DEVELOPMENT OF PRECAST BRIDGE DECK OVERHANG SYSTEM

By

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DEVELOPMENT OF PRECAST BRIDGE DECK

OVERHANG SYSTEM

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CHAPTER I

1.0 INTRODUCTION

The demand for bridge deck repair and replacement is at an all time high and at the same time longer time for bridge closures is unacceptable due to the interference with the traffic flow. Out of the 597,851 bridges nationwide, almost 144,314 (i.e. 24%) are structurally deficient or functionally obsolete (www.betterroads.com 2007). Bridges in major metropolitan and rural areas would benefit from improved methodologies of bridge construction. In order to tackle this problem new and innovative methods are needed for bridge repair and construction that are able to provide cost-effective, long-lasting and rapid systems.

The American Association of State Highway and Transportation Officials Technology Implementation Group (AASHTO-TIG) has made numerous efforts to promote use of prefabricated bridge elements and systems among individual state departments of transportation (Ralls, et al. 2004; Ralls and Tang 2003). According to Federal Highway Association (FHWA) prefabricated concrete bridge elements and systems provides various advantages, such as safe work zone, improved quality of construction, lowering environmental impacts with least disruptions to the traffic flow (FHWA 2004). Use of prefabricated bridge elements reduces the amount of equipment required on the project site, eliminates the need to place piers in stream crossings, and reduces the amount of emissions produced by delayed traffic which in turn lessens the environmental impact.

The bridge element that directly resists the wheel abrasion and has the most exposure to the environment is the bridge deck; hence improved construction methodologies have attracted the attention of several researchers. Conventional bridge deck construction techniques are very linear or dependant on the previous task being completed before the next can begin. Also, bridge deck construction often involves a large amount of wood formwork to be erected and then removed once the bridge deck has reached the desired strength. This work is very labor intensive and can pose some safety risks to workers as they are required to work at an elevated height. Hence there is need for the development of a bridge deck system that provides an improvement in economy, durability, and speed over current construction techniques.

Precast bridge deck systems provide a very effective construction technique which can be implemented for the rehabilitation of existing highway bridges as well as new bridge construction. The development of a satisfactory system has potential to improve safety and speed of bridge construction (Maher 1997; Breger 1983, Ralls et al. 2004). Several of the recommended precast bridge deck systems have been investigated; however, none of them have been widely implemented because they were not able to satisfy the following challenges: (i) difficulty with adjustments to the system to meet construction tolerances; (ii) inability to provide a smooth final riding surface without extensive grinding or an overlay; and (iii) expense due to specialized equipment or materials required to construct the system. In spite of these challenges several systems have shown the ability to increase the speed of construction but with a cost to the durability and economy of the system.

The scope of the current study is to develop a bridge deck system for repair or new construction that addresses these previously mentioned challenges. The constructability and the structural behavior of a prestressed precast bridge deck overhang is investigated under static loading. The proposed system utilizes individual precast panels that are one half of the final bridge deck thickness to be used in the interior spans and an innovative precast panel that has a full depth and half depth section to be used in the overhangs and the first interior span (Fig.1.1D). 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. The investigation entails a detailed evaluation of the prestressed precast bridge deck overhang system under static loading to verify the following concerns about the system: (i) serviceability and functionality of the proposed bridge deck overhang system; (ii) compatibility of the proposed system as compared to traditionally used cast in place (CIP) bridge deck overhang system; and (iii) effect and adequacy of the adjustable haunch form system on the behavior of bridge deck overhang. To accomplish these objectives, a full scale load testing is done by simulating the load area of an AASHTO HL 93 truck that is increased until failure.

1.1 Background Information

It is common for construction to utilize members that are fabricated off site and then transported to the jobsite for construction. Typically these members are made up of prestressed concrete that can be constructed in a plant that has better quality control then at the bridge site. These precast elements are typically used to increase the durability and speed of a construction project but can also increase the economy if a significant number of elements are needed or if the form-work needed can be greatly reduced. For bridge construction it is quite common to use beam elements that are prefabricated and it is becoming more popular to use prefabricated elements for bent caps.

1.1.1 Partial Depth Bridge Decks

One bridge element that was recognized in the 1970s that could greatly benefit from precast construction is the bridge deck. This element is very repeatable and is quite costly to construct due to the labor required for: formwork construction and destruction, construction of the needed reinforcing cage, placement of the concrete and providing adequate curing. Some state DOTs started using partial depth prestressed precast panels as stay in place formwork in the interior portion of the span that receives some mild reinforcing as well as cast-in-place concrete in the early 1970s (Merril, 2002). With this system the cantilever portion is conventionally formed and overhang brackets are used as both formwork and a work platform. This system is shown in Fig.1.1B.

This 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. There was an extensive amount of research on this system by the Texas DOT (Bieschke and Klinger, 1982; Buth et al., 1972; Furr and Jones, 1970). This research found that this system was able to provide an economical bridge deck system with a large amount of reserve capacity. Currently, several states use this system as their standard method of bridge construction because of the improvements in safety, economy and speed over conventionally formed bridge deck construction.

1.1.2 Full Depth Bridge Decks

Beginning in 1985 several DOTs (Texas, Louisiana, New York, New Jersey, Vermont) started investigating the use of full depth precast bridge deck systems (Freeby and Ley, 2005; Badie et al., 2006). 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 Fig.1.1C. 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 to attempt to minimize the amount of cracking in the bridge deck.

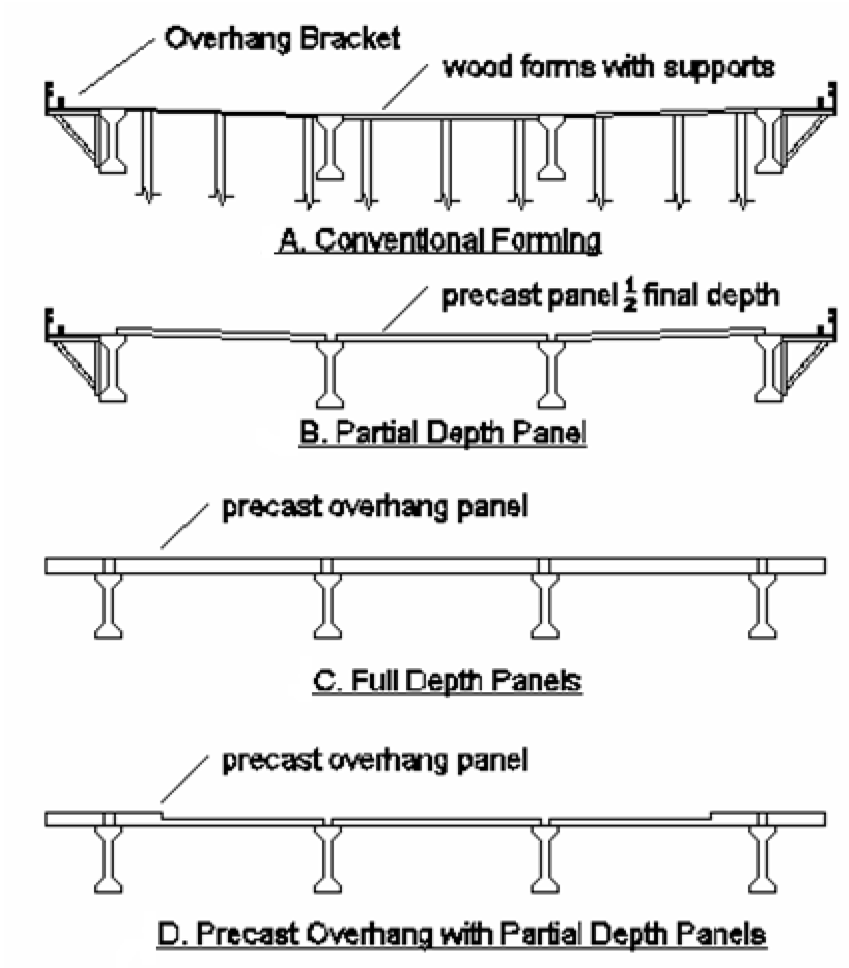


Fig.1.1: Display of various precast and conventional bridge deck systems.

There has been a flourish of recent research over this topic as several states continue to investigate these systems (Scholz et al. 2007, Badie et al., 2006). 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 this systems use. 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 Fig.1.2. 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 (Scholz et al., 2007; Hyzak, 2008). This increase can be attributed to large shipping weights, increase in crane size, and additional wearing surface or grinding.

Fig.1.2: A wooden stick placed at the intersection of two full depth precast panels showing the difference in panel height to be almost ¼".

1.1.3 Partial Depth Bridge Decks with Precast Overhangs

While reviewing the benefits and challenges of the full depth and partial depth bridge decks it was realized that the systems could be combined in a hybrid system. These reviews lead to the creation of the system shown in Fig.1.1D where a precast overhang member is used that has a full and partial depth section. This precast panel in the overhang removes the need for forming or a work platform as the full depth section can provide it. On this platform a screed rail is attached that allows the bridge deck to be finished to provide the desired riding surface. The grade bolts commonly used in the full depth bridge deck systems can be used to adjust the overhang surface.

Because the partial depth panels are used in the interior spans the deck surface can be easily adjusted to meet the desired profile and provide the correct clear cover on the reinforcing steel by adjusting the height of screed and the cantilever with the grade bolts. In the current system a 4” topping of reinforced concrete is placed on the precast panels.

1.2 Research objectives

The basic aim of the current research project is to develop precast bridge deck system with the least amount of form work which will enhance safety in the work zone along with fast and high quality construction technique to construct a bridge deck. To achieve this objective a precast bridge deck overhang system which will reduce the form-work required for construction of a bridge deck overhang for partial depth bridge deck system (stay in place form system) used by several DOTs. As per the aforementioned requirements the hybrid overhang system was developed and tested for static loading as per AASHTO LRFD. To make this system work and eliminate the disadvantages of full-depth bridge deck system there was need to develop innovative haunch form system which will reduce the critical job of adjusting haunch height especially when precast girders are used with the proposed precast bridge deck system. Hence, there was need to develop adjustable haunch form system.

Haunch form-work was neglected during the development of precast bridge deck system in the past. The haunch is the gap between bridge girder and deck slab which plays a vital role for smooth finished surface of bridge deck using precast panels. Chapter 2 focuses on the development of adjustable haunch form work system. This system makes the process of adjusting haunch height simple and provides smooth finishing surface for bridge deck overhang. Investigation of this system was done by testing it for several load cases which it will see during construction.

Chapter 3 discusses about the development of precast bridge deck overhang system. This system totally eliminates the need of form to construct bridge deck overhang. Chapter 3 focuses on the load testing of the proposed bridge deck overhang as per AASHTO LRFD and comparison of the behavior of proposed system with traditionally used cast-in-place (CIP) bridge deck overhang system.

CHAPTER II

2.0 DEVELOPMENT OF ADJUSTABLE HAUNCH FORM

2.1. Introduction

Precast bridge deck systems provide a very effective construction technique which can be implemented for the rehabilitation of existing highway bridges as well as new bridge construction. The development of a satisfactory system has potential to improve safety and speed of bridge construction (Maher 1997; Breger 1983, Ralls et al. 2004).

One area that has not been addressed in previous precast bridge deck systems is the role of the haunch in the construction of a bridge deck. The haunch, the space between the bridge girder and deck (refer fig. 2.1), is used as an area of adjustment between the bridge girders and the deck to provide the correct roadway profile and bridge deck thickness. Determining the height of the haunch can become especially challenging when prestressed concrete beams are used as the camber can be quite variable between the bridge girders of the same design depending on how long the girders are stored before use (Kelly et al. 1987).

There has been several precast bridge deck systems developed in the last two decades and implemented by various DOTs that have realized that adjustment is needed to meet construction and grading tolerances. However previously developed systems have largely ignored the impact of this adjustment on the haunch and often require workers go back under the bridge once the geometry is established to manually complete the forming of the haunch (Tadros et al. 2006; Sullivan 2007).

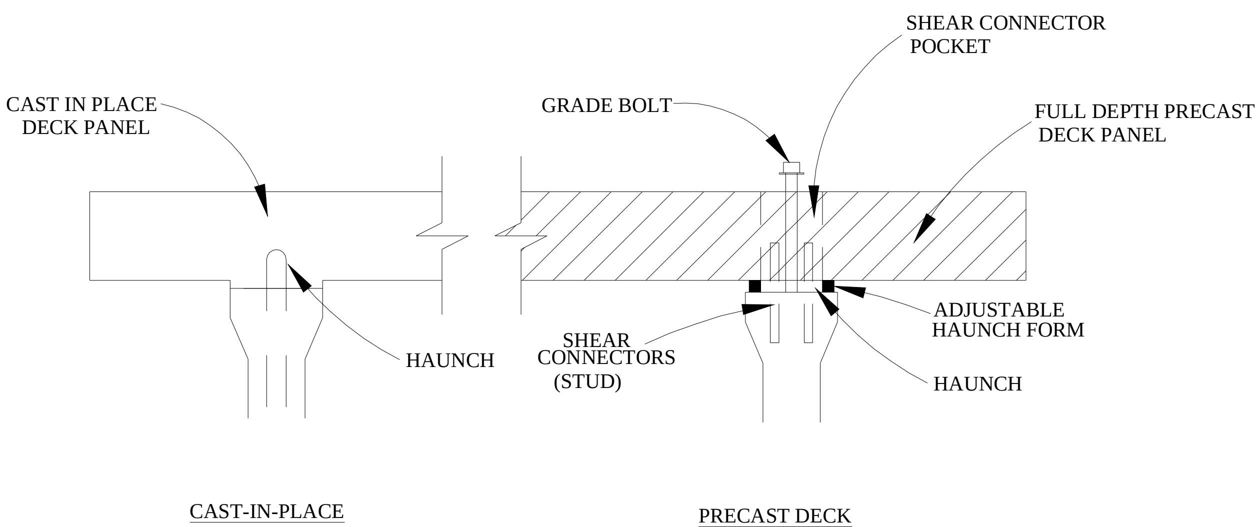


Fig. 2.1: Location of haunch for Cast in Place (CIP) and precast bridge deck system.

While these approaches appear to have been satisfactory for a small number of projects, the performance of precast deck systems can be improved and a wider implementation of the technology would be expected if a forming system is used that does not require work to be completed under the bridge deck. This would require a forming system that is able to resist the lateral pressure from the fresh concrete or grout material filling the haunch, allow for an easy adjustment of the system, and not require workers to work under the bridge deck for either installation or removal.

Location of haunch for Cast in Place (CIP) and precast bridge deck overhang is shown in fig. 1. It also shows the proposed adjustable haunch form for precast bridge deck overhang along with the shear connector pocket, shear connectors and grade bolts. Grade bolts are used for adjustment of haunch height while shear connectors are used for shear transfer from bridge deck to the girders.

In this chapter packing foams are investigated to be used as a stay in place adjustable haunch form for the precast bridge deck construction that is attached with and without adhesive. The foam adhesive combination is easily compressed or elongated, and does not absorb water. Several tests were designed to simulate the performance of the packing foam and adhesives in different phases of bridge deck construction. Based on the results of testing, recommendations were given for using foam as a forming material for an adjustable haunch form for precast bridge deck construction.

2.2. Materials

2.2.1 Packing Foam

Based on conversations with foam manufacturers two different types of closed-cell foams were investigated. These foams were chosen for their high resistance to water absorption and durability. A polyethylene (PE) and a cross link (CL) foam of different densities were investigated. The PE foam is produced by polymerization of ethylene and trapping air bubbles within the matrix. This material is typically extruded into sheets. These sheets can be laminated together to build up different thicknesses. The CL foam is similar but different specialized polymers are used in combination with cross linking reagents. By adding these cross linking reagents the physical properties of the foam is greatly altered and the density of the material is increased which greatly impacts the strength and stiffness of the foam. The CL foam is also extruded and can be laminated to form different thicknesses. Both foams are commonly used as packing foams for computer components, are economical and also widely available. Different densities of the PE and CL foam were investigated as they have a significant impact on the foam properties.

A summary of the foam properties from technical literature is provided in Table 2.1. These properties are typically specified when foams are used as packing materials and are commonly available from distributors. Foam one through three are PE foams and foam four and five are CL foam with different densities. Typically, as a foams density increases so does the elastic modulus and tearing resistance.

Table 2.1 – Summary of the manufacturer reported foam properties (PXL, 2003; Pregis, 2005).

Property	Foam Number 1	2	3	4	5	Test Method ASTM
Type of Foam	PE	PE	PE	CL	CL	-
Density (lb/ft^3)	1	1.2	1.7	2	4	D-3575-W
Stress for a given deflection (psi)						D-3575-D
25%	3	5	5.5	5	9	
50%	6	10	12.5	14	19	
Increase in deflection from a sustained load (%)						D-3575-B
2hrs	30	30	34	-	-	
24hrs	24	24	20	-	-	
Increase in deflection for a1 psi load (%)	12	5	3	-	-	D-3575-BB
Tensile strength (psi)	20	38	26	54.5	84	D-412
Elongation capacity (%)	75	75	59	237	311	D-412

2.2.2. Adhesives

Next adhesives were identified that were compatible with both concrete and the foam. Three types of adhesives investigated including: (A) synthetic elastomer liquid, (B) two part epoxy, and a (C) aerosol adhesive. In the remainder of the discussion each adhesive will be referred to by its corresponding letter. Table 2.2 shows summary of the adhesive properties provided by manufacturers.

Table 2.2 – Summary of the manufacturer reported adhesive properties (Lord 2002, 3M 2002, 3M 2005).

Properties	Type of adhesive			Test Method ASTM
	A	B	C	
Color	Light amber	Blue syrup	Blue	-
Coverage (ft^2/gal)	308	320	213.33	-
Viscosity (cps)	175-275	N/A	N/A	-
Work Time @ 75˚ F or 24˚ C (Hrs.)	0-1	1-2	8	-
Tensile strength (psi)	-	2490	-	D882-83A
Elongation at break	-	31%	-	D882-83A
Coefficient of thermal expansion (mm/mm˚C)	-	365×10^{-6}	-	-

2.3. Experimental Methods

The values in Table 2.1 are useful to the packing industry and values in Table 2.2 are useful for general use of adhesive but do not provide all of the information needed to determine if the foam and adhesive would act as a satisfactory haunch form material. Because of this, tests were developed to evaluate how packing foam and different adhesive combinations would perform as a haunch form for precast bridge deck construction.

These tests investigate the ability of the foam and adhesive combination to resist lateral pressures that may occur from concrete or grout, ability to resist elongation that may occur if the foam is glued in place and then the system is adjusted upward, and a combination of upward adjustment or elongation and then subsequent lateral pressure. Several other tests were also included to investigate the robustness of the system to changes in temperature and at the joint. These tests also investigate the memory of foam specimens.

2.3.1 Test specimens

Each test used a specimen that was prepared with a standard method. The specimen was 3" in height by 1.5" in width with a length of 10.5". A typical specimen is shown in Fig. 2.2. The 3" height was chosen as it was a reasonable upper bound of a typical bridge haunch. A height to width ratio of 1:2 was chosen as it was a typical aspect ratio. The specimen length of 10.5" was chosen as it met the size of the available testing equipment and it was long enough so that the foam behavior could be evaluated at the center and edge related behavior could be minimized.

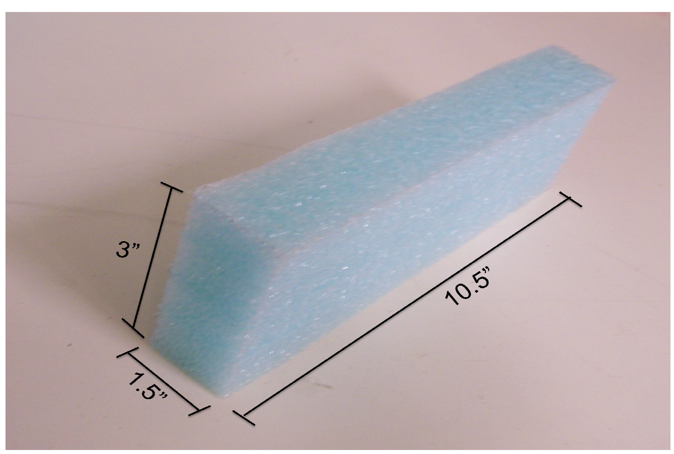


Fig.2.2: Dimensions of foam Specimen used for testing.

The test specimens were prepared according to the following procedure and as shown in Fig. 2.3:

1. The foam was cut into planks that are 10.5” x 3” x 1.5” as shown in Fig. 2.2 with a table saw.

2. Concrete blocks with dimensions 18” x 3” x 3” were made of 5000 psi concrete with 1/2” nominal size aggregate.

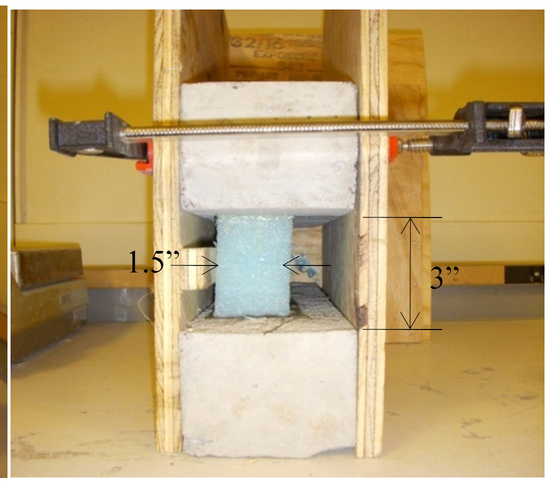
3. A wooden jig was used to support the specimen to ensure that the foam plank remained vertical.

4. First a concrete beam is placed in the jig and 10 grams of adhesive is applied to thoroughly cover the trowel finished surface of the concrete beam (Fig. 2.3a). This is done to simulate the top surface of the precast beam. Next a foam plank is placed on the glued covered surface. Ten grams of glue is then applied to the top surface of the foam in the same manner (Fig. 2.3b). Finally the formed surface of the concrete beam is placed on the foam to mimic the formed surface of the precast panel (Fig. 2.3c). Ten grams of glue was chosen as it was the amount of material needed to thoroughly cover the interface between the concrete block and the foam surface.

5. This setup is then allowed to set under gravity load while supported in the jig for the one day. (Fig.2.3d).

While preparing test specimen it was important to ensure that a surface was used on the concrete blocks that is similar to the surface used in the actual structure. For this reason the foam was glued to a troweled concrete surface to simulate the top surface of the precast beam and to a formed surface to represent the bottom of the precast panel.

(a) (b)



(c) (d)

Fig.2.3: Step wise procedure to prepare test specimen; a) Applying adhesive on trowel finished surface of beam, b) Placing foam plank on glued surface and applying adhesive on foam plank, c) Placing of concrete block on the foam, d) Specimen allowed setting under gravity load.

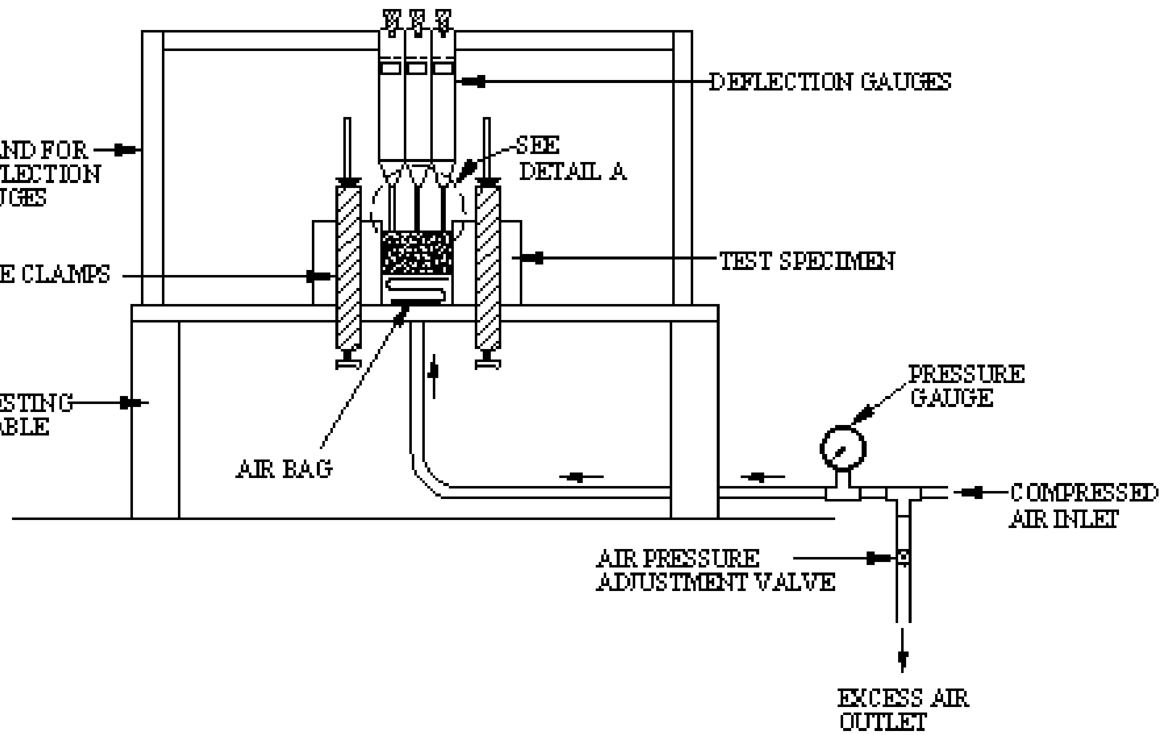
2.3.2 Test Methods

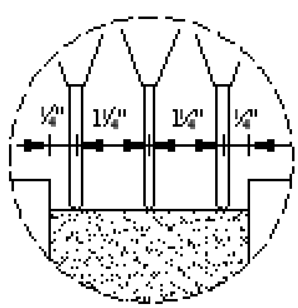
Three major tests were carried out to investigate the different combinations of foam and glue. These tests specifically investigated the ability of the foam and adhesive combinations to provide sufficient lateral pressure resistance, elongation and memory. The lateral pressure tests were further modified to investigate combinations of elongation and lateral pressure as well as investigations with no adhesive or the effects of curing temperature on strength.

2.3.2.1 Lateral Pressure Test

This test is designed to investigate the ability of the foam and adhesive system to resist the lateral pressure that results from the fluid pressure of grout or concrete. This is achieved by examining the capacity of a foam strip glued on its top and bottom to a concrete block with one of the previously mentioned adhesives.

The lateral pressure on the foam is applied using an air bag which is monitored with a pressure gauge and a regulator valve to adjust the pressure. The specimens are supported on their side on a wooden table over an air bag while the concrete blocks are fixed to the table using clamps. The test setup is shown in Fig. 2.4. Care must be taken to insure that the air bag applies pressure uniformly. This was done by placing the specimen over central region of the air bag as shown in Fig. 2.5. Deflection gages were used in the test to measure the deflection at 0.25" from the edge and at the center of the specimen (1.5" from the edge); detailed drawing is shown in fig. 2.4.





Detail A

Fig. 2.4: Experimental setup for Lateral Pressure Test.

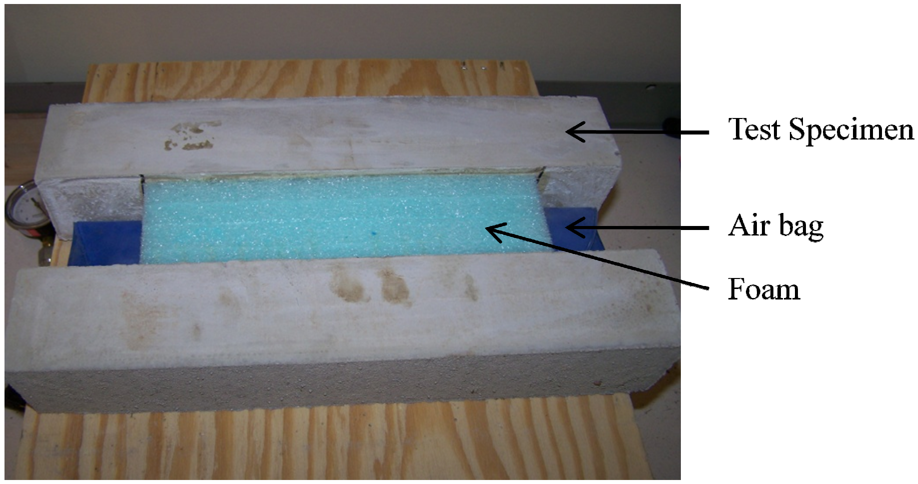


Fig.2.5: Test Specimen placed over air bag.

This test was conducted at either one or two days of adhesive curing. The deflection of the foam specimens were measured at regular intervals starting at 1.5 psi and increasing by 1 psi until 6.5 psi was reached. At each pressure interval the loading was held constant for 1 minute to allow the deflection of the system to stabilize. The value of 6.5 psi was chosen because it was the capacity of the air bag equipment used in the testing and is also a reasonable upper bound on the amount of pressure that one might see from fresh concrete or grout. This would roughly correspond to 6.5' of concrete head or 7.8' of grout head. An example of a failed specimen is shown in Fig. 6.

Three specimens were tested for each result. For each individual test, the lateral pressure at failure and specimen deflections at the different load steps was recorded.

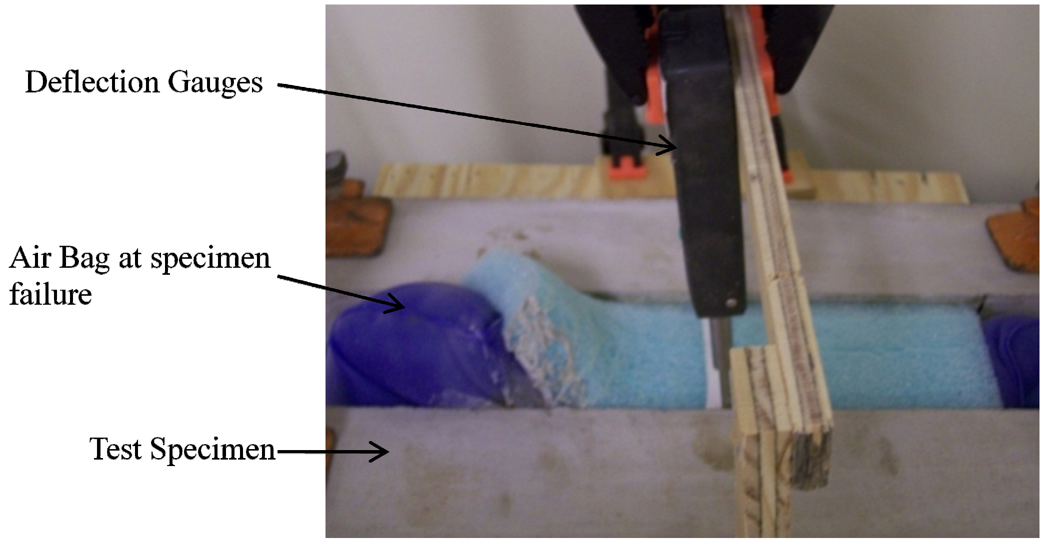


Fig.2.6: Lateral Pressure test specimen at failure.

Several modifications to the lateral pressure test were made and they are discussed in Section 2.3.2.1.1 to 2.3.2.1.4.

2.3.2.1.1 Elongation and Resistance to Lateral Pressure

The lateral pressure test was modified to investigate the ability of the foam and adhesive combination to resist lateral pressure after it has been elongated. This was done to simulate if a form was glued in place and then adjusted upwards after the glue had gained strength and then subjected to lateral pressure. The combination of tension on the glue and then a subsequent shear from the horizontal pressure was thought to possibly be critical. This was evaluated by comparing the lateral pressure capacity of the foam and adhesives after a specimen had been elongated by 0.25”. A value of 0.25” was chosen for the elongation as none of the foam and glue combinations failed at this elongation.

After a specimen was placed in the testing setup as shown in Fig. 2.7 small screw jacks were used to elongate the specimen by 0.25”. The specimen is then clamped to the testing table and a lateral pressure is applied. The deflection at different lateral pressures was completed in a similar manner to the lateral pressure test.

