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**The Structural Evaluation of
Precast Concrete Slab Panels in
Bridge Superstructures**

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Final Report

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**Arkansas State Highway and
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The Structural Evaluation of Precast Concrete Slab Panels in Bridge Superstructures

Abstract

During the period from the mid-1950's through the mid-1970's a large number of bridges were constructed throughout Arkansas using a then standard 19 ft long precast, non-prestressed, concrete channel beam section. Notable about these sections is that they were fabricated without shear reinforcement. Nearly 400 bridges constructed with these beams remain in use in Arkansas alone. A nationwide survey of highway departments has identified thirteen states that currently use a similar bridge section.

Recently, the Arkansas State Highway and Transportation Department (AHTD) discovered that a number of these channel beam sections are exhibiting significant deterioration, particularly corrosion of and concrete spalling around the flexural reinforcement in the stems. Consequently, there is a need to determine if the structural capacity of these sections has been significantly affected.

Thirty-three channel beams, removed from four in-service bridge sites, were tested for their load capacity. While the results varied between the thirty-three beams, in nearly every case shear failure controlled the load capacity of a section.

Based on the results of this research, a field manual was prepared for AHTD inspection crews. The field manual establishes guidelines to classify 19 ft precast channel beams constructed between the mid-1950's through the mid-1970's and to identify those beams that may have deficient structural behavior.

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Chapter 1

Introduction

Recently, the Arkansas State Highway and Transportation Department (AHTD) discovered that several precast, non-prestressed, concrete channel beam bridge superstructures are experiencing extensive deterioration. The deterioration is due, at least in part, to reinforcing steel corrosion. The particular precast channel beam sections studied in this technical report have a 19 ft span length and were constructed without shear reinforcement. Design details for the studied sections were developed in 1952 and were designed for a H15 truck loading. A detailed cross-section of a single interior beam section is shown in Figure 1 with an elevation view shown in Figure 2.

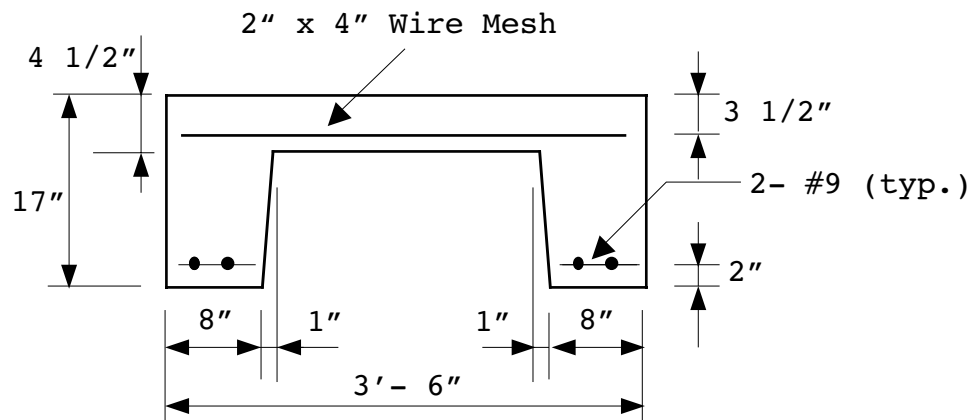


Figure 1: Cross-Section View of Interior Channel Beam

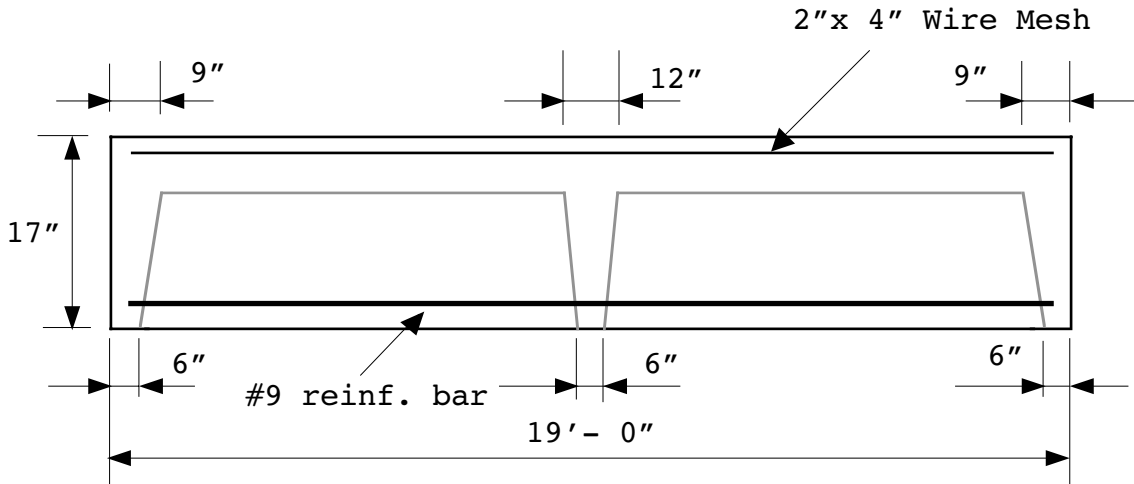


Figure 2: Elevation View of Interior Channel Beam

In Arkansas there are approximately 400 bridges which use the 1952 AHTD details for 19 ft precast channel beams. A national survey of state highway and transportation departments conducted by the authors, indicates that at least thirteen states currently use non-prestressed precast concrete beams with a channel cross-section. These states are listed in Table 1.

Arkansas	Louisiana	Nebraska	Wyoming
Illinois	Minnesota	North Dakota	
Iowa	Mississippi	Pennsylvania	
Kentucky	Missouri	South Dakota	

Table 1: States with Precast Channel Beam Bridges

At a bridge site which includes deteriorated beams, the bridge owner may consider replacement, repair, or no action to an individual beam depending on the severity of the

deterioration. Beam replacement provides the most structurally confident option. However, this option may not be the most economical in terms of material, labor and traffic delays. An alternative approach is to repair deteriorated beams in-place. A third option is to examine the bridge and estimate the remaining service life before one of the two previous options becomes warranted. Any adopted evaluation program by a bridge owner must consider both public safety and economic issues.

This technical report focuses on determining the structural capacity of 19 ft precast channel beam sections that were constructed without shear reinforcement. Included are details of determining the load capacity of the channel beam sections and the material properties of the concrete and tensile strength of the reinforcing steel included in these beams. The project research program included:

- investigating the extensiveness of the problem,
- reviewing other research work related to this specific topic,
- load testing beams at various degrees of deterioration,
- material testing of the beam concrete and reinforcing steel,
- evaluating results,

and

- summarizing research results.

Chapter 2

Background

2.1 Precast Concrete Channel Beam Bridge Description

During the period from the mid-1950's through the mid-1970's a large number of bridges were constructed throughout Arkansas using a then standard, 19 ft long precast, non-prestressed, concrete channel beam. When positioned side-by-side and end-to-end, and subsequently bolted together, these beams become the superstructure of the bridge. This configuration is extremely practical as the stems of the channel resist both flexure and shear forces while the flange acts as the bridge deck, Figure 3.

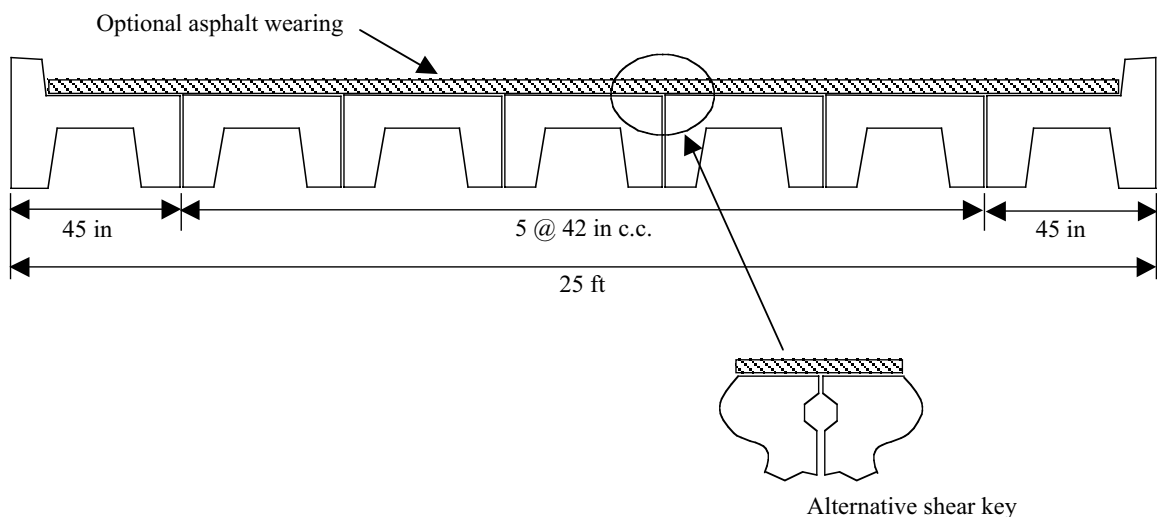


Figure 3: Bridge Cross-Section

Both interior and exterior sections are required for a bridge. The exterior sections are slightly wider than an interior section and have an integrated curb. Some bridges have a grouted longitudinal keyway between sections to improve load transfer to adjacent sections. The beams were cast using Class "S" concrete, $f'_c = 3$ ksi, and flexural reinforcement typically consists of two #9, Grade 40, longitudinal bars in each channel stem. Notable is that shear reinforcement is not present in sections of this vintage.

Details of the bridge beam are provided in the 1952 AHTD “DETAILS OF STANDARD PRECAST SLAB BRIDGE; 19’-0”, 24’-0” CLEAR ROADWAY”.

2.2 Corrosion of Steel Reinforcement

A number of these channel beam bridges in Arkansas are experiencing signs of extensive deterioration. Longitudinal reinforcing steel corrosion appears to initiate this deterioration, with concrete cover loss and bond loss between the concrete and steel as consequences of this corrosion.

The corrosion of steel in concrete occurs when water and oxygen reach the reinforcement, thus creating a galvanic cell. The resulting electrochemical reaction (rust) causes the expansion of the reinforcing bars, which produces tensile stresses in the concrete and eventual cracking of the concrete. Once cracking begins, the corrosion process accelerates and additional bond loss occurs. As a result of extensive bond loss, the structural capacity of the section may be jeopardized.

Reinforcing steel corrosion can be aggravated by the presence of chloride ions in the cement paste and/or water that permeates the concrete. Chloride ions attack the protective oxide film formed on the steel in the highly alkaline chemical environment present in concrete. In precast concrete construction, the use of chloride-based accelerators was a common practice during the 1950’s through 1970’s for shortening the production cycle of precast products. Given that the channel beams examined in this report were fabricated before the introduction of non-chloride accelerators, the possibility of chlorides being introduced into these beams during fabrication was investigated.

Chlorides may also be introduced by the use of deicing salts on bridge decks. As snow and ice melt, the amount of water containing chloride ions will increase due to the presence of deicing salts. This water eventually permeates the concrete, finding its way to the longitudinal steel reinforcement.

Concrete cover acts to protect the reinforcing steel from corrosion. The concrete cover (c) to bar diameter (d_b) ratio is an indicator for the potential of longitudinal cracking and spalling of the concrete, which then exposes beam reinforcement to water seepage [1], Figure 4. Emmons shows that with a c/d_b ratio of 7, concrete begins cracking when the level of corrosion reaches 4% [1]. However, when the c/d_b is lowered to 3, cracking begins with only 1% corrosion.

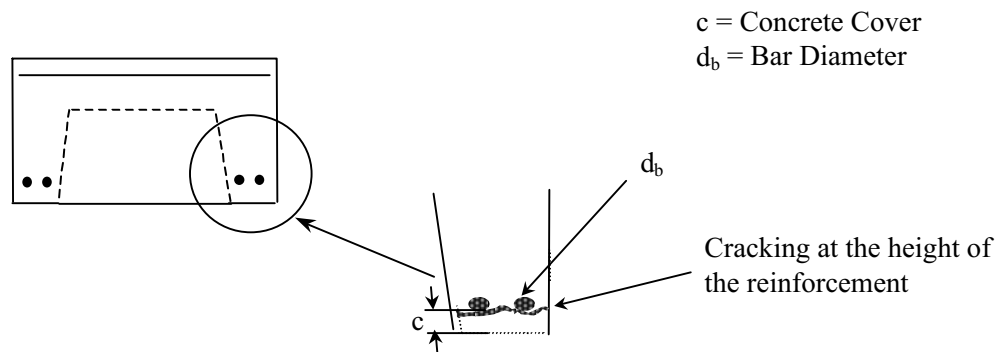


Figure 4: Corrosion-Induced Cracking

The c/d_b ratio for the beams examined in this report is approximately 1.3, suggesting that very little corrosion is required before cracking is initiated. This explains the excessive cracking located adjacent to the stem reinforcement. Most importantly, corrosion leads to

a reduced reinforcing bar cross-sectional area. Since beam flexural strength is a function of steel reinforcing area, a reduced steel cross section area will have a detrimental effect on load capacity. Emmons states that for beams tested in flexure, when steel reinforcement corrosion is greater than 4.5% of the total reinforcement area, the ultimate load capacity is reduced by 12 percent [1]. In many of the beams tested in this report, there was significant loss of reinforcement cross-sectional area due to corrosion; however, this was not quantified. In addition to steel reinforcement corrosion, the lack of shear reinforcement introduces the potential for shear failure and reduced beam ductility. The collective effect of the factors noted above raises concern about the structural capacity, integrity, and longevity of these deteriorated bridge sections.

