OKLAHOMA TRANSPORTATION CENTER

ECONOMIC ENHANCEMENT THROUGH INFRASTRUCTURE STEWARDSHIP

INVESTIGATIONS OF A PRECAST BRIDGE DECK SYSTEM

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16. ABSTRACT

Improved methods of bridge deck construction are greatly needed. Bridge decks are often the first element to require repair or replacement because of its direct exposure to the elements and tire wear. This report presents a new precast overhang system that allows for significant improvements in construction speed, economy, and safety while meeting the AASHTO requirements and providing a serviceable structure. Results also indicate that the system will allow a support beam to be removed in certain circumstances. Welded wire reinforcing were also investigated to replace tied reinforcing bars with partial depth panels to improve the economy, constructability, and construction speed of bridge decks. Bridge decks have been constructed and tested that have used tied reinforcing and welded wire mats. The improved ability of the wire mat to help resist cracking could allow an owner either greater construction tolerances for the reinforcement placement or improved crack control and hence long term durability.

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SI (METRIC) CONVERSION FACTORS

Approximate Conversions to SI Units

Symbol	When you know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	25.40	millimeters	mm
ft	feet	0.3048	meters	m
yd	yards	0.9144	meters	m
mi	miles	1.609	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm
ft^2	square feet	0.0929	square meters	m^2
yd^2	square yards	0.8361	square meters	m^2
ac	acres	0.4047	hectares	ha
mi^2	square miles	2.590	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.0283	cubic meters	m^3
yd^3	cubic yards	0.7645	cubic meters	m^3
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.4536	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	degrees Fahrenheit	(°F-32)/1.8	degrees Celsius	°C
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.448	Newtons	N
lbf/in^2	poundforce per square inch	6.895	kilopascals	kPa

Approximate Conversions from SI Units

Symbol	When you know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.0394	inches	in
m	meters	3.281	feet	ft
m	meters	1.094	yards	yd
km	kilometers	0.6214	miles	mi
		AREA		
mm^2	square millimeters	0.00155	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.196	square yards	yd^2
ha	hectares	2.471	acres	ac
km^2	square kilometers	0.3861	square miles	mi^2
		VOLUME		
mL	milliliters	0.0338	fluid ounces	fl oz
L	liters	0.2642	gallons	gal
m^3	cubic meters	35.315	cubic feet	ft^3
m^3	cubic meters	1.308	cubic yards	yd^3
		MASS		
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.1023	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	degrees Celsius	9/5+32	degrees Fahrenheit	°F
		FORCE and PRESSURE or STRESS		
N	Newtons	0.2248	poundforce	lbf
kPa	kilopascals	0.1450	poundforce per square inch	lbf/in^2

INVESTIGATIONS OF A PRECAST BRIDGE DECK SYSTEM

Final Report

July 1, 2010

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

Recent studies suggest more than 60% of the structurally deficient bridge ratings in Oklahoma are due to severe bridge deck deterioration. Because the bridges in Oklahoma and across the nation are in such dire need of improvement and the associated costs are so overwhelming, the Federal Highway Administration (FHWA) and the Oklahoma Transportation Center (OTC) have made it a priority to seek new methods to economically repair and construct bridges. What is needed is a bridge deck system that is durable, rapid to construct, and economical.

In response to this need, several recent attempts have been made to create a bridge deck system with full depth precast concrete pieces that are lifted into place with large cranes to serve as the bridge deck. These precast deck systems have been attempted in around 10 states, but have not been widely adopted for the following reasons: (i) difficulty adjusting the precast pieces to meet construction tolerances, (ii) inability to provide a smooth final riding surface without extensive grinding, and (iii) expense due to specialized equipment or materials. However, a new system has been developed by the Texas Department of Transportation, Texas A&M University, and Oklahoma State University. The system that is currently being investigated utilizes individual precast panels that are one half of the final bridge deck thickness in the interior spans and a precast panel that has a full depth and partial depth section in the overhangs and the first interior span. These panels serve as structural stay in place formwork, working surface, and support for the screed rail. A 4” topping of cast in place reinforced concrete is placed to tie the structural systems together and provide the final riding surface for the bridge deck.

There were several findings from this research project , the most important of them are:

- The precast overhang bridge deck configuration can be extended to 5’ in length. This allows the number of support beams to be reduced on a standard 30’ road width from four beams to three.
- It was determined that the amount of steel on a bridge deck could be reduced by 30% while also reducing the cracking on the bridge deck by 30%. This is achieved by using a close spacing of bars that is easy to place. In addition several key details were designed that will ease the use of these mats.

The precast overhang system has been used on the Rock Creek Bridge in Cool, Texas. The system has been in service for almost one year and TxDOT has reported satisfactory performance. Two other bridges are being investigated in which to implement the system. One of these bridges will also be using the welded wire mats as reinforcing. The ability to implement these systems in practice before the submitting the final report is a major achievement for the OTC and the research team on this project.

Preliminary estimates suggest these changes could lead to savings of over $10,000, and one week of construction time per span. Furthermore, by completing the second objective the average crack size will be reduced on a bridge deck and hence improve the durability of a bridge deck.

PHASE I– DEVELOPMENT OF A PRECAST OVERHANG FOR BRIDGE DECK CONSTRUCTION

1.1- INTRODUCTION

In the United States and internationally, there is a need for renewal of transportation infrastructure. The American Society of Civil Engineers has estimated that $190 billion is needed over the next 20 years to eliminate deficiencies in US bridges(1). It is in the best interest of society to find ways to provide durable bridge systems in an economic and rapid manner. Currently, the most costly and labor intensive element to construct on a bridge is the bridge deck. Improvements in bridge deck construction would help satisfy these needs.

In response to this need, several attempts have been made to create a concrete bridge deck system that is partially pre-assembled in a manufacturing facility (or precast) and then shipped to the construction site where construction can be completed. However these systems have not been widely adopted for the following reasons: (i) difficulty adjusting the pre-assembled pieces to meet construction tolerances, (ii) inability to provide a smooth final riding surface without extensive grinding, and (iii) expense due to specialized equipment or materials needed for construction.

After careful investigation of these challenges, a new precast bridge deck system was developed and implemented by TxDOT in Ft. Worth, Texas with the help of researchers at Oklahoma State University, Texas A&M University, and Austin Prestressed. This system has addressed each challenge by modifying the form of the precast deck panels so they contain a full depth and partial depth section. This system removes the need for all form work, provides a construction work platform, is adjustable to meet construction tolerances, and provides a support for all needed construction equipment. A 4” topping of cast-in-place reinforced concrete is then used to tie the pre-assembled pieces together and provide the final riding surface for the bridge deck.

This system has yielded drastic improvements in speed of construction, and improvements in economy are projected over modern methods of bridge deck construction in Texas. The TxDOT estimates significant savings in cost and over a week in construction time per bridge span.

This report describes the features of the system, laboratory testing, the construction of the system in Texas, and the planned improvements for the future.

1.1.1- Precast Bridge Deck Construction Techniques

One bridge element that was recognized in the 1970s that could greatly benefit from precast construction is the bridge deck. This element is repeatable and is quite costly to construct due to the labor required for formwork placement and removal, for placement of the needed reinforcement, for placement of the concrete, and for providing adequate curing. A typical conventional forming system is shown in Figure 1A.

1.1.1.1- Partial Depth Bridge Decks

In an effort to improve the economy and constructability of bridge decks several US DOTs began using partial depth prestressed precast panels as stay in place formwork. These panels were typically used in the interior portion of the span and were only half of the bridge deck depth. Next mild reinforcing steel was added above these panels and cast-in-place concrete was placed to finish the bridge(2). While these partial depth stay-in-place forms yield definite benefits over conventional construction methods the cantilever portion of the bridge deck is currently conventionally formed by using overhang brackets that serve as both formwork and a work platform. This system is shown in Figure 1B.

The partial depth system was tried in several states and has had challenges due to slow speed of overhang construction, obtaining the correct elevation of the finished riding surface, and inadequate amount of support under the panel during construction which caused serviceability problems. However, there has been an extensive amount of research on this system by the Texas DOT(3,4,5,6,7). This research found that this system if constructed correctly was able to provide an economical bridge deck system with a large amount of reserve capacity. Currently, several states use this system as a standard method of bridge construction because of the improvements in safety, economy and speed over conventionally formed bridge deck construction.

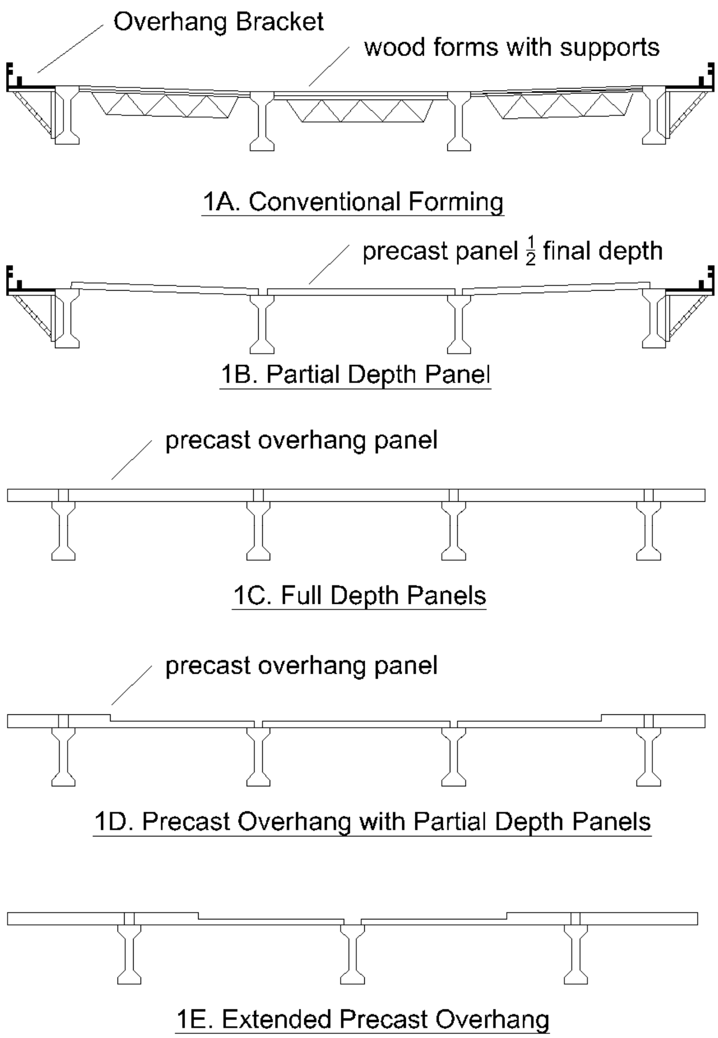


Figure 1: Display of various precast and cast in place bridge decks.

1.1.1.2- Full Depth Precast Bridge Decks

Beginning in 1985 several state DOTs (Texas, Louisiana, New York, New Jersey, Vermont) started investigating the use of full depth precast bridge deck systems[8,9]. Typically, these bridge deck systems consist of thick concrete planks that run the entire width of the bridge deck that are placed on the beams below. An example of one of these systems is shown in Figure 1C. These concrete planks are heavy and are not easy to transport or place. Once these elements are in place, they are connected with reinforcing steel and some cast-in-place grout or concrete. Some systems are then post-tensioned in an attempt to minimize the amount of cracking in the bridge deck.

There was a flourish of recent research over this topic as several states continue to investigate these systems[9,10]. One benefit that these systems have over the partial depth deck panel system is that they remove the need for the conventional forming used in the overhang construction. These systems typically use very little cast-in-place concrete or grout and require the use of several leveling bolts to obtain the correct geometry and riding surface of the bridge deck. While these grade bolts are very useful, they have proven to be challenging to provide adequate flexibility to meet the large number of different geometries required for a bridge deck. Furthermore, due to differential camber between prestressed concrete beams these systems have been found to only be useable on steel girders. This attribute has limited the use of these systems. It is often necessary to provide an asphalt wearing surface or grind the surface of the deck elements where the concrete planks interface to obtain the correct riding surface. An example of an unsatisfactory riding surface provided by one of these full depth panel sections can be found in Figure 2. While the full depth precast section has shown an improvement in speed of construction, it has also shown an increase in the cost of construction[10,11]. This increase can be attributed to large shipping weights, increase in crane size, and additional wearing surface or grinding.

1.1.2- Development of the New System

While reviewing the benefits and challenges of the full depth and partial depth bridge decks, it was realized that some features of both systems could be combined in a hybrid system that is able to achieve significant improvements over the previous systems. An overview of this new hybrid system is shown in Figure 1D and Figure 1E.

In this system, a new precast panel is used in the overhang that extends from the first interior girder to the tip of the cantilever. This precast panel is full depth from the cantilever tip until the compression zone of the exterior bay. The panel is then only partial depth until the first interior girder. Each proportion and size of the precast overhang panel was chosen for specific reasons. The full depth portion of the precast panel at the exterior of the bridge allows for the removal of the overhang forming brackets and also provides a construction work platform and area for the safety rail.

Figure 2: A wooden stick placed at the intersection of two full depth precast panels that have been adjusted using grade bolts. The difference in panel height is over ¼”.

Pockets in this full depth section are used to provide a connection between the precast panel and the exterior girder. Grout is used to fill the haunch area and concrete is used to fill the pockets. These grout pockets also provide a location for the screed rail to be attached to the bridge deck. These panels also have special inset areas in the full depth section to allow for a connection to be made between panels and for grade bolts to be used for altering panel geometry. In addition to this panel, a novel adjustable haunch gasket was developed to be used with this system. This haunch forming system is made with low density polyethylene foam that is glued to the top of the girder allowing it to compress or expand as the grade bolts are adjusted in the precast overhang panel. A detailed summary of the precast overhang element can be found in Figure 3 and Figure 4.

For the interior bays, the partial depth precast panels are used. After the geometry of the precast overhang panel has been established with the grade bolts, the reinforcing steel in the interior span and between panels is placed and concrete is used in the partial depth section. Finally, the haunch of the exterior girder is grouted and then the pockets are filled with a low shrink concrete mixture. The traffic rail for the bridge is then completed, and the deck is finished. A pictorial explanation of the construction process is shown in Figure 5 thru Figure 14.

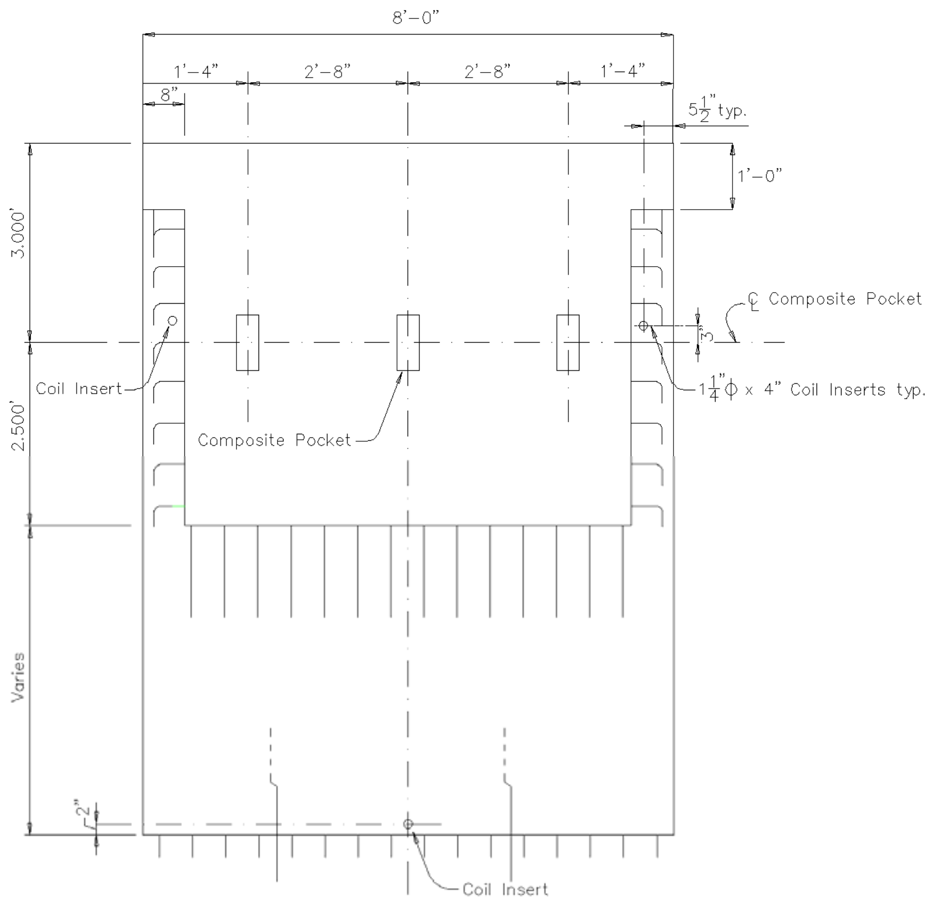


Figure 3: A plan view of the precast overhang panel showing dimensions.

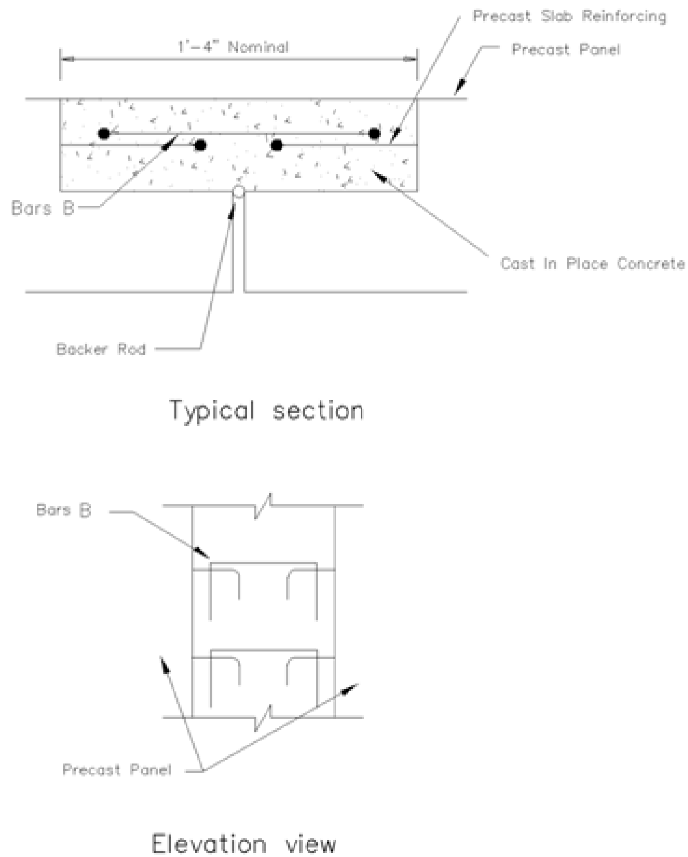


Figure 4: Connection details between the precast overhang panels.

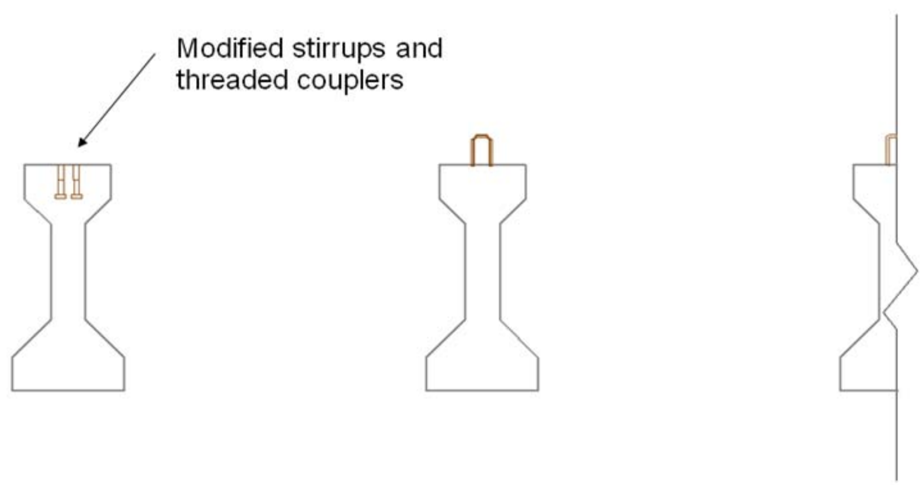


Figure 5: The beams are erected on the bents. A shear connector is used on the external beam for load transfer.

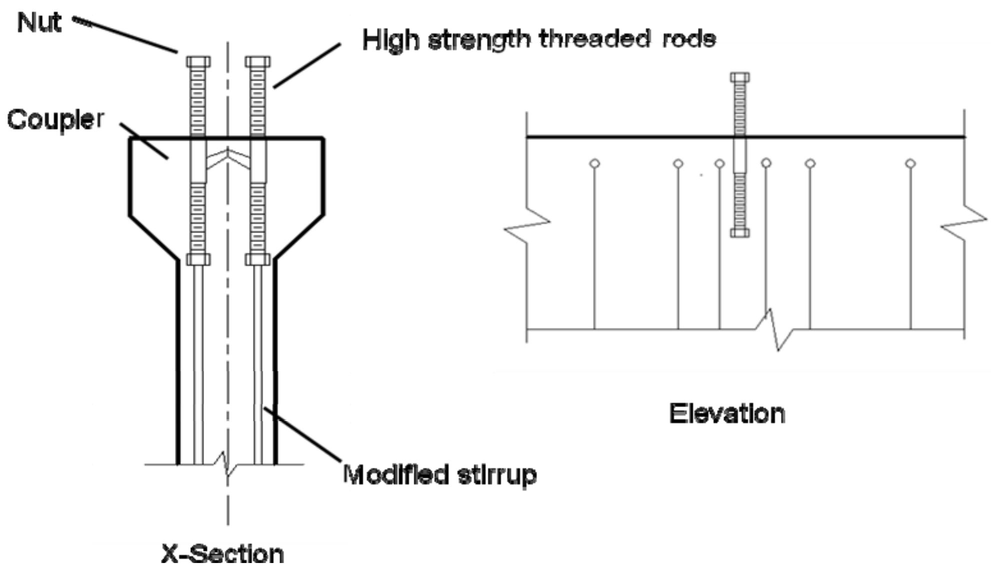


Figure 6: Structural details for the modification of the external beam.

