ABSTRACT

AMORTNONT, ADAM MICHAEL. Behavior of Innovative Precast Shallow Floor Framing System. (Under the direction of Dr. Emmett A. Sumner III.)

Modifications to conventional composite floor systems have been made to increase span lengths and decrease structural depths. The competitive market has also led to the development of systems to improve the ease and speed of construction. Diversakore® is developing an innovative composite shallow floor framing system comprised of precast prestressed steel-concrete composite girders, precast hollow core planks, and a cast-in-place concrete topping slab. The precast girders are constructed with a cambered U-shaped steel plate along the bottom of the section which serves as a stay-in-place form. A composite section is created using normal weight concrete, draped prestressing stands, conventional mild reinforcing steel, and the U-shaped steel plate. 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 cambered precast girders support the hollow core planks during erection and the cast-in-place topping slab is utilized to engage the planks and form a composite T-beam. The camber in the girder eliminates the need for shoring during erection and enhances the constructability of the system. A research program sponsored by the NSF I/UCRC on Repair of Buildings and Bridges with Composites (RB²C) is currently ongoing at the Constructed Facilities Laboratory at North Carolina State University to evaluate the ultimate strength and serviceability performance of this innovative
floor system. The experimental and analytical program includes full-scale tests of representative sub-assemblages and utilizes a layered sectional analysis to predict the behavior. The results of the analytical model and the experimental investigation are presented along with conclusions drawn from the initial phase of the research program.
Behavior of Innovative Precast Shallow Floor Framing System

by
Adam Michael Amortnont

A thesis submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering Raleigh, North Carolina 2010

APPROVED BY:

__________________________________________
Dr. James M. Nau

__________________________________________
Dr. Rudolf Seracino

__________________________________________
Dr. Emmett A. Sumner III
Chair of Advisory Committee
DEDICATION

I dedicate this thesis to my mother and father. Their everlasting love and support has undoubtedly given me the opportunity to accomplish all my goals.
**BIOGRAPHY**

Adam Amortnont was born and raised in Asheville, NC. He began his collegiate career at Appalachian State University in 2003. After two years he transferred to North Carolina State University to pursue a Bachelor’s of Science in Civil Engineering. After graduating Summa Cum Laude in 2007, he continued his education by enrolling in graduate school at North Carolina State University to obtain a Master’s of Science Degree in Civil Engineering with a concentration in structures. Upon completion of his Master’s of Science Degree, he plans to stay in Raleigh to pursue a career in structural engineering.
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# TABLE OF CONTENTS

List of Tables ........................................................................................................................... x

List of Figures ......................................................................................................................... xi

1 Introduction.............................................................................................................................. 1

   1.1 Background ................................................................................................................ 1

   1.2 Objectives ................................................................................................................... 5

   1.3 Scope .......................................................................................................................... 5

   1.4 Organization of Thesis ............................................................................................. 6

2 Literature Review .................................................................................................................. 8

   2.1 Introduction ................................................................................................................ 8

   2.2 Innovative Composite Systems ................................................................................ 8

   2.3 Background ................................................................................................................ 18

      2.3.1 Moment-Curvature Analysis ........................................................................ 18

      2.3.2 Composite Behavior ...................................................................................... 21

      2.3.3 Slab Specific Behavior ...................................................................................... 23

   2.4 Need for Research ...................................................................................................... 30

3 Experimental Investigation ................................................................................................... 31

   3.1 Introduction .............................................................................................................. 31

   3.2 Test Specimens ......................................................................................................... 31

      3.2.1 Specimen Components ................................................................................... 32
3.3 Material Properties .................................................................................................36
  3.3.1 Concrete Compressive Strength ..................................................................36
  3.3.2 Grout Compressive Strength ......................................................................38
  3.3.3 Steel Plate Tensile Strength .......................................................................38
3.4 Construction of Test Specimens .......................................................................40
  3.4.1 Initial Construction ......................................................................................40
  3.4.2 Precast Girder Construction ......................................................................42
  3.4.3 Floor System Construction .........................................................................49
3.5 Test Setup ..............................................................................................................58
3.6 Instrumentation .....................................................................................................61
  3.6.1 Strain Gauges ..............................................................................................61
  3.6.2 PI Gauges ....................................................................................................63
  3.6.3 String Potentiometers ................................................................................64
  3.6.4 Precast Girder Construction ......................................................................65
  3.6.5 Data Acquisition System ............................................................................66
3.7 Loading Protocol ..................................................................................................66
4 Experimental Results ..............................................................................................68
  4.1 Introduction ........................................................................................................68
  4.2 Behavior of Test Specimens .............................................................................68
    4.2.1 Moment-Deflection ..................................................................................68
    4.2.2 Deflection Profiles of Test Specimens .....................................................72
    4.2.3 Steel and Concrete Strain .........................................................................76
LIST OF TABLES

Table 3.1 – Experimental Investigation Test Matrix .............................................................. 31
Table 3.2 – Ultimate Compressive Strength of Concrete ....................................................... 37
Table 3.3 – Material Properties of Steel Plate ........................................................................ 39
Table 3.4 – DKP 1 and DKP 2 Load Steps ............................................................................. 67
Table 3.5 – DKP 1a and DKP 2a Load Steps ......................................................................... 67
Table 4.1 – Summary of Observed Failure Behavior ............................................................. 93
Table 5.1 – Construction Load Deflection Comparison ....................................................... 101
Table 5.2 – Experimental Live Load Moment Deflection Comparison ............................... 114
Table 5.3 – Flexural Capacity ............................................................................................... 119
Table 5.4 – Vertical Shear Capacity ..................................................................................... 121
Table 5.5 – Horizontal Shear between Maximum and Zero Moments ................................. 124
Table 5.6 – Longitudinal Shear between Maximum and Zero Moments ............................. 126
Table 5.7 – Design Limit State Moment Capacity Comparisons ......................................... 129
LIST OF FIGURES

Figure 1.1 – Rendering of Diversakore® Versa:T™ Shallow Floor Framing System....................................................................................................................................... 3

Figure 1.2 – Rendering of Diversakore® Precast Shallow Floor Framing System....................................................................................................................................... 4

Figure 2.1 – U-shaped Steel Plate Configurations............................................................................................................................. 10

Figure 2.2 – Reinforced Concrete Beam with Side Profile Sheets (Oehlers, 1993)......................................................................................................................................... 11

Figure 2.3 – Cross-Section of Beam-Slab Floor (Leskela et al., 1997)................................................................................................... 12

Figure 2.4 – Checkered Surface of Channel (Leskela et al., 1997).................................................................................................... 12

Figure 2.5 – Section of Steel Encased Concrete Composite Beam (Hui et. al, 2005)....................................................................................................................................... 15

Figure 2.6 – Steel-Plate Concrete Composite Beam (Nie and Zhao, 2009).......................................................................................... 16

Figure 2.7 – General Arrangement for Horizontal Push Test (Lam, 2007)..................................................................................... 29

Figure 3.1 – U-shaped Steel Plate......................................................................................................................................................... 32

Figure 3.2 – Delivery of Hollow Core Planks ................................................................................................................................. 34

Figure 3.3 – Concrete Compressive Strength Test Setup and Cylinder Failure .................................................................................. 37

Figure 3.4 – Stress-Strain Curves for Steel Plate............................................................................................................................... 39

Figure 3.5 – U-Shaped Steel Plates with Shear Studs and Bar Stock................................................................................................. 40

Figure 3.6 – Welding of Shear Studs.................................................................................................................................................... 41

Figure 3.7 – Welding of Bar Stock to Underside of Top Steel Return............................................................................................... 42
Figure 3.8 – Cambering Process of U-Shaped Steel Plate ...................................................... 44
Figure 3.9 – Cambering Process with Tie-Down System in Place ......................................... 45
Figure 3.10 – (a) Reinforcement Cage and Camber of Girder, (b) Prestressing Strands, and (c) View of Reinforcement from within Girder .............................................................. 46
Figure 3.11 – Formwork for Sides and Top Curvature of Girder ........................................... 47
Figure 3.12 – (a) Reinforcement Cage and Camber of Girder, (b) Prestressing Strands, and (c) View of Reinforcement from within Girder .............................................................. 46
Figure 3.11 – Formwork for Sides and Top Curvature of Girder ........................................... 47
Figure 3.12 – (a) Casting and Vibration of Concrete, (b) Placement of U-Shaped Steel Bars ....................................................................................................................... 48
Figure 3.13 – Release of Prestressing Strands ........................................................................ 49
Figure 3.14 – Tie Down System from (a) Above the Strong Floor, (b) Below the Strong Floor ....................................................................................................................... 51
Figure 3.15 – Diagram of Tie Down System ........................................................................... 51
Figure 3.16 – Placement of Hollow Core Planks .................................................................. 53
Figure 3.17 – Formwork for Floor System Specimens ........................................................... 54
Figure 3.18 – (a) Bent Reinforcing Bar, (b) Grouting Reinforcing Bars in Keyways, and (c) WWF Reinforcing for Topping Slab .................................................................................. 55
Figure 3.19 – (a) Placement of Concrete and (b) Screeding of Slab ........................................ 56
Figure 3.20 – (a) Screeding and Vibrating of Slab and (b) Finished Slab ............................... 57
Figure 3.21 – Girder Specimen Test Setup ............................................................................. 59
Figure 3.22 – Floor System Specimen Test Setup .................................................................. 60
Figure 3.23 – Diagram of Floor System Test Setup ............................................................... 61
Figure 3.24 – Strain Gauge on Steel Plate and Protective Cover ............................................ 62
Figure 3.25 – Strain Gauges on Rebar and Protective Cover .................................................. 63
Figure 3.26 – (a) PI Gauge and (b) PI Gauge Layout ............................................................. 64
Figure 3.27 – String Potentiometer Layout ......................................................................... 65
Figure 3.28 – LMT’s Used to Measure Slip ......................................................................... 66
Figure 4.1 – Moment vs. Deflection for DKP 2 ................................................................. 70
Figure 4.2 – Moment vs. Deflection for DKP 1a ............................................................... 71
Figure 4.3 – Moment-Deflection Envelopes for All Specimens .......................................... 72
Figure 4.4 – Sequence of Deflection for DKP 1 ................................................................. 73
Figure 4.5 – Deflection of DKP 1a at Construction and Failure Loads .................................. 74
Figure 4.6 – Elevation Profile of Deflections for DKP 1a .................................................. 75
Figure 4.7 – Elevation Profile of Deflections with Camber for DKP 1a .............................. 75
Figure 4.8 – Moment vs. Strain for DKP 1 ....................................................................... 78
Figure 4.9 – Strain Profiles for DKP 1 .............................................................................. 79
Figure 4.10 – Total Strain Profiles at the Quarter Point for DKP 2a ................................... 80
Figure 4.11 – Slip Between Steel Plate and Precast Concrete Beam for DKP 2 ............... 81
Figure 4.12 – Slip Plots for DKP 1a .................................................................................. 82
Figure 4.13 – Concrete Strain Profile Along Flange Width for DKP 1a .............................. 84
Figure 4.14 – Transverse Flexural Crack along Hollow Core Plank Bottom .......... 85
Surface .............................................................................................................................. 85
Figure 4.15 – Concrete Crushing of DKP 1 ....................................................................... 87
Figure 4.16 – Yielded Compression Steel Reinforcement in DKP 1 ..................................... 88
Figure 4.17 – Localized Concrete Crushing and Shear Stud Failure for DKP 2 ............... 89
Figure 4.18 – DKP 1a Longitudinal Shear Cracks .............................................................. 91
Figure 4.19 – Flexural Cracks Along and at Failure Location of Girder Specimens

Figure 5.1 – Moment-Curvature for Partially Prestressed Girder (DKP 1)
Figure 5.2 – Moment-Curvature for Nonprestressed Floor System (DKP 2a)
Figure 5.3 – Moment vs. Deflection for Partially Prestressed Girder (DKP 1)
Figure 5.4 – Moment vs. Deflection for Nonprestressed Floor System (DKP 2a)
Figure 5.5 – Moment-Deflection Envelopes for Girder Specimens
Figure 5.6 – Moment Deflection Envelopes for Floor System Specimens
Figure 5.7 – Mid-span Moment-Deflection for DKP 2a
Figure 5.8 – Moment-Slip for DKP 2a
Figure 5.9 – Mid-span Moment-Deflection for DKP 1
Figure 5.10 – Moment-Slip for DKP 1
Figure 5.11 – Moment-Strain Envelopes for DKP 1
Figure 5.12 – Moment-Strain Envelopes for DKP 1a
Figure 5.13 – Strain Profiles for DKP 2
Figure 5.14 – Experimental and Predicted Mid-span Strain Profiles for DKP 2
Figure 5.15 – Strain Profiles for DKP 2a
Figure 5.16 – Experimental and Predicted Mid-span Strain Profiles for DKP 2a
Figure 5.17 – Variation of Strength Reduction Factor (φ)
Figure 5.18 – Stress Distribution and Force Equilibrium
Figure A.1 – Ramberg Osgood Stress-Strain Relationship for Various Reinforcing Steels

Figure A.2 – Stress-Strain Relationship for Concrete with Modified Popovics Equation

Figure A.3 – Linear Strain Distribution

Figure A.4 – Moment-Curvature Relationship to Determine Deflection at Mid-span

Figure A.5 – DKP 1 Moment-Curvature Relationship

Figure A.6 – DKP 1 Moment-Deflection Relationship

Figure A.7 – DKP 2 Moment-Curvature Relationship

Figure A.8 – DKP 2 Moment-Deflection Relationship

Figure A.9 – DKP 1a Moment-Curvature Relationship

Figure A.10 – DKP 1a Moment-Deflection Relationship

Figure A.11 – DKP 2a Moment-Curvature Relationship

Figure A.12 – DKP 2a Moment-Deflection Relationship

Figure B.1 – DKP 1 Applied Moment vs. Deflection

Figure B.2 – DKP 1 Applied Moment vs. Averaged Mid-span Strain

Figure B.3 – DKP 1 Applied Strain Profiles

Figure B.4 – DKP 1 Applied Moment vs. End Slip

Figure B.5 – DKP 1 Applied Moment vs. Deflection Envelope Comparison

Figure B.6 – DKP 1 Strain Profile Comparisons

Figure B.7 – DKP 1 Failure Photos
Figure B.8 – DKP 2 Applied Moment vs. Deflection........................................................... 161
Figure B.9 – DKP 2 Applied Moment vs. Averaged Mid-span Strain.................................. 162
Figure B.10 – DKP 2 Strain Profiles .................................................................................. 163
Figure B.11 – DKP 2 Applied Moment vs. End Slip............................................................ 164
Figure B.12 – DKP 2 Applied Moment vs. Deflection Envelope Comparison..................... 165
Figure B.13 – DKP 2 Strain Profile Comparisons ................................................................. 166
Figure B.14 – DKP 2 Failure Photos .................................................................................... 168
Figure B.15 – DKP 1a Applied Moment vs. Averaged Deflection ....................................... 170
Figure B.16 – DKP 1 Applied Moment vs. Averaged Mid-span Strain ................................ 171
Figure B.17 – DKP 1a Strain Profiles .................................................................................. 172
Figure B.18 – DKP 1a Applied Moment vs. End Slip.......................................................... 173
Figure B.19 – DKP 1a Applied Moment vs. Deflection Envelope Comparison ................... 174
Figure B.20 – DKP 1a Strain Profile Comparisons ............................................................... 175
Figure B.21 – DKP 1a Failure Photos .................................................................................. 177
Figure B.22 – DKP 2a Applied Moment vs. Averaged Deflection ....................................... 179
Figure B.23 – DKP 2a Applied Moment vs. Averaged Mid-span Strain .............................. 180
Figure B.24 – DKP 2a Strain Profiles .................................................................................. 181
Figure B.25 – DKP 2a Applied Moment vs. End Slip.......................................................... 182
Figure B.26 – DKP 2a Applied Moment vs. Deflection Envelope Comparison ................... 183
Figure B.27 – DKP 2a Strain Profile Comparisons ............................................................... 184
Figure B.28 – DKP 2a Failure Photos .................................................................................. 185
1 INTRODUCTION

1.1 Background

The use of composite steel and concrete structural members was documented well over a century ago. The first systems utilized concrete encased steel beams in which the concrete was mainly used for fireproofing. A series of tests conducted during the 1920’s demonstrated an increase in strength from the composite action developed between steel and concrete materials. This gave rise to utilization of composite construction and further research into the behavior of composites. One of the first behaviors studied was the bond and horizontal shear force developed at the concrete and steel interface. In order to resist the horizontal shear forces due to bending and to ensure interaction between concrete and steel members, mechanical shear connectors were developed. The introduction of the shear stud in the 1950’s quickly became and still is the standard mechanical shear connector. Shear studs are attached to the beam using a rapid welding process and once embedded in the concrete, act to resist the horizontal shear forces. Also during this time new methods of fireproofing made concrete encasement obsolete and the cost of labor and construction brought about steel deck flooring systems. Over the last few decades skyscrapers and other building systems have primarily utilized composite floor systems.

The conventional or typical composite floor system of today consist of a steel I-beam as the supporting member and steel decking or precast concrete planks with cast-in-place concrete
as the slab unit. In order to produce a cheaper, more efficient system, many improvements and modifications have been made. One such modification has been to use reduced height floor systems that lower the overall floor-to-floor heights of building. This can be done by a variety of methods including placing the ribbed decking or precast planks on the bottom flange of the supporting member. Other improvements sought after include ease and speed of construction.

