
<td>0.94</td>
<td>1.09</td>
<td>0.91</td>
<td>0.33</td>
</tr>
<tr>
<td>Vertical Shear ACI</td>
<td>V_{n,ACI}/V_{exp}</td>
<td>1.44</td>
<td>1.69</td>
<td>1.45</td>
<td>0.52</td>
</tr>
</tbody>
</table>

Notes:

1. Shaded cells indicate observed behavior modes
2. Gradient shaded cells indicate initial behavior mode
3. Red values indicate the predicted behavior mode from design limit states
4. Design ratios less than or equal to one are conservative
5. Design ratios greater than one may be unconservative
6. Horizontal shear was not considered for DK 2 on account of no shear studs in the design

4.4 Summary

Observations and comparisons of results from DK specimen tests indicate that longitudinal shear is the primary failure mechanism, responsible for the reduction of the
effective flange during all tests and the premature flexural failure. Although the DK 4 test setup was uncharacteristic results showed that a conservative design approach was taken for vertical shear strength. Horizontal shear failure occurred only within the DK 2 specimen because of the absence of shear stud connectors, but appeared to be adequately design for in all other tests. It is undetermined however, the relationship, if any, between the horizontal and longitudinal shear limit states. Longitudinal shear failure was an unexpected failure mode first discovered during the testing of DK 1 and DK 2 by Willis (2009) for which there were no design provisions. Observations and calculations of design ratios less than one confirm that a conservative design approach has been taken with regard to longitudinal shear failure. Although strengthening for longitudinal shear was done for the testing of the DK 3 and DK 4 specimens no significant strength gain was observed as the design ratios are consistent. A provisional design procedure was developed during the analysis and comparison of all the DK specimen test results that addresses the design limit states of flexural, vertical shear, horizontal shear, and longitudinal shear strength as well as construction dead load deflection and service live load deflection. This provisional design procedure was intended to be verified through subsequent testing but was not because of shortage of project funds and other circumstances ending the project early. The provisional design procedure is presented in Chapter 5.
5 DESIGN PROCEDURE

5.1 Introduction

This chapter presents the proposed design procedure regarding the design of Diversakore® shallow floor framing systems. The proposed design procedure is briefly outlined in the following section to give an overall understanding of the design process. The three phase design procedure outline is followed by the unabridged design procedure after which a discussion of special design considerations is presented. A full detailed design example is presented at the end of this chapter.

5.2 Procedure Outline

The design procedure is a three phase process wherein the engineer will design the steel u-shaped section for the construction loading, design tension and/or compression flexure reinforcement under fire loading conditions, and then design the full composite section for the specified ultimate loading. Appropriate assumptions are to be made of bay size, approximate loading, and relative size of the composite beam to be used. Within all phases deflection checks are to be done for respective loading considering appropriate deflection limits. The design procedure outline is presented in the following section.
5.2.1 Design Procedure Outline

- Select trial steel u-shaped section size.

- Design the appropriate amount of shoring to satisfy the limit states of positive and negative flexure strength under construction loading.

- Check vertical shear strength of the steel section under construction loading.

- Verify dead load deflection limit of steel u-shaped section under construction loading.

- Design the tension and compression flexure reinforcement for the fire loading case.

- Analyze and verify the full composite section strength under ultimate loading conditions.

- Determine the appropriate size and spacing of stirrups to satisfy the ultimate vertical shear capacity.
• Determine the amount of mechanical shear studs to resist the ultimate horizontal shear demand coinciding with the ultimate moment capacity of the composite section.

• Design longitudinal shear reinforcement to satisfy the ultimate longitudinal shear demand coinciding with the ultimate moment capacity of the composite section.

• Verify live load service deflection limit utilizing effective transformed moment of inertia section properties under service live loading.
5.2.2 Design Procedure

The proposed design procedure for the Diversakore® shallow floor framing system is given within this section. This section details a two stage process whereby the designer enters the procedure with initial assumptions of bay size, general member sizes, and approximate construction and ultimate loading conditions. The outcome of the procedure is a section that satisfies the given design criteria and the amount of shoring needed during the construction phase.

5.2.2.1 Strength Design Guidelines

Stage I – Construction

This section applies to the U-shaped steel plate for flexure and vertical shear.

Flexural Strength

This section applies to the U-shaped steel plate bent about the minor axis. The u-shaped plate shall be designed for positive and negative bending, negative bending which may be present due to shoring required for construction.

General Provisions

\[ \phi_b M_n \geq M_u \]  

Equation 5.1
Where:

\[ \phi_b = \text{strength reduction factor for flexure, 0.90 as per AISC 13 Section F1 (AISC 2005)} \]

\[ M_n = \text{nominal flexural strength, lb-in.} \]

\[ M_{fa} = \text{maximum factored moment (negative or positive) within the span of interest, lb-in.} \]

**Nominal Flexural Strength**

The flexural strength shall be determine as the lowest value from the limit states of yielding and flange local buckling as per AISC Section F6 (AISC 2005). The section shall be designed with compact webs and compact return flanges if possible. Provisions are also given for noncompact and slender design.

**Yielding**

\[ M_n = M_p = F_y Z_y \leq 1.6F_y S_y \quad \text{Equation 5.2} \]

Where:

\[ F_y = \text{specified minimum yield stress of the u-shaped plate, psi} \]

\[ Z_y = \text{plastic section modulus about the y-axis, in.}^3 \]

\[ S_y = \text{minimum section modulus, in.}^3 \]
Flange Local Buckling

For sections with compact flanges ($\lambda_f \leq \lambda_{pf}$):

$$M_n = M_p$$  \hspace{1cm} \text{Equation 5.3}

For sections with noncompact flanges ($\lambda_{pf} < \lambda_f \leq \lambda_{rf}$):

$$M_n = \left[ M_p - \left( M_p - 0.7F_yS_y \right) \left( \frac{\lambda_f}{\lambda_{rf}} - \frac{\lambda_{pf}}{\lambda_{rf}} \right) \right]$$  \hspace{1cm} \text{Equation 5.4}

For sections with slender flanges ($\lambda_f > \lambda_{rf}$):

$$M_n = F_{cr}S_y$$  \hspace{1cm} \text{Equation 5.5}

Where:

$$F_{cr} = \frac{0.69Ek_c}{\left( \frac{b_f}{2t_f} \right)^2}$$  \hspace{1cm} \text{Equation 5.6}

$$k_c = \frac{4}{\sqrt{h/t_w}} \text{ and shall not be taken less than}$$  \hspace{1cm} \text{Equation 5.7}

0.35 nor greater than 0.75 for calculation purposes
For positive bending, \( \lambda \) values shall be determined following AISC 13 Table B4.1 (AISC 2005) as:

\[
\lambda_{f_i} = \frac{b_{f_i}}{t_{f_i}}
\]

Equation 5.8

\[
\lambda_{pf_i} = 0.38 \sqrt{\frac{E}{F_y}}
\]

Equation 5.9

\[
\lambda_w = \frac{h}{t_w}
\]

Equation 5.10

\[
\lambda_{wp} = \frac{h_c}{h_p} \sqrt{\frac{E}{F_y}} \leq \lambda_r
\]

Equation 5.11

\[
\lambda_{wr} = 5.70 \sqrt{\frac{E}{F_y}}
\]

Equation 5.12

Where:

\( h_c \) = twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius as shown in Figure 5.1, in.

\( h_p \) = twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used as shown in Figure 5.1, in.

\( M_y \) = yield moment about the axis of bending, kip-in

\( E \) = modulus of elasticity of steel, 29,000 ksi
Figure 5.1 – Steel U-Shaped Plate Flange Width-Thickness for Positive Bending

For negative bending, $\lambda$ values shall be determined following AISC 13 Table B4.1 (AISC 2005) as:

$$\lambda_{f_2} = \frac{b_{f_2}}{t_{f_2}}$$  \hspace{1cm} \text{Equation 5.13}

$$\lambda_{p_{f_2}} = 1.12 \sqrt{\frac{E}{F_y}}$$  \hspace{1cm} \text{Equation 5.14}

$$\lambda_{r_{f_2}} = 1.40 \sqrt{\frac{E}{F_y}}$$  \hspace{1cm} \text{Equation 5.15}

$$\lambda_{wp} = \frac{\frac{h_c}{h_p} \sqrt{\frac{E}{F_y}}}{\left(0.54 \frac{M_p}{M_y} - 0.09\right)^2} \leq \lambda_r$$  \hspace{1cm} \text{Equation 5.16}
\[ \lambda_{wr} = 5.70 \sqrt{E/F_y} \]

Equation 5.17

Where:

\[ h_c = \text{twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius as shown in Figure 5.2, in.} \]

\[ h_p = \text{twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used as shown in Figure 5.2, in.} \]

\[ M_y = \text{yield moment about the axis of bending, kip-in} \]

\[ E = \text{modulus of elasticity of steel, 29,000 ksi} \]

**Figure 5.2 - Steel U-Shaped Plate Flange Width-Thickness for Negative Bending**

**Shoring**

The amount of shoring is designed utilizing AISC 13 Table 3-22c (AISC 2005) such that \( \phi_b M_n \geq M_u \) is satisfied for both positive and negative flexure and \( \phi_v V_n \geq V_u \). Table 5.1 is a representative table of an AISC table presenting moment and shear coefficients for equally
loaded continuous spans. The flexure coefficients, $C_1$, are in terms of $wl^2$ and the shear coefficients, $C_2$, are in terms of $wl$ where $l$ is equal to span divided by the number of shore points plus one as in Equation 5.18. The ultimate applied moments and shear are found using Equation 5.19, Equation 5.20, and Equation 5.21.

$$l = \frac{\text{span}}{n + 1}$$  \hspace{1cm} \text{Equation 5.18}

Where:

$n = \text{number of shore points}$

<table>
<thead>
<tr>
<th>n</th>
<th>$C_1$ Positive Flexure</th>
<th>$C_2$ Negative Flexure</th>
<th>$C_3$ Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.07</td>
<td>-0.125</td>
<td>5/8</td>
</tr>
<tr>
<td>2</td>
<td>0.08</td>
<td>-0.10</td>
<td>6/10</td>
</tr>
<tr>
<td>3</td>
<td>0.077</td>
<td>-0.107</td>
<td>17/28</td>
</tr>
<tr>
<td>4</td>
<td>0.078</td>
<td>-0.105</td>
<td>23/38</td>
</tr>
<tr>
<td>5</td>
<td>0.078</td>
<td>-0.106</td>
<td>63/104</td>
</tr>
<tr>
<td>6</td>
<td>0.078</td>
<td>-0.106</td>
<td>86/142</td>
</tr>
</tbody>
</table>

$$M_u^+ = C_1 wl^2$$  \hspace{1cm} \text{Equation 5.19}

$$M_u^- = C_2 wl^2$$  \hspace{1cm} \text{Equation 5.20}

$$V_u = C_3 wl$$  \hspace{1cm} \text{Equation 5.21}
**Vertical Shear Strength**

This section applies to the U-shaped steel plate. Vertical shear strength shall be checked at the supports and near points of shoring, if applicable.

**General Provisions**

\[ \phi_v V_n \geq V_u \]  \hspace{1cm} \text{Equation 5.22}

Where:

\[ \phi_v = \text{strength reduction factor for shear} = 1.00 \text{ as per AISC 13 Section G2.1 (AISC 2005)} \]

\[ V_n = \text{nominal vertical shear strength, lb} \]

\[ V_u = \text{maximum factored vertical shear strength within the span of interest, lb} \]

**Nominal Vertical Shear Strength**

The vertical shear strength shall be determined according to the limit states of shear yielding and shear buckling. AISC 13 Section G5 states that \( V_n \) of rectangular HSS and box members shall be determined using provisions of Section G2.1 with \( A_w = 2ht \). (AISC 2005)
Shear Yielding and Shear Buckling

\[ V_n = 0.6F_y A_w C_v \]  

Equation 5.23

Where:

\[ A_w = \text{area of the web} = 2ht, \text{ in.}^2 \]

\[ h = \text{clear distance between the flanges less the inside corner radius on the top and bottom as shown in Figure 5.3, in.} \]

\[ t = \text{thickness of steel plate as shown in Figure 5.3, in.} \]

\[ C_v = \text{web shear coefficient} \]

\[ r = \text{corner radius, calculated as } 1.5t \]

\( C_v \) is determined as follows:

(i) For \( h/t \leq 1.10 \sqrt{k_v E/F_y} \)

\[ C_v = 1.0 \]  

Equation 5.24

(ii) For \( 1.10 \sqrt{k_v E/F_y} < h/t \leq 1.37 \sqrt{k_v E/F_y} \)

\[ C_v = \frac{1.10 \sqrt{k_v E/F_y}}{h/t} \]  

Equation 5.25

(iii) For \( h/t > 1.37 \sqrt{k_v E/F_y} \)

\[ C_v = \frac{1.10 k_v}{(h/t)^2 F_y} \]  

Equation 5.26
Where:

The web plate buckling coefficient, $k_v$, is determined as follows:

(i) For unstiffened webs with $h/t < 260$, $k_v = 5$  

Equation 5.27

(ii) For stiffened webs,

\[
  k_v = 5 + \frac{5}{(a/h)^2}
\]

Equation 5.28

Where:

\[ a = \text{clear distance between transverse stiffeners, in.} \]

\[ h = \text{the clear distance between flanges less the fillet or corner radii, in.} \]

The standard U-shaped steel plate is unstiffened, but stiffened designs are acceptable, requiring a different value of $k_v$, the web plate buckling coefficient.

![Figure 5.3 - Steel U-Shaped Plate Web Depth-Thickness](image.png)
Stage II – Fire

This section applies to the composite floor system for the limit states of flexure and shear under a specified fire loading case. The design of the Diversakore® beam under fire loading conditions does not consider the steel u-shaped section as tension flexural reinforcement as the steel section is considered to have no strength at elevated fire temperatures.