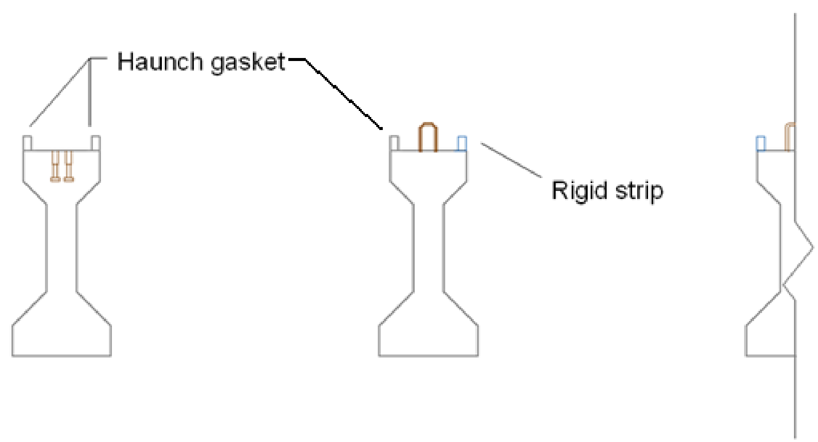


Figure 7: The haunch gasket is glued to the external girder and the outside face of the interior beam.

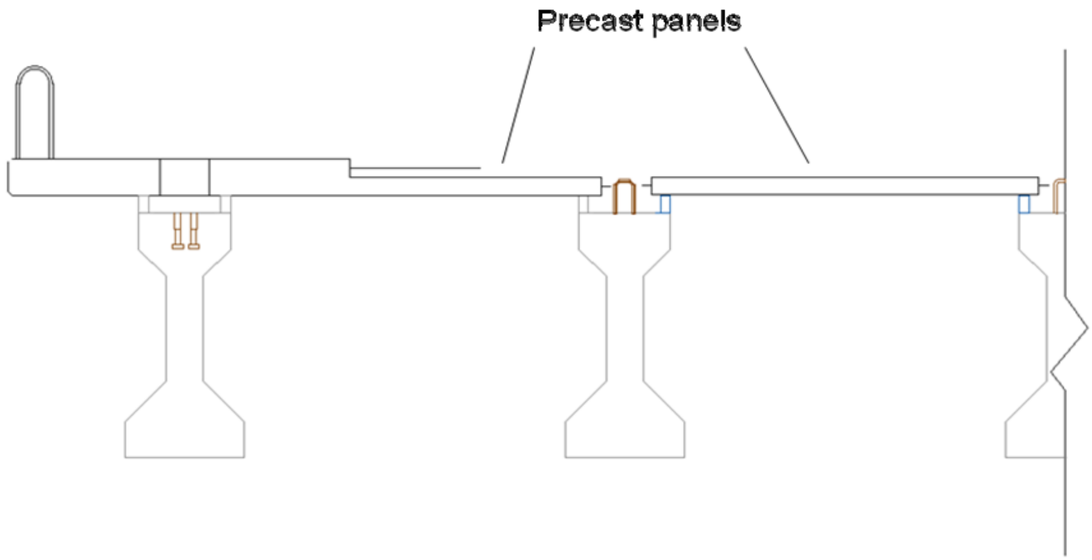


Figure 8: Precast panels are then placed. Precast overhang panels are used in the exterior bay and partial depth panels in the interior bays.

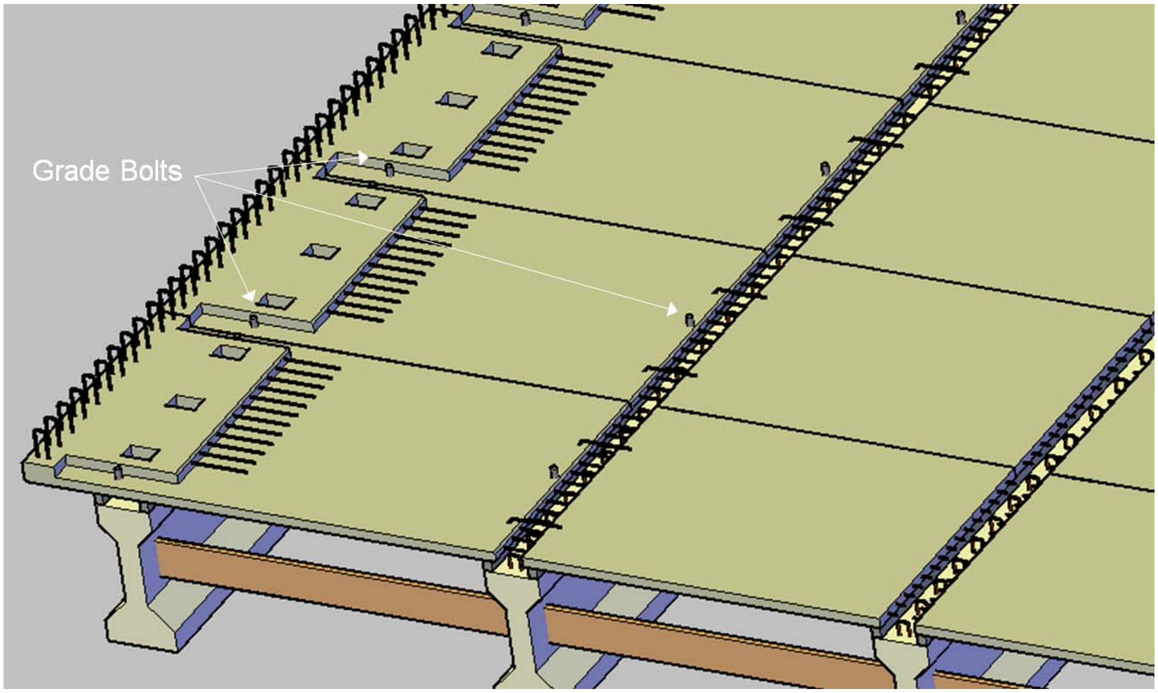


Figure 9: Grade bolts are adjusted in the overhang panels to the desired grade.

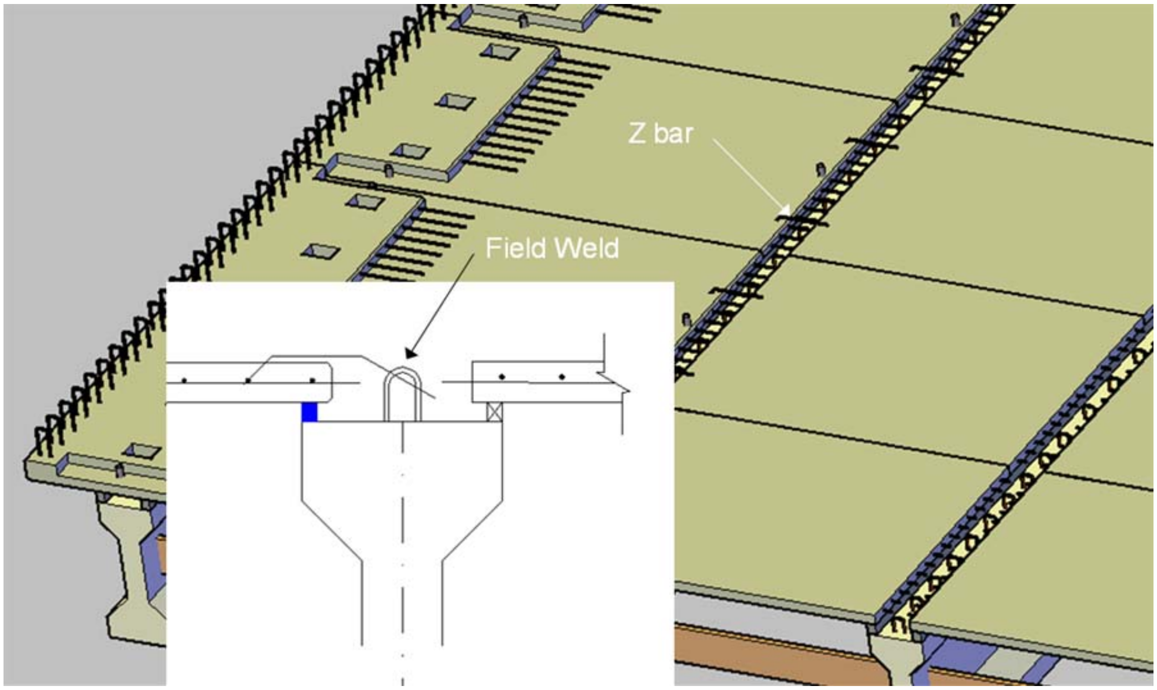


Figure 10: The external rebar is a failsafe bar that is bent down and welded to the stirrups of the first interior beam to prevent overturning.

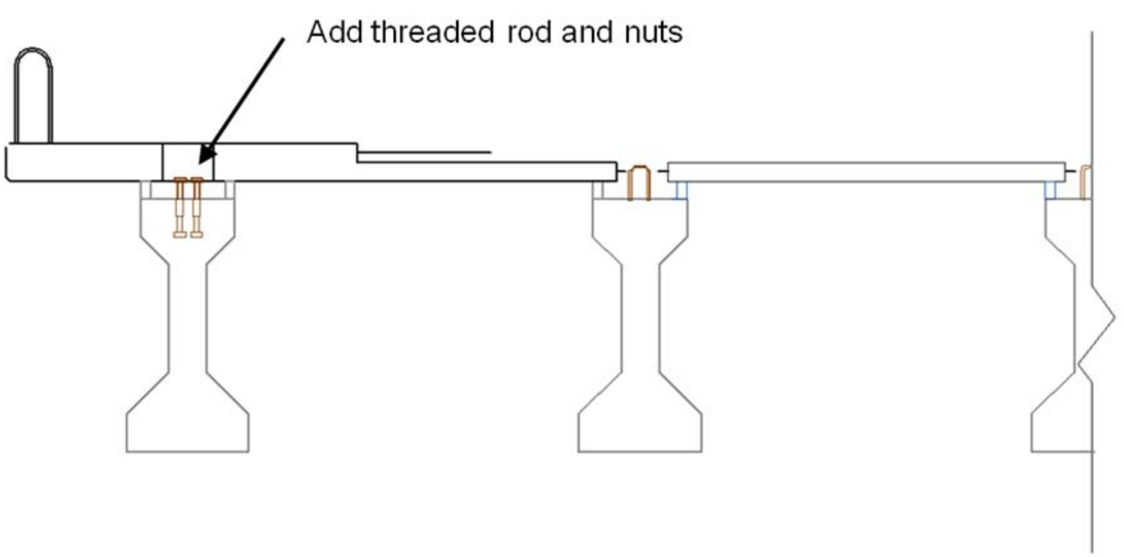


Figure 11: Threaded rods and nuts are added to the grout pocket of the external beam. This step could be carried out before the placement of the overhang panels.

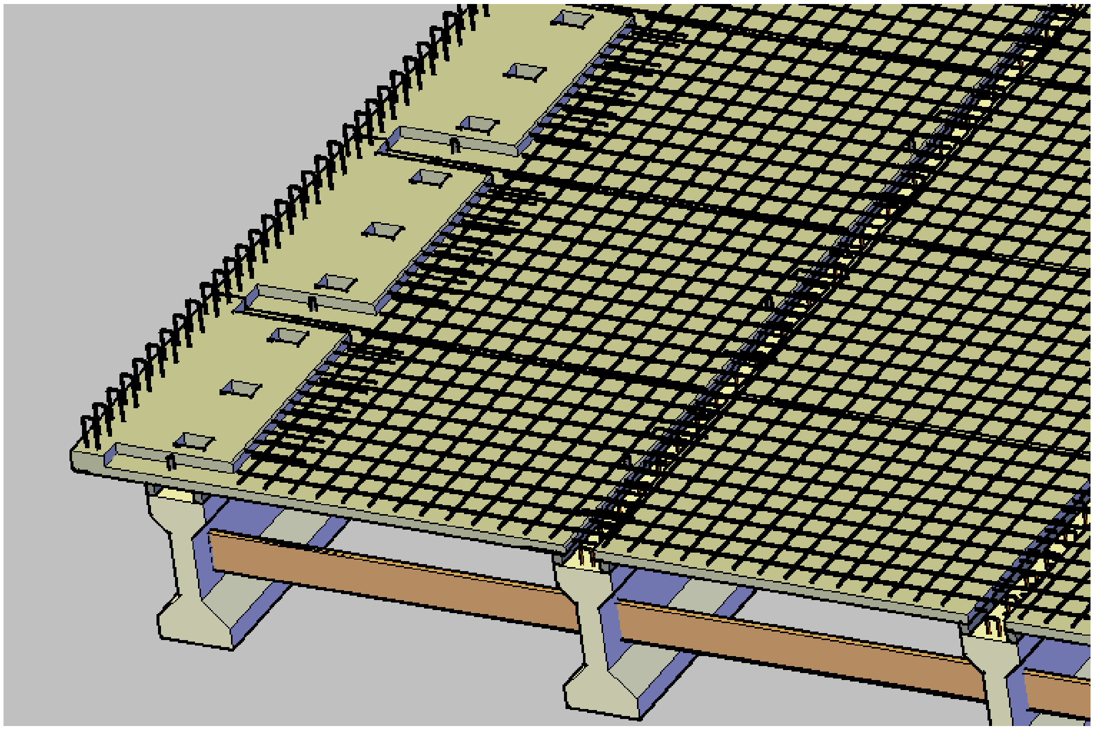


Figure 12: Rebar is placed above the partial depth portions of the deck.

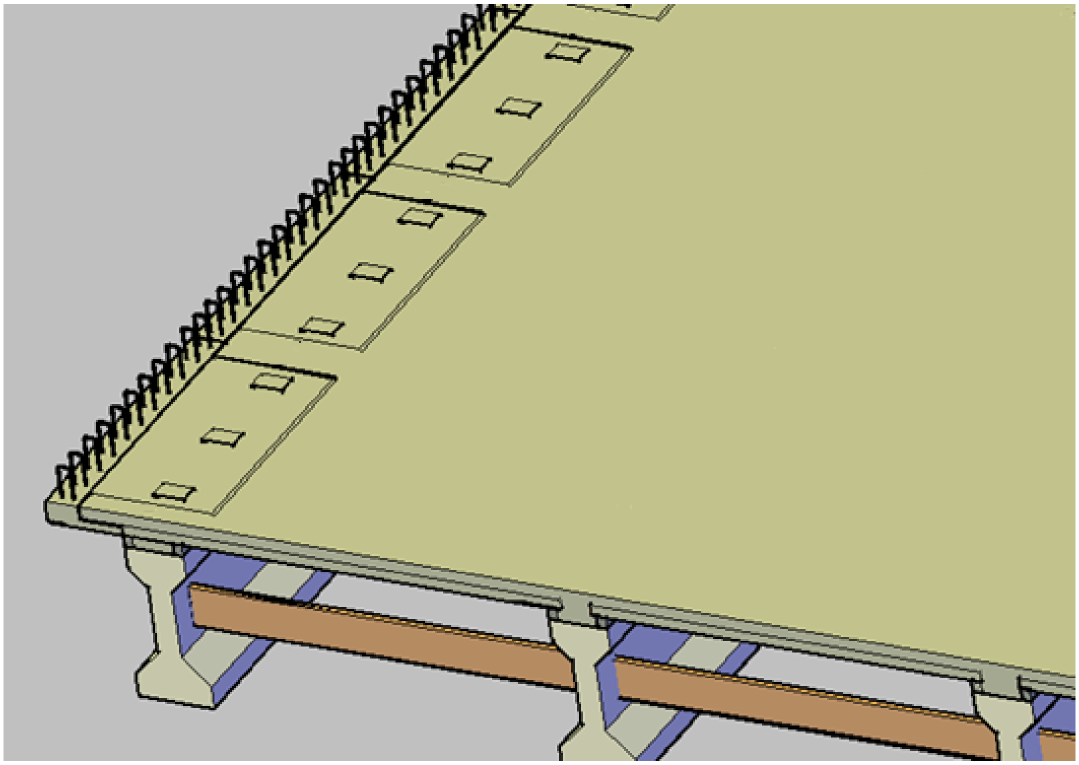


Figure 13: Concrete is placed to tie the precast system together. The haunch of the external girder is filled with grout, and then the composite pockets are filled with concrete.

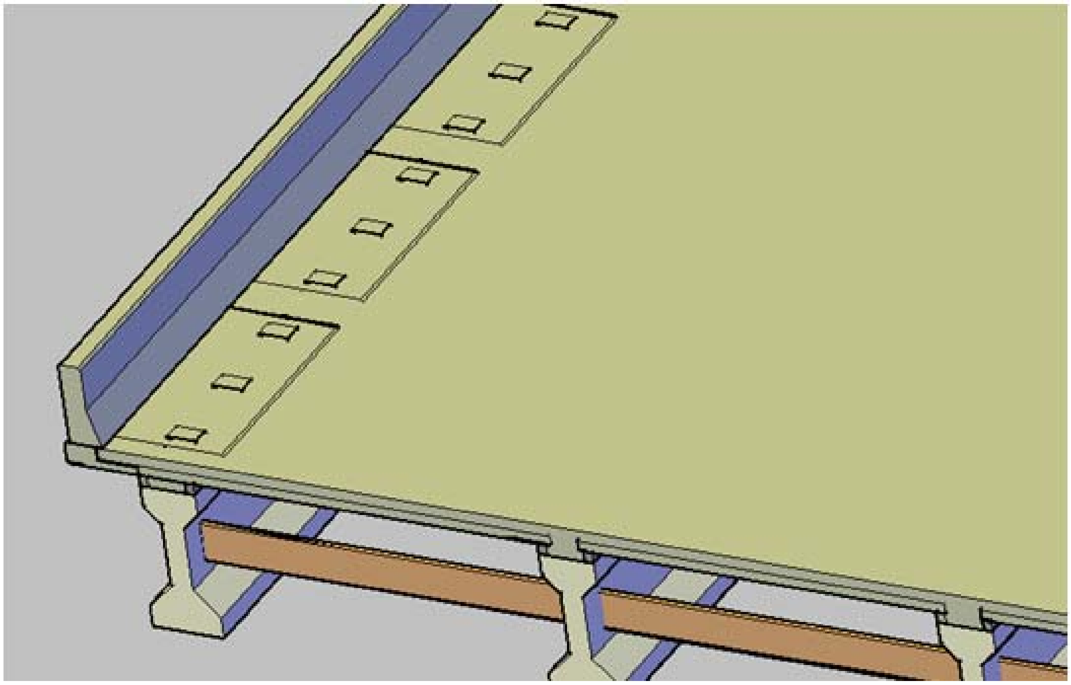


Figure 14: The concrete barrier is constructed by either slip forming or conventional forming.

1.1.2.1- System Attributes

As stated previously, this bridge deck system was specifically designed to combine advantageous features from the partial depth bridge deck with the full depth bridge deck systems in such a manner as to address the challenges of both systems.

This system specifically adapted the full depth section of the bridge deck in the overhang portion as it eliminates the placement and removal of formwork for the overhang and the work platform that is required with the partial depth panel system. Furthermore, this full depth length was sized to create a significant work platform for the screed rail and the construction workers to hand finish the external areas of the bridge deck. The precast panel is designed to be continuous over the exterior girder and extends to the first interior girder to provide a stable support for the panel. Incorporated into the precast overhang panels are threaded inserts for installation of the columns for the contractors hand rail/fall protection system. This allows fall protection to be installed concurrently with the overhang units. While almost any system can be accommodated, the inserts for the Rock Creek bridge were cast into the top slab approximately 3" in from the outside edge (inside the concrete traffic rail footprint). This location negates the need for any patching after the temporary hand rail is removed as the rail concrete covers the inserts.

Grade bolts were used in the precast overhang panel to obtain the desired riding surface, like they are used in full depth bridge deck construction techniques. However, the precast overhang system only requires three grade bolts at the exterior bay, as this is the only full depth portion of the bridge deck. By using a set of non-continuous precast panels, it allows the system to avoid the past challenges that other full depth precast members have seen where construction tolerances from differential beam deflection have caused the need for grinding or an overlay as shown in Figure 2.

One other benefit that may not be obvious is the simplification of the bridge deck construction. When the full depth portion of the precast panel is placed on the exterior beam, it is placing almost the entire dead load on the outside girder before the placement of the remaining cast-in-place concrete. The placement of this dead load on the external girder insures that the height of the bridge deck established by the grade bolts for the full depth section will be very close to the final height of the bridge deck. The reason for this is that no additional dead load deflection will occur. This allows the construction engineer to directly establish the roadway profile to match the desired elevation and ensure that all concrete cover requirements are met. Currently, there are numerous challenges to provide the correct ride and reinforcement cover with partial depth panel systems as one must accurately determine the deflection of the bridge deck from the placement of the fresh concrete. This is often challenging due to the complex construction geometry and differential beam deflection, especially in the cast of precast concrete girders. Again, because of the preloading of the external beam this is not a problem with this system and the desired bridge deck height can be directly established with the grade bolts.

1.2- TESTING METHODS

1.2.1- Specimens

The specimen layout can be seen in Figure 15. Each of the tested slabs was 8.25” thick and 8’ x 18’ or 8’ x 22’ planar dimensions. The slabs were supported on three girders spaced at 6’ center to center with 3’, 5’, or 5’-8” overhangs. The testing setup was restrained at the center beam by using post-tensioned bars and load was applied in the cantilever as shown in Figure 15. The supporting girders were 1’ wide and 1’-2” high and made of reinforced concrete. The 1’ width was chosen to mimic a small but still reasonable flange width for a prestressed or steel support beam.