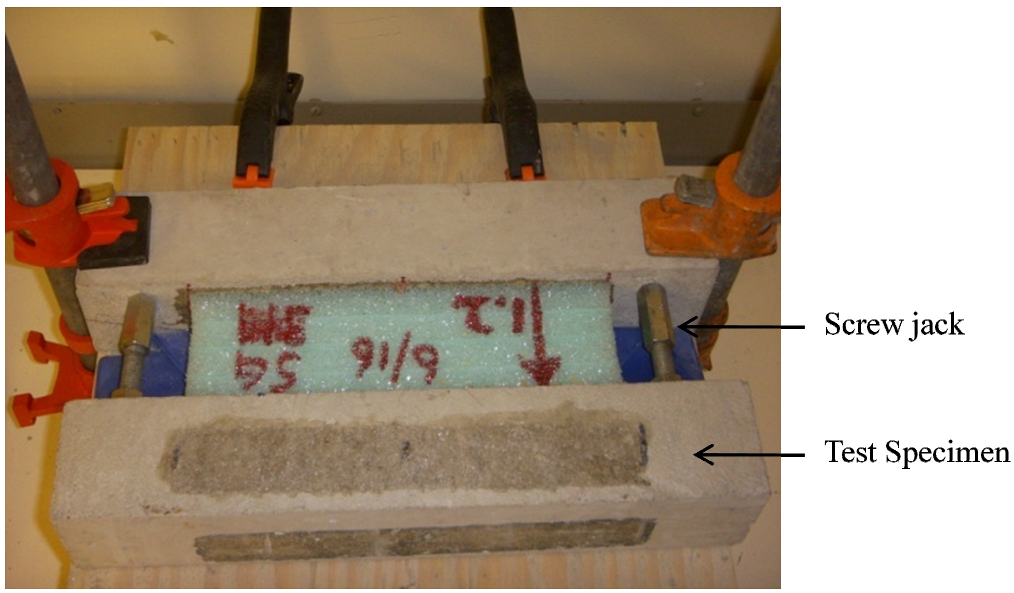


Fig.2.7: Picture showing intermediate stage of elongating test specimen using screw jacks for elongation and resistance to lateral pressure test.

2.3.2.1.2 Lateral Pressure Test with No Glue

This test was conducted to observe performance of foam used as a haunch form without the application of adhesive. Foam specimens were crushed by 0.25”, 0.50”, 0.75” and 1” and then subjected to lateral pressure to examine the capacity of the friction between the concrete specimens and the foam.

2.3.2.1.3 Lateral Pressure Test on specimen cured at 45° F

In this test, specimens where cured at 45° F, instead of the 73° F used in the other testing, to observe the effect of temperature on curing of adhesive. Specimens were cured at 45° F for one day and where tested for lateral pressure capacity. Specimen preparation and test procedure was same as that of lateral pressure test.

2.3.2.1.4 Lateral Pressure Test at a Foam Joint

After conducting Lateral Pressure Test on full length foam specimen, it was decided to cut the specimen so that a joint was in the center as shown in Fig.2.8. This allowed the performance of the haunch foam to be investigated at a joint. The remainder of the test procedure is the same as the lateral pressure test described previously.

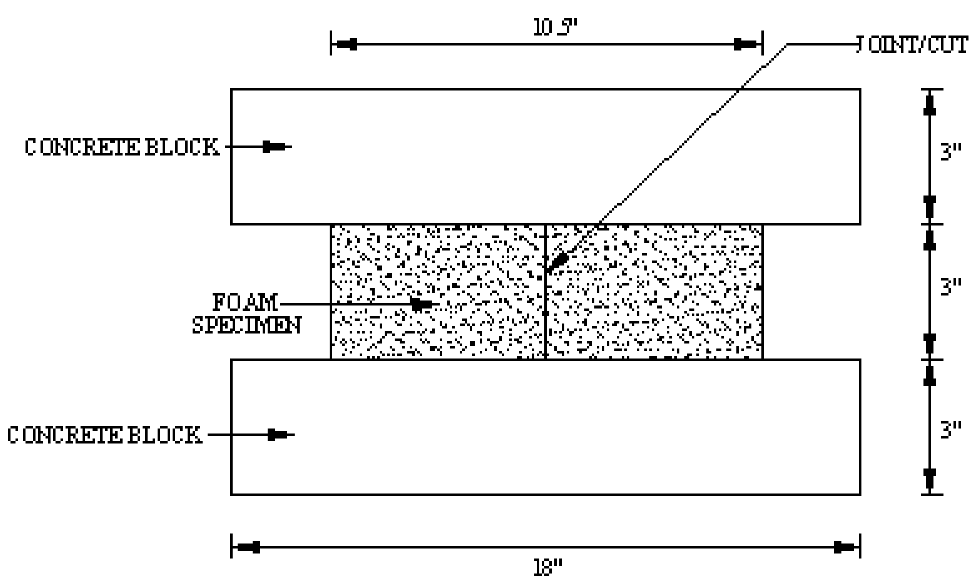


Fig.2.8: Specimen used for joint test.

2.3.2.2 Elongation Test

This test focuses on the elongation or tensile strain capacity of the foam and adhesive in combination. This test was designed to simulate a situation when the haunch form has been installed and then an adjustment is made to the geometry which causes the foam to be elongated. The ability to allow for an adjustment of the precast panel height is crucial to the constructability of a precast bridge deck system.

In order to simulate this, a specimen was placed in a Universal Tinius Olson Machine after different durations of curing. The specimen was then pulled apart at a rate of 10 lb/minute this rate was used as it was easy to observe the load on the specimen and was at a similar rate as might be expected to occur in the field. The specimens are prepared as described previously and then clamped to steel plates that are fixed to the load heads of the machine. A level was used to insure that the specimen was attached with minimal eccentricities. During the testing two deflection gages were used to monitor the deflection of the specimen. The test assembly is shown in Fig.2.9. Care was taken to insure that the foam height was 3” before a specimen was investigated. It was necessary to do this to insure that the height of the foam was not inadvertently changed while securing the specimen.

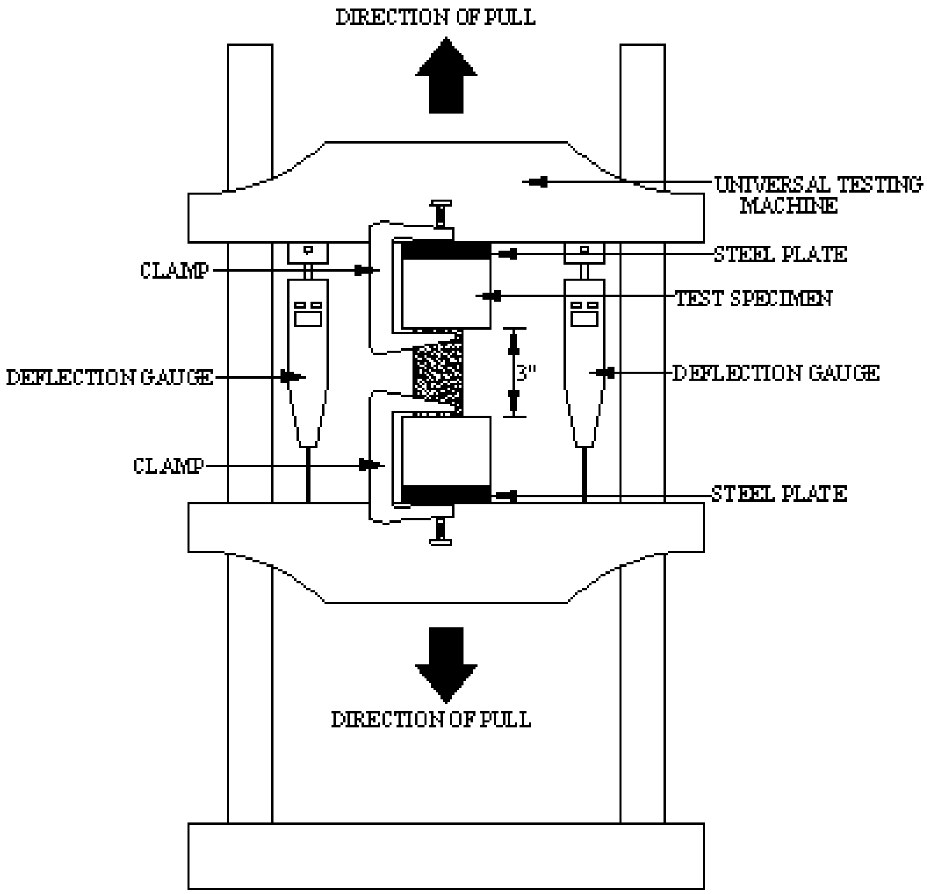


Fig.2.9: Experimental setup for the tension test.

The specimen was loaded until a tear was observed in the foam to adhesive bond that was wide enough for grout to pass through (about 1/16”). Observation of the failure was easy to witness if a light was used behind the specimen so that the tearing would be highlighted. The load is then stopped and the deflection readings on the gages were recorded. Elongation of specimen was measured at failure up to a 1” with a 0.005” precision. If the specimen does not fail at 1” elongation then the test was stopped and an elongation of 1” was reported. The value of 1” was chosen because it was the range of deflection gauge used in testing and is also a reasonable upper bound to the amount of elongation that one might see during adjusting the height of precast overhang panel.

A single tension test consisted of the average of three individual specimens.

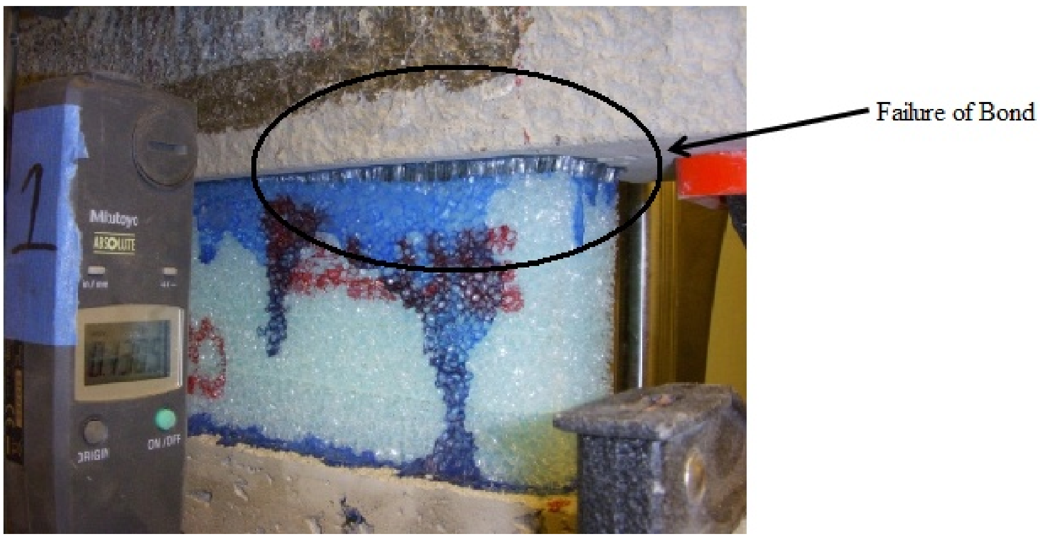
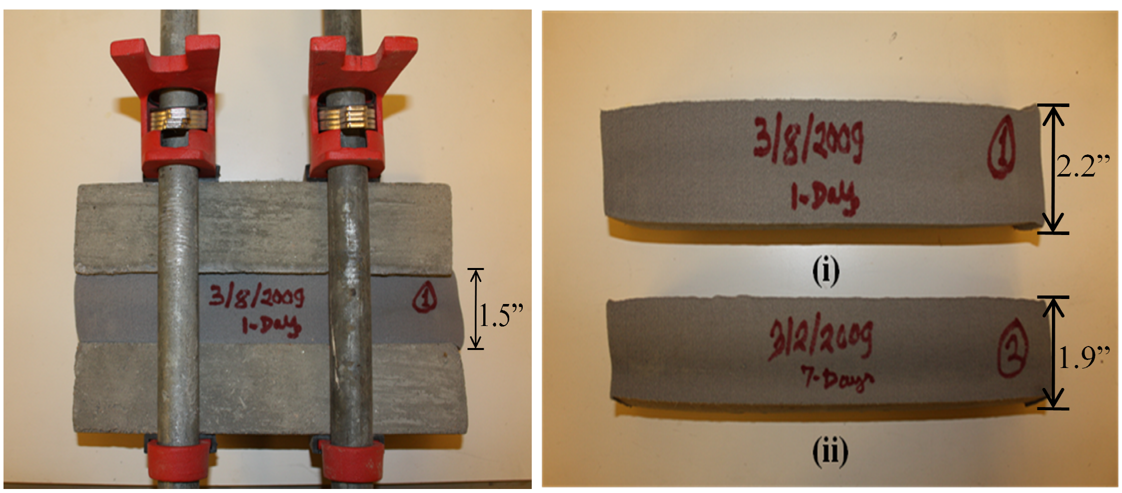


Fig.2.10: Failed adjustable haunch specimen during tension test.

2.3.2.3 Memory Test

A test was conducted to evaluate the ability of a foam to return to its original height after being crushed by 50% of its original height. This ability to return to the original geometry after a loading event is referred to as the memory of the foam. This test provides an estimate of the amount of height change that is expected from the foam if a precast panel is initially placed directly on the foam and then raised upward with grade bolts. If the system is raised upward after the adhesive has gained strength then the adhesive and foam could be placed in tension. If this geometry change occurs before the adhesive has gained strength then the foam will need to adjust upward to remain in contact with the panel above. This information can be useful to evaluate adjustment restrictions on raising panel during construction.

For this testing an unglued foam specimen was crushed between two concrete blocks. Each specimen was crushed to 1.5" or 50% of the original height of the foam specimen using pipe clamps as shown in Fig. 2.11. Two sets of specimen were investigated; one set was left for one day and the other for seven days. Each set consists of three individual specimens. The height of each specimen was measured after release and then at 10 minute time intervals until one hour, then 1 hour intervals were used until 4 hours. The final reading was taken after 24 hours.



(a) (b)

Fig.2.11: Showing stages of memory test: (a) Foam specimen crushed to 50% (1.5") of its height, (b) Foam specimen just after release (i. One day crushing, ii. Seven days crushing).

2.4. Results

Different combinations of packing foam and adhesives were evaluated with the previous tests to investigate the ability to resist lateral pressure, elongation, elongation and resistance to lateral pressure, and some slight modifications of these tests in order to simulate the performance of the haunch at different phases during the construction of bridges.

The results for memory test, lateral pressure test with no adhesive, lateral pressure test, elongation and resistance to lateral pressure test, and elongation test are summarized in Table 2.3. In Table 2.3 the average of air pressure at which specimen was failed and standard deviation is presented for three replicate tests. The maximum pressure investigated in the lateral pressure and elongation and lateral pressure tests was 6.5 psi. If a specimen exceeded this capacity then the value was reported as > 6.5psi. If a standard deviation is reported as zero then this means that all specimens had the same result. The foam adhesive combination in the different tests was investigated with a cure time of either one or two days. This was done to evaluate how the strength of the adhesive changed with time.

Table 2.3: Summary of test results for memory test, lateral pressure test with no adhesive, lateral pressure test, elongation and lateral pressure test, and elongation test.

Foam	Memory Test*				Lateral Pressure Test-No Adhesive**		Adhesive	Cure Time	Lateral Pressure Test***					Elongation and Lateral Pressure Test***					Elongation Test	
	Day 1		Day 7																	
	AHG (%)	σ (%)	AHG (%)	σ (%)	AAP (psi)	σ (psi)		(Days)	AAP (psi)	σ (psi)	Deflection (in.) Top	Center	Bottom	AAP (psi)	σ (psi)	Deflection (in.) Top	Center	Bottom	AE (in)	σ (in)
1	75.0	0.1	61.1	0.0	2.5	0.0	A	1	5.5	0.0	0.102	0.340	0.089	5.6	1.2	0.129	0.335	0.134	0.908	0.130
							A	2	>6.5	0.0	0.139	0.439	0.139	>6.5	0.0	0.130	0.423	0.160	0.888	0.090
2	75.0	0.0	59.8	0.1	2.5	0.0	A	1	4.8	0.6	0.085	0.342	0.092	3.5	0.0	0.114	0.307	0.145	0.363	0.050
							A	2	>6.5	0.0	0.108	0.401	0.113	6.2	0.6	0.125	0.243	0.116	0.825	0.230
3	93.1	0.0	59.0	0.0	3.2	0.6	A	1	6.1	0.6	0.089	0.322	0.100	>6.5	0.0	0.097	0.295	0.089	0.364	0.020
							A	2	6.3	0.0	0.106	0.317	0.101	>6.5	0.0	0.095	0.336	0.108	0.696	0.280
4	87.5	0.0	70.8	0.0	4.5	0.0	A	1	>6.5	0.0	0.059	0.243	0.066	>6.5	0.0	0.057	0.177	0.060	0.754	0.220
							A	2	-	-	-	-	-	-	-	-	-	-	0.661	0.300
5	97.2	0.1	83.3	16.7	5.5	0.0	A	1	5.8	1.2	0.051	0.216	0.053	6.2	0.6	0.043	0.241	0.087	0.310	0.030
							A	2	>6.5	0.0	0.054	0.122	0.055	6.2	0.6	0.078	0.131	0.069	0.396	0.040
2	-	-	-	-	-	-	B	1	4.5	0.0	0.018	0.183	0.046	3.5	1.0	0.013	0.092	0.013	0.324	0.150
							B	2	4.5	0.0	0.019	0.181	0.050	-	-	-	-	-	0.327	0.050
2	-	-	-	-	-	-	C	1	3.8	1.2	0.038	0.182	0.063	-	-	-	-	-	0.318	0.180
							C	2	4.8	0.6	0.141	0.338	0.099	-	-	-	-	-	0.309	0.080

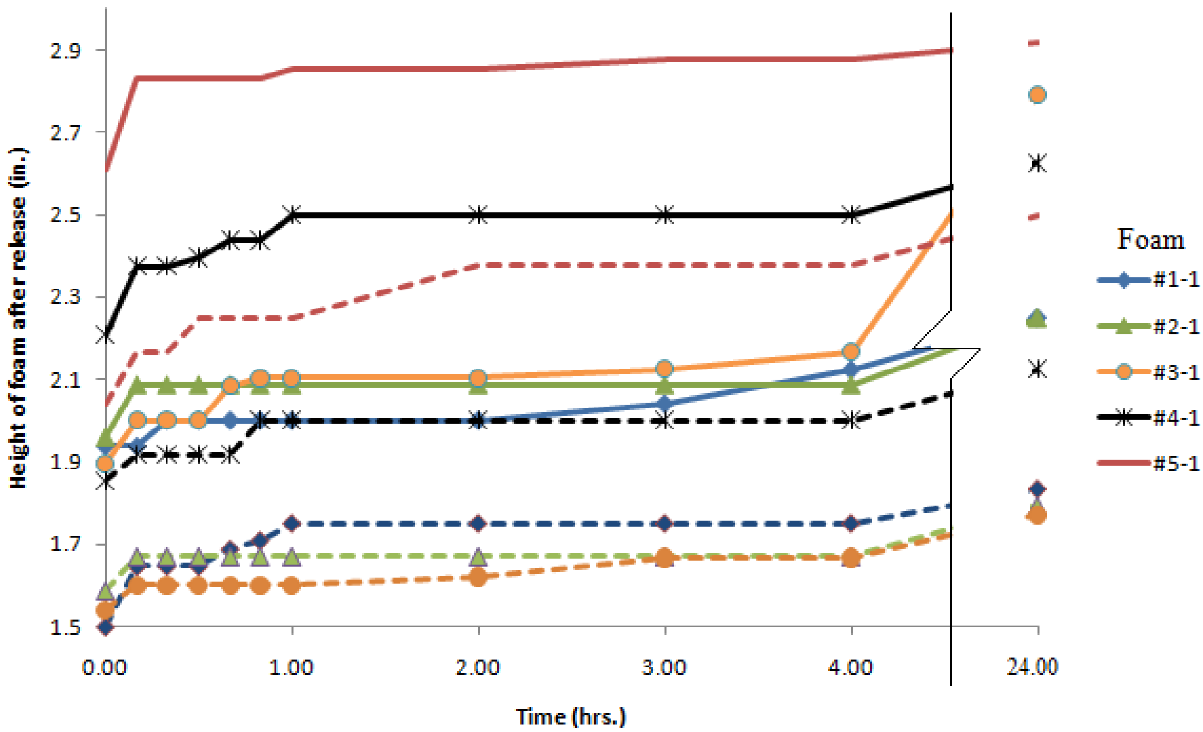
σ :Standard deviation.
AHG: Average height gain.
AAP: Average air pressure.
AE: Average elongation.

**Results of memory test presented represent the average percent of height gain when foam Specimen crushed to 50% of its height after 24 hrs.*
***Results of Lateral Pressure test with no glue represent the air pressure at failure when crushed by 0.75".*
**** The maximum pressure investigated in lateral pressure test and elongation and lateral pressure test for any specimen was 6.50 psi if all specimens the exceeded this capacity then the result was reported as “>6.5”.*

Not all combinations of foam and adhesive were investigated for this testing. From preliminary testing adhesive A appeared the most practical due to constructability and economy. Adhesive A is easy to apply as compared to other adhesives i.e. can be applied by paint brush. Because this adhesive appeared to be the most practical it was used to evaluate the performance of the foams as a haunch form.

In order to make a comparison between adhesives, foam 2 was investigated with all three adhesives A, B, and C to investigate the impact on the physical properties of the specimen.

In Fig.2.12 the memory or average percentage height gain of different foam specimens after release after being compressed for either 1 or 7 days is shown. The solid line represents the data of specimen after release at one day and dotted line data after compressing for seven days. And Fig.2.13 provides the capacity of the foam to resist lateral pressure when there is no adhesive at different levels of compression. The results presented in the Fig.2.13 are average of air pressure at which specimen was failed and standard deviation is presented for three tests. The maximum pressure investigated in this test was 6.50 psi.



Solid line: Data for one day testing
Dotted line: Data for seven day testing

Fig.2.12: Graph showing gain of height by foam in memory test.

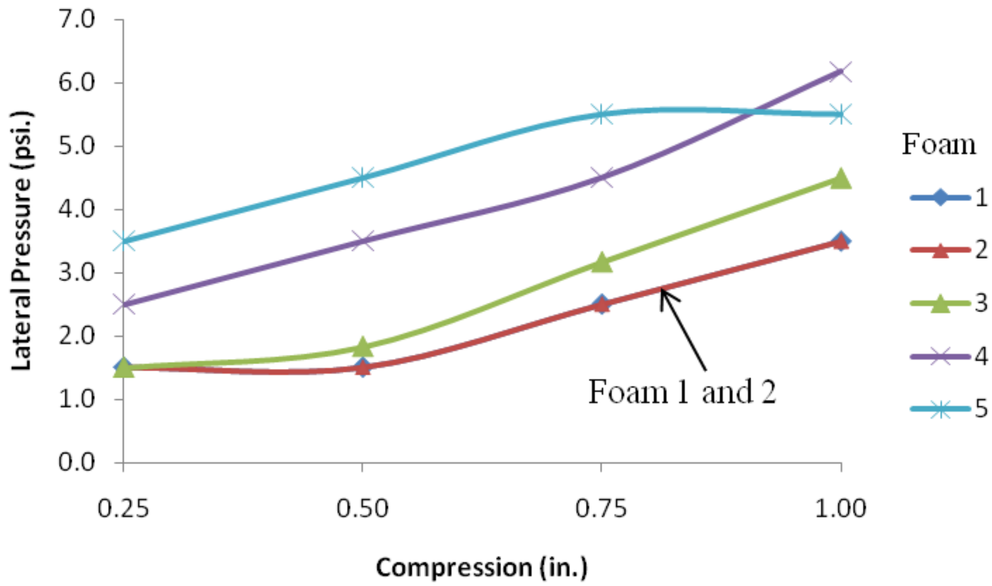


Fig.2.13: Graph showing results of lateral pressure test with no adhesive.

In the aforementioned chart data for foam 1 and 2 are almost same that's why the lines representing data got overlapped for foam 1 and 2.

Results for lateral pressure test on specimen cured at 45° F and lateral pressure test carried out at the joint of foam are tabulated in Table 2.4.

Table 2.4: Results for modified lateral pressure test.

Lateral Pressure Test	Foam	Adhesive	Cure Time (Days)	AAP (psi)	σ (psi)	Deflection (in.)		
						top	center	bottom
Cured at 45⁰ F	2	A	1	4.5	0	0.0928	0.2893	0.1160
Tested at joint	1	A	1	5.17	0.58	0.0908	0.3198	0.0797
Normal	1	A	1	5.5	0.0	0.102	0.340	0.089
	2	A	1	4.8	0.6	0.085	0.342	0.092

σ :Standard deviation.

AAP: Average air pressure.

2.5. Discussion

The performance for the foam and adhesive combinations will be discussed in terms of the results from each test.

2.5.1 Lateral Pressure Test

The lateral pressure test investigates the ability of a foam and adhesive combination to resist the lateral pressure from the fluid grout or concrete material used to make a connection between the precast members. The results for this test would be considered conservative as failure of the foam and adhesive combination always occurred at the ends of the foam members or where the glue was terminated as shown in Fig.2.6. The area where the glue was terminated likely saw not only shear stresses from the airbag but also bending stresses as the bag was not confined outside the length of the foam. While this test setup does not exactly mimic the loading condition that will be used for a haunch form it should provide a conservative estimate of the available strength.

From Table 2.3 it can be seen that after two days of curing foam 2 and adhesive A can resist 6.5 psi while foam 2 and adhesive B can resist 4.5 psi and foam 2 and adhesive C can resist 4.8 psi lateral pressure. Since it is desirable to provide as much lateral pressure resistance as possible to resist a form failure, adhesive A has the best performance of the three adhesives.

Comparing the results for samples cured for one day and two day we can see that adhesive A and C gain some strength on second day while adhesive B has same strength on second day. From this we can infer that it would be ideal to cure the haunch for two days before the grout or concrete is placed. However, the haunch could still be used at one day as it provides 4.8 psi of lateral pressure resistance. This is equivalent to 4.8' of head pressure from a concrete or 5.8' of head pressure from grout pour (assuming that the unit weight of the concrete as 144 lb/ft^3 and grout as 120 lb/ft^3). With conventional gravity feed methods of placement the pressures would not be expected to be exceeded.

2.5.1.1 Elongation and resistance to lateral pressure test

A modification to the lateral pressure test was made to study the behavior of haunch system after elongation of the foam by 0.25". An approximately 20% reduction in strength was measured for foam 2 and adhesive A and foam 2 and adhesive B when the results of this test are compared to the lateral pressure test. However, the strength of the rest of the foam and adhesive combinations were not significantly impacted. This result suggests that the foam and adhesive combinations should be allowed to be raised by 0.25" after the glue has cured. This finding allows the contractors more flexibility during the construction of the bridge deck to insure the proper geometry is obtained.

2.5.1.2 Lateral Pressure Test- No Adhesive

In this test foam specimens were tested with no adhesive applied to them. This testing was done to investigate the need for the adhesive between the foam and the concrete. From the results shown in Fig.2.13 it can be inferred that as stiffness of foam increases, the resistance to lateral pressure also increases. Test results show that foams 1, 2, and 3 has a low lateral pressure resistance in the absence of adhesive. However, foams 4 and 5 show lateral pressure resistance of 4.50 and 5.50 psi respectively at 0.75" compression.

These results suggest that with CL foams it may be possible to use only a foam without using an adhesive to resist the lateral pressures of the grout or concrete. This could be advantageous as this is one less step in the construction process. However it may be difficult to insure uniform loading of the foams by the panels in the field especially if their geometry is adjusted by grade bolts. Also, it is likely necessary to use some glue to insure the panels stay in the correct location. The use of adhesive is especially necessary if the precast panel is to be raised to adjust the system for the needed roadway profile and bridge deck thickness. However, these results do provide some assurance that some lateral pressure resistance would be expected if an inadequate amount of glue was used during construction.

2.5.1.3 Other comments from modified lateral pressure test

Besides the aforementioned tests, adhesive A showed no change in the measured strength when cured at 45° F for one day. Also a haunch system was tested with a joint added to the center of the test setup and there was no difference in the strength or failure mode of the system. This suggests that the performance of the glue is not affected at least at a temperature of 45^{o} F and 73^{o} F.

2.5.2 Elongation test

This test results provides limitations for raising panels while adjusting roadway profile. From Table 2.3 we can see that minimum elongation of 0.30" was found for any combination of foam and adhesive. Test results also show that as the stiffness of the foam increases the resistance of the system to elongation decreases. The combination of foam 1 and adhesive A shows very effective performance for this test and is able to elongate up to 0.90" while foam 5 was only able to resist 0.30" of elongation. The use of foam 1 and adhesive A would allow greater flexibility during construction.

2.5.3 Memory test

This test evaluated the capacity of a foam to regain height after release after it has been crushed to 50% of its original height. From Fig. 2.12 we can infer that the instant height gain of foam 1 and 2 is about 65% of its original height after one day of crushing and about 75% of its original height after 24 hrs. While for foam 3, 4 and 5 an instant height gain of 64%, 74% and 87% respectively of its original height after one day of crushing and height of 93%, 87% and 97% respectively of its original height after 24 hrs. However, after seven days of crushing there is no initial memory for foams 1, 2, and 3.

Whereas, foam 4 and 5 shows very small amount of memory after seven days of crushing. From this we can infer that as the stiffness of foam increases the capability of the foam to regain height also increases. This suggests that, polyethylene foam does not have a high memory; however, this foam did show promise in the elongation test and would still be deemed to be satisfactory.

These results suggest that a precast bridge deck system should be adjusted within 1 day of the panel installation if possible. If adjustments are made after 7 days of being compressed then the adhesive and foam combination would have less ability to elongate before failure then if the adjustments were made after 1 day.

2.6. Synthesis of results

The memory test data suggests that, if a precast bridge deck panel is raised beyond 0.4" using PE or 1.41" using crosslink foam as forming material after initially being crushed to 50% of its original height (i.e. 3") then we need to attach foam to precast panel using adhesive so that there is no gap between foam plank and precast panel.

As per lateral pressure test with no adhesive a crosslink foam (foam 4 and 5) or a foam of a similar stiffness may be able to be used as a haunch form without applying adhesive. But, one should be aware that there could be problems with the use of these foams as it might be difficult to crush them with the self weight of the precast panel or insure uniform compression due to field differential height of the panels. This would require that the foam be cut to a height that is close to the final haunch height.

One important foam parameter is the ability of the material to be compressed by the self weight of the bridge deck system. While this parameter is not discussed directly the information can be inferred from the compressive stiffness information contained in Table 2.1. If a foam is very stiff then the self weight of the precast deck panel may not be able to cause the foam to deflect downwards. Of the foams investigated foam 1 has the lowest compressive stiffness and so it would provide the most flexibility during construction. While buckling of the foam may also become an issue, it was never seen with the 1:2 aspect ratio used for this testing.

In all of the testing the combination of foam 1 and adhesive A showed good performance including the highest lateral pressure and elongation capacity. Hence it is recommended to use this combination for construction of adjustable haunch system.

Another parameter that is not considered in the data presented but also is important is the aesthetics of the foam as may be used on the exterior of the bridge and so in a visible location. The foam manufacturer creates foam in a distinctive color so that it is clear to the customer the properties of the foam. The typical color for foam 1 is a gray which is similar to concrete and so would not cause an aesthetic problem for the bridge.

2.7. Suggested Construction Sequence

The following is a summary of the suggested construction method of an adjustable haunch forming system:

- The surface of the precast beam where the foam is to be placed should be thoroughly covered in adhesive.

- The foam should be cut to height that is approximately 1” higher than the estimated haunch.
- Next the foam should be placed on the adhesive and held in place until it sticks.
- Just before the precast panel is placed the top of the foam should be thoroughly covered in adhesive.
- The grade bolts in the precast panels should be adjusted to provide a haunch depth that closely matches that required for the bridge deck before the panel is placed.
- The panel should be placed and then adhesive should be allowed to cure for a day before adjusting.
- After the glue has cured the height of the panel can still be lowered using grade bolts but should not be raised more than 0.25”.
- The haunch is now ready for grout or concrete placement

2.8. Conclusions

Several combinations of packing foam and adhesive were investigated to be used as a haunch forming system for precast bridge decks. These systems provide a large number of advantages over other conventional forming systems as they allow a precast deck panel to be adjusted during the construction process and do not require work to be performed under the bridge for installation or removal. This increases the safety, constructability and economy of these systems.