Live Load Reduction

Minimum live loads for occupancy or use are given in Table 4-1 in Chapter 4 of ASCE 7-05 (ASCE 2006). A live load reduction can be used in accordance with the provisions of Section 4.8 of ASCE 7-05 and is given by Equation 5.29. A reduced live load must satisfy the minimum requirement of \( K_{LL} A_T \) greater than 400 ft\(^2\) and shall not be used for live loads greater than 100 psf or for live loads less than or equal to 100 psf in public assembly occupancies.

\[
L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right) \quad \text{Equation 5.29}
\]

Where:

\( L \) = reduced design live load per square foot of area supported by the member, psf

\( L_o \) = unreduced design live load per square foot of area supported by the member, psf

\( K_{LL} \) = live load element factor

\( A_T \) = tributary area, ft\(^2\)
The live load element factor, $K_{LL}$, is based on Table 4-2 of ASCE 7-05 as shown in Table 5.2.

**Table 5.2 - Live Load Element Factor**

<table>
<thead>
<tr>
<th>Element</th>
<th>(K_{LL}^a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior columns</td>
<td>4</td>
</tr>
<tr>
<td>Exterior columns without cantilever slabs</td>
<td>4</td>
</tr>
<tr>
<td>Edge columns with cantilever slabs</td>
<td>3</td>
</tr>
<tr>
<td>Corner columns with cantilever slabs</td>
<td>2</td>
</tr>
<tr>
<td>Edge beams without cantilever slabs</td>
<td>2</td>
</tr>
<tr>
<td>Interior beams</td>
<td>2</td>
</tr>
<tr>
<td>All other members not identified including:</td>
<td></td>
</tr>
<tr>
<td>Edge beams with cantilever slabs</td>
<td></td>
</tr>
<tr>
<td>Cantilever beams</td>
<td></td>
</tr>
<tr>
<td>One-way slabs</td>
<td></td>
</tr>
<tr>
<td>Two-way slabs</td>
<td></td>
</tr>
<tr>
<td>Members without provisions for continuous</td>
<td></td>
</tr>
<tr>
<td>shear transfer normal to their span</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
</tbody>
</table>

\(^a\) In lieu of the preceding values, $K_{LL}$ is permitted to be calculated.

**Flexural Strength**

The flexural strength shall be determined using concrete stress block analysis as per ACI 318-8 Chapter 10 (ACI 2008).

**General Provisions**

\[
\phi_b M_n \geq M_u
\]

Equation 5.30

Where:

\[
\phi_b = \text{strength reduction factor for flexure, 0.65-0.90 as per ACI 318-08 (ACI 2008)}
\]

\[
M_n = \text{nominal flexural strength, lb-in.}
\]
\[ M_u = \text{maximum factored moment (negative or positive) within the span of interest, lb-in.} \]

Flexural Fire Loading

AISC Design Guide 19 specifies a fire load combination defined by Equation 5.31 (AISC 2003).

\[ M_u = 1.2D + 0.5L \]  

Equation 5.31

Nominal Flexural Strength

Nominal flexural strength under fire loading is calculated using an equivalent stress block design and analysis procedure. After initial estimation of tension flexural reinforcement, the stress block analysis is performed to confirm the flexural design strength under fire loading. The initial estimation of tension flexural reinforcement is calculated as per ACI 318-08 T-beam design (ACI 2008):

\[ A_s = \frac{M_u}{\phi f_y j d} \]  

Equation 5.32

Where:

\[ \phi = \text{strength reduction factor for flexure, estimated as 0.90 for initial calculation of tension reinforcement as per ACI 318-08 (ACI 2008)} \]

\[ M_u = \text{maximum factored moment, negative or positive, within the span of interest, lb-in.} \]
\( f_y = \text{specified minimum yield stress of the flexural reinforcement, psi} \)

\( j d = \text{flexural lever arm, estimated as 0.95d, in.} \)

**Strength Reduction Factor**

After the initial selection of tension flexural reinforcement, the strength reduction factor, \( \phi_b \), shall be taken as (ACI 2008):

\[
\phi_b = \begin{cases} 
0.65 \quad \text{(Compression Controlled)} \\
0.65 + (\varepsilon_t - 0.002)(250/3) \quad \text{(Transition)} \\
0.90 \quad \text{(Tension Controlled)} 
\end{cases}
\]

Equation 5.33

ACI 318-08 Chapter 10.3 explains that sections are considered compression controlled if the strain in the extreme tension steel is less than 0.002 when the extreme compression fiber reaches the assumed strain limit of 0.003. Sections are considered tension controlled if the strain in the extreme tension steel is greater than 0.005 when the extreme compression fiber reaches the assumed strain limit of 0.003. A transition zone is defined between the compression control limit, 0.002, and the tension control limit, 0.005. The strength reduction factor strain limits are depicted in Figure 5.4.
Effective Flange Width

The effective width of the concrete slab, \( b_{eff} \), is determined as per ACI 318-08 Chapter 8.12 (ACI 2008) or AISC 13 Section I3.1a (AISC 2005). The effective width, depicted in Figure 5.5, is calculated using the thickness of the concrete slab plus the thickness of the top flute of the hollow core planks. The effective width of the concrete slab shall adhere to Equation 5.34.

\[
b_{eff} \leq \frac{L}{4}
\]

Equation 5.34

Where:

\( L = \) beam span, center-to-center of supports, in.
The effective overhanging flange width on each side of the web shall adhere to Equation 5.35.

\[
\text{Overhang} = \min \left[ \frac{bt_{slab}}{L_c/2} \right]
\]

Equation 5.35

Where:

\[
t_{slab} = \text{slab thickness, including the top flute of the hollow core plank, in.}
\]

\[
L_c = \text{clear distance to the next web, in.}
\]

**Figure 5.5 – Effective Flange Width**

**Flexural Design Using Stress Block Analysis**

The ultimate flexural capacity shall be determined by calculating the sum of moments about the top of the stress block using Equation 5.36. If it is found that compression reinforcement is needed then Equation 5.37 shall be used. Figure 5.6 is a depiction of the equivalent stress
block method used to determine the nominal moment capacity. It should be noted that the
steel u-shaped section is not considered within the nominal moment calculation.

Figure 5.6 – Stress Distribution and Force Equilibrium

\[ M_n = -C_c \left( \frac{a}{2} \right) + T_{s_l} \left( d_{s_l} \right) \]  
\[ M_n = -C_c \left( \frac{a}{2} \right) - C_{s_l} \left( d_{c_{s_l}} \right) + T_{s_l} \left( d_{t_{s_l}} \right) \]

Where:

- \( C_c \) = concrete compressive force, lb
- \( C_{s_l} \) = compression steel rebar force, lb
- \( T_{s_l} \) = tension steel rebar force, lb
- \( a \) = depth of compression zone stress block, in.
- \( d_{s_l} \) = distance from extreme compression fiber to centroid of reinforcement, in.
The concrete compressive force within the stress block is given by Equation 5.38. If different concrete compressive strengths are within the stress block, an iterative procedure and appropriate engineering judgment should be exercised to find the correct compression zone depth.

\[ C_c = 0.85 f_c \beta_1 c b_{eff} \]  
Equation 5.38

Where:

\[ f_c' = \text{concrete compressive strength, psi} \]

\[ \beta_1 = \text{factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, 0.65-0.85, given by Equation 5.39} \]

\[ c = \text{neutral axis depth, in.} \]

\[ b_{eff} = \text{effective width, in.} \]

The \( \beta_1 \) factor is dependent on the concrete compressive strength and is determined by the following (ACI 2008):

\[ \beta_1 = \begin{cases} 
0.85 & \text{for } (f_c \leq 4000 \text{ psi}) \\
0.65 - \frac{f_c - 4000}{20,000} & \text{for } (4000 < f_c < 8000 \text{ psi}) \\
0.65 & \text{for } (f_c \geq 8000 \text{ psi}) 
\end{cases} \]  
Equation 5.39
The compressive and tensile forces in the steel rebar are based on the assumption that the rebar steel yields and are given by Equation 5.40 and Equation 5.41, respectively. For increased accuracy it is advised to treat rebar at different depths as separate layers.

\[ C_{s_i} = A_{sci} f_y \ (\varepsilon_{s_i} \geq \varepsilon_y) \]  
Equation 5.40

\[ T_{s_i} = A_{sti} f_y \ (\varepsilon_{s_i} \geq \varepsilon_y) \]  
Equation 5.41

Where:

- \( A_{sci} \) = compression steel rebar cross-sectional area, in.\(^2\)
- \( A_{sti} \) = tension steel rebar cross-sectional area, in.\(^2\)
- \( f_y \) = steel rebar yield strength, psi
- \( \varepsilon_{s_i} \) = strain in the steel rebar at the level, \( d_{s_i} \)
- \( \varepsilon_y \) = yield strain of steel

By equating the compressive and tension forces, the depth of the concrete compressive stress block, \( a \), shall be solved for as in Equation 5.42. If the depth of the concrete compressive stress block exceeds the depth of the slab, Equation 5.43 shall be used.

\[ a = \beta_1 c = \frac{T - C_s}{0.85 f'_c b_{eff}} \text{ for } (a \leq t_{slab}) \]  
Equation 5.42

\[ a = \beta_1 c = \frac{1}{b_{w}} \left( \frac{T - C_s}{0.85 f'_c} - b_{eff} t_{slab} \right) + t_{slab} \text{ for } (a > t_{slab}) \]  
Equation 5.43
Where:

\[ T = \text{total tension force in steel rebar} = \Sigma T_{sl}, \text{ lb} \]
\[ C_s = \text{total compression force in steel rebar} = \Sigma C_{sl}, \text{ lb} \]
\[ b_w = \text{width of concrete web between hollow core plank faces, in.} \]

The assumptions of compression and tension steel yielding shall be checked. Using the calculated neutral axis depth and strain compatibility the strain at any depth can be determined as in Equation 5.44. If the strain in steel components is found to be less than the yield strain, the force contribution of the steel components shall be adjusted as in Equation 5.45 and Equation 5.46 resulting in new neutral axis depth. This process shall be iterated until the neutral axis depth satisfies both force equilibrium and strain compatibility.

\[ \varepsilon_l = \varepsilon_{cu} \left( \frac{|d_l - c|}{c} \right) \quad \text{Equation 5.44} \]

\[ C_{sl} = A_{scl} E_s \varepsilon_{s_{sl}} (\varepsilon_{s_{sl}} < \varepsilon_y) \quad \text{Equation 5.45} \]

\[ T_{sl} = A_{sti} E_s \varepsilon_{s_{sl}} (\varepsilon_{s_{sl}} < \varepsilon_y) \quad \text{Equation 5.46} \]

Where:

\[ \varepsilon_{cu} = \text{assumed strain limit of concrete in compression} = 0.003 \]
\[ E_s = \text{elastic modulus of steel, psi} \]
**Vertical Shear Strength**

The vertical shear strength shall be determined as the strength of core concrete and steel stirrups as per ACI 318-08 Chapter 11 (ACI 2008).

**General Provisions**

\[ \phi_v V_n \geq V_u \]  
Equation 5.47

Where:

- \[ \phi_v \] = strength reduction factor for shear = 0.75 as per ACI 318-08 (ACI 2008)
- \[ V_n \] = nominal vertical shear strength, lb
- \[ V_u \] = maximum factored vertical shear strength within the span of interest, lb

**Nominal Vertical Shear Strength**

The nominal vertical shear strength shall be determined by the sum of the concrete and steel reinforcing components.

**Vertical Shear Design**

\[ V_n = V_c + V_s \]  
Equation 5.48

Where:

- \[ V_c \] = nominal shear strength provided by concrete, lb
$V_s = \text{nominal shear strength provided by shear reinforcement, lb}$

**Concrete Strength ($V_c$)**

For members subject to shear and flexure only, concrete shear strength is determined by either Equation 5.49a or Equation 5.49ab (ACI 2008).

$$V_c = 2\lambda \sqrt{f'_c} b_w d$$  \hspace{1cm} \text{Equation 5.49a}

$$V_c = \left(1.9\lambda \sqrt{f'_c} + 2500\rho_w \frac{V_{fu}d}{M_{fu}}\right)b_w d \leq 3.5\lambda \sqrt{f'_c} b_w d$$  \hspace{1cm} \text{Equation 5.49ab}

Where:

$b_w = \text{concrete web width, in.}$

$d = \text{distance from extreme compression fiber to centroid of longitudinal steel reinforcement, in}$

$\lambda = \text{modification factor reflecting the reduced mechanical properties of lightweight concrete}$

$\rho_w = \text{ratio of } A_s \text{ to } b_w d$

$V_{fu} = \text{factored shear force at different sections of beam, lbs}$

$M_{fu} = \text{factored moment at different sections of beam, lb-in}$
Shear Reinforcement Strength ($V_s$)

The shear strength provided by the shear reinforcement is given by Equation 5.50 (ACI 2008).

$$V_s = \frac{V_u}{\phi_v} - V_c = \frac{A_v f_{yt} d}{s}$$  \hspace{1cm} \text{Equation 5.50}

Where:

- $A_v = \text{area of shear reinforcement within spacing } s, \text{ in}^2$
- $f_{yt} = \text{specified yield strength of shear reinforcement, psi}$
- $s = \text{center-to-center spacing of shear reinforcement, in}$

Spacing Limits and Minimum Shear Reinforcement Requirements

The requirement for shear reinforcement is denoted by Equation 5.51 (ACI 2008):

If $V_u \leq \frac{\phi V_c}{2}$, no stirrups required

If $\frac{\phi V_c}{2} < V_u \leq \phi V_c$, minimum stirrup spacing required  \hspace{1cm} \text{Equation 5.51}

If $V_u > \phi V_c$, design stirrup spacing and check minimums
The spacing for shear reinforcement is limited by the following:

If \( V_s < 2V_c \),

\[
s = \min \left( \frac{d/2}{24} \frac{A_vf_{yt}}{0.75\sqrt{f'_c b_w}} \right) \text{ in.}
\]

Equation 5.52

If \( 2V_c < V_s < 4V_c \),

\[
s = \min \left( \frac{d/4}{12} \frac{A_vf_{yt}}{0.75\sqrt{f'_c b_w}} \right) \text{ in.}
\]

If \( V_s > 4V_c \),

Existing beam dimensions are inadequate to ensure proper development of the shear capacity.
**Stage III – Ultimate**

This section applies to the composite floor system for the limit states of flexure, vertical shear, horizontal shear, and longitudinal shear. It assumes that the floor system acts as a composite member with full interaction between all components.