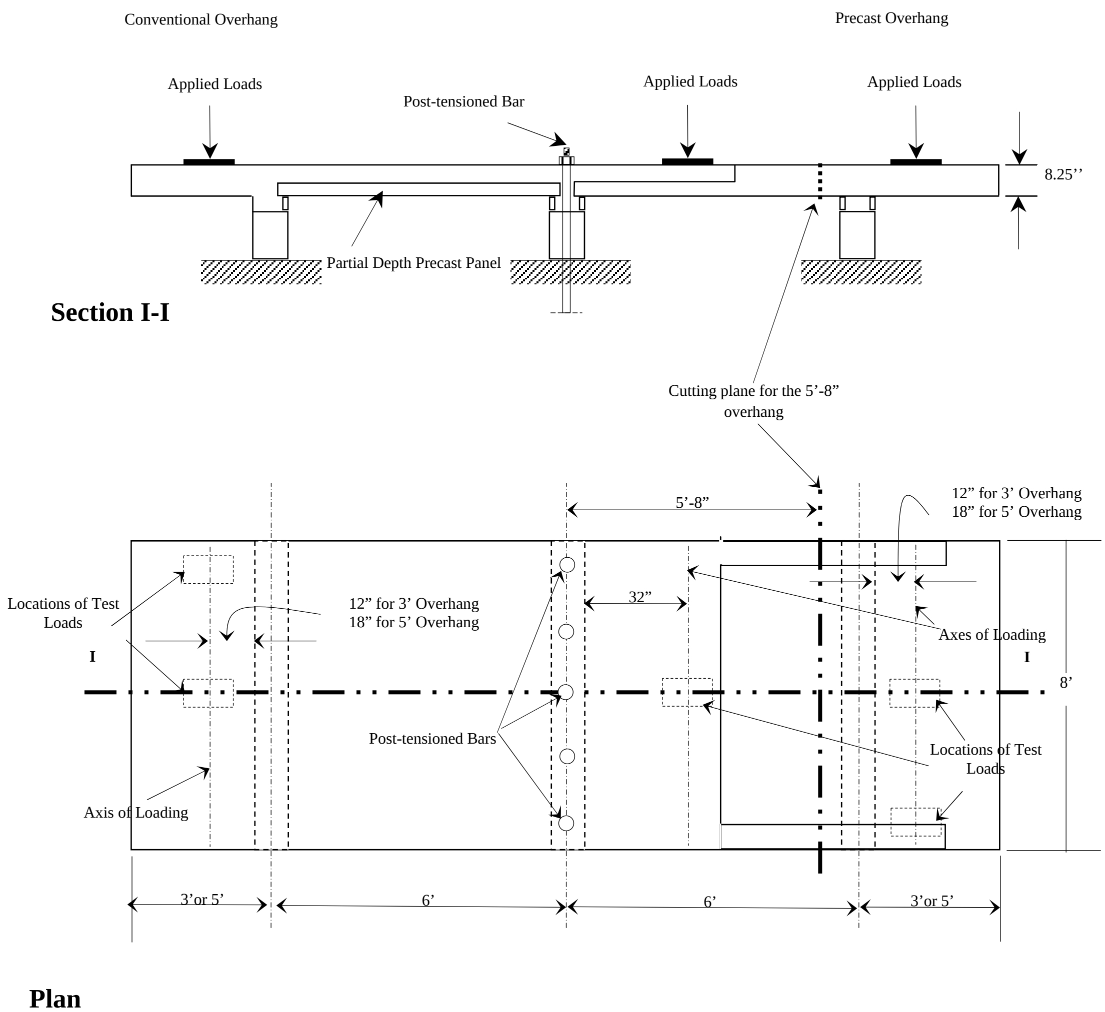


Figure 15: Test specimen: Typical Overall layout

The novel precast overhang system has prestressing strands in the transverse direction and mild steel in the longitudinal direction in the bottom layer and mild reinforcing steel in both directions in the top layer. This layout was chosen so that the existing forms for partial depth precast panels could be used to construct the bottom portion of the precast overhang panel. The opposing cantilever was made with cast-in-place (CIP) concrete and had mild steel in both the top and bottom layers. Reinforcement details can be found in Figure 16 and Figure 17.

A 4" partial depth precast panel was used for the interior span that received a 4.25" topping of concrete with mild reinforcement in both directions. This specimen construction style allowed investigation of the performance of each side independently with a minimal behavioral interference; and hence gave the chance to compare the strength and stiffness of both structural systems by using a single specimen. By restricting the bridge decks to these sizes it forces all load transfer to be made in the 8' width of the specimen. In addition this specimen construction style allows for the CIP concrete used for both specimens to be as similar as possible between the tested specimens as they were from the same concrete mixture and were placed at the same time.

The authors recognize this test protocol does not mimic the actual performance of a bridge deck; as the support beams on the ground are continuously supported. Furthermore, the center beam is restrained at the center. While the supports are different than actual practice, both systems are evaluated with equivalent support conditions; therefore the results from the testing are comparable. With this support condition the specimen response are conservative when compared to bridge decks in the field. This is because this test setup did not allow the beam supporting the cantilever to deflect and would therefore not allow load to be shed to other parts of the bridge. The 6' beam spacing used in the testing was chosen because it is a reasonable beam spacing for prestressed bridge construction and it allowed the specimen to be tested with the facilities available. The results from this testing would not be expected to vary with the spacing of the interior beams but would vary with changes in the cantilever length as investigated in the testing.

Precast elements were created by Austin Prestressed of Austin, Texas. The cast-in-place concrete for the specimens were from a local ready mix company and the grout used to fill the haunch of the system was Sika 212™. The grout was mixed by the research team.

Typical reinforcing details used in this study are given in Figure 16 and Figure 17. Reinforcing bars consisting of #5 bars at 6" spacing transversely and #4 bars at 9" longitudinally were used in the top mat of steel. A lap splice was used at the interface between the precast panel and the CIP concrete topping. The partial depth precast panel reinforcing was 3/8-in diameter, stress-relieved, Grade 270 prestressing strands at 6-in centers in the transverse direction and 0.22 in^2/ft of welded wire mats in the longitudinal direction. The specified prestressing force during casting was 16.1 kips per strand. This prestressing force was 54% of the general ultimate tensile strength for the strand. This value matches the requirements by the Texas Department of Transportation in precast panel construction. The bottom layer of steel in the cast-in-place overhang consisted of #4 bars at 1'-6" centers for the majority of the specimens. One specimen was constructed with these bars at 6" centers. One would not expect that this change would have an impact on the results since this bar was in compression. During the construction of the precast overhang panels by Austin Prestressed the reinforcing bars in the top of the slab were inadvertently switched for the 3' overhang corner testing. After the error was discovered it was decided to use this same reinforcing detail throughout the top layer of reinforcing in specimens 1 and 2. This change in height of approximately 0.5" is estimated based on flexural failure to reduce the ultimate strength of the specimen by approximately 10% and would be expected to reduce the cracking resistance of the specimen. This change is shown in Figure 18.

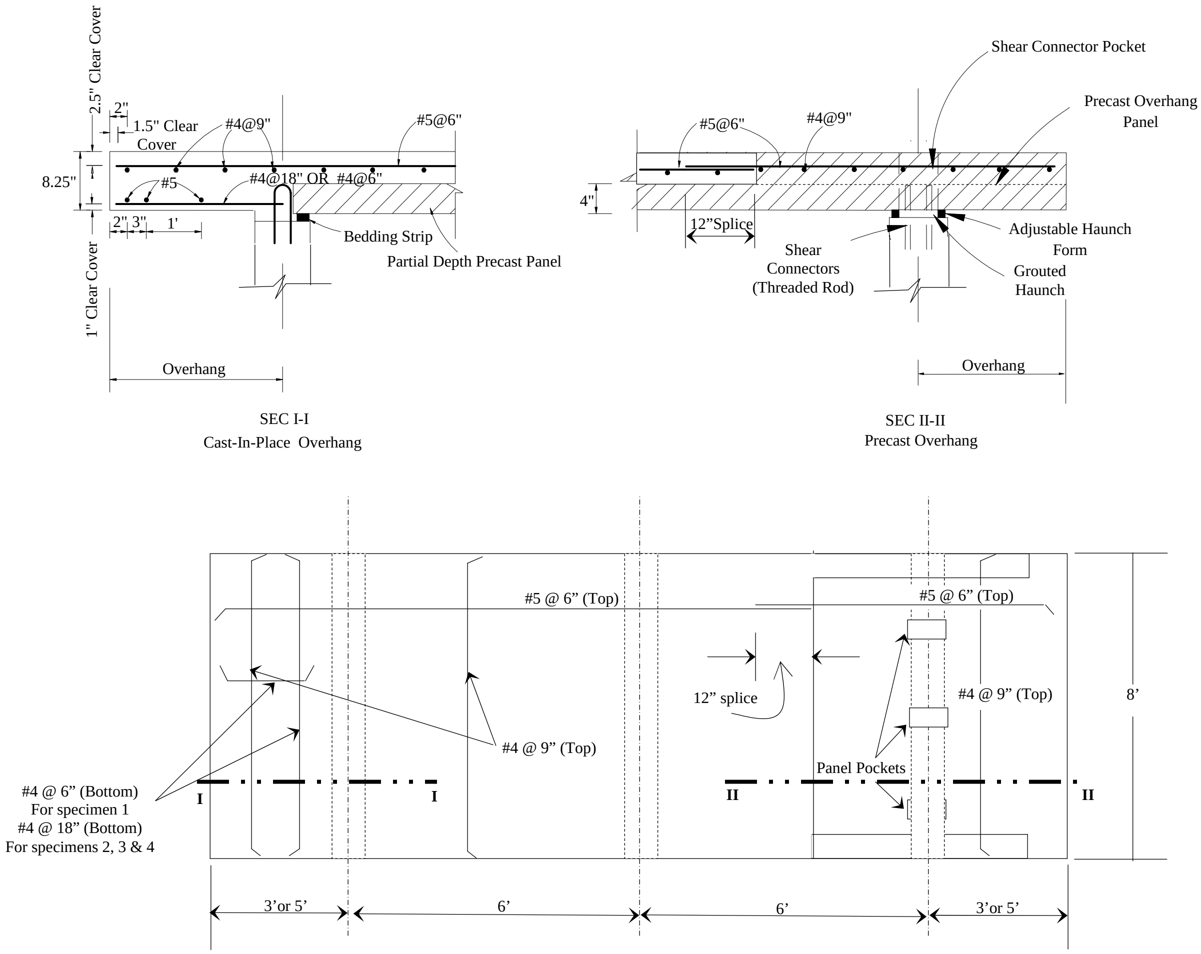
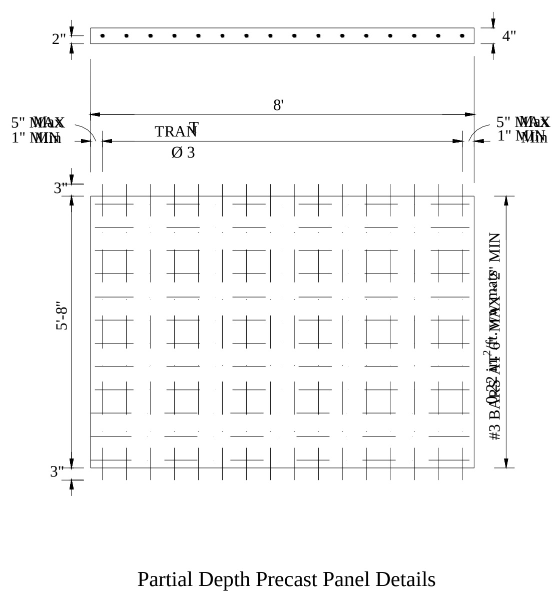
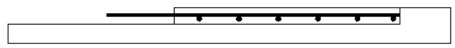


Figure 16: Bridge decks reinforcement details



Partial Depth Precast Panel Details

Figure 17: Precast panel reinforcement details



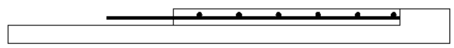


Figure 18: The intended detail and the detail actual used in the 3' overhang specimens.

1.2.2- Test Set-up

The two cantilevers of each test specimen were tested by either loading at the specimen center or at the corner by applying concentrated loads with hydraulic rams as shown in Figure 19. On the final specimen after both cantilevers were tested, a cut was made just to the inside of the external beam, as shown in Figure 19d, to create another cantilever to be tested. This cantilever was cut so that it had a span length of 5'-8". This specimen was then tested. For each test a 10" x 20" steel plate was used to represent an AASHTO HL 93 tire patch. The edge of the tire patch was placed at 1'-2" away from the face of the cantilever.

These loading conditions were chosen to simulate an HL 93 truck traveling at the very edge of the guard rail at midspan and where the bridge deck terminates such as at the approach slab. For the cantilevers of 3', 5', and 5'-8" this lead to an eccentricity of 12", 18" and 32" respectively. It should be mentioned that when loading the conventional side midspan loading of the 3' overhang that the load area HL93 AASHTO tire patch was inadvertently rotated 90^o. The correct loading orientation was used for the remainder of the specimens. This modification should be conservative as the midpoint of the load is in the same point but the clear distance between the edge of the plate and the edge of the beam was increased by 5".

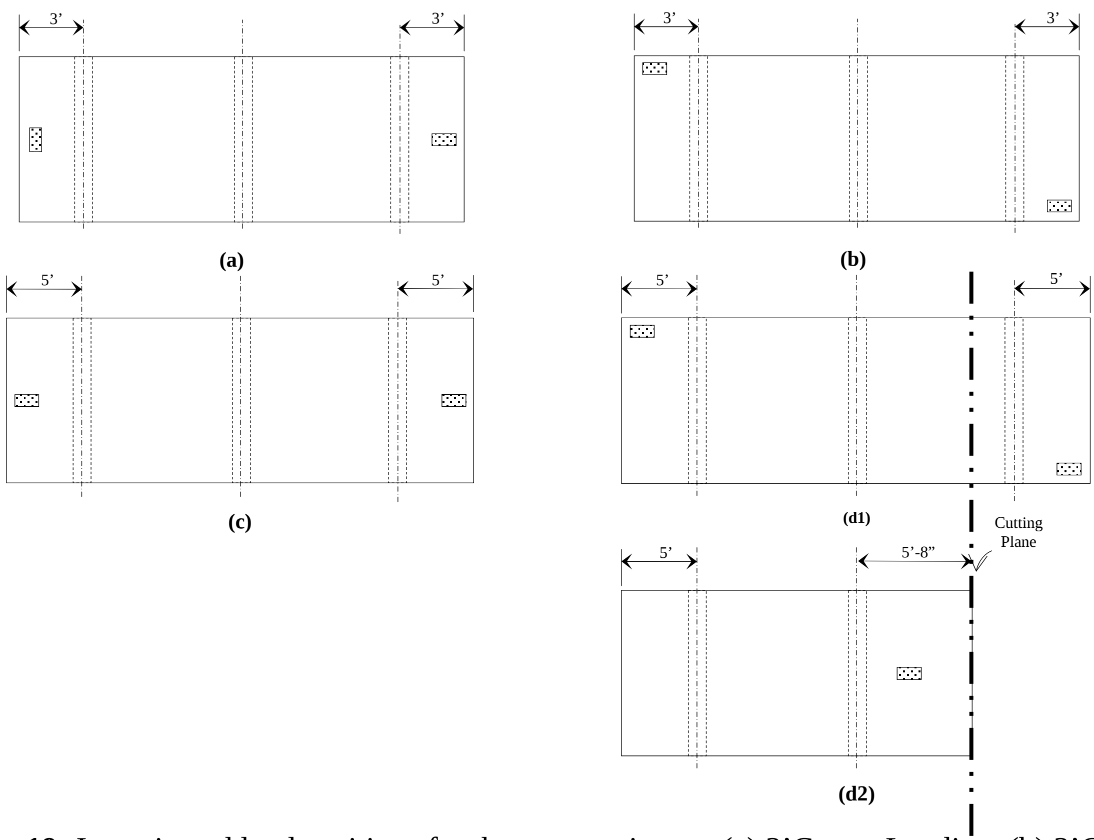


Figure 19: Investigated load positions for the test specimens: (a) 3'Center Loading, (b) 3'Corner Loading, (c) 5'Center Loading, (d1) 5'Corner Loading, (d2) 5'-8" Center Loading

1.2.3- Materials

The average compressive strength, modulus of elasticity and splitting tensile strength of the four specimens for concrete and grout mixtures are shown in Table 1. These tests were conducted according to ASTM C873/C873M-04e1, ASTM C469/C469-02e1, and ASTM C496/C496M-04e1 respectively. The average age of the cast-in-place concrete at the time of testing was 7 days. The properties of concrete were measured on 4" x 8" concrete cylinders.

The grout to fill the haunch is SikaGrout 212™ high performance grout. This material is used to fill the haunch on the precast overhang portion of the bridge. This requires the grout to be sufficiently fluid to flow through the haunch while maintaining dimensional stability and later attain sufficient strength. Obtaining both of these criteria can have conflicting effects. To evaluate these characteristics the flowability, segregation, bleeding, early age dimensional stability, fresh density, and strength were evaluated. Details of the grout investigation can be found in Trejo et al. [12]. After the grout had obtained initial set then a concrete mixture was used to fill the remaining space in the pocket.

The mechanical properties of the reinforcement bar measured for various diameters met TxDOT 440 and ASTM A 615/A615M-08a grade 60 requirements. Table 2 provides the average stress and strain magnitudes for the samples tested. All bars had a well defined yield plateau.

Table 1: Summary of the average material properties of the mixtures used in Test Specimens.

Specimen	Test	CIP	Precast Panel (Stage I)	Precast Panel (Stage II)	Grout	Pocket Concrete	Depth Panel
3' Overhang Center Loading	Compression, psi	6980	9100	7100	8140	4090	8480
	Tension, psi	660	729	620	544	524	693
3' Overhang Corner Loading	Compression, psi	5370	9150	6860	6290	4880	8480
	Tension, psi	514	774	550	600	458	693
5' Overhang Center Loading	Compression, psi	5730	9680	8740	6800	5370	8480
	Tension, psi	514	713	792	507	---	693
5' Overhang Corner, and 5'-8" Center Loadings	Compression, psi	3370	9310	9480	---	4560	9910
	Tension, psi	220	600	600	---	530	770

Table 2: Stress values for steel reinforcement

Specimen	Yield Stress, ksi	Yield Strain	Ultimate Stress, ksi
#5 Samples	70	0.00244	100
Precast wire mesh[12]	63	0.00215	69

1.2.4- Measurements

During loading continuous measurements of the applied loads were recorded at the hydraulic jack. Deflections of the slab with electronic linearly variable displacement transducers (LVDTs) with (0.0005 in) accuracy and surface strain readings were taken at selected load stages by using a rectangular grid of stainless steel targets spaced at about 8" that was measured by a portable DEMAC gauge with 4.4 microstrain accuracy. The DEMAC gauge has machined ends that match the machined holes in the stainless steel discs. These systems provided flexible and accurate methods to investigate the performance of the overhang systems.

1.2.5- Determination of Principal Strains

The maximum average principal strain (ε_{max}) was found for each set of DEMACs. This was found by averaging the perpendicular strains at the sides of each grid squares in both the x and y direction. This is shown in Figure 20 as ε_x and ε_y; where:

$$\varepsilon_x = \frac{\varepsilon_{x1} + \varepsilon_{x2}}{2} \text{ and } \varepsilon_y = \frac{\varepsilon_{y1} + \varepsilon_{y2}}{2} \quad \text{……………………..} \ (1)$$

therefore,

$$\varepsilon_{max} = \sqrt{\varepsilon_x^{\ 2} + \varepsilon_y^{\ 2}} \quad \text{……………………………..……….} \ (2)$$

and the orientation of this maximum principal strain is,

$$\alpha = \tan^{-1}\left(\varepsilon_y / \varepsilon_x \right) \quad \text{……………………..…………..} \ (3)$$

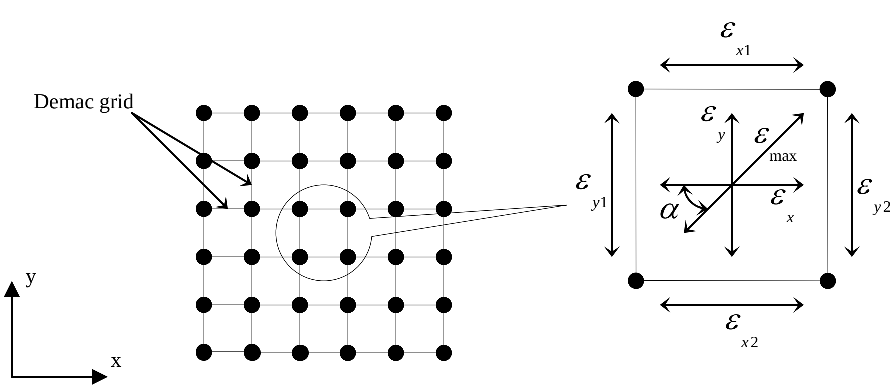


Figure 20: Determination of the principal strains

1.3- RESULTS

The load, deflection, crack location, and surface strain of each specimen were measured at each loading step. A summary of the measurements taken during testing as well as the surface strains is shown in Figure 21 through Figure 29. These graphs were displayed beginning with the cracking stage. Also, the top surface deflection, progression graphs, and the gauges locations have been accompanied to the former graphs.