In order to address some of these issues Diversakore® has developed an innovative shallow floor framing system that uses their Versa:T:™ beam along with precast hollow core planks as shown in Figure 1.1. A composite section is created with a U-shaped steel plate, precast hollow core planks, mild reinforcing steel, and cast-in-place concrete. As mentioned by Willis (2009), the floor system is constructed with the U-shaped steel plate that supports the hollow core planks and acts as stay in place formwork for the cast-in-place concrete. Shear studs are used as the shear connection and the cast-in-place concrete is cast monolithically to tie the hollow core planks and U-shaped steel plate into a composite T-beam. The construction process is expedited by comparison to a conventional system. The U-shaped plates arrive on-site followed by the hollow core planks and are hoisted into position. This in turn creates a safe, platform to work upon. The reinforcing cage is then tied and placed into the girders and welded wire mesh is placed on top of the hollow core planks as reinforcement for the cast-in-place concrete slab. Lastly the concrete tipping is cast for an entire floor, creating a monolithic frame.
The Diversakore® system allows for its components to be manufactured off-site under quality control monitoring and delivered to the construction site, saving space and cost of on-site supervision. The simplified assembly speeds the construction process and reduces the cost of labor. There is no form work needed for the floor system but temporary shoring is required along the girders until the section acts compositely. Based on the need to further increase the quality of the product and speed, ease, and cost of construction, Diversakore® has developed a precast girder system.

The precast Versa:™ system consists of the same components the cast-in-place system described above, but does not require shoring during final construction. A rendering of the precast floor system is shown in Figure 1.2. The precast girders are constructed by
cambering the U-shaped steel plate and casting a reinforced concrete, inverted T-section. With the system under development, two methods for cambering the plates were considered. The first method consists of cambering the U-shaped steel plates before the concrete is cast while the other method consists of cambering and prestressing the members. The girders and desired cambers are designed to resist and level out under the construction load. The precast girders are constructed at a precast plant and transported to the jobsite, where they are hoisted into place. The hollow core planks are supported by the girders and a cast-in-place concrete slab is utilized to engage the planks and form a composite T-beam. The quality control and constructability of the system are again enhanced by reducing the number of components produced on-site and needed for assembly. The result is an efficient shallow floor system that does not sacrifice strength and serviceability.

Figure 1.2 – Rendering of Diversakore® Precast Shallow Floor Framing System
1.2 Objectives

The objective of this research program was to investigate and evaluate the flexural performance of the innovative precast shallow floor framing system. In order to better understand the behavior of the system, the following were proposed:

1. Investigate the behavior of the Diversakore® girder and floor system in flexure through experimental investigation.

2. Develop analytical moment-curvature models to predict the response of the structural systems.

3. Identify the failure modes and the corresponding design limit states.

4. Develop guidelines for design and construction of the systems.

1.3 Scope

Experimental and analytical investigations were carried out to achieve the goals set forth in the objectives of the research program. The results from the experimental and analytical investigations were then compared and evaluated. Additional analysis and research examined the performance and limit states observed by the system to develop and improve the current design procedure. The following were performed to accomplish the objectives:
1. Tested full-scale girder and floor systems specimens in flexure. Each girder system and floor system consisted of one partially prestressed and one mild reinforced concrete girder, for a total of four tests.

2. Developed a moment-curvature layered sectional analysis to predict the response of the structural system.

1.4 Organization of Thesis

The organization of this thesis is divided into several chapters. Brief descriptions of each chapter are as follows:

Chapter 2 presents a review of relevant research. Analytical and experimental research into behavior of composite members was considered. Topics include moment-curvature modeling, experimental behavior, failure modes, and design considerations of composite floor systems. Lastly, the need for research is discussed.

Chapter 3 provides a detailed description of the experimental investigation. This includes the component utilized in the system, construction of the specimens, testing configuration, instrumentation used to measure various parameters, and the loading protocol.
Chapter 4 presents the results from the experimental investigation. Representative results are shown for each system with data from all parameters. The observed behaviors as well as modes of failure were also described for each specimen.

Chapter 5 evaluates the performance of the system by comparing the results from the experimental investigation to the predicted behavior from the moment-curvature layered sectional analysis. Various parameters were compared including deflection, slip, and strain behavior. Results were also compared against the strength and serviceability limit states. Design limit state for the failure modes are described and compared to current design codes.

Chapter 6 summarizes and gives the conclusions drawn from the testing program. It also provides recommendations for the current design procedure and future research.

In addition to the main content, two appendices are included. The first appendix explains the procedure and presents all the results from analytical investigation. The second appendix provides the results for each specimen from the experimental investigation.
2 LITERATURE REVIEW

2.1 Introduction

This chapter presents a comprehensive but not all encompassing review of the behavior of steel-concrete composite systems. This chapter is divided into the three main sections of innovative systems, background, and the need for research. The innovative systems section provides details of previous research involved with a similar composite system designed by Diveraskore® as well as research conducted on other types of slim floor systems. The background section presents experimental and analytical studies on composite behavior with various aspects of the system considered. The last section explains the gaps in research and need for research for this particular composite system.

2.2 Innovative Composite Systems

There have been many research programs devoted to the study of shallow or slim floor composite systems. In particular, two studies have focused on very similar products of the system considered in this research. The initial pilot study was conducted as the Georgia Institute of Technology by Lindsey, Leon, and Kim (2002). Willis (2009) followed up this research and began the initial testing program at North Carolina State University. Experimental and analytical investigations of slim floor composite systems have been performed by Leskela, Inha, and Iso-Mustajarvi (1997), Leskela (2002), and Kuhlmann and Rieg (2006). Additional investigations have looked at the behavior of components similar to
this research and include work by Oehlers (1993), Hui, Aiquin, and Derun (2005), and Nie and Zhao (2009).

The initial pilot study for Diversakore’s® shallow floor framing system was undertaken at the Georgia Institute of Technology by Lindsey, Leon, and Kim (2002). The research program had two objectives. One was to establish a definitive strength design methodology and to verify it through three full scale tests. The second was to perform a preliminary assessment of the fire resistance of the system. The composite floor comprised of a U-shaped supporting steel members and a hollow core plank and concrete slab floor system. Three floor systems were tested in flexure and differed in length, shear connection, reinforcing, and supporting member size. The U-shaped steel sections were made with varying configurations and consisted of steel plates, channels, and angles welded together as shown in Figure 2.1. The load was applied by either two or four point loads. Failure modes observed included separation of hollow core planks and concrete topping slab and shear failure of the hollow core planks. They determined that the experimental results correlated well with the theoretical calculated values. Analytically tests for fire resistance were performed using a finite element analysis and determined that the supporting steel members offered little strength and rapid loss of stiffness during a fire if left unprotected.
Willis (2009) initiated a testing program at the North Carolina State University (NCSU) to further investigate the flexural behavior of the Diversakore® shallow floor framing system. The objectives of the research program were to evaluate the current design procedure, determine what effect shear studs had on the behavior of the system, and to make recommendations for changes to the current construction and design of the system. Two tests were performed in flexure and loaded using four point loads. The specimens consisted of a ¼” U-shaped steel plate supporting hollow core planks and cast-in-place concrete. Mild reinforcing steel was added as positive moment reinforcement and concrete was cast monolithic to form a composite reinforced concrete T-beam (Figure 1.1). The two specimens differed only in the shear connection used. One specimen relied entirely on the bond between the steel section and cast-in-place concrete while the second specimen utilized headed shear studs. An analytical study was also performed to predict the response of the system. Failure modes included longitudinal shear cracking and loss of shear connection. From the results of the study, it was recommended that current design procedures needed to be modified to include the design for longitudinal and horizontal shear. Two methods for
determining the horizontal shear demand were presented. Lastly it was recommended that the thickness of the steel plate be reduced to optimize the efficiency of the system.

Oehlers (1993) performed three large scale flexure and shear tests on composite reinforced concrete beams with side profiled sheets as shown in Figure 2.2. For use in the field the composite girder’s profiles were to be produced at a factory, shipped to the site and used as permanent form work for the cast in place concrete leading to a more efficient construction process. The structural advantages of the system included increased stiffness, flexural strength, and shear strength. Composite action was provided by ribs along the sides of the profiled sheets that resisted the normal forces, ensuring the same curvature of all components was maintained. Variations in the reinforcement demonstrated the additional flexural and shear strength provided by the side profiles. Both sets of test also revealed an increase in strength without a decrease in ductility.

Figure 2.2 – Reinforced Concrete Beam with Side Profile Sheets (Oehlers, 1993)
Leskela, Inha, and Iso-Mustajarvi (1997) performed flexural tests and push out tests to determine the behavior and bond strength of U-section composite beams. A cross section of the specimens is shown in Figure 2.3. The inner surface of the U-section consisted of a checkered shutter surface to create the bond between the concrete and steel as shown in Figure 2.4. The results of a finite element analysis closely modeled the actual behavior of the flexural tests performed.

Leskela (2002) discussed and compared the behavior of shallow floor composite beams. The study looked at asymmetric I-sections supporting hollow core planks, composite slabs with
profiled decking, and precast concrete planks. Although the actual behaviors of these systems differ, many general observations were made. One main difference was that premature vertical shear failure occurred in hollow core slabs near the supports at the onset of steel yielding. The failure planes for shallow composite beams typically consisted of vertical shear interfaces just to the outside of the steel section. It was noted that the failure plane differed from that of typical composite beams, where an exact failure plane is defined. Tests of three degrees of shear connection showed that the bond between steel and concrete interfaces could not be relied upon. It was concluded the shear connection does not affect the serviceability of the system, but does have an effect on the ultimate capacity.

Kuhlmann and Rieg (2006) tested six girders and reported on the analytical and numerical evaluation of reduced height composite girders and their corresponding effective width. He stated that reduced height composite sections are different from typical composite sections because the concrete is typically cracked under service loads and the concrete chord cannot be neglected as part of the moment of inertia and is important to the bending moment of the section. Since the codes neglected the bending stiffness of concrete, deflections were overestimated resulting in a conservative design. Results of the experimental investigation showed concrete strain along the top of the slab to be bell shaped with less shear lag than assumed by some codes. The concrete strain had the opposite effect with the onset of cracking. Concrete strains were weaker near the center due to a relatively higher compression zone which resulted in less crack growth. Some codes also neglected the bending stiffness of the concrete resulting in conservative deflection calculations. The
investigations concluded that the effective width is greater than given by codes and that with increased slab thickness or width come increased effective width. A more effective design approach for calculation of effective width was proposed for future research.

Hui, Aiqun, and Derun (2005) experimentally tested 11 steel encased concrete beams in flexure. Eight specimens had full shear connection while three specimens only had partial shear connection. The results were compared to a numerical and finite element analysis study provided by the authors. The specimens consisted of encased concrete in a U-shaped steel form made up of cold-formed profiled steel welded to a thicker bottom steel plate as shown in Figure 2.5. Shear connection was provided by headed shear studs along the bottom of the section and on the channels top flange returns. Suggested advantages of this type of composite section were improved flexural capacity, economics, and constructability. The two failure modes reported were longitudinal shear failure on the vertical plane between the concrete slab and steel flanges and flexural failure typical of a reinforced concrete beam. The numerical analysis was only applied to the flexural failure mode. Assumptions made by Hui et al. in the analysis of beams with full shear connection included:

1. Planar sections and linear strain profile throughout the section.
2. Uniform yielding stress of steel in tension and compression and local bucking was ignored.
4. The steel and concrete share the same curvature.
Three possibilities for the calculation of ultimate flexural capacity considered the plastic neutral axis within the concrete flange, steel flange, and web of the composite beam. Partial shear connection can be desirable and more efficient at service level than full shear connected sections because they do not typically yield at the service limit state. A numerical analysis was also applied to the case of partial shear connection and included the following additional assumptions made by Hui et al.:

1. Planar sections in their own respect.
2. Shear connection was sufficient for transformation capacity.
3. Shear connection was ductile and worked at the same load.
4. Bond is neglected between the concrete and steel surfaces.

Partial shear connection ultimate flexural capacity calculations considered the cases where the plastic neutral axes are within their own flanges, the steel web and concrete flange, the concrete web and steel flange, and their own webs. The numerical calculations and finite element analysis correlated well with and were validated by the experimental results.

Figure 2.5 – Section of Steel Encased Concrete Composite Beam (Hui et al, 2005)
Nie and Zhao (2009) tested five steel plate-concrete composite beams, as shown in Figure 2.6, in four-point and three-point bending to investigate their flexural behavior. Test specimens consisted of the same cross section and span, but varied in steel plate thickness and degree of shear connection. Advantages mentioned were reduced weight due the elimination of concrete cover, unexposed crack formation, and resistance to stress in all directions. Failure modes of flexure and combined flexure and horizontal slip were observed and analyzed. Strain values indicated a linear profile was maintain at different levels of loading and verified the assumption that plane sections remain plane. A theoretical analysis showed that the ultimate flexural strength could be accurately calculated but the actual service load deflections were greater than those calculated by code methods. The increased deflections resulted from a lower stiffness associated with slip between the plate and concrete. It was concluded that without full shear connection and accounting for the slip, the method for calculating deflections needed modification.

![Steel-Plate Concrete Composite Beam](image)

**Figure 2.6 – Steel-Plate Concrete Composite Beam (Nie and Zhao, 2009)**
2.3 **Background**

Although an abundance of research has been conducted on composite floor systems and partially prestressed beams, no direct research has been conducted on the behavior of the precast composite shallow floor framing system presented in this thesis. This section presents research dealing with the moment-curvature analysis of partially prestressed beams, the general behavior of composite sections, and additional slab specific design considerations. Since this research is part of the ongoing study at NCSU and was performed in conjunction with the work by Willis some parts of this study may overlap. As part of the analytical study and to predict the response of the structural system many moment-curvature analyses were looked at. They included work by Burns (1964), Naaman (1983), Al-Zaid and Naaman (1986), Al-Zaid, Naaman, and Nowak (1988), and Shushkewich (1990). Research by Hofbeck, Ibrahim, and Mattock (1969), Johnson (1970), Oehlers and Park (1992), Kemp and Trinchero (1993) and Poitter (2001) focused on the flexural behavior, longitudinal shear strength, and shear connector strength of composite beams. Many other studies have looked at what effects the slab system has on the flexural capacity of the section, shear connector strength, and longitudinal shear cracking. Studies that focused on composite systems with ribbed metal decking have been completed by Nie, Cia, and Wang (2005), Fahmy and Abu-Amra (2008), and Jeong (2008). Studies that focused on composite systems with precast hollow core planks have been completed by Bode et al. (1997), Fontana and Brogogno (1997), Lam, Elliot, and Nethercot (2000), Hicks, Lawson, and Lam (2006), and Lam (2007).
2.3.1  **Moment-Curvature Analysis**

Increasing the ultimate moment capacity of a prestressed concrete beam can be done in a number of ways with the addition of mild reinforcing steel as one method. Burns (1964) presented a moment-curvature analysis to determine the effects of varying the prestressing force. His analysis used stress-strain curves typical for high strength stress-relieved steel and a concrete stress strain curve similar to the Hognestad concrete model. The analysis calculated the applied moment with a given strain of the top fiber using an iterative process and force balance to determine the neutral axis. The ultimate moment capacity was assumed to occur at a strain of .0035 at the top fiber of concrete. Deflection calculations were taken using the area-moment principal and closely matched existing experimental data. From his analysis he determined varying the prestressing force resulted in a variation of cracking moments, but resulted in the same ultimate moment. Also, varying the area and prestressing force to keep a constant effective prestressing force resulted in a similar cracking moment with the higher ultimate moments developed for larger areas of steel.

According to Naaman (1983) most codes accounted for the design of reinforced concrete and prestressed concrete separately with hardly any mention of partially prestressed concrete. To determine the ultimate flexure capacity of partially prestressed concrete beam, he presented an approximate nonlinear design procedure. The procedure takes into account the amount of steel and the actual behavior of the prestressing, reinforcement, and concrete ultimate compressive strain. Several stress-strain models for steel were considered. The model
presented by Mentegotto and Pinto, that most accurately described the behavior of prestressing steel, was recommended. The analysis also calculated the forces in concrete using the ACI equivalent rectangular stress block. The results of the approximate nonlinear analysis followed closely with the results from the exact nonlinear analysis while the ACI code underestimated the predicted curvatures, rotations, and deflections.

Most designs for prestressed concrete beams for service loads used a linear elastic analysis that did not account for a cracked section. To allow for cracking in new structures and to account for overloading of existing members, Al-Zaid and Naaman (1986) presented a general analysis procedure for cracked and uncracked partially prestressed concrete composite sections within the elastic range. The analysis satisfied equilibrium, strain compatibility, and linear elastic stress-strain relations and included the following assumptions by Al-Zaid and Naaman:

1. It applies to reinforced, prestressed, and partially prestressed concrete sections with and without compressive reinforcement, given a rectangular, I, or T-shaped precast beam.
2. It applies to both shored and unshored construction.
3. It takes into account the change in the prestressing force due to the application of load.

For the analysis of cracked and uncracked sections they considered the following assumptions:

1. Steel and concrete are linear elastic in the range of stresses considered.
2. Plane sections remain plane under bending.

3. Perfect bond exists between steel and concrete.

4. Full interaction between the precast beam and the cast-in-place slab is insured (i.e., interface bond is sufficient).

For the analysis of the cracked section they considered the following additional assumptions:

1. Concrete does not withstand any tensile stress.

2. The prestressed girder does not crack under the effect of its own weight and the prestressing force.

3. The crack will not propagate into the cast-in-place slab under service loading.

4. Any external load applied after composite action leads to same change in curvature in the precast beam and the cast-in-place slab.

A set of equations were developed and evaluated the effect of various material properties and parameters. From their results they determined future design for partially prestressed composite structures should utilize this design method.