**Flexural Strength**

The flexural strength shall be determined using concrete stress block analysis as per ACI 318-08 Chapter 10 (ACI 2008). Under ultimate load conditions the steel u-shaped section is considered as tension flexural reinforcement.

**General Provisions**

\[
\phi_b M_n \geq M_u
\]

Equation 5.53

Where:

\( \phi_b \) = strength reduction factor for flexure, 0.65-0.90 as per ACI 318-08 (ACI 2008)

\( M_n \) = nominal flexural strength, lb-in.

\( M_u \) = maximum factored moment (negative or positive) within the span of interest, lb-in.
Nominal Flexural Strength

Flexural Design Using Stress Block Analysis

![Figure 5.7 – Stress Distribution and Force Equilibrium](image)

The nominal moment capacity of the section is calculated by Equation 5.54. Figure 5.7 is a depiction of the equivalent stress block used to determine the nominal moment capacity of the section.

\[
M_n = -C_c \left( \frac{a}{2} \right) - C_{s_i}(d'_i) + T_{s_i}(d_{s_i}) + T_{pl}(d_{pl})
\]

Equation 5.54

Where:

\[C_c\] = concrete compressive force, lb

\[C_{s_i}\] = compression steel rebar force, lb

\[T_{s_i}\] = tension steel rebar force, lb

\[T_{pl}\] = u-shaped steel plate tension force, lb
\[ a = \text{depth of compression zone stress block, in.} \]

\[ d'_i = \text{distance from extreme compression fiber to centroid of compression reinforcement, in.} \]

\[ d_{si} = \text{distance from extreme compression fiber to centroid of tension reinforcement, in.} \]

\[ d_{pl_i} = \text{distance from extreme compression fiber to centroid of u-shaped plate steel, in.} \]

The concrete compressive force within the stress block is given by Equation 5.55. If different concrete compressive strengths are within the stress block, an iterative procedure and appropriate engineering judgment should be exercised to find the correct compression zone depth.

\[ C_c = 0.85f_c\beta_1cb_{eff} \quad \text{Equation 5.55} \]

Where:

\[ f_c' = \text{concrete compressive strength, psi} \]

\[ \beta_1 = \text{factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, 0.65-0.85 given by Equation 5.56} \]

\[ c = \text{neutral axis depth, in.} \]

\[ b_{eff} = \text{effective width, in.} \]
The $\beta_1$ factor is dependent on the concrete compressive strength and is determined by the following (ACI 2008):

$$\beta_1 = \begin{cases} 0.85 & \text{for } (f_c \leq 4000 \text{ psi}) \\ 0.85 - \left( \frac{f_c - 4000}{20,000} \right) & \text{for } (4000 < f_c < 8000 \text{ psi}) \\ 0.65 & \text{for } (f_c \geq 8000 \text{ psi}) \end{cases} \quad \text{Equation 5.56}$$

The compressive and tensile forces in the steel rebar and plate are based on the assumption that the rebar and tension plate steel yields and are given by Equation 5.57, Equation 5.58, and Equation 5.59, respectively. For increased accuracy it is advised to treat rebar at different depths as separate layers and the u-shaped plate as three separate layers.

$$C_{si} = A_{sc_i} f_y \ (\varepsilon_{si} \geq \varepsilon_y) \quad \text{Equation 5.57}$$

$$T_{si} = A_{st_i} f_y \ (\varepsilon_{si} \geq \varepsilon_y) \quad \text{Equation 5.58}$$

$$T_{pt_i} = A_{pt_i} F_y \ (\varepsilon_{pt_i} \geq \varepsilon_y) \quad \text{Equation 5.59}$$

Where:

- $A_{sc_i} = \text{compression steel rebar cross-sectional area, in.}^2$
- $A_{st_i} = \text{tension steel rebar cross-sectional area, in.}^2$
- $A_{pt_i} = \text{steel plate cross-sectional area, in.}^2$
By equating the compressive and tension forces, the depth of the concrete compressive stress block, \( a \), shall be solved for as in Equation 5.60. If the depth of the concrete compressive stress block exceeds the depth of the slab, Equation 5.61 shall be used.

\[
a = \beta_1 c = \frac{T - C_s}{0.85f_c' b_{eff}} \text{ for } (a \leq t_{slab})
\]

Equation 5.60

\[
a = \beta_1 c = \frac{1}{b_w} \left( \frac{T - C_s}{0.85f_c'} - b_{eff} t_{slab} \right) + t_{slab} \text{ for } (a > t_{slab})
\]

Equation 5.61

Where:

\( T = \text{total tension force in steel rebar and U-shaped steel plate} = \Sigma T_{si} + \Sigma T_{pli}, \text{ lb} \)

\( C_s = \text{total compression force in steel rebar} = \Sigma C_{si}, \text{ lb} \)

\( b_w = \text{width of concrete web between hollow core plank faces, in.} \)

The assumptions of compression and tension steel yielding shall be checked. Using the calculated neutral axis depth and strain compatibility the strain at any depth can be
determined as in Equation 5.62. If the strain in steel components is found to be less than the yield strain, the force contribution of the steel components shall be adjusted as in Equation 5.63, Equation 5.64, and Equation 5.65 resulting in new neutral axis depth. This process shall be iterated until the neutral axis depth satisfies both force equilibrium and strain compatibility.

\[ \varepsilon_i = \varepsilon_{cu} \left( \frac{|d_i - c|}{c} \right) \]  

Equation 5.62

\[ C_{si} = A_{sc_i} E_s \varepsilon_{si} \ (\varepsilon_{si} < \varepsilon_y) \]  

Equation 5.63

\[ T_{si} = A_{st_i} E_s \varepsilon_{si} \ (\varepsilon_{si} < \varepsilon_y) \]  

Equation 5.64

\[ T_{pi} = A_{pt_i} E_s \varepsilon_{pi} \ (\varepsilon_{pi} < \varepsilon_y) \]  

Equation 5.65

Where:

\[ \varepsilon_{cu} = \text{assumed strain limit of concrete in compression} = 0.003 \]

\[ E_s = \text{elastic modulus of steel, psi} \]
**Vertical Shear Strength**

The proposed vertical shear strength is determined by the summation of the shear strength of core concrete and stirrups and one half of the web shear strength of the steel u-shaped section. The proposed provisions are presented below.

**General Provisions**

\[ \Sigma \phi_{nl} V_{ni} \geq V_u \]  
Equation 5.66

Where:

\[ \phi_{nl} = \text{strength reduction factor for shear for respective component, 0.75 for concrete and 1.00 for steel u-shaped section} \]

\[ V_{ni} = \text{nominal vertical shear strength for respective components; steel u-shaped section strength divided by two; concrete component shear strength, lb} \]

\[ V_u = \text{maximum factored vertical shear strength within the span of interest, lb} \]

The expanded vertical shear strength is given in Equation 5.67.

\[ \Sigma \phi_{nl} V_{ni} = 0.75 V_{nc} + 1.00 \frac{V_{ns}}{2} \]  
Equation 5.67

Where:

\[ V_{nc} = \text{nominal shear strength provided by concrete and stirrups, calculated during fire design phase, lb} \]

\[ V_{ns} = \text{nominal shear strength provided by steel u-shaped section, calculated during construction design phase, lb} \]
If the vertical shear strength is determined to be inadequate, stirrup spacing and/or size shall be adjusted to increase strength to satisfy the ultimate shear design requirements.

**Horizontal Shear Strength**

The horizontal shear strength shall be determined as per AISC Sect. 13.2d to provide full shear connection to the section (AISC 2005).

**General Provisions**

\[ \Sigma Q_n \geq V' \quad \text{Equation 5.68} \]

Where:

\[ \Sigma Q_n = \text{sum of nominal strengths of shear connectors between the points of maximum positive moment and the point of zero moment, lb} \]

\[ V' = \text{nominal horizontal shear demand, lb} \]

**Horizontal Shear Strength Design**

The horizontal shear forces at the interface, as shown in Figure 5.8, between the steel beam and concrete slab are assumed to be transferred by shear connectors.
Figure 5.8 – Horizontal Shear Interface

The total horizontal shear force, \( (V') \), between the point of maximum positive moment and the point of zero moment is given as the lowest value for the limit states of concrete crushing or steel tensile yielding given by Equation 5.69 (AISC 2005).

\[
V' = \min \left[ 0.85f_c'A_c + A_{st}f_y + A_{sc}f_y \right] \frac{1}{F_yA_{pl}} \quad \text{Equation 5.69}
\]

Where:

- \( A_c \) = area of concrete within compression zone between return flanges of the steel u-shaped section, \( \text{in.}^2 \)
- \( A_{pl} \) = area of u-shaped steel plate cross section, \( \text{in.}^2 \)
- \( A_{st} \) = area of tension reinforcing steel, \( \text{in.}^2 \)
- \( A_{sc} \) = area of compression reinforcing steel, \( \text{in.}^2 \)
The nominal strength of one stud shear connector embedded in solid concrete or in a composite slab is calculated by Equation 5.70 (AISC 2005).

\[ Q_n = 0.5 A_{\text{stud}} \sqrt{f'_c E_c} \leq R_g R_p A_{\text{stud}} F_u \]  

Equation 5.70

Where:

- \( A_{\text{stud}} \) = cross sectional area of stud shear connector, in.\(^2\)
- \( E_c \) = modulus of elasticity of concrete = \( w_c^{1.5} 33 \sqrt{f'_c} \), psi
- \( F_u \) = specified minimum tensile strength of a shear stud connector, psi
- \( R_g \) = coefficient to account for group effect = 1.0 for any number of studs welded in a row directly to the steel shape
- \( R_p \) = position effect factor for shear studs = 1.0 for studs welded directly to the steel shape
- \( w_c \) = weight of concrete per unit volume (90 \( \leq w_c \leq 155 \)), lb/ft\(^3\)

Shear Connector Placement and Spacing

The required number and longitudinal spacing of shear studs shall be determined by Equation 5.71 and Equation 5.72, respectively.

\[ n = \frac{V'}{Q_n} \]  

Equation 5.71

\[ s = \frac{l}{n} \]  

Equation 5.72
Where:

\[ n \quad \text{required number of shear studs between the point of maximum positive moment and the point of zero moment} \]

\[ s \quad \text{longitudinal spacing of shear studs, in.} \]

\[ l \quad \text{span between the point of maximum positive moment and the point of zero moment, in.} \]

The placement and spacing of shear connectors shall adhere to the following guidelines:

1) Shear connectors shall be distributed uniformly between the point of maximum and zero moments.

2) Shear connectors shall have at least one inch of lateral concrete cover.

3) The diameter of the shear stud shall not exceed two and one half times the thickness of the supporting member.

4) The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis and four diameters transverse to the longitudinal axis of the supporting composite member.

5) The maximum center-to-center spacing of stud connectors shall not exceed eight times the slab thickness or thirty six inches.
Longitudinal Shear Strength

The longitudinal shear strength shall be determined as per ACI 318-08 Chapter 11.6 (ACI 2008).

General Provisions

\[ \phi_v V_{ln} \geq V_{lu} \]  
Equation 5.73

Where:

\[ \phi_v \]  = strength reduction factor for shear = 0.75 as per ACI 318-08

\[ V_{ln} \]  = nominal longitudinal shear strength, lb

\[ V_{lu} \]  = maximum factored longitudinal shear strength, lb

Nominal Longitudinal Shear Strength

Longitudinal Shear Demand

The longitudinal shear demand per unit length of a single plane of the member shall be determined as per AASHTO LRFD Bridge Design Specification 2004 and ACI 318-99 and multiplied by the longitudinal length of the shear plane. (AASHTO 2007) This provision is presented in the Alternative Design Method in Appendix A of ACI Codes prior to 2002. The total demand shall have the force carried by concrete between shear planes, Equation 5.10, subtracted away and the remainder divided by two to calculate the longitudinal shear force for one plane as shown in Equation 5.74.
Figure 5.9 - Concrete Between Longitudinal Shear Planes

\[ V_{lu} = \frac{\left( \frac{V_u}{b d_e} \right) (l \times b) - 0.85 f'_c A_{cs}}{2} \]

Equation 5.74

Where:

\( V_u \) = maximum factored vertical shear in span of interest, lbs

\( d_e \) = distance between the centroid of the tension force to the centroid of the compression force in the transformed cracked section, in.

\( l \) = longitudinal length of shear plane, in.

\( b \) = width of the web at the level under consideration, in.

\( A_{cs} \) = area of equivalent stress block within longitudinal shear planes, in\(^2\)

The distance, \( d_e \) is calculated assuming cracked moment of inertia section properties.
Longitudinal Shear Strength Design

Shear transfer shall be considered across planes with existing or potential cracks, planes with dissimilar materials, or an interface between two concretes cast at different times as shown in Figure 5.10. A shear friction design method is utilized to calculate the longitudinal shear capacity of a single plane as in Equation 5.75 as per ACI 318-08 (ACI 2008).

\[ V_{ln} = \mu A_{vf} f_y \]  

Equation 5.75

Where:

\[ A_{vf} = \text{area of shear friction reinforcement, in.}^2 \]
\[ f_y = \text{specified yield strength of shear friction reinforcement, not to exceed 60,000 psi} \]
\[ \mu = \text{coefficient of friction} \]
\[ f'_c = \text{specified concrete compressive strength, psi} \]
The coefficient of friction shall be taken as:

1) Concrete placed monolithically .........................1.4λ

2) Concrete place against hardened concrete
with surface intentionally roughened.....................1.0λ

3) Concrete place against hardened concrete
not intentionally roughened ................................ 0.6λ

4) Concrete anchored to as-rolled structural
steel by headed studs or by reinforcing bars...........0.7λ

where λ = 1.0 for normal weight concrete and 0.75 for all lightweight concretes.