Packing foam 1 and adhesive A combination has been identified to be an acceptable candidate to be used as a haunch form. This system showed the ability to resist a lateral pressure over 6.5 psi or approximately 6.5' of concrete and an elongation of 0.9" before failing. It also showed satisfactory performance when adhesive was cured at 45^{o}F and also when tested at joint of foam. Besides this the foam manufacturer creates this foam in a gray color that is similar to concrete which doesn't create any aesthetic problems for the final bridge.

While this testing was completed with specific sizes of haunch forms other heights and configurations could be estimated based on the data presented in this report. Furthermore, while the focus of this paper has been on providing this forming system for haunches of bridge girders the same concepts could be extended to be investigated for use in several other applications involving precast elements.

CHAPTER III

3.0 FULL SCALE TESTING OF BRIDGE DECK OVERHANGS

3.1. Introduction

There is need to develop new innovative bridge construction techniques which will minimize the impact of traffic and the impact on the surrounding environment. Prefabricated bridge elements have shown the potential to meet these needs.

The use of prefabricated bridge girders and bent caps has become an acceptable method of precast construction. But, another element which has attracted the attention of researchers is the development of prefabricated bridge deck system. The development of such a system would be very advantageous as the bridge deck often is the item that needs first maintenance and ultimately replacement. A system that allowed for rapid construction or replacement would be very helpful.

Several attempts in the past have been made to develop precast bridge deck system. However these systems have not been widely adopted as they have challenges providing a smooth riding surface without grinding while also being constructible. However, partial depth bridge deck systems have been used since the 1970s in Texas, Missouri, and Illinois. These systems have been shown to improve safety, economy and speed over conventionally formed bridge deck construction. While these systems have been widely adopted in several states they can still be improved as the overhang construction is still largely constructed with wood forms and cast in place concrete. Other challenges include obtaining the correct elevation of the finished riding surface. These bridge decks are somewhat challenging to provide adequate cover on the reinforcing and an adequate riding surface.

In 1985, attempts were made to eliminate the problems in the partial depth bridge system by introducing the full depth bridge deck system (Freeby and Ley, 2002; Badie et al., 2006). Typically these bridge deck systems consist of thick concrete planks that runs the entire width of the bridge deck that are placed on the beams below. This system has shown potential over partial depth bridge deck system by removing the need for the conventional forming used in the overhang construction. But, still the problem of obtaining the smooth finished riding surface was not corrected. 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. Also these full depth bridge deck panels are heavy and are not easy to transport or place. Furthermore, considerable construction labor is needed in erecting the construction forms for the area between the precast panel and the prestressed girders. This area is often referred to as the haunch. This dimension often varies with the construction geometry due to camber in the girders and the change in the bridge deck profile.

After reviewing the pros and cons of the full depth and partial depth bridge decks it was realized that the systems could be combined in a hybrid system. This hybrid system is composed of a precast overhang member with a full and partial depth section. The precast panels for proposed bridge deck system are one half of the final bridge deck thickness to be used in the interior spans and an innovative precast panel that has a full depth and half depth section to be used in the overhangs and the first interior span respectively. These panels serve as structural stay in place form-work, working surface, and support for the screed rail. A 4.25” topping of cast-in-place reinforcement concrete is placed to tie the structural systems together and provide the final riding surface for the bridge deck.

This proposed system has not been investigated previously for strength. Also, the behavior of the proposed system is unclear due to the use of prestressing strands in the compression zone. The current chapter is dedicated to the static load testing of the proposed precast bridge deck panels by mimicking the AASHTO HL 93 truck and studying their behavior.

3.2. Design and Fabrication of Prototype Bridge System

3.2.1 Prototype bridge system details

A full-scale two-lane, full-depth overhang and partial depth interior span bridge system was constructed. These systems were designed to reflect typical reinforcement ratios that are used by US DOTs. The precast concrete panels were designed for transverse flexure with conventional mild steel reinforcement and standard prestressing strands in accordance with the current TxDOT deck design provisions for slab design with the main reinforcement perpendicular to traffic flow. A layout of the test specimen is shown in Fig.3.1. The prototype bridge was 8’ in the longitudinal direction and 18’ in the transverse direction. The bridge deck was constructed on 3 girders that had 6’ center to center spacing with typical 3’ overhangs. The bridge decks investigated were 8.25” thick with 2.25” of cover from the bridge deck surface to the top reinforcing bar. One exterior span and cantilever was built with a precast overhang panel system and the other side of the deck system was built using a 4” precast panel and a conventionally formed 8.25” overhang. By constructing the specimens in this manner it allowed the capacity of the two overhang systems to be compared using a single specimen.

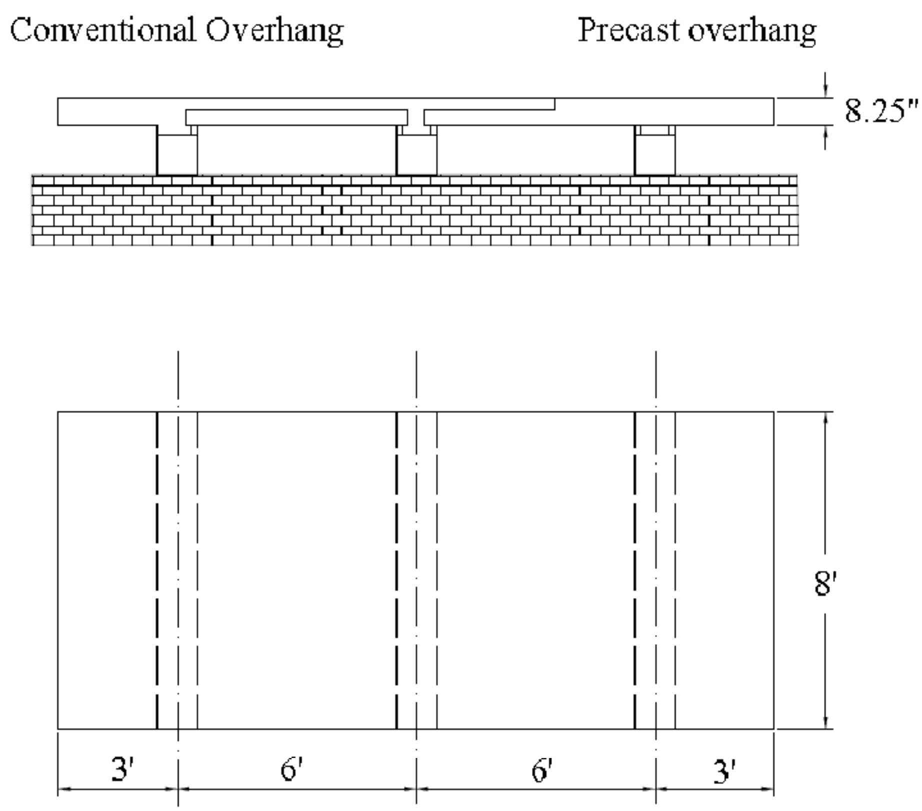


Fig.3.1: Arrangement of precast concrete bridge deck system components.

Precast concrete panels were installed on the concrete girders and made fully composite with shear connector pockets and high strength threaded rods with nuts (refer fig.3.2). The girders used in this testing had a top flange width of 12” and were 14” in height and rested directly on the ground. While this specimen configuration does not directly reflect the performance in practice it does provide a conservative estimate of the performance of the bridge deck system.

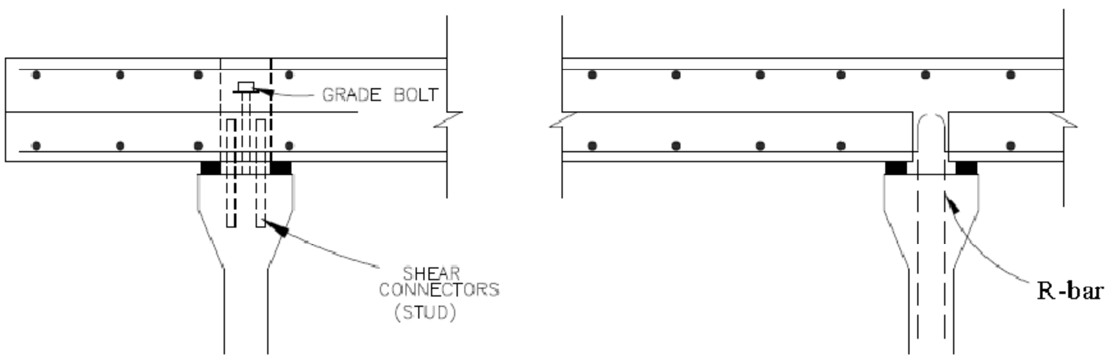


Fig.3.2: Details of shear connector pockets.

3.2.2 Fabrication of specimen and reinforcement details

Specimens of bridge deck where composed of three basic elements i. Conventional bridge deck, ii. Partial depth precast panels, iii. Precast overhang panel. These three basic elements are shown in Fig3.1. Fabrication and reinforcement details are discussed in detail in following sections.

3.2.2.1 Conventional bridge deck

For conventional bridge deck, a cast-in-place (CIP) full depth overhang and partial depth concrete topping with reinforcement and partial depth precast panel which act as a stay-in-place form are used. Conventional overhang was 3’ from the center of the girder and 8.25” in depth. The concrete strengths for specimen 1 and 2 are reported in Table 3.1 and Grade 60 steel was used for longitudinal and transverse reinforcement. Transverse deck reinforcement consists of straight #5 bars at 6” spacing in the top layer and # 4 bars at 9” spacing for specimen 1 and 18” spacing in the bottom layer. Longitudinal temperature and shrinkage steel were also provided using #4 bars at 9” spacing in the top layer and #5 bars at the bottom layer. The bottom longitudinal reinforcement with # 5 bars was provided in three rows; first row was at 2” from the edge of the overhang, second was at 3” spacing from the first row and third was at 1’ spacing from the second row for conventional overhang. For partial depth concrete topping transverse reinforcement consists of straight #5 bars at 6” spacing and longitudinal temperature and distribution steel was provided using #4 bars at 9” spacing. The clear cover over the top layer was 2.5” while the bottom layer had a clear cover of 2” (refer fig.3.3).

The aforementioned reinforcement was tied after installing the precast overhang on the other end and intermediate panels on the interior bays. The specimens were cured with wet burlap for 7 days.

The deck slab was designed as a beam in flexure supported by the girders using the AASHTO Service Load design provisions using HL 93 loads for conservatism. TxDOT's bridge deck design requirements limit the calculated stress in the reinforcing steel (f_s) to 24,000 psi and the concrete stress (f_c) to 1,600 psi using a modular ratio (n) of 8 (Merrill 2002).

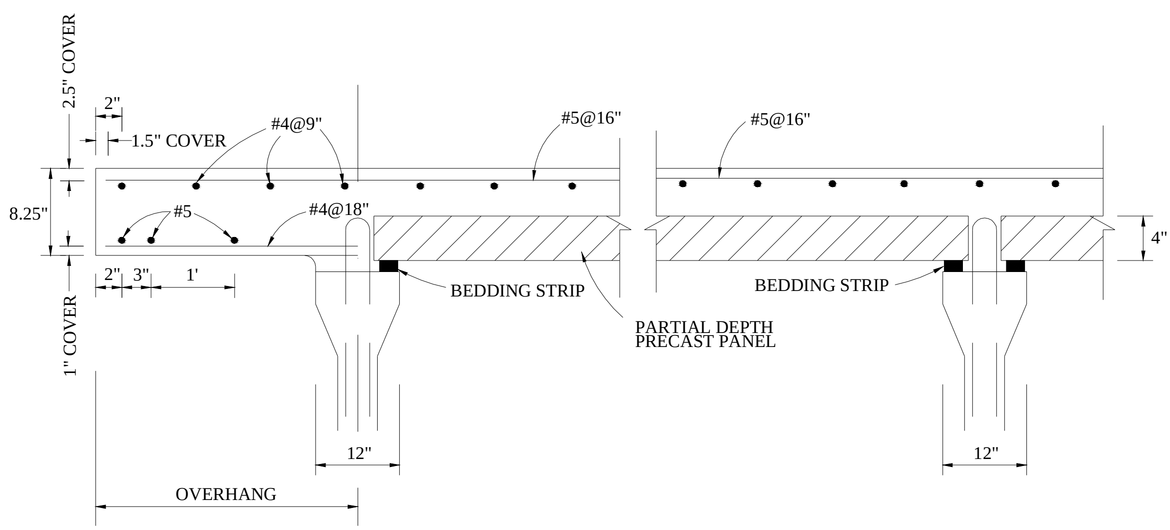


Fig. 3.3: Reinforcement details for cast in place (CIP) bridge deck overhang.

3.2.2.2 Partial depth precast panels

Partial depth precast panels are used for both conventional bridge deck construction method as well as construction of precast overhang system for interior spans. For testing on this project standard partial depth precast panel developed by TxDOT were used for intermediate span of bridge specimen. Partial depth precast panels were of 8' x 5'-8" x

4”. The strength of concrete for specimen 1 and 2 are mentioned in Table 3.1, Grade 60 reinforcing bars were used for longitudinal reinforcement and Grade 270 strands were used for transverse reinforcement. Transverse deck reinforcement consists of the 3/8” strands at 6” spacing located at mid-depth in the 4” thick panel and were prestressed to 16.1 kips. Longitudinal temperature and distribution steel were also provided using #3 bars at 6” c/c. These panels have a 3” strand extension that goes into the connection between the girders and the bridge deck (refer fig.3.4).

These partial depth panels are generally casted in 8’ wide casting beds ranging from 350’ to 500’ in length using self-stressing forms. The required concrete strength is 5000 psi, but most fabricators use a high-range water reducer along with Type III cement so that the concrete reaches 4000 psi in about 14 hours for strand release. This allows panels to be cast in a given bed every other day. The panels were given a broom finish to aid in the development of bond between the panel and the cast in place concrete topping.

Partial depth precast panel designs are highly standardized, and they are intended to follow the AASHTO Bridge Design Specifications. Service Load design is used but ultimate strength is checked at mid-span. The panels alone support the dead load. The panels are generally not wide enough to develop larger strands, so the design is based on the amount that can be developed rather than full development (Merrill 2002).

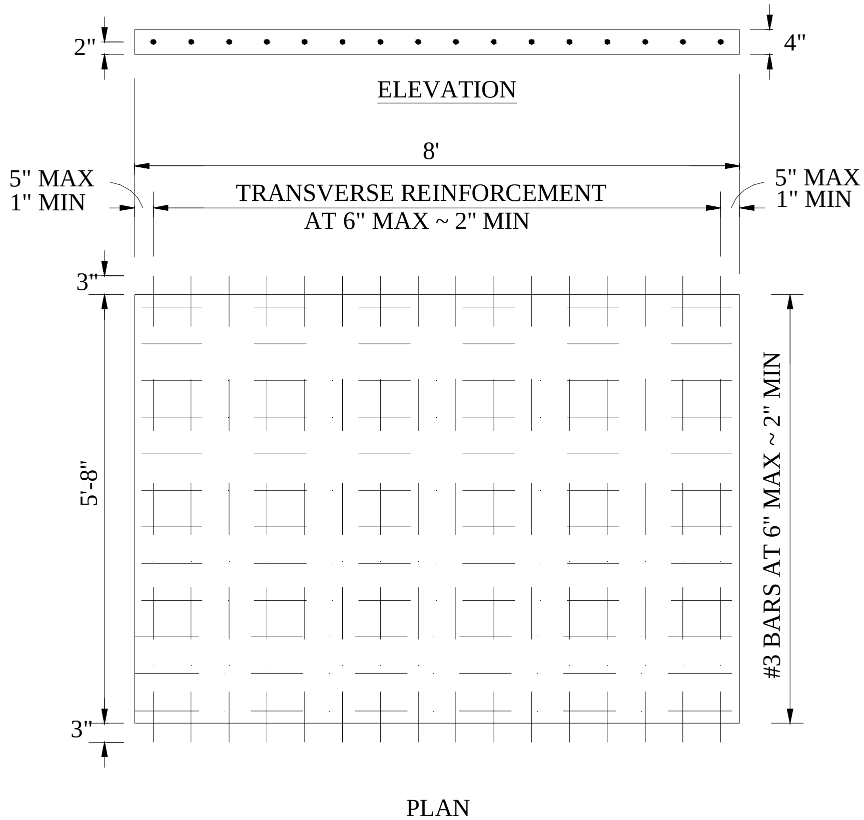


Fig.3.4: Reinforcement details for partial depth precast panel (stay in place form).

3.2.2.3 Precast overhang panel

The precast overhang panels are used in place of conventional overhangs. These are hybrid panels composed of precast prestressed panel in the bottom layer with a 4.25” reinforced concrete topping.

Full depth precast overhang panels were 8’ x 8’-8” and of varying depth. It was having depth of 8.25” until 5’-6” in the longitudinal direction and then 4” depth for remaining length. And also depth of panel was 4” at offset of 8” on both sides of panel starting from 1’ in the longitudinal direction till 5’-6” in the same direction. This offset was intentionally kept to make connection of two panels in transverse direction monolithic using C_2 bars (shown in Fig.3.5). Each precast overhang panel consists of three shear connector pockets of 10” x 7” and three grade bolts to adjust roadway profile. Shear connectors are used to generate monolithic connection between precast overhang panel and bridge girders. Layout of shear connector pockets and grader bolts is shown in Fig.3.2. The strength of concrete for specimen 1 and 2 are given in Table 3.1, Grade 60 reinforcing bars were used for longitudinal reinforcement in bottom layer and for longitudinal as well as transverse reinforcement for top layer and Grade 270 strands were used for transverse reinforcement in bottom layer.

Construction of the precast overhang panels were done in two stages. Stage 1 was construction of precast prestressed bottom layer panel of 8’-4” x 8’ x 4” and stage 2 was construction of precast reinforced concrete layer on the top of precast prestressed bottom layer panel upto 5’-6” in the longitudinal direction with 4.25” depth. Concreting of stage 2 was done with 4” margin in longitudinal direction so as to cover the 3” extension of strands on the overhang side. Construction of stage 1 was similar to that of construction of partial depth precast panel except for addition of U bars for traffic rail reinforcement and Z bars (see Fig.3.6). Construction of stage 2 was done one day after casting stage 1. Top reinforcement was tied as mentioned above and concrete was poured to construct full depth overhang panel. Care was taken to keep three shear connector pockets empty during the construction of both the stages and also to place grader bolts (coil) in specified position as shown in Fig.3.5. For reinforcement detail of stage 1 and stage 2 refer fig.3.6 and 3.7 respectively.

Transverse deck reinforcement consists of straight #5 bars at 6” c/c in the top layer and the 3/8” strands at 6” c/c located at mid-depth in the 4” thick panel and is prestressed to 16.1 kips in the bottom layer. Longitudinal temperature and distribution steel were also provided using #4 bars at 9” c/c in the top layer and #3 bars at 6” c/c in the bottom layer. The clear cover over the top layer was 2.5” while the bottom layer had a clear cover of 2” (refer fig.3.8).

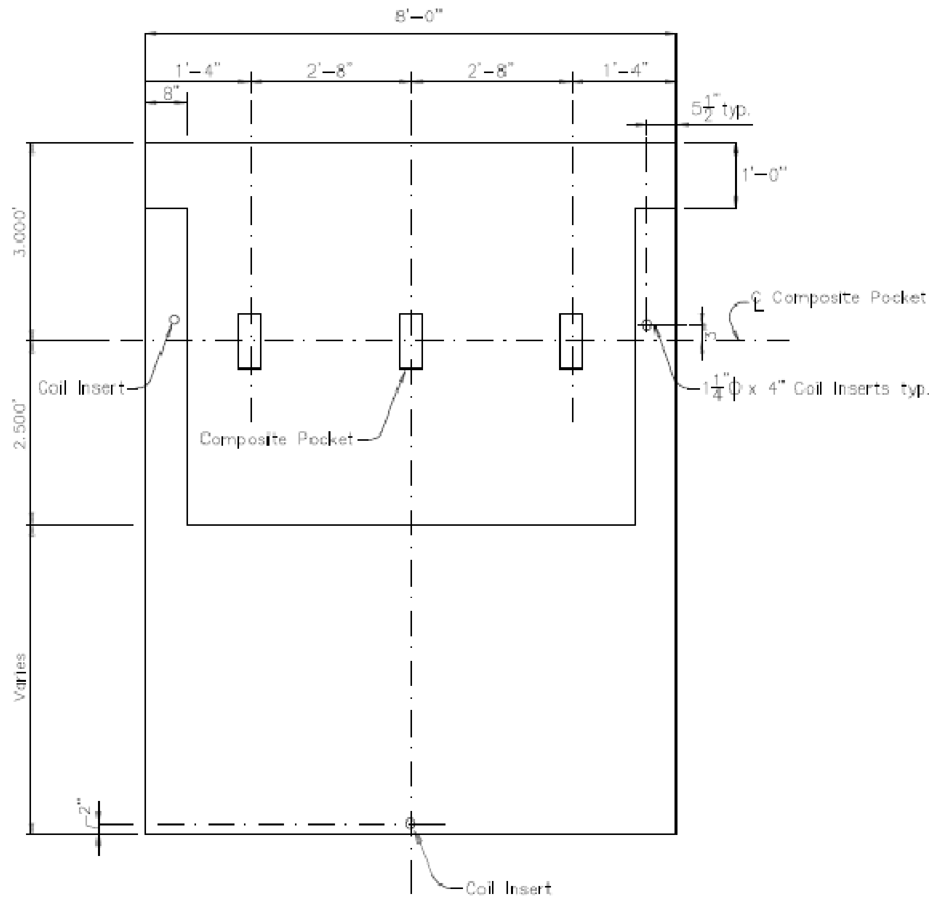
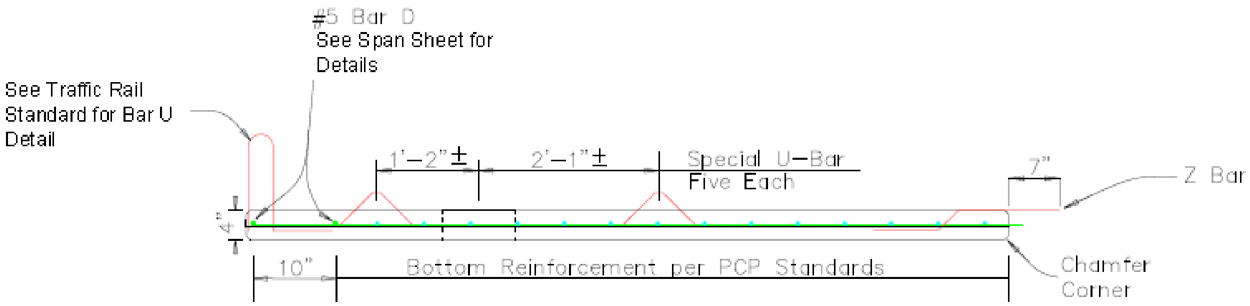
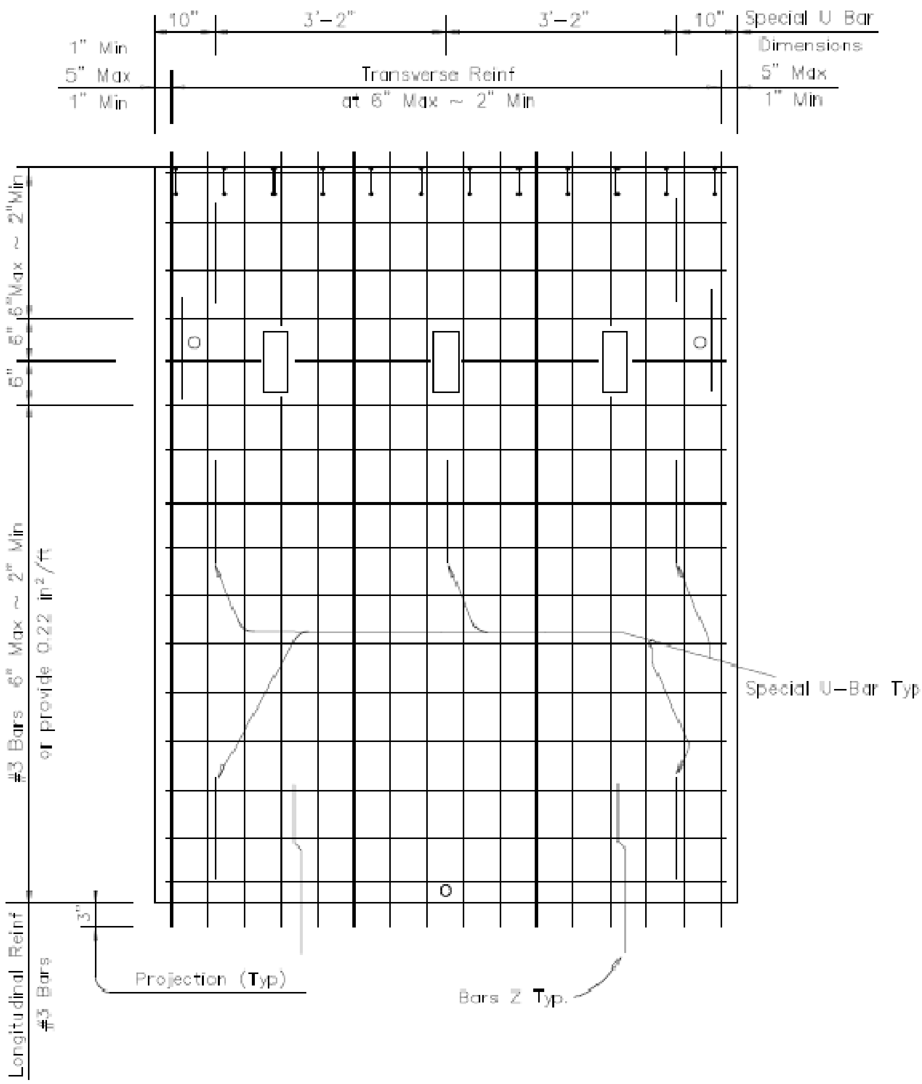


Fig.3.5: Dimensions of the precast bridge deck overhang panel.

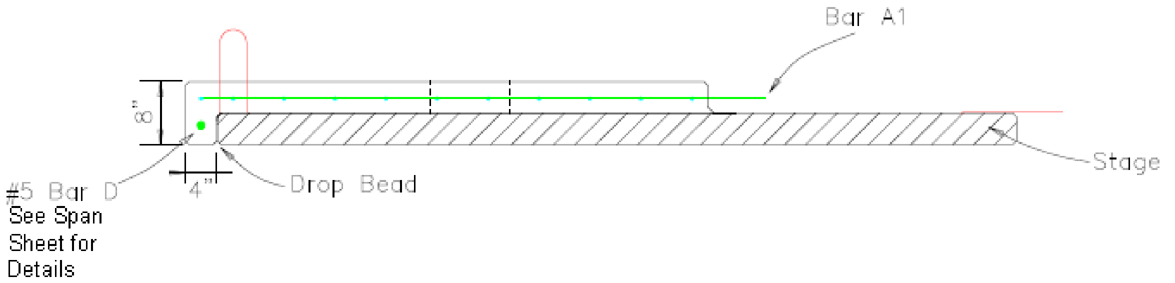


Elevation

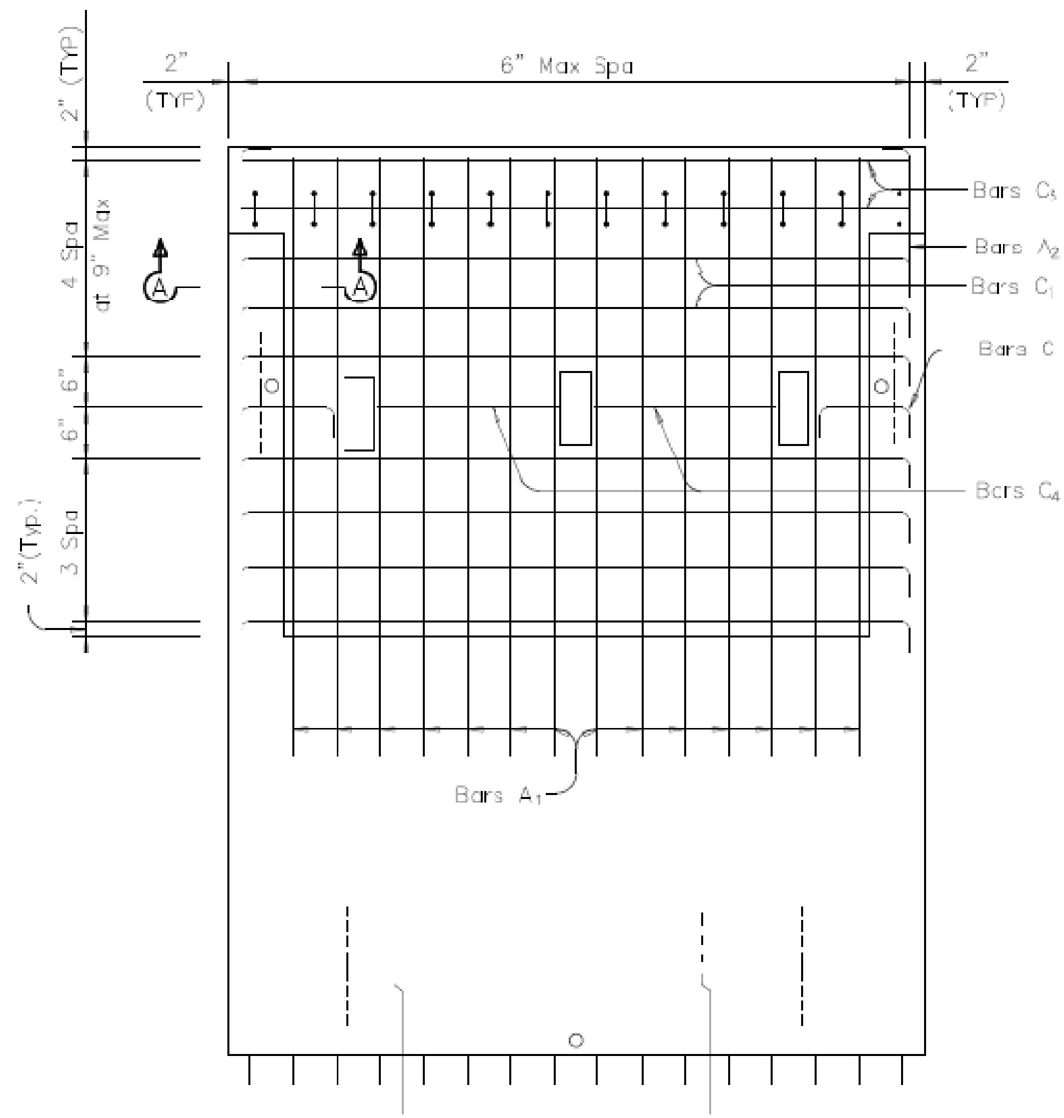


Plan

Fig.3.6: Reinforcement detail for stage 1 of the precast bridge deck overhang panel.



Elevation



Plan

Fig.3.7: Reinforcement detail for stage 2 of the precast bridge deck overhang panel.