Al-Zaid, Naaman, and Nowak (1988) continued the previous work to incorporate the effects of sustained and cyclic loads. Time dependent factors such as shrinkage and creep of concrete and relaxation of steel were also analyzed.

Shushkewich (1990) presented a unified moment-curvature model for a variety of shapes and reinforcement. A set of equations were used to determine stress and strains of the cracked or uncracked section given an applied moment. Based on this model, the moment-curvature
relationship of partially prestressed members in the elastic range can be determined regardless of its shape or the number layers and types of reinforcing steel.

2.3.2 Composite Behavior

Hofbeck, Ibraham, and Mattock (1969) tested 38 push off specimens to determine the shear transfer strength typical at the interface of a precast beam and cast-in-place slab. Comparisons were made between initially cracked and uncracked specimens. The results indicated that the initially cracked section increased the slip and reduced the shear transfer capacity of the section. Other results concluded that dowel action played a significant role in initially cracked sections and that shear-friction theory conservatively estimates the concrete contribution to the shear transfer strength.

Johnson (1970) proposed an ultimate strength design method for the required amount of transverse reinforcement needed to prevent longitudinal shear failure in the slab of a composite beam. The design method included the contribution of all transverse reinforcement and stated that longitudinal bending had no effect on longitudinal shear strength. Based on the study, the need for transverse reinforcement was greatest with the absence of negative transverse bending. Minimum and calculated amounts of transverse reinforcements were suggested for the total, top, and bottom regions. The design also assumed transverse bending and longitudinal shear could be resisted by the top reinforcement.
Oehlers and Park (1992) studied the distribution of longitudinal shear across the dowel action of individual shear studs. Previous tests looked at the behavior of this dowel action with respect to the transverse reinforcement of the slab. 25 push out tests were performed on specimens to study the behavior when longitudinal cracks form. From the study it was determined that the effects due to the transverse reinforcement were affected by their stiffness rather than their strength.

Kemp and Trinchero (1993) conducted a design study and compared it to the measurements of four experimental tests to investigate the limits of stress with special interest in deflection. The purpose of their research was to eliminate the need of stress checks that some design codes called for by looking at serviceability deflections due to nonlinear stresses. More specifically, attention was given to the stress and strains at about 20% above the load causing first yield. Three examples of simply-supported composite beams were compared to six codes including, the AISC LRFD - 1987, British BS5950 (3) - 1990, Canadian S16.1 - 1989, Eurocode (Draft), and South African Sabs 0162 - 1984 and New. For the three examples serviceability deflections were less critical than ultimate flexure and current codes with the elastic stress criterion limiting the design. Two experimental tests were conducted on 6 m scaled down, simple supported, unpropped specimens with full and partial shear connections. To represent the distribution of loading, four point loads were applied to both specimens. The theoretical results determined by the moment curvature analysis performed by the author closely matched the experimental behavior in which the critical limit state was the ultimate
bending condition. The author concluded that the stress limit set by some international codes led to unwarranted conservatism in the design of composite beams.

Poitter (2001) conducted a study at the Virginia Polytechnic Institute and State University on longitudinal splitting of composite girders. Full scale tests were performed on hot rolled and joist girder composite systems. The objective of the research was to determine the flexural behavior of the system with the presence of longitudinal shear cracks and to determine the minimum amount of transverse reinforcement required to prevent longitudinal shear cracking. A comparative analytical study was also performed to generate design procedure for the minimum amount of transverse reinforcement required. Results from the experimental and analytical study concluded that the required longitudinal shear strength can be determined by two methods. It also presented two equations for determining the longitudinal shear strength. Lastly, it determined that transverse reinforcement was not required for flexural design as long as the flexural strength was calculated based on the concrete within the shear planes.

2.3.3 Slab Specific Behavior

Nie, Cai, and Wang (2005) studied the effects of slip on the behavior of steel-concrete composite beams with profiled sheeting. Five specimens with various degrees of interaction and shear stud spacing were tested and compared to an analysis method. The model developed accurately models the behavior observed and more accurately predicts the increased deflection and reduced stiffness associated with slip.
Fahmy and Abu-Amra (2008) conducted an analytical study on longitudinal cracking of concrete slabs with ribbed metal decking. A finite element analysis was used to determine the effects of changing multiple parameters on the occurrence and location of longitudinal crack formation. The first parameter looked at one point, two point, and uniformly distributed loading. Based on the results the longitudinal crack occurred at the top of the slab for uniformly distributed loading due to bending in the transverse direction and occurred in the bottom of the slab for one and two point loading from transverse tensile stress due to interaction shear forces. The initial cracking moment was increased with increased concrete compressive strength, beam to slab width ratio, steel beam size, existence of ribbed metal deck, height of metal deck, and amount of transverse reinforcement. Design curves were developed for varying degrees of connection with a range of ratios of ultimate to longitudinal cracking moment, steel to concrete strength, beam span to slab width, and transverse reinforcement.

Jeong (2008) developed a simplified model to determine the performance of steel-concrete composite slabs with partial interaction. Nine specimens were tested in flexure with variations in span length and spacing of studs along with a more expansive finite element analysis. The model related the strength and stiffness properties corresponding to the degree of shear connection and degree of shear interaction, respectively. Failure modes observed included flexure and interaction. Interaction failure occurred before the full flexural capacity of the member was reached. Results of the flexural tests and simplified model compared well to the “m-k method” in the Eurocode. The simplified model also improved on the
limitation of the “m-k method” by only requiring push-out test to determine the degree of partial interaction.

Bode et al. (1997) performed five full scale flexural tests on slim floors made of hollow cores resting on the bottom flange of a steel wide flanged section to determine the influence of interaction to the bending of the beams and transverse shear capacity of the hollow cores. It was determined that the interaction had little to no influence on the bending capacity of the beam. However, transverse bending on the bottom flange (flexible support) of the steel section resulted in biaxial bending in the hollow cores that must be considered. This resulted in cracking along the prestressing and the reduction of shear capacity for the hollow core planks.

With the number of composite slim floor systems growing, the design of such systems must consider a system effect. The placement of hollow core on the bottom plate/flange of a wide flange section can be considered a flexible support and can decrease the transverse shear strength of the hollow core sections and therefore must be accounted for in design (Fontana and Borgogno, 1997).

Lam, Elliot, and Nethercot (2000) conducted a parametric study on composite steel beams with precast concrete hollow core slabs. The finite element analysis performed extended a previous experimental study on the effects of shear studs, transverse reinforcement, and in situ concrete on composite action. The objective of the study was to determine the behavior
of the slab in bending and the ultimate capacity of the composite section. The first study performed was on the compression slab with variations in concrete strength and transverse steel reinforcement area and strength. The transverse reinforcement influenced the mode of failure which was either tensile splitting or compression crushing of the in situ concrete. The finite element model showed little contribution from increased concrete strength because it only considered tension stiffening and neglected aggregate interlock and bond strength. The second study compared 45 finite element full composite beam models to the results of three experimental tests. The parameters changed for this study were transverse reinforcing, depth of hollow core, shear stud spacing, and supporting beam size. The results concluded that changes in the transverse reinforcement led to no changes to the stiffness and increased the moment capacity, but decreased the ductility of the beam. Increasing the slab height resulted in increased moment capacity, but led to undesirable tensile failure in the hollow cores when smaller steel section were used. Increase in shear stud spacing resulted in reduction of the moment capacity and increased deflection. Increased size in steel sections also resulted in increased moment capacity, but beam yielding and slab compression failures were observed. Based on the results of the parametric study design procedures and charts for sizing of composite beams are to be developed.

Hicks, Lawson, and Lam (2006) investigated the design and behavior of precast hollow core slabs in composite construction. Some of the advantages they listed included:

1. Reduced weight and depth of steel section resulting in reduced cost and building height.
2. Fewer secondary beams due to longer span lengths of hollow core slabs.

3. Enhanced durability in semi-exposed applications.

4. Shear connectors welded prior to delivery.

5. Number and spacing of shear connectors not dictated as in use with profiled steel decking.

6. Pre camber of hollow cores and steel able to offset imposed dead loads.

Unbalanced loading from the installation sequence of the hollow cores and vertical shear force due to the hollow core resting on a non-rigid support were among a few design considerations. The authors argue that the presence of the hollow cores no longer make the slab construction monolithic thereby reducing the effective flange width. Recommendations on transverse reinforcement were given along with possible failure planes that were either vertically through the slab or directly around the shear connector (Hicks et al.).

With most research concentrated on reinforced concrete and metal deck construction, Lam (2007) performed 72 full-scale push tests to determine the capacity of headed shear studs in precast hollow core slabs. Longitudinal shear capacity depends upon the strength of the shear connector and the slabs resistance to longitudinal cracking. To determine the capacity of the shear connection and degree of slip, push out tests were developed. The standard push vertical push test was not suitable for use with hollow core slabs so a horizontal push test arrangement (Figure 2.7) was developed and validated the results obtain from the standard push test. The parameters in the study included stud sizes, hollow core depth, end profile, in-situ concrete strength, and level of transverse reinforcement. Concrete crushing around the
connections, stud yielding, and the combination of the two were observed failure modes. Increased strength of the shear connection was observed with increased transverse reinforcement and in-situ concrete gap width and strength with little effect from the hollow core height. The transverse reinforcement limited longitudinal splitting, provided shear transfer between the beam and slab, and provided confinement against concrete splitting. It was also concluded that the transverse reinforcement had the most influence on the shear capacity and slip ductility of the section.
Figure 2.7 – General Arrangement for Horizontal Push Test (Lam, 2007)
2.4 Need for Research

A great deal of research has been completed on the behavior of steel-concrete composite beams. Most of the research to date has dealt with typical composite beams. However, limited research has been conducted on innovative composite systems such as slim floor systems, steel plate composite beams, and reinforced concrete composite beams. While all of the previously discussed research can in some way be applied, additional research is needed to further understand the behavior of this system.

The research that has been conducted on the innovative composite floor systems includes monolithic, nonprestressed and non-cambered floor systems. This research focused on the flexural behavior of the system and studied the shear connection strength developed from shear studs and the natural bond between the concrete and steel. This Diversakore® system is unique from previously tested systems because it utilizes cambered precast concrete composite beams and is assembled using unpropped construction. The camber and additional strength from prestressing the girders also needed to be considered. Additional research was needed to define the behavior and interaction between the concrete girder, hollow core planks, and cast-in-place concrete.
3 EXPERIMENTAL INVESTIGATION

3.1 Introduction

In order to better understand the ultimate strength and serviceability performance of the innovative precast shallow floor framing system, an experimental program was undertaken at the North Carolina State University (NCSU) Constructed Facilities Laboratory (CFL) in Raleigh, North Carolina. The experimental program consists of four full-scale specimens, including prestressed and nonprestressed precast girder specimens and prestressed and nonprestressed precast girder floor systems.

3.2 Test Specimens

All precast girders were constructed by Atlanta Structural Concrete Co in Buchanan, GA. The test specimens and their reinforcing type and test configuration are listed in Table 3.1.

Table 3.1 – Experimental Investigation Test Matrix

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Reinforcing</th>
<th>Test Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>DKP 1</td>
<td>Partially Prestressed</td>
<td>Precast Composite Girder</td>
</tr>
<tr>
<td>DKP 2</td>
<td>Mild</td>
<td></td>
</tr>
<tr>
<td>DKP 1a</td>
<td>Partially Prestressed</td>
<td>Composite Floor System</td>
</tr>
<tr>
<td>DKP 2a</td>
<td>Mild</td>
<td></td>
</tr>
</tbody>
</table>
3.2.1 Specimen Components

3.2.1.1 U-Shaped Steel Plate

A U-shaped steel plate was used as the base for each precast girder. It was bent using a hydraulic press brake and cut using a computer controlled plasma cutter. The plate was \( \frac{1}{4} \) inch thick A572 Steel and was thirty feet in length. The cross section measured 30 inches wide along the bottom, 4 inches high and had 2.5 inch long returns along the top as shown in Figure 3.1.

![Figure 3.1 – U-shaped Steel Plate](image)

3.2.1.2 Steel Reinforcement

Grade 60 steel rebar was used as tension and compression reinforcement for all specimens. Five No. 9 bars with a cover of 2 inches above the steel plate were utilized as tension
reinforcement and four No. 8 bars with 2 inches of cover from the top of the precast girder were utilized as compression reinforcement. No. 4 stirrups were used as shear reinforcement in compliance with the minimum requirements specified in ACI 318-08. WWF 6 x 6 x W2.9 x W2.9 was utilized to reinforce the concrete slab of the floor systems from shrinkage and temperature effects.

3.2.1.3 Prestressing Steel

Prestressing steel was utilized for specimens DKP 1 and DKP 1a. Six – ½ inch 7-wire strand low relaxation prestressing wire was used as additional tension reinforcement as well as to induce camber into the specimens.

3.2.1.4 Shear Studs

Shear studs were placed with the same stud pattern on all test specimens. The shear studs used were 5/8 inches in diameter and 5 inches tall. They were placed at a 12 inch staggered spacing along the steel plate with each stud offset 3 inches from either side of the centerline of the steel plate.

3.2.1.5 Bar Stock

Three inch by ¼ inch A36 Bar stock was welded to the underside of the top returns at 4 ft spacing o.c. in order to stabilize and prevent opening or overturning of the U-shaped steel section during construction.
3.2.1.6  *Hollow Cores*

8” Gate Core - untopped hollow core planks were supplied by Gate Precast Company and delivered to the CFL as shown in Figure 3.2. Hollow core planks 4 feet wide and cut to a length of 32 inches were placed transverse to and on the top returns of the steel plate for the floor system specimens.

![Figure 3.2 – Delivery of Hollow Core Planks](image)

3.2.1.7  *Concrete for Precast Girders*

All concrete for the precast girders was supplied by Atlanta Structural Concrete Co. at an onsite manufacturing facility. The concrete strength required at release for the prestressed specimens was 3500 psi and 7000 psi at 28 days.
3.2.1.8 Concrete for Topping Slab

Concrete used for the prestressed and nonprestressed floor systems was provided by S.T. Wooten Concrete. The concrete mix was specified to have a compressive strength of 5000 psi, 6 inch slump, and utilized 3/8 inch pea gravel aggregate.

3.2.1.9 Bent Bars

No. 4 Bent bars were placed within the keyways between each hollow core and spanned across the precast girder in order to support the hollow core planks.

3.2.1.10 Grout

Cementitious grout was placed into the hollow core plank keyways and was used to anchor the bent bars. The mix was 2-½ parts sand to 1 part Portland cement with water added for workability per PCI Manual for Design of Hollow Core Slabs. Grout was mixed at the CFL in a portable concrete mixer.

3.2.1.11 Perforated Angle

14 gauge slotted steel angle 3-1/8” x 1-5/8” was placed along the bottom outside edge of the hollow cores in order to simulate continuity of the hollow cores along their length and as a safety measure to reduce the risk of hollow cores from falling during testing.
3.3 Material Properties

Testing was performed to verify the material properties of concrete, steel, and grout used to construct the each specimen.

3.3.1 Concrete Compressive Strength

Concrete cylinders were produced and tested by ASC in order to determine the compressive strength of concrete for each specimen. ASC tested two cylinders for each specimen at one day for prestressed specimens and at three days for nonprestressed specimens. Additional tests were performed at 7 and 28 days for each specimen. Tests for concrete cylinders made by ASC were also performed at the CFL to determine the test day concrete compressive strength of each specimen.

Concrete cylinders were also produced at the CFL for the casting of the slab for each floor system. All cylinders were 4” x 8” and cast in accordance with ASTM C31. Cylinders were placed alongside their corresponding specimens to closely match the curing conditions. Tests were performed at 7, 21, 28 days, and on test day. Table 3.2 presents the average compressive test day strength of concrete for each specimen. The diameter of each cylinder was taken with a vernier caliper at two locations at mid height and averaged to calculate the cross area. Cylinders were tested in accordance with ASTM C39 at a rate of 35 ± 7 psi/s using a 500 kips capacity Forney machine. The test setup and a typical cylinder are shown in
Figure 3.3. The maximum load recorded was used to calculate the ultimate stress of each specimen.

Table 3.2 – Ultimate Compressive Strength of Concrete

<table>
<thead>
<tr>
<th>Test ID</th>
<th>DKP 1 Precast</th>
<th>DKP 2 Precast</th>
<th>DKP 1a Precast</th>
<th>DKP 1a Slab</th>
<th>DKP 2a Precast</th>
<th>DKP 2a Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate</td>
<td>7400</td>
<td>6900</td>
<td>7900</td>
<td>6300</td>
<td>7200</td>
<td>7000</td>
</tr>
<tr>
<td>Compressive</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength (psi)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.3 – Concrete Compressive Strength Test Setup and Cylinder Failure
3.3.2 Grout Compressive Strength

Three grout cubes were made in 2 inch molds for each floor system. Grout cubes were tested to verify the compressive strength of grout used to tie in the bent bars and hollow core planks. Two sides were measured with vernier calipers at mid height and were used to determine the cross sectional area of each cube. Three grout cubes were tested on the day of each test in the Forney machine at a rate of 35 ± psi/s.