For normal weight concrete either placed monolithically or placed against hardened concrete
with surface intentionally roughened, $V_{ln}$ shall not exceed the value obtained from Equation
5.76a. For all other cases, $V_{ln}$ shall not exceed the value obtained from Equation 5.76ab (ACI
2008).

$$V_{ln} = \min \left\{ \frac{0.2 f'_{c}A_{c}}{(480 + 0.08f'_{c})A_{c}}, \frac{1600A_{c}}{800A_{c}} \right\}$$

Equation 5.76a

$$V_{ln} = \min \left\{ \frac{0.2 f'_{c}A_{c}}{800A_{c}} \right\}$$

Equation 5.76ab
Where:

\[ A_c = \text{area of concrete section resisting shear transfer, in.}^2 \]

Where concretes of different strengths are cast against each other, the value of \( f'_c \) used to evaluate \( V_{in} \) shall be that of the lower-strength concrete.
5.2.2.2 Serviceability Design Guidelines

Stage I – Construction

Dead Load Deflection

This section applies to the dead load deflection of the u-shaped steel section under dead loading.

General Provisions

\[ \Delta_D \leq \Delta_{Limit} \]  

Equation 5.77

Where:

\[ \Delta_D = \text{dead load deflection calculated under applied construction loading, in.} \]

\[ \Delta_{Limit} = \text{dead load deflection limit calculated using appropriate design code and/or engineering judgment, in} \]

If the dead load deflection limit is not satisfied, redesign the section with a greater moment of inertia. The engineer should consider redesigning the amount of shoring required, however it should be noted that shoring design is typically governed by the negative moment capacity of the steel u-shaped section.
**Stage II – Service**

This section applies to the composite floor system for deflections at service levels under live loading conditions. It assumes that the floor system acts as a composite member with full interaction between all components.

**Live Load Deflection**

The live load deflection under service loading conditions can be determined for various loading conditions as in AISC 13 Table 3-23 (AISC 2005).

**General Provisions**

To satisfy the deflection serviceability limit state, deflections at service load levels must not exceed limits set by building codes. The 2009 IBC limits the unfactored live load deflection to \( L/360 \) for floor systems (IBC 2009).

**Deflection Calculations**

Deflections shall be calculated using the transformed effective moment of inertia of the composite member. For better accuracy, layers of steel and concrete should be broken into several layers or sections. Under service loading conditions the steel section shall be considered in the calculation of transformed area or moments of inertia.

**Transformed Area**

The area of steel shall be transformed into an equivalent area of concrete using the modular ratio, \( n \), as shown in Equation 5.78.
\[ n = \frac{E_s}{E_c} \]  \hspace{1cm} \text{Equation 5.78}

The transformed area is given by Equation 5.79.

\[ A_t = A_c + nA_s \]  \hspace{1cm} \text{Equation 5.79}

Where:

\[ A_t = \text{transformed area, in.}^2 \]
\[ A_c = \text{concrete area, in.}^2 \]
\[ A_s = \text{steel area, in.}^2 \]
\[ A_g = \text{gross area, in.}^2 \]

\textit{Moments of Inertia}

The effective transformed moment of inertia, \( I_e \), can be determined using in Equation 5.80 (ACI 2008).

\[ I_e = \left( \frac{M_{cr}}{M_a} \right)^3 I_g + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \leq I_g \]  \hspace{1cm} \text{Equation 5.80}

Where:

\[ M_{cr} = \text{cracking moment, lb-in.} \]
\[ M_a = \text{maximum moment in member, due to service load, lb-in.} \]
\[ I_g \quad \text{moment of inertia of gross transformed section, in.}^4 \]

\[ I_{cr} \quad \text{moment of inertia of cracked transformed section, in.}^4 \]

In order to determine the effective moment of inertia, the cracking moment, the gross moment of inertia, and the cracked moment of inertia must also be determined (ACI 2008). The cracking moment is found using Equation 5.81.

\[ M_{cr} = \frac{I_g f_r}{y_t} \quad \text{Equation 5.81} \]

Where:

\[ f_r \quad \text{modulus of rupture of normal weight concrete, } 7.5\sqrt{f_c}, \text{ psi (ACI 2008)} \]

\[ y_t \quad \text{distance from centroidal axis of gross section to extreme fiber in tension, in.} \]

The gross moment of inertia of the transformed section is determined by Equation 5.82.

\[ I_g = \Sigma (I_{oi} + A_i d_i^2) \quad \text{Equation 5.82} \]

Where:

\[ I_{oi} \quad \text{moment of inertia about each sections own axis, in.}^4 \]

\[ A_i d_i^2 \quad \text{moment of inertia of each section due to the parallel axis theorem, where } d \text{ is measured from the centroid of the member to the centroid of each individual section, in.}^4 \]

The centroid from the top of the member is given by Equation 5.83.
\[ \bar{y} = \frac{\sum (A_i y_i)}{\sum A_i} \]  
Equation 5.83

Where:

\[ y_i = \text{distance from the top of the member to the centroid of each individual section} \]

\[ A_i = \text{area of each individual section, in.}^2 \]

**Figure 5.11 – Gross Transformed Area**

The cracked moment of inertia is determined similarly to the gross moment of inertia, calculated using Equation 5.82, but negates concrete in tension. The neutral axis of the cracked section is determined by equating the compression and tension forces. From strain compatibility and transformed areas, the neutral axis and depth of uncracked concrete can be determined by equating moments produced by compression and tension areas. Since the member under consideration does not have a constant width, an iterative process must be performed to determine the neutral axis depth. The initial neutral axis depth is assumed and the calculated depth must satisfy this assumption in order to be correct. The generic form of the solution is given in Equation 5.84, where \( c \) is the unknown. Figure 5.11 and Figure 5.12
depict transformed areas of the composite member under service loading. The gross transformed section under service loading is depicted in Figure 5.11 while the cracked transformed section is depicted in Figure 5.12.

\[ A_{c_t}(|c - d_t|) + (n - 1)A'_{s_t}(|c - d_t|) = nA_{s_t}(|c - d_t|) \]  

Equation 5.84

Where:

- \( A_{c_t} \) = area of each concrete section above the neutral axis, in.\(^2\)
- \( A'_{s_t} \) = area of compression steel above neutral axis, in.\(^2\)
- \( A_{s_t} \) = area of tension steel below neutral axis, in.\(^2\)
- \( c \) = depth to neutral axis of transformed cracked section, in.
- \( d_t \) = distance from the top of the member to the centroid of each individual section, in.

\[ A_{c_t} = \text{area of each concrete section above the neutral axis, in.}^2 \]
\[ A'_{s_t} = \text{area of compression steel above neutral axis, in.}^2 \]
\[ A_{s_t} = \text{area of tension steel below neutral axis, in.}^2 \]
\[ c = \text{depth to neutral axis of transformed cracked section, in.} \]
\[ d_t = \text{distance from the top of the member to the centroid of each individual section, in.} \]

Figure 5.12 – Cracked Transformed Area
The effective transformed moment of inertia can then be used to calculate the deflection at any stage of loading. To simplify the design the following are considered accurate for practical purposes.

For $M_a < M_{cr}$ \( \rightarrow I_e = I_g \) \hspace{1cm} \text{Equation 5.85}

For $M_a > 3M_{cr}$ \( \rightarrow I_e \approx I_{cr} \) \hspace{1cm} \text{Equation 5.86}
5.3 Special Design Considerations

The proposed design procedure covers strength and serviceability aspects of the Diversakore® shallow floor framing system. Additional elements of the system such as steel straps between u-shaped section return flanges, shear key reinforcement between hollowcores, and hollowcore concrete infill should be considered and/or incorporated into the design of the framing system. Following are discussions of the aforementioned elements and their contribution to the design of the system.

5.3.1 Steel Straps

It is recommended to weld mild steel straps between the return flanges of the steel u-shaped section at intervals along the span of the beam. The steel straps should be the same thickness as the design thickness of the u-shaped section and measure 4” in width while spanning the distance between return flanges. Steel straps are recommended to be welded at both ends of the steel section and on 4’ intervals along the beam span. The purpose for the inclusion of steel straps within the section is to inhibit the tendency of the steel section to open outward or to collapse inward under construction loading. Although there are no design provisions for including steel straps it was deemed necessary to include as a special design consideration.
5.3.2 Shear Key Reinforcement

During construction of the Diversakore® shallow floor framing system it is recommended to grout the shear keyways between hollowcore planks to increase the shear friction bond between planks. Grouting of the shear keyways provides an advantage of reducing concrete loss during the pouring sequence as concrete is unable to flow between hollowcore units. In addition to grouting shear keyways, mild reinforcing steel may be placed at the keyway ends and extend into the core concrete area to reinforce and tie the system together in a more complete manner.

When shear key reinforcement extending into core concrete falls within the compression zone it may be appropriate to include the steel as longitudinal shear reinforcement. The designer will need to consider that change of coefficient of friction because the steel passes through interfaces between concrete against concrete not intentionally roughened. The coefficient of friction is taken as $0.6\lambda$. The strength of the reinforcing steel located in the shear keyways is added to the strength of welded wire fabric within the concrete slab and the summation of strengths is multiplied by $\phi_v = 0.75$ as per ACI 318-08 to acquire the longitudinal shear design strength (ACI 2008).

5.3.3 Hollowcore Concrete Infill

A conservative approach was taken within the design procedure with regard to concrete flowing into hollowcore cores. The assumption of the design procedure is that no
concrete flows into the cores as during construction it is not possible to determine the amount of concrete flow into cores. If the designer chooses, he/she may specify a distance to cut hollowcores back to allow concrete to flow into the cores. Allowing hollowcore core concrete inflow will increase the longitudinal shear strength but to an unknown degree. The distance of concrete infill may be specified to be great enough to fully develop transverse longitudinal shear steel reinforcing bars. It is at the engineer’s discretion to specify concrete infill distances as concrete infill or transverse steel located within hollowcore cores is not the primary longitudinal shear reinforcement. Typical designs do not include concrete infill or transverse reinforcement through the cores; however designs can incorporate transverse steel extending from hollowcores on one side of the beam to hollowcores on the opposite side of the beam.

5.3.4 Hollowcore Surface Roughening

Intentional roughening of the hollowcore planks is also recommended to improve the shear friction bond between hollowcores and cast-in-place concrete. Delamination of concrete from hollowcore slabs can be prevented thus keeping the system tied together in a more complete manner. Roughening of the hollowcore planks can be done mechanically during the manufacturing process.
6 OBSERVATIONS, CONCLUSIONS, RECOMMENDATIONS

6.1 Summary of Research

As the need for increased efficiency of structural members has increased new slim floor framing systems have been developed. The Diversakore Versa :T:® system is one example. The framing system was developed to meet the needs of an industry pushing for increased strength, faster construction, and lighter framing members. North Carolina State University has taken on a three phase research program to evaluate the behavior and performance of the Diversakore® slim floor framing system. The purpose of this phase of the research was to conclude the research program by completing the evaluation of the behavior of the Diversakore® system. An experimental and analytical investigation was performed to evaluate strength and serviceability design limit states. The experimental investigation consisted of testing two full scale specimens from which the flexural strength and vertical shear strength were evaluated. Limit states of horizontal and longitudinal shear were also evaluated as part of the investigation of flexural and vertical shear strength. The analytical investigation consisted of a layered section analysis of the sections along with a detailed limit state analysis for each section. Comparisons were made between experimental data and analytical models from which analysis and design procedures for future design or reference for further research have been developed. General observations, conclusions, and recommendations for future research are presented in the subsequent sections of this chapter.
6.2 Observations

The testing of the two specimens yielded significant observations with regard to the limit states analyzed during the research project. Observations from the testing of the specimens are as follows:

1. It was observed that both specimens failed prematurely in a flexural manner. Neither the DK 3 or DK 4 specimen reached their predicted ultimate flexural design strength but failed at a reduced flexural strength due to the loss of effective flange width. Steel yielding and concrete crushing.

2. The DK 4 specimen was tested to induce a vertical shear failure. However, it was observed that the vertical shear strength of the composite specimen was much greater than predicted and as a result was greater than the flexural strength even when loaded in a manner designed to induce shear failure.

3. Longitudinal shear failure was observed to be the primary failure mode during the DK 3 and DK 4 tests. Cracking occurred along both shear planes extending the entire span of both specimens and from the surface down to the return flanges of the steel section at the interface between the ends of the hollowcore slabs and cast-in-place concrete.
4. The layered sectional analysis was observed to accurately predict the load deflection of the DK 3 specimen up to the point of observed longitudinal shear failure. Deviation of experimental load deflection from predicted load deflection occurred as the effective flange width was lost and the section strength was reduced. The DK 4 experimental load deflection stiffness and strength was not accurately modeled by layered section analysis. The predicted stiffness and strength of the DK 4 specimen was greater than the experimental values.

5. Horizontal shear failure was not observed during the testing of the DK 3 or DK 4 specimen. Slip plots indicated that no significant slip occurred between core concrete and the steel u-shaped section. However, significant slip was measured between the hollowcores and the steel u-shaped section indicating longitudinal shear failure.

6.3 Conclusions

The following conclusions were drawn from the observations and analysis carried out during the research project. The conclusions are as follows:

1. Flexural behavior can be accurately predicted but special consideration must be given to individual strength design limit states to ensure the sections reach their ultimate moment capacities.
2. The longitudinal shear strength, based upon ultimate moment capacity, can be accurately predicted using the provisions presented in the proposed design procedure.