Chapter 3

Literature Review

3.1 Bridge Strengthening Project - Boone County, Missouri

Three bridges in Boone County, Missouri were constructed between 1970 and 1976 using single-span, simply supported precast reinforced concrete channel sections [2]. Eight channel sections comprise the bridge cross-section. Each section is 38 in. wide and 24 in. deep. The bridges were designed in accordance with the American Association of State Highway and Transportation Officials (AASHTO). After evaluating the three bridges in 1979, a 15 ton load rating was assigned to each of the bridges based on the existing beam condition. An improved bridge rating was warranted at these sites; however, bridge replacement was not an option. Instead, the beams were retrofitted using carbon fiber reinforced polymer (CFRP) strips because of ease and speed of application.

Although the design concrete compressive strength was 3.0 ksi, field tests gave measured strengths exceeding 9.0 ksi. For retrofitting purposes, concrete compressive strength was taken as 5.0 ksi and 40 ksi was used for reinforcing steel yield strength. A CFRP composite was used on the sides of the beam sections to provide shear strength and also applied along the bottom of the stems to provide flexural strength. After the retrofit, load testing showed a decrease in mid-span deflection and reduced beam internal stresses.

3.2 Channel Beam Bridge – Grant County, Arkansas

In the fall of 2000, a structurally deficient precast concrete channel beam bridge was documented in the Arkansas Democrat Gazette [3]. The bridge is located on State

Highway 46 in Grant County and consists of 56 spans using precast channel beams, described in this report, and 3 spans using steel beams. The total length of the bridge is 1245 ft. State Highway 46 is a major transportation route for local lumber mills and a fish processing plant. After an inspection of the bridge in October of 2000, the AHTD lowered the weight limit to three tons. However, many of the logging trucks that pass over this bridge weigh in excess of 85.0 k, which is nearly fourteen times the lowered weight limit. To remedy the problem, deteriorated beam sections were replaced [3].

3.3 Case Study – Iowa State University

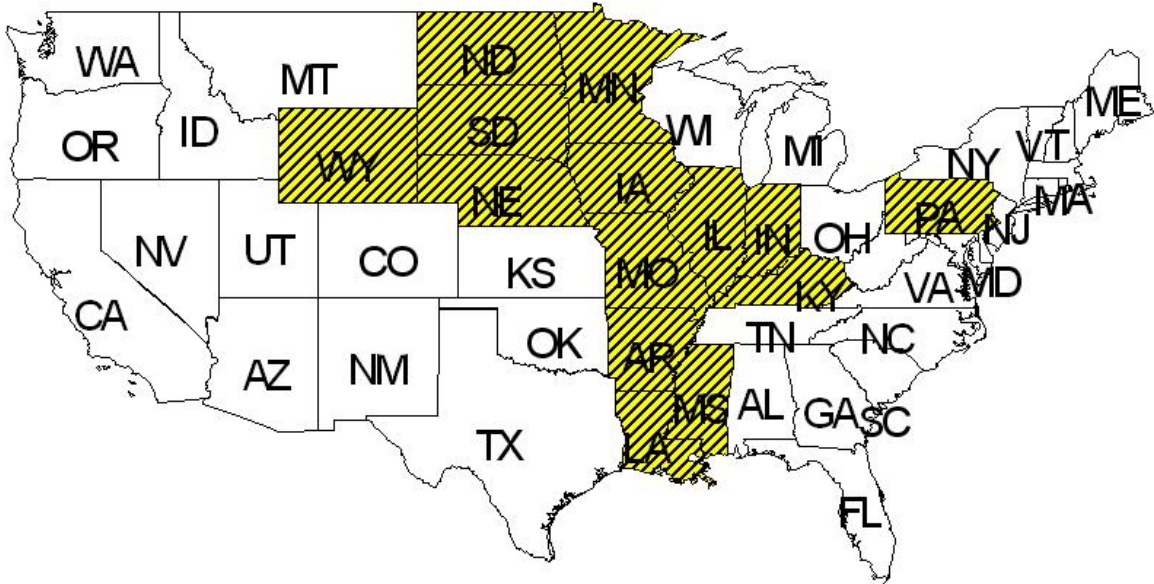
A recent study conducted at Iowa State University (ISU) examined precast channel beam bridges located in Iowa [4]. The study included field and laboratory tests on these channel sections. Load tests were performed on four deteriorated precast concrete deck bridges; two with shear keys in place and two without. Strain gauges were used to measure the strain in the concrete and steel, as well as transducers to measure vertical beam deflection. Findings showed that when shear keys were properly installed between panels, satisfactory load distribution and ample strength was developed. Conversely, when a shear key was not present or when the shear key was not filled with grout, there was a significant reduction in live load distribution. A minimal loss of concrete cover was not a concern since hooked reinforcing bars were used. In addition to hooked bars, these beams contained shear reinforcement. Twelve beams were obtained from bridges in the state and tested at ISU using a four point bending arrangement. Eleven beams tested exceeded the theoretical load capacity. All the beams tested failed from a compressive failure of the concrete deck surface.

Service tests and ultimate strength tests were conducted on four joined panels to determine the effect of various shear connection configurations. The connection configurations were made of tight and loose bolts, only tight bolts, and shear connectors with tight bolts. The data collected was used to calibrate a precast concrete deck bridge finite element model to determine live load distribution factors for bridge rating. In addition, a retrofit was designed to reinforce the deteriorated bridges for flexural strength. The retrofit consisted of a strut located under the diaphragm and a post-tensioned tendon extending the length of each panel. The strand underneath the strut is tensioned to produce a moment on the precast section equal and opposite to the moment produced from the weight of the beam. Consequently, the removal of the effect of dead load increases the live load capacity.

3.4 Continental United States Precast Survey

A general knowledge of the number and condition of precast concrete channel sections in Arkansas was known; however, the use of this section in other states was unknown. To obtain a better understanding of the use of precast channel sections, a survey was sent to all states located within the continental US. The survey consisted of questions ranging from the number of precast channel bridges that make up their highway system to deterioration problems and methods of rehabilitation. The survey is presented in Appendix A. Of the 26 states that responded to the survey, 13 currently use and one state, Indiana, had previously used precast concrete channel beam sections. States which have used these sections are identified in Figure 5. The use of this bridge superstructure represents a small percentage of the total number of bridges in each reporting state with

percentages ranging from 0.2% to 8.5%. The typical age of these bridges ranges from 5 years in Louisiana to 50 years in Iowa, with the majority 30 years and older.



 **States Having PCB Bridges**

Figure 5: States Having Precast Non-Prestressed Channel Beam Bridges

Longitudinal cracking, concrete spalling and steel corrosion in the channel beams are evident in 11 of the 14 states. Repair strategies used by these states include patching and bridge replacement. A list of these eleven states is given in Table 2.

Arkansas	Iowa	Mississippi	South Dakota
Illinois	Kentucky	Missouri	Wyoming
Indiana	Minnesota	Nebraska	

Table 2: States with Deteriorating Precast Channel Beam Bridges

Chapter 4

Problem Statement

During the 2000 inspection of a channel beam bridge located near Jenkins' Ferry, AR, a lack of concrete cover and longitudinal reinforcing steel corrosion in the beam stem raised a concern for the structural integrity of the bridge. An underside picture of span 43 of the bridge is shown in Figure 6.



Figure 6: Deterioration of Jenkins' Ferry Bridge

As a result of the severe deterioration found at the Jenkins' Ferry bridge, the AHTD decided to investigate the condition of other similar bridges throughout the state. This preliminary investigation suggested that similar problems existed with this section type at many other locations. As a result of the investigation, many precast channel beam bridges throughout the state are currently posted at low vehicle loads.

In order to more accurately load rate these bridges, research is warranted to evaluate the structural capacity of sections in varying states of deterioration. With approximately 400

such bridges in the Arkansas state highway system, it is essential to address this problem in an appropriate manner.

This report documents load testing results performed on 37 precast channel beams. In addition to load testing, concrete material properties of compressive strength, permeability, chloride content, and alkali silica reactivity were determined. Yield strength of the reinforcing steel was also measured.

Chapter 5

Description of Tasks

The primary objective of the authors was to develop a relationship between the visual appearance of the beam and its load carrying capacity. With this information, bridges consisting of precast channel beams could be more appropriately posted for weight limits. In order to accomplish this objective the following tasks were performed:

1. Categorize beam conditions.
1. Load test beams
1. Collect data on beam material properties.

To achieve these objectives, 33 beams were obtained from the field and load tested to failure. Concrete samples were cored from the beams and tested for compressive strength. Reinforcing steel samples were cut from a few of the sections to evaluate yield strength. In addition, 4 newly fabricated channel sections (2 with stirrups and 2 without stirrups) were load tested to failure. Details of these tasks are provided below.

5.1 Task 1: Selection of Channel Sections to be Tested

A minimum of three beams were identified in each of four condition categories: good, average, poor, and repaired. The aim of testing sections in these various conditions was to produce load - deflection data considering various states of deterioration. Beams for load testing were taken from three bridge sites and a maintenance facility. These four locations are shown in Figure 7. In addition, two precast channel beams were cast

without shear reinforcement and two beams cast with shear reinforcement to investigate the significance of shear reinforcement.

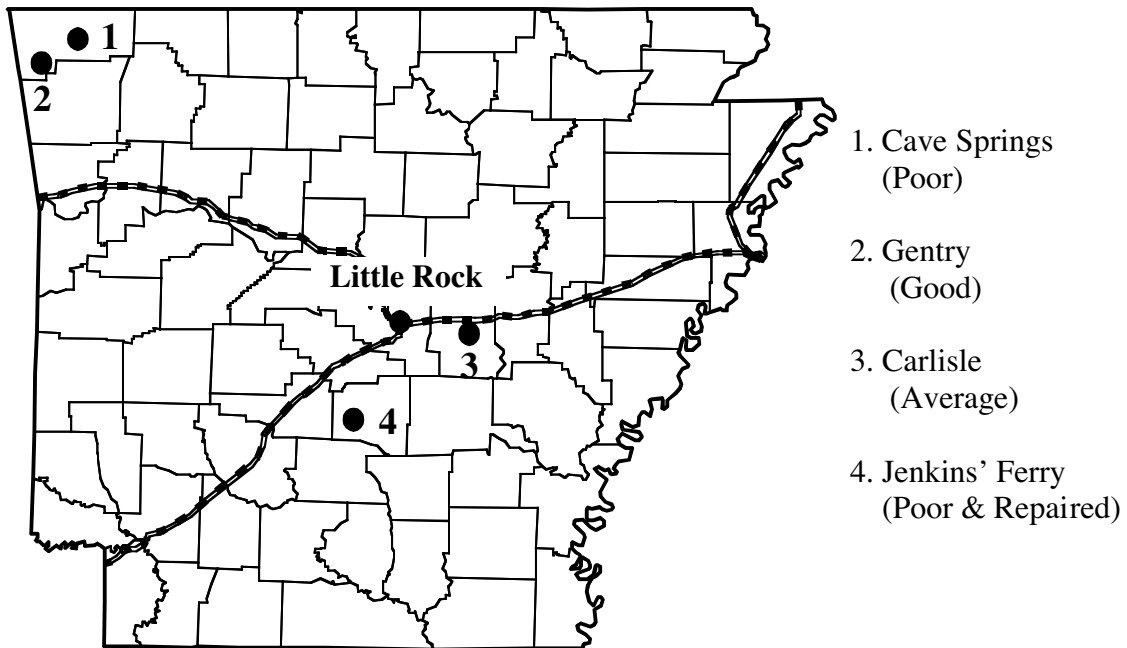


Figure 7: Location of Selected Precast Channel Beams

Visual inspection was used to categorize the beam sections as New, Good, Average, Poor, or Repaired. Categorizing the beams was subjective and primarily based on the percentage and location of exposed longitudinal reinforcing steel. A “New” section, Figure 8, is a newly fabricated section. A “Good” section, Figure 9, is one with little evidence of deterioration. A section defined as “Average,” Figure 10, has visible cracks, may have concrete staining on the beam stem, and may have limited spalling. Sections with extensive spalling and reinforcing steel corrosion are denoted as “Poor” as shown in Figure 11. A “Repaired” section is one that has undergone a field repair typically using a shotcrete technique, Figure 12.



Figure 8: Precast Channel Beam in “New” Condition



Figure 9: Precast Channel Beam in “Good” Condition



Figure 10: Precast Channel Beam in “Average” Condition



Figure 11: Precast Channel Beam in “Poor” Condition



Figure 12: Precast Channel Beam in “Repaired” Condition

After beams in these categories were identified, they were transported by the AHTD to the Engineering Research Center (ERC) at the University of Arkansas. A detailed description of the beams included in this report follows.

5.1.1 Jenkins’ Ferry Bridge

Six beams were selected by AHTD personnel from the Jenkins’ Ferry Bridge. This bridge spans over the Saline River on State Highway 46 in Grant County, Arkansas. The bridge cross section was consistent with that depicted in Figure 3. An asphalt overlay was removed from the surface of the beams prior to load testing at ERC. The sections were noted as being in “Poor” condition due to extensive longitudinal cracking, reinforcing steel corrosion, concrete spalling, and exposed reinforcement. Half of the beams tested were previously field repaired using a shotcrete method. These three sections are labeled “Repaired” in this report.

5.1.2 Cave Springs, Little Osage Creek Bridge

Eight beams were selected from the Little Osage Creek Bridge on State Highway 12 near Cave Springs, AR, in Benton County. This two-lane bridge is near a Wal-Mart® distribution center. Consequently, these units were subjected to a large number of heavy truck loadings over several years. Longitudinal cracking and concrete spalling accompanied by poor quality concrete, resulted in the “Poor” classification of these beams. An examination of the underside of this bridge revealed the widespread presence of white-colored “stalactites”, Figure 13. Many of these “stalactites” were located along the longitudinal joints between the precast panels, suggesting that water is traveling between the beams and seeping to the steel reinforcement. An asphalt overlay was removed prior to load testing and revealed a heavily deteriorated concrete surface with concrete chunks that could easily be removed by hand.