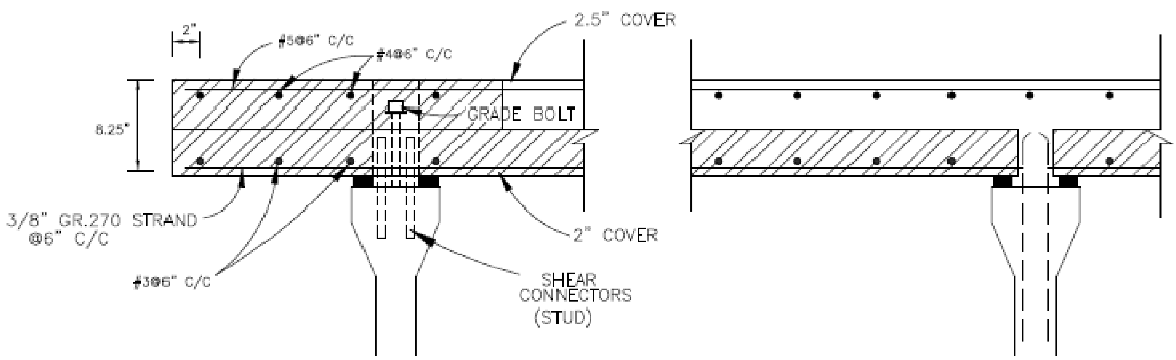


Fig.3.8: Reinforcement detail for precast bridge deck overhang.

To ensure the composite action between the precast overhang panels and bridge girder, the haunch was filled using SikaGrout™ 212. SikaGrout 212 is a non-shrink, cementitious grout with a unique 2-stage shrinkage compensating mechanism (SikaGrout 212, 2003). Mixing of grout was done with 0.17 water grout ratio and was used to fill the haunch. Filling of haunch was done by pouring the grout mixture through shear connector pockets.

A summary of the concrete and grout mixtures is provided in Table 3.1 along with the relevant material properties. All mixtures were representative of bridge deck concrete. The grout used in the haunch did not contain coarse aggregate. The location where each mixture was used in the specimen is shown in Fig.3.9.

Table 3.1: Summary of the average material properties of the mixtures used in specimen 1 and 2.

Specimen	Test	CIP	Stage I	Stage II	Grout	Pocket Concrete	Partial Depth Panel
1	Compression, psi (MPa)	6976 (48)	9098 (63)	7096 (49)	8137 (56)	4085 (28)	8475 (58)
	Tension, psi (MPa)	660 (5)	729 (5)	620 (4)	544 (4)	524 (4)	693 (5)
2	Compression, psi (MPa)	5371 (37)	9151 (63)	6857 (47)	6287 (43)	4881 (34)	8475 (58)
	Tension, psi (MPa)	514 (4)	774 (5)	550 (4)	600 (4)	458 (3)	693 (5)

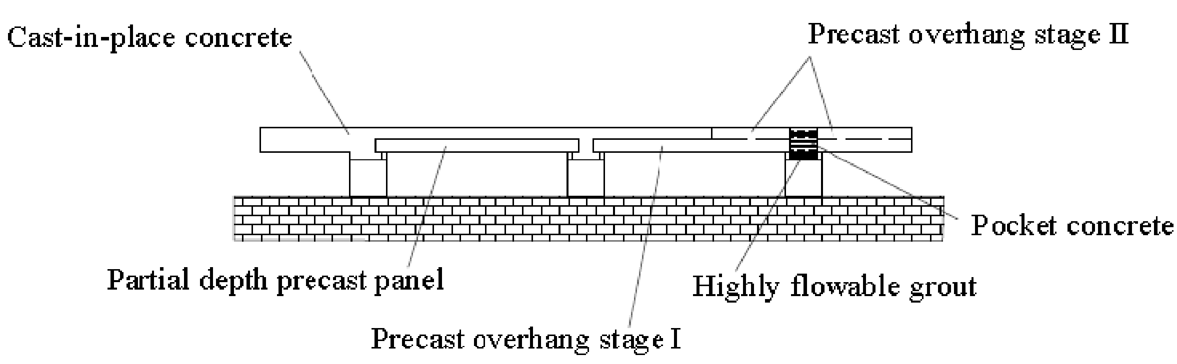


Fig.3.9: Locations of materials used in Specimens 1 and 2.

The reinforcing used in Specimens 1 and 2 were as per TxDOT 440 and ASTM A 615 grade 60 requirements.

3.2.3 Placement of panels

Installation of the adjustable haunch form was done for the precast overhang panels prior to the placement of panels i.e. foam was glued down on the bridge girder at least a day before placement of panels. Application of top layer adhesive on foam was done just prior to the placement of panels. Panels were placed on desire locations with the help of a crane and the height of grade bolts were roughly adjusted equal to the height of haunch. Fine adjustment to the height of haunch was done using grade bolts the next day after the placement of panels so that adhesive achieves the target strength. The precast panels were adjusted in such a manner that it provides a minimum haunch of about 2 in. and achieves straight roadway profile. The locations of the grade bolts on each panel were deliberately chosen so as to maintain the stability of panels before filling the pockets. Once desired roadway profile was achieved then Z bars of the full depth overhang panel were bent and welded to R bars of bridge girder. This was done for safety purpose and also to make sure that panel does not rotate or move from surface. All adjustments to the panel height should be made before the Z bar is welded to the R-bars. Bottom of conventional overhang and sides of full specimen were formed using plywood. Concrete topping of 4.25” on partial depth portion of bridge deck and 8.25” slab on conventional side was poured once reinforcement was tied as per TxDOT specifications. The whole bridge deck was then cured with wet burlap for 7 days. Grouting of the pockets was done on the next day using SikaGrout 212. The process of applying the grout consisted of wetting the concrete surfaces, mixing, and placing the grout in the pockets. Grout was first poured in the center pocket until it flows toward the adjacent pockets; grout was poured in the next pocket once the center pocket was half filled. All pockets were filled and vibrated until the grout filled the haunch. The next day the remainder of the pocket was filled with concrete.

3.3 Test Setup and Instrumentation

The test setup was designed to incorporate the conditions prevalent in the real structure as well as in the type of the imposed loading. A reaction frame and test specimens were mounted on the strong floor using post tension bolts. The center girder and bridge deck were fixed with the help of post tension bolts to restrict rotation of the center girder during testing. The cross section of the strong floor, loading frame and the bridge deck slab are shown in Fig.3.11. The loading points for Specimens are shown in Fig.3.10. For each test a 10” x 20” steel plate was used to represent a 16 kip AASHTO HL 93 tire patch. The center of the tire patch was placed 11” from the edge of the exterior beam. Two different load cases were investigated. In specimen 1 a load at the midspan of the cantilever was applied and in Specimen 2 the load was placed at the corner. This loading condition was chosen to simulate an HL 93 truck traveling at the very edge of the guard rail away from the edge of the panels and at the location where a bridge deck terminates such as at the approach slab.

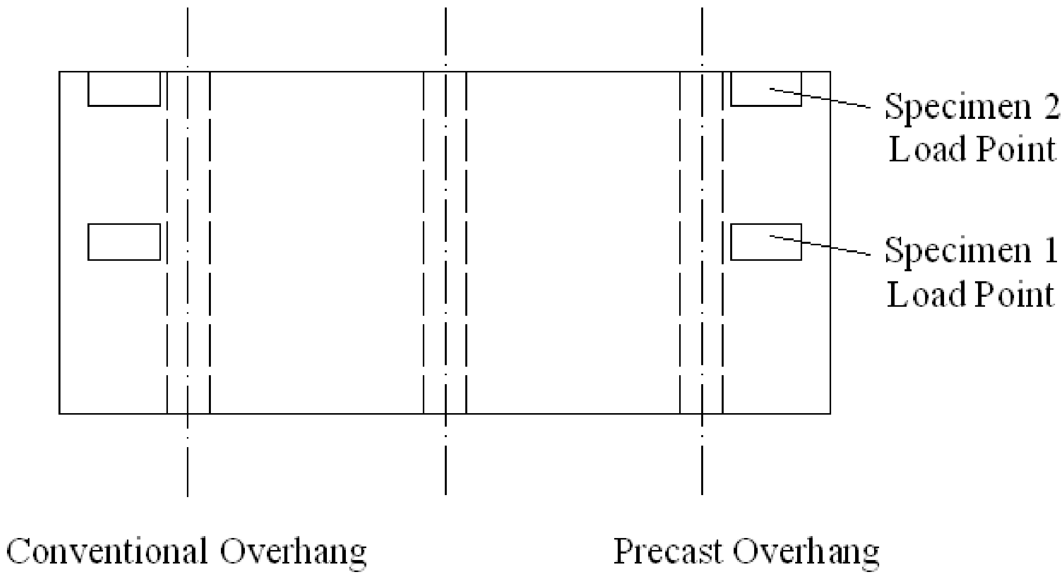


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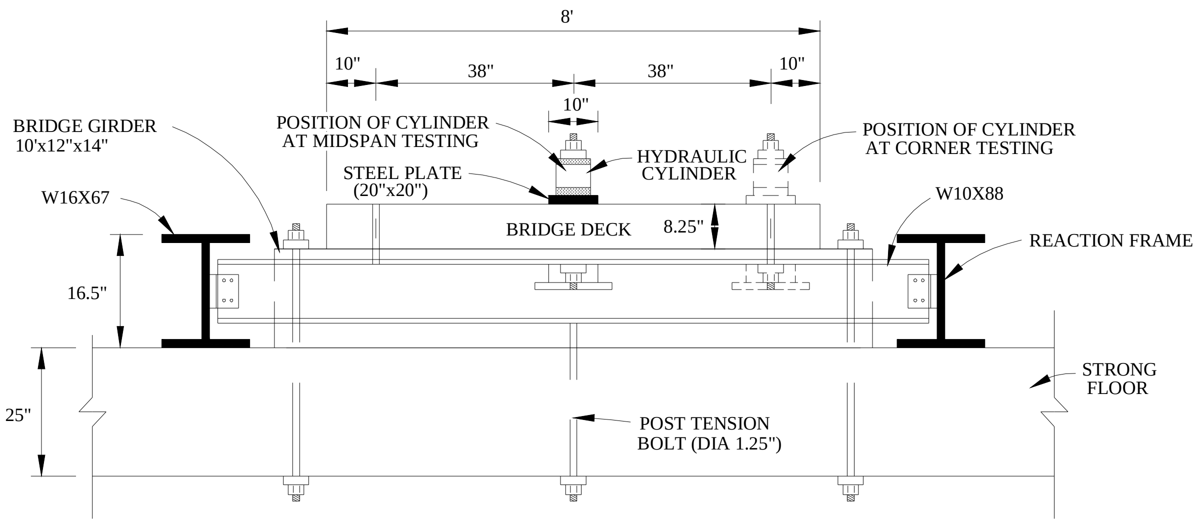
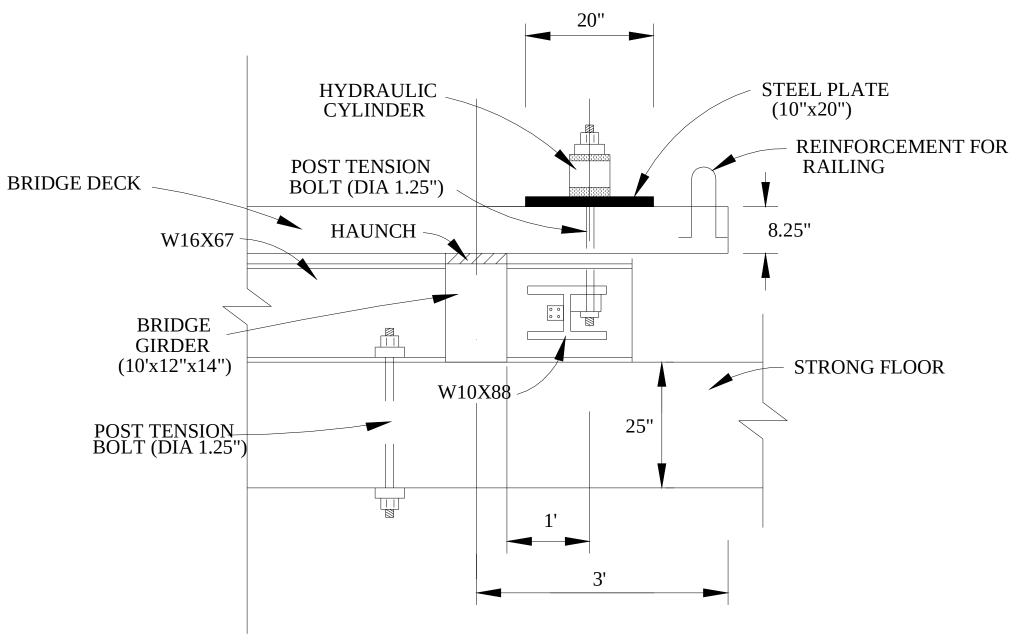
Fig.3.10: Load points investigated for specimen 1 and 2.



(a)



(b)

Fig.3.11: (a) Cross sectional front view of the test setup for 3’ bridge deck overhang testing, (b) cross sectional side view of the test setup for 3’ bridge deck overhang testing.

A hydraulic ram was used to load the bridge. The loading was transferred to the bridge deck from the hydraulic cylinder using steel plate of 10” x 20”. Hydrocal was applied on the surface of the bridge deck at the location of testing before placing the steel plate to make a level surface and to insure an even distribution of pressure on the contact areas of the slab surface.

The structural response of the specimens was evaluated with surface demec strain readings with 4.014 microstrain accuracy and by deflection measurements using linear variable displacement transducers (LVDTs) with 0.0005” accuracy. These systems provided flexible and accurate methods to investigate the performance of the overhang systems. 51 demec points were used to measure surface strain and 6 LVDTs were used to measure deflection while loading at midspan of the overhang and 47 demec points and 9 LVDTs were used to capture response of bridge deck while loading at corners. The response of bridge deck was monitored and recorded continuously using aforementioned tools at interval of 8 K.

Demec readings were taken at each load increment and contour plots were created. For this purpose the demec point stations were arranged in the form of a rectangular grid of approximately 7.75” x 7.75”. It was intended to have different mesh layout for overhang testing at center and at a corner due to the location of testing. But, slight differences in the demac point grid for CIP overhang and precast overhang tested was due to the inadvertent change in the orientation of the tire patch during testing. The Fig.3.12, 3.13, 3.14 shows the orientation of the tire patch and layout of demec point stations along with the details of mesh such as distance of first line of the demacs from the edge of overhang and size of the grid.

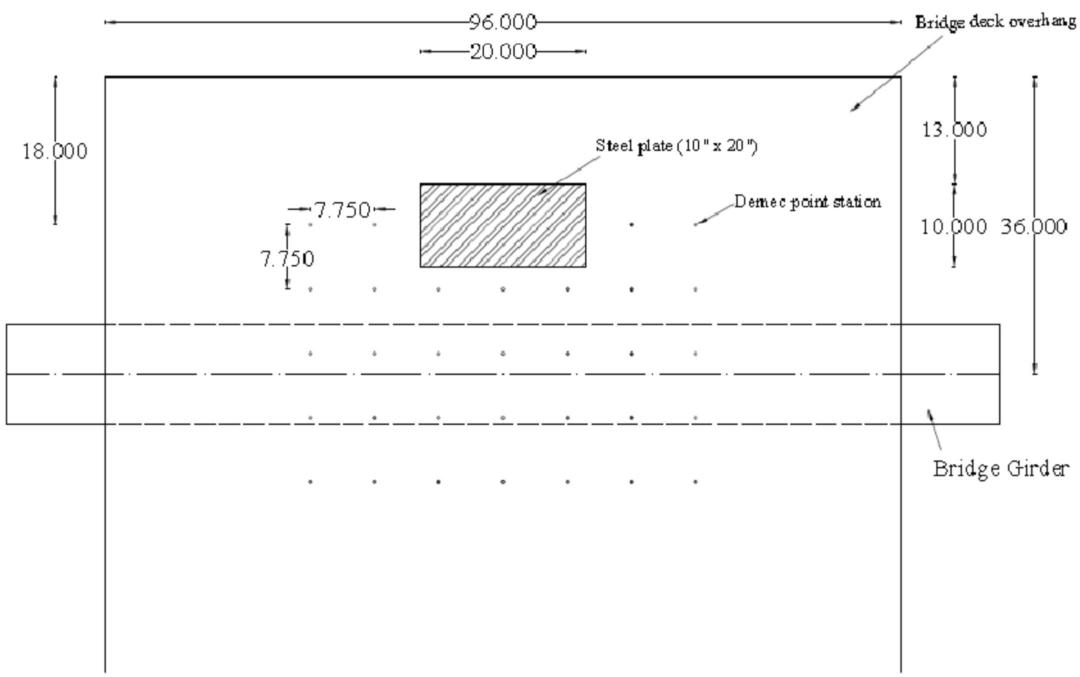


Fig.3.12: Orientation of tire patch and layout of demec point stations for CIP overhang tested at center.

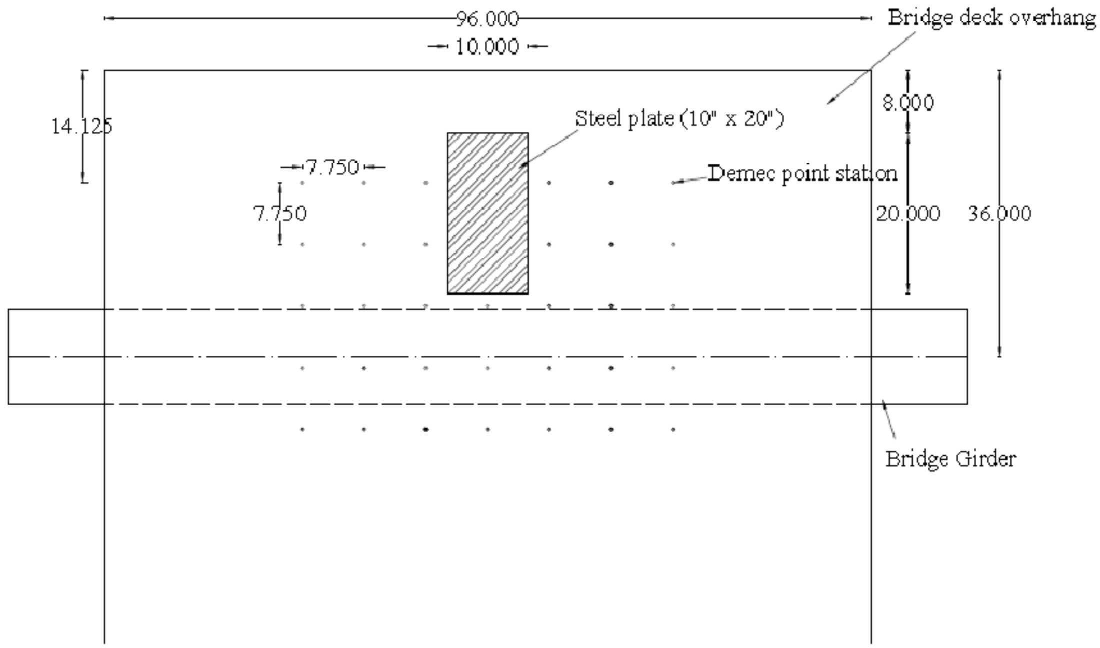


Fig.3.13: Orientation of tire patch and layout of demec point stations for prestressed precast overhang tested at center.

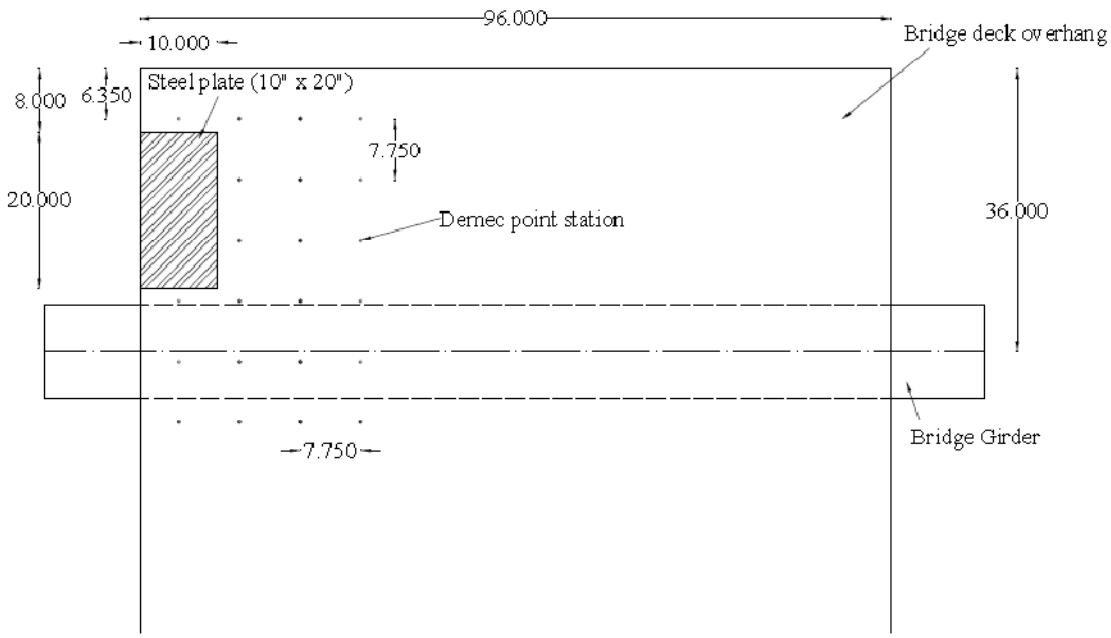


Fig.3.14: Orientation of tire patch and layout of demec point stations for CIP and prestressed precast overhang tested at corner.

3.4 Results

The full scale prototype bridge deck overhang system was tested with the same surface area of the AASHTO HL 93 truck at center of overhang and at corners (fig.3.10). In the entire test cases, the load was applied in regular increments and stopped every 8 or 16 kips to record surface strain, deflection and for inspection of the bridge deck overhang. At each loading interval the following were measured: load, surface strain, deflection, and crack pattern. The surface strains recorded during testing are presented in the form of contours which are plotted considering lateral strain. There was couple of reasons for considering only lateral strain for comparison of both system and studying the behavior of bridge deck under static loading as per AASHTO. Firstly, results inferred from the contour drawn on the basis of longitudinal strain and lateral strains were same. Secondly, Von Mesis method which is used to combine both the longitudinal and lateral strains and represent the surface strain in the form of resultant is not effective for the current layout of Demec stations as averaging of the surface strain varies the original location of the surface strain measured during testing. And also the ultimate aim of the current study to compare different bridge deck system is satisfied through the contours drawn on the basis of longitudinal strains as both the systems are having same boundary conditions and load cases. The contour graphs are plotted using Minitab software. The intermediate data points on contours are interpolated considering linear relationship between the actual recorded data points.

Test results are summarized in table 3.2 for comparison of cast in place overhang (CIP) with precast overhang with respect to cracking moment, maximum applied moment and loading location for test specimens.

Table 3.2: Summary of test results for bridge deck overhang.

Specimen	Type	Loading position	Cracking moment (kip-in)	Maximum applied moment (kip-in)	Max. deflection at service load (in)	Max. deflection at max. applied load (in)	Saftey factor (max. load / 16 K)	Failure Mode
1	Cast in place	Centre	1008	1872*	0.0055	0.0530*	6.5*	-
	Precast	Centre	864	1296*	0.0035	0.1220*	4.5*	-
2	Cast in place	Corner	720	1008	0.0070	1.2190	3.5	Punching shear
	Precast	Corner	720	1440	0.0130	0.5880	5.0	Punching shear
	Cast in place**	Corner	720	1152	-	-	4.0	Punching shear
	Precast**	Corner	864	1296	-	-	4.5	Punching shear

** The maximum loads for these specimens were limited by the loading equipment and do not reflect the actual strength of the specimen.*
*** The test results mentioned for these specimens were reported for corner testing conducted after testing corner for first time at farthest end of same overhang.*

First four tests listed in table 3.2 are discussed in detail in the following sections.

3.4.1 Cast in Place (CIP) bridge deck overhang center testing

Flexural cracks were formed at 56 kips loading and at the deflection of 0.0105 in. Three cracks were observed at this stage, one above the bridge girder closer to the interior edge of the bridge girder and of the remaining two one was at the interior panel and other was on overhang near the exterior edge of bridge girder. During the successive loading stages it was observed that the first crack above the girder closer to the interior edge of the bridge girder was widening until another crack was observed at 72 kips while the first crack at interior panel and on overhang remained same. Other cracks observed during successive loading until maximum load was reached are mapped in detail in fig.3.15. The numbers next to the crack denotes the loading at which the respective cracks were first noticed. The rectangular shape represents the orientation of the AASHTO HL 93 tire patch. The support girder is shown as a dotted line.

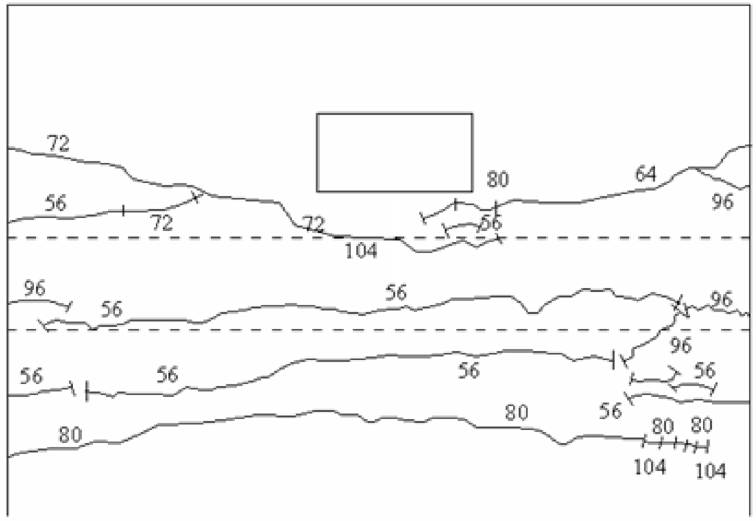
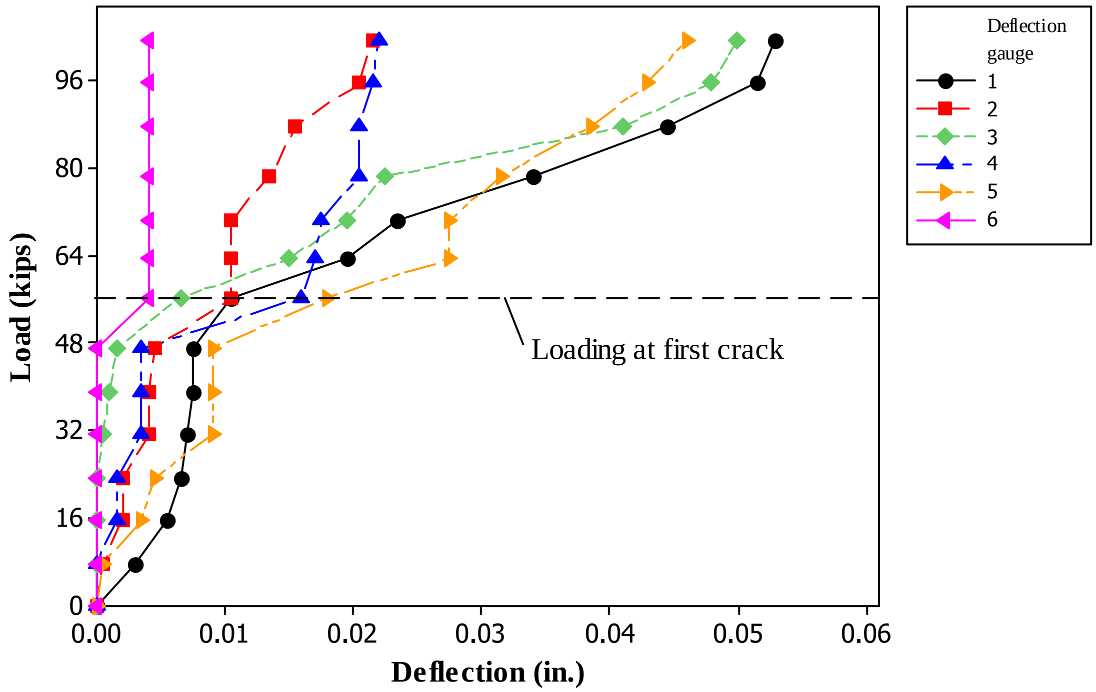


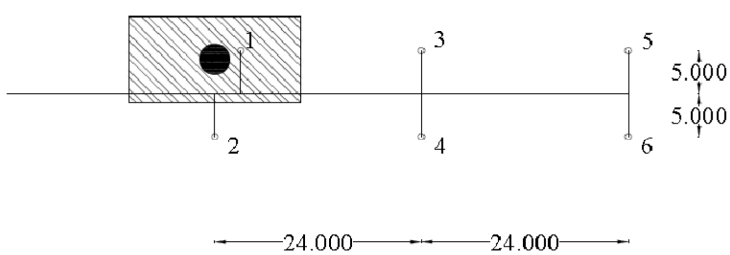
Fig.3.15: Crack pattern for the cast in place (CIP) overhang tested at center.

Figure 3.16 shows the location of the deflection gauges with respect to the steel plate (AASHTO standard tire patch for HL 93 truck) and load deflection graph for CIP overhang tested at center. The dotted line in the graph represents the loading at the first crack. The load deflection graph shows the bilinear relationship where stiffness of the bridge deck decreases after the first crack. Decrease in stiffness is observed by the drastic increase in the deflection of bridge deck after first crack.

The failure pattern of the bridge deck was not able to classified as loading was stopped at 104 kips due to limitation of the loading frame and hydraulic jack. The maximum moment applied (refer table 3.2) at this point was almost 6.5 times the design loading. Result of this test suggests that current bridge deck is having more potential than it was intended for.



(a)



(b)

Fig.3.16: a) Load deflection graph for CIP overhang tested at center, b) Location of deflection gauges with respect to the steel plate.

3.4.2 Prestressed precast bridge deck overhang center testing

The bridge deck first cracked at the load of 48 kips and at a deflection of 0.0150 in. This first crack was observed over the bridge girder. At a load of 56 kips this crack extended to the edges of the bridge deck. At the next loading stage of 64 kips the initial crack at the corner of the bridge deck closer to the exterior edge of the girder connected with a new crack on the overhang next to the exterior edge of the bridge girder. This cracking suggests the development of a flexural crack near the exterior edge of the bridge girder. Crack mapping of other cracks at several loading stages was completed and is shown in fig.3.17. The numbers next to the crack in below figure denotes the loading at which respective cracks were observed and the rectangular shape represents the orientation of the AASHTO HL 93 truck tire patch. Also the dotted lines depict the edges of the bridge girder.