3. The vertical shear strength calculated using the AISC or ACI approach is conservative when compared to results from the DK 4 shear test. Design provisions for the limit state of vertical shear have been developed allowing the design vertical shear strength of the composite section to be taken as the summation of the concrete with stirrups and half of the steel section design vertical shear strength.

4. Allowing core concrete to flow into cores of the hollowcores does not contribute significantly to an increase in longitudinal shear strength as the longitudinal shear failure was observed in both the DK 3 and DK 4 tests.

5. The proposed design procedure will produce a conservative yet efficient slim floor framing design. The proposed design provisions address the limit states of flexure, vertical shear, horizontal shear, and longitudinal shear. Dead load construction deflection and live load service deflection checks are also a component of the procedure.
6. The proposed vertical shear strength of the composite section will produce a conservative shear design as the vertical shear strength of the sections greatly exceeded the initial predicted vertical shear strength.

6.4 Recommendations for Future Work

After assessment of the observations and conclusions of this research project several recommendations have been made to proceed with future research. Future research may include, but should not be limited to the following:

1. Verification and/or improvement of proposed design procedure through additional full scale testing of shallow floor framing systems with further investigation of the longitudinal shear limit state.

2. Further examination of the component contribution of the steel plate section and reinforced concrete section on the vertical shear strength of the framing system.

3. Investigation the affects of negative bending on the framing system by analyzing continuous beam configurations.

4. Studying the behavior of untopped slim floor framing systems.
5. A long term study to examine the effects of creep and the additional benefits that the steel plate section may add to the structure.

6. Research the behavior of the slim floor system utilizing ribbed metal decking as a substitute for hollowcore slabs.
7 REFERENCES


APPENDIX A: DESIGN EXAMPLE

A.1 Design Criteria

A full design example is presented for the Diversakore® shallow floor framing system under an estimated live load with approximate bay size and tributary area of loading. Typical bay sizes for which Diversakore® shallow floor framing systems are designed are approximately 30’ x 30’ with Diversakore Versa :T:® girders spanning 30’ center to center of column supports. The columns were assumed to measure 2’ x 2’ from which a clear span of 28’ is obtained for the Versa :T:® girders. Hollowcores rest on the return flanges of the steel u-shaped section and span to the next parallel Versa :T:® girder. Six hollowcore planks measuring 4’ wide and 3’ long and one hollowcore plank measuring 2’ wide and 3’ long are aligned perpendicular to the steel sections upon which a cast-in-place concrete deck is placed. The overall bay, girder, and precast hollowcore planks layout is shown in Figure A.1. The example design member is subjected to a superimposed dead load of 10 psf and a live loading of 100 psf which is reducible according to ASCE Live Load Reduction provisions. The steel u-shaped section is also subjected to the full dead load of the system along with a construction live load of 20 psf. Design calculations along with final design drawings are presented at the end of the design example to illustrate the comprehensive design procedure. For this example a 2” cast-in-place concrete topping and 8” Oxford Gate Core Untopped Hollowcore slabs are used in the design.
Hollowcores placed perpendicular to steel sections in both directions beginning at centerline of building.

30' x 30' tributary area

Figure A.1 - Design Example Bay, Girder, and Hollowcore Layout
Material properties for the design example are as follows:

\[ F_y = 50,000 \text{ psi} \]
\[ f_y = 60,000 \text{ psi} \]
\[ F_u = 65,000 \text{ psi} \]
\[ E_s = 29,000,000 \text{ psi} \]
\[ w_c = 150 \text{ pcf} \]
\[ f_c' = 4,000 \text{ psi} \]
\[ f_h' = 6,000 \text{ psi} \]
\[ E_c = 3,834,253.5 \text{ psi} \]
\[ E_h = 4,098,386.7 \text{ psi} \]
\[ \varepsilon_{y,s} = 0.002069 \]
\[ \varepsilon_{y,pl} = 0.001724 \]
A.2 Flexural Strength – Steel U-Shaped Section

Trial section dimensions are selected to begin the design procedure. The dimensions are selected such that return flanges and webs of the steel u-shaped section are compact. The initial thickness of the steel u-shaped section is selected to be 0.375”. The return flange width is 3 ½” and the distance between top and bottom flanges is selected to be 6”. The total bottom flange width is 22”. The bending radius at each corner of the steel section is calculated as 0.5625”. Figure A.2 depicts the steel section and corresponding dimensions of elements.

**Figure A.2 - Steel Section Dimensions**
A.2.1 Flexural Strength

A.2.1.1 Positive Moment Design Capacity

\[ \lambda_{pf} = 0.38\sqrt{29,000,000 \text{ psi}/50,000 \text{ psi}} = 9.15 \quad \text{AISC Table B4.1 Case 2} \]

\[ \lambda_f = \frac{3.25 \text{ in}}{0.375 \text{ in}} = 8.67 \leq 9.15 \rightarrow \text{Compact} \]

\[ \lambda_{wp} = \frac{7.305 \text{ in}}{12.06 \text{ in}} \sqrt{\frac{29,000,000 \text{ psi}}{50,000 \text{ psi}}} = 33.53 \quad \text{AISC Table B4.1 Case 11} \]

\[ \lambda_{wr} = 5.70\sqrt{29,000,000 \text{ psi}/50,000 \text{ psi}} = 137.27 \quad \text{AISC Table B4.1 Case 11} \]

\[ \lambda_w = \frac{7.315 \text{ in}}{0.375 \text{ in}} = 19.51 \leq 32.44 \rightarrow \text{Compact} \]

\[ A_{pl} = 2(3.25 \text{ in})(0.375 \text{ in}) + 2(0.375 \text{ in})(6 \text{ in}) + (0.375 \text{ in})(22 \text{ in}) = 15.1875 \text{ in}^2 \]

\[ ENA_{bot} = \frac{2(3.25 \text{ in})(0.375 \text{ in})(6.5625 \text{ in}) + 2(0.375 \text{ in})(6 \text{ in})(3.375 \text{ in}) + (0.375 \text{ in})(22 \text{ in})(0.1875 \text{ in})}{2(3.25 \text{ in})(0.375 \text{ in}) + 2(0.375 \text{ in})(6 \text{ in}) + (0.375 \text{ in})(22 \text{ in})} = 2.16 \text{ in} \]

\[ ENA_{top} = 6.75 \text{ in} - 2.16 \text{ in} = 4.59 \text{ in} \]

\[ r = (0.375 \text{ in})(1.5) = 0.5625 \text{ in} \]

\[ h_c = 2(4.59 \text{ in} - 0.375 \text{ in} - 0.5625 \text{ in}) = 7.305 \text{ in} \]
\( PNA_{bot} = 0.35 \text{ in} \)

\( PNA_{top} = 6.75 \text{ in} - 0.35 \text{ in} = 6.4 \text{ in} \)

\( h_p = 2(6.4 \text{ in} - 0.375 \text{ in}) = 12.05 \text{ in} \)

<table>
<thead>
<tr>
<th>Moment of Inertia</th>
<th>b</th>
<th>h</th>
<th>(y_{bot})</th>
<th>(I_{elemnt})</th>
<th>(Ay^2)</th>
<th>(I_{total})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Return Flanges</td>
<td>2(3.25)</td>
<td>0.375</td>
<td>6.5625</td>
<td>0.0286</td>
<td>47.349</td>
<td>47.3776</td>
</tr>
<tr>
<td>Webs</td>
<td>2(0.375)</td>
<td>6</td>
<td>3.375</td>
<td>13.5</td>
<td>6.698</td>
<td>20.198</td>
</tr>
<tr>
<td>Bottom Flange</td>
<td>22</td>
<td>0.375</td>
<td>0.1875</td>
<td>0.0967</td>
<td>31.939</td>
<td>32.0357</td>
</tr>
</tbody>
</table>

\( I = 99.6113 \text{ in}^4 \)

\( S = I/ENA_{top} = 99.6113 \text{ in}^4 / 4.59 \text{ in} = 21.7018 \text{ in}^3 \)

\( Z = A_1y_t + A_2y_c = (7.59375 \text{ in}^2)(0.173 \text{ in}) + (7.49375 \text{ in}^2)(4.14 \text{ in}) = 32.75 \text{ in}^3 \)

\( M_y = SF_y = (21.7018 \text{ in}^3)(50,000 \text{ psi}) = 1,085,090 \text{ lb} - \text{in} \)

\( M_p = ZF_y = (32.75 \text{ in}^3)(50,000 \text{ psi}) = 1,637,500 \text{ lb} - \text{in} \)

\( M_n = M_p \leq 1.6M_y = 1,637,500 \text{ lb} - \text{in} \)

\( \phi_b M_n^+ = \frac{(0.9)(1,637,500 \text{ lb} - \text{in})}{12} = 122,813 \text{ lb} - \text{ft} \)
\( \lambda_{pf_2} = 1.12 \sqrt{29,000,000 \text{ psi}/50,000 \text{ psi}} = 26.97 \quad \text{AISC Table B4.1 Case 12} \)

\( \lambda_{rf_2} = 1.40 \sqrt{29,000,000 \text{ psi}/50,000 \text{ psi}} = 33.72 \quad \text{AISC Table B4.1 Case 12} \)

\( \lambda_f = \frac{22 \text{ in}}{0.375 \text{ in}} = 58.67 > 33.72 \rightarrow \text{Slender} \)

\( \lambda_{wp} = \frac{7.305 \text{ in}}{12.06 \text{ in} \sqrt{ \frac{29,000,000 \text{ psi}}{50,000 \text{ psi}}} \left( 0.54 \frac{1,505,467 \text{ lb} - \text{in}}{1,083,920 \text{ lb} - \text{in}} - 0.09 \right)^2} = 33.53 \quad \text{AISC Table B4.1 Case 11} \)

\( \lambda_{wr} = 5.70 \sqrt{29,000,000 \text{ psi}/50,000 \text{ psi}} = 137.27 \quad \text{AISC Table B4.1 Case 11} \)

\( \lambda_w = \frac{7.305 \text{ in}}{0.375 \text{ in}} = 19.48 \leq 32.44 \rightarrow \text{Compact} \)

\( A_{pl} = 15.1875 \text{ in}^2 \)

\( ENA_{bot} = 4.59 \text{ in} \)

\( ENA_{top} = 6.75 \text{ in} - 4.59 \text{ in} = 2.16 \text{ in} \)

\( r = 0.5625 \text{ in} \)

\( h_c = 2(2.16 \text{ in} - 0.375 \text{ in} - 0.5625 \text{ in}) = 2.445 \text{ in} \)

\( PNA_{bot} = 6.4 \text{ in} \)
\[ PNA_{top} = 6.75 \text{ in} - 6.4 \text{ in} = 0.35 \text{ in} \]

\[ h_p = 2(0.35 \text{ in} - 0.375 \text{ in}) = 0 \text{ in} \]

\[ I = 99.6113 \text{ in}^4 \]

\[ S = 21.7018 \text{ in}^3 \]

\[ F_{cr} = \frac{0.9(29,000,000 \text{ psi})(0.76)}{\left(\frac{22 \text{ in}}{0.375 \text{ in}}\right)^2} = 5,763.3 \text{ psi} \]

\[ k_c = \frac{4}{\sqrt{4.875 \text{ in}/0.375 \text{ in}}} = 1.24434 \quad \therefore \quad k_c = 0.76 \]

\[ M_n = F_{cr}S = (5,763.3 \text{ psi})(21.7018 \text{ in}^3) = 125,074 \text{ lb - in} \]

\[ \phi_b M_n^\gamma = -\frac{(0.9)(125,074 \text{ lb - in})}{12} = -9,380.55 \text{ lb - ft} \]
A.2.2  **Flexural Demand – Design of Shoring**

A.2.2.1  **Ultimate Applied Moments**

The distributed dead and live load is calculated. The distributed dead load consists of the hollowcore, cast-in-place concrete, and steel section weight along with a superimposed live load of 20 psf.

\[
\begin{align*}
\omega_{D,h} &= (30 \text{ ft})(56 \text{ psf}) = 1,680 \text{ plf} \\
\omega_{D,e} &= \left( \frac{22 \text{ in}}{12 \text{ in/ft}} \right) \left( \frac{6 \text{ in}}{12 \text{ in/ft}} + \frac{22 \text{ in} - 2(3.25 \text{ in})}{12 \text{ in/ft}} \right) \left( \frac{8 \text{ in}}{12 \text{ in/ft}} + \frac{2 \text{ in}}{12 \text{ in/ft}} \right)(30 \text{ ft})(150 \text{ psf}) \\
&= 1016.67 \text{ plf} \\
\omega_{D,s} &= \left( \frac{(22 \text{ in})(0.375 \text{ in}) + (2)(0.375 \text{ in})(6 \text{ in}) + (2)(3.25 \text{ in})(0.375 \text{ in})}{144 \text{ in}^2} \right)(490 \text{ pcf}) \\
&= 51.68 \text{ plf} \\
\omega_{L} &= (30 \text{ ft})(20 \text{ psf}) = 600 \text{ plf} \\
\omega_{u} &= 1.2(1,680 \text{ plf} + 1016.67 \text{ plf} + 51.68 \text{ plf}) + 1.6(600 \text{ plf}) = 4,258.02 \text{ plf}
\end{align*}
\]

Specify 6 shoring points. Compare strength of section to ultimate moments.

\[
\begin{align*}
M_{u}^+ &= (0.078)(4,258.02 \text{ plf})(28 \text{ ft/7})^2 = 5,314.01 \text{ lb-ft} < 122,813 \text{ lb-ft} \\
M_{u}^- &= (-0.106)(4,258.02 \text{ plf})(28 \text{ ft/7})^2 = -7,221.6 \text{ lb-ft} < -9,380.55 \text{ lb-ft}
\end{align*}
\]
The positive and negative moment demands are satisfied utilizing 6 shoring points as shown in Figure A.3. Typically negative moment design will govern over positive moment design when determining number of shoring points due to lesser negative moment strength.