3.3.3 Steel Plate Tensile Strength

A large area of the steel plate was cut from the end of one of the test specimens after failure. From the plate three steel coupons were cut 10 inches in length and ¾ inch in width. These coupons were then milled to ½ inch along a reduced length of 4 inches. To determine the strength properties of the steel, specimens were tested in a 20 kips Q-test MTS machine. The dimensions of the coupons were measured using vernier calipers and all were tested in accordance with ASTM A370. A gauge length was marked along the reduced section and a 2 inch extensometer was placed at the center of the coupon. The program Test Works 4 monitored the load and stroke data and calculated strain from the extensometer readings. The elongation was measured after rupture based on the marked gauge length. The data was then used to determine steel properties and to develop the stress-strain curves for each coupon (Figure 3.4). The yield strength, ultimate strength, and elongation for each coupon are presented in Table 3.3.
Table 3.3 – Material Properties of Steel Plate

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Yield Strength (ksi)</th>
<th>Ultimate Strength (ksi)</th>
<th>% Elongation at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon 1</td>
<td>62.3</td>
<td>73.7</td>
<td>31.9</td>
</tr>
<tr>
<td>Coupon 2</td>
<td>62.4</td>
<td>74.0</td>
<td>32.3</td>
</tr>
<tr>
<td>Coupon 3</td>
<td>62.8</td>
<td>74.2</td>
<td>31.9</td>
</tr>
<tr>
<td>Average</td>
<td>62.5</td>
<td>74.0</td>
<td>32.0</td>
</tr>
</tbody>
</table>
3.4 Construction of Test Specimens

This section describes the construction of all specimens from the initial construction at the CFL to the girder construction at ASC back to the final construction for the floor systems at the CFL.

3.4.1 Initial Construction

The construction of the shear studs and bar stock was performed at the CFL. The completed specimens are shown in Figure 3.5.

Figure 3.5 – U-Shaped Steel Plates with Shear Studs and Bar Stock
3.4.1.1 Shear Studs

Shear studs were welded to the steel plate at the CFL using a Nelson Nelweld Model 6000 Stud Welder. Shear studs were placed in the welding gun and welded to the steel plate at a setting of 1200 amps and a plunge time of 0.7 seconds. Ceramic ferrules were placed at the base of the shear studs to encase the weld and to facilitate a consistent weld collar around the stud. The stud welding process and finished product are shown in Figure 3.6.

Figure 3.6 – Welding of Shear Studs
3.4.1.2  *Steel Bar Stock*

All steel bar stock was placed underneath and spanned between the top returns of the steel plate. They were welded using a Miller tig welder. The welding process and close up of a weld are shown in Figure 3.7

![Welding of Bar Stock to Underside of Top Steel Return](image)

**Figure 3.7 – Welding of Bar Stock to Underside of Top Steel Return**

3.4.2  *Precast Girder Construction*

The construction of the precast girder specimens took place at Atlanta Concrete Structural Concrete, Co. in Buchanan GA. The innovativeness of the precast system resulted in unique construction methods with adjustments made as needed.
3.4.2.1 Cambering of U-Shaped Steel Plates

The first task in the construction process was cambering of the U-shaped steel plate. The steel plate for the prestressed precast girder and nonprestressed precast girder were constructed with slightly different methods.

One end of prestressed girder’s plate was placed against and welded to the casting bed. Angles were welded to the casting bed at the mid points and quarter points to provide the required camber. For this case the camber was 2-½ inches at the midpoint and 1-7/8 inches the quarter points. The steel plate was then bent along the top edges of the angles and casting bed. In order to pull the other end to the casting bed a large concrete block was used to add additional load. This caused an undesirably higher deflection at the midpoint. In order to reduce the camber at mid-span, steel rods were welded to the casting bed to either side of the steel plate. An angle that spanned across the steel plate was placed on the rods and tightened down to stress the steel plate to the angle underneath. The bottom of steel plate was then welded to the angle using a stick weld. The second end of the steel plate was welded to the casting bed to complete the process. Figure 3.8 shows the welded angles on the casting bed to produce the cambered shape, the concrete block method used to stress the second end of the plate down to the casting bed, and the end of the plate welded to the casting bed.
The nonprestressed girders plates were cambered in a similar but slightly different fashion. Angles were again welded to the casting bed at mid-span and at the quarter points, but for this case, the design called for a camber of 3 inches at the midpoint and 1-7/8 inches at the quarter points. The same process was used in welding the steel plate to the bed and angle
with the exception of using the rod and angle tie down system as shown in Figure 3.9 in lieu of the concrete block in order to complete the cambering process.

![Figure 3.9 – Cambering Process with Tie-Down System in Place](image)

3.4.2.2 Reinforcement Construction

The prestressed girders reinforcement was placed within the U-shaped steel plate after the cambering process. Stirrups were set in place on 1 inch chairs. Five 29-½ foot No. 9 bars were run through the stirrups and tied in place with rebar ties. Four No. 8 bars of the same length were then run through and tied in place at the top of the closed loop stirrups. The six
prestressing stands ran through both girders and were tensioned with a force of 31 kips resulting in an initial stress of about 202.5 ksi, 75% of the ultimate tensile stress.

The nonprestressed girders reinforcement was built in the same manner, but was done prior to the cambering of the steel plate. The steel plate and reinforcement were picked up and placed on the casting bed together. There was no additional reinforcement for the nonprestressed girders. Figure 3.10 shows (a) a cambered specimen with the reinforcement cage, (b) prestressing strands, and (c) view of all reinforcement from inside the cage.

Figure 3.10 – (a) Reinforcement Cage and Camber of Girder, (b) Prestressing Strands, and (c) View of Reinforcement from within Girder
3.4.2.3 *Form Work and Casting of Concrete*

Form work was placed flush against and secured to the sides of the U-shaped steel plate. The design called for the top surface of the concrete to follow the same curvature as the camber so Styrofoam was used as form work for the web of the girder. Two layers of 2 inch thick Styrofoam rested on the top returns of the steel plate and lined the sides of the girder as shown in Figure 3.11. They were taped with double sided tape and then glued with silicon to the steel plate flange and side of the formwork.

![Figure 3.11 – Formwork for Sides and Top Curvature of Girder](image)

Concrete was provided on site and was unloaded into the forms from the shoot of the concrete truck (Figure 3.12(a)). It was then vibrated to help consolidate the concrete. The top surface was finished using two different methods. For the girders specimens a smooth
finish was given and for the floor system specimens a raked finish was given. U-shaped steel bars were also placed in these girders in order to provide additional interaction between the girder and slab as shown in Figure 3.12(b).

![Figure 3.12 – (a) Casting and Vibration of Concrete, (b) Placement of U-Shaped Steel Bars](image)

The morning following the casting of the prestressed girders, the strands were cut and released individually and simultaneously from both ends of the girders using blow torches as shown in Figure 3.13.
3.4.2.4 Storage and Delivery

The girders were stored at Atlanta Structural Concrete, Co’s yard following casting. Once the testing schedule permitted the girders were shipped to the CFL.

3.4.3 Floor System Construction

The floor system test specimens required additional construction for the slab and is described in the following section.

3.4.3.1 Tie Down System

In order to simulate the actual construction load experienced by the girders in the field, a tie down system was employed. Although a uniform load could not be simulated, two point loads were used at the third points to closely match the actual moment produced.
Steel plates, ½ inch thick were welded to the verticals on both sides of the U-shaped steel plate at the third points. A hole in the plate allowed for the bolting of a clevis to the plates. Bolts were then placed into the end of clevis to support an HSS. The HSS4x4x½ were 40 inches in length and spanned transverse to the length of the girder. Rods were bolted to the HSS and passed through the strong floor. Chairs were assembled to the underside of the strong floor with a plate and nut in order to lock the system off at any point. A load cell was placed between the strong floor and chair to record the load from the tension in the system. A steel plate and 10 ton hydraulic jack were then loosely tightened on the rod against the bottom of the chair. Hydraulic lines were run from each jack, through the strong floor, to a 6-way hub. A hydraulic line was run to the hub and connected to hydraulic hand pump with a close off valve in between. Figure 3.14 (a) shows the tie down system from above the strong floor while Figure 3.14 (b) shows it from below the strong floor. A diagram of the tie down system is shown in Figure 3.15. The tie down systems, located at the third points, were engaged and locked off to load the girder to the simulated construction loads.
Figure 3.14 – Tie Down System from (a) Above the Strong Floor, (b) Below the Strong Floor

Figure 3.15 – Diagram of Tie Down System

- **Hydraulic Jack**
- **Chai Load Cell**
- **HSS4x4x1/2 Clevis**
- **PL1/2x6x7-1/2 Threaded Rod**
- **Strong Floor**
- **Dywidag Bar**
- **Load Cell**
- **Chai Hydraulic Jack**
3.4.3.2  *Hollow Core Planks*

In the field hollow core planks span girder to girder, but for the test specimens the planks are cantilevered to both sides of the precast girder. In order to support the planks for test specimen construction, stud wall supports were used. Walls were built to the same height of the ledge of the girder. After the girder was stressed down to simulate the construction load, the planks were flown in using the overhead crane and put into place spanning the girder and stud walls as shown in Figure 3.16. In order to support the planks after removal of the stud walls bent No. 4 bars were placed and grouted into the keyways between the planks. The core of the hollow core planks were blocked out flush with the edge of the plank by 1-½ inch Styrofoam disks. The Styrofoam disks were added to simulate the worst case scenario in which concrete did not fully flow and fill the cores.
3.4.3.3  *Form Work and Casting*

Before grouting or placement of concrete took place, formwork was installed along the perimeter of the specimen (Figure 3.17). Holes were drilled into the hollow cores planks using a hammer drill and Tapcon concrete screws held 1” x 6” wood boards that provided for a 2 inch slab.
Duct tape was also used to cover the rest of any exposed joints between the hollow core planks from the bottom and sides. The bent reinforcing bars were then placed and grouted into the keyways between adjacent hollow core planks as shown in Figure 3.18 (a) and (b). Figure 3.18 (c) shows the placement of the WWF on the surface of the girder and hollow cores planks as slab reinforcement for shrinkage and temperature effects.
The next series of figures (Figure 3.19 and Figure 3.20) show the casting process with a completed floor system specimen shown last. Concrete arrived and was placed using a concrete hopper, maneuvered by the overhead crane. The concrete was spread across the slab and consolidated using a concrete vibrator. It was then screeded and finished with a smooth surface.
Figure 3.19 – (a) Placement of Concrete and (b) Screeding of Slab
Figure 3.20 – (a) Screeding and Vibrating of Slab and (b) Finished Slab
The nonprestressed floor system had 1 inch chairs supporting the WWF, resulting in the mesh protruding through the slab in a few places. The temperature of the concrete was hot and began setting up towards the end of the cast, resulting in more difficult placement and honeycombing in a few areas. The placement and finishing of the concrete was performed by an outside contractor.

The prestressed floor system did not use chair to support the WWF, instead the mesh was pulled up into the slab during the cast. The concrete was placed and finished by students and staff at the CFL.

3.5 Test Setup

Test Specimens were positioned onto a pin and roller system on top of 24 inch high concrete blocks with a clear distance of 27 feet. A 440 kips MTS Actuator applied a gravity load that was distributed between four equally spaced point loads at a distance of 5.4 feet apart to simulate a uniformly distributed load. In order to distribute the load evenly three spreader beams were used in a loading tree. The main spreader beam was directly attached to the actuator and was a built up section consisting of two C15x40’s for the web and PL1-½x12’s for the flanges. It also had a total length of 156 inches and had stiffeners welded vertically to the web. The secondary spreaders beams were W14x176’s, 90 inches in length and had ½ inch plates welded to the ends. Between the main and secondary spreaders another pin and roller system was utilized in the configuration.
The girder specimens had additional pin and roller systems below the secondary spreader and girder that bore on ¾ inch neoprene rubber pads. Pins were employed at the inside while rollers were employed to the outsides of the secondary spreader beams. The bearing surface area of the load points was 6 inches in width and spanned transversely across the entire top surface of the girders. Figure 3.21 shows a typical setup for the girder specimens.

Figure 3.21 – Girder Specimen Test Setup
The floor systems specimens also had additional pin and roller systems below the secondary spreader beam system. These pin and roller systems were made up of a ¾ inch thick steel plate as the pin and Teflon coated bearing pad and a thin polished steel plate as the roller. These systems were configured in the same manner as the girder specimens; however they bore on the 8 inch side of steel HSS8x6x½. The HSS’s rested on ½ inch thick, 8 inch wide neoprene rubber pads. They were 90 inches in length and spanned transversely across the entire slab to create an evenly distribute line load. Figure 3.22 shows the typical setup used for the floor system specimens while a diagram of the test setup can be seen in Figure 3.23.

![Figure 3.22 – Floor System Specimen Test Setup](image-url)
3.6 **Instrumentation**

Instrumentation was used on all tests specimens to measure strains, deflections, and slip during the testing.

3.6.1 **Strain Gauges**

In order to observe a complete strain profile at different sections of the specimens, a total of 36 electrical resistant strain gauges with a resistance of 120 ohms were placed at the

---

**Figure 3.23 – Diagram of Floor System Test Setup**
midpoint and quarter points of each specimen. Gauges of 0.250 inch length were placed directly on the U-shaped steel plate and gauges of 0.125 inch length were placed on the rebar. Before the concrete was cast, gauges were placed on the inside of the U-shaped steel plate along the bottom, sides, and on the underside of the top flange returns, and on the top and bottom reinforcing steel. In order to protect the gauges from moisture and disturbance during casting and testing rubber butyl protective pads were placed over each gauge and wiring was protected by plastic tubing. Strain gauges were also placed along the bottom and sides of the U-shaped steel plate prior to testing. Figure 3.24 and Figure 3.25 show the application of the strain gauges on the steel plate and rebar with the protective coverings.

![Figure 3.24 – Strain Gauge on Steel Plate and Protective Cover](image)
3.6.2 PI Gauges

Five 100 mm PI gauges were used to measure concrete strains along the top surface. A single PI gauge was placed at each quarter point and three PI gauges were placed along the mid-span of each specimen (Figure 3.26). For the girder specimens, one was placed along the centerline with the other two offset 8 inches to either side. For the floor systems, gauges were offset 32-½ inches from the center or 12 inches from the edge in order to observe any differential strain along the width of the topping slab during the test. PI gauges were positioned ¾” above the surface of the specimens and therefore read the strain at that point above the surface.
3.6.3 String Potentiometers

String potentiometers (string pots) measured the deflection of the specimens at the mid-point and quarter points. For the girder systems only one string pot was used at the quarter points and two at the mid-point. The mid-point string pots were placed 8 inches off center, in-line with the PI gauges on the top surface. The floor system added two string pots beneath the hollow core section at all locations and were offset 32-½ inches from the centerline of the specimen as show in Figure 3.27.
3.6.4 Precast Girder Construction

Slip was measured using linear motion transducers (LMT) at the ends of the specimens. For the bare specimens two LMT’s were used on each end to measure slip at the concrete to steel interface. The floor system added two LMT’s to each end to measure slip between the hollow cores and steel plate. The relative slip between the hollow cores and concrete slab could then be calculated using the other two measurements. The positioning of the LMT’s is shown in Figure 3.28.
3.6.5 Data Acquisition System

Data was continuously recorded by a Vishay 5000 data acquisition system. Load, displacement, slip, and strain data was recorded at a rate of one scan every five seconds and monitored using Strain Smart software.

3.7 Loading Protocol

All test specimens were loaded statically up to failure and were displacement controlled. Load steps were cyclic with complete unloading between cycles. This allowed observation of any residual effects after each cycle. Specimens were loaded at a rate of 0.0333 inches per minute and unloaded at a rate of 0.0667 inches per minute to match static loading as closely
as possible. Variations of load steps were used for the bare specimens and floor system specimens. The bare system was loaded to its predicted construction load, service load, 1.25 service load, factored load, and failure load. The floor systems load cycles included their predicted construction load, a preload, service load, 1.25 service load, and failure load. The specific load cycles for each specimen are listed in Table 3.4 and Table 3.5.

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<th>DKP 1 Applied Moment (k-ft)</th>
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4 EXPERIMENTAL RESULTS

4.1 Introduction

This chapter presents representative results and the general behavior observed from the experimental testing program. Particular failure modes are also presented and discussed. Specific results for each test are given in Appendix B.

4.2 Behavior of Test Specimens

The general behavior exhibited by the specimens and representative results are presented in this section. Plots and figures accompany a discussion of the test data given for the deflection, strain, slip, and other observed behavior of the system.

4.2.1 Moment-Deflection

String potentiometers were placed under the steel plate as well as under the hollow core planks of the floor system specimens to record deflections. Recordings were taken at mid-span and at the quarter points. The load was monitored by a load cell mounted to the actuator. The moment produced by the four point loads was calculated and plotted against deflections. Figure 4.1 and Figure 4.2 show typical moment vs. deflection relationships at mid-span and quarter points for the precast girder and floor system specimens, respectively. Loading was performed in cycles up to failure, resulting in some residual deflection at zero
load for subsequent load steps. To enhance the moment vs. deflection relationship the envelope of moment-deflections was plotted for each specimen by removing the unloading portion of each load cycle with the exception of the failure load cycle. A comparison of all of the specimens’ moment-deflection envelopes is presented in Figure 4.3.
Figure 4.1 – Moment vs. Deflection for DKP 2
Figure 4.2 – Moment vs. Deflection for DKP 1a
4.2.2 Deflection Profiles of Test Specimens

The sequence of deflection at various loads steps is demonstrated in Figure 4.4 and Figure 4.5. The sequence of photos in Figure 4.4 shows the amount of deflection due to camber, construction load, and at failure for DKP 1. Figure 4.5 shows the amount of deflection at construction load and at failure for DKP 1a. As shown, the camber of both specimens flattened out under construction loads. Figure 4.6 and Figure 4.7 show the elevation profile
of deflections for DKP 1a. This deflection behavior was typical for all test specimens. The deflection values shown in Figure 4.6 are absolute and do not indicate the measured camber. Figure 4.7 shows the actual deflection when the initial camber is included.