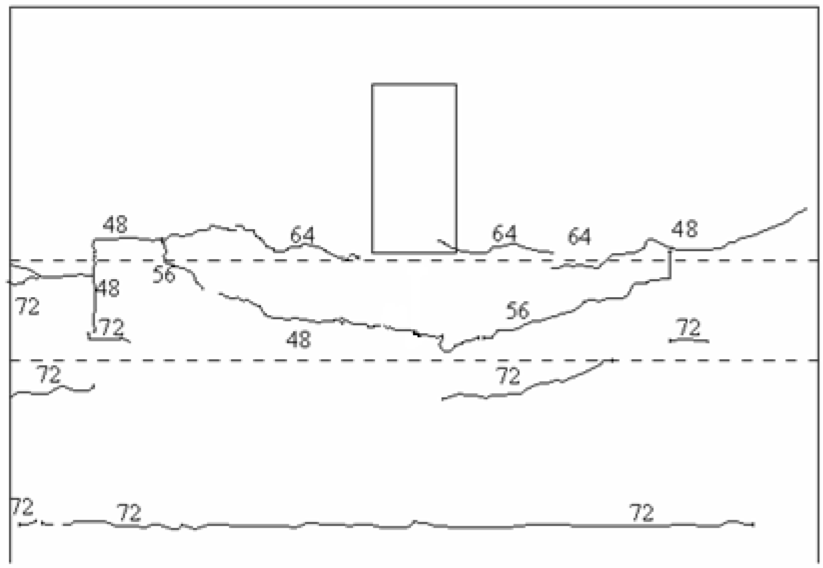
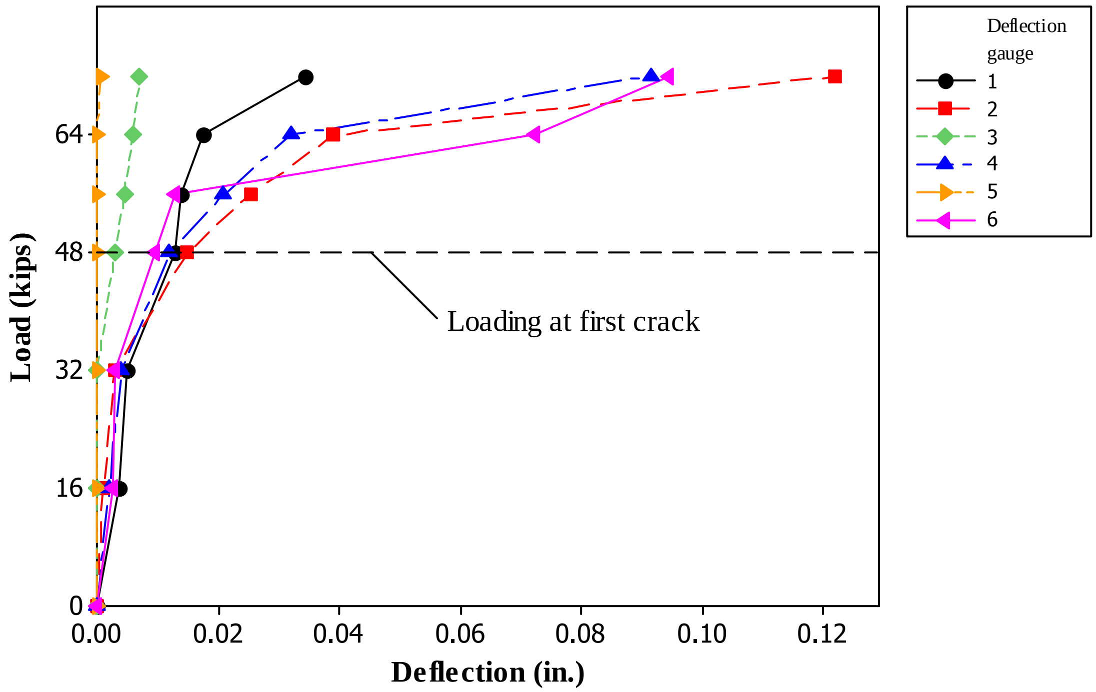


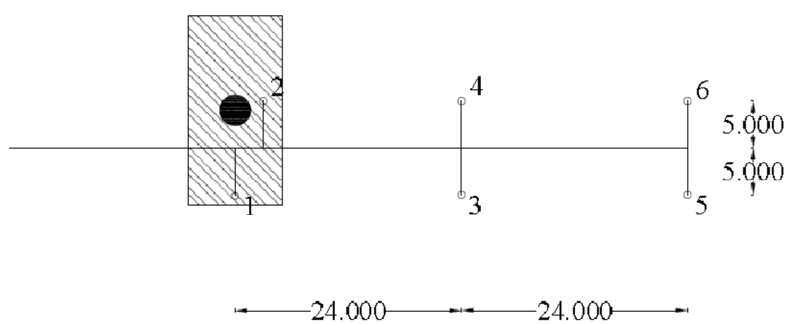
Fig.3.17: Crack pattern for the prestressed precast overhang tested at center

The locations of the deflection gauges with respect to the steel plate (AASHTO standard tire patch for HL 93 truck) and load deflection curve are shown in fig.3.18. The loading at the first crack is shown using dotted line in the graph. The load deflection graph for prestressed precast bridge deck overhang showed the bilinear relationship which is same as that for CIP bridge deck overhang. This system also showed the reduction in the stiffness after the first crack. This bridge deck showed more deflection than that of CIP bridge deck overhang.

For this case also it was not possible to classify the failure pattern as loading was stopped at 72 kips due to some technical difficulties. The maximum moment applied (refer table 3.2) at this point was almost 4.5 times the design loading. These test results showed that this system is having lot of reserved potential before first cracking and till failure.



(a)



(b)

Fig.3.18: a) Load deflection graph for prestressed precast overhang tested at center, b) Location of deflection gauges with respect to the steel plate.

3.4.3 Cast in Place (CIP) bridge deck overhang corner testing

First crack was observed at the load of 40 kips and at a deflection of 0.0515 in. The first crack emerged at this time was almost at an angle of 45^o to the bridge girder indicating the development of shear crack. It showed the widening of the first crack until the specimen was broken at 56 kips with development of the new crack. At this time failure was brittle failure and looking toward the cracking pattern which was at 45^o angle to the bridge girder it was categorized as punching shear failure. The fig.3.19 shows the crack pattern for the CIP bridge deck tested at corner. The loading at which crack emerges is noted next to the crack in below figure and the rectangular shape represents the orientation of the AASHTO HL 93 truck tire patch. The bridge girder is denoted by the dotted line.

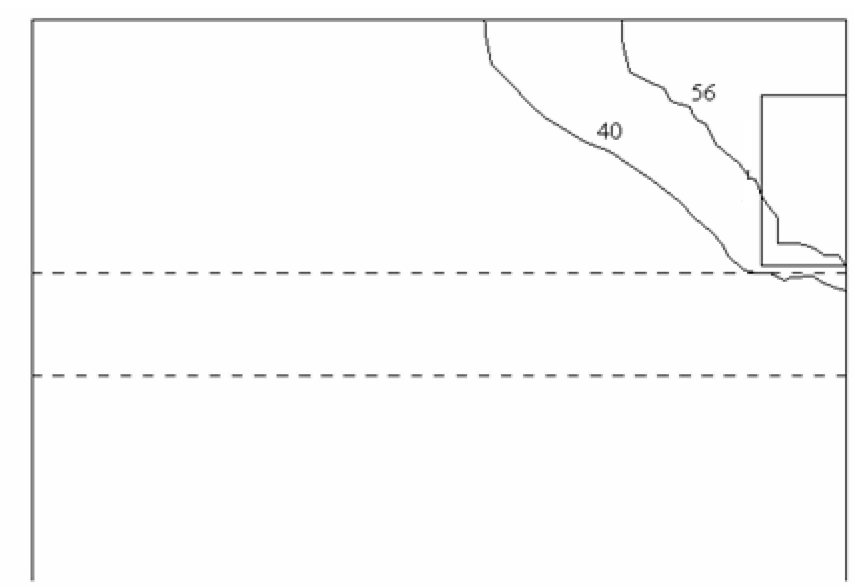
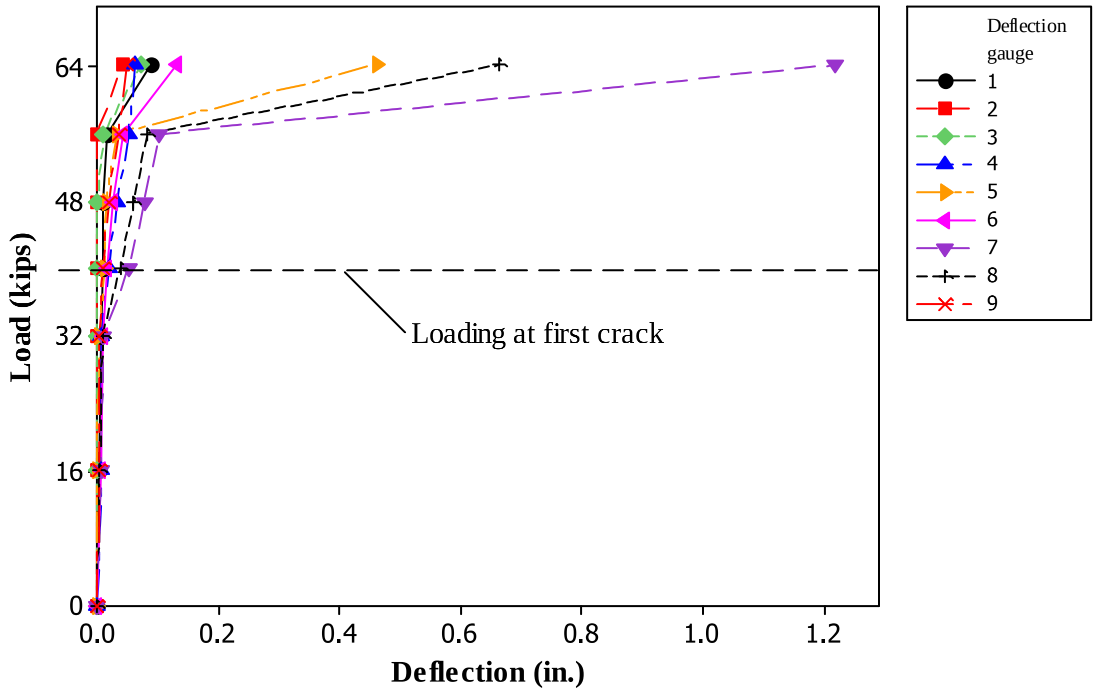


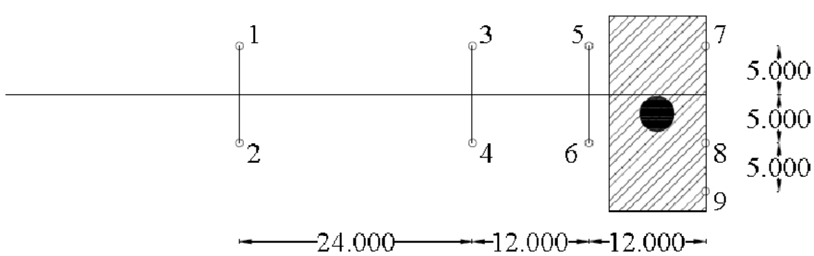
Fig.3.19: Crack pattern for the cast in place (CIP) overhang tested at corner.

The locations of the deflection gauges with respect to the steel plate (AASHTO standard tire patch for HL 93 truck) and load deflection curve are shown in fig.3.20 for CIP bridge deck overhang tested at corner. The dotted line in the graph represents the loading at the first crack. The overall behavior of the CIP bridge deck tested at corner is similar with respect to the load deflection curve for bridge deck overhang tested at center. For this specimen also we can observe the reduction of the stiffness after first crack.

Failure of the bridge deck was classified as punching shear failure since the cracking pattern showed the inclination of 45^o with the bridge girder. The maximum moment reached at this point was almost 3.5 times than that of designed loading (refer table 3.2). Results indicate that corner is the weakest part of the bridge deck overhang but still the current system showed some reserved potential beyond service load.



(a)



(b)

Fig.3.20: a) Load deflection graph for CIP overhang tested at corner, b) Location of deflection gauges with respect to the steel plate.

3.4.4 Prestressed precast bridge deck overhang corner testing

The cracking load for this test specimen was 48 kips and at the deflection of 0.0525 in. Couple of cracks emerged at this time with inclination to bridge girder showing presence of punching shear cracks. This bridge deck showed presence of more number of cracks than that of CIP bridge deck with emergence of a new crack for almost every load increment. Punching shear failure was occurred at 80 kips loading with almost 45^{o} inclination to the bridge girder. Some other minor cracks were also observed at this time which is mapped in detail in the following figure.

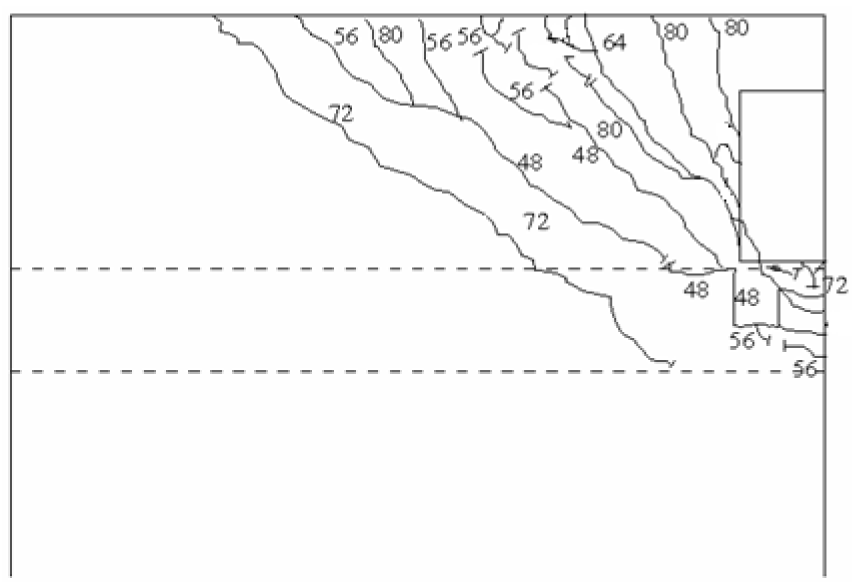
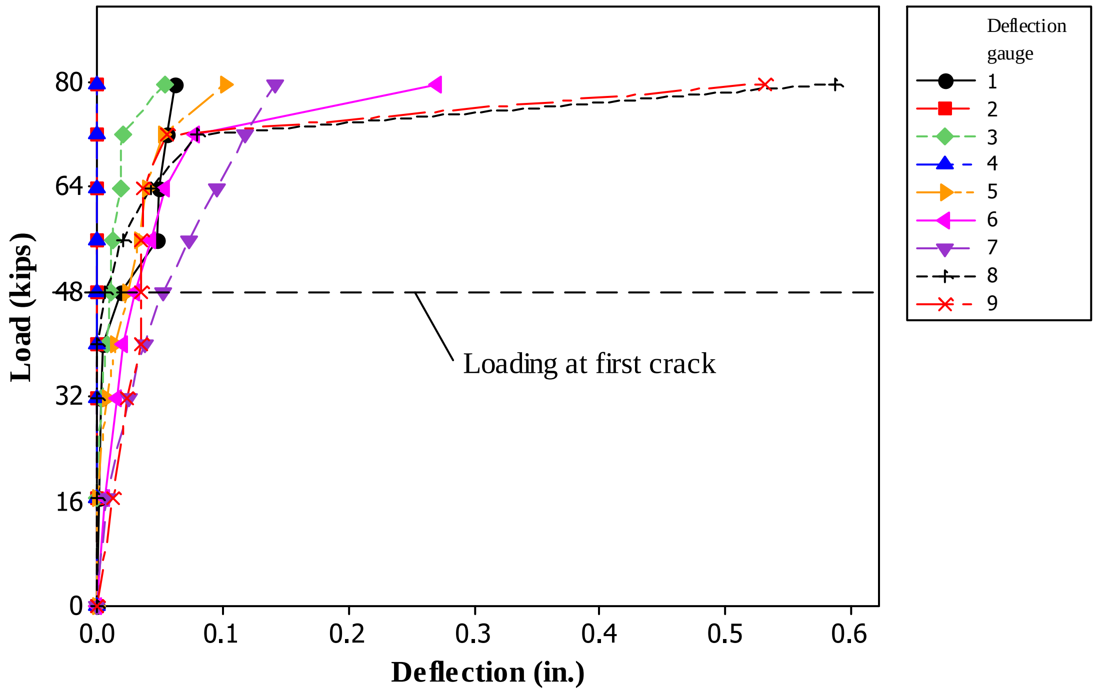


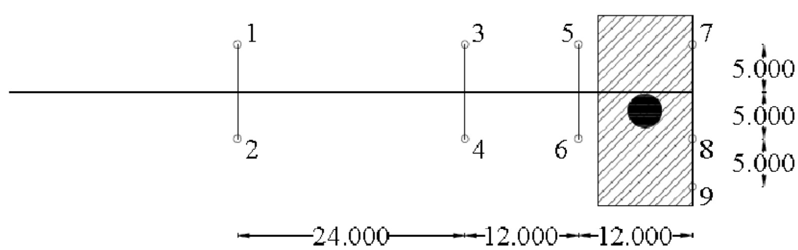
Fig.3.21: Crack pattern for the prestressed precast bridge deck overhang tested at corner.

The load deflection curve showed the similar bilinear behavior as that of previous tests with decrease in stiffness of bridge deck after first crack. Prestressed precast overhang showed the less deflection as that of CIP overhang tested at corner. The maximum deflection for the prestressed precast overhang was almost half that of CIP overhang tested at corner. The first cracking load is shown by the dotted line in the graph.

Observing the failure of the bridge deck at an inclination to the bridge girder this failure was categorized as punching shear. The maximum moment reached at the time of failure was almost 5 times that of design loading (refer table 3.2). Form the results it can be inferred that prestressed precast overhang is having lot more reserved potential to carry load beyond service load as compared to the CIP bridge deck overhang.



(a)



(b)

Fig.3.22: a) Load deflection graph for the prestressed precast bridge deck overhang tested at corner, b) Location of deflection gauges with respect to the steel plate.

Both the systems showed the ability to strain significantly after the initial cracking. This is the performance that is consistent with the ductile behavior of structure. For comparative study of the generalized behavior of the bridge deck during various load cases and loading stages; graphs for load versus microstrain are plotted (see fig 3.23, 3.24).

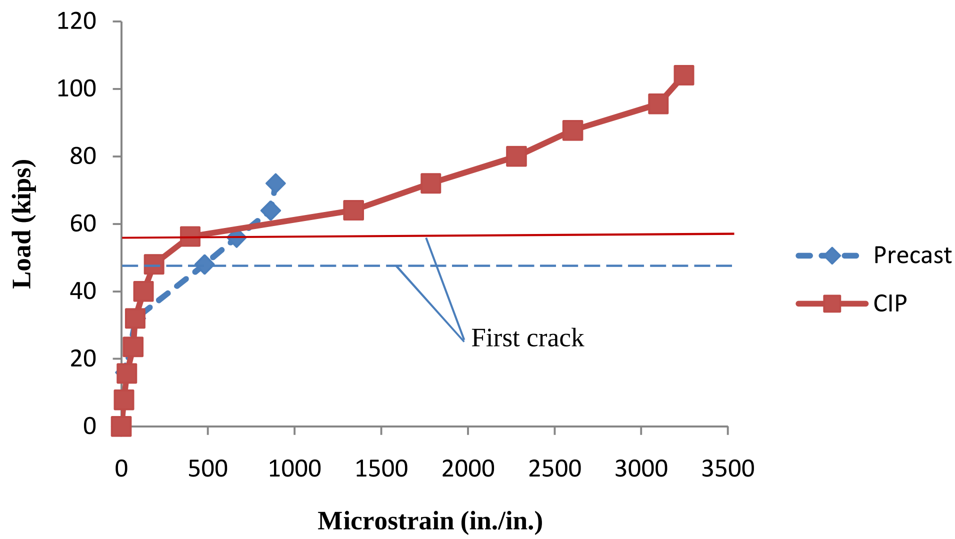


Fig.3.23: The load verses surface strain for the precast and conventional overhang for the center loading of specimen.

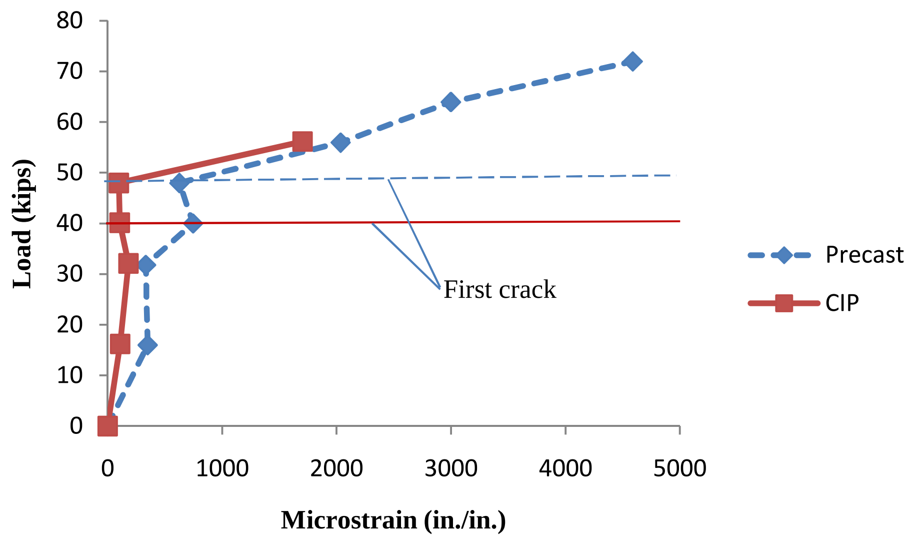


Fig.3.24: The load verses surface strain for the precast and conventional overhang for the corner loading of specimen.

3.5 Discussion

The previous tests show that both the systems evaluated provide a satisfactory capacity well beyond the design load. Furthermore, it was also verified that both systems satisfies the service limit state, fatigue and fracture limit state, and strength limit states as per AASHTO. Also as per AASHTO LRFD 2007 section 9.5.3 it is not necessary to test concrete bridge deck for fatigue performance as the large factor of safety encountered indicates that the service level stresses are expected to be low. Limit states for specimens tested as per AASHTO LRFD 2007 are listed in table 3.3. Besides this it was also observed that corners were the weakest portion of the bridge deck for both the systems. However both systems showed satisfactory performance with a safety factor of 4.5 for the precast overhang and 3.5 for the CIP overhang at the weakest portion of the bridge deck.

Table 3.3: AASHTO LRFD 2007 limit states for tested specimens.

Check	Limit state	AASHTO LRFD 2007 Section
1. Service limit state	Deflection should be > L/1200 i.e. 36/1200=0.03 in	9.5.2
2. Fatigue and Fracture limit state	N.A	9.5.3
3. Strength limit state	First crack loading should be > 16 K (service load)	9.5.4

3.5.1 Cast in Place (CIP) bridge deck overhang center testing

The maximum deflection of 0.0055 in. was observed at service load (16 kips). This deflection corresponds to L/6545, which is less than the specified AASHTO limit for serviceability of L/1200. For the tested system with 3' overhang, the AASHTO limit (L/1200) will be 0.03 in. The overall behavior of the CIP overhang was outstanding under service load because no cracks were detected. The first crack was observed at the 56 kips loading which is 3.5 times of the designed service load. The results also indicate that the system performance satisfies all of the requirements of serviceability and ability to transfer the loads. The large factor of safety encountered indicates that the service level stresses are expected to be low, which would indicate satisfactory fatigue performance for the system in service.

The load-microstrain graph shown in fig.3.23 signifies the brittle behavior of the cast in place (CIP) bridge deck overhang tested at centre. The drastic increase in surface strain i.e. loss in load carrying capacity after first crack (refer fig.3.23) for CIP bridge deck overhang tested at centre closely resembles the brittle behavior of structure.

Figure 3.25 shows the development of surface strain during all the phases of loading. Dotted line on the contour plots represents the location of the bridge girder and the rectangular hatched portion represents the orientation of tire patch during testing. Residual strains (i.e. surface strain after the release of loading) clearly show the development of the flexural failure of CIP bridge deck overhang. This phenomenon is supported by the load deflection curve (fig.3.18) showing very little linear decrease in the deflection form the loading point toward the edge of the overhang.

3.5.2 Prestressed precast bridge deck overhang center testing

The maximum service load (16 kips) deflection for this system was recorded as 0.0035 in. This deflection corresponds to L/10286, which is far less than the specified AASHTO limit for serviceability of L/1200. For the tested system with 3' overhang, the AASHTO limit (L/1200) will be 0.03 in. This system showed outstanding performance for load testing beyond the service load with no evidence of cracking until 48 kips which is almost 3 times that of service load as per AASHTO strength limit state for HL 93 truck. This system also satisfies all the limit state requirements set as per AASHTO.

From fig.3.23 it can be observed that this system shows significant inelastic deformation after first cracking until loss of load carrying capacity occurs; this behavior infers that precast bridge deck overhang system posses more ductility than that of CIP bridge deck system.

Cracks due to construction joint between full depth and partial depth portion of bridge deck is clearly visible at the interior span of the bridge deck. This cracking should not be the issue as it emerged at 72 kips which is about 4.5 times the service load.

The contour plots for prestressed precast bridge deck overhang tested at center during the later stages of loading (i.e. 48 kips) infers that this system shows the uniform load distribution throughout the edge of the girder. This phenomenon is clearly illustrated by the orientation of contour plots parallel to the edge of the girder which implies that precast prestressed bridge deck panels are having capability to distribute load uniformly throughout the panel. But, the pattern of residual surface strains indicates the development of punching shear failure. This phenomenon is also supported by the behavior of the bridge deck observed in fig.3.18. From fig.3.18 it is clearly seen that gauge 2 had deflected about 30% more than that of remaining two gauges (4 and 6) which are closer to the edge and same behavior is observed in the case of gauge 1 with respect to gauges 3 and 5; which implies the development of the localized failure.

3.5.3 Cast in Place (CIP) bridge deck overhang corner testing

This system has shown the maximum deflection of 0.007 in. at the service load (16 kips) which corresponds to L/5143. This is far less than the specified AASHTO limit L/1200 (i.e. 0.03 in.) for the system tested with 3’ overhang. First crack was observed at 2.5 times the factored service loading as per AASHTO HL 93 truck. This implies that current system performs satisfactorily under service loads and posses some potential to resist loading beyond the service load.

Figure 3.24 shows the graph of load versus microstrain plotted for specimen tested for static loading as per AASHTO LRFD at corner. This graph characterizes the ductile performance of the structure. From the aforementioned graph we can see that CIP bridge deck overhang system shows the sudden loss of load-carrying capacity after first crack (i.e. after elastic limit of structure) which implies that CIP bridge deck system shows the brittle behavior.

As per load deflection curve (refer fig.3.20) almost all deflection gauges showed no deflection until 32 kips loading and after 32 kips deflection gauge 7 showed maximum deflection throughout the loading. This implies that there was localized failure near the location of deflection gauge 7 (i.e. near the edge of girder). Crack map (refer fig.3.19) clearly shows that first crack originated near the location of deflection gauge 7 and also from surface strain contours it can be clearly seen that the region near the deflection gauge was strained more as compared to the other region under observation which gives another evidence for beginning of localized failure. Throughout the loading phases after first crack it was observed that there was growth in the first crack until the brittle failure of bridge deck at 56 kips with emergence of second crack. The surface strain contours also provide the evidence for load path during testing. From figure 3.26 we can see the surface strain was high at the inclination of 45^o to the bridge girder which represents the orientation of the first crack. During the later stages of loading the load transfer through the first crack was observed with the growth in surface strain around the region of first crack until the brittle failure at 56 kips with emergence of second crack. The localized failure with load transfer through only one path and then sudden failure closely characterized the behavior of brittle i.e. less ductile material.

3.5.4 Prestressed precast bridge deck overhang corner testing

For prestressed precast bridge deck overhang corner testing the maximum deflection of 0.0130 in. was observed at service load (16 kips). This deflection corresponds to L/2769, which is less than the specified AASHTO limit for serviceability of L/1200. For the tested system with 3’ overhang, the AASHTO limit (L/1200) will be 0.03 in. The performance of prestressed precast bridge deck overhang during corner testing was outstanding under service load as it showed no cracks before service load (16 kips). The first event of crack was noticed at 48 kips loading which is 3 times that of the designed service load. Form the aforementioned results it can be inferred that this system performance satisfies all of the requirements of serviceability and ability to transfer the loads. With all this norms the proposed system satisfies all the limit state requirements as per AASHTO.

As per fig.3.24 prestressed precast bridge deck overhang system showed significant inelastic deformation before failure. This shows the characteristics of the ductile behavior of the structure. This increased ductility for the prestressed precast bridge deck overhang system as that of CIP bridge deck system is predicted due to the confinement of material because of the prestressing force at the bottom of the proposed panels.

Prestressed precast bridge deck overhang system tested at corner showed uniform distribution of load on the edge of girder until the first crack occurred at 40 kips loading. This behavior can be clearly illustrated form the contour plots of surface strains (refer fig.3.26). Load transfer path which was along the edge of the girder prior to the first crack was changed after first crack at 48 kips loading and was at inclination to the bridge girder along the crack. This crack acted as a hinge and all the load transfer was observed along this hinge or crack until second crack or hinge formation was observed. Several cracks were observed in succession during load increment and most of the load transfer was observed through the nearest crack to the loading point (i.e. redistribution of load was observed due to emergence of new cracks). Failure of bridge deck occurred with ample warning before breaking of bridge deck.

3.5.5 Comparison of CIP and prestressed precast bridge deck overhang

Form load-microstrain curve for both the systems tested at centre (fig.3.23) it can be seen that both the systems shows the same stiffness until 32 kips which is twice the service load. But, the proposed bridge deck system shows more flexibility than that of conventionally used CIP bridge system before cracking. Also for bridge decks tested at corner, proposed bridge deck system showed more ductility throughout the loading phases with 16% increase in first cracking load (refer fig.3.24).

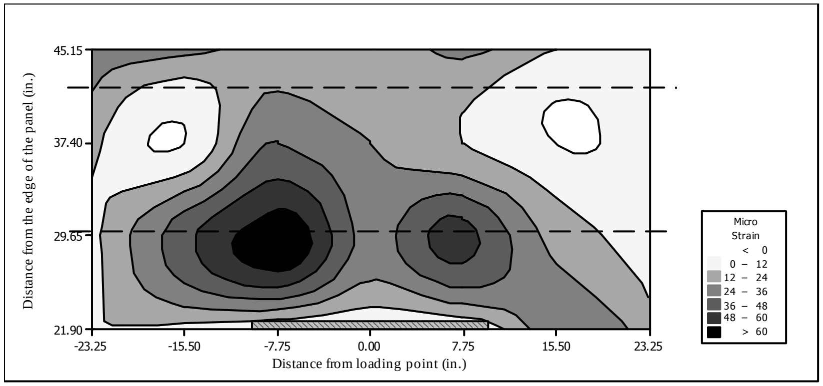
Furthermore, first cracking load for proposed system was same (i.e. 48 kips) regardless of the location of loading point; this is expected due to the behavior of this system to distribute load uniformly which is characterized due to ductility of the proposed system. Whereas, for conventionally used CIP bridge deck system first cracking load for bridge deck tested at centre was 56 kips and at corner was 40 kips. This reduction in first cracking load when bridge deck tested at corner is expected due to its brittle nature.

Both the systems showed satisfactory performance under factored service load as per AASHTO LRFD and posses the reserved potential to transfer load beyond the service limit. But, the proposed prestressed precast bridge deck has shown the outstanding performance with more ductility over the performance of conventionally used CIP bridge deck overhang. This can be clearly seen from the figures of first cracking moment and maximum moment attended by both the systems under different load cases (refer table 3.2). Also, the data for deflection and surface strain for both the loading cases shows that the proposed prestressed precast bridge deck overhang system is less prone to the cracking with enhanced ductility.

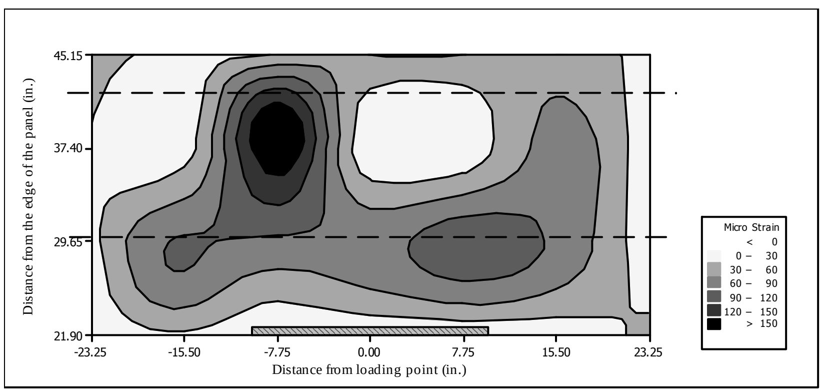
Fig.3.25: Comparison of surface strain of Cast in place (CIP) and precast bridge deck overhang tested at centre.