Figure 13: Stalactites on Underside of Panel

5.1.3 Gentry Maintenance Yard

Seven sections were taken from the AHTD Maintenance Yard, in Gentry, Arkansas, located in Benton County. These sections had been salvaged and stock piled for use as replacement sections. These sections had no visible evidence of deterioration, and therefore were categorized as “Good.”

5.1.4 Carlisle Bridge

Twelve “Average” sections were taken from Carlisle, Arkansas during an AHTD bridge replacement project. These sections were examined in-place by members of the research team prior to their removal and shipment to ERC for testing. These sections exhibited few signs of distress. However, the presence of longitudinal cracks along with some limited exposed reinforcing steel resulted in an “Average” condition rating. In two of the twelve sections, strain gauges were attached to the exposed outer longitudinal reinforcing bars to monitor steel behavior during load testing.

5.1.5 New Precast Concrete Channel Beams

Four “New” sections were fabricated by Hanson Pipe and Products. Because of limited availability of Grade 40 steel, Grade 60 steel was used for the steel reinforcing bars. Two of the sections were constructed with the same design parameters as that of the beams being examined in this project. In addition, two of the sections were fabricated with shear reinforcement consisting of Grade 60, #3 rebars spaced at 8 inches apart. The four “new” beams were instrumented at five locations along the longitudinal reinforcing steel and at two locations on the shear stirrups.

5.2 Task 2: Load Testing

The precast channel beams were load tested to failure to determine the load carrying capacity of each section. During Task 2, the correlation between applied load and midspan beam deflection, mode of failure, crack propagation, and elastic-plastic behavior was investigated. A four-point loading scheme was used to determine the load capacity of each beam. Two, 200 k reversible hydraulic jacks located 5 ft – 6-in from each beam end were used to apply two concentrated forces to the beam. This loading configuration produces a constant and maximum bending moment in the center 8 ft section of the beam, and a constant and maximum shear in the outer spans. The loading, bending moment, and shear diagrams due to the applied loading are shown in Figure 14. The side view and cross-sectional views of the load frame are given in Figure 15 and Figure 16 respectively.

The channel beams were positioned onto the testing bed using an 8 k all-terrain forklift. Each end of a beam rested on an 8-in x 8-in x 48-in solid oak timber block. The length of the end support is similar to that provided by a pier cap in the field. Once the beam was positioned, four 2-1/4-in diameter threaded steel bars, 5 ft in length, were screwed into the reaction beam, Figure 16. A rod was positioned on each side of the beam adjacent to the point loads. A four-inch diameter PVC pipe, approximately 3 ft - 5-in long, was placed over each steel rod for protection and to support a welded double channel cross-member. Nuts were threaded onto each rod, above the built-up channel section, to lock the channels in place.

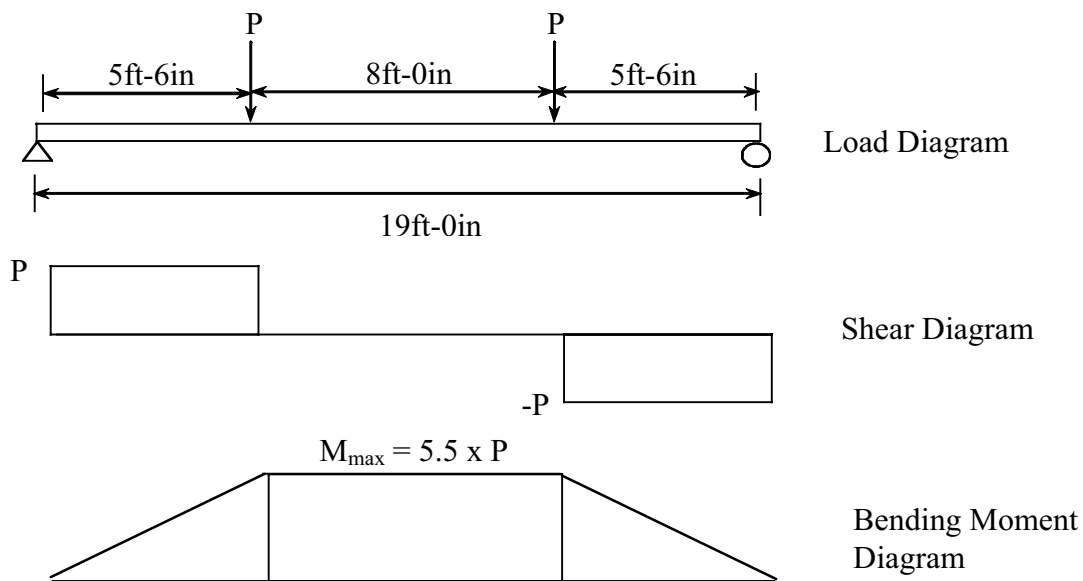


Figure 14: Equivalent Force Diagram

A hydraulic jack was placed on the concrete beam beneath each of the two double welded channel sections to produce a concentrated force on the beam. A 12-in x 42-in x 0.5-in steel plate was used beneath each jack in order to uniformly distribute the load across the width of the beam. The jacks were linked in parallel, each applying an equal force to the two concentrated load points. Load was applied using a manually actuated pump and monitored by a digital pressure gauge attached to the hydraulic lines supplying each jack. Pressure was then converted to load using the known jack surface area. During loading, midspan deflection was continuously recorded using a digital video camcorder. The camcorder was attached to the beam and measured the midspan beam deflection relative to a stationary measuring stick. Load and midspan deflection were recorded in load increments during testing.

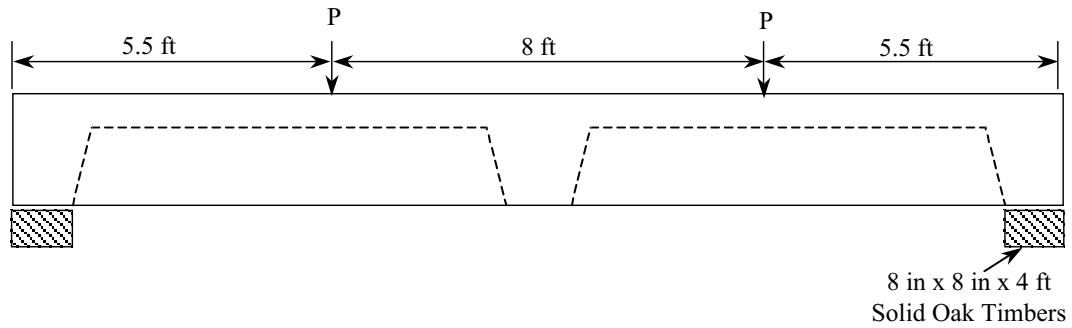


Figure 15: Side View of Loading Configuration

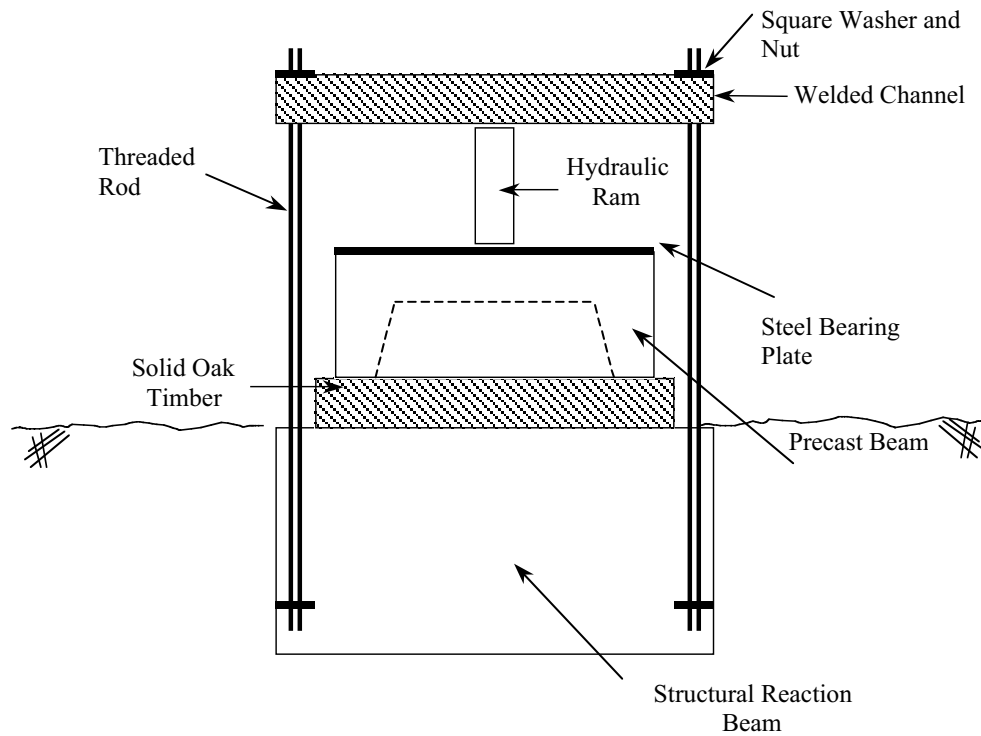


Figure 16: Cross-Section of Loading Configuration

All beams were tested to failure to determine beam load capacity, where failure was defined either by concrete deck crushing due to flexure, or by a wide diagonal tension crack near the supports due to shear.

Elastic-plastic behavior was investigated for three Carlisle beams to establish the load at which permanent (inelastic) deformation occurs. Crack propagation was monitored and recorded on the beam side as a function of the applied load. The 2P load was applied in 20 k increments by attaining the desired load and then removing the applied load. This procedure continued up to a 2P load of 80 k.

5.3 Task 3: Determination of Material Properties

In order to analyze the data obtained from the load tests, it was necessary to determine the engineering properties of the concrete and reinforcing steel. It was also deemed necessary to examine any evidence of concrete degradation.

The concrete's compressive strength was determined by obtaining 4-in diameter cores from the mid-height along one stem of the beam, as shown in Figure 17. The cores were taken after the beam was load tested so as to not influence the load carrying characteristics of the beam. Cores were taken from the beam stem near the ends of the beam, since only limited cracking occurred in this region. To obtain the cores, the beams were placed on their side allowing the coring device to stand up-right. The core ends were saw cut perpendicular to the axis of the cylinder and tested in accordance with ASTM C39/C 39M - 01, "Test Method for Compressive Strength of Cylindrical Concrete Specimens" [5].

The yield and ultimate tensile strength of the reinforcing steel was determined by testing three pieces of steel obtained from three different beams from each bridge site after load testing. These steel samples were taken at arbitrary sections along the span. The samples

were cut with an acetylene torch to a 12-in length. Each #9 bar was then cut to an 8-in length and turned down by a machinist to a 0.5-in diameter. An extensometer was mounted on each specimen and tension loaded in an MTS machine. The yield and ultimate strengths were identified from the test data using the “Direct Method,” as described in ASTM E8. The yield strength was recorded as the average of the three bars tested at each bridge location.

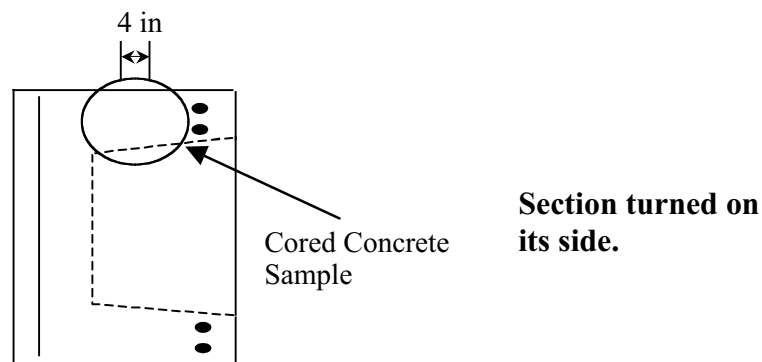


Figure 17: Method of Obtaining Concrete Compressive Sample

Chloride content and concrete permeability tests were performed to investigate these as possible causes of the extensive reinforcement corrosion identified in the tested beams. Chloride content of the concrete was determined for all six of the Jenkins’ Ferry Bridge beams using ASTM C 1218/C 1218M – 99, “Standard Test Method for Water-Soluble Chloride in Mortar and Concrete” [6]. To determine chloride content, three powder samples weighing at least 20 grams each were taken through the beam stem at stem mid-

height at a distance of 1.5-in, 3-in and 5-in from the outside surface of the beam stem as shown in Figure 18. The chloride samples were then analyzed by Challenge Environmental Laboratory, LLC of Fayetteville, Arkansas.

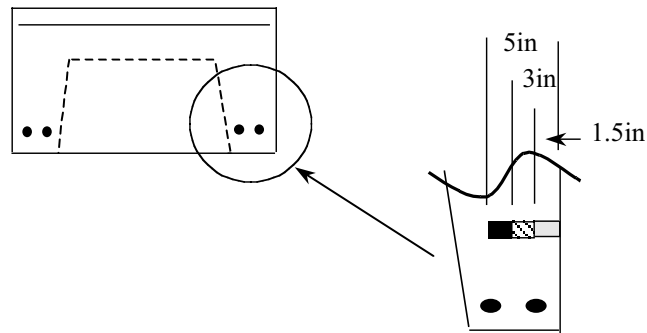


Figure 18: Method of Obtaining Chloride Sample

Water seepage due to high concrete permeability was investigated as a possible cause of reinforcing steel corrosion using a non-standard, but highly accurate permeameter, the Kuss Permeameter. The Kuss Permeameter is a recently developed testing device to quickly establish a relative permeability for a given concrete sample. In the procedure, a 2-in wide ring with a 12-in outside diameter is sealed on the surface with a clear spray-on sealer. An 8-in diameter pressure difference plate is then sealed to the ring. A nominal pressure difference is applied to the plate to measure permeability. The permeability setup is shown in Figure 19. The volume of air required to maintain the pressure difference is measured to determine a relative permeability.