Figure 4.4 – Sequence of Deflection for DKP 1
Figure 4.5 – Deflection of DKP 1a at Construction and Failure Loads
Figure 4.6 – Elevation Profile of Deflections for DKP 1a

Figure 4.7 – Elevation Profile of Deflections with Camber for DKP 1a
4.2.3 Steel and Concrete Strain

Strain readings were taken at mid-span and both quarter points. Strain gauges measured the strain along the inside and outside of the U-shaped steel plate and on the top and bottom mild steel reinforcing. Pi gauges were used to measure the strain along the top surface of concrete slab. The strain readings were then averaged at each level along the profile of the specimens. Figure 4.8 shows a sample moment vs. strain plot of the test specimens with positive strain values indicating members in tension while negative values indicate members in compression. Strain profiles were also plotted for each load cycle at mid-span and at the quarter points as shown in Figure 4.9. The strain profiles illustrate the linear relationship of the specimens throughout all load cycles. Profiles were drawn separately for the gauges placed on the U-shaped steel plate and gauges placed on the concrete interface and reinforcing steels. This was done in order to show the differential strain profiles resulting from the loss of shear connection. At lower load levels the strain profiles overlie demonstrating the assumption that plane sections remain plane. However, this assumption becomes invalid at larger loading cycles as the shear connection is lost. The disparity between the plate and concrete strain profiles is greater at the quarter points than at mid-span as the shear demand is greatest near the ends. From here the precast concrete member and steel plate act with partial to no interaction. The floor system specimens exhibited a slightly more complicated stain profile. Since the system was unpropped, strains are present within the precast member due to construction loads while zero strain is present in the slab at the time of the cast. To determine the total strain profile due to loading, strain values were
zeroed before the construction load and before the application of additional load cycles. The total strain for the precast member was calculated by summing these values while the strain for the slab only used readings from the additional load cycles. The total strain profile for the quarter points of a floor system specimen is shown in Figure 4.10. It shows the discontinuity between the slab and precast members and again shows the differential strain profiles of the plate and precast member due to loss of shear connection.
Figure 4.8 – Moment vs. Strain for DKP 1

Note: Pi gauge sensors lost due to failure
I – Construction Load Step (210 k-ft)  
II – Service Load Step (310 k-ft)  
III – 1.25 Service Load Step (388 k-ft)  
IV – Factored Load Step (476 k-ft)  
V – Failure Load Step (537 k-ft)

Figure 4.9 – Strain Profiles for DKP 1
4.2.4 Longitudinal Slip

The relative slip between the precast concrete beam and steel plate was measured for all specimens. Additional slip measurements for the floor system specimens were recorded between the steel plate and hollow core planks and calculated between the precast concrete beam and hollow core planks. DKP 1 was the only beam that did not have a significant amount of slip recorded with a maximum slip of approximately 0.025 inches. All other specimens recorded significant amounts of slip during testing and at ultimate. The most significant amount of slip recorded neared 1 inch during the final loading cycle of DKP 2 and is shown in Figure 4.11. Furthermore slip typically occurred most significantly on only one end of the specimens as a result of shear connection failure. The recorded slip between the steel and precast concrete and steel and hollow core planks was almost identical for all
specimens resulting in relatively no movement between the precast concrete member and hollow core planks. Figure 4.12 illustrates the observed moment-slip relationship for DKP 1a. The axis of slip is reduced to better illustrate the behavior at the onset of slip.

Figure 4.11 – Slip Between Steel Plate and Precast Concrete Beam for DKP 2
Figure 4.12 – Slip Plots for DKP 1a

Note: 0.75" maximum slip recorded

HC - Steel I
HC - Steel II
Concrete-Steel I
Concrete-Steel II
4.2.5 Effective Flange Width

The floor systems effective flange width was calculated in accordance to ACI 318-08 and was controlled by eight times the slab thickness for each overhang outside the web. The specimens were constructed with a width of 89 inches with the effective flange width calculated as 78 inches. It should be noted that a three inch slab was used in the analysis and included the top flute of the hollow core planks and two inches of the cast-in place slab. PI gauges were placed within the effective flange width on the centerline and 32.5 inches from the centerline or 12 inches from the edge of the slab. As shown in Figure 4.13, the effective flange width is constant for the preload but becomes concentrated along the centerline of the cross section at and beyond the service load. However, the strain readings from the outer PI gauges indicate a majority of strain is still present within the effective flange width even at failure. The compressive strain along the cross section for DKP 1a is shown in Figure 4.13. The disparity in MP PG I came at the onset of longitudinal shear cracking and reduction of the effective flange width. Effective flange width strain results for DKP 2a were unreliable due to erroneous PI gauge readings.
4.2.6 Transverse Hollow Core Splitting

Flexural splitting cracks formed along the bottom surface of the hollow planks as shown in Figure 4.14. They also were typically near the location of the load points, the areas of higher load concentration. The flexural splitting began as hairline cracks, but widened and propagated through the bottom flute and into the cores as the curvature increased as additional load was applied. However, no loss of strength was noticed due to these flexural cracks.
4.2.7 Creep, Shrinkage, and Temperature Effects

Additional observed behavior included creep and crack formation in the slab of the floor systems due to shrinkage and temperature effects. Hairline cracks formed in the slab prior to external loading along the keyways of the hollow core planks. Although the slab was reinforced, cracks propagated transverse from the edge to the center of the slab. Additional deflections due to creep of the system were also noticed during the slab curing period and prior to the addition of external loading. This resulted in deflections of up to one half of an inch.
4.3 **Observed Failure Behavior**

The primary mode of failure observed during the experimental program included concrete crushing and shear connection failure. Additional failure behavior observed included steel plate and mild reinforcing yielding, longitudinal shear cracking, and flexural shear cracking.

4.3.1 **Concrete Crushing**

ACI 318-08 assumes failure when the concrete strain at the extreme fiber exceeds 0.003 in/in. Both precast girder specimens exhibited concrete crushing at failure. PI gauge readings at the top concrete surface at mid-span exceeded 0.003 in/in for the DKP 1 with observable concrete crushing at failure as shown in Figure 4.15. Strain readings also exceeded the crushing strain for DKP 2 at mid-span with the concrete strain approaching crushing and observed at the third quarter point. Strain values indicated that concrete crushing did not occur at the extreme level of concrete for the precast beam or the composite slab for the floor system specimens. Although strain readings were not directly taken at the level of the precast member, a linear relationship drawn between the levels of the reinforcing was extended to the top of the precast member and did not indicate values in excess of the concrete crushing strain. PI gauges values were also taken at the critical level of the slab and did not indicate crushing of the concrete.
4.3.2 Steel Yielding

The tension reinforcement and steel plate yielded in both partially prestressed specimens. The steel was considered to have yielded beyond a strain of 0.002 in/in as per ACI 318-08. The bottom steel plate of DKP 1 yielded in tension at mid-span prior to the observed ultimate moment capacity of the girder. The reinforcing steel in compression exceeded 0.003 in/in and buckled at mid-span as shown in Figure 4.16. All of the tension reinforcement and the steel plate yielded for DKP 1a prior to the observed ultimate moment capacity of the floor system. The steel plate and tension reinforcement did not yield in DKP 2 or DKP 2a with the exception of the tension reinforcement in DKP 2a yielding past ultimate. It should be noted
that the measured strains for the tension reinforcement in both nonprestressed specimens did exceed 0.0018 in/in and would have likely yielded without slip and loss of shear connection.

![Yielded Compression Steel Reinforcement in DKP 1](image)

**Figure 4.16 – Yielded Compression Steel Reinforcement in DKP 1**

4.3.3 Shear Connection Failure

The slip was a result of the ductility and eventual failure of the shear connections. Headed shear studs 5/8” in diameter and 5 inches in length were utilized for the shear connection. The analysis considered the shear connection to provide full interaction between the steel and precast concrete beam. However, only DKP 1 did not have shear connection failure and as a result did not have any significant slip. Shear stud failure did occur in the remaining specimens. Shear connection failure directly resulted in the relative slip between the various components of the specimens. As noted previously significant slip generally occurred on one
end of the specimens and resulted from failure of shear studs. Once some studs failed the remaining studs became more vulnerable as the forces were redistributed. Since the longitudinal shear is transferred to the shear connectors through dowel action, the shear studs must resist shear and flexure forces. An embedded shear stud and localized concrete crushing was noticed and shown in Figure 4.17. A close up of the failure plane in the shank of the shear stud just above the weld collar and shear stud is also shown in Figure 4.17. A combination of localized concrete crushing, tension and shearing of the shear studs resulted in the failure of the shear connection and was the primary failure mode for the specimens.

Figure 4.17 – Localized Concrete Crushing and Shear Stud Failure for DKP 2
4.3.4 Longitudinal Shear Cracks

Longitudinal shear cracks were also observed for the floor system specimens. Transverse reinforcement of the specimens was provided by WWF in the slab and bent No. 4 rebar grouted in the keyways between adjacent hollow core planks. Although transverse reinforcement is typically designed for the transverse negative bending of the slab, it also resists longitudinal shear forces. Furthermore, in-situ concrete within the voids of the hollow core planks contributes to the longitudinal shear strength. However, block outs were placed flush with the surface of the voids to produce a worst case scenario and therefore did not contribute to the longitudinal shear strength of the specimens. Exceptions to this may have taken place as a result of the block out being pushed into the voids during casting of the slab as shown in Figure 4.18. The formation of longitudinal vertical shear crack occurred during the final or failure load cycle for DKP 1a and 2a. Two possible weak surfaces where the crack could have formed were at the interfaces of the precast beam and hollow core planks. However, it was determined to have occurred along the hollow core plank interface for both specimens as shown in Figure 4.18. This result was due to the combination of transverse negative bending of the slab and longitudinal shear forces. The formation of the crack directly affected the capacity of the specimens by reducing the effective slab width and lowering the neutral axis depth of the section. As a result, the specimens reverted to the strength of the rectangular portion of the composite beam.
Figure 4.18 – DKP 1a Longitudinal Shear Cracks
4.3.5 Flexural-Shear Cracks

Flexural-shear cracks were observed during testing for both precast girder specimens. Cracks propagated from the interface of the steel plate top flange returns and extended upwards towards the center of the girder. Figure 4.19 shows the propagation of cracking for DKP 2 near the third quarter point. The cracks become more vertical and flexural in nature as they approach the middle of the specimen and the location of failure. The addition of and confinement provided by the steel plate most likely helped inhibit the onset and propagation of flexural cracks.

Figure 4.19 – Flexural Cracks Along and at Failure Location of Girder Specimens
4.3.6 **Summary of Observed Failure Behavior**

All specimens failed due to a combination of the failure modes described in the previous sections with the primary mode of failures as concrete crushing and shear connection failure. A summary of the failure modes exhibited and the primary mode of failure are listed in Table 4.1.

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Where:

SY = steel yielding  
CC = concrete crushing  
FSC = flexural shear cracks  
SCF = shear connection failure  
LSC = longitudinal shear cracking  
✗ = observed failure behavior  
✗✗ = primary mode of failure
5 ANALYSIS

5.1 Introduction

This chapter presents the analysis and comparison of results obtained from the experimental and analytical investigations. Results from experimental tests were compared to the predicted moment-curvature response. Limit states are also examined and includes serviceability and strength limit states.

5.2 Moment-Curvature Analysis

A moment-curvature layered sectional analysis was modified and utilized to determine the behavior of each specimen based on their material properties, reinforcement details, and stress-strain constitutive models. Plots of representative moment-curvature relationship are given in Figure 5.1 and Figure 5.2. In both figures the depth of the neutral axis of the section is plotted on the secondary y-axis. Figure 5.1 shows the relationship for the partially prestressed girder (DKP 1) while Figure 5.2 shows the relationship for the nonprestressed floor system (DKP 2a). The precast girders stiffness’s were linear prior to failure while both floor systems demonstrated a two tiered stiffness relationship once the cast-in-place concrete cured and the system acted compositely. The initial slope of the floor system took into account the precast girder behavior from the construction load due to the hollow core planks and cast-in-place concrete slab. From the moment-curvature sectional analysis the load vs.
deflection behavior was determined using the moments of area theorem. The moment was plotted against deflection to easily compare experimental behavior to the predicted deflections. Figure 5.3 and Figure 5.4 show the moment-deflection relationships for the partially prestressed precast girder (DKP 1) and the nonprestressed floor system (DKP 2a), respectively. A detailed discussion of the moment curvature analysis is given in Appendix A.
Figure 5.1 – Moment-Curvature for Partially Prestressed Girder (DKP 1)

Figure 5.2 – Moment-Curvature for Nonprestressed Floor System (DKP 2a)
Figure 5.3 – Moment vs. Deflection for Partially Prestressed Girder (DKP 1)

Figure 5.4 – Moment vs. Deflection for Nonprestressed Floor System (DKP 2a)
5.2.1 Deflection

The measured mid-span moment-deflection response for each specimen was compared to its corresponding moment-deflection response from the moment curvature analyses. The predicted and experimental curves for the precast girder and floor system specimens are presented in Figure 5.5 and Figure 5.6, respectively. For simplicity, the predicted moment-deflections were zeroed and compared to the absolute deflections from the experimental program. Moment-deflection envelopes were also utilized instead of the full load cycles to easily compare the responses.

Figure 5.5 compares the moment-deflection behavior of the partially prestressed girder (DKP 1) and the nonprestressed girder (DKP 2). The initial stiffness of the experimental specimens closely follows the stiffness predicted by the moment-curvature analyses. The stiffness was slightly greater for DKP 1 than DKP 2 for both the predicted and experimental results as expected due to the addition of the prestressing steel reinforcement. The experimental results for DKP 1 closely matched the predicted strength, but did not fully match the predicted stiffness. This was a result of the relatively small amount of slip recorded and the residual displacement sustained from each load cycle. DKP 2 followed the same type of behavior until the moment exceeded 200 k-ft. At that point the slip increased significantly at the back end of the specimen as the load continued to be applied. The stiffness of the member began to decrease as the degree of shear connection decreased. As a result of the decreased
stiffness and longitudinal slip, the deflection of the specimens also exceeded their predicted values.

The initial behavior of the floor system specimens was virtually identical to the behavior of their girder specimen counterparts. The initial loading was applied with the tie down system at the third points. Since 4-point bending was used to simulate the construction load, the specimens were stressed to approximately 95% of the expected moment in order to accurately produce the moment response due to a uniformly distributed load. The final construction moment produced was approximately 224 k-ft for both specimens. The predicted moment-deflection response considered the transition of the stiffness of the precast girder to the floor system to be instantaneous and therefore did not account for any creep resulting from the construction load. This is shown in Figure 5.6 as the deflection increased with no additional applied load at the construction load. To compare the behavior and stiffness of both systems, the predicted response of the floor system was offset to account for the creep and to better match the initial values of the experimental results. The close relationship in the secondary stiffness of the floor systems is also shown in Figure 5.6. The experimental results for DKP 1a closely followed the predicted behavior until approximately 600 k-ft at which point considerable slip began to occur. The same behavior was observed for DKP 2a with considerable slip occurring just beyond a moment of 400 k-ft.
Figure 5.5 – Moment-Deflection Envelopes for Girder Specimens

Figure 5.6 – Moment Deflection Envelopes for Floor System Specimens
The experimental deflections at mid-span were also compared against the predicted deflections from the moment-curvature analysis and using the transformed cracked moment of inertia of the section. The deflection was calculated from the equation given by AISC Table 3-22a for four point loads when using the transformed cracked moment of inertia. The experimental and predicted deflections for the dead load or construction load at a moment of 224 k-ft are presented in Table 5.1. The ratio of the experimental to predicted deflections were all within 10% of the experimental deflection. There is not a clear trend on the whether the methods used for the predicted deflections were conservative or unconservative for all specimens, but they tended to under predict the deflections that were observed. This can be attributed to small amounts of slip and the loss of shear connection at the construction load.