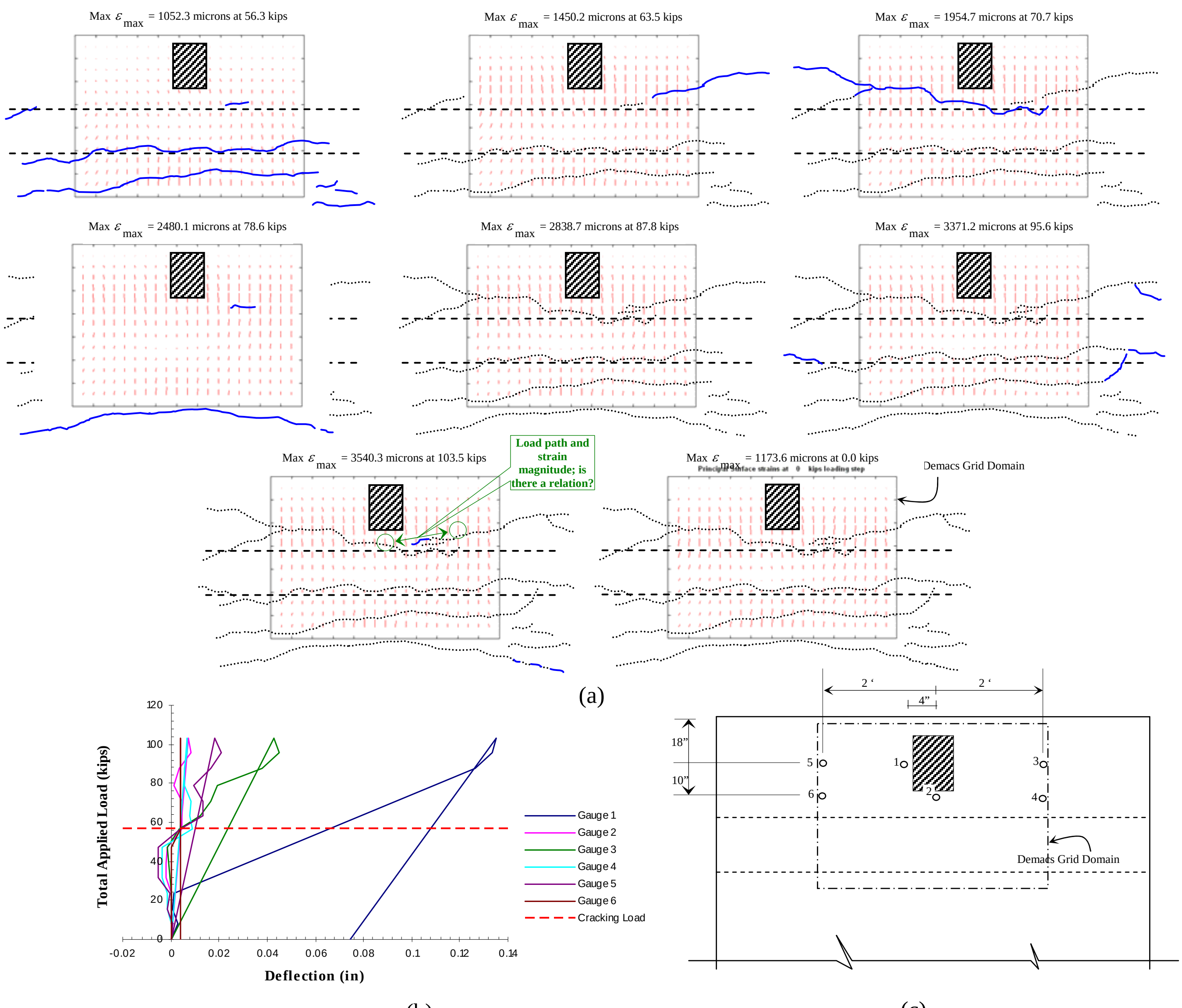


Figure 21: 3-ft. Overhang/ Conventional Side/ Center Loading.: a) Top surface cracks progression plots accompanied with maximum principal top surface strains, b) Deflection progress at different loading stages, c) Deflection gauges' locations

Note: *Failure Load has not been reached*

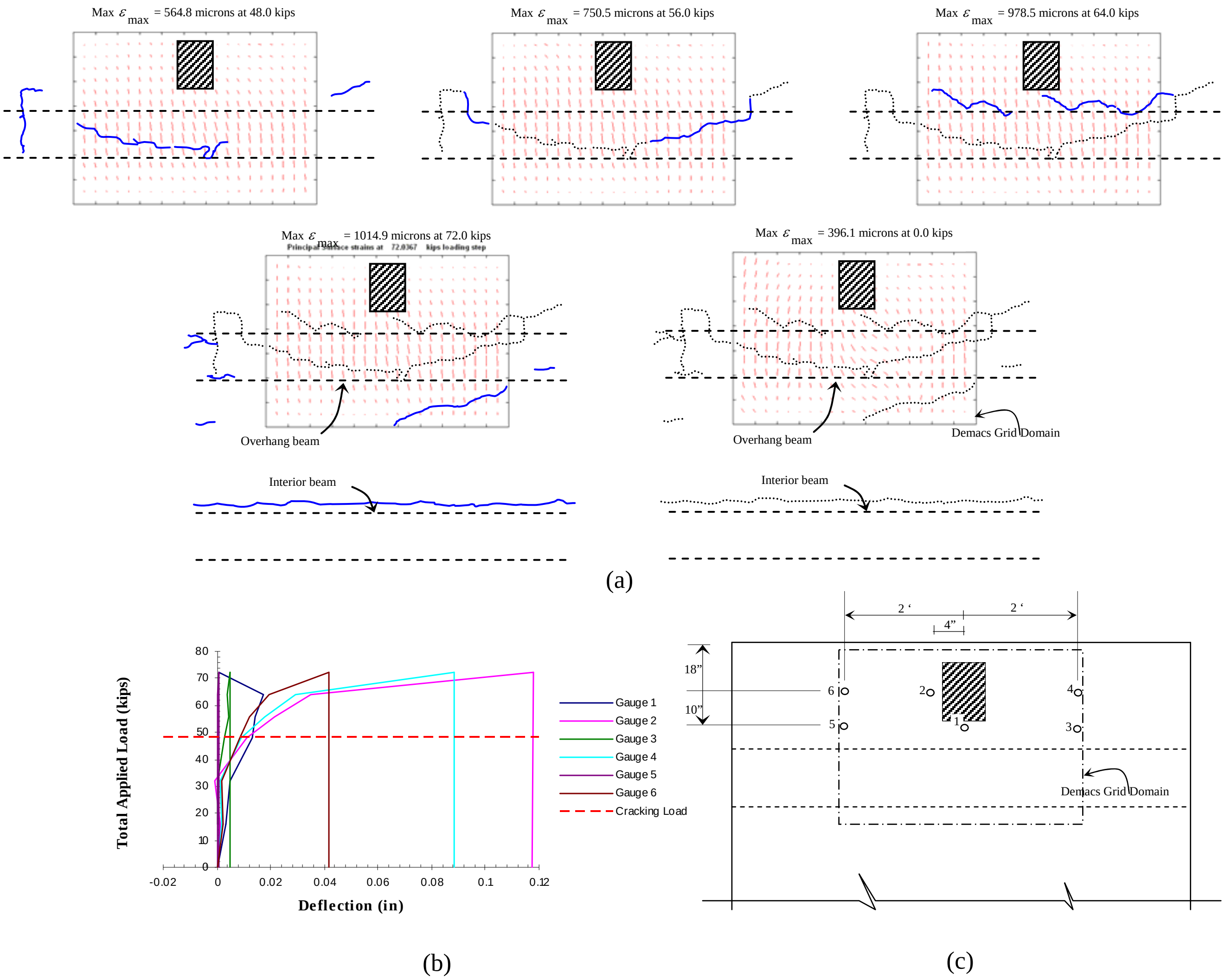


Figure 22: 3-ft. Overhang/ Precast Side/ Center Loading: a) Top surface cracks progression plots accompanied with maximum principal top surface strains, b) Deflection progress at different loading stages, c) Deflection gauges' locations

***Note:** Failure Load has not been reached*

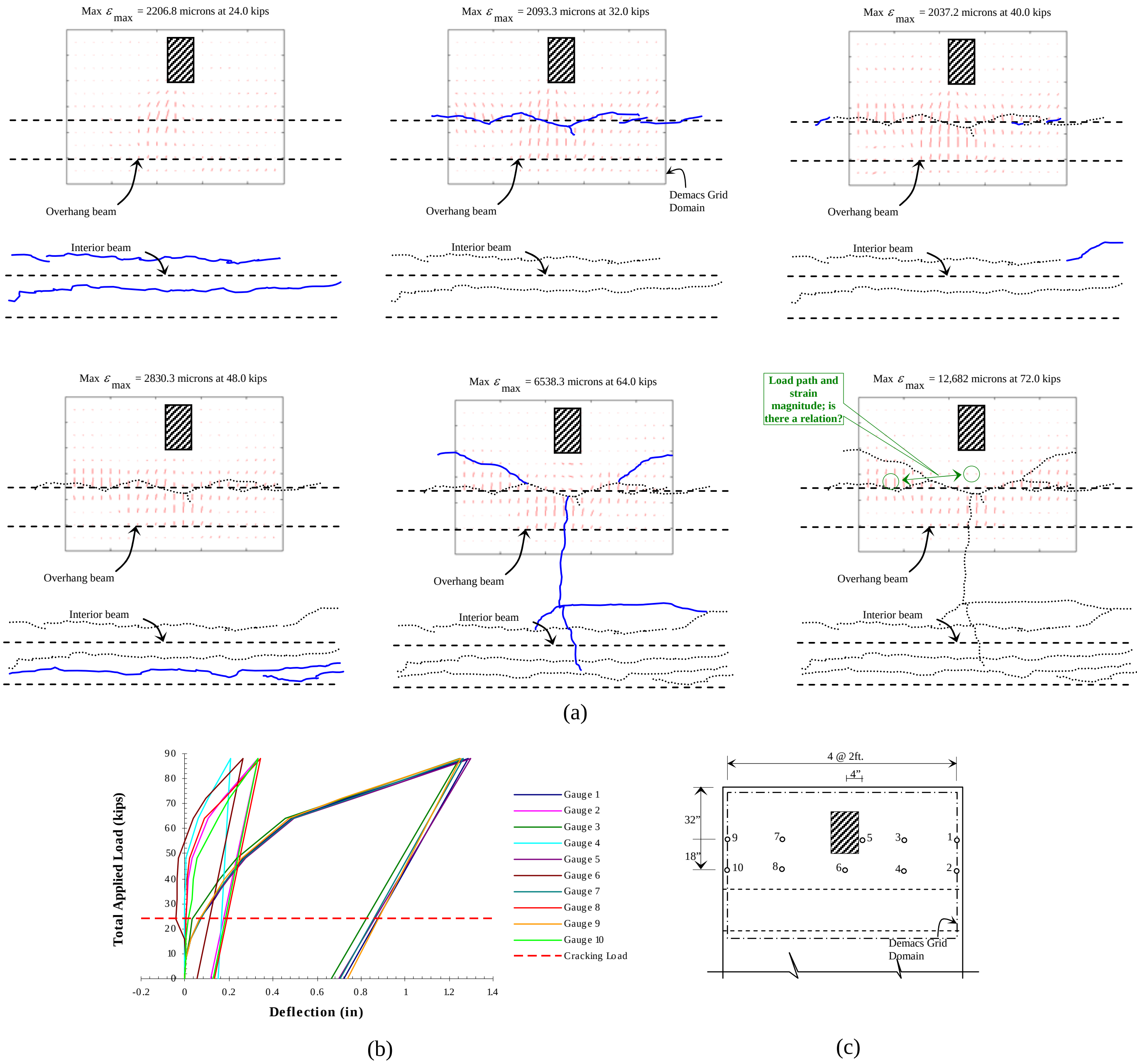


Figure 23: 5-ft. Overhang/ Conventional Side/ Center Loading: a) Top surface cracks progression plots accompanied with maximum principal top surface strains, b) Deflection progress at different loading stages, c) Deflection gauges' locations

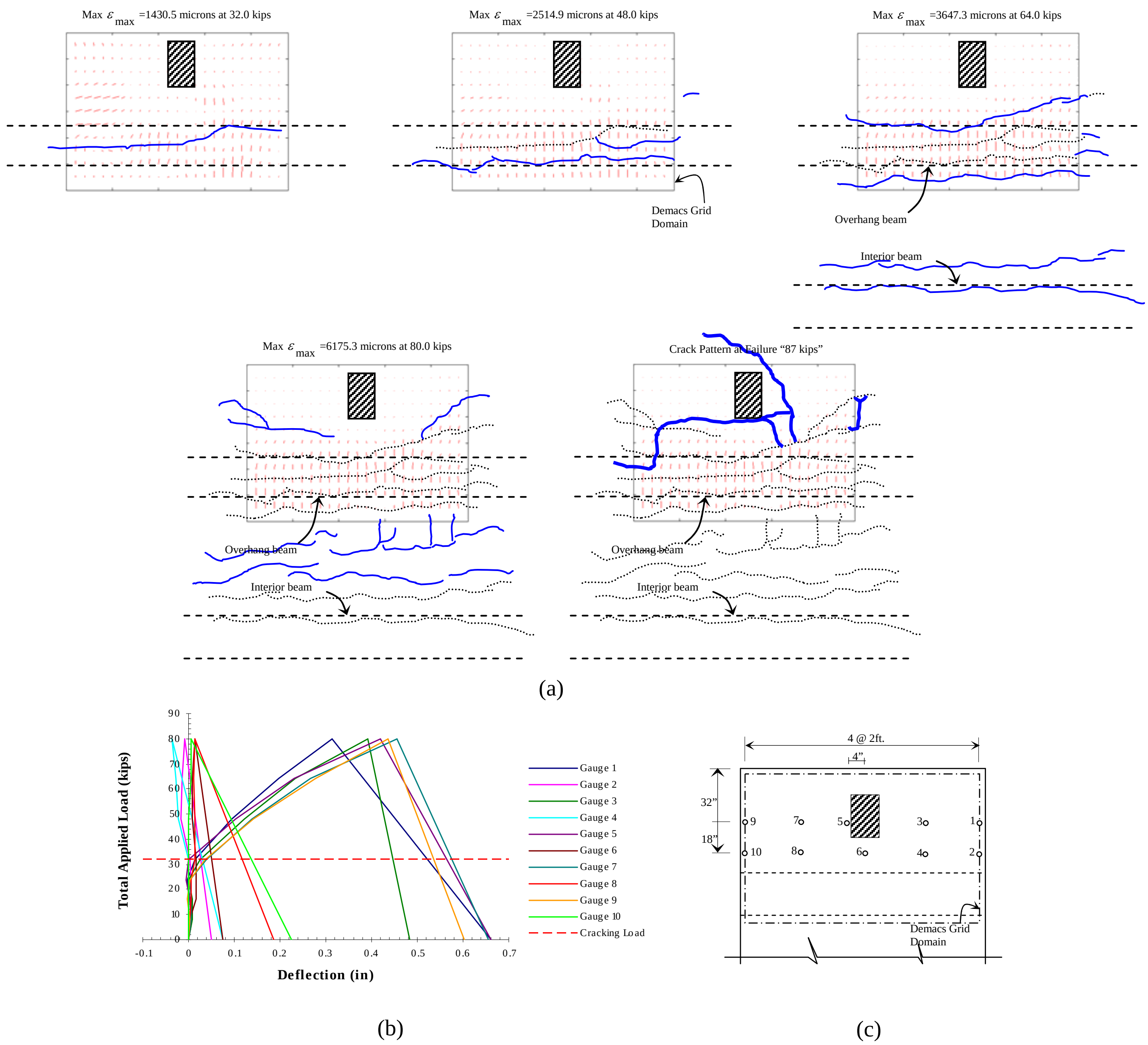


Figure 24: 5-ft. Overhang/ Precast Side/ Center Loading: a) Top surface cracks progression plots accompanied with maximum principal top surface strains, b) Deflection progress at different loading stages, c) Deflection gauges' locations

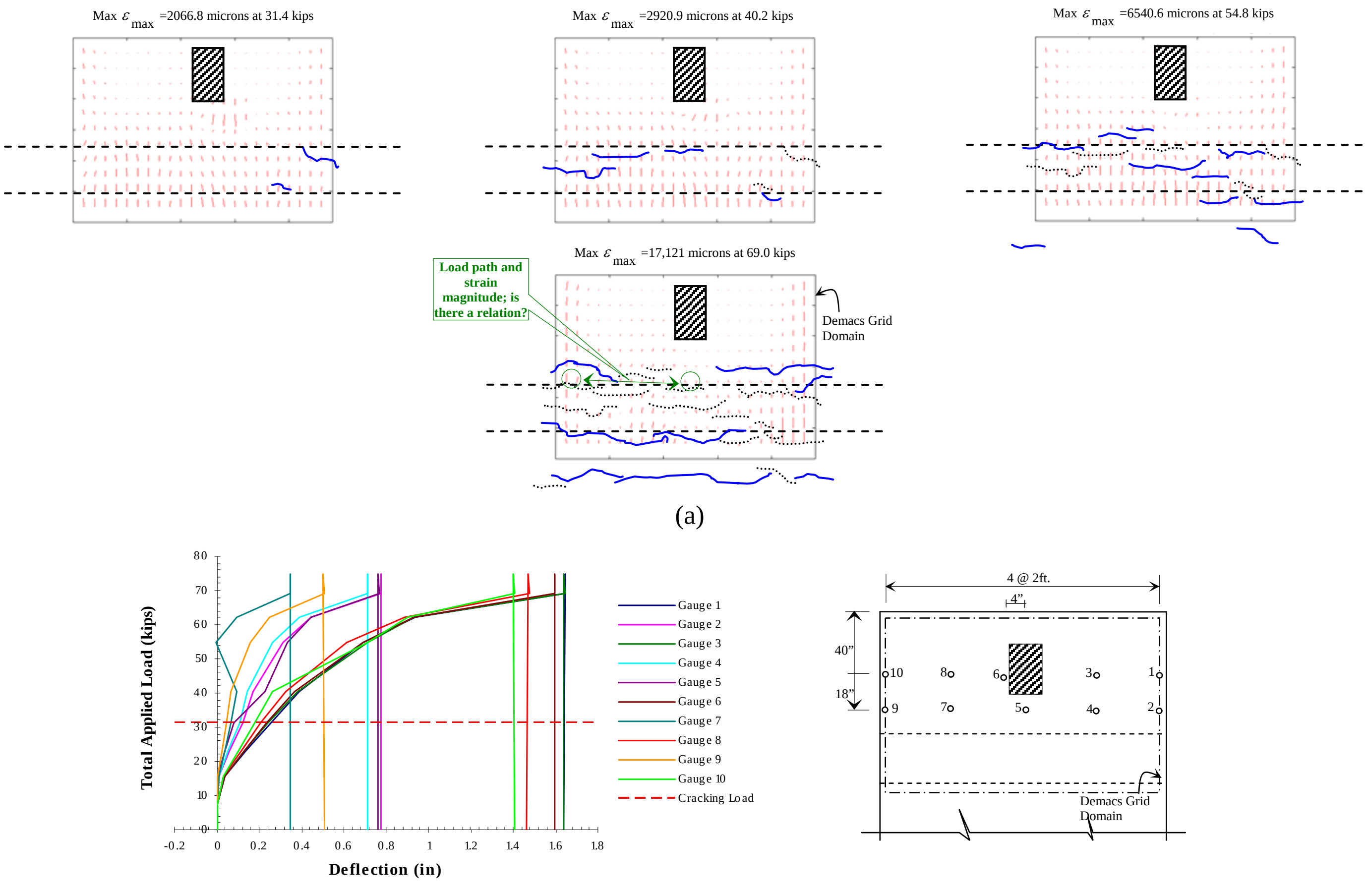


Figure 25: 5ft.-8in. Overhang/ Precast Side/ Center Loading: a) Top surface cracks progression plots accompanied with maximum principal top surface strains, b) Deflection progress at different loading stages, c) Deflection gauges' locations

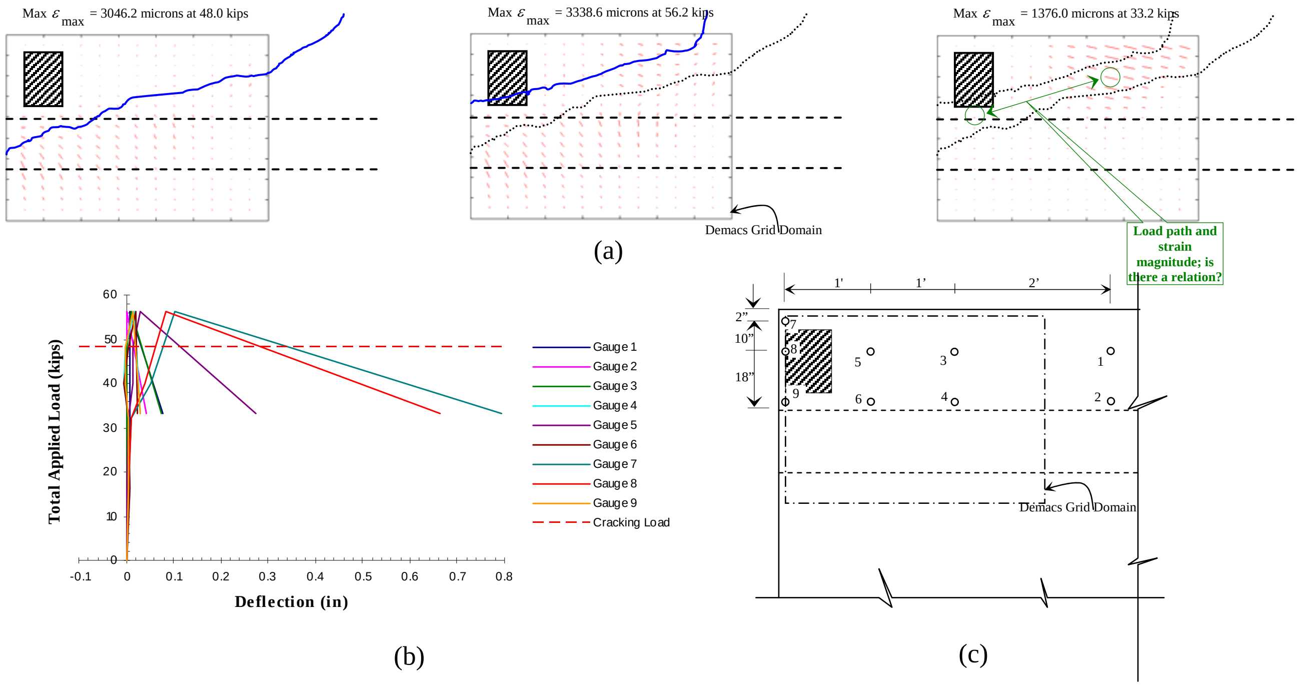


Figure 26: 3-ft. Overhang/ Conventional Side/ Corner Loading: a) Top surface cracks progression plots accompanied with maximum principal top surface strains, b) Deflection progress at different loading stages, c) Deflection gauges' locations