Cast in place bridge deck overhang

Loading: 16 Kips (service load)

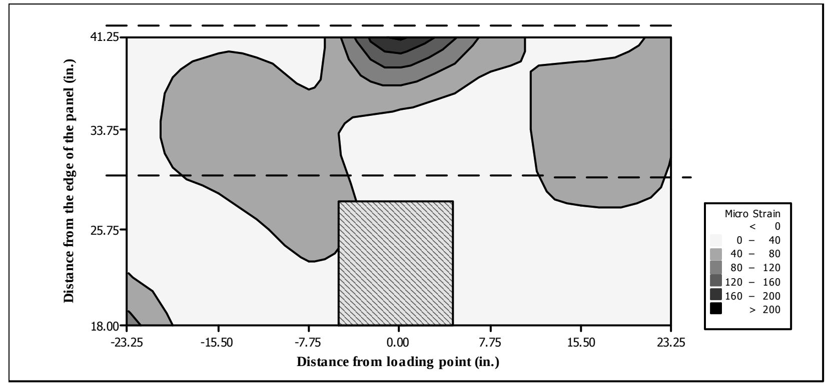


Loading: 32 Kips



Precast bridge deck overhang

Loading: 16 Kips (service load)



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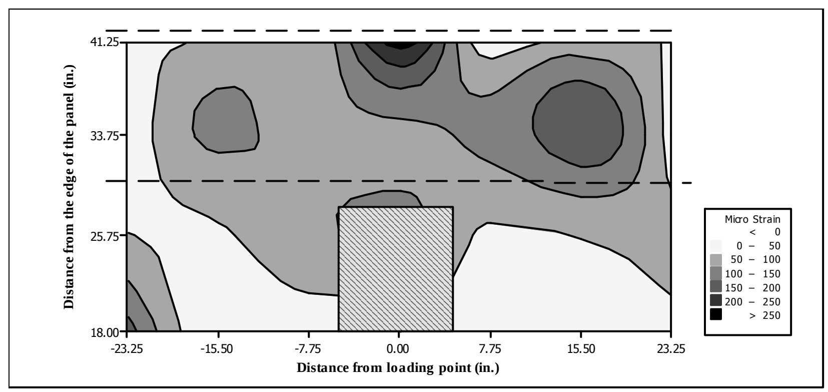

Cast in place bridge deck overhang

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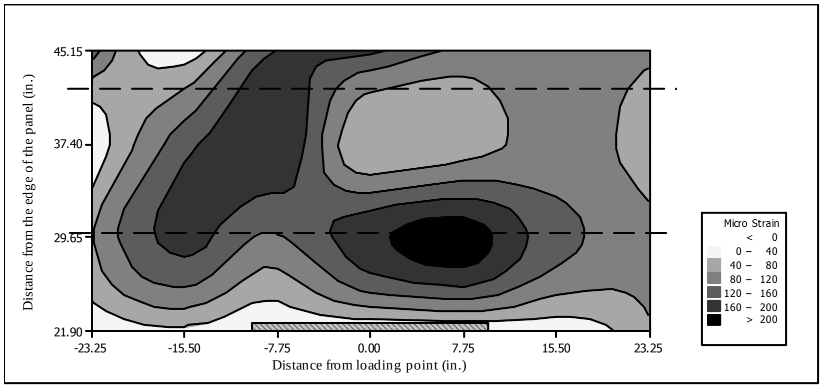

Precast bridge deck overhang

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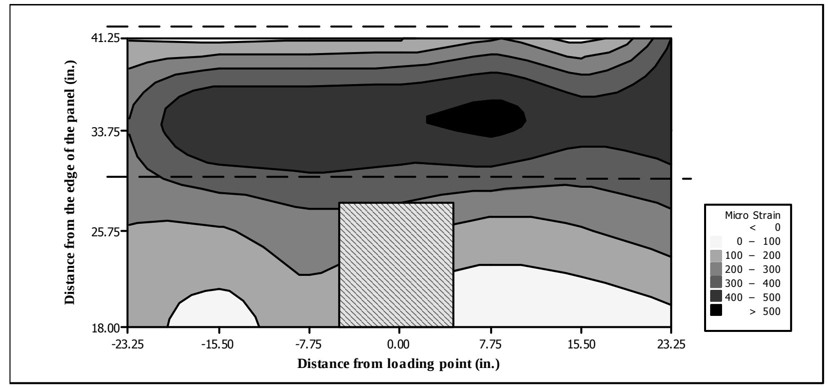

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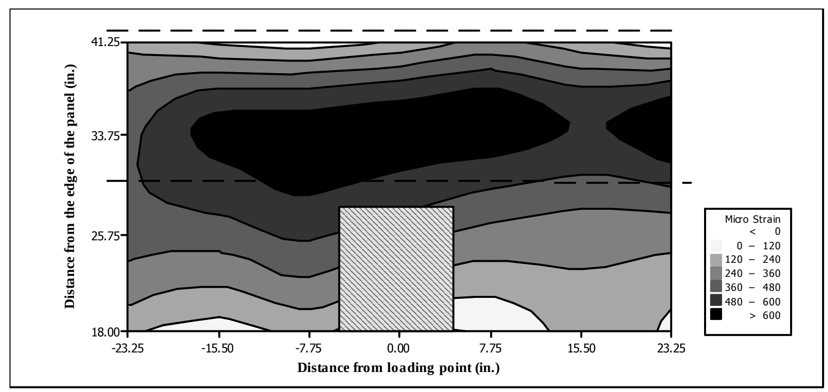

Cast in place bridge deck overhang

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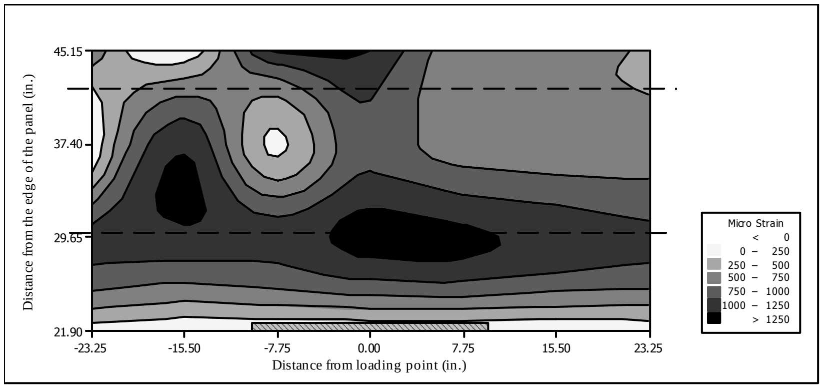

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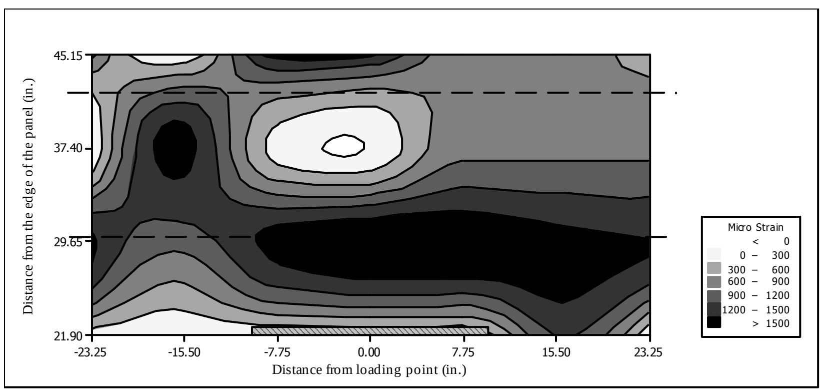

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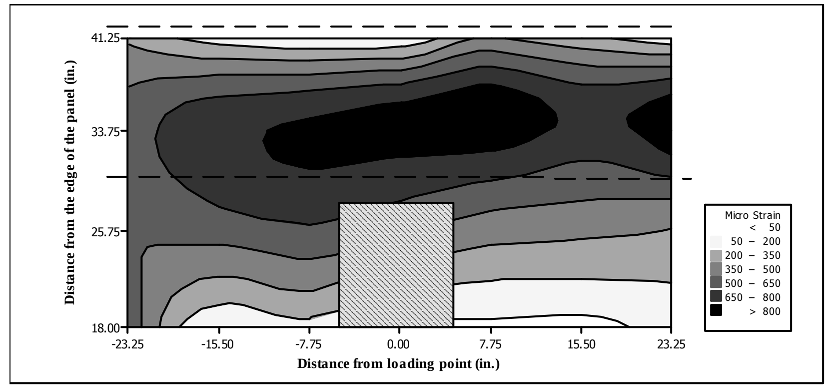

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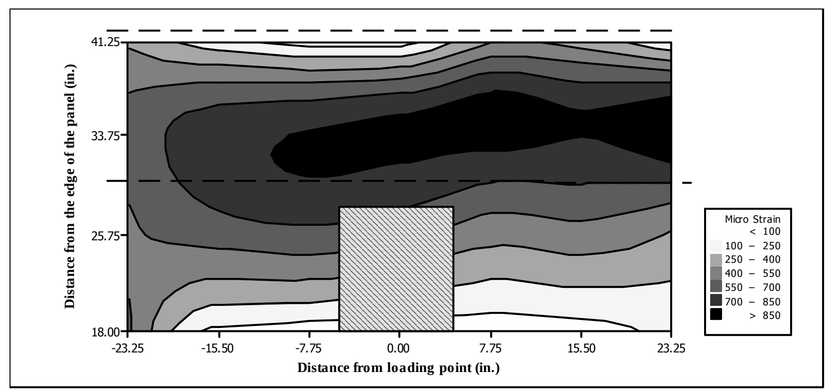

Cast in place bridge deck overhang

Precast bridge deck overhang

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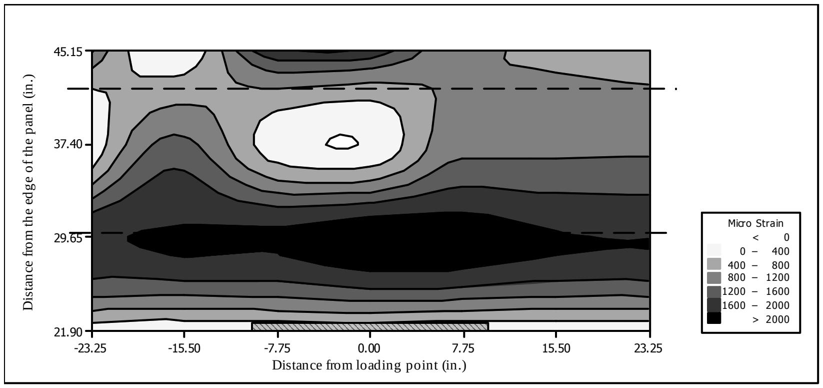

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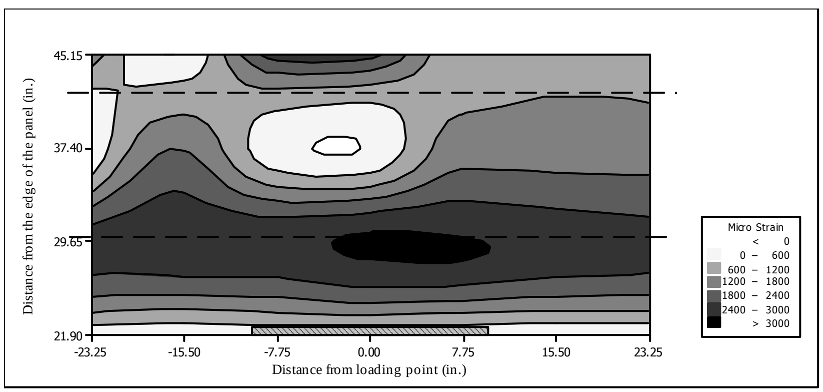

Cast in place bridge deck overhang

Precast bridge deck overhang

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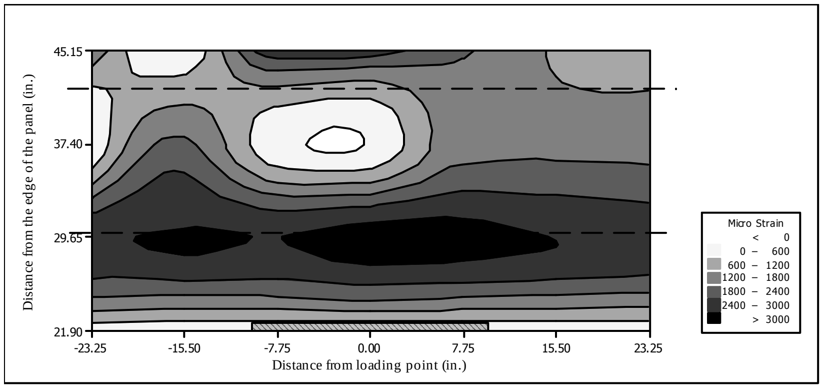

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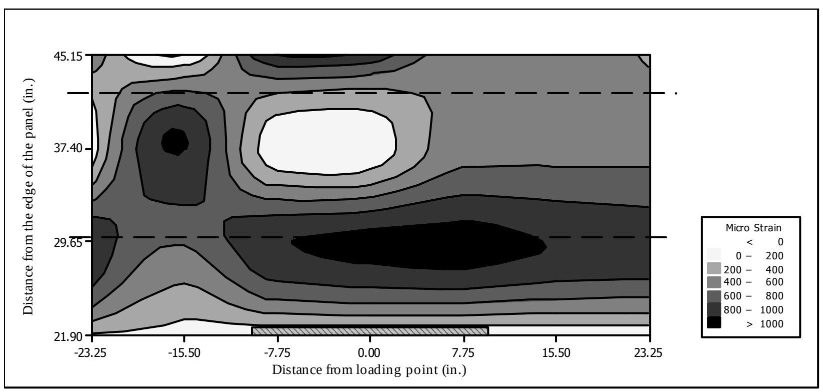

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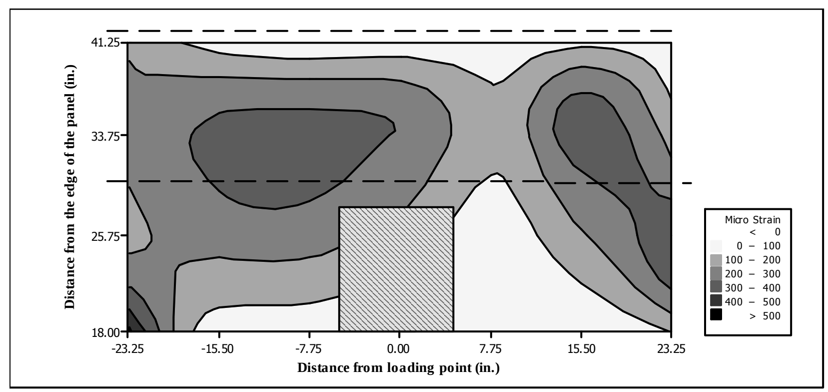
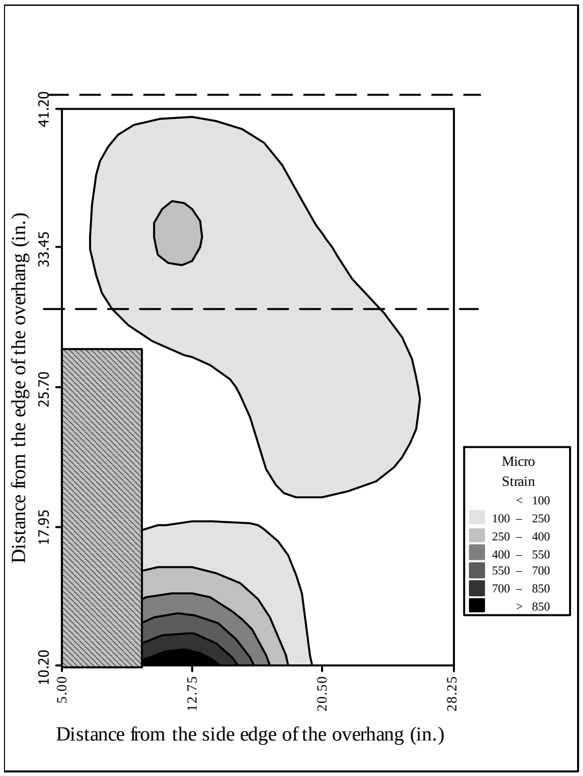


Fig.3.26: Comparison of surface strain of Cast in place (CIP) and precast bridge deck overhang tested at corner.

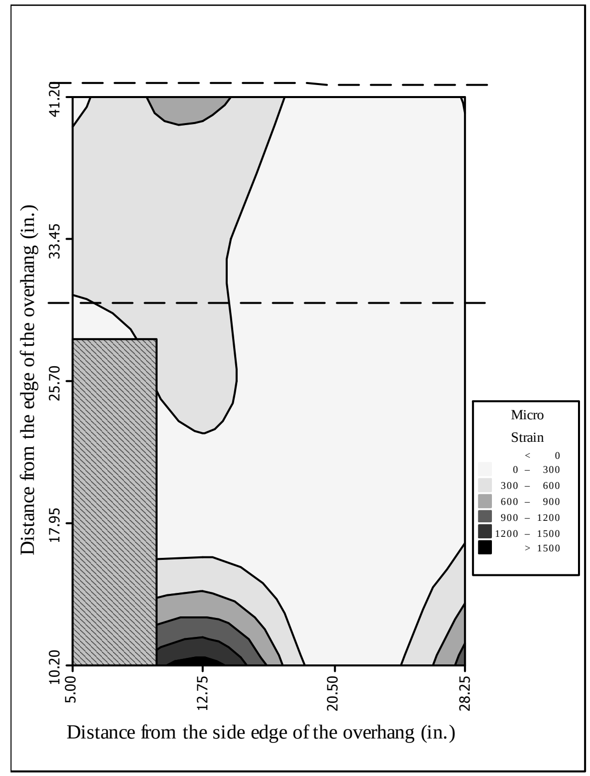
Cast in place bridge deck overhang

Loading: 16 Kips (service load)



Precast bridge deck overhang

Loading: 16 Kips (service load)



Cast in place bridge deck overhang

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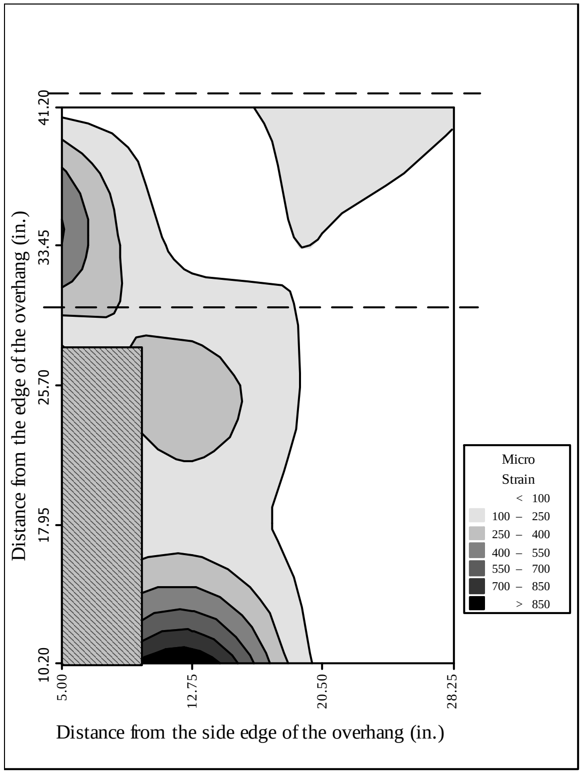

Precast bridge deck overhang

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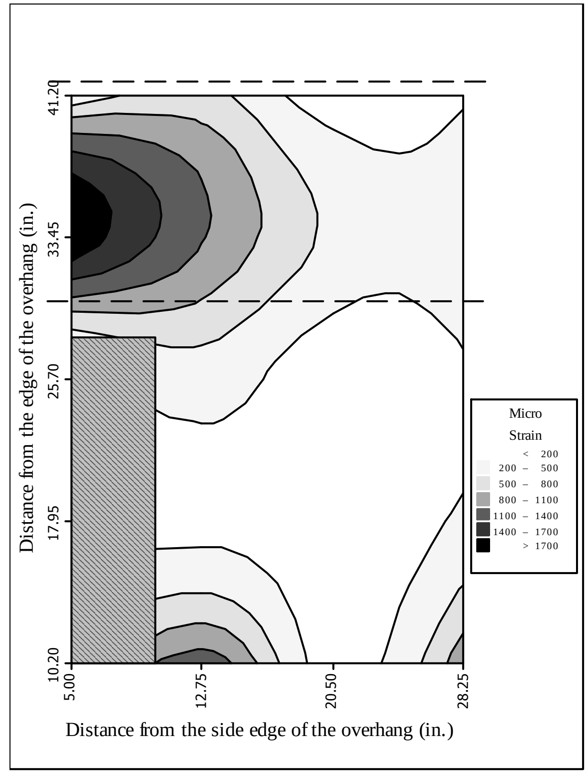

Cast in place bridge deck overhang

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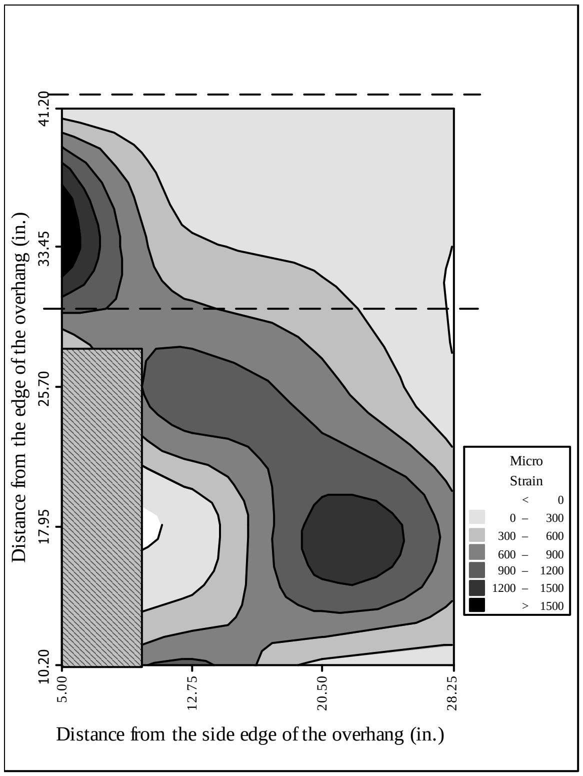

Precast bridge deck overhang

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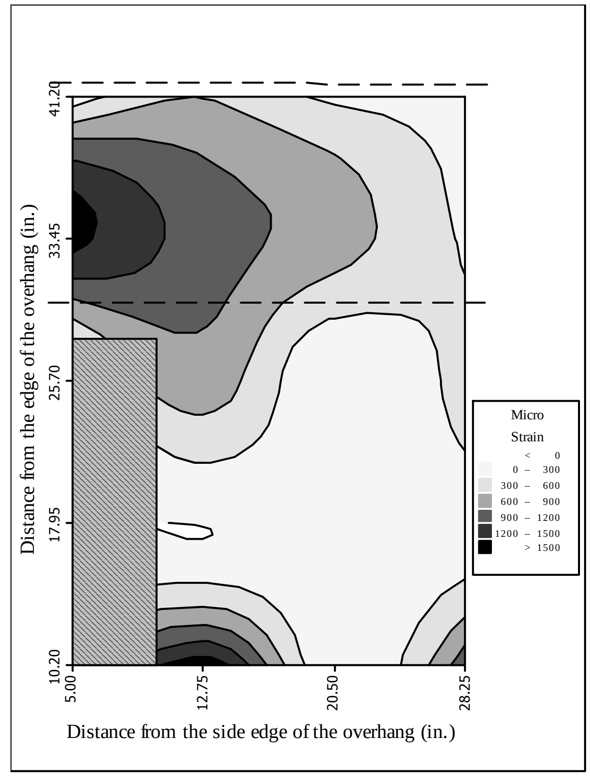

Cast in place bridge deck overhang

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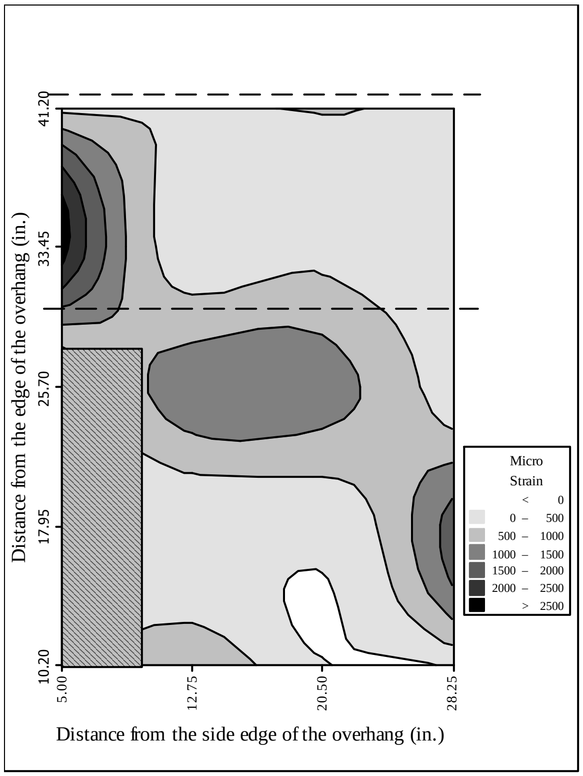

Precast bridge deck overhang

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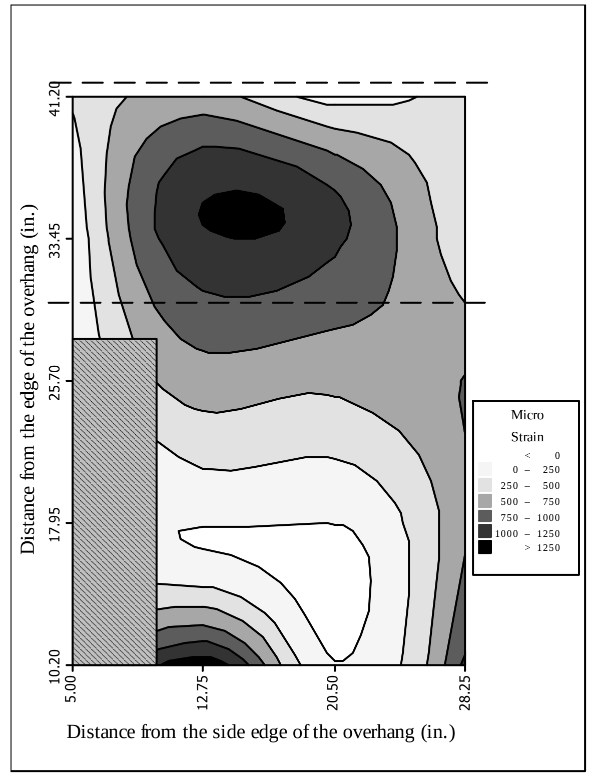

Cast in place bridge deck overhang

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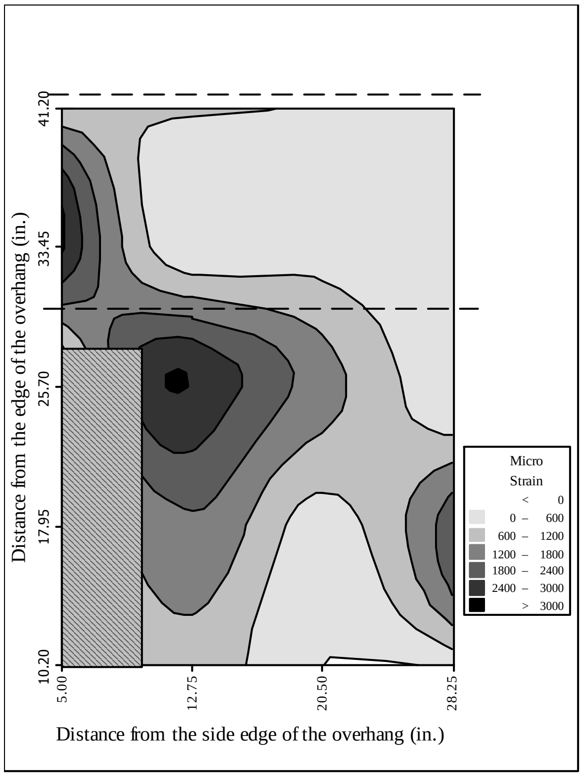

Precast bridge deck overhang

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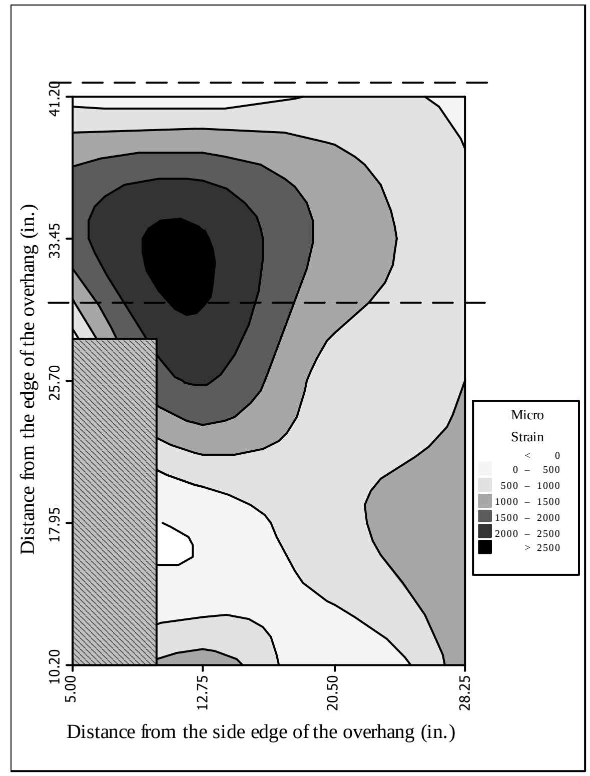
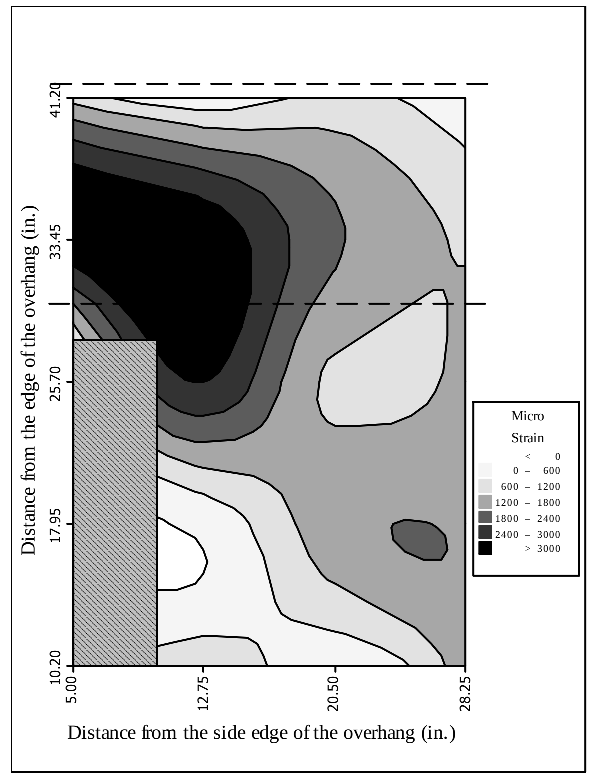


Cast in place bridge deck overhang

Precast bridge deck overhang

Loading: 64 Kips



Cast in place bridge deck overhang

Precast bridge deck overhang

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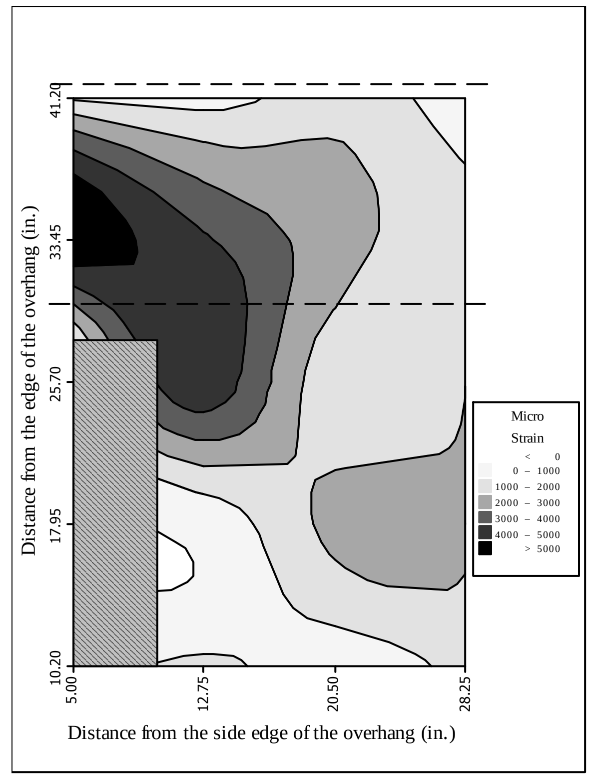
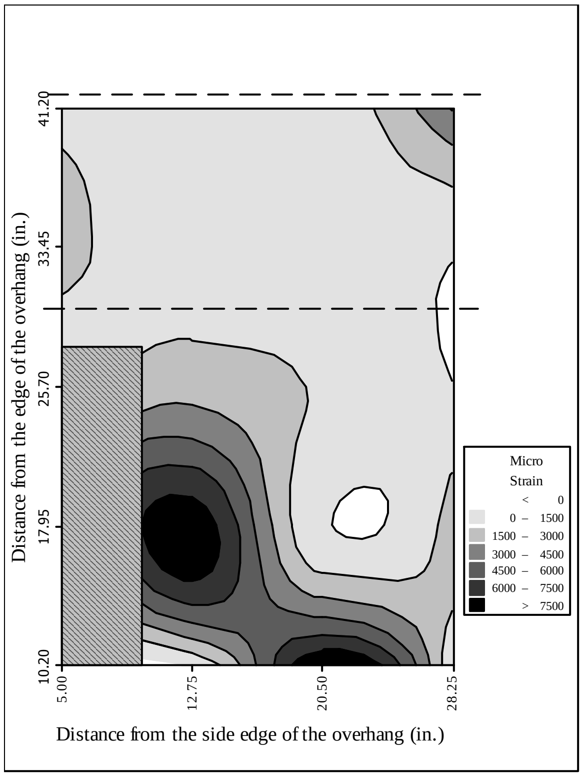


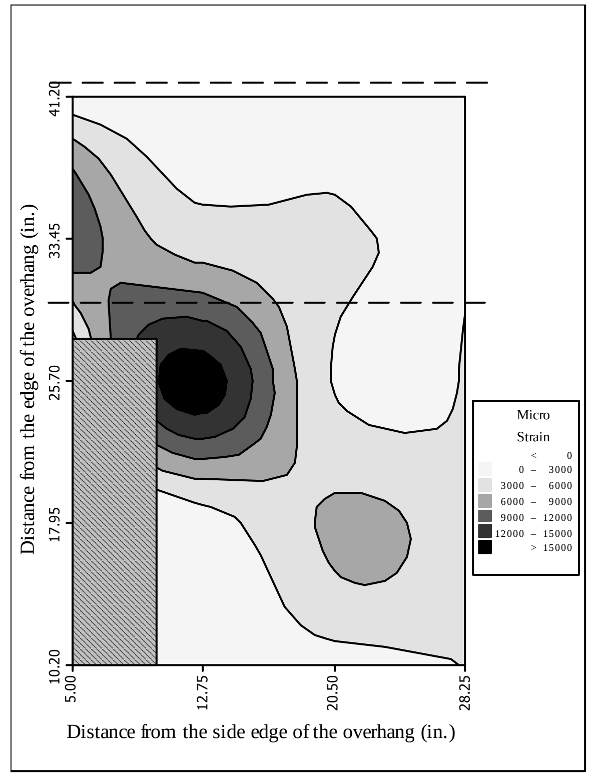
Cast in place bridge deck overhang

Loading: Breaking load



Precast bridge deck overhang

Loading: Breaking load



3.6 Conclusions

Based on the experimental test results for both loading cases, the following conclusions can be drawn:

1. Both bridge deck systems demonstrated an acceptable structural behavior without any cracking under service loads.
2. The deflection for both the system under both load cases tested was much lower than the AASHTO limit for serviceability.
3. For the overload case (i.e. loading beyond service load), both the system has shown reserved potential to transfer load beyond service loading. Looking toward the deflection at service load and first cracking load, it may be possible to utilize this reserved potential by extending the length of bridge deck overhang.
4. Proposed bridge deck system has shown excellent performance as compared to the conventionally used CIP bridge deck system in all respect for both the load cases.