Figure A.3 - Steel Section with Designed Shoring Layout
A.3 Vertical Shear Strength – Steel U-Shaped Section

The vertical shear capacity is calculated and checked against the ultimate shear.

\[ h = 6.75\text{ in} - 2(0.375\text{ in}) - 2(0.5625\text{ in}) = 4.875\text{ in} \]

\[ A_w = 2(4.875\text{ in})(0.375\text{ in}) = 3.65625\text{ in}^2 \]

Determine \( C_v \).

\[
\frac{h}{t} = \frac{4.875\text{ in}}{0.375\text{ in}} = 13 \leq 1.10 \sqrt{k_v E / F_y} = 1.10 \sqrt{5 \left( \frac{29,000,000\text{ psi}}{50,000\text{ psi}} \right)} = 59.23
\]

\[ C_v = 1 \]

\[ k_v = 5 \text{ for } \frac{h}{t} < 260 \]

\[ \phi_v V_u = 1.00(0.6)(50,000\text{ psi})(3.65625\text{ in}^2)(1) = 109,688\text{ lbs} \]

Calculate ultimate shear for six shoring points and check against design capacity.

\[
V_u = \left( \frac{86}{142} \right) (4,258.02\text{ p.f.l}) \left( \frac{28\text{ ft}}{7} \right) = 10,315.2\text{ lbs} < 109,688\text{ lbs}
\]

The vertical shear demand is satisfied.
A.4 Flexural Strength – Composite Section under Fire Conditions

A.4.1 Flexural Fire Ultimate Loading

The distributed dead and live load is calculated. The distributed dead load consists of the hollowcore, cast-in-place concrete, and steel section weight along with a superimposed dead load of 10 psf. The live load is 100 psf. Under fire loading conditions the load combination used for design of the section is 1.2D + 0.5L.

\[ w_{D,h} = (30 \text{ ft})(56 \text{ psf}) = 1,680 \text{ plf} \]

\[ w_{D,c} = \left( \frac{22 \text{ in}}{12 \text{ in/ft}} \right) \left( \frac{6 \text{ in}}{12 \text{ in/ft}} \right) + \left( \frac{22 \text{ in} - 2(3.25 \text{ in})}{12 \text{ in/ft}} \right) \left( \frac{8 \text{ in}}{12 \text{ in/ft}} \right) + \left( \frac{2 \text{ in}}{12 \text{ in/ft}} \right) (30 \text{ ft}) (150 \text{ psf}) \]

\[ = 1016.67 \text{ plf} \]

\[ w_{D,s} = \left( \frac{(22 \text{ in})(0.375 \text{ in}) + (2)(0.375 \text{ in})(6 \text{ in}) + (2)(3.25 \text{ in})(0.375 \text{ in})}{144 \text{ in}^2} \right) (490 \text{ pcf}) \]

\[ = 51.68 \text{ plf} \]

\[ w_{SD} = (30 \text{ ft})(10 \text{ psf}) = 300 \text{ plf} \]

\[ w_L = (30 \text{ ft})(100 \text{ psf}) = 3,000 \text{ plf} \]

\[ w_u = 1.2(1,680 \text{ plf} + 1016.67 \text{ plf} + 51.68 \text{ plf} + 300 \text{ plf}) + 0.5(3,000 \text{ plf}) \]

\[ = 5,158.02 \text{ plf} \]

\[ M_u = \frac{(5,158.02 \text{ plf})(28 \text{ ft})^2}{8} = 505,486 \text{ lb} - \text{ft} \]
A.4.2 Flexural Fire Capacity

The amount of flexural reinforcement is estimated, from which the section capacity is calculated. If the capacity is less than the demand an increase in flexural reinforcement is needed. It should be noted that the steel section is not considered under fire conditions.

\[
A_s = \frac{(505,486 \text{ lb} - \text{ft})(12 \text{ in}/1 \text{ ft})}{0.9(60,000 \text{ psi})(13.3 \text{ in})} = 8.45 \text{ in}^2
\]

\[
jd = 0.95d = 0.95(14 \text{ in}) = 13.3 \text{ in}
\]

Select amount of reinforcement steel. Select seven No. 10 bars with \( A_s = 8.89 \text{ in}^2 \). Recalculate flexural design capacity of section with consideration of effective flange width and differing concrete strengths of cast-in-place concrete and hollowcore slabs. The effective flange width overhang of the section is governed by eight times the thickness of the slab and the 1” hollowcore flute. The thickness is taken as 3” which is the summation of 2” of concrete slab and the 1” hollowcore flute.

\[
b_{eff} = 2(8)(2 \text{ in} + 1 \text{ in}) + 22 \text{ in} = 70 \text{ in}
\]

\[
\beta_1 = 0.85 - \left( \frac{4,000 \text{ psi} - 4,000}{20,000} \right) = 0.85
\]

The total tension force within the rebar is calculated to be equated to the total compressive force. Steel is assumed to have yielded.
\[ T = (8.89 \text{ in}^2)(60,000 \text{ psi}) = 533,400 \text{ lbs} \]

The concrete compressive force is calculated by summing the force within cast-in-place concrete and the force within hollowcore flute. The hollowcore concrete is converted to cast-in-place concrete by using the modular ratio. The depth of compression zone, \( c \), is iteratively solved for by equating the total compression force to the total tension force to solve for the equivalent stress block depth, \( a \). Initially it is assumed that \( a \) is within the 2” cast-in-place portion of the slab. The equivalent stress block depth is solved for by equating \( T \) and \( C_c \).

\[ C_c = 0.85(4,000 \text{ psi})(70 \text{ in})a \]

\[ a = \frac{533,400 \text{ lbs}}{0.85(4,000 \text{ psi})(70 \text{ in})} = 2.24 \text{ in} > 2 \text{ in} \rightarrow \text{Recalculate} \ C_c \text{ and} \ a \]

\( C_c \) is recalculated assuming that the full 2” cast in place concrete slab is within the equivalent stress block depth and a portion of the 1” hollowcore flute is within this depth.
\[ C_c = 0.85(4,000 \text{ psi})(70 \text{ in})(2 \text{ in}) + 0.85(4,000 \text{ psi})(15.5 \text{ in})(a - 2 \text{ in}) + 0.85(4,000 \text{ psi})(48 \text{ in}) \left(\frac{4,098,386.7 \text{ psi}}{3,834,253.5 \text{ psi}}\right)(a - 2 \text{ in}) \]

\[ C_c = 462,400 \text{ lbs} + 52,700a \text{ lbs} - 105,400 \text{ lbs} + 174,442a \text{ lbs} - 348,885 \text{ lbs} \]

\[ a = \frac{533,400 \text{ lbs} - 8,115 \text{ lbs}}{227,142} = 2.31 \text{ in} < 3 \text{ in} \rightarrow \text{Check yield assumption} \]

\[ c = \frac{a}{\beta_1} = \frac{2.31 \text{ in}}{0.85} = 2.72 \text{ in} \rightarrow \text{Check yield assumption} \]

\[ \varepsilon_s = 0.003 \left(\frac{|14 \text{ in} - 2.72 \text{ in}|}{2.72 \text{ in}}\right) = 0.0124 > 0.002069 \rightarrow \text{Steel has yielded} \]

The steel strain is greater than 0.002 indicating that the steel has yielded. The steel strain is also greater than 0.005 allowing \( \phi_b = 0.9 \) because of the tension controlled nature of the section. The section capacity is now calculated. The reinforcement depth, \( d \), is calculated with a cover according to ACI 318-08 for interior beams.
\[ C_c = 462,400 \text{ lbs} + 52,700(2.31 \text{ in}) \text{ lbs} - 105,400 \text{ lbs} + 174,442(2.31 \text{ in}) \text{ lbs} - 348,885 \text{ lbs} = 532,813 \text{ lbs} \]

\[ d = 16.75 \text{ in} - 0.375 \text{ in} - 1.5 \text{ in} - (1.693 \text{ in}/2) = 14.03 \text{ in} \rightarrow \text{Specify 14 in} \]

\[ M_n = (-532,813 \text{ lbs}) \left(\frac{2.31 \text{ in}}{2}\right) + (533,400 \text{ lbs})(14 \text{ in}) = 6,852,200 \text{ lb} - \text{ in} \]

\[ \phi_b M_n = 0.9(6,852,200 \text{ lb} - \text{ in})(1 \text{ ft}/12 \text{ in}) = 513,915 \text{ lb} - \text{ ft} \]

\[ \phi_b M_n = 513,915 \text{ lb} - \text{ ft} > M_u = 505,486 \text{ lb} - \text{ ft} \rightarrow \text{Adequate Strength} \]

The tension steel reinforcement layout is depicted in Figure A.4. The spacing between bars is specified as 2 ¾” and the depth to centroid of the tension reinforcement is 14”. The steel u-shaped section is included in the figure but does not contribute to the moment capacity.

![Figure A.4 - Cross Section with Tensions Reinforcement Detail](image-url)
A.5  Vertical Shear Strength – Composite Section under Fire Conditions

A.5.1  Vertical Shear Fire Ultimate Load

The ultimate vertical shear loading under fire conditions is determined for the section. The distributed loading was found in the flexural fire section.

\[
w_u = 1.2(1,680\, \text{plf} + 1016.67\, \text{plf} + 51.68\, \text{plf} + 300\, \text{plf}) + 0.5(3,000\, \text{plf}) \\
= 5,158.02\, \text{plf}
\]

\[
V_u = \frac{(5,158.02\, \text{plf})(28\, \text{ft})}{2} = 72,212.3\, \text{lbs}
\]

A.5.2  Vertical Shear Capacity

The vertical shear capacity is found as the summation of the concrete strength and the stirrups within. The stirrups are designed using No. 4 double legged hat shaped stirrups. Concrete is assumed to be normal weight. The spacing is determined after finding the required force for the stirrups to carry.

\[
V_c = 2(1.0)(\sqrt{4,000\, \text{psi}})(15.5\, \text{in})(14\, \text{in}) = 27,448.6\, \text{lbs}
\]

\[
V_s > \frac{V_u}{\phi_v} - V_c = \frac{72,212.3\, \text{lbs}}{0.75} - 27,448.6\, \text{lbs} = 68,834.5\, \text{lbs}
\]

\[
s < \frac{A_v f_y d}{V_s} = \frac{2(0.2\, \text{in}^2)(60,000\, \text{psi})(14\, \text{in})}{68,834.5\, \text{lbs}} = 4.88\, \text{in}
\]
Specify stirrup spacing equal to 4 inches and recalculate vertical shear capacity.

\[
V_s = \frac{A_v f_{yt} d}{s} = \frac{2 (0.2 \text{ in}^2) (60,000 \text{ psi}) (14 \text{ in})}{4 \text{ in}} = 84,000 \text{ lbs}
\]

\[
V_n = V_c + V_s = 27,448.6 \text{ lbs} + 84,000 \text{ lbs} = 111,448.6 \text{ lbs}
\]

\[
\phi_v V_n = 0.75 (111,448.6 \text{ lbs}) = 83,586.5 \text{ lbs} > 72,212.3 \text{ lbs}
\]

→ Adequate Strength

The shear reinforcement is detailed in Figure A.5. The dimensions of the stirrups are given along with spacing of stirrups along the span.

No. Stirrups spaced 4” o.c. along span
All dimensions center to center

Figure A.5 - Cross Section with Shear Reinforcement Detail
A.6  Flexural Strength – Composite Section under Ultimate Conditions

A.6.1  Flexural Ultimate Loading

The ultimate flexural loading is calculated using the load combination 1.2D + 1.6L.

\[ w_u = 1.2(1,680 \text{ plf} + 1016.67 \text{ plf} + 51.68 \text{ plf} + 300 \text{ plf}) + 1.6(3,000 \text{ plf}) \]
\[ = 8,458.02 \text{ plf} \]

\[ M_u = \frac{(8,458.02 \text{ plf})(28 \text{ ft})^2}{8} = 828,886 \text{ lb – ft} \]

A.6.2  Flexural Ultimate Capacity

The ultimate capacity of the section must be evaluated including the steel u-shaped section. The effective flange width has already been determined. The total tension force must be recalculated to include the steel section. The total tension force is then equated to the total compression force. Iteration is done to find the correct compression zone depth while satisfying steel yield assumptions.

\[ b_{eff} = 2(8)(2 \text{ in} + 1 \text{ in}) + 22 \text{ in} = 70 \text{ in} \]

\[ \beta_1 = 0.85 - \left(\frac{4,000 \text{ psi} - 4,000}{20,000}\right) = 0.85 \]
The total tension force within the rebar and steel section is calculated to be equated to the total compressive force. All steel is assumed to have yielded.

\[ T = (8.89 \text{ in}^2)(60,000 \text{ psi}) + 2(3.25 \text{ in})(0.375 \text{ in})(50,000 \text{ psi}) \]
\[ + 2(0.375 \text{ in})(6 \text{ in})(50,000 \text{ psi}) \]
\[ + (22 \text{ in})(0.375 \text{ in})(50,000 \text{ psi}) = 1,292,780 \text{ lbs} \]

The concrete compressive force is calculated by summing the force within cast-in-place concrete and the force within hollowcore flute. The hollowcore concrete is converted to cast-in-place concrete by using the modular ratio. The equivalent stress block depth, \( a \), is iteratively solved for by equating the total compression force to the total tension force. Initially it is assumed the equivalent stress block depth is within the two inch cast-in-place portion of the slab. The equivalent stress block depth is solved for by equating \( T \) and \( C_c \).

\[ C_c = 0.85(4,000 \text{ psi})(70 \text{ in})a \]

\[ a = \frac{1,292,780 \text{ lbs}}{0.85(4,000 \text{ psi})(70 \text{ in})} = 5.43 \text{ in} > 2 \text{ in} \rightarrow \text{Recalculate } C_c \text{and } a \]