Table 5.1 – Construction Load Deflection Comparison

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Experimental (in)</th>
<th>Moment Curvature (in)</th>
<th>Transformed Cracked Section (in)</th>
<th>$\Delta_{exp}/\Delta_{M-\phi}$</th>
<th>$\Delta_{exp}/\Delta_{tcs}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>DKP 1</td>
<td>1.89</td>
<td>1.74</td>
<td>1.89</td>
<td>1.09</td>
<td>1.00</td>
</tr>
<tr>
<td>DKP 2</td>
<td>2.11</td>
<td>2.14</td>
<td>1.95</td>
<td>0.98</td>
<td>1.08</td>
</tr>
<tr>
<td>DKP 1a</td>
<td>1.70</td>
<td>1.60</td>
<td>1.87</td>
<td>1.06</td>
<td>0.91</td>
</tr>
<tr>
<td>DKP 2a</td>
<td>2.13</td>
<td>2.00</td>
<td>1.94</td>
<td>1.07</td>
<td>1.10</td>
</tr>
</tbody>
</table>
5.2.2 Slip

As mentioned in the previous chapter, the loss of shear connection was a main element in the failure of all specimens. Although the loss of shear connection was not directly measured, the recorded slip gives great insight into the behavior of the specimens. All specimens closely followed their corresponding predicted moment-deflection response up until a considerable amount of slip was recorded. The predicted and experimental moment-deflections responses for DKP 2a are shown in Figure 5.7 and the moment-slip behavior is shown in Figure 5.8. Comparing the two graphs shows a direct correlation between the loss of stiffness and degree of slip encountered. The same correlation was exhibited by all specimens including DKP 1 which had slip on the magnitude of only two hundredths of an inch. The correlation is less obvious, but can be seen in Figure 5.9 and Figure 5.10.
Figure 5.7 – Mid-span Moment-Deflection for DKP 2a

Considerable amount of slip began to occur

Figure 5.8 – Moment-Slip for DKP 2a

Deviation from predicted stiffness

Note: 0.75” maximum slip recorded
Figure 5.9 – Mid-span Moment-Deflection for DKP 1

Figure 5.10 – Moment-Slip for DKP 1
5.2.3  **Strain**

From the moment-curvature analysis, a complete moment-strain response for any level of material or reinforcement was determined. The complete predicted and experimental DKP 1 moment-strain response for the top of the concrete slab, steel reinforcing, and steel plate bottom flange are shown in Figure 5.11. The strain for each layer followed its predicted strain values closely until failure. However, the strain in the concrete slab and compression reinforcing slightly exceeded their predicted strain near the service load. Figure 5.12 shows the moment-strain behavior of DKP 1a with the total strains values plotted. The experimental strain values again closely followed the predicted strain values. The behavior of the floor systems was quite different than that of the girder systems. The strain followed the same trend up to the construction load. At that point the concrete slab had zero strain, but the precast girder had strains due to the construction load. After the construction load, the girder acted compositely as a floor system, altering the strain behavior. The neutral axis rose towards the slab reversing the trend of strains in the precast girder. The mild steel reinforcing at the top of the girder that was originally in compression due to the construction load, dropped below the neutral axis of the floor system and tended towards positive, tension strain. Both DKP 2 and DKP 2a also followed the same behavior associated with their girder and floor system counterparts.
Figure 5.11 – Moment-Strain Envelopes for DKP 1

Note: Pi gauge sensors lost due to failure

Figure 5.12 – Moment-Strain Envelopes for DKP 1a

Zero strain in slab at construction load
Strain profiles can be generated or determined at any load level. For simplicity, strain profiles were taken for the predetermined load cycles at mid-span and the quarter points. The strain profiles for all load cycles for DKP 2 are shown in Figure 5.13. These plots show a linear strain profile for the section at low loads at mid-span, confirming the assumption that plane sections remain plane. However, this assumption breaks down at the quarter points at low loads and at mid-span at higher loads. The strain readings from the U-shaped steel plate are plotted separately from the concrete and rebar strain readings to show the effects from the loss of shear connection. As the section continued to lose connection, the two sections began to act more independently from one another. As the strain profiles suggest, both sections remained plane about their own axis. It should also be noted that since the deflections were the same for both sections, the curvatures of both sections were equivalent. This is shown in Figure 5.13 as well as Figure 5.14 with each strain profile having matching slopes, thus matching curvatures. Figure 5.14 breaks down the load steps separately and compares the experimental strain profiles at mid-span to the predicted moment-curvature strain profiles for an equivalent moment. The experimental strain profiles very closely match the predicted strain profiles until larger loads were encountered. Again, the loss of shear connection creates a discontinuity in the strain in the precast concrete and steel plate, although both have matching curvatures.
Figure 5.13 – Strain Profiles for DKP 2
Figure 5.14 – Experimental and Predicted Mid-span Strain Profiles for DKP 2
The behavior of the floor system was similar, but a little more complex with the addition of the slab in an unpropped system. The unpropped system caused a discontinuity to occur at the surface of the precast girder resulting in zero strain in the topping slab at the time of construction. The total strain due to external loading for DKP 2a is plotted against the depth of the section for all load steps and locations in Figure 5.15. The total strain is plotted for the construction load plus externally applied loads at each load step for the steel plate and the rebar. The total strain is plotted at for the top of the slab and is only due to externally applied loads at each load step. Additionally, the strain at the bottom of the slab is extrapolated using strain readings from the rebar due to externally applied loads. This resulted in the most accurate representation of the total strain in each component of the system at the various load steps. The experimental and predicted mid-span strain profiles are compared for each load cycle in Figure 5.16. As with the precast specimens, shear connection was lost resulting in differential strain profiles for the precast concrete and steel plate. The same curvature appeared to be maintained at all locations regardless of the loss of shear connection. The experimental strain profiles for DKP 2a followed the predicted strain profiles, but breaks down once shear connection becomes lost as shown in Figure 5.16.
Figure 5.15 – Strain Profiles for DKP 2a

- Quarter Point 1
- Midspan
- Quarter Point 3

- Construction Load (218 k-ft)
- Preload (350 k-ft)
- Service Load (555 k-ft)
- Failure Load (604 k-ft)
Figure 5.16 – Experimental and Predicted Mid-span Strain Profiles for DKP 2a
5.3 Limit States

The analysis of the system included the examination of different limit states. The first limit state checked was the serviceability limit state in regards to live load deflection. The next limit checked was the ultimate limit state for calculating the moment capacities of the systems. Strength limit states associated with additional design considerations and observed failure modes were also analyzed.

5.3.1 Serviceability Limit State

To satisfy the deflection serviceability limit state, deflections at service load levels must fall below limits set by building codes. The 2009 IBC limits the unfactored live load deflection to L/360 for floor systems. In the case for the test specimens, the deflection is limited by 0.9 inches. A live load of 100 psf was used to calculate the service load cycle during testing, but a live load reduction was calculated in accordance with ASCE 7-05 and is given by Equation 5.1.

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

Equation 5.1

Where:

\[ L \] = reduced design live load per square foot of area supported by the member

\[ L_o \] = unreduced design live load per square foot of area supported by the member
\[ K_{LL} = \text{live load element factor} \]
\[ A_T = \text{tributary area in ft}^2 \]

For the case presented, the unreduced live load of 100 psf was used based on Table 4-1 of ASCE 7-05 for public rooms in multi-residential buildings and lobbies, and first floor corridors in office buildings. The live load element factor, \( K_{LL} \), was taken as 2 based on Table 4-2 for an interior beam. The tributary area for the system was calculated to be 810 ft\(^2\), which satisfied the minimum requirement of \( K_{LL}A_T \) greater than 400 ft\(^2\). Based on this criteria the reduced live load factor was 0.377 resulting in a reduced live load of 66.3 psf. To determine if the deflection limit was satisfied, the deflection due to the live load was measured based off of the initial deflection due to the construction load. The corresponding moments produced by the reduced live loads was 170 k-ft. Table 5.2 shows the experimental and predicted deflections for the live load moment. The observed deflections for both floor specimens were well within the deflection limit of 0.9 in. They also corresponded well with both methods used to predict deflection. DKP 2a deflected more than expected due to slip and loss of shear connection close to the live load moment.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Experimental (in)</th>
<th>Moment Curvature (in)</th>
<th>Transformed Cracked Section (in)</th>
<th>( \Delta_{exp}/\Delta_{M-\Phi} )</th>
<th>( \Delta_{exp}/\Delta_{tcs} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>DKP 1a</td>
<td>0.58</td>
<td>0.61</td>
<td>0.53</td>
<td>0.95</td>
<td>1.08</td>
</tr>
<tr>
<td>DKP 2a</td>
<td>0.71</td>
<td>0.64</td>
<td>0.55</td>
<td>1.11</td>
<td>1.29</td>
</tr>
</tbody>
</table>
5.3.2 **Design Limit States**

The limit states examined for design include flexural, vertical shear, horizontal shear, and longitudinal shear strength of the system.

5.3.2.1 **Flexural Strength**

In order to satisfy the ultimate strength limit state, the structure must be able to withstand forces or stresses that it is designed for. As a factor of safety, loads are typically magnified while the capacity of the structure is reduced to meet this criterion. A strength reduction factor ($\phi$) is applied to account for variations in material properties and dimensions, imprecision in design equations, to reflect the degree of ductility and required reliability of the member for the loading being considered, and to reflect the importance of the member (ACI 318-08). Depending on the type of failure mode, a strength reduction factor is used ranging from 0.65 to 0.90. According to ACI 318-08 (10.3), a section is considered compression controlled if the concrete in compression reaches the assumed strain limit of 0.003 prior to the net tensile strain in the extreme tension steel reaching the strain limit of 0.002. Sections are considered tension controlled if the net tensile strain in the extreme tension steel reaches a strain of 0.005 before the concrete in compression reaches the assumed strain limit of 0.003. Furthermore, sections within these limits fall into the transition range. From Figure 5.17 the strength reduction factor can be determined for the type of failures considered.
For the test program all specimens fell within the compression controlled zone of failure. This resulted in a strength reduction factor of 0.65 for all specimens. DKP 1 and DKP 2 both reached the assumed strain limit of 0.003 prior to any steel reinforcement reaching the strain limit of 0.002. However, DKP 1a and DKP 2a did not reach the assumed strain limit of 0.003 at the extreme fiber of the precast member or concrete slab because of shear connection failure.

The nominal flexural strength of the section is determined in accordance with AISC (Section 11.1) and ACI 318-08 (10.2). The AISC specification uses either the strain compatibility approach or the plastic stress distribution method to determine the nominal strength of composite sections. To simplify the design, the plastic stress distribution method was recommended. This method assumes full interaction is achieved and a linear strain profile throughout the depth of the section. It also assumes that the concrete reaches its crushing
strain of 0.003 and that all the steel exceeds its yield strain. An equivalent rectangular concrete stress block is used to model the nonlinear stress-strain relationship of the concrete and the concrete stress is taken to be $0.85f'_c$ across an equivalent compression zone to a depth $a = \beta_1 c$ from the extreme compression fiber as shown in Figure 5.18. The factor $\beta_1$ is 0.85 for concrete compressive strengths of 4000 psi or less, is reduced by 0.05 for each 1000 psi in excess of 4000 psi, and is taken as a minimum of 0.65 for all other concrete compressive strengths. The concrete compressive force shown in Equation 5.2 is balanced with the tension force from the steel reinforcing given by Equation 5.3. To maintain force equilibrium the concrete compressive force is set equal to the steel tension forces and used to solve for the depth of the stress block as shown in Equation 5.4.

$$C = 0.85f'_c \beta_1 cb_{eff}$$  

Equation 5.2

Where:

$f'_c$ = concrete compressive strength, psi

$\beta_1$ = factor for accounting for concrete compressive strength
\( c = \) neutral axis depth, in

\( b_{\text{eff}} = \) effective width, in

\[ T_i = A_{s_i}f_{y_i} \quad \text{Equation 5.3} \]

\[ a = \frac{\Sigma T_i}{0.85f'c'b_{\text{eff}}} \quad \text{Equation 5.4} \]

Where:

\( A_{s_i} = \) steel cross-sectional area, in \(^2\)

\( f_{y_i} = \) steel yield strength, ksi

\( a = \) depth of compression zone, in

The nominal moment capacity is calculated by summing the moments about the depth of the stress block as shown in Equation 5.5:

\[ M_n = C \left( \frac{a}{2} \right) + \sum_{i=1}^{n} T_i(d_i - a) \quad \text{Equation 5.5} \]

Where:

\( d_i = \) distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in

The ultimate flexural capacity of each test specimen, using stress block analysis and moment curvature analysis, is given and compared in Table 5.3. The results of the stress block
analysis and the moment curvature analysis are very closely correlated with the stress block analysis being slightly conservative for all specimens.

### Table 5.3 – Flexural Capacity

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Stress Block Analysis (k-ft)</th>
<th>Moment Curvature Analysis (k-ft)</th>
<th>Stress Block/Moment Curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>DKP 1</td>
<td>530</td>
<td>557</td>
<td>0.95</td>
</tr>
<tr>
<td>DKP 2</td>
<td>501</td>
<td>517</td>
<td>0.97</td>
</tr>
<tr>
<td>DKP 1a</td>
<td>1130</td>
<td>1135</td>
<td>1.00</td>
</tr>
<tr>
<td>DKP 2a</td>
<td>967</td>
<td>969</td>
<td>1.00</td>
</tr>
</tbody>
</table>

#### 5.3.2.2 Shear Strength

AISC (Section I3-1b) uses a conservative approach when considering the shear strength of a composite beam by assigning all the shear strength to the steel section web. This neglects any shear contribution from the concrete slab, but does not account for the shear strength contribution from the reinforced beam in the case of the specimens under consideration. However, shear strength provisions (AISC Section I2.1d) for composite axial members take into account the steel section and steel reinforcement or the concrete and steel reinforcement. It suggests that superimposing the shear strengths of the steel section and reinforced concrete is logical, but little research is available to verify this assumption. Based on these provisions, the shear strength for the specimens under consideration should be taken by the steel section alone or that of the reinforced concrete beam. The shear strength of the steel section is given in AISC (Section G2.1) by Equation 5.6.
\[ V = 0.6 f_y A_w C_v \]  
\text{Equation 5.6}

Where:

\[ A_w = \text{web area, in}^2 \]
\[ C_v = \text{web shear coefficient} \]

When the shear strength of the steel section alone is inadequate, the shear strength design is based on the reinforced concrete section. The nominal shear strength \((V_n)\) specified by the ACI code (11.1) is given by the combination of the concrete shear strength \((V_c)\) and steel reinforcement strength \((V_s)\) as in Equation 5.7.

\[ V_n = V_c + V_s \]  
\text{Equation 5.7}

The concrete and steel reinforcement contributions to the shear strength are calculated by Equation 5.8 and Equation 5.9, respectively.

\[ V_c = 2 \sqrt{f'_c b_w d} \]  
\text{Equation 5.8}
\[ V_s = \frac{A_v f_y d}{s} \]  
\text{Equation 5.9}

Where:

\[ b_w = \text{web width, in} \]
\[ A_v = \text{area of shear reinforcement spacing, in} \]
\[ s = \text{center-to-center spacing of transverse reinforcement, in} \]
Minimum spacing requirements are given as the lower value of \(d/2\) for nonprestressed members, \(0.75h\) for prestressed members, and 24 inches. Additional requirements are given where \(V_s\) exceeds \(2V_c\) by dividing the previous requirement in half.

The shear capacity provided by the webs of the steel plate and reinforced concrete section are given in Table 5.4. The shear capacity of each section alone is insufficient to develop the full flexural capacity of the section and should be redesigned based on the approach used, but not based on the results of the experimental tests. It appears that contributions from both types of shear reinforcement can be added to determine the shear capacity of the section, but was not the main focus of this research. Additionally, the shear contribution from the pretressing was ignored when determining the results for the reinforced concrete section capacity.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Plate Web Shear Capacity (k)</th>
<th>Reinforced Concrete Section Capacity (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DKP 1</td>
<td>56.3</td>
<td>64.2</td>
</tr>
<tr>
<td>DKP 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DKP 1a</td>
<td></td>
<td>76.8</td>
</tr>
<tr>
<td>DKP 2a</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5.3.2.3  *Horizontal Shear Strength*

According to AISC specification the entire horizontal shear at the interface between the steel beam and concrete slab is assumed to be transferred by shear connectors. 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 (Equation 5.10), steel tensile yielding (Equation 5.11), or strength of the shear connectors (Equation 5.12).

\[
V' = 0.85 f_c' A_c \\
V' = f_y A_s \\
V' = \Sigma Q_n
\]

Equation 5.10
Equation 5.11
Equation 5.12

Where:

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

\[ A_s = \text{area of steel cross section, in}^2 \]

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

The nominal strength of one stud shear connector embedded in solid concrete or in a composite slab is then calculated by Equation 5.13.

\[
Q_n = 0.5 A_{sc} \sqrt{f_c'E_c} \leq R_y R_p A_{sc} F_u
\]

Equation 5.13

Where:

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

\[ E_c = \text{modulus of elasticity of concrete} = w_c^{1.5} \sqrt{f_c'}, \text{ksi} \]
\( F_u \) = specified minimum tensile strength of a shear stud connector, ksi

\( R_g \) = coefficient to account for group effect

\( R_p \) = position effect factor for shear studs

\( w_c \) = weight of concrete per unit volume (90 ≤ \( w_c \) ≤ 155), lb/ft³

However, for the case of concrete encased beams, the entire horizontal shear is not assumed to be transferred by shear connectors. The AISC specification requires that the nominal flexural strength of concrete-encased and filled members to be determined by one of three methods.

1. The superposition of elastic stresses on the composite section for the limit state of yielding (yield moment)

2. The plastic stress distribution on the steel section alone for the limit state of yielding (plastic moment)

3. The plastic stress distribution on the composite section or from the strain compatibility method if shear connectors are provided and the concrete meets specified strength requirements.

Based on the plastic stress distribution on the composite section the nominal flexural strength is the same as given in Equation 5.5. The required number of shear connectors for half the span is thus given by the horizontal shear force given by the minimum of Equation 5.10 or Equation 5.11 divided by the nominal strength of one connector.
AISC also specifies spacing requirements for shear connector placement. The minimum center-to-center spacing of shear connectors is given as six diameters along the longitudinal axis and four diameters transverse to the longitudinal axis of the supporting composite beam. Maximum center-to-center spacing’s are not to exceed eight times the slab thickness or 36 inches. Additionally, the diameter of the studs is not to exceed 2.5 times the thickness of the supporting composite member.