Figure 19: Kuss Permeability Setup

Additional testing was performed on a Cave Springs beam to examine the possibility that concrete spalling may be due to alkali-silica reaction (ASR). Samples for the ASR testing were taken from the material remaining after the extraction of the compressive strength cores. These samples were then tested according to the Strategic Highway Research Program C - 315 “Handbook for the Identification of Alkali - Silica Reactivity in Highway Structures” [7].

Chapter 6

Data Analysis and Test Results

At completion of the load and material testing, the results were analyzed to develop correlations between visual inspection, material properties, and load capacity. Beams were grouped and categorized according to the degree of deterioration of the unit. Units within a category were then compared according to load capacity. “New” beams containing shear stirrups approached a load capacity, 2P (Figure 14), of 135 k and an average midspan deflection of 6.126-in. However, for “New” beams without shear reinforcement, failure occurred at a 2P load of 78 k with a corresponding midspan deflection of just over 1-in. For a beam in either “Good” or “Average” condition, the load capacity (2P) ranged from 84 k to 124 k, with an average of approximately 100 k. Beams rated in “Poor” or “Repaired” conditions had considerably lower load capacities (2P), ranging from 40 k to 100 k, with an average of 74 k. Maximum midspan deflection at failure for “Good” and “Average” beams ranged between 1.5-in to 6.6-in. However, for “Poor” and “Repaired” sections the maximum midspan deflection was significantly lower, in the range of 0.875-in to 4.6-in. Figure 20 shows typical load-midspan deflection behavior for each of the beam groups.

With the exception of the Cave Springs beams, the actual concrete compressive strength of the beams was considerably higher than the 3.0 ksi design strength of the AHTD Design Details. The average concrete compressive strength in the Cave Springs’ beams was found to be 4.2 ksi. However, the average compressive strength was 9.8 ksi for the Jenkins’ Ferry beams, 10.8 ksi for the Gentry beams, and 12.7 ksi for the Carlisle beams. Visual inspection of the Cave Springs’ concrete showed a poor paste-aggregate bond.

The 1952 AHTD Bridge Details specify a 40 ksi reinforcing steel yield strength. However, testing showed the average reinforcing steel yield strength was 40 ksi for the Cave Springs and Gentry beams, and 50 ksi for the Jenkins' Ferry and Carlisle beams.

Low permeability was found in all the beams except for those from Cave Springs. The measured permeability for Jenkins' Ferry panels was 0 ml/min of water penetration after the asphalt overlay was removed. This indicates that the asphalt overlay binder layer on the beam deck surface was impermeable. Gentry and Carlisle beams had permeability readings of 0.06 ml/min and 0.04 ml/min of water penetration respectively. This low permeability value concurs with the minimal reinforcing steel corrosion in these beams. However, extremely high permeability readings of 23.5 ml/min and 32.2 ml/min were recorded for the Cave Springs beams. This high permeability is consistent with the poor concrete condition in these beams.

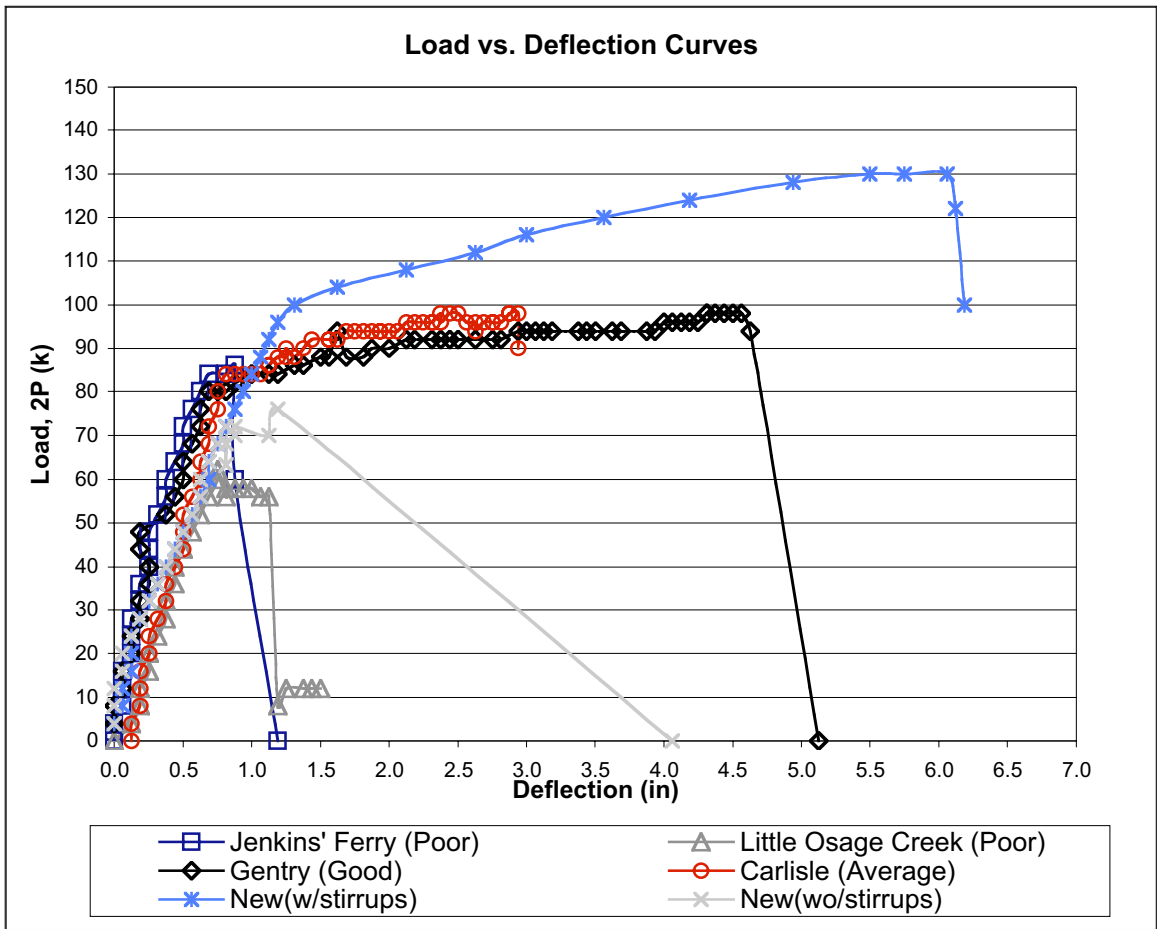


Figure 20: Typical Load Versus Deflection Behavior

6.1 Jenkins' Ferry Bridge

Six precast panels from the Jenkins' Ferry Bridge were the first to be tested and analyzed. Results for load capacity, concrete compressive strength, and steel tensile strength are summarized in Table 3. Five of the six sections ultimately failed in shear.

Beam ID	Original Location	Beam Condition	Failure Mode	Load Capacity, 2P (kip)	Mid-Span Deflection (in)	Core Strength # (ksi)	Density (kcf)	Steel Tensile Strength * (ksi)
RA	JF	Poor	Shear	100	2.563	8.677	N/A	N/A
RB	JF	Poor	Shear	100	3.375	9.514	N/A	57.9
RC	JF	Poor	Shear	48	2.875	7.683	N/A	53.1
R1	JF	Poor/Repaired	Shear	40	1.125	9.216	N/A	48.7
R2	JF	Poor/Repaired	Flexural	95	4.563	11.578	N/A	N/A
R3	JF	Poor/Repaired	Shear	86	1.188	12.242	N/A	N/A

- Values reflect one test per beam.

* - Values reflect one test per beam.

Table 3: Jenkins' Ferry Test Results

6.1.1 Visual Inspection

By visual inspection, all of the Jenkins' Ferry Bridge beams were categorized as "Poor". This assessment was based on noticeable shear cracks near the beam ends, extensive longitudinal cracking, Figure 21, and exposed and corroded flexural reinforcing steel, Figure 22. The concrete for these beams was produced using a smooth, rounded, coarse aggregate. Three of the six beams had been repaired in the field using a shotcrete technique, Figure 23. Sections from this bridge were delivered to the ERC testing facility with an asphalt overlay. This overlay was removed manually prior to load testing. Upon removal of the overlay, water ponding at the asphalt-concrete interface was evident. Because asphalt is inherently more permeable than good quality concrete, water permeates through the asphalt overlay and then ponds between the asphalt – asphalt

binder interface at the concrete deck surface. The water is trapped at this interface since the overlay drastically reduces water evaporation. Considering this condition, a possible seepage path is from the deck surface to the longitudinal reinforcing steel causing bar corrosion.



Figure 21: Longitudinal Cracking at the Height of the Reinforcement



Figure 22: Concrete Spalling and Reinforcement Corrosion



Figure 23: Shotcrete Technique Repair

6.1.2 Chloride Content

Chloride penetration results from sections showed decreasing chloride concentration as a function of distance from the face of the beam stem. The maximum chloride concentration was found in beam R2, where the chloride concentration ranged from 598

ppm (parts per million) at 0-in to 1.5-in from the beam stem surface, decreasing to 368 ppm between 1.5-in and 3-in, and 226 ppm from 3-in to 5-in. This reduction in chloride concentration as the distance from the surface of the beam stem increases indicates that a chloride-based accelerator was not likely used during the fabrication process of these beams.

6.1.3 Load Capacity and Deflection

On average, the maximum load capacity, $2P$, for these sections was 78.2 k, which translates to a bending moment, excluding self-weight, of 215.1 ft-k. On average, a 2.6-in maximum midspan deflection was measured. Even though these sections were categorized as “Poor”, two units had failure loads of 100 k with 2.6-in and 3.4-in midspan deflections. Figure 24 is a plot showing the load – deflection relationship for this beam group.

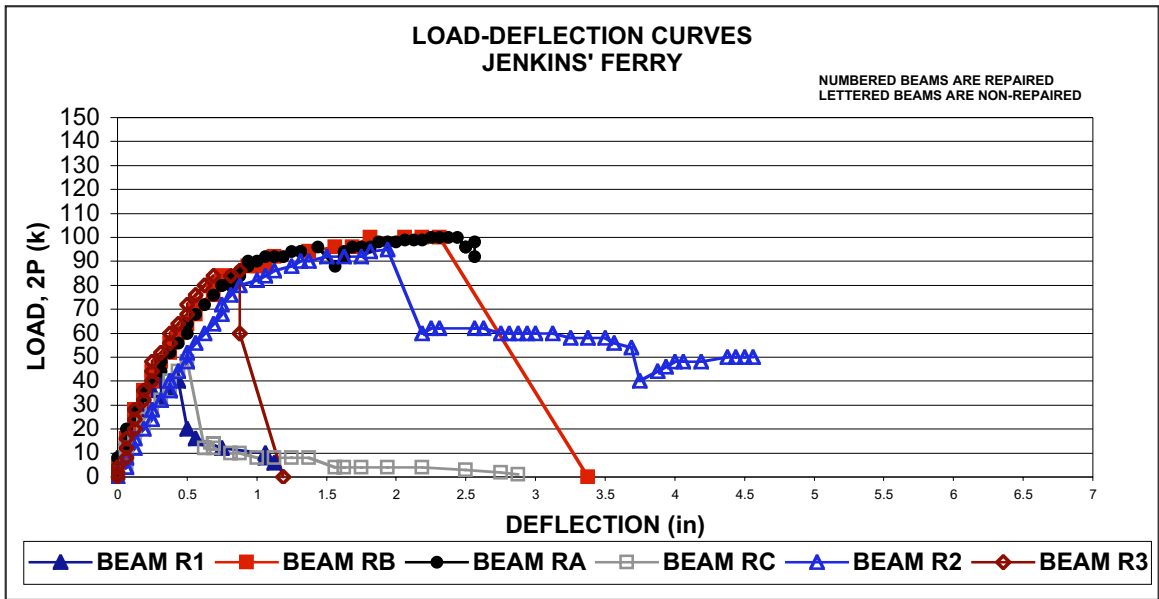


Figure 24: Jenkins' Ferry Load Versus Deflection Results

6.2 Cave Springs, Little Osage Creek Bridge

Eight beams taken from the Little Osage Creek Bridge were evaluated and tested. Results for structural load capacity, concrete compressive strength, and steel tensile strength of these beams are summarized in Table 4. All sections from this bridge ultimately failed in shear.

Beam ID	Original Location	Beam Condition	Failure Mode	Load Capacity, 2P (kip)	Mid-Span Deflection (in)	Core Strength # (ksi)	Density (kcf)	Steel Tensile Strength * (ksi)
C1	CS	Poor	Shear	64	0.875	4.948	0.1372	43.9
C2	CS	Poor	Shear	68	1.125	5.045	0.1424	43.6
C3	CS	Poor	Shear	44	1.188	2.804	0.1427	46.5
C4	CS	Poor	Shear	62	1.500	4.000	0.1409	N/A
C5	CS	Poor	Shear	84	2.000	N/A	N/A	N/A
C6	CS	Poor	Shear	84	0.938	N/A	N/A	N/A
C7	CS	Poor	Shear	84	1.031	7.351	N/A	N/A
C8	CS	Poor	Shear	80	1.000	5.088	N/A	N/A

- Values reflect one test per beam.

* - Values reflect one test per beam.

Table 4: Cave Springs Test Results

6.2.1 Visual Inspection

All of the Cave Springs units were categorized as being in "Poor" condition by visual inspection. Similar to the Jenkins' Ferry Bridge, the Cave Springs bridge beams had an asphalt overlay. When the overlay was removed, the underlying concrete was severely deteriorated, Figure 25. Deterioration was severe enough that concrete pieces could manually be removed from the beam. Overall, the concrete had a very white and powdery appearance due to lime leaching from the cement paste. Permeability readings were very high for these sections, which agrees with the poor concrete quality.