\( C_c \) is recalculated assuming that the full 2” cast in place concrete slab is within the equivalent stress block and a portion of the 1” hollowcore flute is within the equivalent stress block.
\[ C_c = 0.85(4,000 \text{ psi})(70 \text{ in})(2 \text{ in}) + 0.85(4,000 \text{ psi})(15.5 \text{ in})(a - 2 \text{ in}) \]
\[ + 0.85(4,000 \text{ psi})(48 \text{ in}) \left( \frac{4,098,386.7 \text{ psi}}{3,834,253.5 \text{ psi}} \right) (a - 2 \text{ in}) \]
\[ C_c = 476,000 \text{ lbs} + 52,700a \text{ lbs} - 105,400 \text{ lbs} + 174,442a - 348,885 \text{ lbs} \]
\[ a = \frac{1,292,780 \text{ lbs} - 21,715 \text{ lbs}}{227,142} = 5.59 \text{ in} > 3 \text{ in} \rightarrow \text{Recalculate } C_c \text{ and } a \]

\[ C_c \text{ is recalculated assuming that the full 2" cast in place concrete slab is within the equivalent stress block depth along with the 1" hollowcore flute and 1" deep concrete portion between hollowcore flutes. The equivalent stress block is now assumed to extend down below the hollowcore flute.} \]

\[ C_c = 0.85(4,000 \text{ psi})(70 \text{ in})(2 \text{ in}) + 0.85(4,000 \text{ psi})(15.5 \text{ in})(1 \text{ in}) \]
\[ + 0.85(4,000 \text{ psi})(48 \text{ in}) \left( \frac{4,098,386.7 \text{ psi}}{3,834,253.5 \text{ psi}} \right) (1 \text{ in}) \]
\[ + 0.85(4,000 \text{ psi})(15.5 \text{ in})(a - 3 \text{ in}) \]
\[ C_c = 476,000 \text{ lbs} + 52,700a \text{ lbs} + 174,442 \text{ lbs} + 52,700a \text{ lbs} - 158,100 \text{ lbs} \]
\[ a = \frac{1,292,780 \text{ lbs} - 545,042 \text{ lbs}}{52,700} = 14.19 \text{ in} \rightarrow c = 16.69 \text{ in} \]
\[ \rightarrow \text{Add compression steel and recalculate } C_c \text{ and } a \]

Specify four No. 11 bars as compression reinforcement at 2 ½” from the top of the section. \( A_s \) is equal to 6.24 in\(^2\). \( C_s \) is calculated as the force within the bar minus the area of concrete it takes up as to not double count that concrete force.
\[ C_s = (6.24 \text{ in}^2)(60,000 \text{ psi} - 0.85(4,000 \text{ psi})) = 353,184 \text{ lbs} \]

\[ C_c = 0.85(4,000 \text{ psi})(70 \text{ in})(2 \text{ in}) + 0.85(4,000 \text{ psi})(15.5 \text{ in})(1 \text{ in}) \]
\[ + 0.85(4,000 \text{ psi})(48 \text{ in}) \left( \frac{4,098,386.7 \text{ psi}}{3,834,253.5 \text{ psi}} \right) (1 \text{ in}) \]
\[ + 0.85(4,000 \text{ psi})(15.5 \text{ in})(a - 3 \text{ in}) \]

\[ C_c = 476,000 \text{ lbs} + 52,700 \text{ lbs} + 174,442 \text{ lbs} + 52,700a \text{ lbs} - 158,100 \text{ lbs} \]

\[ a = \frac{1,292,780 \text{ lbs} - 545,042 \text{ lbs} - 353,184 \text{ lbs}}{52,700} = 7.49 \text{ in} < 9 \text{ in} \]

→ Check yield assumption

\[ c = \frac{a}{\beta_1} = \frac{7.49 \text{ in}}{0.85} = 8.81 \text{ in} \]

Once an acceptable equivalent stress block depth has been found, yielding of each of the steel layers must be checked and \( C_s \) and \( T \) adjusted accordingly. The calculation of the equivalent stress block depth, compression zone depth, and yielding of steel members should then be iterated until the correct value of the compression zone depth corresponding to steel yielding strains is calculated. The first iteration is shown below and along with the results of this iterative procedure determined utilizing the Microsoft Excel 2007 solver function.
A.6.2.1 Iteration 1

Compression Steel:

\[ \varepsilon_{cs} = 0.003 \left( \left| \frac{2.5 \text{ in} - 8.81 \text{ in}}{8.81 \text{ in}} \right| \right) = 0.002149 > 0.002069 \]

→ compression steel has yielded

Tension Steel:

\[ \varepsilon_{ts} = 0.003 \left( \left| \frac{14 \text{ in} - 8.81 \text{ in}}{8.81 \text{ in}} \right| \right) = 0.001767 < 0.002069 \]

→ tension steel has not yielded

Steel Section:

\[ \varepsilon_{T_{pl1}} = 0.003 \left( \left| \frac{10.1875 \text{ in} - 8.81 \text{ in}}{8.81 \text{ in}} \right| \right) = 0.000496 < 0.001724 \]

→ steel section return flanges have not yielded

\[ \varepsilon_{T_{pl2}} = 0.003 \left( \left| \frac{13.375 \text{ in} - 8.81 \text{ in}}{8.81 \text{ in}} \right| \right) = 0.001554 < 0.001724 \]

→ steel section webs have yielded

\[ \varepsilon_{T_{pl3}} = 0.003 \left( \left| \frac{16.5625 \text{ in} - 8.81 \text{ in}}{8.81 \text{ in}} \right| \right) = 0.00264 \geq 0.001724 \]

→ steel section bottom flange has yielded
\[ T = (8.89 \text{ in}^2)(29,000,000 \text{ psi})(0.001767) \]
\[ + 2(3.25 \text{ in})(0.375 \text{ in})(29,000,000 \text{ psi})(0.000496) \]
\[ + 2(0.375 \text{ in})(6 \text{ in})(29,000,000 \text{ psi})(0.001554) \]
\[ + (22 \text{ in})(0.375 \text{ in})(50,000 \text{ psi}) = 1,105,910 \text{ lbs} \]

\[ C_s = (6.24 \text{ in}^2)((50,000 \text{ psi}) - (0.85)(4,000 \text{ psi})) = 290,784 \text{ lbs} \]

\[ a = \frac{1,105.910 \text{ lbs} - 545,042 \text{ lbs} - 290,784 \text{ lbs}}{52,700} = 5.12 \text{ in} < 9 \text{ in} \]

\[ c = \frac{a}{\beta_1} = \frac{5.12 \text{ in}}{0.85} = 6.06 \text{ in} \]

Iteration is continued utilizing the solver function within Microsoft Office Excel 2007. The final iteration results are given with calculation of the design moment capacity of the section.

**A.6.2.2 Final Iteration Results**

\[ a = 6.68 \text{ in} < 9 \text{ in} \]

\[ c = \frac{6.68}{0.85} \text{ in} = 7.86 \text{ in} \]
Compression Steel:

$$\varepsilon_{cs} = 0.003 \left( \frac{|2.5 \text{ in} - 7.86 \text{ in}|}{7.86 \text{ in}} \right) = 0.002046 < 0.002069$$

→ compression steel has not yielded

Tension Steel:

$$\varepsilon_{ts} = 0.003 \left( \frac{|14 \text{ in} - 7.86 \text{ in}|}{7.86 \text{ in}} \right) = 0.002344 \geq 0.002069$$

→ tension steel has yielded

Steel Section:

$$\varepsilon_{r_{pl1}} = 0.003 \left( \frac{|10.1875 \text{ in} - 7.86 \text{ in}|}{7.86 \text{ in}} \right) = 0.000888 < 0.001724$$

→ steel section return flanges have not yielded

$$\varepsilon_{r_{pl2}} = 0.003 \left( \frac{|13.375 \text{ in} - 7.86 \text{ in}|}{7.86 \text{ in}} \right) = 0.002105 \geq 0.001724$$

→ steel section webs have yielded

$$\varepsilon_{r_{pl3}} = 0.003 \left( \frac{|16.5625 \text{ in} - 7.86 \text{ in}|}{7.86 \text{ in}} \right) = 0.003322 \geq 0.001724$$

→ steel section bottom flange has yielded

A.6.2.3 Design Capacity

The concrete compression force and compression steel force is calculated using the equivalent stress block depth, a, found utilizing the Excel software. The $\phi_b$ factor is
calculated using the strain within the tension reinforcing steel. The design moment capacity is finally calculated and compared to the ultimate moment demand.

\[ C_c = 476,000 \text{ lbs} + 52,700 \text{ lbs} + 174,442 \text{ lbs} + 52,700(6.68 \text{ in}) \text{ lbs} \]
\[ - 158,000 \text{ lbs} = 897,178 \text{ lbs} \]

\[ C_s = (6.24 \text{ in}^2)((29,000,000 \text{ psi})(0.002046) - (0.85)(4,000 \text{ psi})) \]
\[ = 349,028 \text{ lbs} \]

\[ \phi_b = 0.65 + (0.002344 - 0.002)(250/3) = 0.68 \]

\[ M_n = (-476,000 \text{ lbs})(1 \text{ in}) - (52,700 \text{ lbs})(2.5 \text{ in}) - (174,442 \text{ lbs})(2.5) \]
\[ - ((52,700)(6.68 \text{ in}) - 158,000 \text{ lbs})\left(\frac{6.68 \text{ in} - 3 \text{ in}}{2} + 3 \text{ in}\right) \]
\[ + (8.89 \text{ in}^2)(60,000 \text{ psi})(14 \text{ in}) \]
\[ + 2(3.25 \text{ in})(0.375 \text{ in})(29,000,000 \text{ psi})(0.000888)(10.1875 \text{ in}) \]
\[ + 2(0.375 \text{ in})(6 \text{ in})(50,000 \text{ psi})(13.375 \text{ in}) \]
\[ + (22 \text{ in})(0.375 \text{ in})(50,000 \text{ psi})(16.5625 \text{ in}) \]
\[ = 15,965,500 \text{ lb} - \text{ in} \]

\[ \phi_b M_n = 0.68(15,965,500 \text{ lb} - \text{ in})(1 \text{ ft}/12 \text{ in}) = 904,711 \text{ lb} - \text{ ft} \]

\[ \phi_b M_n = 904,711 \text{ lb} - \text{ ft} > M_u = 828,886 \text{ lb} - \text{ ft} \rightarrow \text{Adequate Strength} \]
Upon calculation of the ultimate design capacity of the section, the capacity of the section under fire loading conditions must be reanalyzed to include added compression steel. The results of this analysis are given. The analysis was completed in the same manner as the ultimate analysis utilizing the Microsoft Office Excel 2007 solver function.

\[ a = 2.23 \text{ in} < 9 \text{ in} \]

\[ c = \frac{2.23}{0.85} \text{ in} = 2.62 \text{ in} \]

Compression Steel:
\[ \varepsilon_{cs} = 0.00014 < 0.002069 \rightarrow \text{compression steel has not yielded} \]

Tension Steel:
\[ \varepsilon_{ts} = 0.013 \geq 0.002069 \rightarrow \text{tension steel has yielded} \]
\[ C_c = 528,644 \text{ lbs} \]
\[ C_s = 4,756 \text{ lbs} \]
\[ \phi_b = 0.9 \]
\[ M_n = 6,868,322 \text{ lb} - \text{in} \]
\[ \phi_b M_n = 0.9(6,868,322 \text{ lb} - \text{in})(1 \text{ ft}/12 \text{ in}) = 515,124 \text{ lb} - \text{ft} \]
\[ \phi_b M_n = 515,124 \text{ lb} - \text{ft} > M_u = 505,486 \text{ lb} - \text{ft} \rightarrow \text{Adequate Strength} \]
The section has been determined to have sufficient strength under fire loading conditions with added compression steel. The compression reinforcement is detailed in Figure A.6.

4 No. 11 bars spaced at 3” o.c.

Figure A.6 - Cross Section with Compression Reinforcement Detail
A.7 Vertical Shear Strength - Composite Section Under Ultimate Conditions

A.7.1 Vertical Shear Ultimate Demand

The ultimate vertical shear loading is determined for the section. The distributed loading was found in the flexural ultimate section.

\[ w_u = 1.2(1,680 \text{ plf} + 1016.67 \text{ plf} + 51.68 \text{ plf} + 300 \text{ plf}) + 1.6(3,000 \text{ plf}) \]
\[ = 8,458.02 \text{ plf} \]

\[ V_u = \frac{(8,458.02 \text{ plf})(28 \text{ ft})}{2} = 118,412 \text{ lbs} \]

A.7.2 Vertical Shear Capacity

The proposed vertical shear strength is determined as the summation of the shear strength of core concrete and stirrups and one half of the web shear strength of the steel u-shaped section.

\[ \Sigma \phi_{ni}V_{ni} = 0.75V_{nc} + 1.00 \frac{V_{ns}}{2} \]

\[ 0.75V_{nc} = 83,586.5 \text{ lbs} \]

\[ 1.00 \frac{V_{ns}}{2} = 54,844 \text{ lbs} \]

\[ \Sigma \phi_{ni}V_{ni} = 138,430.5 \text{ lbs} \geq 118,412 \text{ lbs} \rightarrow \text{Adequate Strength} \]
A.8  Horizontal Shear Strength - Design of Shear Studs

A.8.1  Horizontal Shear Demand

The total horizontal shear demand force, $V'$, between the point of maximum positive moment and the point of zero moment is given as the lowest value for the limit states of concrete crushing or steel tensile yielding.

$$V' = \min \left[ 0.85(4 \text{ ksi})(15.5 \text{ in})(6.68 \text{ in}) + (8.89 \text{ in}^2)(60 \text{ ksi}) + (6.24 \text{ in}^2)(60 \text{ ksi}) \right]$$

$$V' = (15.1875 \text{ in}^2)(50 \text{ ksi})(1000 \text{ lbs/kip}) = 759,375 \text{ lbs}$$

A.8.2  Strength of Shear Connector

The nominal strength of one stud shear connector embedded in solid concrete or in a composite slab is calculated from which the spacing is determined. The shear studs used within this design example are $\frac{3}{4}$ inch diameter 8 inch deep headed shear studs. $F_u$ of the shear studs was taken to be 65,000 psi.

$$Q_n = 0.5(0.44 \text{ in}^2)\sqrt{(4,000 \text{ psi})(3,834,253.5 \text{ psi})} \leq (1.0)(1.0)(0.44 \text{ in}^2)(65,000 \text{ psi})$$

$$Q_n = 27,245.4 \text{ lbs} \leq 28,600 \text{ lbs} \therefore Q_n = 27,245.4 \text{ lbs}$$
A.8.3 Shear Connector Placement and Spacing

The required number and longitudinal spacing of shear studs are determined. The placement and spacing of shear connectors adhere to the following guidelines:

1) Shear connectors shall be distributed uniformly between the point of maximum and zero moments.