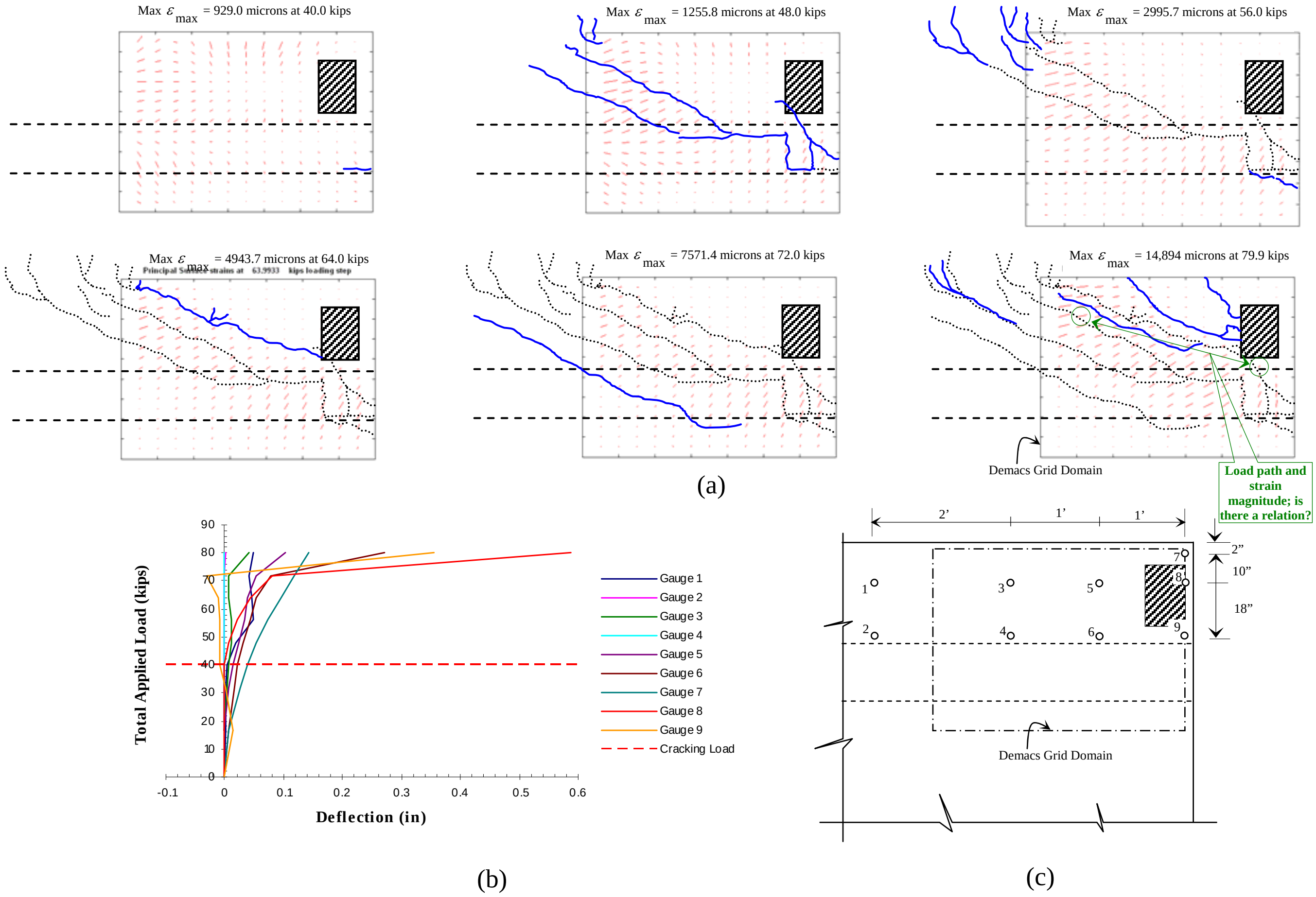


Figure 27: 3-ft. Overhang/ Precast Side/ Corner Loading: a) Top surface cracks progression plots accompanied with maximum principal top surface strains, b) Deflection progress at different loading stages, c) Deflection gauges' locations

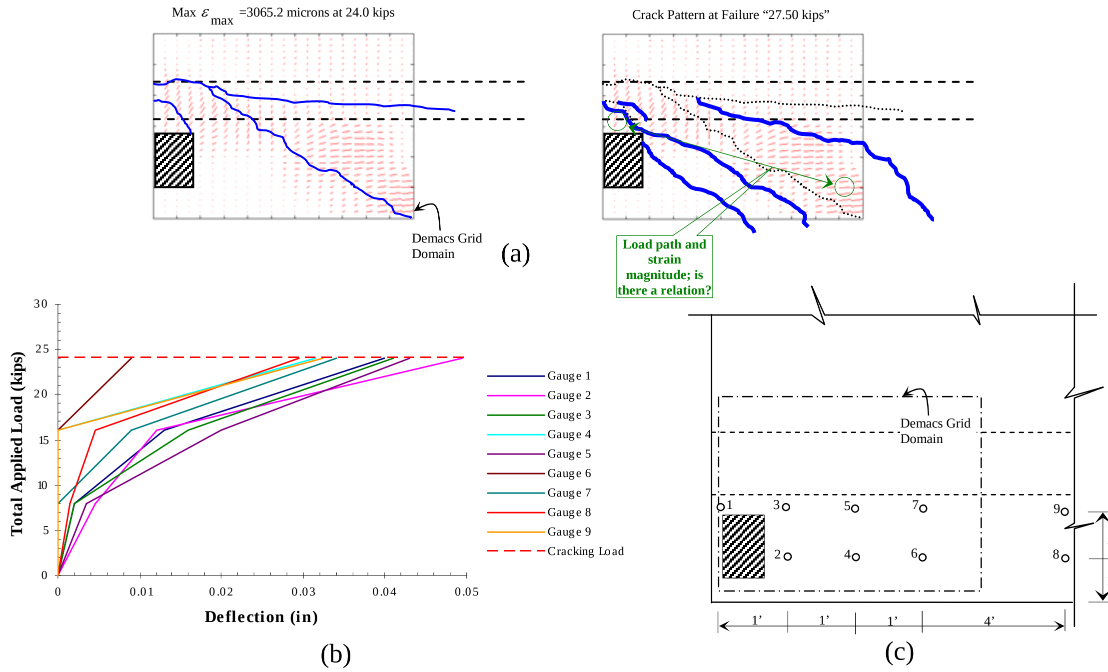


Figure 28: 5-ft. Overhang/ Conventional Side/ Corner Loading: a) Top surface cracks progression plots accompanied with maximum principal top surface strains, b) Deflection progress at different loading stages, c) Deflection gauges' locations

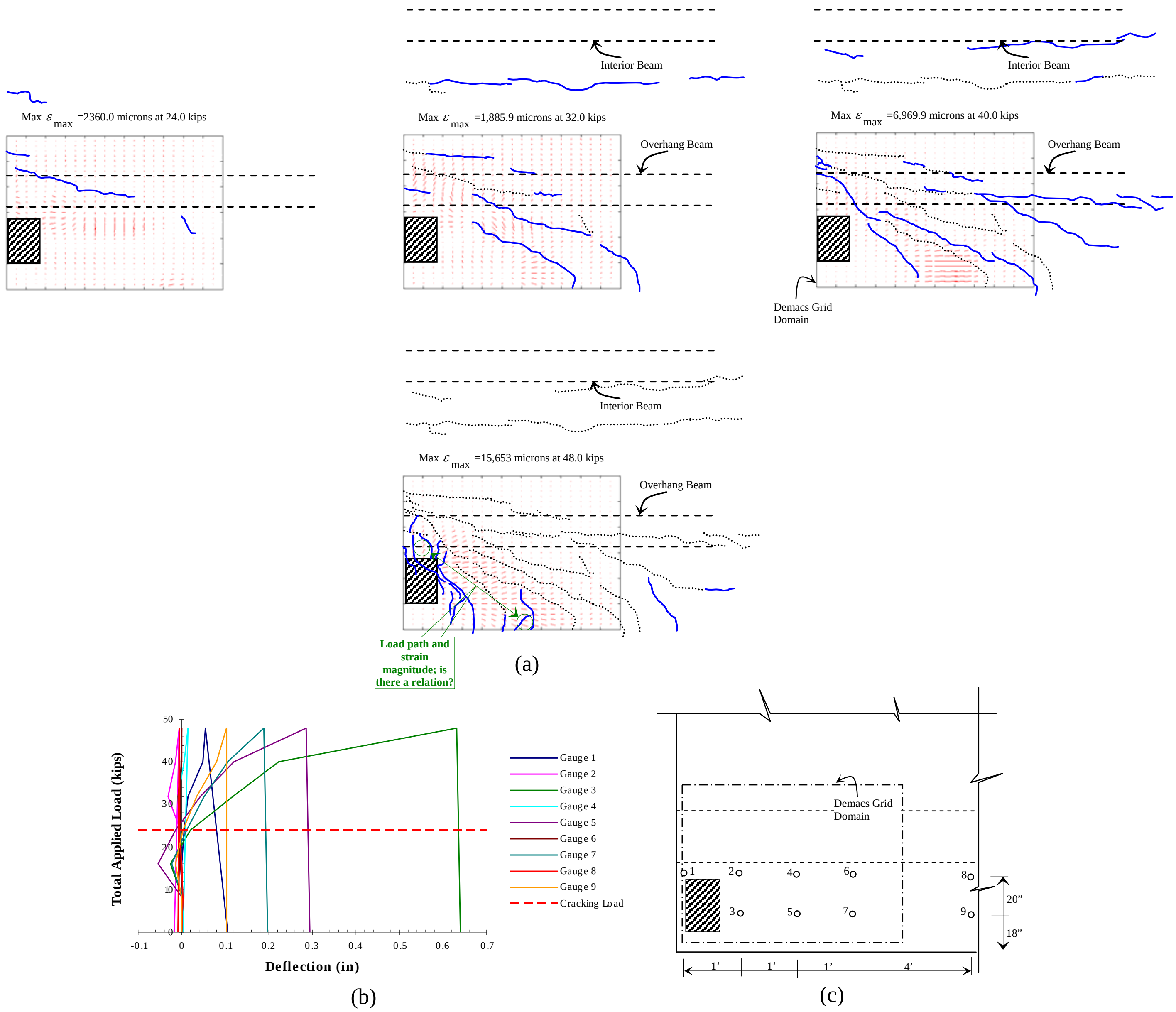
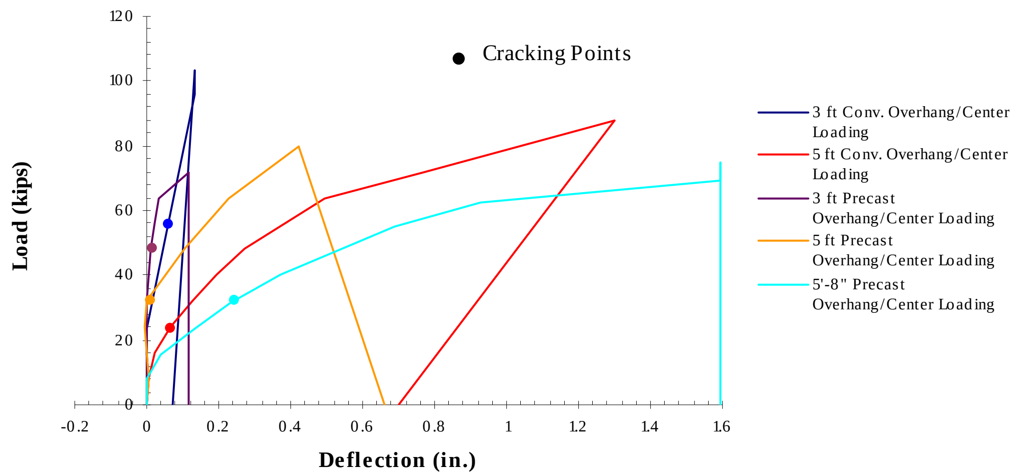
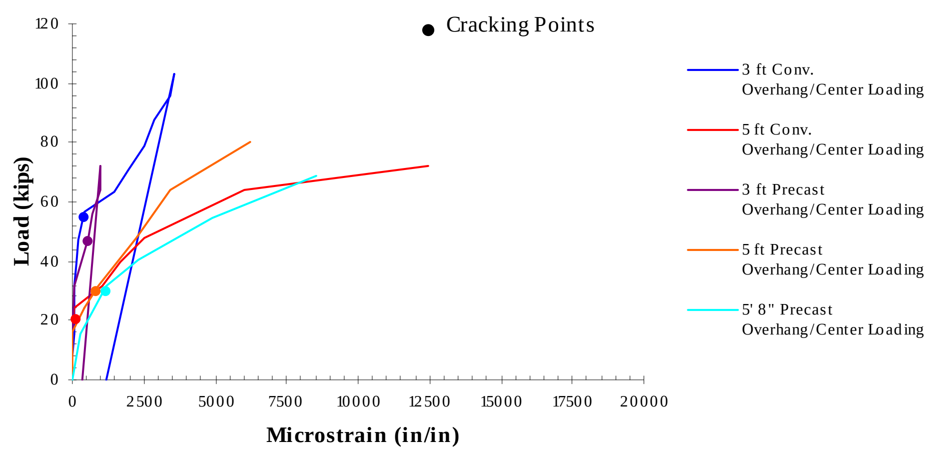


Figure 29: 5-ft. Overhang/ Precast Side/ Corner Loading: a) Top surface cracks progression plots accompanied with maximum principal top surface strains, b) Deflection progress at different loading stages, c) Deflection gauges' locations

Comparisons of each overhang type, conventional and precast, for both overhang lengths, 3 ft. and 5 ft, have also been made via plotting the deflection progress at locations having maximum magnitudes. The same is applied for the top surface strains. Cracking loads have also been included as a reference. See Figure 30 and Figure 31.

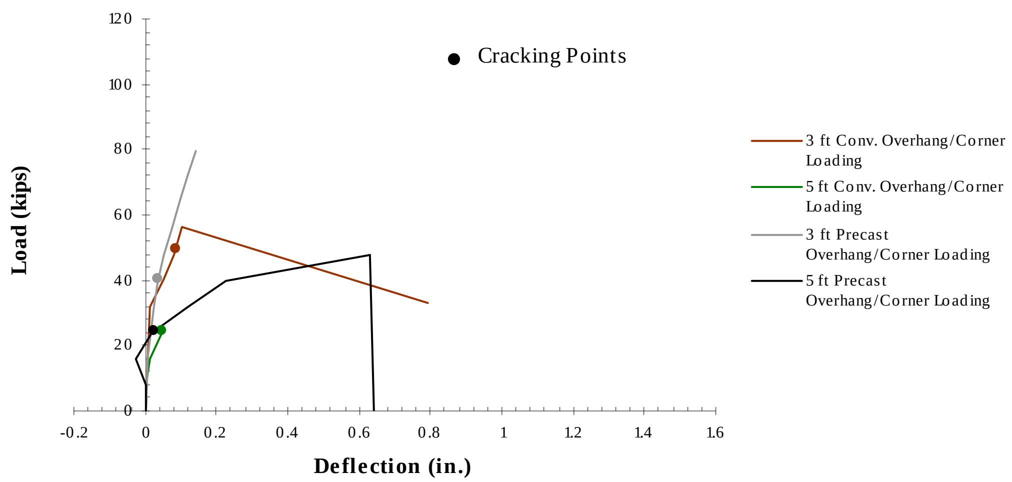


(a)

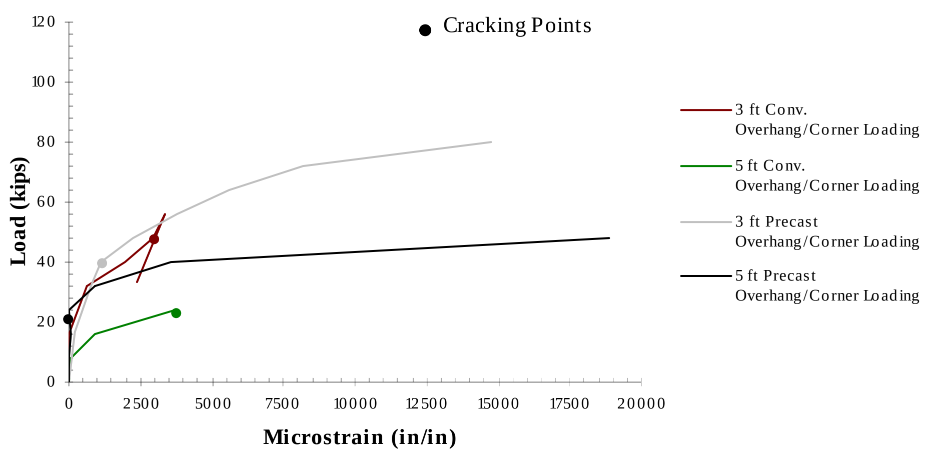


(b)

Figure 30: Center Loading: Comparison of a) Maximum Top Surface Deflections Progression, and b) Top Surface Strains for DEMACs Maximally influenced



(a)



(b)

Figure 31: Corner Loading: Comparison of a) Maximum Top Surface Deflections Progression, and b) Top Surface Strains for DEMACs Maximally influenced

Considering the AASHTO's 16 kips design load as a reference, Table 3, Table 4, and Table 5 highlight the performance of all test specimens. Also, Figure 32 provides some sample photos for two of the test specimens after failure.

Table 3: AASHTO LRFD 2007 limit states for tested specimens.

Check	Limit state	AASHTO LRFD 2007 Section
Service limit state	Deflection should be > L/1200	9.5.2
Fatigue and Fracture Limit state	N.A	9.5.3
Strength limit state	First crack loading should be > 16 kips (service load)	9.5.4

Table 4: Performance of Test Specimens (Loads and strains).

Specimen	Construction Type	Performance								Remarks
		At Cracking				At Failure				
		Load ((kips)	Ratio to AASHTO Design Load	Max. Defln. (in)	Max. Max. Surface Strain (x 10^{-6}) (in/in)	Load, (kips)	Ratio to AASHTO Design Load	Max. Defln. (in)	Max. Max. Surface Strain (x 10^{-6}) (in/in)	
3' Overhang Center Loading	*Conventional*	56.3	3.5	0.093	1052.3	104	6.5	0.135	3540.3	Failure loads have not been reached
	Precast	48.0	3.0	0.011	564.8	72.0	4.5	0.118	1014.9	
5' Overhang Center Loading	*Conventional*	24.0	1.5	0.068	2206.8	72.0	4.5	1.300	12,682	
	Precast	32.0	2.0	0.001	1430.5	87.0	5.4	0.662	6175.3	
5'-8" Overhang Center Loading	*Precast*	31.4	2.0	0.235	2066.8	69.0	4.3	1.596	17,121	
3' Overhang Corner Loading	*Conventional*	48.0	3.0	0.078	3046.2	56.2	3.5	0.794	3338.6	
	Precast	40.0	2.5	0.038	929.0	79.9	5.0	0.143	14,894	
5' Overhang Corner Loading	*Conventional*	24.0	1.5	0.050	3065.2	27.5	1.7	0.050	---	
	Precast	24.0	1.5	0.021	2360.0	48.0	3.0	0.641	15,653	

Table 5: Performance of Test Specimens (Deflections).

Specimen	Construction Type	Max. deflection at service load (in)	Max. deflection at max. applied load (in)	Deflection limit state at service load (in)	Remarks
3' Overhang Center Loading	*Conventional*	0.0925	0.1350	0.03	
	Precast	0.0110	0.118	0.03	
5' Overhang Center Loading	*Conventional*	0.0675	1.2995	0.05	
	Precast	0.0390	0.6565	0.05	
5'-8" Overhang Center Loading	*Precast*	0.2345	1.596	0.05	
3' Overhang Corner Loading	*Conventional*	0.078	0.7940	0.03	
	Precast	0.0375	0.1425	0.03	
5' Overhang Corner Loading	*Conventional*	0.0495	0.0495	0.05	
	Precast	0.0210	0.6300	0.05	

Figure 32: Sample photos for failures after testing.

1.4- DISCUSSION

All specimens that were loaded to failure developed a failure surface around the concentrated loads and failed in punching shear except the 5’-8” precast overhang and the 5’ conventional overhang with center loading which failed in flexure. The specimens that failed in punching shear failed in a brittle manner; however, all of the bridge decks failed at loads much higher than the design loads. It should be noted that the failure loads were not reached in the 3’ specimens with center loading because of the limitations of the rams. However the 3’ specimens provided a significant safety factor when compared to the design loads, a minimum of 2.5 for corner loading at cracking. In each of the specimens flexural cracks developed for all tests on the top surface at the external support beam, refer to Figure 21a through Figure 29a, (cracks along the longitudinal direction). Such cracks increased their widths during the test, reaching at failure values between 0.013” and 0.215”. The following observations can be made:

- When comparing the performance of each specimen to the 16 kip AASHTO design load satisfactory performances were obtained. A minimum factor of safety of 4.3 was obtained for center loading, and a minimum of 1.7 against failure for the corner loading.
- As shown in Table 4, the precast overhang system has a consistently higher ultimate strength than the conventional overhang specimens but similar cracking loads.
- Generally, losses of stiffness for the 5’ overhangs are faster than those of the ones that are 3’. This can be seen by looking at the maximum deflections in
- Figure 30a and Figure 31a. This is expected as the longer cantilevers have a lower amount of stiffness. For the corner loaded specimens this means that the increase in the cantilever length is more significant than the increase in load transfer area. The cracking of the two systems at the surface of the exterior beam was quite different. This performance can be seen in Figure 21 to Figure 29. In the conventional overhang system cracks were observed at the interface between the beam and deck, while the precast overhang system showed cracking at several locations over the top of the beam. This difference in behavior is likely attributable to the presence of a continuous prestressed panel in for the precast overhang system.