Figure 25: Poor Aggregate-Paste Bonding on Top of Beam

6.2.2 Alkali-Silica Reaction

Mild to moderate ASR was identified in these sections; however, this level of ASR does not concur with the level of concrete deterioration present at these beams.

6.2.3 Load Capacity and Deflection

The average load capacity, $2P$, of the Cave Springs beams was 71.3 k, equivalent to a 196.0 ft-k bending moment, excluding self-weight. Load versus deflection plots are shown in Figure 26 and drawn to the same scale as the other load-deflection plots for comparison. Concrete spalling along the underside of the beam occurred during loading. All of the Cave Springs sections failed in shear at small midspan deflections, with minimal visual or audible warning.

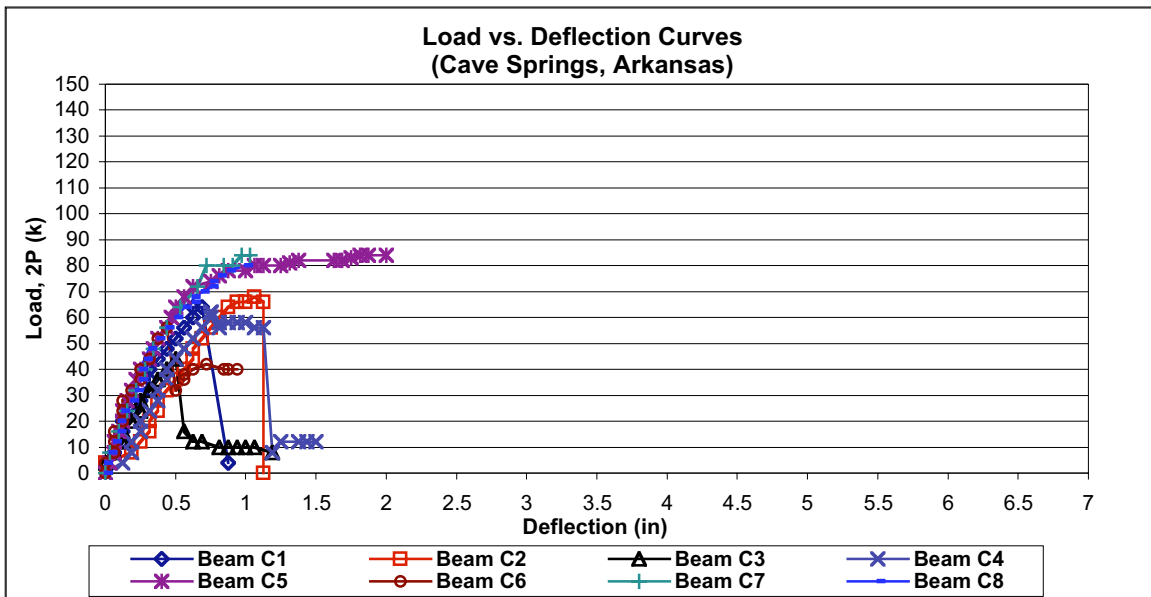


Figure 26: Cave Springs Load Versus Deflection Results

6.3 Gentry Maintenance Yard

Table 5 shows results for load capacity, concrete compressive strength, and reinforcing steel tensile strength tests of the seven Gentry maintenance yard beams. Six of the seven sections ultimately failed in shear.

Beam ID	Original Location	Beam Condition	Failure Mode	Load Capacity, 2P (kip)	Mid-Span Deflection (in)	Core Strength # (ksi)	Density (kcf)	Steel Tensile Strength * (ksi)
G1	G	Good	Shear	96	2.625	11.854	0.1455	45.24
G2	G	Good	Flexural	104	5.688	10.433	0.1459	N/A
G3	G	Good	Shear	98	5.125	10.983	0.1434	41.18
G4	G	Good	Shear	106	6.125	10.059	0.1435	44.08
G5	G	Good	Shear	84	1.500	N/A	N/A	N/A
G6	G	Good	Shear	94	3.063	N/A	N/A	N/A
G7	G	Good	Shear	94	2.813	8.857	N/A	N/A

- Values reflect one test per beam.

* - Values reflect one test per beam.

Table 5: Gentry Test Results

6.3.1 Visual Inspection

Beams obtained from the Gentry Maintenance Yard showed no signs of distress; therefore, these beams were considered to be in "Good" condition. There was no evidence of spalling or exposed steel reinforcement and no asphalt-wearing surface was evident. A typical Gentry beam is shown in Figure 27.

6.3.2 Load Capacity and Deflection

The average load capacity, 2P, of the Gentry beams was 96.5 k, which corresponds to an average maximum bending moment of 265.4 ft-k, excluding self-weight. Midspan deflections reached a maximum of just over 6-in as shown in Figure 28.



Figure 27: Gentry Beam with No Signs of Distress

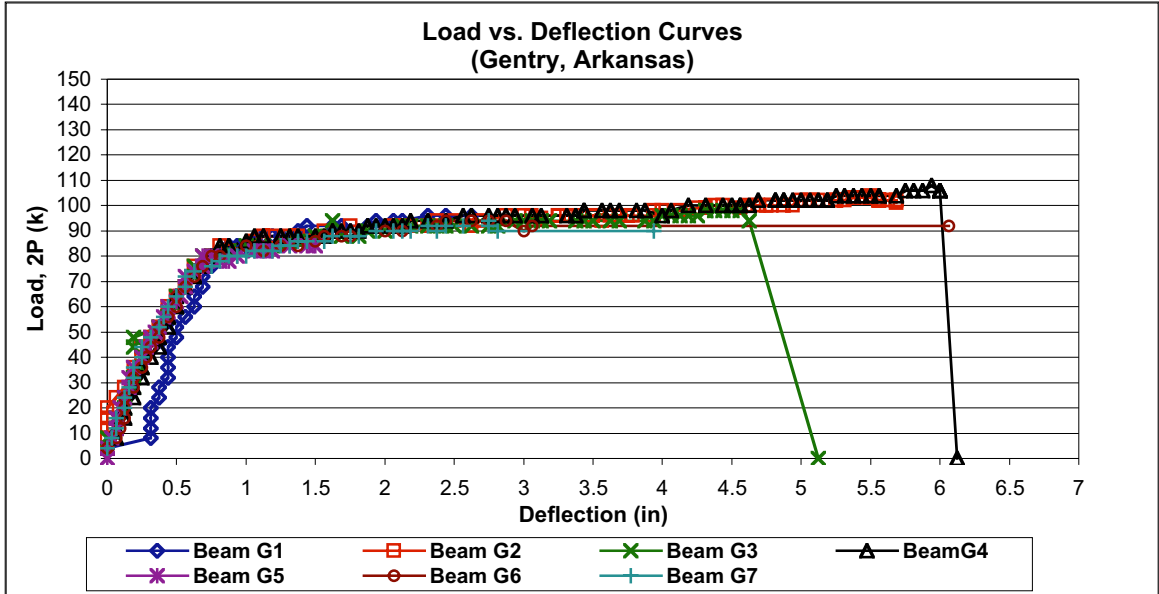


Figure 28: Gentry Load Versus Deflection Results

6.4 Carlisle Bridge

Results for the twelve Carlisle beams tested for load capacity, concrete compressive strength, and reinforcing steel tensile strength are summarized in Table 6. All of the twelve sections from this group ultimately failed in shear.

Beam ID	Original Location	Beam Condition	Failure Mode	Load Capacity, 2P (kip)	Mid-Span Deflection (in)	Core Strength # (ksi)	Density (kcf)	Steel Tensile Strength * (ksi)
L1	C	Average	Shear	102	4.375	N/A	N/A	50.64
L2	C	Average	Shear	102	3.563	13.398	0.1464	N/A
L3	C	Average	Shear	98	2.938	N/A	N/A	N/A
L4	C	Average	Shear	102	4.125	13.849	0.1458	54.41
L5	C	Average	Shear	100	2.250	11.002	0.1449	N/A
L6	C	Average	Shear	94	1.875	11.619	N/A	50.71
L7	C	Average	Shear	90	4.375	9.274	N/A	N/A
L8	C	Average	Shear	94	3.250	9.766	N/A	N/A
L9	C	Average	Shear	100	5.063	7.351	N/A	N/A
L10	C	Average	Shear	112	6.438	9.396	N/A	N/A
L11	C	Average	Shear	112	6.250	9.522	N/A	N/A
L12	C	Average	Shear	124	6.625	13.167	N/A	N/A

- Values reflect one test per beam.

* - Values reflect one test per beam.

Table 6: Carlisle Test Results

6.4.1 Visual Inspection

There was no extensive beam deterioration of the twelve Carlisle beams. The condition of the deck surface of these beams indicates that no asphalt-wearing surface was used at the Carlisle location. Limited longitudinal cracks were identified along the length of these sections and only minor concrete spalling was identified, Figure 29. Therefore, the Carlisle beams were categorized as being in "Average" condition.

6.4.2 Crack Propagation

In addition to investigating the load capacity of the beams, a crack propagation analysis was performed on one precast beam to relate flexural cracking length to applied load. As load was applied, flexural cracking was closely monitored. The first flexure cracks

appeared at an applied load, $2P$, of 60 k. Cracks were clearly marked and labeled in conjunction with loading. Crack propagation as a function of loading, $2P$, is shown in Figure 30. Load capacity, $2P$, for this beam was approximately 102 k.



Figure 29: Carlisle Beam with Minor Evidence of Deterioration

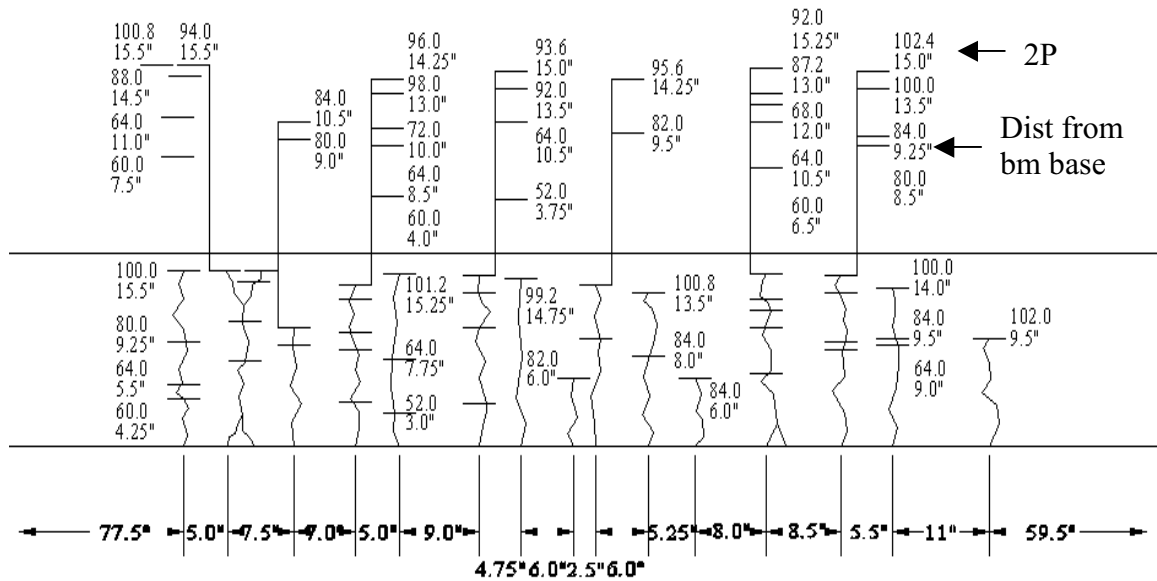


Figure 30: Crack Propagation of Carlisle Beam L4 as a Function of Load, 2P (k)

6.4.3 Elastic-Plastic Behavior

Elastic – plastic behavior was examined on three sections. The deflection was measured as a function of the applied load. The load, 2P, was applied up to 20 k and then released to 0 k. The process was repeated to attain loads of 40 k, 60 k, and 80 k. Permanent deflection was recorded after each load release. All three beams had a permanent deflection of 0.125-in when released from 80 k. Elastic-plastic behavior for beams L1, L2, and L3 is shown in Figure 31, Figure 32, and Figure 33 respectively.

6.4.4 Load Capacity and Deflection

The average load capacity, 2P, for the Carlisle beams was 102.5 k. This load capacity corresponds to a 281.9 ft-k bending moment, excluding self-weight, with an average maximum deflection of 4.3-in. Shear controlled the mode of ultimate failure. Failure

occurred quickly after a shear crack formed at approximately d , depth of beam, from the support. The load-deflection plot for all Carlisle beams is shown in Figure 34.