2) Shear connectors shall have at least one inch of lateral concrete cover.

3) The diameter of the shear stud shall not exceed two and one half times the thickness of the supporting member.

4) The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis and four diameters transverse to the longitudinal axis of the supporting composite member.

5) The maximum center-to-center spacing of stud connectors shall not exceed eight times the slab thickness or 36 inches.

\[
n = \frac{759,375 \text{ lbs}}{27,245.4 \text{ lbs}} = 27.87 \text{ studs} \rightarrow Specify 28 \text{ studs}
\]

\[
s = \left( \frac{(28 \text{ ft})}{2} \right) \left( \frac{12 \text{ in/ft}}{28 \text{ studs}} \right) = 6 \text{ in} \rightarrow Specify double line 6 in stagger space
\]
The first stud is placed three inches on center from the center line of the span. Successive studs are placed in a double line with six inch staggered spacing with distance between lines satisfying minimum spacing requirements. A total of 50 studs are required as each half of the beam must be reinforced for horizontal shear. The shear stud detail is depicted in Figure A.7.

¾” Dia. 8” deep headed shear studs on double line 6” staggered spacing

Figure A.7 - Cross Section with Shear Stud Details
A.9 Longitudinal Shear Strength - Design of Transverse Reinforcement

A.9.1 Longitudinal Shear Demand

The longitudinal shear demand per unit length of half member is determined and multiplied by the longitudinal length of the shear planes. To determine the demand for one plane the half member demand has the force carried by concrete between shear planes subtracted and then divided by two.

\[
V_{lu} = \left( \frac{118,412 \text{ lbs}}{(15.5 \text{ in})(11.21 \text{ in})} \right) \left( (15 \text{ ft} \times 12 \text{ in/ft})(15.5 \text{ in}) - 0.85(4,000 \text{ psi})(6.68 \text{ in})(15.5 \text{ in}) \right) \div 2
= 774,658 \text{ lbs}
\]

A.9.2 Longitudinal Shear Strength

The longitudinal shear reinforcement is designed by determining a required amount of transverse shear reinforcement to span across the longitudinal shear planes. Weld wire mesh is sized according to ACI 318-08 Appendix E. The \( \phi_v \) value associated with longitudinal shear strength design is 0.75 because concrete and steel reinforcement are subject to shearing forces rather than direct tension or compression forces.

\[
\phi_v V_{ln} = \mu A_{vf} f_y
\]

\[
\mu A_{vf} f_y = \frac{774,658 \text{ lbs}}{0.75} = 1,032,880 \text{ lbs}
\]

\[
A_{vf} \geq \frac{1,032,880 \text{ lbs}}{(60,000 \text{ psi})(1.4)} = 12.3 \text{ in}^2
\]
\[
\frac{A_{vf}}{ft} \geq \frac{12.3 \text{ in}^2}{15 \text{ ft}} = \frac{0.82 \text{ in}^2}{ft} \rightarrow \text{Specify WWF W28 x 4 x 4} \rightarrow A_{vf} = \frac{0.84 \text{ in}^2}{ft}
\]

\[
A_{vf} = \left( \frac{0.84 \text{ in}^2}{ft} \right) 15 \text{ ft} = 12.6 \text{ in}^2
\]

\[
\phi_v V_{in} = 0.75(1.4)(12.6 \text{ in}^2)(60,000 \text{ psi}) = 793,800 \text{ lbs}
\]

\[
\phi_v V_{in} = 793,800 \text{ lbs} \geq 774,658 \text{ lbs} \rightarrow \text{Adequate Strength}
\]

Longitudinal shear reinforcement is placed at mid depth of the concrete slab one inch from the surface as shown in Figure A.8.

WWF W28 x 4 x 4 placed with a 1” cover

---

![Figure A.8 - Cross Section with Longitudinal Shear Reinforcement Detail](image-url)
A.10 Deflection

A.10.1 Live Load Deflection Service Loading

The live load deflection under service loading is calculated using the transformed effective moment of inertia. The live load deflection limit is given by IBC 2009 as \( L/360 \) (IBC 2009). The transformed gross and cracked moments of inertia were calculated from which the effective moment of inertia is calculated.

\[
I_{g,t} = 20,670.3 \text{ in}^4
\]

\[
I_{cr,t} = 16,576.72 \text{ in}^4
\]

\[
f_r = 7.5\sqrt{4,000 \text{ psi}} = 474.342 \text{ psi}
\]

\[
M_{cr} = \frac{(474.342 \text{ psi})(20,670.3 \text{ in}^4)}{8.14 \text{ in}} = 1,204,520 \text{ lb-in}
\]

\[
M_a = \frac{\left(\frac{(100 \text{ psf})(30 \text{ ft})}{(12 \text{ in/ft})(12 \text{ in/ft})}\right)^2}{8} = 3,528,000 \text{ lb-in}
\]

\[
l_{eff,t} = \left(\frac{1,204,520 \text{ lb-in}}{3,528,000 \text{ lb-in}}\right)^3 (20,670.3 \text{ in}^4) + \left[1 - \left(\frac{1,204,520 \text{ lb-in}}{3,528,000 \text{ lb-in}}\right)^3\right](16,576.72 \text{ in}^4)
\]

\[
= 16,739.6 \text{ in}^4
\]

\[
\Delta_L = \frac{5\left(\frac{(100 \text{ psf})(30 \text{ ft})}{(12 \text{ in/ft})(12 \text{ in/ft})}\right)^4}{384(3,834,253.5 \text{ psi})(16,739.6 \text{ in}^4)} = 0.65 \text{ in}
\]

\[
\Delta_L = 0.65 \text{ in} < L/360 = \left(\frac{(28 \text{ ft})(12)}{360}\right) = 0.93 \text{ in} \rightarrow \text{Deflection satisfied}
\]
APPENDIX B: EXPERIMENTAL INVESTIGATION TEST DATA

B.1 DK 3

Flexural Test Summary Sheet

<table>
<thead>
<tr>
<th>Test Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test ID</td>
<td>DK 3</td>
</tr>
<tr>
<td>Test Date</td>
<td>8/11/2009</td>
</tr>
<tr>
<td>Test Location</td>
<td>Constructed Facilities Laboratory (CFL), North Carolina State University</td>
</tr>
<tr>
<td>Tested By</td>
<td>Kurtis Kennedy, E.I. and Adam Amortmont, E.I.</td>
</tr>
<tr>
<td>Sponsor</td>
<td>Diversakore®</td>
</tr>
<tr>
<td>Configuration</td>
<td>Simple supports with four equal loads at the 1/5 points.</td>
</tr>
<tr>
<td>Span</td>
<td>27 ft.</td>
</tr>
<tr>
<td>Loading Rate</td>
<td>0.033 in/min</td>
</tr>
<tr>
<td>Steel U-Beam</td>
<td></td>
</tr>
<tr>
<td>Height</td>
<td>6 in. (nominal)</td>
</tr>
<tr>
<td>Thickness</td>
<td>0.25 in. (nominal)</td>
</tr>
<tr>
<td>Width</td>
<td>20 in. (nominal)</td>
</tr>
<tr>
<td>Length</td>
<td>30 ft. (nominal)</td>
</tr>
<tr>
<td>Steel Grade</td>
<td>ASTM A572</td>
</tr>
<tr>
<td>Yield Strength</td>
<td>50 ksi (nominal)</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>65 ksi (nominal)</td>
</tr>
<tr>
<td>Thickness (measured)</td>
<td>62.5 ksi</td>
</tr>
<tr>
<td>Width (measured)</td>
<td>62.5 ksi</td>
</tr>
<tr>
<td>Length (measured)</td>
<td>62.5 ksi</td>
</tr>
<tr>
<td>Reinforcement</td>
<td></td>
</tr>
<tr>
<td>Tension</td>
<td>6 - No. 9 bars</td>
</tr>
<tr>
<td>Compression</td>
<td>2 - No. 5 bars</td>
</tr>
<tr>
<td>Shear</td>
<td>s/2 from supports 23 No. 4 stirrups spaced 3 in. rest 6 in. spacing</td>
</tr>
<tr>
<td></td>
<td>Shear studs 3/4 in. dia by 8 in. long at 10 in. staggered spacing</td>
</tr>
<tr>
<td>Hollow Core Slab</td>
<td></td>
</tr>
<tr>
<td>Depth</td>
<td>8 in.</td>
</tr>
<tr>
<td>Width</td>
<td>48 in.</td>
</tr>
<tr>
<td>Length</td>
<td>36 in.</td>
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<tr>
<td>Concrete Compressive Strength</td>
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<tr>
<td>Reinforcement Grade</td>
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</tr>
<tr>
<td>Min. Cover</td>
<td>1.5&quot;</td>
</tr>
<tr>
<td>Concrete Slab</td>
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</tr>
<tr>
<td>Depth</td>
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</tr>
<tr>
<td>Width</td>
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</tr>
<tr>
<td>Length</td>
<td>30 ft.</td>
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<tr>
<td>Concrete Compressive Strength</td>
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<tr>
<td>Reinforcement Grade</td>
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</tr>
<tr>
<td>Top Cover</td>
<td>0.75-1.0 (in)</td>
</tr>
<tr>
<td>Test Results</td>
<td></td>
</tr>
<tr>
<td>Maximum Applied Moment</td>
<td>790 k-ft</td>
</tr>
<tr>
<td>Failure Description</td>
<td>Longitudinal Shear Cracking, Concrete Crushing</td>
</tr>
</tbody>
</table>

Test Observations
- Slip occurred on the half of the beam coinciding with quarter point 1
- Longitudinal shear cracks developed along the interface of the hollowcore and core beam along the entire span of the specimen
- Failure at applied moment of 790 k-ft due to concrete crushing
- No significant measure of slip between steel u-shaped section and core concrete
Figure B.1 - DK 3 Applied Moment vs. Deflection
Figure B.2 - DK 3 Applied Moment vs. Average Midpoint Strain
Figure B.3 - DK 3 Strain Profiles

I – 0.5 Service Load Step (220 k-ft)  
II – Service Load Step (444 k-ft)  
III – 1.5 Factored Load Step (663 k-ft)  
IV – Failure Load Step (790 k-ft)
Figure B.4 - DK 3 Applied Moment vs. End Slip

Note: 0.52” maximum slip recorded
Figure B.5 - DK 3 Applied Moment vs. Midspan Deflection Comparison
Figure B.6 - DK 3 Strain Profile Comparisons
a) Global view of beam at the point of concrete crushing at midpan

b) Global view longitudinal shear cracking failure

Figure B.7 - DK 3 Failure Photos
B.2 DK 4

Shear Test Summary Sheet

Test ID: DK Shear
Test Date: 9/18/2009
Test Location: Constructed Facilities Laboratory (CFL), North Carolina State University
Tested By: Kurtis Kennedy, E.I. and Adam Amortnont, E.I.
Sponsor: Diversakore®

Test Parameters
Configuration: Simply supported with 4' 8" overhang and load at 28" from support within 6" stirrup spacing length.
Span: 8.25 ft
Loading Rate: 0.01 in/min

Steel U-Beam
Height: 6 in. (nominal)  Steel Grade: ASTM A572
Thickness: 0.25 in. (nominal)  Yield Strength: 50 ksi (nominal)
Width: 20 in. (nominal)  Tensile Strength: 62.5 ksi (measured)
Length: 15 ft. (nominal)

Reinforcement
Tension: 6 - No. 9 bars
Compression: 2 - No. 5 bars
Shear: No. 4 stirrups spaced 6 in. 3/4 in. dia by 8 in. long at 8 in. staggered spacing

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 Slab
Depth: 2.0 in  Concrete Compressive Strength: 5200 psi
Width: 87 in.  Reinforcement: 6 x 6 - W2.9 x W2.9
Length: 15 ft.  Reinforcement Grade: ASTM A185
Top Cover: 0.75-1.0 (in)

Test Results
Maximum Applied Shear: 274 k
Maximum Applied Moment: 637 k-ft
Failure Description: Longitudinal Shear Cracking, Concrete Crushing, Shear and moment interaction

Test Observations
- Failure occurred in the region between the load point and near support
- Longitudinal shear cracks developed the entire length of the specimen
- Concrete crushing occurred at the load point
- Debonding of concrete slab from hollowcores
Figure B.8 - DK 4 Applied Moment vs. Load Point Deflection
Figure B.9 - DK 4 Applied Moment vs. Average Mid Shear Span Strains
Figure B.10 - DK 4 Strain Profiles
Figure B.11 - DK 4 Applied Moment vs. End Slip

Note: 0.22" maximum recorded slip
Figure B.12 - DK 4 Applied Moment vs. Load Point Deflection Comparisons
a) Global view of longitudinal shear cracking failure

b) Longitudinal shear cracking over the shear span

Figure B.13 - DK 4 Failure Photos