The horizontal shear demand and the capacity provided by the shear studs for each specimen are given in Table 5.5. Clearly the provided shear stud size and layout was insufficient to produce the ultimate flexural capacity of the section, and therefore needs to be redesigned. The provided and required spacing of 5/8” Dia. shear studs needed to produce the ultimate flexural capacity of the section are also given in Table 5.5. Due to the geometry of the section and the depth of the shear plane at which shear studs are placed, Equation 5.11 will always control.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Provided Shear Stud Capacity (k)</th>
<th>Provided Spacing</th>
<th>Horizontal Shear Demand (k)</th>
<th>Required Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>All Specimens</td>
<td>281</td>
<td>12” Stagger Spacing</td>
<td>656</td>
<td>Double Line at 10” Spacing</td>
</tr>
</tbody>
</table>
5.3.2.4  \textit{Longitudinal Shear Strength}

One limit state not initially considered in design was that of longitudinal shear crack formation along the cast-in-place concrete and hollow core plank interface. In the case where the shear strength of the slab may govern the design, AISC (Comm. 13.1) recommends that transverse reinforcement be at least 0.002 times the concrete area in the longitudinal direction and uniformly distributed. ACI (11.6) provides a conservative shear-friction design method to determine the nominal shear transfer strength given by Equation 5.14.

\[ V_r = A_{vf} f_y \mu \quad \text{Equation 5.14} \]

Where:

- \( A_{vf} \) = area of shear friction reinforcement, in\(^2\)
- \( \mu \) = coefficient of friction, 1.4 for concrete placed monolithically

This assumes that cracks will form along the shear plane and that the shear forces are resisted by the friction between the faces, resistance to the shearing off of protrusions on the crack faces, and dowel action of the reinforcement crossing the crack. The total horizontal shear force to be transferred at the critical shear planes is given by the total horizontal shear demand minus the contribution of the concrete force within the cracked planes (Equation 5.15).

\[ V_{uh} = V_{nh} - 0.85 f'_c A_{cs} \quad \text{Equation 5.15} \]
Where:

\[ A_{sc} = \text{area of concrete within shear planes, in}^2 \]

The total area of shear reinforcement required in one plane to resist the longitudinal shear forces is given by combining the above equations in Equation 5.16.

\[ A_{vf} = \frac{1}{2} \frac{V_{uh}}{f_y \mu} \]  

Equation 5.16

This design method is based a composite section with a concrete slab and thus the topping slab for the floors system is the only section under consideration. The contact surface between the cast-in-place concrete and hollow core planks, as well as the flow of concrete into the voids, may also need to be taken into consideration when determining the nominal longitudinal shear strength of the section. The critical longitudinal shear plane is vertical along the interface of the cast-in-place concrete and hollow core planks interface. Some transverse reinforcement was provided by WWF in the slab and by bent bars in the keyways between hollow core planks, but was not sufficient to prevent longitudinal shear cracking. The longitudinal shear capacity of the floor system specimens are presented in Table 5.6.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Longitudinal Shear Demand (k)</th>
<th>Longitudinal Shear Capacity (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DKP 1a</td>
<td>1263</td>
<td>740</td>
</tr>
<tr>
<td>DKP 2a</td>
<td>1094</td>
<td>689</td>
</tr>
</tbody>
</table>

Table 5.6 – Longitudinal Shear between Maximum and Zero Moments
The longitudinal shear demand was computed using three different methods as presented by Kamel (1996). These methods included the working stress method (Equation 5.17), code method (Equation 5.18), and equilibrium method (Equation 5.19). Each of these methods gave comparative results for the interface shear stresses along the length of the specimens. The longitudinal shear force was then calculated by multiplying by the width and span between maximum and zero moment. It is recommended to use the code method which is presented in ACI-308-95 as an alternate design method. The longitudinal shear capacity of the floor system specimens are also presented in Table 5.6.

\[ v = \frac{VQ}{I_{tc}b} \]  

Equation 5.17

Where:

- \( V \) = vertical shear force at the location under consideration, lb
- \( Q \) = first moment of area of the position above the level under consideration, in\(^3\)
- \( I_{tc} \) = transformed cracked moment of inertia of the section, in\(^4\)
- \( b \) = width of the web at the level under consideration, in

\[ v = \frac{V_n}{d_e b} \]  

Equation 5.18

Where:

- \( V_n \) = nominal vertical shear force, lb
\[ d_e = \text{distance between the centroid of the compression and tension forces, in} \]

\[ v = \frac{C}{bl} \quad \text{Equation 5.19} \]

Where:

\[ C = \text{compressive force between point of maximum and zero moment, lb} \]

\[ l = \text{length between point of maximum and zero moment, lb} \]

5.3.2.5 Design Limit State Comparison

To compare the experimental results to the design limit states the moment capacity produced by each limit state was evaluated and is shown in Table 5.7. The predicted strengths are compared to the observed experimental strength or to the maximum applied moment as appropriate. The predicted failure mode, as predicted by evaluating all the design limit states, was horizontal shear failure with the exception of DKP 1 failing in flexure due to concrete crushing. The method used to predict failure due to horizontal shear is conservative with additional horizontal shear resistance taken by the system, but is still the limiting design limit state. The design vertical shear capacity was also exceeded in two cases, but did not cause failure of the systems. This again results in a conservative method for design of vertical shear. Longitudinal shear cracks formed in both floor system specimens, but occurred during the failure load stage and after shear stud failure occurred. Beyond failure of the shear studs it is difficult to accurately compare the other limit states to the method of their design. One result, DKP 1, however did show good correlation with failure due to flexural.
Both the stress block analysis and moment curvature analysis of DKP 1 closely predicted the ultimate moment capacity, with horizontal shear failure not occurring.

**Table 5.7 – Design Limit State Moment Capacity Comparisons**

<table>
<thead>
<tr>
<th>Test ID</th>
<th>DKP 1</th>
<th>DKP 2</th>
<th>DKP 1a</th>
<th>DKP 2a</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Observed Moment Capacity (k-ft)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( M_{ult} )</td>
<td>537</td>
<td>398</td>
<td>916</td>
<td>604</td>
</tr>
<tr>
<td>( M_{vs,obs} )</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( M_{hs,obs} )</td>
<td>-</td>
<td>252</td>
<td>891</td>
<td>439</td>
</tr>
<tr>
<td>( M_{ls,obs} )</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>Design Limit States (k-ft)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( M_{m-\phi} )</td>
<td>557</td>
<td>517</td>
<td>1135</td>
<td>969</td>
</tr>
<tr>
<td>( M_{sb} )</td>
<td>530</td>
<td>501</td>
<td>1130</td>
<td>967</td>
</tr>
<tr>
<td>( M_{vs} )</td>
<td>520</td>
<td>520</td>
<td>622</td>
<td>622</td>
</tr>
<tr>
<td>( M_{hs} )</td>
<td>359</td>
<td>250</td>
<td>438</td>
<td>294</td>
</tr>
<tr>
<td>( M_{ls} )</td>
<td>-</td>
<td>-</td>
<td>665</td>
<td>634</td>
</tr>
<tr>
<td><strong>Design Ratios</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( M_{m-\phi}/M_{ult} )</td>
<td>1.04</td>
<td>1.30</td>
<td>1.24</td>
<td>1.60</td>
</tr>
<tr>
<td>( M_{sb}/M_{ult} )</td>
<td>0.99</td>
<td>1.26</td>
<td>1.23</td>
<td>1.60</td>
</tr>
<tr>
<td>( M_{vs}/M_{ult} )</td>
<td>0.97</td>
<td>1.31</td>
<td>0.68</td>
<td>1.03</td>
</tr>
<tr>
<td>( M_{hs}/M_{hs,obs} )</td>
<td>0.67</td>
<td><strong>0.99</strong></td>
<td><strong>0.49</strong></td>
<td><strong>0.67</strong></td>
</tr>
<tr>
<td>( M_{ls}/M_{ult} )</td>
<td>-</td>
<td>-</td>
<td>0.73</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Notes:  
1. Dash “-” indicates that a parameter was not applicable  
2. Shaded cell indicated an observed failure mode  
3. **Bold italic** values indicate the predicted failure mode from design limit states  
4. Design ratios \( \leq 1.0 \) are conservative
6 CONCLUSIONS

6.1 Summary

The need to increase span lengths and decrease structural depths of buildings has led to modifications to the conventional composite floor system in recent years. The development of the Diversakore® Versa:T™ floor system is no exception. Previous research was performed at both Georgia Tech University and North Carolina State University on a cast-in-place shallow floor framing system. With the goal to increase speed and ease of construction a precast system has been developed. In order to determine the behavior of the innovative precast composite shallow floor framing system an experimental and analytical investigation was conducted.

The experimental investigation consisted of four full scale tests loaded in flexure with 4 point loads. Testing was performed for two precast girder and two floor system specimens. Additionally, each test configuration consisted of one prestressed and nonprestressed precast member. Each specimen was loaded to ultimate with observed failure modes noted. The failure modes included concrete crushing, steel yielding, longitudinal shear cracks, shear connection failure, and flexural shear cracks. The results from the experimental investigation were compared with the predicted response from a moment-curvature layered sectional analysis. Based on the results from the study guidelines for design and construction of the systems were developed.
6.2 Conclusions and Recommendations

Several of the results from this investigation were consistent with those found by previous research. The following conclusions and recommendations were found for the systems presented in this research.

1. The reinforcement ratio resulted in brittle failures for both the precast girders and floor systems. This must be accounted for in design with the applicable reduction factored applied. A reduction in steel area may produce more ductile behavior and result in a tension controlled failure. It should also be noted that full composite action was not achieved and consequently did not utilize the steel reinforcing efficiently.

2. The addition of prestressing steel did not significantly increase the initial stiffness, but did increase the ultimate capacity of the system. Under the construction load both prestressed and nonprestressed systems performed as expected with the camber in the girder leveling out. A comparison on the cost effectiveness of prestressing the girders should be performed to warrant its use for cambering the girders. To improve the comparison between prestressed and nonprestressed systems, tests should be performed where the calculated ultimate moment capacities are equivalent.
3. The amount of deflection increased due to the sustained construction load. This occurred during the concrete slab curing period and before any external loading was applied. Additional attention is needed to examine the creep of the system.

4. Adequate shear connection was provided by the headed shear studs for the service loads considered in this study with the exception to the non prestressed composite girder. However, the shear connection was inadequate to produce the calculated ultimate moment capacity of the system with the exception of the prestressed composite girder. Design for shear connector strength should be considered to obtain the calculated ultimate capacity of the section in accordance with the AISC guidelines presented in this study.

5. The failure of the floor systems was due to the formation of longitudinal shear cracks coupled with the loss of shear connection. The amount of transverse reinforcement was inadequate to resist the formation and opening of the cracks along the cast-in-place concrete and hollow core plank interface. The shear transfer strength of the slab should be considered. The ACI shear friction design method is presented to determine the minimum amount of transverse reinforcement necessary to resist longitudinal shear cracks. Given that the voids in the hollow core planks were blocked out for this study, additional attention should be given to the interactions between the hollow core planks or the precast member and the cast-in-place concrete.
6. The moment-curvature layered sectional analysis accurately predicted the initial response of the structural system. Deviation from the predicted stiffness appeared to occur at the onset of slip at the concrete-steel interface. Special consideration is needed to account for partial interaction due to the loss of shear connection and the reduction of the effective width due to longitudinal shear cracking, but is not presented in this study.

6.3 Future Work

As part of the ongoing program at NCSU, a cast-in-place system with concrete allowed to flow into the hollow core plank voids has already been conducted. A vertical shear test has also been performed with the same system. The analyses of the results from these tests are currently underway. Testing is planned to investigate the behavior of the system at the joint location and due to negative bending followed by a three span continuous test.

As a continuation of the recommendations mentioned previously, additional research is needed to determine all the behavior relevant to the precast composite shallow floor framing system. Potential research could include, but is not limited to:

1. Full scale tests with the design limit states of horizontal and longitudinal shear demand accounted for.

2. Push out tests to quantify the shear connection strength.

3. Modify analytical model to account for partial interaction.
4. Investigate the effects of creep with full scale tests.

5. Parametric study to investigate changes to reinforcement ratio, shear connections, transverse reinforcement, etc.

Additional research topics include tests with varying slab systems such as ribbed metal decking. There is also a desire to test the system with hollow core planks without a structural topping slab.
7 REFERENCES


Lindsey, S. D., Leon, R. T., & Kim, D (2002). Flexural Strength Tests on Three Diversakore Beams. The Georgia Institute of Technology. Atlanta, GA.


APPENDIX A: MOMENT CURVATURE ANALYSIS

A.1 Introduction

A moment-curvature layered sectional analysis procedure was utilized to determine the behavior of each specimen based on their material properties, reinforcement details, and stress-strain constitutive models. The moment curvature response was determined such that both force equilibrium and strain compatibility was satisfied. The following assumptions were also applied in the analysis:

1. No slip occurred between sections.
2. Full interaction was achieved.
3. Plane sections remained plane.
4. Tensile strength of concrete was ignored.
5. Ultimate failure occurred when the extreme compression fiber of concrete reached a strain of 0.003 in/in.
6. No other modes of failure were considered.

A.2 Constitutive Models

Constitutive models were used to model the behavior of the concrete and steel materials. To accurately model the stress-strain response of the different reinforcing steels, the Ramberg-Osgood model was utilized. The stress strain relationship is expressed in Equation A.1 and
shown in Figure A.1 for the various reinforcing steels with factors approximated for each material.

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

Equation A.1

Where:

\( f \) = stress caused by strain \( \varepsilon_{pf} \)

\( E_p \) = tangent stiffness when \( \varepsilon_{pf} \) equals 0

\( A \) = factor associated with the slope of the post yield response

\( B \) = factor associated with the location of first yield

\( C \) = transition curve factor from elastic behavior to post yield response

Figure A.1 – Ramberg Osgood Stress-Strain Relationship for Various Reinforcing Steels
The stress-strain relationship for the precast members, hollow core planks, and cast-in-place concrete was determined using Popovics equation modified by Thornfeldt, Tomaszewicz, and Jensen as shown in Equation A.2 (Collins 1997). Figure A.2 shows the stress-strain relationship for each concrete material.

\[
\frac{f_c}{f'_c} = \frac{n(\varepsilon_{cf} / \varepsilon'_c)}{n - 1 + (\varepsilon_{cf} / \varepsilon'_c)^{nk}} \quad \text{Equation A.2}
\]

Where:

- \( f_c \) = stress caused by strain \( \varepsilon_{pf} \)
- \( f'_c \) = peak stress observed in cylinder test
- \( \varepsilon'_c \) = strain when \( f_c \) reaches \( f'_c \)
- \( n \) = curve fitting factor
- \( k \) = factor to increase the post peak decay

To utilize this relationship several other properties and factors were determined. The calculation for the modulus of elasticity of the concrete is given in Equation A.3.

\[
\begin{align*}
E_c &= w_c 33 \sqrt{f'_c} & f'_c \leq 6000 \text{ psi} \\
E_c &= 40,000 \sqrt{f'_c} + 1,000,000 & f'_c > 6000 \text{ psi}
\end{align*} \quad \text{Equation A.3}
\]

Where:

- \( w_c \) = unit weight of concrete lb/ft^3
The final factors were determined by Equation A.4, Equation A.5, and Equation A.6.

\[ n = 0.8 + \frac{f'_c}{2,500} \]  
\[ k = 1.0 \quad \varepsilon_{cf} \leq \varepsilon'_c \]  
\[ k = 0.67 + \frac{f'_c}{9,000} \quad \varepsilon_{cf} > \varepsilon'_c \]  
\[ \varepsilon'_c = \frac{f'_c}{E_c} \frac{n}{n - 1} \]

Equation A.4  
Equation A.5  
Equation A.6

Figure A.2 – Stress-Strain Relationship for Concrete with Modified Popovics Equation

A.3 Procedure

The layered sectional analysis was based on a series of selected strain values for the extreme concrete fiber in compression. The strain value was increased beyond the assumed concrete
crushing strain of 0.003 in/in. From the top strain value ($\varepsilon_{\text{top}}$) and an assumed neutral axis depth (c) the strain was determined at any level of the section (d) using a linear relationship calculated in Equation A.7. The linear strain distribution is shown in Figure A.3.

$$\varepsilon_d = \left(\frac{c - d}{c}\right)\varepsilon_{\text{top}}$$

Equation A.7

Several sections were created based on their geometry and material properties and then divided into several layers. For each level of strain at the extreme compression fiber ($\varepsilon_{\text{top}}$) the corresponding strain was calculated for each layer of every section. The stress distribution was determined for each layer by inputting the calculated strains into their appropriate constitutive models. The force was then calculated by integrating the width (b) and stress ($f(x)$) over the depth (d) of each layer as shown in Equation A.8. The integration was approximated using Simpson’s rule. The force contribution from each section was summed and force equilibrium was satisfied by iterating the depth of the neutral axis until the sum of the forces equaled zero. The moment resistance contribution from each section was
determined by multiplying Equation A.8 by the moment arm (y), the distance from the centroid of each section to the neutral axis, as shown in Equation A.9.

\[ F = \int_{d_1}^{d_2} b f(x)dx \]  
Equation A.8

\[ M = \int_{d_1}^{d_2} yb f(x)dx \]  
Equation A.9

The curvature was determined by Equation A.10.

\[ \phi = \frac{\varepsilon_{\text{top}}}{c} \]  
Equation A.10

The process was repeated, for increasing values of \( \varepsilon_{\text{top}} \). Plots of the moment-curvature relationship for each specimen and neutral axis depth were generated and are shown in Figure A.5, Figure A.7, Figure A.9, and Figure A.11.

The predicted moment-deflection response was determined for each specimen using the predicted moment-curvature relationship and moment of area theorem. To determine the deflection, the specimen was broken into several segments along its length. The moment was calculated at several locations along its length based on the four point loads scheme utilized in the experimental program. The corresponding curvature from each calculated moment was determined and the area for each segment along the specimen was calculated. The moment arm was calculated from the support to the centroid of each area and multiplied by
the area to determine the deflection. The total deflection at mid-span was determined by summing the deflections from each area as shown in Equation A.11. This was done at multiple load steps to develop the full moment-deflection curve.