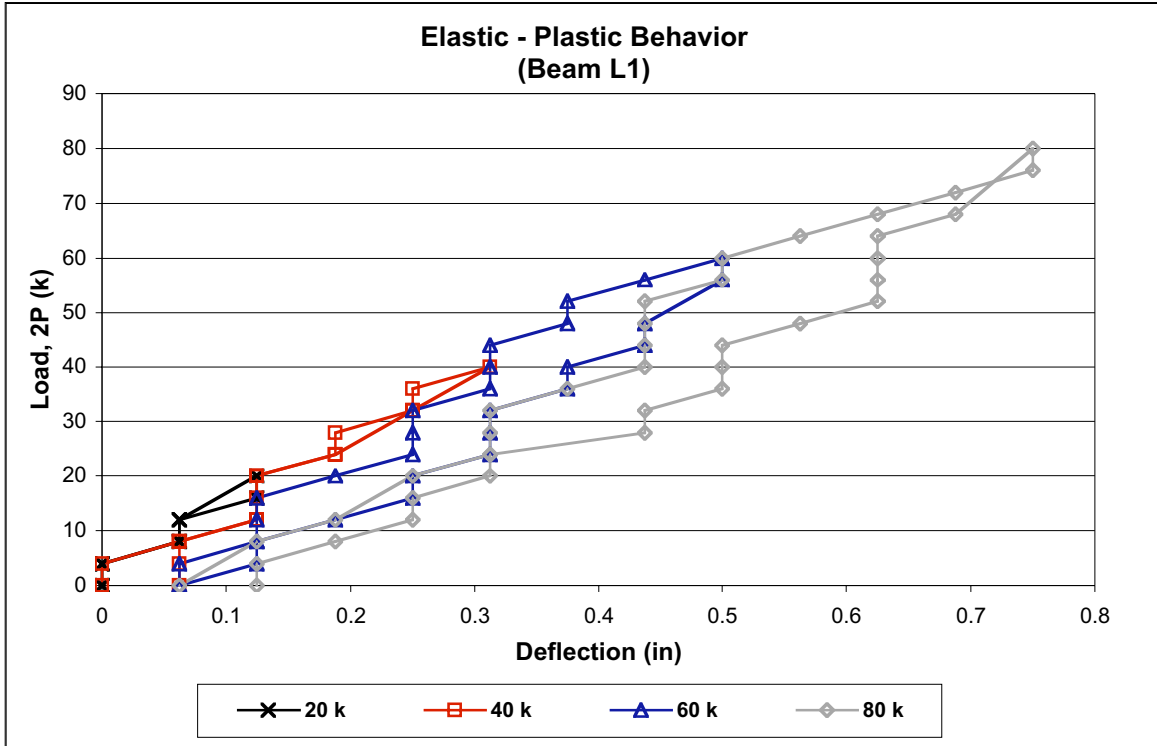


Figure 31: Elastic – Plastic Behavior for Beam L1

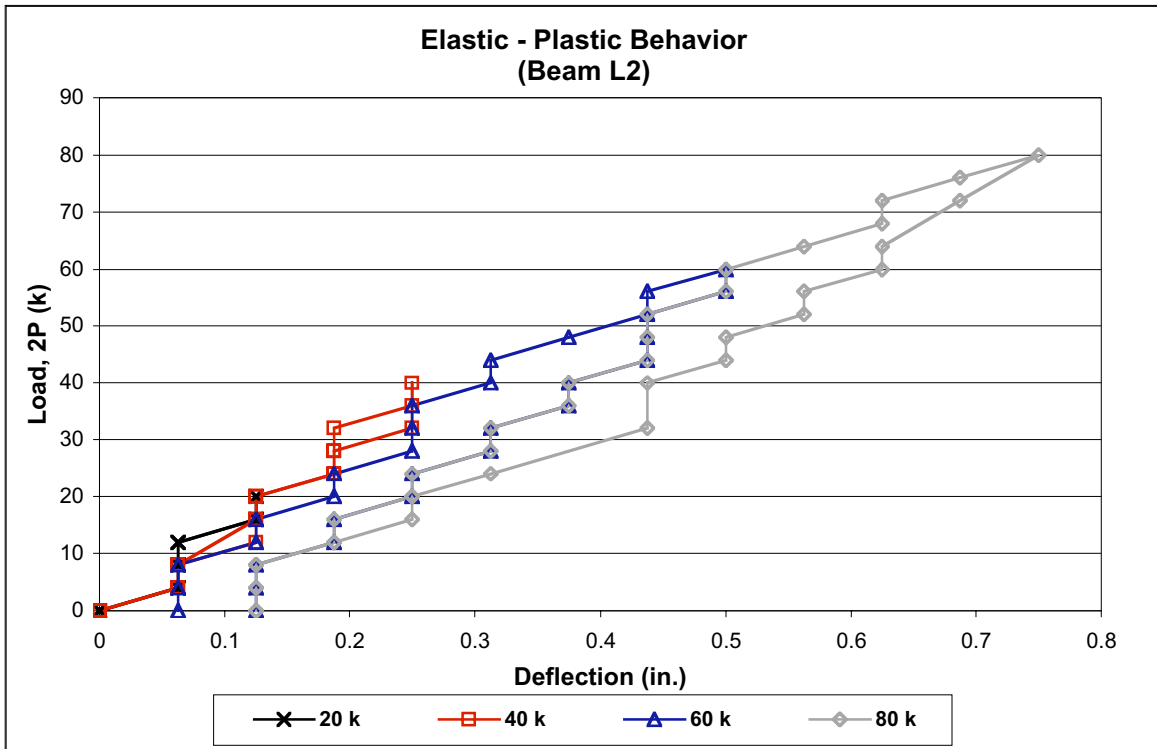


Figure 32: Elastic – Plastic Behavior for Beam L2

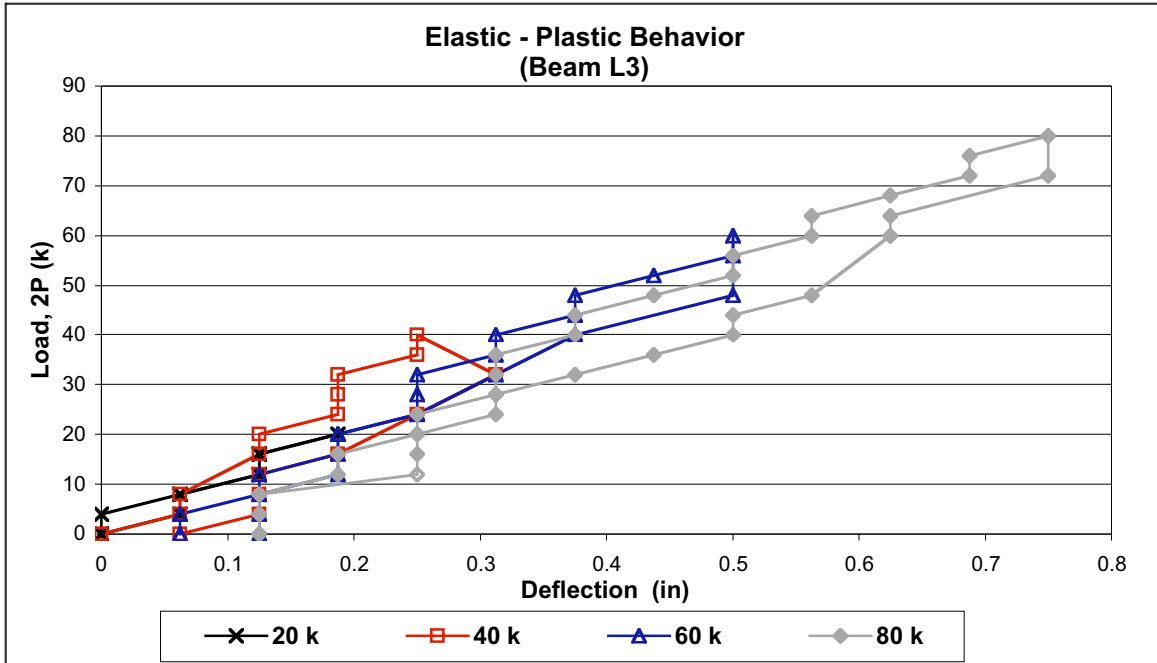


Figure 33: Elastic – Plastic Behavior for Beam L3

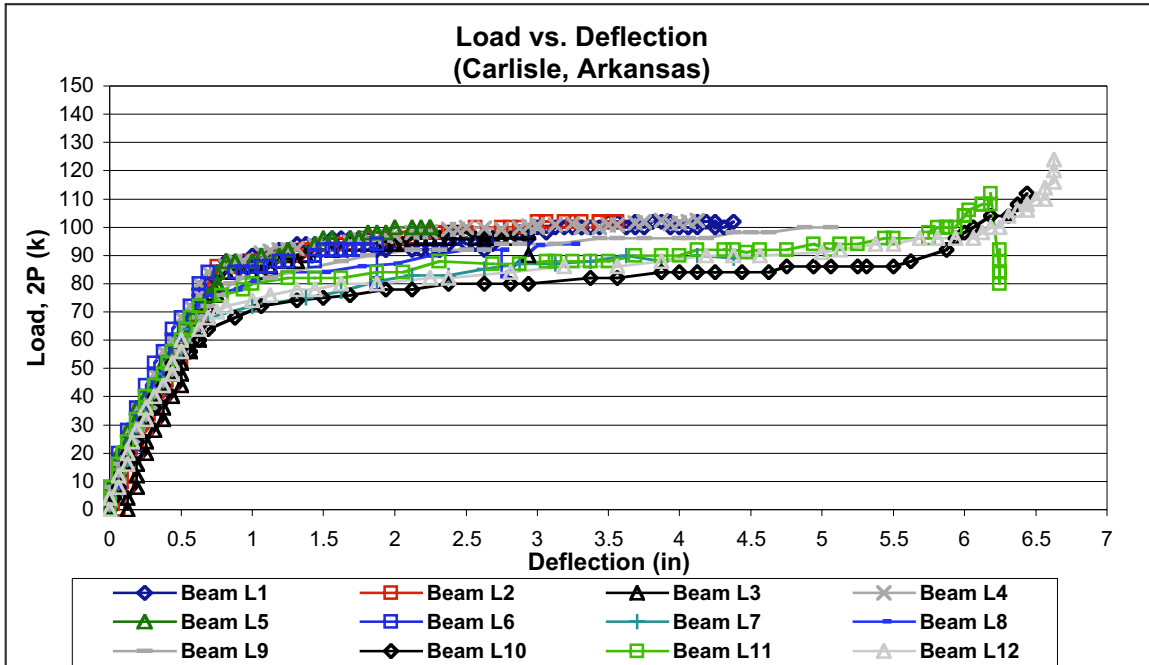


Figure 34: Carlisle Load Versus Deflection Results

6.5 New Precast Concrete Channel Beams

Four newly fabricated beams were load tested to failure. Beams N1 and N2 were made with shear reinforcement while beams N3 and N4 were made without shear reinforcement. Grade 60 steel was used for the reinforcing steel. #9 bars were used for longitudinal reinforcement and #3 bars were used for the stirrups. Table 7 shows results for load capacity, concrete compressive strength and failure mode.

Beam ID	Original Location	Beam Condition	Failure Mode	Load Capacity, 2P (kip)	Mid-Span Deflection (in)	Core Strength (ksi)	Density (kcf)	Steel Tensile Strength (ksi)
N1	N/A	New	Flexure	140	6.063	7.393	N/A	60
N2	N/A	New	Flexure	130	6.188	7.393	N/A	60
N3	N/A	New	Shear	80	1.000	7.996	N/A	60
N4	N/A	New	Shear	76	1.188	7.996	N/A	60

Table 7: New Beams Test Results

Reinforcing steel behavior was monitored using strain gauges. Gauges were located approximately 18-in, 78-in, and 114-in from each beam end. In addition, two strain gauges were placed on the shear stirrups of beams N1 and N2. A plan view of the beam showing relative gauge locations is given in Figure 35.

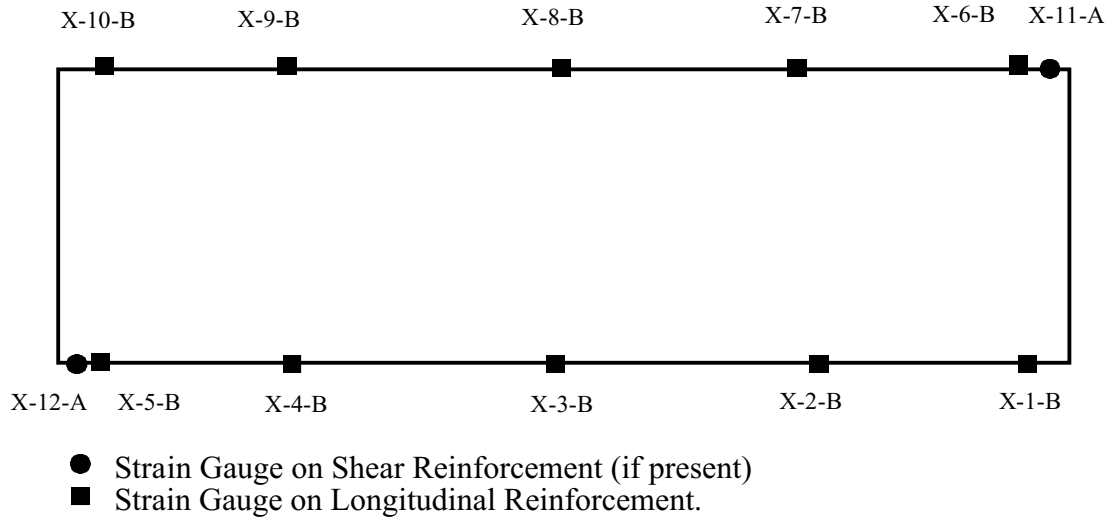


Figure 35: Plan View Showing Approximate Locations of Strain Gauges on “New” Channel Beams

Concrete cylinders were cast during beam fabrication to determine the concrete compressive strength. Three cylinders were tested for 28 day strength and averaged for each of the two concrete batches used in the fabrication process. The overall average concrete compressive strength for the newly fabricated beams was 7.695 ksi.

6.5.1 Visual Inspection

These sections showed no signs of distress. A “New” beam is shown in Figure 36.



Figure 36: Newly Fabricated Channel Beam

6.5.2 Strain Gauge Data Collection

The internal behavior of the beams along the reinforcing steel was continually monitored at the strain gauges during the load application. Results from these tests conducted on the beams with shear reinforcement, N1 and N2, are shown in Figures 37 and 38. Results for the beams without shear reinforcement, N3 and N4, are shown in Figures 39 and 40. A moving average of every 20 data points was used to compensate for noise in the data collection.