For a given load, the cracking of the precast overhang system was much more distributed than the conventional overhang system as shown in Figure 21 to Figure 29. This dispersion of cracks should lead to cracks that are smaller in size. Surface strain measurements shown in

- Figure 30b and Figure 31b also reinforce this same observation as the maximum surface strains are lower for the precast systems when compared to the conventional overhangs. This made it possible to reduce surface strain by an average of 23% prior to failure stages. As a result, the expected average crack widths should also be 23% smaller therefore providing an increased durability of bridge decks for the same loading conditions.
- It was observed in the testing that the location of the maximum principal strains were not necessarily within the expected load path from the load point to the support beam. This can is seen by observing the low levels of surface strains between the load point and the support beam in Figure 21a, Figure 23a, Figure 25a, Figure 26a, Figure 27a, Figure 28a, and Figure 29a. This is due to the fact that surface strains are more related to, and directly affected by deformations rather than loads that were present in these instances. As might be observed, the only exceptions are the centrally loaded precast sides for the 3' and 5' overhangs. Presence of prestressing with the available load symmetry led to these two exceptions.
- In Figure 30 and Figure 31 it can be observed that the CIP specimens showed the greatest increase in surface strain and deflection magnitudes when compared to their precast companions. This suggests that the precast system is stiffer and should exhibit less cracking for the same amount of exterior load. The deflection for both systems under the load cases tested were much lower than the AASHTO limit for serviceability, see Table 5.

This research investigated only static loading. Based on the significant reserve capacity of the specimens it would be expected for the system to show satisfactory fatigue performance based on service load levels. This is further supported by AASHTO LRFD 2007 section 9.5.3 which states it is not necessary to investigate the failure of concrete bridge decks under fatigue loading.

1.5- CONCLUSIONS

The research performed in this study evaluated the performance of the precast, prestressed full-depth bridge overhang system. Three overhang lengths were tested; 3’, 5’, and 5’-8” under center and corner loading. The findings are:

- All specimens provided significant safety factors when comparing the service loading specified to AASHTO to the cracking and ultimate loads. A minimum factor of safety of 1.5 for cracking, and 3.0 at ultimate were both obtained for the 5’ overhang loaded at corner.
- A punching shear failure was observed in all specimens tested except for the 5’ cast-in-place overhang and 5’-8” precast overhang with center loading which showed a flexural failure mode.
- The precast overhang specimens showed the ability to allow a much greater dispersion of cracks when compared to the cast-in-place overhangs. This was reflected in the reduction in surface strains by an average of 23% between the two systems when compared at the same loading conditions. This reduction in surface strain must lead to a similar reduction in crack sizes.

In conclusion the study recommends implementation of the 5’ precast overhang system as it showed satisfactory performance from the center and corner loading under service and ultimate load states.

PHASE II – USE OF WELDED WIRE MATS FOR BRIDGE DECK CONSTRUCTION

2.1- INTRODUCTION

Past research indicates that concrete bridge decks that use flexural design methods show significant safety factors against failure. This was first noticed in testing by the Ontario Ministry of Transportation(13). This research pointed out that bridge decks of typical dimensions did not fail due to flexure, but instead showed a significant amount of load caring capacity after flexural yielding of the reinforcing steel and then failed suddenly due to punching shear. Similar load testing has been completed with bridge decks that use stay in place partial depth bridge panels, and capacities similar to bridge decks with mild reinforcing steel were observed(3,4,5,6). The arching action capacity is used in the AASHTO LRFD Design Manual (2007)(14) with the bridge deck direct design method, which has lead to a significant reduction in the amount of reinforcing steel in bridge decks.

Past research has shown that bridge decks are able to provide significant safety factors against failure; however, they continue to show serviceability problems in the field. These problems result from cracks in a bridge deck that expose the reinforcing steel and concrete to outside chemicals, which ultimately cause durability problems. These cracks are typically largest in the negative moment region over the beams as this area has the greatest tension on the bridge deck surface from typical loading. Because of this, it seems that the primary role of bridge deck reinforcing steel is to minimize the surface cracks and keep the cracks that do form as small as possible in order to promote a long service life.

Typically, the reinforcement for a bridge deck consists of tied reinforcing bars. While bridge decks with these bars have been used satisfactorily for years, the research team feels that the performance of these bridge decks could be improved if deformed pre welded wire mats, in accordance with ASTM standards A496 and A497 for deformed wire and deformed welded wire reinforcement and in a combined standard ASTM A1064, were substituted for these bars. Some of these advantages include:

- Wire mats can be pre-constructed by a machine and then shipped to the jobsite thus minimizing labor and increasing construction speed

- Mats with a similar density to current reinforcing designs can be used in the areas of high tension and lighter mats can be used in the temperature and shrinkage areas; this will allow for a reduction in the amount of required steel
- Since the mats are constructed with a machine, closely spaced reinforcing bars with smaller diameters could be used that would not be economical to place by hand
- Close bar spacing provides superior crack control over rebar of the same weight per foot that uses bars with a larger diameter and spacing
- This ability to improve crack control provides opportunities for a greater tolerance on the clear covers of bridge decks, which will result in improved constructability of the bridge deck

One primary challenge in constructing a bridge deck is to insure that a minimum amount of clear cover is uniformly provided over the reinforcing steel. It is common for construction crews to make significant adjustments to the reinforcing steel height during construction to insure that this specified amount of clear cover is provided at all locations. If a bridge deck was allowed to have a greater clear cover than what was specified then this would increase its constructability and lower the cost. One challenge with increasing the clear cover of the reinforcing steel is that the size of the surface cracking may increase. However, by using a welded wire mat to economically use a tighter spacing of reinforcing bars, cracking can be controlled, which allows for an improvement in the constructability with an increased cover tolerance or an increase in the durability by using a similar clear cover.

2.2- EXPERIMENTAL METHODS

It was realized early in this research that it would be challenging to accurately simulate the long term performance of a bridge deck in the laboratory. Because of this it was decided that a standard test setup would be used to compare the performance of different structural systems to load applied by hydraulic jacks and examine their cracking and ultimate strength. While measuring the response of these structures to loading from external load does not replicate how a bridge deck will perform in the field, this loading can still be a useful method to compare the performance of two different reinforcing layouts as long as similar testing is completed on representative control specimens. If a specimen showed improved or equivalent performance in the testing program under external loading then it would be expected to show similar performance when implemented in the field.

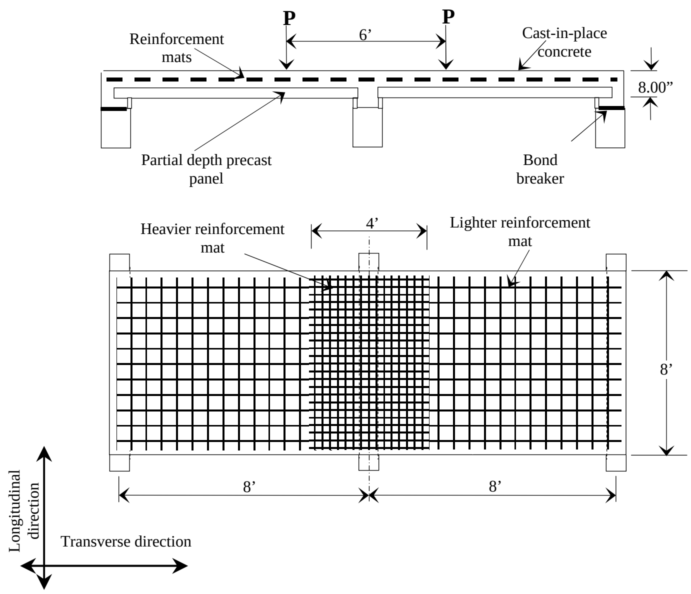
In this project three control bridge decks were used for comparison purposes to the bridges that used wire mats. These control bridge decks included:

- 8” partial precast bridge deck that uses a 4” precast panel and 4” of cast in place concrete with no steel in the cast in place concrete as shown in Figure 38, specimen A.
- 8” partial precast bridge deck that uses a 4” precast panel and 4” of cast in place concrete with #5 bars at 6” transversely and #4 bars at 9” longitudinally for the top layer of steel with 2” of clear cover with a 4” stay in place precast panel as shown in Figure 38, specimen B. (standard TxDOT design)
- 8” cast in place bridge deck with a top layer of #4 bars at 12” in both directions with 2” of clear cover and a bottom layer of reinforcing steel with #5 bars at 12” in both directions with 1” of clear cover as shown in Figure 38, specimen C. (standard AASHTO Direct Design Method)

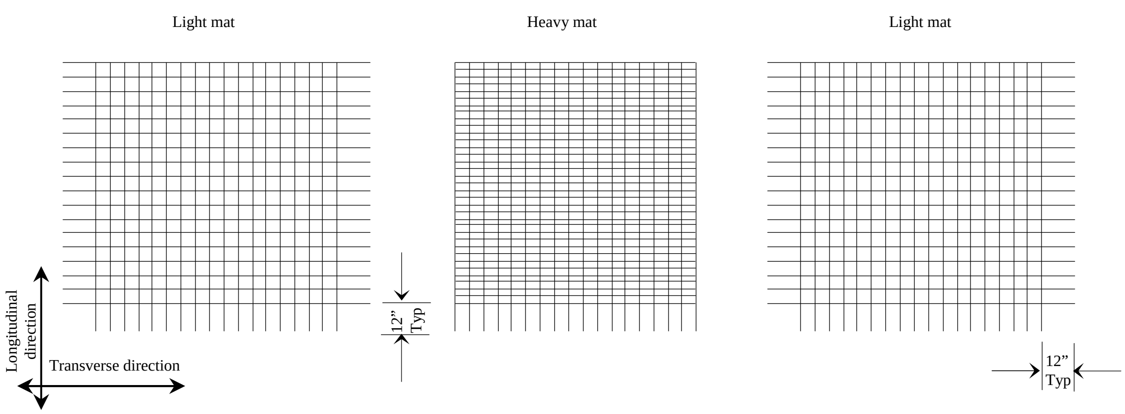
These control specimens were chosen to provide a benchmark for the testing of two different styles of bridge deck design, and an extreme case of using no reinforcing steel in the top layer of the partial precast bridge deck. These control specimens allow for a direct comparison of the cracking, surface strains, and ultimate load with the test methods used and bridge decks that use pre-constructed wire mats with different covers. It is the goal of this project to use the wire mats to develop a bridge deck system that either provides a reduction in cracking with similar covers or equivalent cracking at increased covers.

2.2.1- Test Setup

To investigate the performance of these systems the test setup shown in Figure 33 was used. Different deck thicknesses with different reinforcement arrangements were investigated, Figure 38. This load setup uses a three support beam system with two large point loads symmetrically placed over the center beam. A spacing of 6’ between the load points was used as this matched the transverse wheel spacing of an AASHTO HL 93 truck axle. The load areas used for the testing were 10” x 20” AASHTO tire patches. A beam spacing of 8’ was used for the testing. This beam spacing was chosen as it was a reasonable spacing for a typical DOT bridge deck and could be tested with the available strong floor space. If a larger beam spacing was used then the ultimate loads in the testing may be decreased but the relative ultimate strengths and surface cracking of the different systems should still be similar. The width of the specimen was 8'. This was chosen as it was the dimension of a standard precast panel.



(a)



(b)

Figure 33: a) Loading Setup for Bridge Deck, b) wire mat overview showing the splice detail used in the testing.

When constructing the load transfer area between the external beams and the bridge deck a construction detail was used where the precast panels were extended until about the beam centerline and then a plastic sheet was used between the panel and the concrete below. This was done to minimize the moment or horizontal load transfer between the bridge deck and the outside support beams. This simplifies the system to behave as if it is a two span structure that is continuous over the center support. The layout for the wire mats used for the testing in this research is shown in Figure 33b, Figure 34, and Figure 35. As shown in Figure 34, a heavier wire mat was used over the beams and a lighter mat was used in the areas between the beams. Figure 35 shows a finger splice detail that was used between the two mats. Figure 36 shows how the finger splices between four adjacent mats. This detail was chosen as it provides a full transfer of loads at the lap for the bars used. This detail also minimizes the amount of overlap of the wire mats, improves the constructability, and economy of the system.

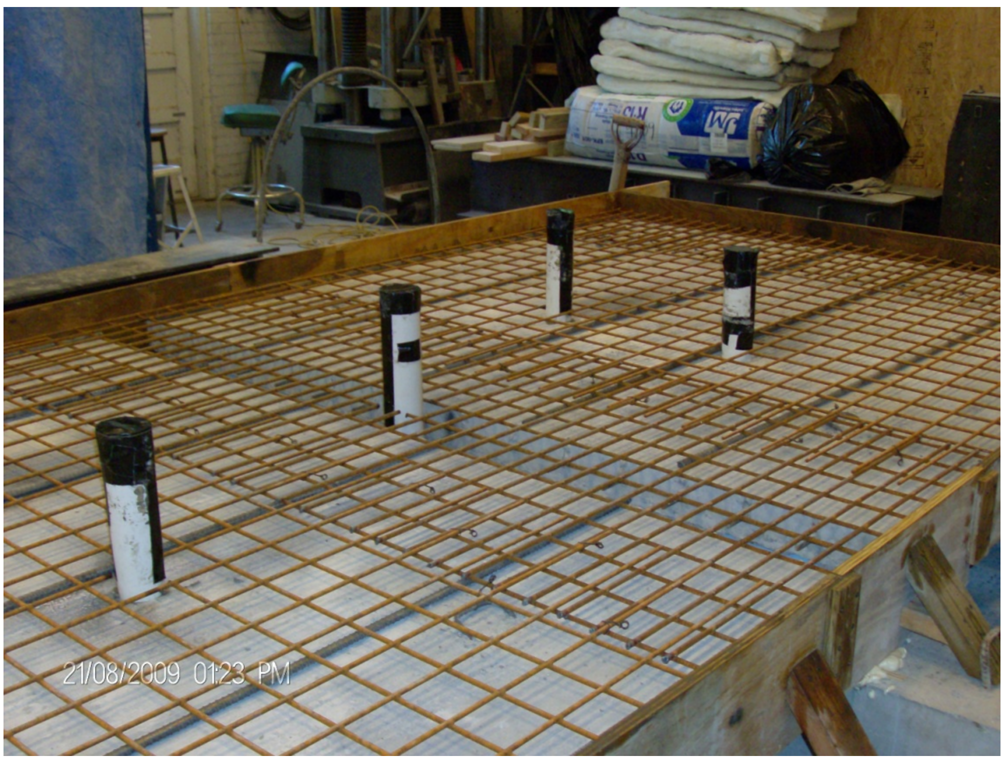


Figure 34: Wire mat layout in specimen that used #5 bars as chairs. Note the heaver wire mesh used over the interior beam.

The width of the wire mat over the beam was chosen to be 25% of the adjacent span *length plus the width of the beam. This was chosen based on a beam analysis of an HL 93 design truck that was systematically moved over the surface of the bridge deck while inspecting the locations* of the inflection point. The controlling load case was a three beam bridge with a HL

93 truck centered in one span. The negative moment in the non loaded span was small enough at 25% of the span length that the design moment would be lower than the cracking moment and so only temperature and shrinkage steel could be used.

The lighter wire mat that was used between the beams was chosen to satisfy the temperature and shrinkage steel requirements. Since this area would always be expected to be in compression or a low amount of tension under typical loading conditions, then temperature and shrinkage steel could be used. D8 bars at 4" in both directions provided an area of 0.24 in^2 per ft were used because they satisfied ACI and AASHTO specifications. This mat size was not modified during this testing. By using a lighter reinforcement mat in the areas between the beams, one can significantly reduce the amount of steel that is used in the top mat of the bridge deck compared to conventional bridge decks that carry the same reinforcing steel across the entire bridge.

Figure 35: A splice between the two wire mats.

The designer should keep in mind that each mat should be designed to weigh around 150 lbs each and should not be wider than 8' to insure easy shipping. This would allow them to be easily placed by two workers. Also, in order to insure that the mats are not incorrectly switched during the construction the designer should take the needed precautions and specify the mats to be dissimilar sizes. This should not be hard since the mats over the beams will be long and slender and the lighter reinforcement mats are closer to square.

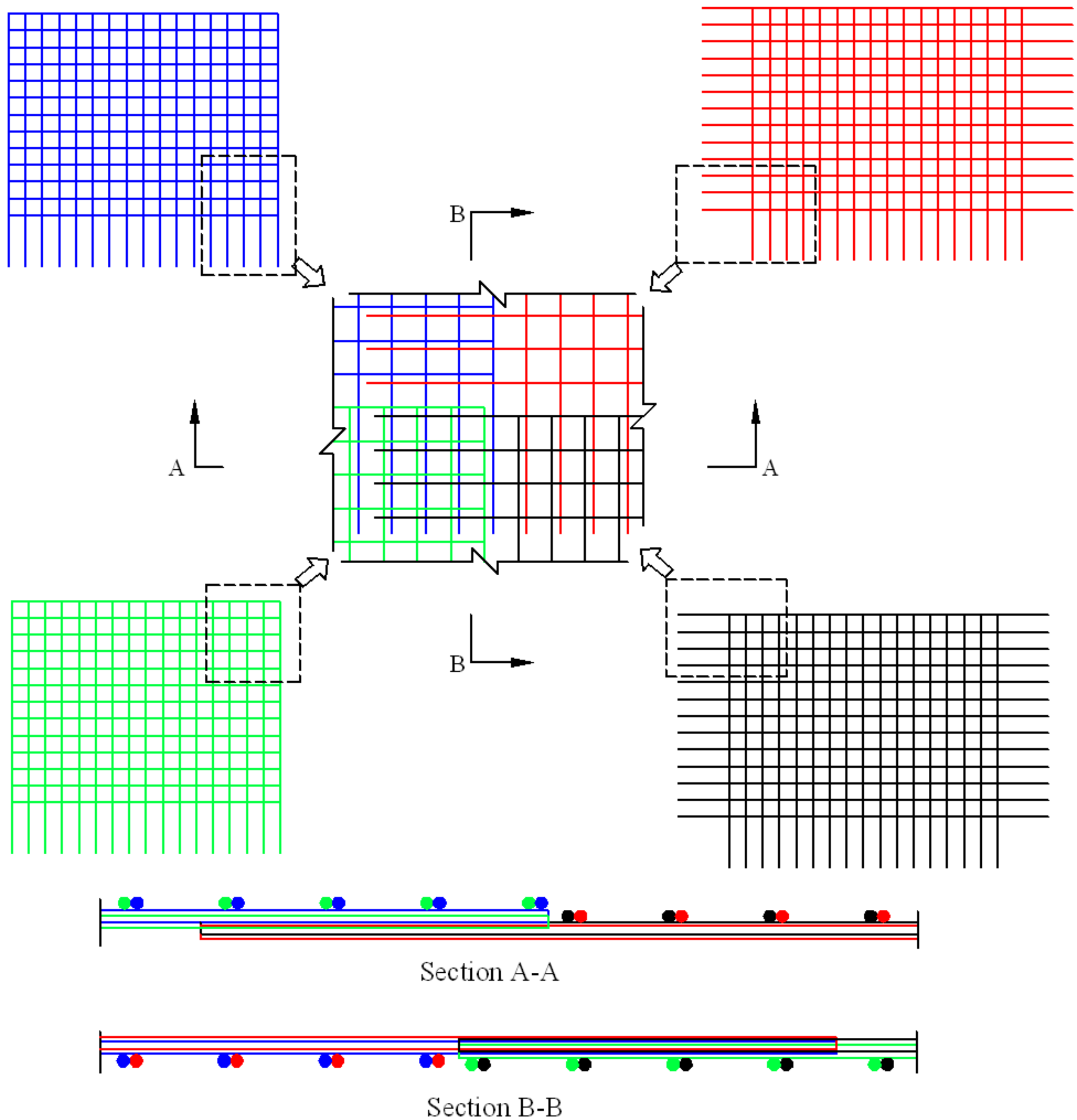


Figure 36: Details for a splice between four wire mats(15)

The instrumentation used to evaluate the performance of the specimens included the measurement of the load, specimen deflection near the load application, crack mapping, and the measurement of the surface strains. These measurements were taken initially and then at discrete load points through the testing. Measurements were typically taken in loading increments of 8 kips per load point, or at a total load of 16 kips until initial cracking was observed. After that load increments of approximately 16 kips per point load, or 32 kips total, were used until failure. After each load step measurements were taken from the instrumentation.

The deflection of the specimen was measured by using six linearly variable displacement transducers (LVDTs) with (0.0005") accuracy. These measurements were taken at the midspan and quarter points of the specimen. The surface strains of the specimens were measured by using stainless steel targets placed on an 8" rectangular grid and fixed to the surface with epoxy prior to loading. The movement of these targets with load in the longitudinal and transverse direction could be measured by using a portable demec stain gage that used special machined points that match a machined cone shaped void in the stainless steel discs. The accuracy of this system is 4.4 microstrain. This measurement technique has been used by a number of researchers to measure the surface strains of concrete specimens. A typical layout of the DEMAC points is shown in Figure 37. The crack maps for each specimen are shown in Appendix A. This measuring system allowed the research team to economically capture a significant amount of data that will help evaluate the performance of the different specimens.