Figure A.4 represents the moment-curvature relationship used to determine the deflection at mid-span due to four point loads. Plots of the moment-deflection relationship for each specimen were generated and are shown in Figure A.6, Figure A.8, Figure A.10, and Figure A.12.

\[ \Delta_{total} = \sum_{i=1}^{n} \left( \frac{\Phi_i + \Phi_{i+1}}{2} \right) (x_{i+1} - x_1)(arm)_i \]  

Equation A.11
Figure A.4 – Moment-Curvature Relationship to Determine Deflection at Mid-span
Figure A.5 – DKP 1 Moment-Curvature Relationship

Figure A.6 – DKP 1 Moment-Deflection Relationship
Figure A.7 – DKP 2 Moment-Curvature Relationship

Figure A.8 – DKP 2 Moment-Deflection Relationship
Figure A.9 – DKP 1a Moment-Curvature Relationship

Figure A.10 – DKP 1a Moment-Deflection Relationship
Figure A.11 – DKP 2a Moment-Curvature Relationship

Figure A.12 – DKP 2a Moment-Deflection Relationship
APPENDIX B: EXPERIMENTAL INVESTIGATION TEST DATA
**B.1 DKP 1**

**Flexural Test Summary Sheet**

**Test ID:** DKP 1  
**Test Date:** 5/20/2009  
**Test Location:** Constructed Facilities Laboratory (CFL), North Carolina State University  
**Tested By:** Adam Amortmont, E.I. and Meade Willis, E.I.  
**Sponsor:** RB̈C

**Test Parameters**

**Configuration:** Simply supported with four equal loads at the 1/5 points.  
**Span:** 27 ft  
**Loading Rate:** 0.033 in/min  
**Load Cycles:**  
- Construction (210 k-ft), Service (310 k-ft), 1.25 Service (388 k-ft),  
- Factored (476 k-ft), Failure (537 k-ft)

**Steel U-Beam**

- **Height:** 4 in. (nominal)  
- **Thickness:** 0.25 in. (nominal)  
- **Width:** 30 in. (nominal)  
- **Length:** 30 ft (nominal)  
- **Steel Grade:** ASTM A572  
- **Yield Strength:** 50 ksi (nominal)  
- **Tensile Strength:** 62.5 ksi (measured)  
- **Yield Strength:** 65 ksi (nominal)  
- **Tensile Strength:** 74.0 ksi (measured)

**Reinforcement**

- **Tension Reinforcement:** 5 - No. 9 bars  
- **Compression Reinforcement:** 4 - No. 8 bars  
- **Prestressing Reinforcement:** 6 - 1/2 in. dia. 7-wire Prestressing Strands  
- **Shear Studs:** 5/8 in. dia. by 5 in. long at 12 in. stagger spacing

**Precast Girder**

- **Web Depth:** 7.25 in.  
- **Web Width:** 22 in.  
- **Bottom Flange Depth:** 4 in.  
- **Bottom Flange Width:** 30 in.  
- **Length:** 30 ft  
- **Concrete Compressive Strength:** 7400 psi

**Test Results**

- **Maximum Applied Moment:** 537 k-ft  
- **Maximum Deflection Values:** Construction (1.7 in.), Service (2.7 in.), 1.25 Service (3.6 in.),  
  - Factored (4.8 in.), Failure (7.1 in.)  
- **Failure Description:** Concrete Crushing, Flexural Shear Cracking, Yielding of the U-Shaped Steel Plate, and Yielding of Steel Reinforcement

**Test Observations**

- No visible signs of distress through 1.25 service load (388 k-ft)  
- Formation of flexural shear cracks at factored load (476 k-ft)  
- Concrete crushing observed at midpoint prior to failure  
- Sudden failure at midpoint due to concrete crushing (537 k-ft)  
- No visible signs of slip between u-shaped steel plate and concrete
Figure B.1 – DKP 1 Applied Moment vs. Deflection
Note: pie gauge sensors lost due to failure
I – Construction Load Step (210 k-ft)  
II – Service Load Step (310 k-ft)  
III – 1.25 Service Load Step (388 k-ft)  
IV – Factored Load Step (476 k-ft)  
V – Failure Load Step (537 k-ft)

Figure B.3 – DKP 1 Applied Strain Profiles
Figure B.4 – DKP 1 Applied Moment vs. End Slip
Figure B.5 – DKP 1 Applied Moment vs. Deflection Envelope Comparison
Figure B.6 – DKP 1 Strain Profile Comparisons

1.25 Service Load (388 k-ft)

Factored Load (476 k-ft)

Failure Load (537 k-ft)
a) Global view of beam at failure with concrete crushing at the midpoint

b) Concrete crushing at midpoint just after failure

c) Concrete crushing at midpoint from other side

Figure B.7 – DKP 1 Failure Photos
B.2 DKP 2

Flexural Test Summary Sheet

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<thead>
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<th>DKP 2</th>
</tr>
</thead>
<tbody>
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<td>5/27/2009</td>
</tr>
<tr>
<td>Test Location</td>
<td>Constructed Facilities Laboratory (CFL), North Carolina State University</td>
</tr>
<tr>
<td>Tested By</td>
<td>Adam Amortnont, E.I. and Meade Willis, E.I.</td>
</tr>
<tr>
<td>Sponsor</td>
<td>RB'C</td>
</tr>
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</table>

Test Parameters

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<tr>
<th>Configuration</th>
<th>Simply supported with four equal loads at the 1/5 points.</th>
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<tbody>
<tr>
<td>Span</td>
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</tr>
<tr>
<td>Loading Rate</td>
<td>0.033 in/min</td>
</tr>
<tr>
<td>Load Cycles</td>
<td>Construction (210 k-ft), Service (298 k-ft), 1.25 Service (372 k-ft), Failure (398 k-ft)</td>
</tr>
</tbody>
</table>

Steel U-Beam

<table>
<thead>
<tr>
<th>Height</th>
<th>4 in. (nominal)</th>
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<tbody>
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<tr>
<td>Width</td>
<td>30 in. (nominal)</td>
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<tr>
<td>Length</td>
<td>30 ft (nominal)</td>
</tr>
</tbody>
</table>

Steel Grade: ASTM A572

Yield Strength: 50 ksi (nominal)

Tensile Strength: 65 ksi (nominal)

Tensile Strength: 74.0 ksi (measured)

Reinforcement

Tension Reinforcement: 5 - No. 9 bars

Compression Reinforcement: 4 - No. 8 bars

Prestressing Reinforcement: N/A

Shear Studs: 5/8 in. dia. by 5 in. long at 12 in. stagger spacing

Concrete Beam

<table>
<thead>
<tr>
<th>Web Depth</th>
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<tbody>
<tr>
<td>Web Width</td>
<td>22 in.</td>
</tr>
<tr>
<td>Bottom Flange Depth</td>
<td>4 in.</td>
</tr>
<tr>
<td>Bottom Flange Width</td>
<td>30 in.</td>
</tr>
<tr>
<td>Length</td>
<td>30 ft</td>
</tr>
</tbody>
</table>

Concrete Compressive Strength: 6900 psi

Test Results

Maximum Applied Moment: 398 k-ft

Maximum Deflection Values: Construction (1.9 in.), Service (3.2 in.), 1.25 Service (4.8 in.), Failure (8.6 in.)

Failure Description: Shear Connection Failure, Concrete Crushing, and Flexural Shear Cracking

Test Observations

- Visible slip between u-shaped steel plate and concrete at service load (298 k-ft)
- Formation of flexural shear cracks at service load
- Additional slip and flexural shear cracks at 1.25 service load (372 k-ft)
- Significant slip between u-shaped steel plate and concrete during failure load step (398 k-ft)
- Opening up of the u-shaped steel plate during failure load step
- Concrete crushing observed at third quarter prior to failure
- Specimen did not reach factored load strength (490 k-ft)
Figure B.8 – DKP 2 Applied Moment vs. Deflection
Figure B.9 – DKP 2 Applied Moment vs. Averaged Mid-span Strain
Figure B.10 – DKP 2 Strain Profiles
Figure B.11 – DKP 2 Applied Moment vs. End Slip

Note: 0.97" maximum slip recorded

Front Concrete-Steel I  Front Concrete-Steel II
Back Concrete-Steel I  Back Concrete-Steel II

Concrete-Steel I  Concrete-Steel II
Figure B.12 – DKP 2 Applied Moment vs. Deflection Envelope Comparison
Figure B.13 – DKP 2 Strain Profile Comparisons
a) Global view of beam at failure with concrete crushing at the third quarter point

b) Concrete crushing at the third quarter point

c) Significant slip between U-shaped steel plate and concrete and “opening” of steel section
d) Shear stud retrieved after failure and steel plate with failure just above the weld collar

e) Embedded shear stud in concrete with local crushing of the concrete

Figure B.14 – DKP 2 Failure Photos
B.3  DKP 1a

Flexural Test Summary Sheet

Test ID : DKP 1a
Test Date : 7/22/2009
Test Location : Constructed Facilities Laboratory (CFL), North Carolina State University
Tested By : Adam Amontmont, E.I.
Sponsor : RB

Test Parameters

Configuration : Simply supported with four equal loads at the 1/5 points.
Span : 27 ft
Loading Rate : 0.033 in/min
Load Cycles : Construction (220 k-ft), Preload (374 k-ft), Service (556 k-ft), 1.25 Service (682 k-ft), Failure (906 k-ft)

Steel U-Beam

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<th>Steel Grade</th>
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</tr>
<tr>
<td>Thickness</td>
<td></td>
</tr>
<tr>
<td>0.25 in.</td>
<td></td>
</tr>
<tr>
<td>Width</td>
<td>Yield Strength</td>
</tr>
<tr>
<td>30 in.</td>
<td>50 ksi (nominal)</td>
</tr>
<tr>
<td>Length</td>
<td>Tensile Strength</td>
</tr>
<tr>
<td>30 ft</td>
<td>65 ksi (nominal)</td>
</tr>
</tbody>
</table>

Reinforcement

Tension Reinforcement : 5 - No. 9 bars
Compression Reinforcement : 4 - No. 8 bars
Prestressing Reinforcement : 6 - 1/2 in. dia. 7-wire Prestressing Strands
Shear Studs : 5/8 in. dia. by 5 in. long at 12 in. stagger spacing

Precast Girder

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<tr>
<th>Web Depth</th>
<th>Concrete Compressive Strength</th>
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<td>7900 psi</td>
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<td>Web Width</td>
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<td>22 in.</td>
<td></td>
</tr>
<tr>
<td>Bottom Flange Depth</td>
<td></td>
</tr>
<tr>
<td>4 in.</td>
<td></td>
</tr>
<tr>
<td>Bottom Flange Width</td>
<td></td>
</tr>
<tr>
<td>30 in.</td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td></td>
</tr>
<tr>
<td>30 ft</td>
<td></td>
</tr>
</tbody>
</table>

Hollow Core Slab

<table>
<thead>
<tr>
<th>Depth</th>
<th>Concrete Compressive Strength</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 in.</td>
<td>6000 psi</td>
<td>7-wire low relaxation strand</td>
</tr>
<tr>
<td>Width</td>
<td></td>
<td>Reinforcement Grade : ASTM A 416</td>
</tr>
<tr>
<td>48 in.</td>
<td></td>
<td>Min. Cover : 1.5 in.</td>
</tr>
<tr>
<td>Length</td>
<td></td>
<td></td>
</tr>
<tr>
<td>32 in.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Concrete Slab

<table>
<thead>
<tr>
<th>Depth</th>
<th>Concrete Compressive Strength</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0 in.</td>
<td>6300 psi</td>
<td>6 x 6 - W2.9 x W2.9</td>
</tr>
<tr>
<td>Width</td>
<td></td>
<td></td>
</tr>
<tr>
<td>89 in.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td></td>
<td>Reinforcement Grade : ASTM A185</td>
</tr>
<tr>
<td>30 ft</td>
<td></td>
<td>Top Cover : 0.75-1.0 in.</td>
</tr>
</tbody>
</table>

Test Results

Maximum Applied Moment : 906 k-ft
Maximum Deflection Values : Construction (2.2 in.), Preload (2.9 in.), Service (3.6 in.), 1.25 Service (4.1 in.), Failure (6.4 in.)
Failure Description : Steel Yielding, Shear Connection Failure, & Longitudinal Shear Cracking

Test Observations

- Small amount of slip noticed at service load (556 k-ft)
- Small separation of concrete slab from hollow core planks at 1.25 service load (682 k-ft)
- Additional slip noticed during 1.25 service load step
- Significant amount of slip and shear connection failure during failure load step (906 k-ft)
- Longitudinal shear crack formation during failure load step
- Specimen did not reach factored load strength (1084 k-ft)
Figure B.15 – DKP 1a Applied Moment vs. Averaged Deflection
Figure B.16 – DKP 1 Applied Moment vs. Averaged Mid-span Strain
Figure B.17 – DKP 1a Strain Profiles
Figure B.18 – DKP 1a Applied Moment vs. End Slip

Note: 0.75" maximum slip recorded
Figure B.19 – DKP 1a Applied Moment vs. Deflection Envelope Comparison
Figure B.20 – DKP 1a Strain Profile Comparisons
a) Global view of beam at failure with concrete crushing at the midpoint

b) Cross section of specimen with longitudinal shear cracks vertically through the cast-in-place concrete and hollow core plank interface

c) Slip between U-shaped steel place and concrete interface
d) Cored sample from the U-shaped steel plate with base of shear stud and embedded shear stud with localized concrete crushing

Figure B.21 – DKP 1a Failure Photos
B.4  DKP 2a

Flexural Test Summary Sheet

<table>
<thead>
<tr>
<th>Test ID</th>
<th>DKP 2a</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Date</td>
<td>7/8/2009</td>
</tr>
<tr>
<td>Test Location</td>
<td>Constructed Facilities Laboratory (CFL), North Carolina State University</td>
</tr>
<tr>
<td>Tested By</td>
<td>Adam Amornont, E.I.</td>
</tr>
<tr>
<td>Sponsor</td>
<td>RB^2C</td>
</tr>
</tbody>
</table>

Test Parameters

Configuration: Simply supported with four equal loads at the 1/5 points.  
Span: 27 ft  
Loading Rate: 0.033 in/min  
Load Cycles: Construction (218 k-ft), Preload (350 k-ft), Service (555 k-ft), Failure (604 k-ft)

Steel U-Beam

<table>
<thead>
<tr>
<th>Height</th>
<th>4 in. (nominal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>0.25 in. (nominal)</td>
</tr>
<tr>
<td>Width</td>
<td>30 in. (nominal)</td>
</tr>
<tr>
<td>Length</td>
<td>30 ft (nominal)</td>
</tr>
</tbody>
</table>

Steel Grade: ASTM A572  
Yield Strength: 50 ksi (nominal)  
Tensile Strength: 62.5 ksi (measured)  
Tensile Strength: 74 ksi (measured)

Reinforcement

Tension Reinforcement: 5 - No. 9 bars  
Compression Reinforcement: 4 - No. 8 bars  
Prestressing Reinforcement: N/A  
Shear Studs: 5/8 in. dia. by 5 in. long at 12 in. stagger spacing

Precast Girders

<table>
<thead>
<tr>
<th>Web Depth</th>
<th>7.25 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web Width</td>
<td>22 in.</td>
</tr>
<tr>
<td>Bottom Flange Depth</td>
<td>4 in.</td>
</tr>
<tr>
<td>Bottom Flange Width</td>
<td>30 in.</td>
</tr>
<tr>
<td>Length</td>
<td>30 ft</td>
</tr>
</tbody>
</table>

Concrete Compressive Strength: 7200 psi

Hollow Core Slab

<table>
<thead>
<tr>
<th>Depth</th>
<th>8 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>48 in.</td>
</tr>
<tr>
<td>Length</td>
<td>32 in.</td>
</tr>
</tbody>
</table>

Concrete Compressive Strength: 6000 psi  
Reinforcement: 7-wire low relaxation strand  
Reinforcement Grade: ASTM A 416  
Min. Cover: 1.5 in.

Concrete Slab

<table>
<thead>
<tr>
<th>Depth</th>
<th>2.0 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>89 in.</td>
</tr>
<tr>
<td>Length</td>
<td>30 ft</td>
</tr>
</tbody>
</table>

Concrete Compressive Strength: 7000 psi  
Reinforcement: 6 x 6 - W2.9 x W2.9  
Reinforcement Grade: ASTM A185  
Top Cover: 0.75-1.0 in.

Test Results

Maximum Applied Moment: 604 k-ft  
Maximum Deflection Values: Construction (2.3 in.), Preload (2.9 in.), Service (4.4 in.), Failure (5.1 in.)

Failure Description: Shear Connection Failure and Longitudinal Shear Cracking

Test Observations

- Small amount of slip noticed during service load step (555 k-ft)  
- Significant amount of slip and shear connection failure during failure load step (604 k-ft)  
- Longitudinal shear crack formation during failure load step  
- Specimen did not reach factored load strength (882 k-ft)
Figure B.22 – DKP 2a Applied Moment vs. Averaged Deflection
Figure B.23 – DKP 2a Applied Moment vs. Averaged Mid-span Strain
I – Construction Load Step (218 k-ft)  
II – Preload Step (350 k-ft)  
III – Service Load Step (555 k-ft)  
V – Failure Load Step (604 k-ft)

Figure B.24 – DKP 2a Strain Profiles
Figure B.25 – DKP 2a Applied Moment vs. End Slip

Note: 0.75” maximum slip recorded
Figure B.26 – DKP 2a Applied Moment vs. Deflection Envelope Comparison
Figure B.27 – DKP 2a Strain Profile Comparisons
a) Global view of beam at failure with longitudinal shear cracks forming around the first quarter point

b) Significant slip between U-shaped steel plate and concrete and “opening” of steel section

c) Cored sample from the U-shaped steel plate with base of shear stud and embedded shear stud with localized concrete crushing

Figure B.28 – DKP 2a Failure Photos