6.5.3 Load Capacity and Deflection

The load capacity values were significantly different for the beams reinforced for shear and the beams without shear reinforcement. The shear reinforced beams reached a load capacity, $2P$, of approximately 135 k with an average deflection of 6.126-in. Shear cracks developed in the shear reinforced beams at approximately a $2P$ load of 80 k, but

were constrained by the stirrups. Conversely, beams without stirrups failed at an average 2P load of 78 k with 1.094-in deflection. Based on the AASHTO nominal concrete shear strength, the predicted load carrying capacity, 2P, is 76 k. The load versus deflection plot for the “New” beams is shown in Figure 41.

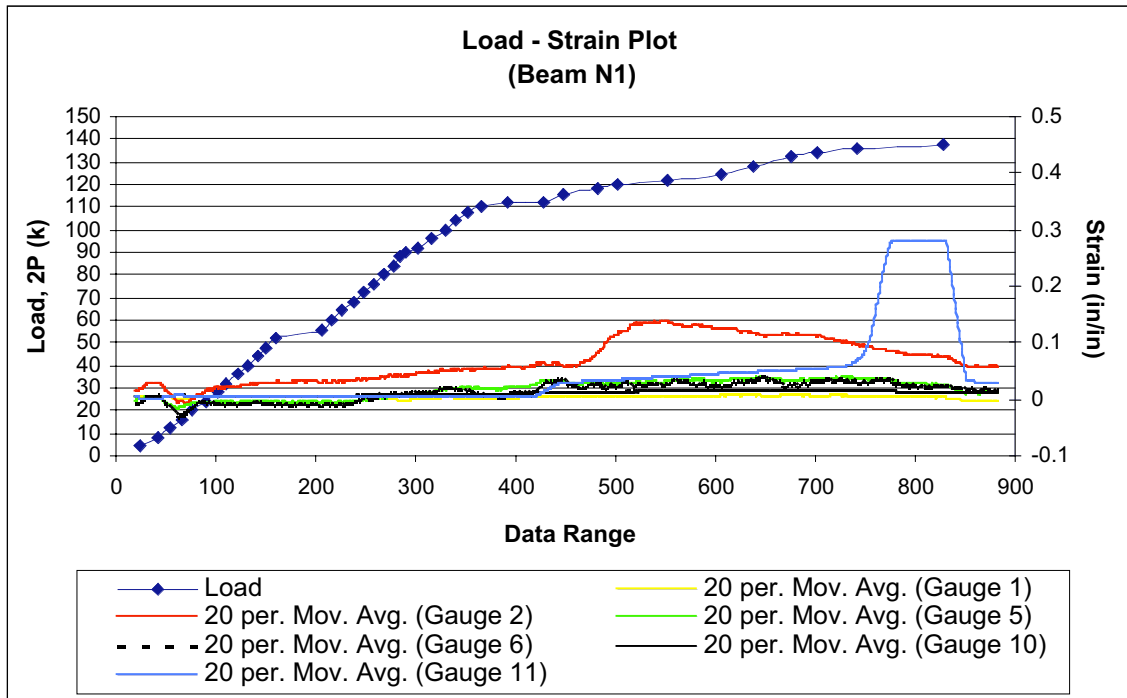


Figure 37: Load – Strain Plot for Shear Reinforced Channel Beam N1

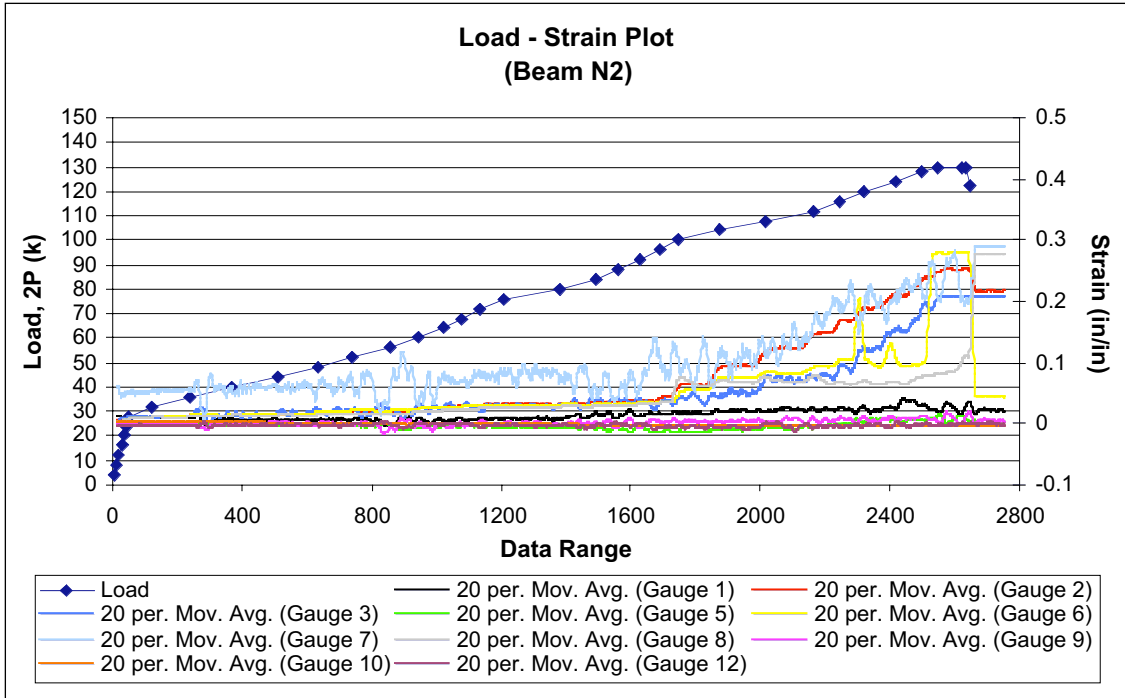


Figure 38: Load – Strain Plot for Shear Reinforced Channel Beam N2

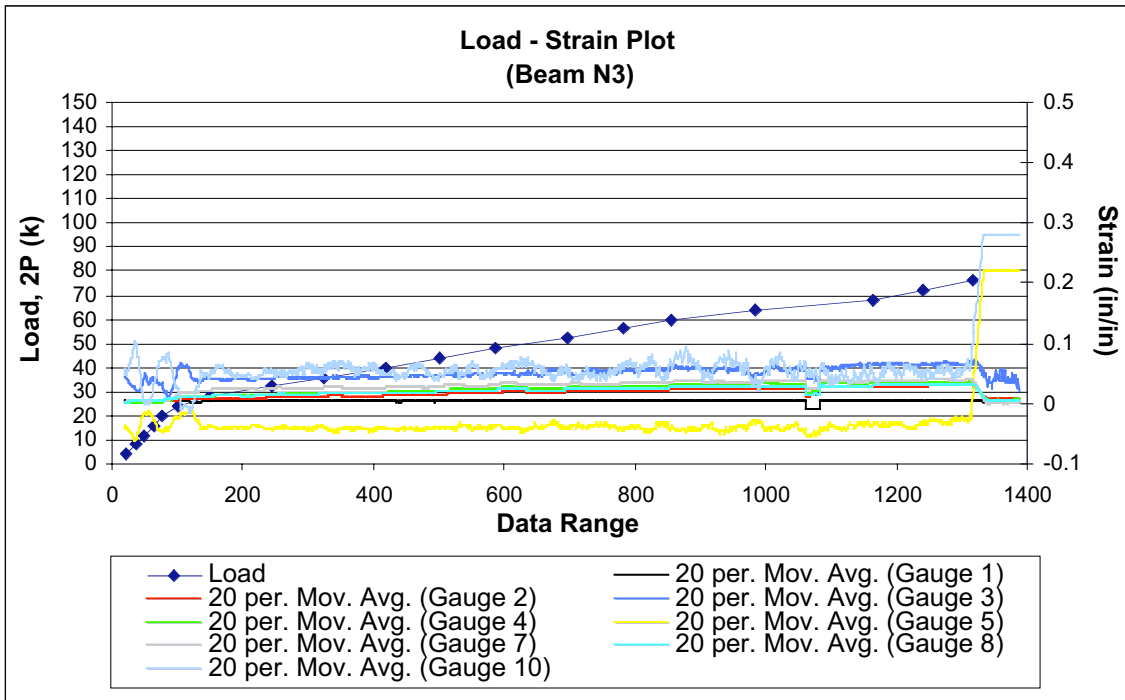


Figure 39: Load – Strain Plot for Non-Shear Reinforced Channel Beam N3

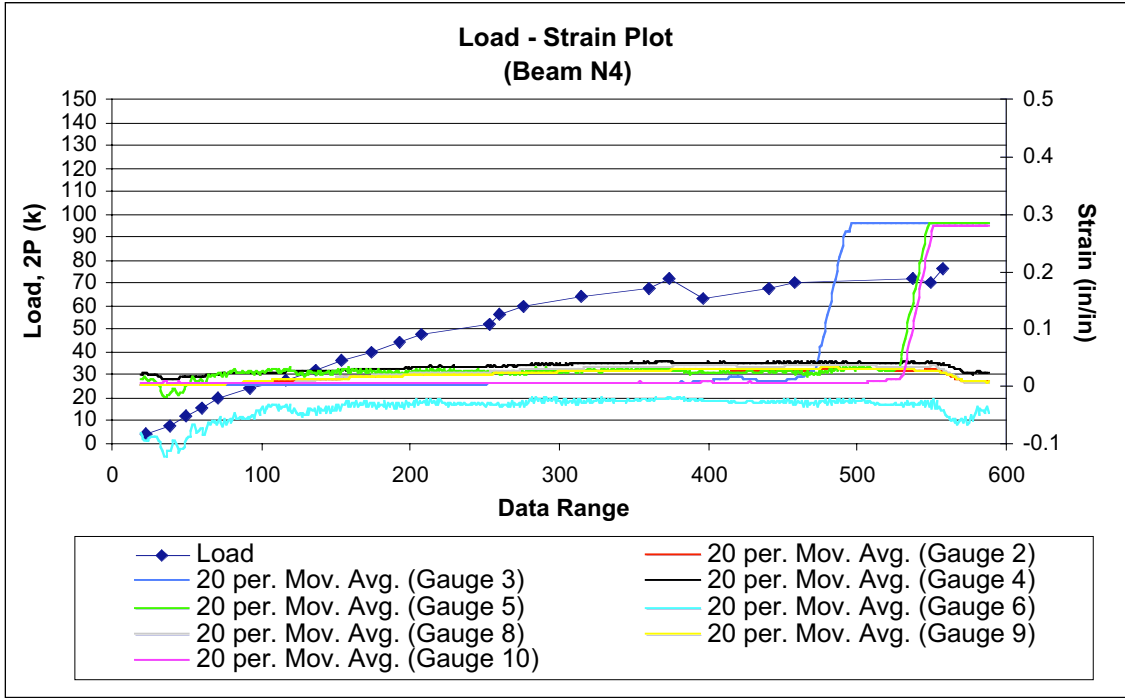


Figure 40: Load – Strain Plot for Non-Shear Reinforced Channel Beam N4

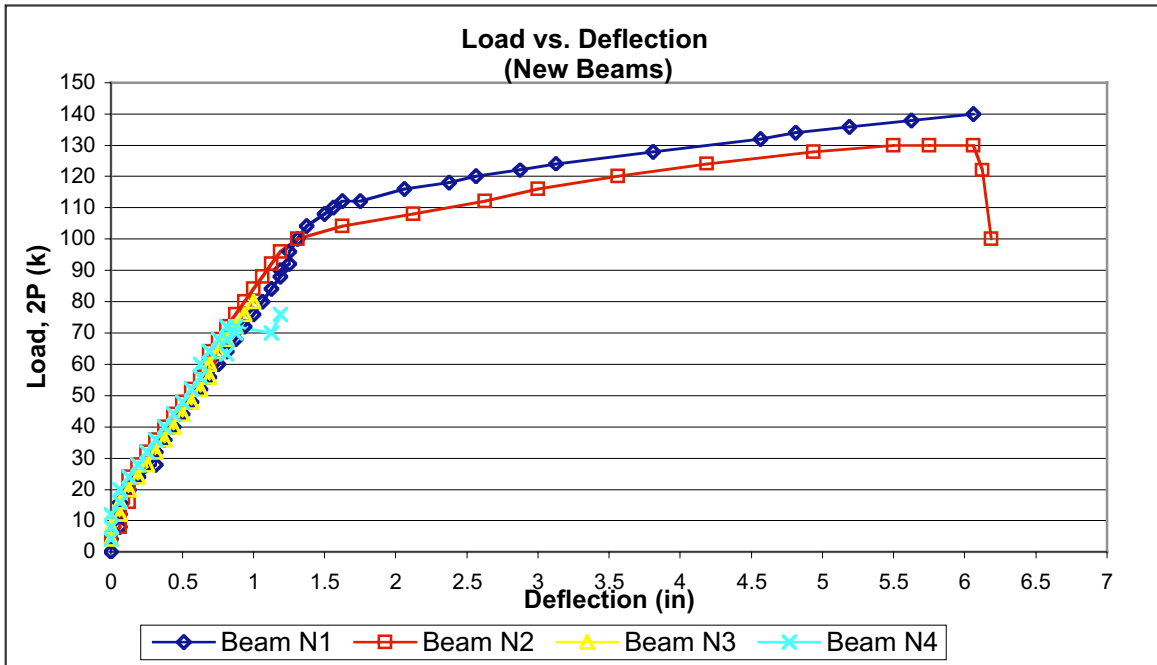


Figure 41: New Beams Load Versus Deflection Results

Chapter 7

Interpretation of Results

Thirty-seven beams were tested to determine their load capacity. The objective of this testing was to determine a correlation between beam appearance and structural strength. The results of these tests are discussed in the following text.