CHAPTER IV

CONCLUSIONS AND RECOMMENDATIONS

The prime objective of this research thesis is to develop the precast bridge deck overhang construction technique. This construction technique will enhance safety, accelerate construction, simplify construction, and decrease costs. Several attempts have been made in the past to accelerate bridge deck construction by implementing full depth and partial depth panels, but each of them have some limitations. The proposed system is developed in such a way that it will address these limitations and drawbacks of the past bridge deck construction techniques.

The research in this thesis focuses on two prime objectives which are necessary to make the proposed system implementable:

a. Development of an adjustable haunch form for precast bridge decks.
b. Full scale testing of a precast bridge deck overhang.

The haunch plays a vital role while using precast elements for the bridge deck construction as it consumes significant amount of time and labor to achieve the correct road alignment. Adjustable haunch form will reduce the time and labor necessary to form the haunch. The ideal material for an adjustable haunch form will adjust in size as the geometry of the precast overhang is adjusted. The combination of five different foams and three adhesives were tested for their suitability using seven tests. These tests were specially designed to examine the performance of proposed adjustable haunch form during several different phases of bridge deck construction.

The implementation of precast bridge deck overhang system will considerably reduce the time and labor for construction of bridge decks with enhanced work safety and quality. The proposed bridge deck system was tested for static load capacity under two different load cases. The performance of the proposed bridge deck system was compared with the conventional cast-in-place (CIP) bridge deck system with respect to the surface strain, deflection and cracking.

4.1 Conclusions:

a. Development of an adjustable haunch form for precast bridge decks.

1. The combination of packing foam 1 and adhesive A has been identified to be an acceptable candidate to be used as a haunch form. This system showed the ability to resist a lateral pressure over 6.5 psi or approximately 6.5' of concrete and an elongation of 0.9" before failing. It also showed satisfactory performance when adhesive was cured at 45^{o}F and also when tested at joint of foam. Besides this the foam manufacturer creates this foam in a gray color that is similar to concrete which doesn't create an aesthetic problem for the final bridge.

2. While this testing was completed with specific sizes of haunch forms other heights and configurations could be estimated based on the data presented in this report. Furthermore, while the focus of this research program has been on providing this forming system for haunches of bridge girders the same concepts could be extended to be investigated for use in several other applications involving precast elements.

b. *Full scale testing of a precast bridge deck overhang.*

4. Both bridge deck systems demonstrated an acceptable structural behavior without any cracking under service loads.
5. The deflection for both the system under both load cases tested was much lower than the AASHTO limit for serviceability.
6. For the overload case (i.e. loading beyond service load), both the system has shown reserved potential to transfer load beyond service loading.
7. It may be possible to utilize the reserved capacity by extending the length of bridge deck overhang.

5. The proposed bridge deck system has shown excellent performance as compared to the conventionally used CIP bridge deck system in all respect for both the load cases.

4.2 Recommendations:

1. More data should be collected for adjustable haunch testing so as to come up with generalized behavior of foam and adhesive to resist lateral pressure due to fresh concrete or grout. This would help to model and design the adjustable haunch form under various circumstances.
2. From the current evaluation and the failure pattern of the bridge deck overhang tested at corner it is observed that punching shear failure is predominant for the case of bridge deck overhang. It might be useful to study the mechanism of the bridge deck along the cross section so as to develop strut and tie model for designing the bridge deck overhangs.
3. As per AASHTO LRFD 1.3.2.5, statically ductile but dynamically non-ductile response characteristics should be avoided. Examples of this behavior are shear and bond failures in concrete members and loss of composite action in flexural components. Hence it is recommended that proposed system should be investigated for the response of dynamic loading due to moving vehicles on the interface between cast in place topping and precast panel.

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APPPENDIX A

TEST DATA FOR ADJUSTABLE HAUNCH TESTING

Lateral Pressure Test (Foam 1 and Adhesive A)

One Day curing strenght.

Date 4/22/2008

Type of foam 1

Amount of glue **8** gm

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.105	0.0945	0.0195	0.0455	0.1905	0.0465	0.0825	0.2795	0.0805	0.114	0.3435	0.11
A #2	0.0195	0.112	0.025	0.037	0.193	0.0425	0.0655	0.2925	0.064			**fail**
A #3	0.0155	0.103	0.0195	0.037	0.18	0.035	0.0635	0.2555	0.0425	0.093	0.3235	0.065
Glue	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.146	0.4025	0.139	**fail**								
A #2												
A #3			**fail**									

One Day curing strenght.

Date 5/28/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.044	0.095	0.017	0.755	0.155	0.0295	0.1825	0.3125	0.061	**fail**		
A #2	0.0335	0.091	0.0195	0.0715	0.174	0.039	0.1845	0.3675	0.072	**fail**		
A #3	0.028	0.0835	0.0435	0.076	0.135	0.0525	0.189	0.2485	0.1045	**fail**		
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1												
A #2												
A #3												

Two Day curing strenght.

Date 4/23/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.015	0.0835	0.0135	0.0355	0.1685	0.0345	0.056	0.255	0.0565	0.0735	0.317	0.0755
A #2	0.0155	0.1035	0.0215	0.038	0.1955	0.047	0.062	0.304	0.0845	0.089	0.3795	0.1115
A #3	0.0055	0.077	0.0155	0.0145	0.1375	0.0345	0.049	0.2185	0.0595	0.078	0.2745	0.0815
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.095	0.3825	0.102	0.121	0.4445	0.131						
A #2	0.128	0.46	0.138			fail						
A #3	0.122	0.3455	0.116	0.167	0.413	0.148						

Lateral pressure Test (Foam 2 and Adhesive A)

One Day curing strenght.

Date 2/19/2008

Type of foam 2

Amount of glue **8** gm

Adhesive		1.5	psi		2.5	psi		3.5	psi		4.5	psi
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0315	0.1095	0.0275	0.0615	0.2165	0.0560	0.1095	0.3445	0.1195			fail
A #2	0.0230	0.1150	0.0180	0.0480	0.2280	0.0360	0.0660	0.3005	0.0450	0.0930	0.3975	0.0520
A #3	0.0195	0.0980	0.0345	0.0340	0.1940	0.0650	0.0520	0.2840	0.1030			fail
Adhesive		**5.5**	**psi**		**6.5**	**psi**		**7.5**	**psi**		**8.5**	**psi**
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1												
A #2			fail									
A #3												

Two Day curing strenght.

Date 2/20/2008

Adhesive		1.5	psi		2.5	psi		3.5	psi		4.5	psi
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0170	0.0710	0.0135	0.0395	0.1620	0.0310	0.0605	0.2405	0.0465	0.0845	0.3010	0.0605
A #2	0.0145	0.0980	0.0135	0.0320	0.1740	0.0275	0.0515	0.2550	0.0460	0.0725	0.3225	0.0650
A #3	0.0085	0.0710	0.0165	0.0195	0.1340	0.0375	0.0345	0.2075	0.0655	0.0530	0.2870	0.0965
Adhesive		**5.5**	**psi**		**6.5**	**psi**		**7.5**	**psi**		**8.5**	**psi**
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.1060	0.3620	0.0780			fail						
A #2	0.0955	0.3805	0.0795	0.1175	0.4210	0.0965						
A #3	0.0735	0.3520	0.1285	0.1010	0.4185	0.1645						

Lateral pressure Test (Foam 2 and Adhesive B)

One Day curing strenght.
Date 2/26/2008
Type of foam 2
Amount of glue **10** gm

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
B #1	0.0035	0.067	0.016	0.007	0.1185	0.0305	0.014	0.178	0.05			fail
B #2	0.002	0.07	0.0135	0.005	0.127	0.027	0.011	0.19	0.0455			fail
B #3	0.0085	0.063	0.0095	0.017	0.1185	0.023	0.0275	0.1805	0.0425			fail
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
B #1												
B #2												
B #3												

Two Day curing strenght.
Date 2/27/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
B #1	0.0085	0.0535	0.013	0.019	0.12	0.0305	0.0335	0.186	0.053			fail
B #2	0.002	0.068	0.0155	0.0045	0.1375	0.036	0.0095	0.2045	0.059			fail
B #3	0.003	0.0535	0.009	0.0065	0.098	0.021	0.013	0.151	0.039			fail
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
B #1												
B #2												
B #3												

Lateral Pressure Test (Foam 2 and Adhesive C)

One Day curing strenght.
Date 6/19/2008
Type of foam 2
Amount of glue 18 gm

Adhesive		1.5	psi		2.5	psi		3.5	psi		4.5	psi
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
C #1	0.016	0.1045	0.0275	fail								
C #2	0.012	0.096	0.0225	0.024	0.138	0.0505	0.0465	0.2085	0.0775	fail		
C #3	0.0165	0.0845	0.0295	0.029	0.145	0.0515	0.0505	0.233	0.083			fail
Adhesive		**5.5**	**psi**		**6.5**	**psi**		**7.5**	**psi**		**8.5**	**psi**
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
C #1												
C #2												
C #3												

Two Day curing strenght.
Date 7/1/2008

Adhesive		1.5	psi		2.5	psi		3.5	psi		4.5	psi
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
C #1	0.002	0.0815	0.0185	0.018	0.153	0.044	0.0405	0.238	0.075	fail		
C#2	0.0185	0.0845	0.019	0.033	0.1425	0.037	0.0565	0.227	0.067	0.1055	0.359	0.116
C #3	0.0275	0.1025	0.0205	0.0685	0.1925	0.0525	0.276	0.418	0.1045	fail		
Adhesive		**5.5**	**psi**		**6.5**	**psi**		**7.5**	**psi**		**8.5**	**psi**
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
C #1												
C #2	fail											
C #3												

Lateral pressure Test (Faom 3 and Adhesive A)

One Day curing strenght.

Date 3/11/2008

Type of foam 3

Amount of glue **8** gm

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0055	0.0455	0.0085	0.019	0.115	0.0245	0.0355	0.1745	0.039	0.053	0.2445	0.0615
A #2	0.009	0.0565	0.0105	0.0205	0.1055	0.0225	0.036	0.165	0.0365	0.056	0.22	0.055
A #3	0.0095	0.0435	0.0145	0.0245	0.088	0.027	0.047	0.1465	0.043	0.0685	0.1965	0.0575
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.086	0.3325	0.1025			fail						
A #2	0.0865	0.3075	0.0835	0.126	0.4005	0.124						
A #3	0.01	0.2625	0.077	0.1465	0.347	0.1085						

One Day curing strenght.

Date 5/6/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0005	0.0065	0.001	0.009	0.045	0.009	0.0225	0.0965	0.0215	0.0435	0.1575	0.0395
A #2	0.0095	0.0685	0.0195	0.0235	0.125	0.0395	0.0455	0.2095	0.064	0.062	0.2645	0.087
A #3	0.0075	0.085	0.027	0.021	0.161	0.0505	0.0405	0.25	0.0785	0.0635	0.3595	0.1185
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.066	0.2195	0.0575	0.095	0.281	0.085						
A #2			fail									
A #3			fail									

Two Day curing strenght.

Date 3/12/2008

Adhesive	**1.5 psi**			**2.5 psi**			**3.5 psi**			**4.5 psi**		
	top slip	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**
A #1	0.021	0.0465	0.011	0.0435	0.0985	0.028	0.0825	0.1725	0.0535	0.1305	0.2553	0.0825
A #2	0.012	0.047	0.0008	0.025	0.0955	0.0205	0.0435	0.1515	0.0365	0.0655	0.214	0.058
A #3	0.011	0.0495	0.0085	0.0265	0.1035	0.024	0.0465	0.161	0.0435	0.072	0.228	0.065
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**
A #1			**fail**									
A #2	0.0885	0.276	0.081	0.114	0.333	0.1045						
A #3	0.107	0.3055	0.0945			**fail**						

Two Day curing strenght.

Date 5/9/2008

Adhesive	**1.5 psi**			**2.5 psi**			**3.5 psi**			**4.5 psi**		
	top slip	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**
A #1	0.0055	0.033	0.01	0.0265	0.0815	0.026	0.035	0.107	0.028	0.0495	0.1675	0.058
A #2	0.0075	0.055	0.0235	0.0165	0.0955	0.0415	0.0315	0.1565	0.069	0.051	0.225	0.1015
A #3	0.0065	0.0465	0.017	0.016	0.0925	0.037	0.0295	0.1515	0.065	0.0435	0.2115	0.094
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**
A #1	0.0765	0.2605	0.074	0.097	0.3475	0.1						
A #2	0.0755	0.266	0.0845	0.1045	0.355	0.118						
A #3	0.0655	0.2445	0.079	0.0845	0.306	0.1055						

Lateral Pressure Test (Foam 4 and Adhesive A)

One Day curing strenght.

Date 3/3/2008

Type of foam **4**

Amount of glue **8 gm**

Adhesive	15	psi		25	psi		35	psi		45	psi	
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0015	0.023	0.001	0.0045	0.0535	0.0065	0.009	0.0925	0.015	0.015	0.125	0.0235
A #2	0.009	0.0325	0.008	0.018	0.064	0.015	0.03	0.1035	0.0245	0.0495	0.173	0.0445
A #3	0.0065	0.0215	0.0055	0.0155	0.0495	0.0125	0.0275	0.0845	0.022	0.04	0.121	0.033
Adhesive	**55**	**psi**		**65**	**psi**		**75**	**psi**		**85**	**psi**	
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0215	0.1805	0.0325	0.0295	0.2315	0.045						
A #2	0.063	0.212	0.055	0.0755	0.279	0.088						
A #3	0.057	0.1705	0.048	0.073	0.2175	0.065						

Lateral Pressure Test (Foam 5 and Adhesive A)

One Day curing strenght.

Date 6/5/2008

Type of foam 5

Amount of glue 8 gm

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.01	0.0415	0.0005	0.0175	0.0765	0.008	0.028	0.1225	0.013	0.0395	0.173	0.019
A #2	0.0115	0.0385	0.009	0.0255	0.0775	0.0215	0.036	0.127	0.0445			fail
A #3	0.003	0.035	0.0095	0.009	0.0685	0.0205	0.018	0.108	0.033	0.0275	0.1515	0.047

Adhesive	5.5 psi			6.5 psi			7.5 psi			8.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.051	0.226	0.0275	0.0635	0.275	0.037						
A #2												
A #3	0.0385	0.197	0.062	0.052	0.2445	0.0785						

Two Day curing strenght.

Date 6/11/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.006	0.0165	0.005	0.013	0.035	0.013	0.0215	0.056	0.0215	0.0305	0.0795	0.032
A #2	0.006	0.015	0.003	0.0115	0.0285	0.0095	0.0185	0.046	0.0165	0.028	0.067	0.0235
A #3	0.007	0.0165	0.0045	0.0135	0.031	0.0115	0.022	0.049	0.0205	0.032	0.066	0.0295

Adhesive	5.5 psi			6.5 psi			7.5 psi			8.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0415	0.1045	0.0435	0.0545	0.1365	0.062						
A #2	0.04	0.094	0.0335	0.0515	0.1185	0.0465						
A #3	0.0435	0.0885	0.0435	0.055	0.112	0.0575						

<u>Elongation Test (Foam 1 and Adhesive A)</u>

Type of foam	1	
Amount of adhesive	8	gm
Loading rate	10	lb/min

One day curing strength.

Date 4/22/2008

Adhesive	Pull (in)	Pull (lb)
A	0.6615	110
A	DNF	-
A	0.935	130

Date 5/13/2008

Adhesive	Pull (in)	Pull (lb)
A	1	170
A	0.854	150
A	1	180

Two days curing strength.

Date 5/14/2008

Adhesive	Pull (in)	Pull (lb)
A	0.789	150
A	0.91	160
A	0.9635	160

Note: DNF stands for specimen did not failed until 1" of elongation.

Elongation Test (Foam 2 and Adhesive A)

Type of foam	2	
Amount of adhesive	8	gm
Loading rate	10	lb/min

One day curing strength.

Date 2/20/2008

Adhesive	Pull (in)	Pull (lb)
A	0.379	40
A	0.3985	40
A	0.312	-

Two days curing strength.

Date 4/16/2008

Adhesive	Pull (in)	Pull (lb)
A	DNF	-
A	0.4205	90
A	0.697	120

Date 5/14/2008

Adhesive	Pull (in)	Pull (lb)
A	0.875	160
A	0.96	170
A	DNF	-

Note: DNF stands for specimen did not failed until 1" of elongation.

Elongation Test (Foam 2 and Adhesive B)

Type of foam	2	
Amount of adhesive	10	gm
Loading rate	10	lb/min

One day curing strength.

Date 2/26/2008

Adhesive	Pull (in)	Pull (lb)
B	0.368	60
B	0.4465	90
B	0.157	20

Two days curing strength.

Date 2/27/2008

Adhesive	Pull (in)	Pull (lb)
B	0.386	70
B	0.279	60
B	0.316	70

Elongation Test (Foam 2 and Adhesive C)

Type of foam	2	
Amount of adhesive	18	gm
Loading rate	10	lb/min

One day curing strength.

Date 6/3/2008

Adhesive	Pull (in)	Pull (lb)
C	0.341	60
C	0.4	60
C	0.5695	80

Date 6/19/2008

Adhesive	Pull (in)	Pull (lb)
C	0.369	50
C	0.097	10
C	0.132	20

Two days curing strength.

Date 7/2/2008

Adhesive	Pull (in)	Pull (lb)
C	0.404	60
C	0.259	20
C	0.2645	20

Elongation Test (Foam 3 and Adhesive A)

Type of foam	3	
Amount of adhesive	8	gm
Loading rate	10	lb/min

One day curing strength.

Date 4/15/2008

Adhesive	Pull (in)	Pull (lb)
A	0.3485	70
A	0.361	70
A	0.382	80

Two days curing strength.

Date 4/16/2008

Adhesive	Pull (in)	Pull (lb)
A	0.39	100
A	0.5233	150
A	0.5685	120

Adhesive	Pull (in)	Pull (lb)
A	DNF	-
A	0.421	90
A	0.697	120

Adhesive	Pull (in)	Pull (lb)
A	0.8185	150
A	0.263	70
A	DNF	-
A	DNF	-
A	0.9785	190

Note: DNF stands for specimen did not failed until 1” of elongation.

Elongation Test (Foam 4 and Adhesive A)

Type of foam	4	
Amount of adhesive	8	gm
Loading rate	10	lb/min

One day curing strength.

Date 3/5/2008

Adhesive	Pull (in)	Pull (lb)
A	0.7155	220
A	0.631	200
A	DNF	-

Date 3/4/2008

Adhesive	Pull (in)	Pull (lb)
A	DNF	-
A	0.4485	120
A	0.729	250

Two days curing strength.

Date 3/5/2008

Adhesive	Pull (in)	Pull (lb)
A	DNF	-
A	0.541	350
A	0.443	140

Note: DNF stands for specimen did not failed until 1” of elongation.

Elongation Test (Foam 5 and Adhesive A)

Type of foam	5	
Amount of adhesive	8	gm
Loading rate	10	lb/min

One day curing strength.

Date 6/6/2008

Adhesive	Pull (in)	Pull (lb)
A	0.3365	170
A	0.308	80
A	0.284	60

Two days curing strength.

Date 6/11/2008

Adhesive	Pull (in)	Pull (lb)
A	0.4355	440
A	0.405	320
A	0.347	120

Elongation and Lateral Pressure Test (Foam 1 and Adhesive A)

One Day curing strenght.

Date 6/5/2008

Type of foam 1

Amount of glue 8 gm

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0205	0.081	0.0155	0.044	0.157	0.044	0.0795	0.254	0.0785	0.1115	0.326	0.1085
A #2	0.0195	0.0778	0.007	0.042	0.153	0.0215	0.0755	0.244	0.046	0.1055	0.313	0.0705
A #3	0.037	0.0825	0.0145	0.077	0.1615	0.0335	0.129	0.2555	0.053			**fail**
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.1605	0.4195	0.1595			**fail**						
A #2	0.1485	0.3855	0.1065	**fail**								
A #3												

One Day curing strenght.

Date 6/17/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0215	0.0885	0.037	0.0545	0.1835	0.088	0.0785	0.2495	0.126	0.108	0.323	0.1695
A #2	0.026	0.0585	0.0345	0.0545	0.1205	0.0715	0.0995	0.2085	0.128			**fail**
A #3	0.028	0.0685	0.0395	0.0555	0.1375	0.078	0.0895	0.2045	0.1205	0.125	0.2855	0.173
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.1445	0.411	0.2205			**fail**						
A #2												
A #3	0.153	0.339	0.2075	0.19	0.4045	0.2525						

One Day curing strenght.

Date 6/19/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0155	0.059	0.0225	0.033	0.1185	0.04	0.0535	0.1905	0.0615	0.0785	0.2615	0.0845
A #2	0.01	0.0945	0.0173	0.0225	0.1565	0.0305	0.0405	0.236	0.0535	0.062	0.312	0.08
A #3	0.0145	0.0985	0.0265	0.113	0.2725	0.0755	**fail**					
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1			**fail**									
A #2	0.094	0.397	0.1265	**fail**								
A #3												

Two Day curing strenght.

Date 6/5/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0105	0.062	0.019	0.0335	0.1435	0.059	0.0485	0.2015	0.0855	0.0795	0.293	0.1335
A #2	0.0205	0.067	0.017	0.038	0.1235	0.0365	0.056	0.1795	0.0555	0.083	0.25	0.0815
A #3	0.021	0.075	0.008	0.04	0.1415	0.019	0.0595	0.1975	0.0325	0.087	0.2735	0.055
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.103	0.352	0.1685	0.119	0.391	0.1905						
A #2	0.1055	0.302	0.1035	0.128	0.485	0.1865						
A #3	0.114	0.328	0.0765	0.144	0.3915	0.103						

Elongation and Lateral Pressure Test (Foam 2 and Adhesive A)

One Day curing strenght.

Date 2/22/2008

Type of foam 2

Amount of glue **8** gm

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.017	0.081	0.014	0.077	0.1925	0.041	fail					
A #2	0.021	0.1035	0.0435	0.0875	0.2965	0.166			fail			
A #3	0.0345	0.1005	0.0345	0.178	0.4305	0.229			fail			
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1												
A #2												
A #3												

Two Day curing strenght.

Date 2/22/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0175	0.0575	0.035	0.062	0.104	0.035	0.098	0.169	0.062	0.1605	0.2745	0.1075
A #2	0.014	0.051	0.007	0.033	0.0965	0.025	0.056	0.1545	0.039	0.0875	0.232	0.0565
A #3	0.0085	0.033	0.007	0.019	0.0775	0.0225	0.0295	0.1195	0.0395	0.0465	0.1865	0.063
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1			fail									
A #2	0.1225	0.13	0.1235	fail								
A #3	0.066	0.2735	0.0955	0.092	0.324	0.118						

Elongation and Lateral Pressure Test (Foam 2 and Adhesive B)

One Day curing strenght.

Date 2/27/2008

Type of foam 2

Amount of glue **10** gm

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
B #1	0.006	0.0825	0.0045			**fail**						
B #2	0.0355	0.042	0.0065	0.0165	0.1025	0.019			**fail**			
B #3	0.006	0.0395	0.004	0.017	0.0895	0.014	0.033	0.153	0.025			**fail**
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
B #1												
B #2												
B #3												

Elongation and Lateral Pressure Test (Foam 3 and Adhesive A)

One Day curing strenght.
Date 6/4/2008
Type of foam **3**
Amount of glue **8 gm**

Adhesive	**1.5 psi**			**2.5 psi**			**3.5 psi**			**4.5 psi**		
	top slip	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**
A #1	0.0155	0.047	0.0135	0.023	0.082	0.0205	0.042	0.134	0.0345	0.0755	0.2275	0.0595
A #2	0.014	0.0515	0.011	0.024	0.092	0.0285	0.0465	0.156	0.047	0.073	0.226	0.069
A #3	0.009	0.038	0.0085	0.022	0.0825	0.022	0.037	0.134	0.0375	0.0535	0.1985	0.0565
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**
A #1	0.105	0.2945	0.0835			**fail**						
A #2	0.1125	0.315	0.1005			**fail**						
A #3	0.0735	0.274	0.0825	**fail**								

Two Day curing strenght.
Date 5/9/2008

Adhesive	**1.5 psi**			**2.5 psi**			**3.5 psi**			**4.5 psi**		
	top slip	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**
A #1	0.0095	0.0335	0.007	0.023	0.0715	0.0185	0.0365	0.1195	0.0315	0.0555	0.1945	0.0535
A #2	0.0115	0.0405	0.0135	0.023	0.0795	0.026	0.037	0.134	0.042	0.058	0.2085	0.0645
A #3	0.008	0.035	0.01	0.0185	0.0715	0.022	0.0315	0.1155	0.037	0.0485	0.1785	0.058
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**	**top slip**	**centre deflection**	**bottom slip**
A #1	0.074	0.2605	0.074	0.097	0.3475	0.1						
A #2	0.0755	0.266	0.0845	0.1045	0.355	0.118						
A #3	0.0655	0.2445	0.079	0.0845	0.306	0.1055						

Elongation and Lateral Pressure Test (Foam 4 and Adhesive A)

One Day curing strenght.

Date 3/7/2008

Type of foam crosslink #1

Amount of glue 8 gm

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0045	0.018	0.0075	0.0125	0.0445	0.016	0.022	0.0715	0.0255	0.0335	0.106	0.037
A #2	0.0035	0.0165	0.0055	0.0095	0.038	0.013	0.0155	0.0605	0.02	0.022	0.087	0.0285
A #3												

Adhesive	5.5 psi			6.5 psi			7.5 psi			8.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.047	0.142	0.0495	0.075	0.203	0.0725						
A #2	0.0295	0.1175	0.0375	0.0385	0.1505	0.047						
A #3												

Elongation and Lateral Pressure Test (Foam 5 and Adhesive A)

One Day curing strenght.

Date 6/6/2008

Type of foam 5

Amount of glue 8 gm

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.005	0.043	0.009	0.014	0.082	0.018	0.0215	0.1255	0.0275	0.031	0.171	0.038
A #2	0.005	0.0485	0.0165	0.0165	0.0915	0.0295	0.0285	0.138	0.0445	0.041	0.19	0.0595
A #3	0.005	0.045	0.017	0.0075	0.0875	0.04	0.0095	0.1405	0.0705	0.0125	0.213	0.121
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.0445	0.223	0.05			fail						
A #2	0.0565	0.243	0.0755	0.071	0.2865	0.089						
A #3			fail									

Two Day curing strenght.