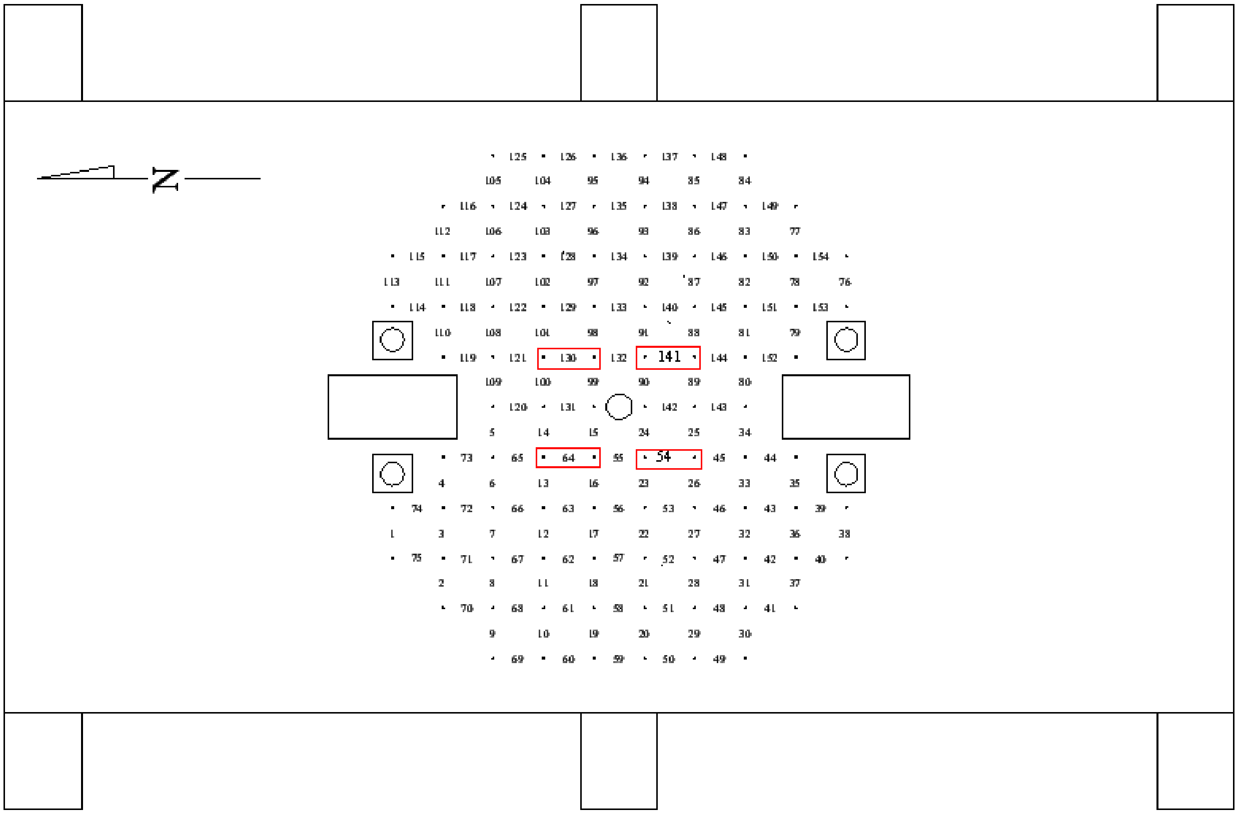


Figure 37: A typical demec gauge layout. The locations shown with a red box were the highest strains for the specimens investigated. The average readings from the side that failed were used to compare the performance of the different specimens.

Each specimen was constructed with a typical DOT bridge deck concrete with a 3" slump, 20% fly ash replacement, 0.42 w/cm, ¾" maximum nominal size aggregate, and 5% air content. Although the specified 28 day compressive strength of the concrete was 4,000 psi, the compression strength when evaluated at approximately 7 days for all of the specimens was around 5,500 psi. Based on the research team's experience with past bridge deck mixtures this would be a typical value for the strength gain of these mixtures. A summary of the measured strengths is presented in Table 6.

All specimens were constructed by using 4" partial depth precast panels with a cast-in-place concrete topping except for specimen B which was entirely cast-in-place. All of the specimens were 8" in depth except for specimen G which was 9". In specimen A no reinforcement was used in the cast-in-place section. In specimens D and E the density of the transverse reinforcement was varied. In specimens E, F, and G the same density of reinforcement was used at different clear covers. All of the specimen construction details are shown in Figure 38.

Table 6: A summary of the concrete specimen test results.

Specimen	Test	CIP	Precast Panel (Stage II)
A	Compression, psi	6490	10050
	Tension, psi	540	790
B	Compression, psi	5220	10540
	Tension, psi	410	760
C	Compression, psi	6240	10220
	Tension, psi	380	790
D	Compression, psi	5300	10130
	Tension, psi	430	510
E	Compression, psi	4500	10130
	Tension, psi	510	790
F	Compression, psi	4920	10380
	Tension, psi	380	790
G	Compression, psi	8850	---
	Tension, psi	730	---

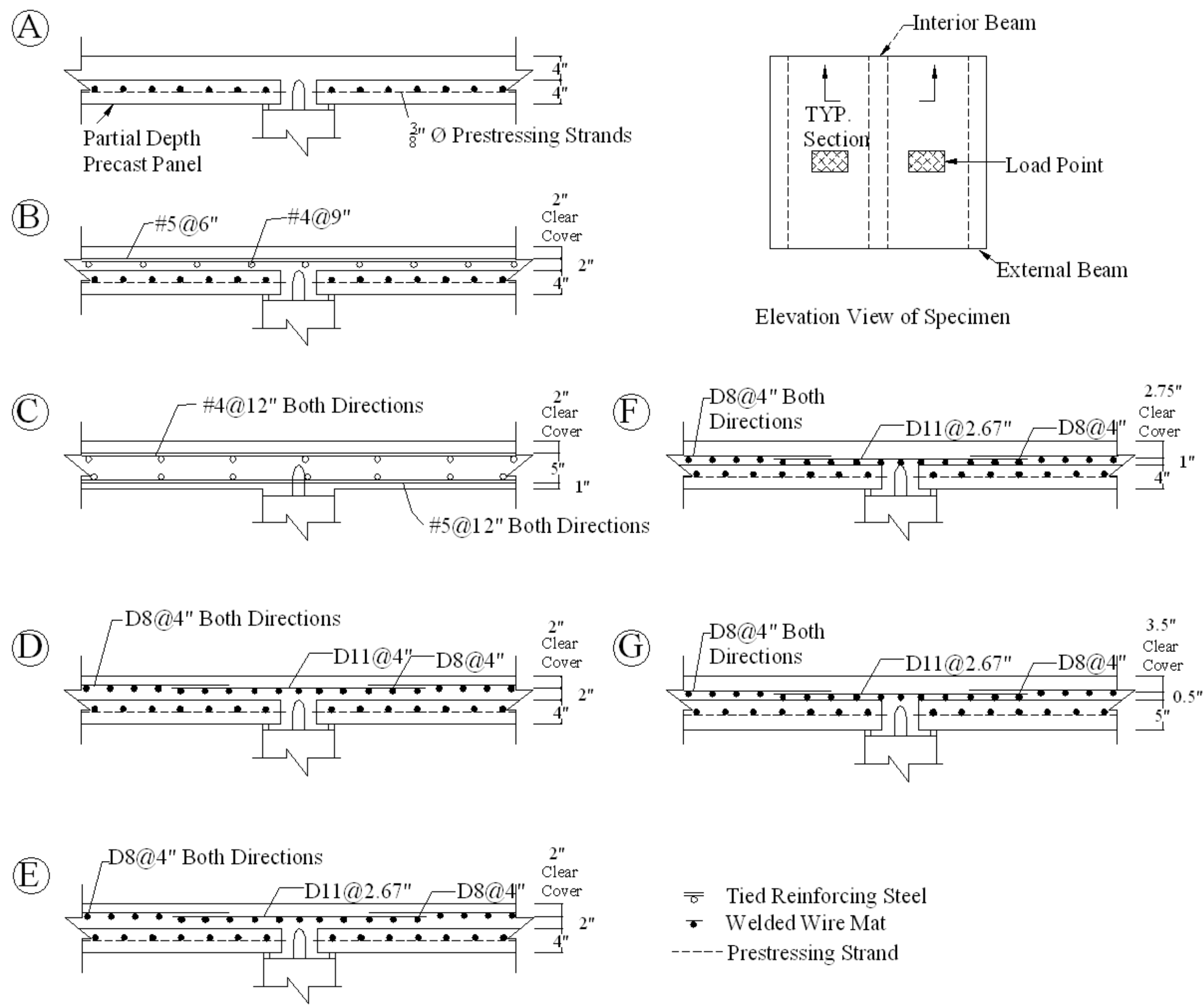


Figure 38: A graphical representation of the specimens tested.

2.3- RESULTS

An overview of the specimen details and results can be found in Table 7. The results given in Table 7 are for the total load placed on the specimen and so would need to be divided by two to determine the point load applied at each location. The cracking load corresponded to the load at which the first crack was visually observed. All of the specimens failed in either punching shear, a bond failure between the precast panel and the cast in place concrete, or a combination of the two. Some typical failures are shown in Figure 39, Figure 40, and Figure 41.

Table 7: A summary of the specimens tested.

Specimen Name	Construction Type: Negavite Moment Reinforcement at the Support Beam: transverse	Construction Type: Negavite Moment Reinforcement at the Support Beam: longitudinal	Construction Type: Partial Depth Precast Panel	Construction Type: Clear Cover (in)	Construction Type: Depth (in)	Cracking: Load (kips)	Cracking: Ratio to AASHTO Design	Failure: Load (kips)	Failure: Ratio to AASHTO Design	Load vs. Strain, Initial slope (kips/(in/in))
A	--	--	yes	N/A	8	27	0.9	283	8.9	82200
B	#5 @ 6"	#4 @ 12"	no	2	8	49	1.5	279	8.7	112000
C	#4 @ 12"	#4 @ 12"	yes	2	8	79	2.5	212	6.6	219000
D	D11 @ 4"	D8 @ 4"	yes	2	8	36	1.1	287	9.0	103000
E	D11 @ 2.67"	D8 @ 4"	yes	2	8	49	1.5	204	6.4	145000
F	D11 @ 2.67"	D8 @ 4"	yes	2.75	8	49	1.5	215	6.7	124000
G	D11 @ 2.67"	D8 @ 4"	yes	3.5	9	51	1.6	314	9.8	92600

The load reported is the sum of both load points.

The AASHTO Design Load is 32 kips per axle.

Figure 39: A punching shear failure of specimen A.

Figure 40: A sliding failure between the precast concrete panel and the cast in place concrete topping for Specimen B (standard TxDOT bridge deck). Note that this failure occurred at 8.7 times the design load.

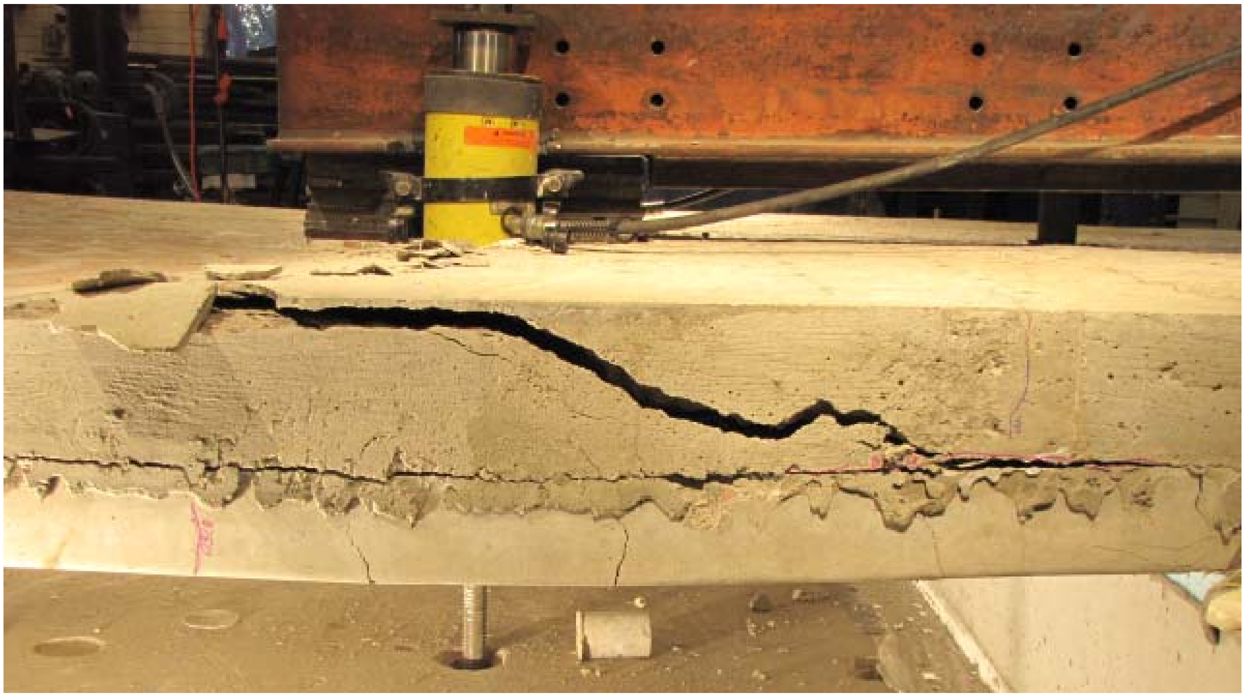


Figure 41: A combination punching shear and sliding failure of Specimen G.

One useful method of comparison between the specimens was to compare the magnitude of the maximum average surface strains on the failure side. This was always found to be at the edge of the interior beam and the bridge deck as shown in Figure 37. This point corresponded to the location of the largest crack during testing, as well as the largest moment.

The raw data from the average maximum surface strains from the failure side of the bridge deck can be seen in Figure 42. A smoothing technique was used so that an easier comparison of the data could be made. This was done by fitting a line to the two linear portions of the data. The typical procedure for this smoothing process is shown in Figure 43. The results of the smoothing technique for all of the bridge deck specimens can be seen in Figure 44. A summary of the slopes of the initial load versus surface strain measurements is given in Table 7. Please note that the final surface strain readings correspond to the last surface strain reading before failure. Because the measurements were manually taken then the measurement of strain at failure was not possible. This limitation should not be a problem as the general behavior of the system has been characterized.

One should note that the location in the bilinear behavior was not at the point of first crack for the specimens. Since the values for the load at cracking for the specimen was determined visually it corresponds to the load when the first localized cracking occurred. These cracks had to be much larger and more distributed before the stiffness of the system was noticed to change.

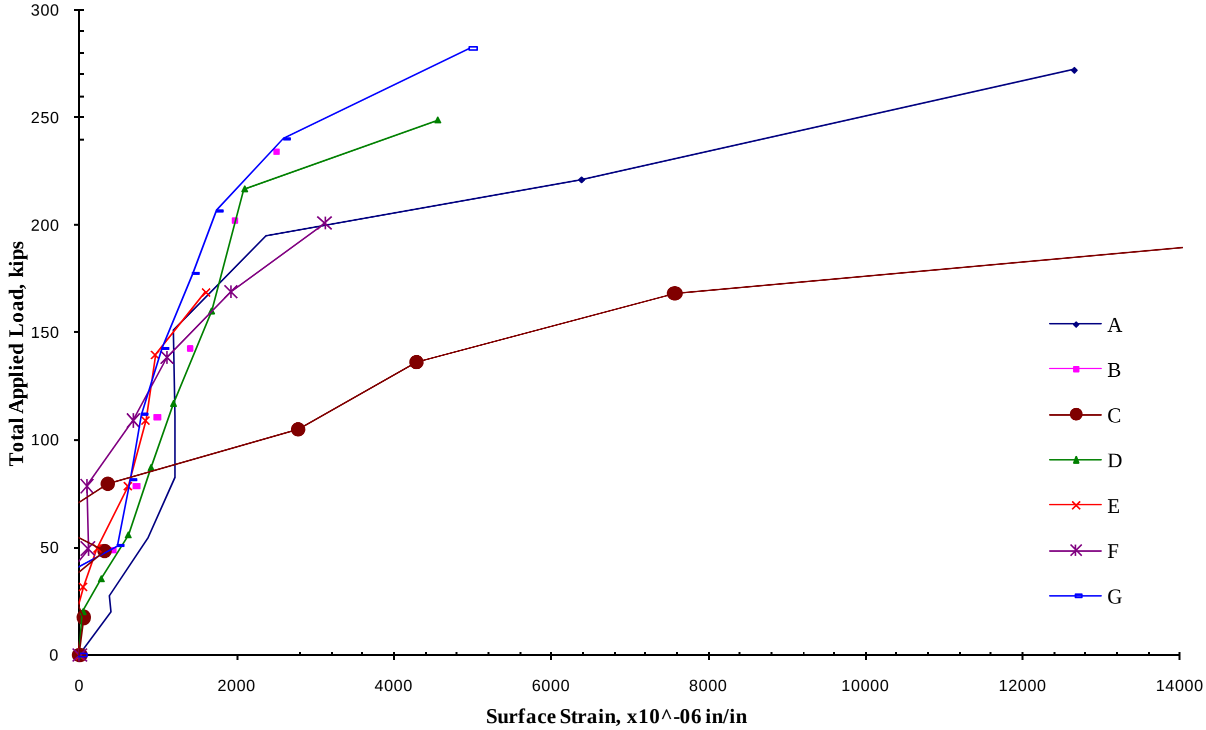


Figure 42: Raw data from the average maximum surface strains at the failure side of the bridge deck.

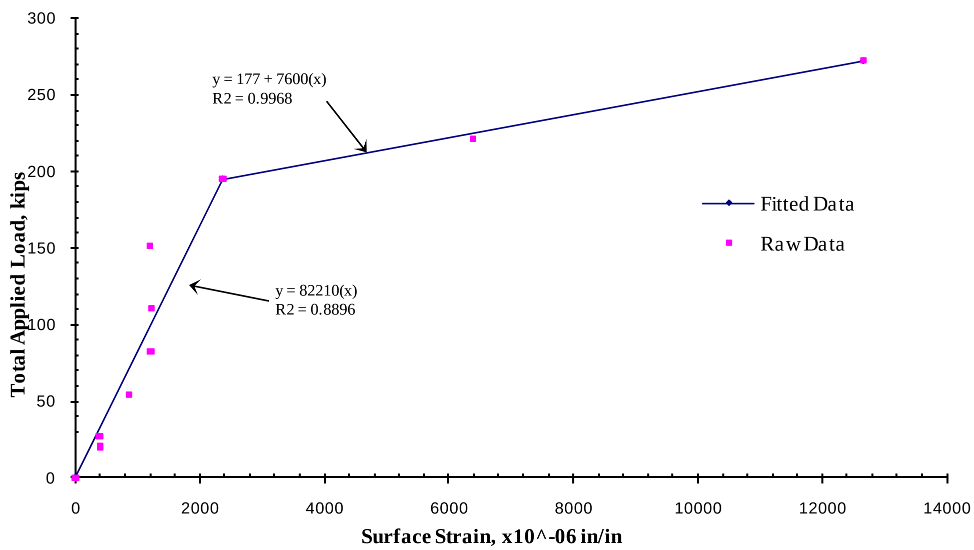


Figure 43: An example of the smoothing technique used for the data analysis in this report.

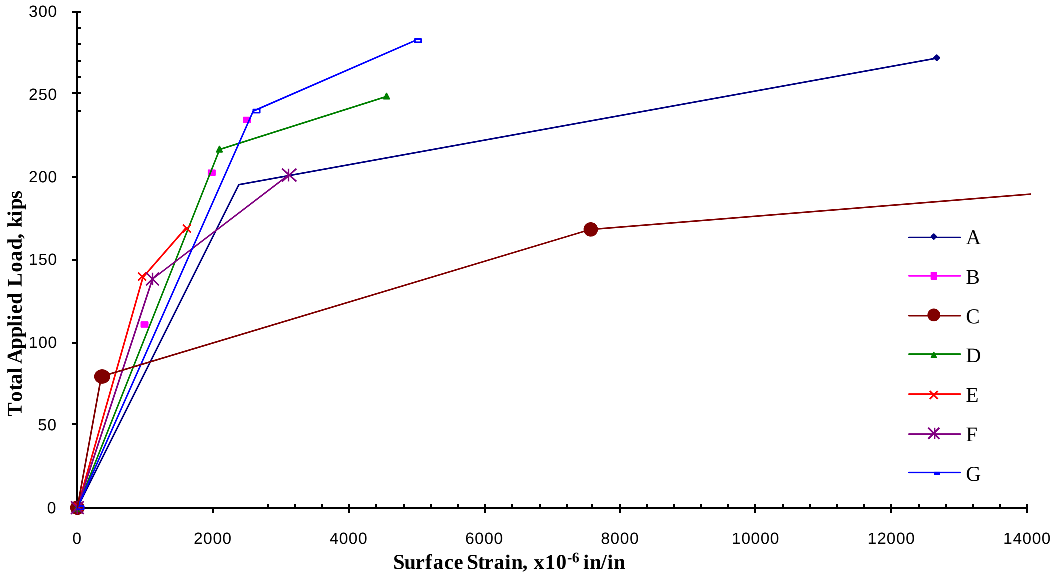


Figure 44: The smoothed results from the average maximum surface strains at the failure side of the bridge deck.

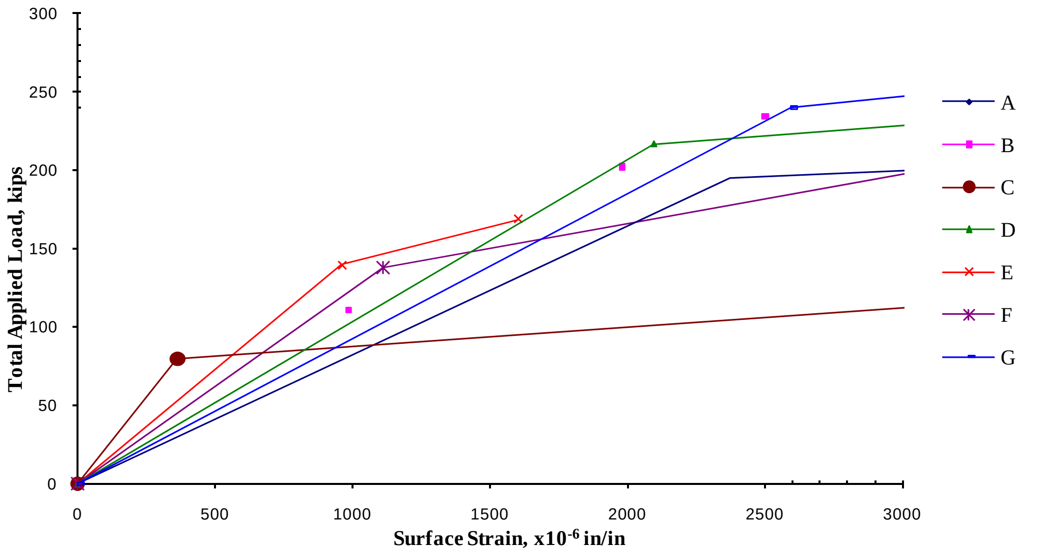


Figure 45: The smoothed results from the average maximum surface strains at the failure side of the bridge deck showing only the first 3000 microstrain for each specimen.

2.4- DISCUSSION

As can be seen in Table 7, every specimen tested showed a significant safety factor. The smallest ratio of the design load versus the actual load was 6.4 when compared to the HL 93, 16 kip point load or 32 kip axle load.