The load capacity of the beams from Cave Springs, categorized as “Poor”, was significantly lower than that found for sections from the other bridges. However beams from Jenkins’ Ferry, also categorized as “Poor”, compared favorably in terms of load capacity to beams categorized as “Good” or “Average”. Maximum midspan deflections of the Jenkins’ Ferry Bridge beams were similar in magnitude to the “Average” condition beams from Carlisle. Other than the “New” beams, the beams from the Gentry maintenance yard, categorized as “Good”, were the most structurally sound of the units tested. Excluding the “New” beams cast with shear reinforcing, the load capacity of the “Good” beams were the highest among those tested. Overall from the load capacity results, no direct relationship was able to be made between physical appearance and load carrying capacity, except for five “poor” beams. Each of these five “poor” beams failed in shear before yielding of the longitudinal reinforcing steel.

The “new” beams instrumented to evaluate reinforcing steel behavior during loading showed that longitudinal reinforcing steel experiences high strains only at crack

locations. Load testing “new” beams showed that when shear stirrups are added, load capacity significantly increases and is accompanied with more favorable ductile behavior.

The load capacity of all the beams tested surpassed the required strength for H15 loading.

Several factors contribute to this excess capacity. Factors include:

- Shear resistance of the concrete, V_c .
- Aggregate interlocking, similar to frictional forces caused by the irregularity of the aggregate along each side of the shear crack, V_a .
- Dowel action, the resistance of the flexural reinforcement to transverse forces, V_d .
- Arch action

When examining the behavior of beams without shear reinforcement, the primary concern is shear transfer [8]. The ability of the beam to carry additional load after a shear crack has developed is dependent upon the beam’s ability to redistribute the shear forces across the inclined crack. Without shear reinforcement, the shear strength of a section is directly dependent on the concrete shear strength. Figure 42 presents data comparing the compressive strength obtained from cores taken from the tested beams with the shear force at measured load capacity ($P + V_{DL}$). In addition, the theoretical AASHTO concrete shear strength, V_c , is superimposed on the figure as a dashed line.

Except for two sections in “Poor” condition from Jenkins’ Ferry Bridge, the measured load capacity was consistent with the calculated shear capacity of the section. The two sections that fall significantly below the superimposed concrete shear strength line were beams that had shear cracks along with excessive concrete spalling prior to testing.

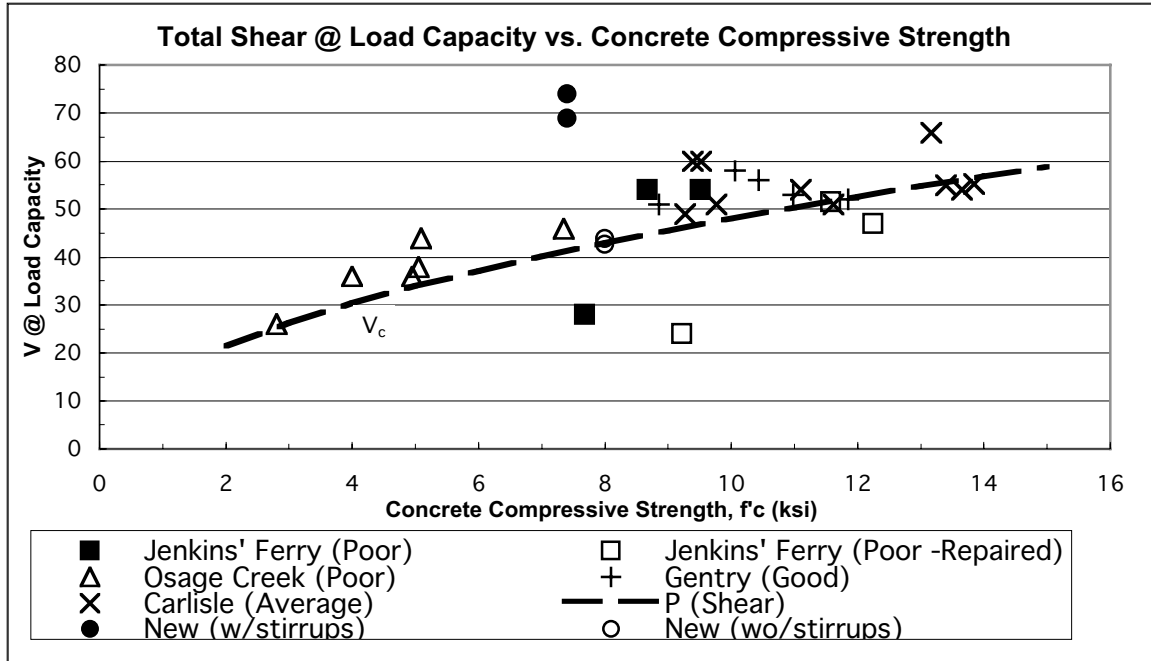


Figure 42: Load Capacity Versus Concrete Compressive Strength

Of the thirty-seven tested beams, only one “poor” beam from Cave Springs had a compressive strength less than the 3.0 ksi specified in the 1952 AHTD Bridge Design Details. The majority of the beams had a concrete compressive strength significantly greater than 3.0 ksi. Longitudinal reinforcing steel was Grade 40 and Grade 50, which is consistent with the original AHTD bridge design details for this beam section.

Minimal chloride content was found in the sections analyzed, suggesting that a chloride-based accelerator was probably not used in the initial fabrication of the beams. Therefore, if chlorides are contributing to the corrosion of the flexural steel reinforcement, it is most likely from deicing salts. Only minor evidence of alkali-silica reactivity was found in the concrete.

Chapter 8

Conclusions and Recommendations

8.1 Conclusions

This technical report investigated the structural strength of 19 ft precast channel beam sections cast without shear reinforcing steel. The report includes the material properties and the load capacity of thirty-five such beams and two additional beams cast with shear reinforcing steel. The beams were categorized according to their physical appearance where physical appearance was a subjective measure based primarily on the extensiveness of exposed longitudinal reinforcing steel. The beams were categorized in four groups: new, good, average, and poor. The poor category was further subdivided into poor and poor with shotcrete repair. The authors found that beam load capacity was primarily a function of the concrete compressive strength of the beam. Only two beams had a shear strength that was significantly less than the nominal AASHTO shear strength for the corresponding concrete compressive strength of the beam.

The lowest capacity load was $P = 20$ k. Only one beam had a concrete compressive strength less than the AHTD specified 3 ksi and this beam failed at $P = 22$ k or after including beam self-weight, 0.42 k/ft, a total shear force of 26 k. The 26 k shear force is approximately equal to the calculated shear strength for 3 ksi concrete, 26.3 k. Of the thirty-three used beams tested, thirty beams failed at a single concentrated load, P , greater than 31 k, which is equal to a total shear force of 35 k.

The beams in this technical report were initially designed for H15 loading. Assuming no load distribution between beams, one wheel line from an unfactored H15 truck plus 30%

impact produces a 16.6 k shear force, which is less than the minimum P force of 20 k. These results indicate that although “poor” beams have sustained substantial deterioration, their load capacity still exceeds the shear force of an unfactored H15 truck with impact.

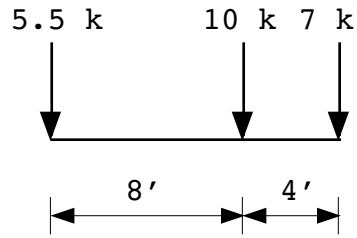
Rating factors are next reviewed for weight limit posting vehicles used by AHTD, Figure 43. Rating factors for these vehicle loads are given in Table 8 considering the lowest capacity load, P = 20 k, of the thirty-three tested beams. The shear capacity, V(CAPACITY), was taken as the minimum applied capacity load, P = 20 k, plus the shear due to the self weight of the beam, V(BEAM), of 3.97 k. The rating factors were based on the Load Factor method and calculated using a modified rating factor equation:

$$RF = \frac{V(CAPACITY) - 1.3 * V(DL)}{1.3 * V(LL + I)} \quad (1)$$

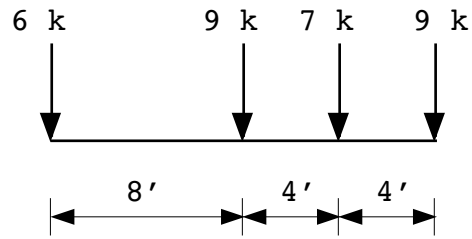
In the equation, V(CAPACITY) is the actual shear capacity of the beam from load testing and V(DL) is the combined shear due to the weight of the beam, V(BEAM), plus the weight of an assumed wearing surface, V(WS). The weight of the wearing surface was taken as 0.025 ksf.

From Table 8, all of the rating factors corresponding to the lowest applied capacity load are less than one. However, when considering the average beam shear capacity, V(CAPACITY), of the thirty-three beams, 48.59 k, all rating factors are above 1.

T3 VEHICLE (45 k Legal) CODE 4



T4 (62 k Legal) CODE 9



T3S2 (80 k Legal) CODE 9

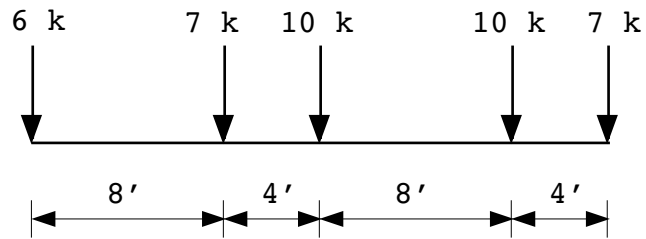


Figure 43. Weight Limit Posting Vehicles (wheel loads)

WT LIMIT POSTING VEHICLE	V (BEAM) (kip)	V (WS) (kip)	V (LL+I) (kip)	V (CAPACITY) (kip) P(LD CAP) + V(BM)	RATING FACTOR
T3 (45 k LEGAL) CODE 4	3.97	0.84	22	23.97	0.620
T4 (62 k LEGAL) CODE 9	3.97	0.84	26.88	23.97	0.507
T3S2 (80 k LEGAL) CODE 5	3.97	0.84	25.58	23.97	0.533

Table 8. Rating Factors for Weight Limit Posting Vehicles

Compressive strengths were determined for twenty-seven of the thirty-three used beams and these values are compared with their rating factors in Figure 44. Of the twenty-seven beams, six beams had rating factors below one. These six beams were taken from Jenkins' Ferry and Cave Springs, which were considered "poor" by visual inspection before load testing.

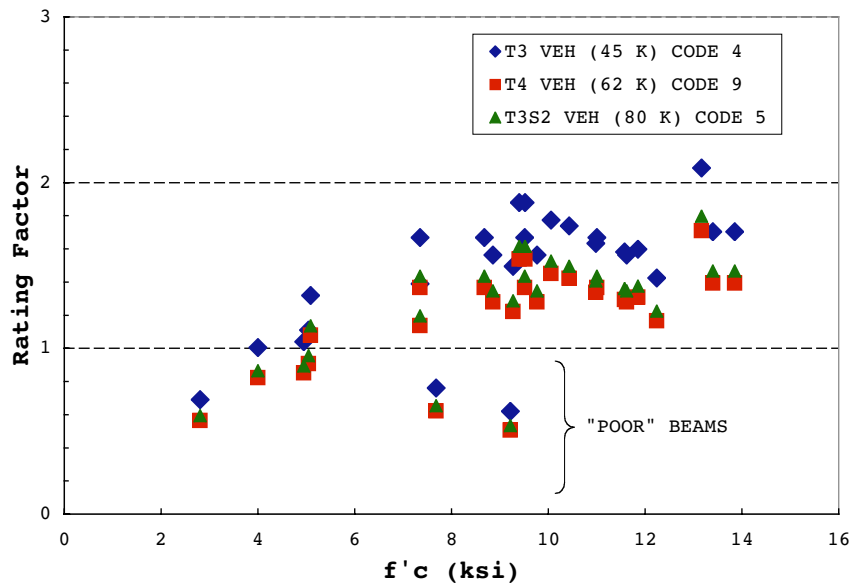


Figure 44. Rating Factor as a Function of Concrete Compressive Strength

Instead of beam strength, what is of major concern are sections that fail without ductile behavior and therefore display no warning before failure. Five of fourteen tested beams in “poor” condition failed in shear before exhibiting a large midspan deflection indicative of ductile behavior. It is this beam behavior that the authors are most concerned with, and which needs to be remedied to prevent a sudden failure of a precast channel beam section.

8.2 Recommendations

- Structural capacity of a beam can be related to the concrete compressive strength of a beam; therefore, a non-destructive testing approach needs to be developed which will allow the compressive strength of a beam to be easily determined in-place.
- Bridge beams need to be categorized according to the guidelines established in the field guide to highlight “poor” beams.
- A retrofit technique needs to be developed that can be easily implemented for “poor” beams that will guarantee ductile behavior before failure.
- The retrofit approach needs to be field tested at an existing bridge to prove its adequacy.

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Appendix A

**Survey of Precast Concrete Channel Beams
in the
Continental United States**

1. Are precast concrete beams used in the bridges located in your district? Yes
No

If yes to question one, please continue this survey.

2. What percentage of your total bridges are precast concrete?

3. What is the average age of these precast concrete bridges?

4. Have you experienced any problems with these precast concrete sections?
Explain? _____

5. If there are problems, what type of sections are they? _____

6. Are asphalt overlays popular on state highway bridges in your district? Yes No

7. If so, have you noticed more problems with these than bridges without the asphalt overlay? Yes No

8. Does your area experience a lot of snow and ice? Yes No

9. Are de-icing salts used on bridges in your district during winter weather? Yes No

10. If you do not use de-icing salts, what method(s) do you use? _____

11. Any additional comments. _____