From the results one can see that all of the bridge decks tested provided satisfactory ultimate strength including the specimen that used no top reinforcing. Therefore, it appears that the steel provided in the top mat of a bridge deck is primarily used for resisting cracking.

Only two specimens showed a lower average maximum surface strain then the TxDOT bridge deck. Both specimens (E and F) consisted of a wire mat with D11 bars at 4” and D8 bars at 2.67” with 2” and 2.75” of clear cover. While the same wire mat at 3.5” of clear cover (specimen G) showed a performance less than the TxDOT standard bridge deck this data is still useful as it can be used as a point of interpolation. From interpolation between the 2.75” and 3.5” specimen a clear cover of 3” with this mat would prove to show a cracking performance equal to a TxDOT standard bridge deck with 2” of clear cover. Therefore, if one wanted to use the wire mats at an increased depth to optimize the construction tolerances then a mat with D11 bars at 2.67” transversely and D8 bars at 4” longitudinally could be placed at 3” of clear cover and an equivalent maximum surface strain or cracking performance should be expected between the bridge decks. If one used a bridge deck clear cover of 2” then by comparing the slopes of the average maximum surface strains the load at the failure side would be expected to be reduced by 30%. This reduction in maximum surface strain should also correlate to a reduction in crack sizes for the bridge deck by approximately 30%. While it is difficult to quantify, this reduction in crack size should correspond to the extension of service life of the bridge deck. If one used this wire mat at either depth of clear cover, then for a 4 beam bridge with 8’ beam spacing the steel used would be reduced by 30%. By using wire mats one would expect to significantly increase the speed of construction and reduce the amount of labor needed to construct a bridge deck. For bridge decks with larger beam spacing or with more beams, these improvements in economy and construction speed would be expected to increase.

It should be noted that while specimen C showed sufficient strength and outstanding surface strain performance up until the first cracking, the surface strain after first cracks were observed was not satisfactory. Of the specimens that contained reinforcing steel this specimen used the lowest amount and also performed the worst after first cracking. It is unfortunate that there was not enough funding in this project in order to investigate this behavior in more detail as this specimen had a much different load versus surface strain performance than the other specimens investigated. This behavior should be investigated with further research but is likely due to the presence of higher strength concrete in Specimen C and not using partial depth precast panels.

2.5- CONCLUSIONS

In this work welded wire mats were used to replace tied reinforcing bars with partial depth panels to improve the economy, constructability, and construction speed of bridge decks. Bridge decks have been constructed and tested that have used tied reinforcing and welded wire mats. The testing results suggest that:

- The specimen with no top reinforcing steel showed ultimate strengths similar to the other specimens but high levels of surface strain. Therefore, it appears that the top mat of reinforcing steel is primarily responsible for keeping the surface cracks of a bridge deck small before failure.
- A wire mat with D11 bars at 2.67” and D8 bars at 4” with 2” of clear cover provides a reduction in the average maximum surface strain by 30% when compared to the performance of a TxDOT standard bridge deck from first loading up until an axle load of 150 kips.
- A wire mat with D11 bars at 2.67” and D8 bars at 4” with 3” of clear cover should provide the same average maximum surface strain as a typical TxDOT standard bridge deck.

The improved ability of the wire mat to help the concrete bridge deck to resist the initial cracking could allow an owner a construction tolerance for the placement of the top mat of reinforcing. This would allow the contractor to place the wire mats with a clear cover near 3” and any geometry changes in the mats of up to 1” upwards could be ignored. The tolerance on the grading of bridge deck steel would allow for significant improvements in constructability of bridge decks as grading of bridge decks would be greatly improved.

RECOMMENDATIONS

From the work in Phase I "Development of a Precast Overhang for Bridge Deck Construction" the research team recommends that the cantilever on the proposed precast overhang system can be extended in length up to 5' while still providing satisfactory strength and serviceability performance. By allowing this extension of length of this system, the number of beams on a 30' roadway can be reduced from four to three. This can lead to a significant savings in the bridge construction costs.

From the work in Phase II "Use of Welded Wire Mats for Bridge Deck Construction", the research team recommends that, based on the testing in this research project, welded wire mats can be substituted for tied reinforcing steel in the top mat of a bridge deck while using stay-in-place concrete panels. The research found that by using a wire mat with D11 bars at 2.67" spacing transversely and D8 bars at 4" longitudinally with 2" of cover over the beams and then D8 bars at 4" in both transverse and longitudinal directions, a bridge deck can be produced with a sufficient amount of strength and improved durability while using about 30% less steel than a typical bridge deck. This same steel layout can be used with a clear cover of 3" with equivalent performance in strength and durability to current TxDOT bridge decks.

The improved ability of the wire mat to help the concrete to resist the initial cracking could allow an owner a construction tolerance for the placement of the top mat of reinforcing. This would allow the contractor to place the wire mats with a clear cover near 3" and any geometry changes in the mats of up to 1" upwards could be ignored. The tolerance on the grading of bridge deck steel would allow for significant improvements in constructability of bridge decks as grading of bridge decks would be greatly improved.

IMPLEMENTATION /TECHNOLOGY TRANSFER

The research in this project has been implemented in the construction of one bridge that is currently in service and two that are in the design phases. The Texas Department of Transportation is pleased with this system and is looking to implement it in a larger number of locations. Presentations have been given to a number of owners and contractors and several have been interested. The research team has been heavily involved in the production and review of the created plan sets and has used research results to answer several important DOT questions.
In addition four technical presentations have been made from this research and conference proceedings paper were published at the 2010 PCI/FIB International Conference. Two other peer-reviewed journal papers are being prepared for submission soon.

Also under this project a research symposium has been organized between OU, OSU, and Langston Universities. This meeting was held in Oklahoma City on July 26, 2010. The meeting featured 17 different technical presentations and was attended by over 60 people from the OTC partner institutions.

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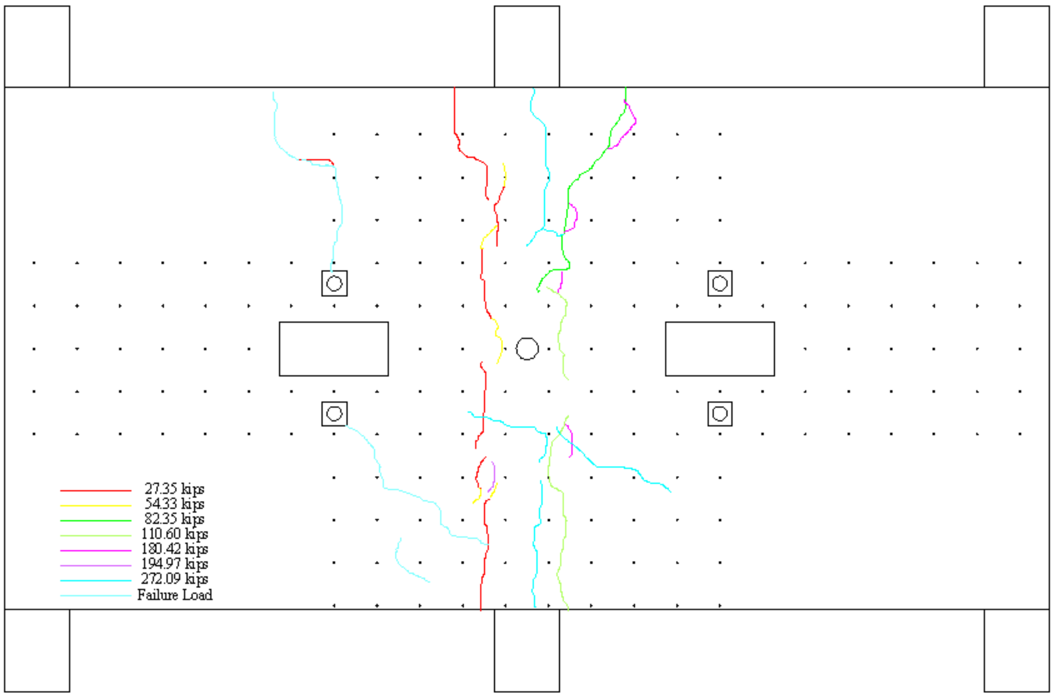
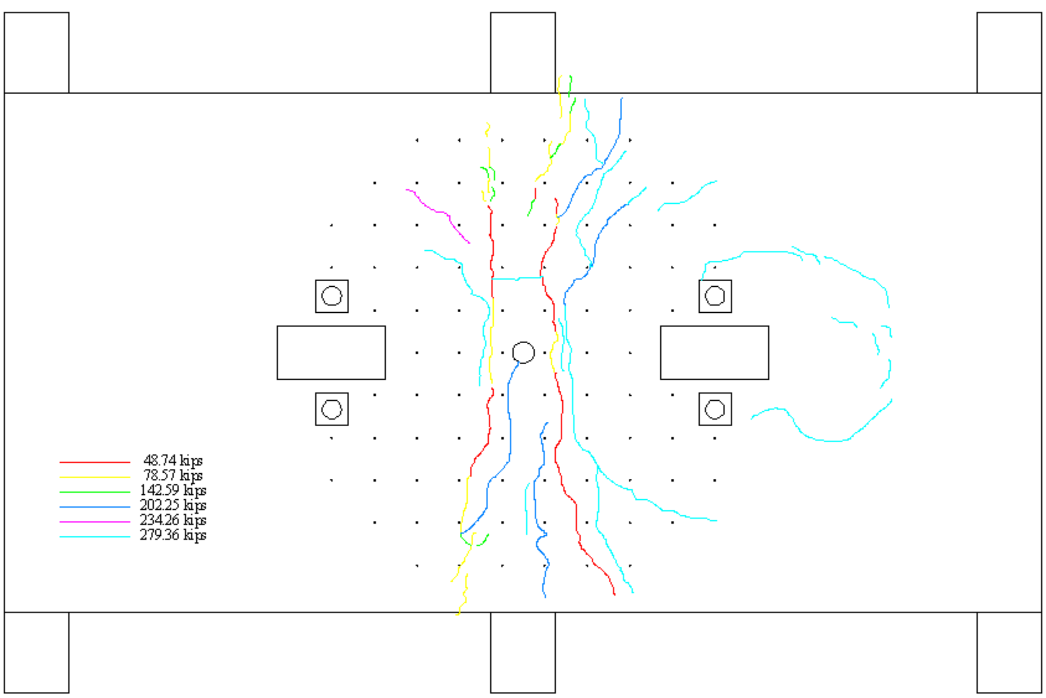
APPENDICES

Crack Maps and Demec gauge layout



Specimen **A**



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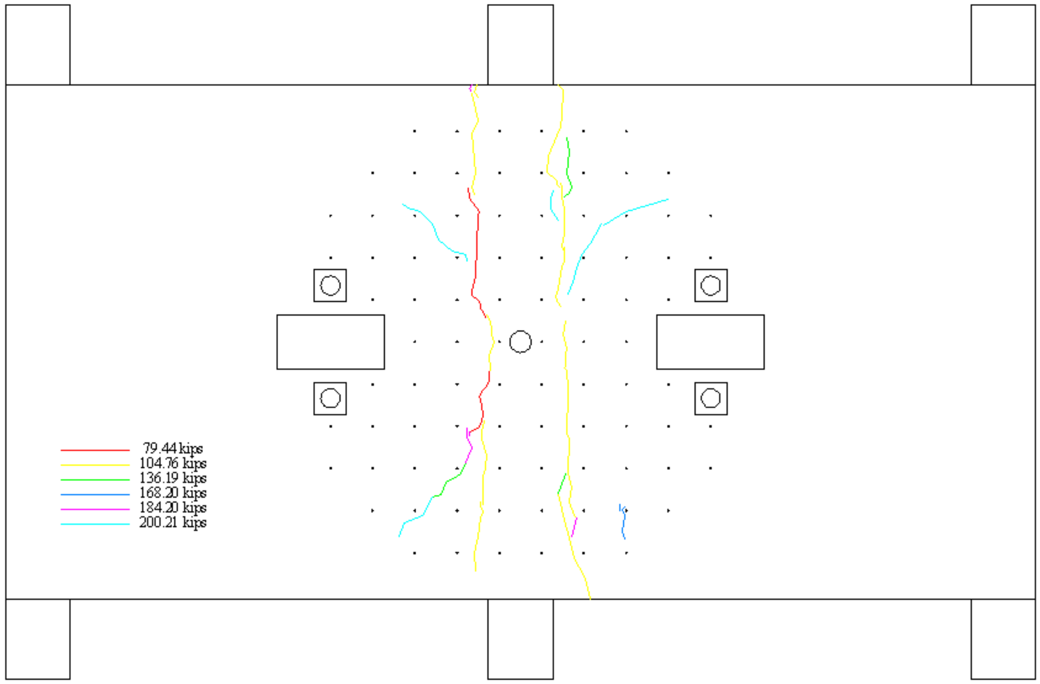

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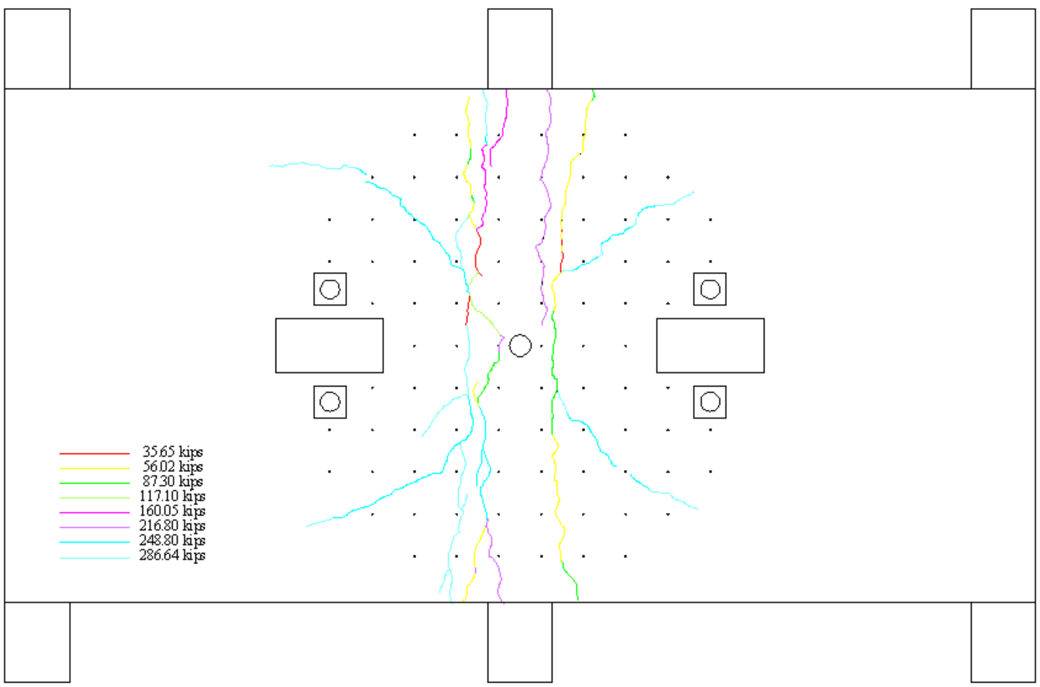

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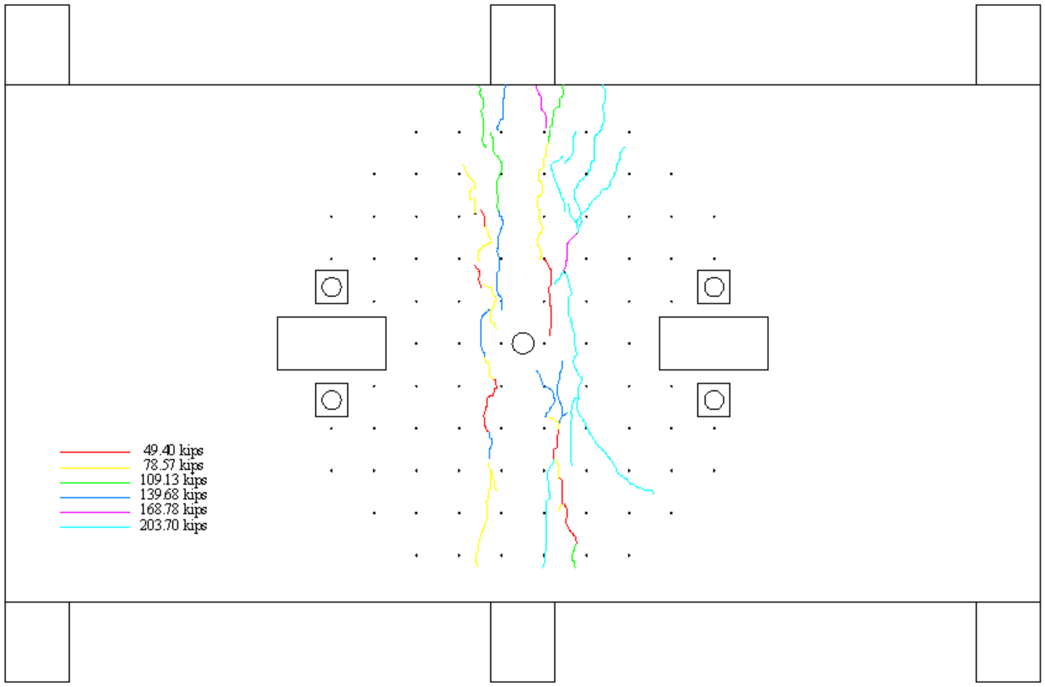
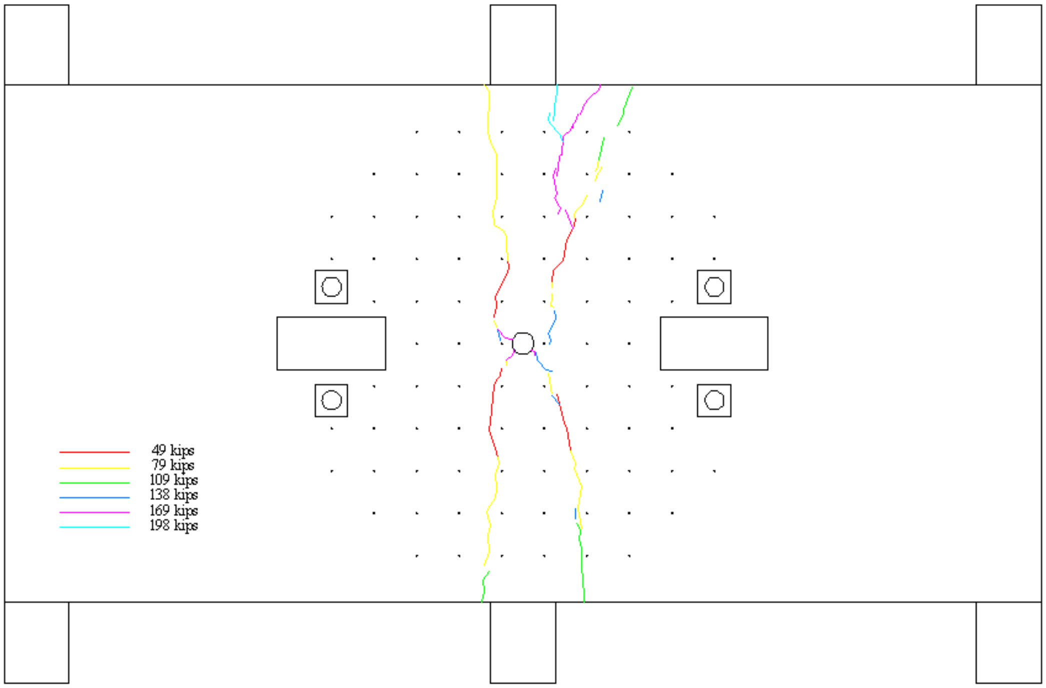
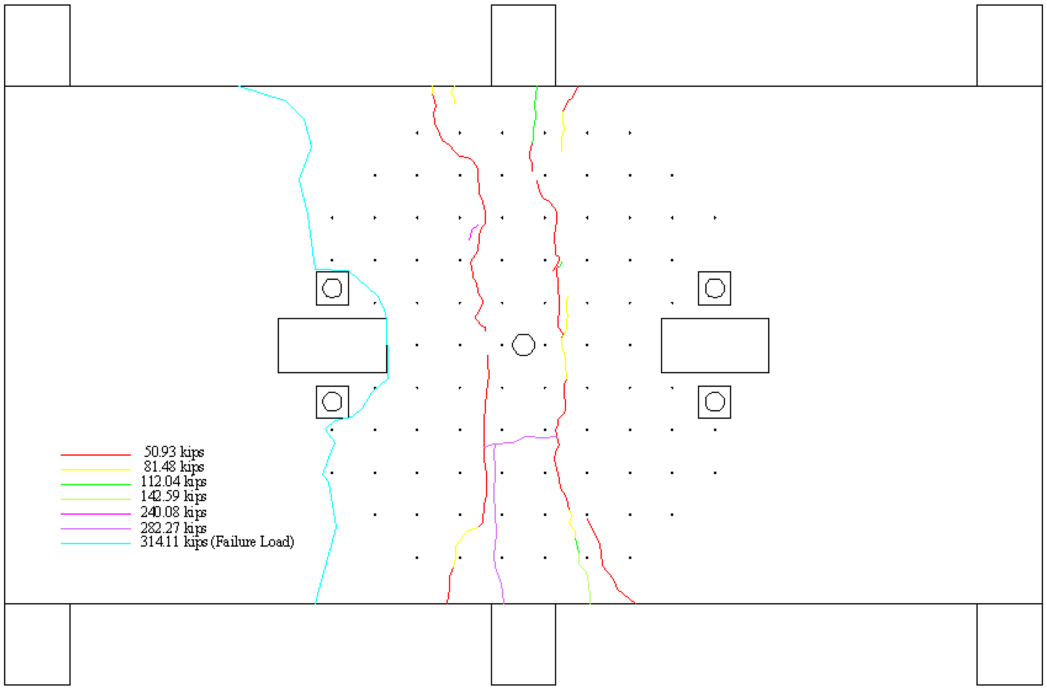


Specimen **E**



Specimen **F**



50.93 kips
81.48 kips
112.04 kips
142.59 kips
240.08 kips
282.27 kips
314.11 kips (Failure Load)

Specimen **G**