Date 6/11/2008

Adhesive	1.5 psi			2.5 psi			3.5 psi			4.5 psi		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.007	0.0165	0.0055	0.016	0.032	0.014	0.0325	0.054	0.027	0.0535	0.0785	0.0435
A #2	0.004	0.014	0	0.0165	0.0375	0.0095	0.031	0.066	0.022	0.047	0.0945	0.0355
A #3	0.0095	0.027	0.0105	0.0165	0.043	0.028	0.0305	0.069	0.0435	0.046	0.099	0.0585
Adhesive	**5.5 psi**			**6.5 psi**			**7.5 psi**			**8.5 psi**		
	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip	top slip	centre deflection	bottom slip
A #1	0.077	0.104	0.06	0.103	0.1335	0.0835						
A #2	0.07	0.1345	0.0525	0.0845	0.159	0.0645						
A #3			fail									

Memory test for #1.0 Foam

Start Date	5/28/2008
Release Date	5/29/2008
Foam Used	# 1
Duration under load (days)	1
Original Height (in.)	3
Crushed	50%

Time (Hrs)	**Height of foam after release (in.)**					**Percentage Height Loss**	**Percentage Height Gain**
	Sample 1	**Sample 2**	**Sample 3**	**Average**	**SD**		
0.00	2.0	1.9	1.9	1.9	0.1	35.44	64.56
0.17	2.0	1.9	1.9	1.9	0.1	35.44	64.56
0.33	2.1	2.0	1.9	2.0	0.1	33.33	66.67
0.50	2.1	2.0	1.9	2.0	0.1	33.33	66.67
0.67	2.1	2.0	1.9	2.0	0.1	33.33	66.67
0.83	2.1	2.0	1.9	2.0	0.1	33.33	66.67
1.00	2.1	2.0	1.9	2.0	0.1	33.33	66.67
2.00	2.1	2.0	1.9	2.0	0.1	33.33	66.67
3.00	2.1	2.1	2.0	2.0	0.0	32.00	68.00
4.00	2.2	2.1	2.1	2.1	0.1	29.22	70.78
24.00	2.3	2.3	2.2	2.3	0.1	25.00	75.00

Memory test for #1.0 Foam

Start Date	5/28/2008
Release Date	5/29/2008
Foam Used	# 1
Duration under load (days)	7
Original Height (in.)	3
Crushed	50%

Time (Hrs)	Height of foam after release (in.)					Percentage Height Loss	Percentage Height Gain
	Sample 1	Sample 2	Sample 3	Average	SD		
0.00	1.5	1.5	1.5	1.5	0.0	50.00	50.00
0.17	1.7	1.6	1.6	1.6	0.0	45.14	54.86
0.33	1.7	1.6	1.6	1.6	0.0	45.14	54.86
0.50	1.7	1.6	1.6	1.6	0.0	45.14	54.86
0.67	1.7	1.7	1.7	1.7	0.0	43.75	56.25
0.83	1.8	1.7	1.7	1.7	0.0	43.06	56.94
1.00	1.8	1.8	1.8	1.8	0.0	41.67	58.33
2.00	1.8	1.8	1.8	1.8	0.0	41.67	58.33
3.00	1.8	1.8	1.8	1.8	0.0	41.67	58.33
4.00	1.8	1.8	1.8	1.8	0.0	41.67	58.33
5.00	1.8	1.8	1.8	1.8	0.0	41.67	58.33
6.00	1.8	1.8	1.8	1.8	0.0	40.28	59.72
24.00	1.9	1.8	1.8	1.8	0.0	38.89	61.11

Memory test for #1.2 Foam

Start Date	11/3/2008
Release Date	11/4/2008
Foam Used	#1.2
Duration under load (days)	1
Original Height (in.)	3
Crushed	50%

Time (Hrs)	Height of foam after release (in.)					Percentage Height Loss	Percentage Height Gain
	Sample 1	Sample 2	Sample 3	Average	SD		
0.00	2.0	1.9	2.0	2.0	0.1	34.67	65.33
0.17	2.1	2.0	2.1	2.1	0.1	30.44	69.56
0.33	2.1	2.0	2.1	2.1	0.1	30.44	69.56
0.50	2.1	2.0	2.1	2.1	0.1	30.44	69.56
0.67	2.1	2.0	2.1	2.1	0.1	30.44	69.56
0.83	2.1	2.0	2.1	2.1	0.1	30.44	69.56
1.00	2.1	2.0	2.1	2.1	0.1	30.44	69.56
2.00	2.1	2.0	2.1	2.1	0.1	30.44	69.56
3.00	2.1	2.0	2.1	2.1	0.1	30.44	69.56
4.00	2.1	2.0	2.1	2.1	0.1	30.44	69.56
24.00	2.3	2.3	2.3	2.3	0.0	25.00	75.00

Memory test for #1.2 Foam

Start Date	10/29/2008
Release Date	11/4/2008
Foam Used	#1.2
Duration under load (days)	7
Original Height (in.)	3
Crushed	50%

Time (Hrs)	**Height of foam after release (in.)**					**Percentage Height Loss**	**Percentage Height Gain**
	Sample 1	**Sample 2**	**Sample 3**	**Average**	**SD**		
0.00	1.5	1.6	1.6	1.6	0.1	47.11	52.89
0.17	1.6	1.8	1.6	1.7	0.1	44.33	55.67
0.33	1.6	1.8	1.6	1.7	0.1	44.33	55.67
0.50	1.6	1.8	1.6	1.7	0.1	44.33	55.67
0.67	1.6	1.8	1.6	1.7	0.1	44.33	55.67
0.83	1.6	1.8	1.6	1.7	0.1	44.33	55.67
1.00	1.6	1.8	1.6	1.7	0.1	44.33	55.67
2.00	1.6	1.8	1.6	1.7	0.1	44.33	55.67
3.00	1.6	1.8	1.6	1.7	0.1	44.33	55.67
4.00	1.6	1.8	1.6	1.7	0.1	44.33	55.67
24.00	1.8	1.9	1.8	1.8	0.1	40.22	59.78

Memory test for #1.7 Foam

Start Date	7/1/2008
Release Date	7/2/2008
Foam Used	# 1.7
Duration under load (days)	1
Original Height (in.)	3
Crushed	50%

Time (Hrs)	Height of foam after release (in.)					Percentage Height Loss	Percentage Height Gain
	Sample 1	Sample 2	Sample 3	Average	SD		
0.00	1.9	1.9	1.9	1.9	0.0	36.81	63.19
0.17	2.0	2.0	2.0	2.0	0.0	33.33	66.67
0.33	2.0	2.0	2.0	2.0	0.0	33.33	66.67
0.50	2.0	2.0	2.0	2.0	0.0	33.33	66.67
0.67	2.0	2.1	2.1	2.1	0.1	30.56	69.44
0.83	2.1	2.1	2.1	2.1	0.0	29.86	70.14
1.00	2.1	2.1	2.1	2.1	0.0	29.86	70.14
2.00	2.1	2.1	2.1	2.1	0.0	29.86	70.14
3.00	2.1	2.1	2.1	2.1	0.0	29.17	70.83
4.00	2.1	2.2	2.2	2.2	0.0	27.78	72.22
24.00	2.8	2.8	2.8	2.8	0.0	6.94	93.06

Memory test for #1.7 Foam

Start Date	5/29/2008
Release Date	6/9/2008
Foam Used	# 1.7
Duration under load (days)	7
Original Height (in.)	3
Crushed	50%

Time (Hrs)	**Height of foam after release (in.)**					**Percentage Height Loss**	**Percentage Height Gain**
	Sample 1	**Sample 2**	**Sample 3**	**Average**	**SD**		
0.00	1.6	1.6	1.5	1.5	0.0	48.67	51.33
0.17	1.6	1.6	1.6	1.6	0.0	46.67	53.33
0.33	1.6	1.6	1.6	1.6	0.0	46.67	53.33
0.50	1.6	1.6	1.6	1.6	0.0	46.67	53.33
0.67	1.6	1.6	1.6	1.6	0.0	46.67	53.33
0.83	1.6	1.6	1.6	1.6	0.0	46.67	53.33
1.00	1.6	1.6	1.6	1.6	0.0	46.67	53.33
2.00	1.6	1.6	1.6	1.6	0.0	46.00	54.00
3.00	1.7	1.7	1.6	1.7	0.0	44.44	55.56
4.00	1.7	1.7	1.6	1.7	0.0	44.44	55.56
5.00	1.7	1.7	1.6	1.7	0.0	44.44	55.56
6.00	1.7	1.7	1.6	1.7	0.0	44.44	55.56
24.00	1.8	1.8	1.8	1.8	0.0	41.00	59.00

Memory test for CL#1 Foam

Start Date	3/9/2009
Release Date	3/10/2009
Foam Used	CL#1
Duration under load (days)	1
Original Height (in.)	3
Crushed	50%

Time (Hrs)	**Height of foam after release (in.)**					**Percentage Height Loss**	**Percentage Height Gain**
	Sample 1	**Sample 2**	**Sample 3**	**Average**	**SD**		
0.00	2.3	2.3	2.1	2.2	0.1	26.39	73.61
0.17	2.4	2.4	2.4	2.4	0.0	20.83	79.17
0.33	2.4	2.4	2.4	2.4	0.0	20.83	79.17
0.50	2.4	2.4	2.4	2.4	0.0	20.14	79.86
0.67	2.4	2.4	2.4	2.4	0.0	18.77	81.23
0.83	2.4	2.4	2.4	2.4	0.0	18.77	81.23
1.00	2.5	2.5	2.5	2.5	0.0	16.67	83.33
2.00	2.5	2.5	2.5	2.5	0.0	16.67	83.33
3.00	2.5	2.5	2.5	2.5	0.0	16.67	83.33
4.00	2.5	2.5	2.5	2.5	0.0	16.67	83.33
24.00	2.6	2.6	2.6	2.6	0.0	12.50	87.50

Memory test for CL#1 Foam

Start Date	3/3/2009
Release Date	3/10/2009
Foam Used	CL#1
Duration under load (days)	7
Original Height (in.)	3
Crushed	50%

Time	**Height of foam after release (in.)**					**Percentage Height Loss**	**Percentage Height Gain**
(Hrs.)	**Sample 1**	**Sample 2**	**Sample 3**	**Average**	**SD**		
0.00	1.9	1.9	1.8	1.9	0.0	38.19	61.81
0.17	1.9	1.9	1.9	1.9	0.0	36.12	63.88
0.33	1.9	1.9	1.9	1.9	0.0	36.12	63.88
0.50	1.9	1.9	1.9	1.9	0.0	36.12	63.88
0.67	1.9	1.9	1.9	1.9	0.0	36.12	63.88
0.83	2.0	2.0	2.0	2.0	0.0	33.33	66.67
1.00	2.0	2.0	2.0	2.0	0.0	33.33	66.67
2.00	2.0	2.0	2.0	2.0	0.0	33.33	66.67
3.00	2.0	2.0	2.0	2.0	0.0	33.33	66.67
4.00	2.0	2.0	2.0	2.0	0.0	33.33	66.67
24.00	2.1	2.1	2.1	2.1	0.0	29.17	70.83

Memory test for CL#2 Foam

Start Date	11/3/2008
Release Date	11/4/2008
Foam Used	CL#2
Duration under load (days)	1
Original Height (in.)	3
Crushed	50%

Time (Hrs)	**Height of foam after release (in.)**					**Percentage Height Loss**	**Percentage Height Gain**
	Sample 1	**Sample 2**	**Sample 3**	**Average**	**SD**		
0.00	2.6	2.6	2.6	2.6	0.0	13.11	86.89
0.17	2.9	2.9	2.8	2.8	0.1	5.67	94.33
0.33	2.9	2.9	2.8	2.8	0.1	5.67	94.33
0.50	2.9	2.9	2.8	2.8	0.1	5.67	94.33
0.67	2.9	2.9	2.8	2.8	0.1	5.67	94.33
0.83	2.9	2.9	2.8	2.8	0.1	5.67	94.33
1.00	2.9	2.9	2.8	2.9	0.1	4.89	95.11
2.00	2.9	2.9	2.8	2.9	0.1	4.89	95.11
3.00	2.9	2.9	2.8	2.9	0.1	4.11	95.89
4.00	2.9	2.9	2.8	2.9	0.1	4.11	95.89
24.00	2.9	3.0	2.8	2.9	0.1	2.78	97.22

Memory test for CL#2 Foam

Start Date	4/8/2009
Release Date	4/15/2009
Foam Used	CL#2
Duration under load (days)	7
Original Height (in.)	3
Crushed	50%

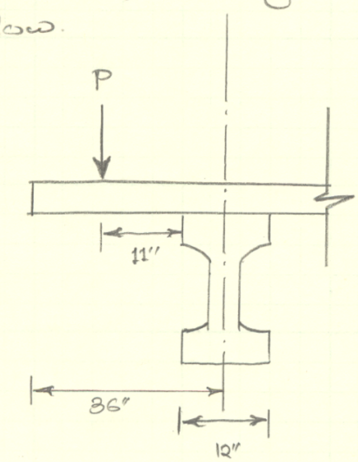
Time (Hrs)	**Height of foam after release (in.)**					**Percentage Height Loss**	**Percentage Height Gain**
	Sample 1	**Sample 2**	**Sample 3**	**Average**	**SD**		
0.00	2.0	2.0	2.1	2.0	0.1	31.94	68.06
0.17	2.0	2.3	2.3	2.2	0.1	27.78	72.22
0.33	2.0	2.3	2.3	2.2	0.1	27.78	72.22
0.50	2.3	2.3	2.3	2.3	0.0	25.00	75.00
0.67	2.3	2.3	2.3	2.3	0.0	25.00	75.00
0.83	2.3	2.3	2.3	2.3	0.0	25.00	75.00
1.00	2.3	2.3	2.3	2.3	0.0	25.00	75.00
2.00	2.4	2.4	2.4	2.4	0.0	20.72	79.28
3.00	2.4	2.4	2.4	2.4	0.0	20.72	79.28
4.00	2.4	2.4	2.4	2.4	0.0	20.72	79.28
24.00	2.5	2.5	2.5	2.5	0.0	16.67	83.33

APPPENDIX B

ANALYSIS OF CAST IN PLACE BRIDGE DECK OVERHANG

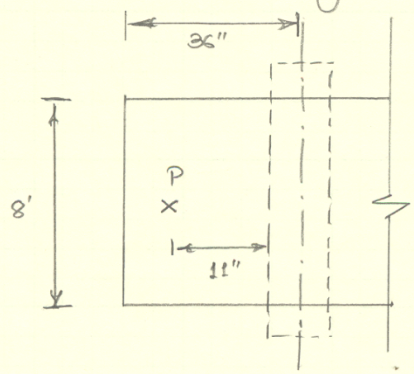
* **Analysis of Cast-in-place bridge deck overhang, designed as per TxDOT specifications.**

For static loading of CIP bridge deck overhang 11" of leverarm was used. The reason for using 11" of leverarm was to maximize the overhang length, this was done by using 12" of flange width for bridge girder (i.e. the minimum flange width of bridge girder in use) and using railtype which is having least width. The line diagram of loading on overhang is shown in figure below.

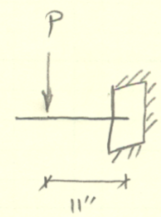


Here, we will analyze bridge deck overhang for calculating 'P' using following different cases.

Case 1: 'P' considering only cantilever action w.r.t moment carrying capacity.



a) Actual loading of bridge deck over hang



b) Modeled as simple cantilever.

This case considers that bridge deck will act as cantilever. This is the most conservative approach to analyze the overhang as it neglects the contribution of longitudinal slab. Sometimes contribution of longitudinal salb is taken into consideration as bridge decks are buit as a contineous slab in real life.

Using strip method to investigate. considering 2' wide beam.

$$M_{capacity\ cantilever} = A_s \cdot f_y \cdot (d - a/2)$$

where, $a = \dfrac{A_s \cdot f_y}{0.85 \times f_c' \times b}$

here,

$A_s = (0.31\ in^2) \times 4 \quad = 1.24\ in^2$

$f_y = 60\ ksi$

$f_c' = 3.6$ ksi

$b = 24$ in , $d = 8 - 2 = 6$ in .

$$\therefore a = \frac{1.24 \times 60}{0.85 \times 3.6 \times 24} = 1.013 \text{ in}$$

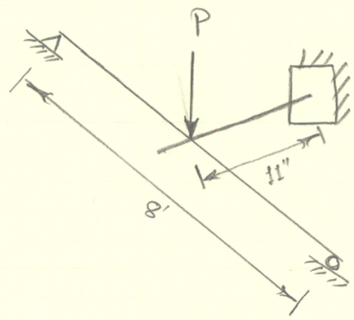
$$\therefore M_{capacity\ cantilever} = 1.24 \times 60 \times \left(6 - \frac{1.013}{2}\right) = 408.72 \text{ k.in}$$

where, $M_{capacity\ cantilever} = P_{capacity\ cantilever} \times$ lever arm .

$$\therefore P = \frac{408.72}{11}$$

$$\therefore \underline{\underline{P = 37.16 \text{ kips}}}$$

<u>Case 2</u>: Calculating 'P' considering the contribution of lateral support from adjucent panels (slabs).



Bridge deck overhang modeled as proped-cantilever supported by pin-pin beam.

Assuming the width of pin-pin beam as 1'.

we know,

$$M_{capacity\ pin\text{-}pin} = A_s f_y (d - a/2)$$

where, $a = \dfrac{A_s \cdot f_y}{0.85 \times f_c' \times b}$

here, $A_s = 0.20 \times \dfrac{12}{9} = 0.27\ in^2$

$f_y = 60\ ksi$

$f_c' = 3.6\ ksi$

$b = 12\ in.$

$\therefore d = 8 - 2 - \dfrac{5}{8} = 5.375\ in$

$\therefore a = \dfrac{0.27 \times 60}{0.85 \times 3.6 \times 12} = 0.44\ in$

$$M_{capacity\ pin\text{-}pin} = 0.27 \times 60 \times \left(5.375 - \frac{0.44}{2}\right) = 83.51\ k.in$$

Also,

$$M_{capacity\ pin\text{-}pin} = \frac{P_{capacity\ pin\text{-}pin} \times l}{4}$$

where, $l = 8'$

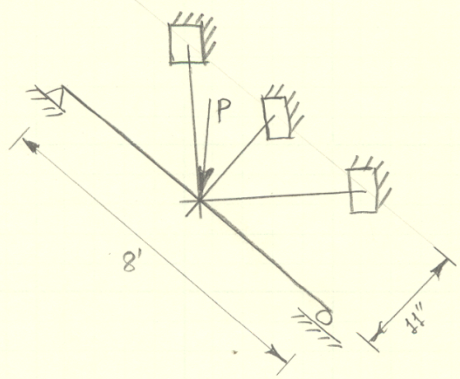
$$\therefore \quad P_{capacity\ pin\text{-}pin} = \frac{4}{8 \times 12} \times 83.51 = 3.48\ k.$$

$$\therefore P_{total} = P_{capacity\ cantilever} + P_{capacity\ pin\text{-}pin}$$

$$= 37.16 + 3.48$$

$$\therefore \underline{\underline{P = 40.64\ k}}.$$

Case 3 : Calculating 'P' considering two more struts supporting load 'P' in addition to the case 2.



As for this case leverarm being too small as compared to the length of longitudinal slab, practically thinking true cantilever action is not possible. It is too conservative to consider cantilever action alone. To more closely model the bridge deck we can consider load transfer path with three cantilevers one perpendicular to the loading point and other two at 45° inclination conversing at point of loading.

∴ from previous calculations we know,

$P_{cantilever} = 37.16 \text{ k}$

$P_{pin-pin} = 3.48 \text{ k}$

Now, calculating $P_{inclined\ cantilevers} = \dfrac{37.16 \times \cos 45}{\sqrt{2} \times 11''} = 1.69 \text{ k}$

$\therefore P_{total} = P_{cantilever} + 2 \times (P_{inclined\ cantilevers}) + P_{pin-pin}$

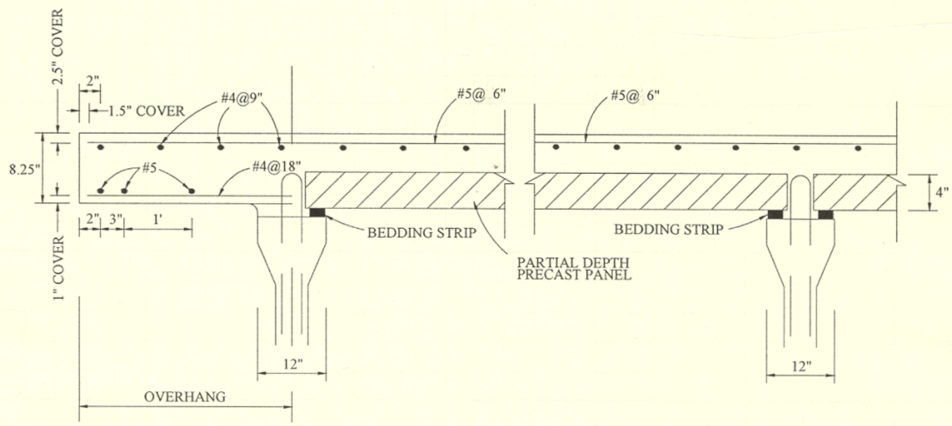
$= 37.16 + 2 \times (1.69) + 3.48$

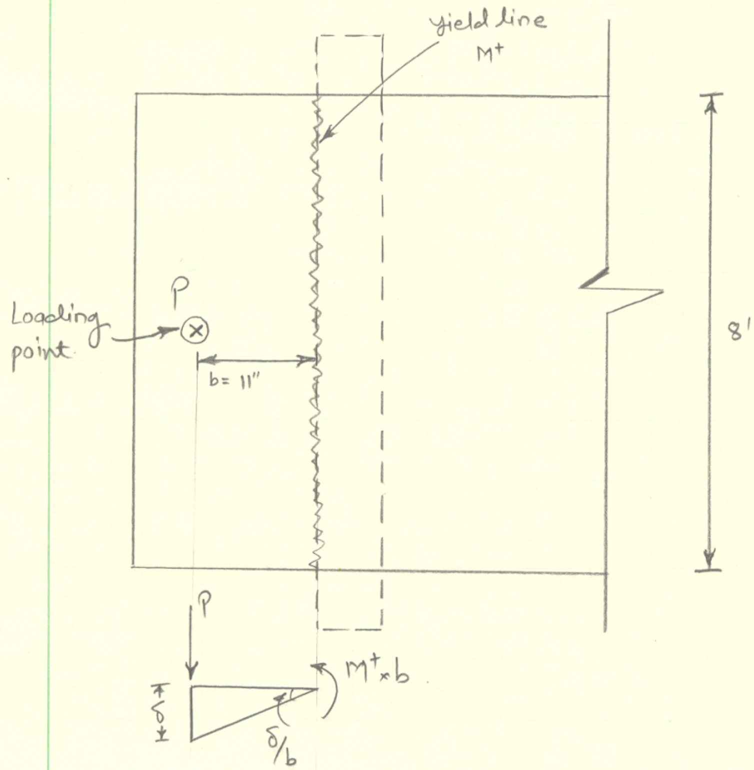
$\therefore \underline{P = 44.02 \text{ k}}$

<u>Case 4</u>: Using yield line theory to calculate 'P'.

<u>Data</u>: 1) TxDOT standard cast-in-place bridge deck overhang.

2) TxDOT type A girder (flange width = 12").





Top reinforcement #5 @ 6" $\therefore$ $A_s = 0.62$ in²/ft.

$f_c' = 3600$ psi

$f_y = 60$ ksi.

cover = 1"

Calculating capacity per feet.

$$\omega = \frac{A_s}{bd} \times \frac{f_y}{f_c'}$$

where, $d = 8.25 - \frac{3}{4} - 1 = 6.5"$

$$\therefore \omega = \frac{0.62}{12 \times 6.5} \times \frac{60}{3.6} = 0.1324 \text{ k/ft.}$$

$$\therefore M_u = \omega b d^2 f_c' (1 - 0.59\omega)$$

$$= 0.1324 \times 12 \times 6.5^2 \times 3.6 (1 - 0.59 \times 0.1324)$$

$$= 222.78 \text{ k-in/ft.}$$

$$\therefore M_u = 18.56 \text{ k-ft/ft}$$

Now,

$$W_{ext} = W_{int}$$

$$P \times \cancel{\delta} = M_u \times \cancel{b} \times \frac{\cancel{\delta}}{\cancel{b}}$$

$$\therefore P = M_u$$

$$\therefore P = 18.56 \text{ k ft/ft.}$$

$$\therefore P = 18.56 \times \left(\frac{96}{12}\right)$$

$$\therefore \underline{\underline{P = 148.48 \text{ k}}}$$

APPPENDIX C

EFFECT OF E AND I ON CAMBER OF STANDARD TxDOT BRIDGE GIRDER

Effect of E and I on camber of TxDOT standard bridge girders

We know midspan deflection, $\Delta = (5/384)*(wl^4/EI)$

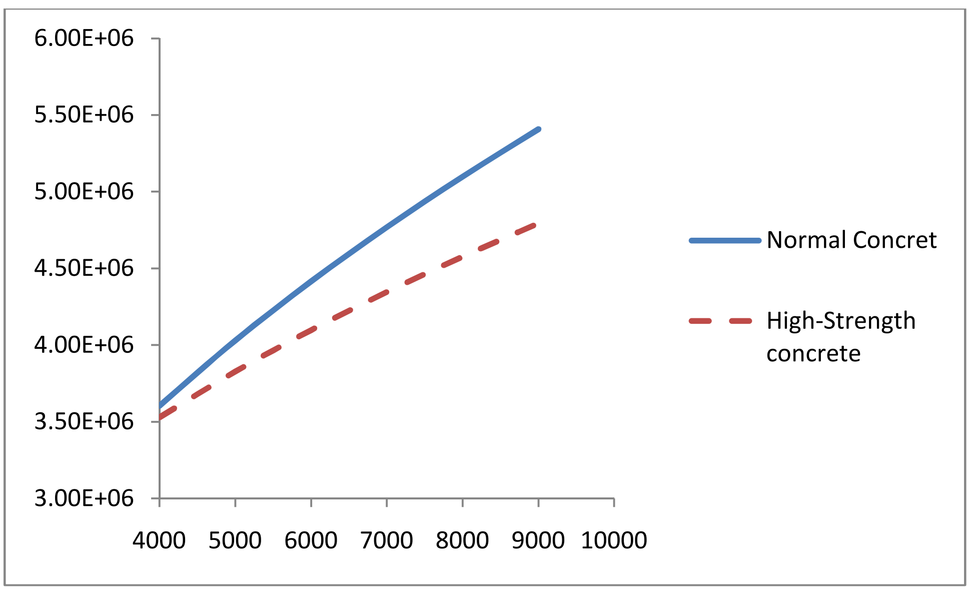
This means that deflection/camber is inversely proportional to E and I. In this chapter we will study how E and I will vary as per change in strength of concrete and area of steel respectively.

Effect of change in concrete strength on modulus of concrete E_C

Table1: Effect of change in concrete strength on modulus of concrete

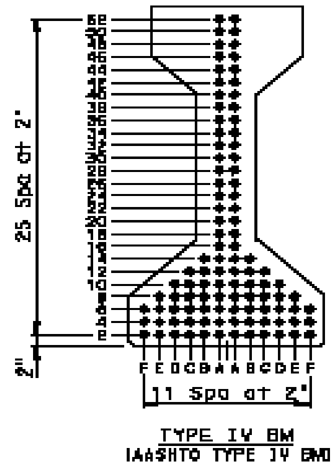
F'_C	Modulus of Elasticity of Concrete (E_C)	
	Normal Concrete	High strength concrete
4000	3.60E+06	3.53E+06
5000	4.03E+06	3.83E+06
6000	4.42E+06	4.10E+06
7000	4.77E+06	4.35E+06
8000	5.10E+06	4.58E+06
9000	5.41E+06	4.79E+06

Fig.1: Effect of change in concrete strength on modulus of concrete



We can observe that as the strength of concrete increases from 4000psi to 9000psi there is drastic chance in modules of elasticity (E). In case of normal concrete percentage increase in E is 49.99% while that for high strength concrete it is 35.83%.

Effect of change in area of steel on I of TxDOT standard type IV bridge girder



Layers	No. of strands	Concrete			Strand			Y' Concrete	Y' Steel	Y'(tr) Transformed	I Concrete	I Transformed
		Area	Y(top)	A*Y(top)	Area	Y(top)	A*Y(top)					
L-1	12	788.40	29.25	23060.70	1.84	52.00	95.47	0.05	22.70	29.30	82602.00	83550.04
L-2	24	788.40	29.25	23060.70	3.67	51.00	187.27	0.10	21.65	29.35	82602.00	84331.03
L-3	36	788.40	29.25	23060.70	5.51	50.00	275.40	0.14	20.61	29.39	82602.00	84957.08
L-4	46	788.40	29.25	23060.70	7.04	49.13	345.78	0.18	19.70	29.43	82602.00	85358.91
L-5	54	788.40	29.25	23060.70	8.26	48.37	399.63	0.20	18.92	29.45	82602.00	85591.05
L-6	60	788.40	29.25	23060.70	9.18	47.73	438.16	0.21	18.27	29.46	82602.00	85700.98
L-7	64	788.40	29.25	23060.70	9.79	47.50	465.12	0.22	18.03	29.47	82602.00	85823.34
L-8	66	788.40	29.25	23060.70	10.10	47.32	477.84	0.23	17.84	29.48	82602.00	85857.55

Fig.2: Change in Y' (tr) with change in area of steel for type IV beam

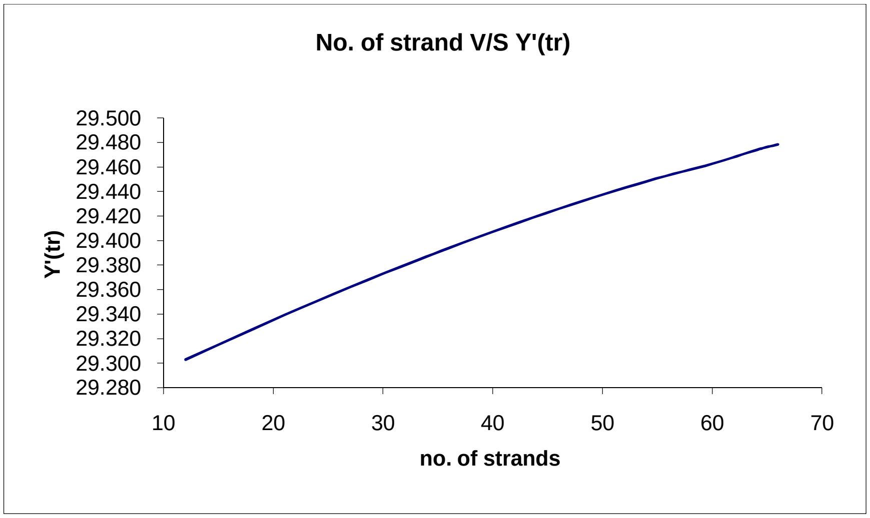
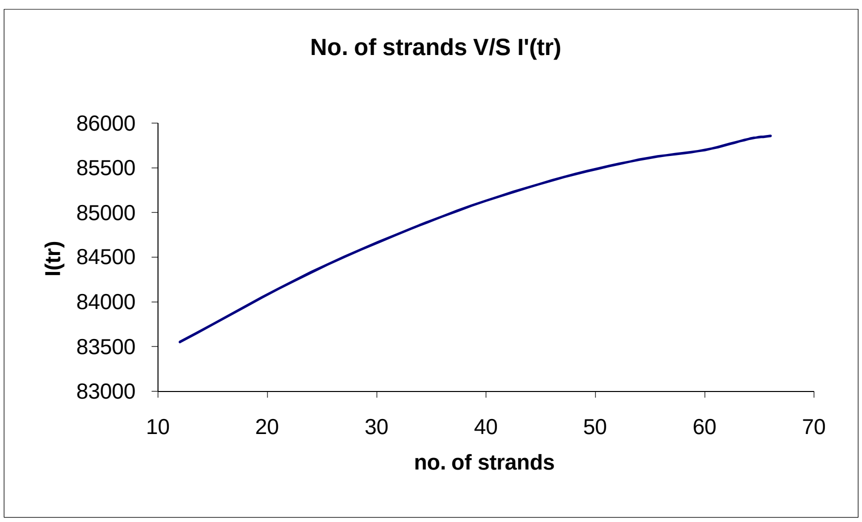
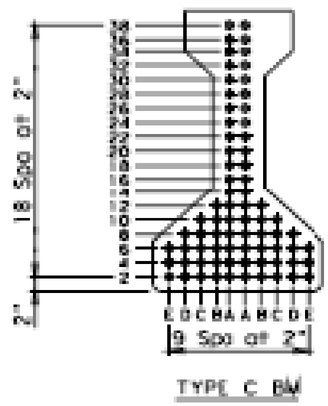


Fig.3: Change in I (tr) with change in area of steel for type IV beam



Here, we can observe that after increasing the steel in Type IV beam layer there is increase in $I_{tr.}$ In the case of Type IV BM it is 2.76%.

<u>Effect of change in area of steel on I of TxDOT standard type C bridge girder</u>



Layers	No. of	Concrete			Strand			Y'	Y'	Y'	I	I
	strands	Area	Y(top)	A*Y(top)	Area	Y(top)	A*Y(top)	Concrete	Steel	Transformed	Concrete	Transformed
L-1	10	494.90	22.91	11338.16	1.53	38.00	58.14	0.05	15.04	22.96	82602.00	82949.32
L-2	20	494.90	22.91	11338.16	3.06	37.00	113.22	0.09	14.00	23.00	82602.00	83205.76
L-3	30	494.90	22.91	11338.16	4.59	36.00	165.24	0.12	12.97	23.03	82602.00	83381.26
L-4	38	494.90	22.91	11338.16	5.81	35.16	204.42	0.14	12.11	23.05	82602.00	83464.33
L-5	44	494.90	22.91	11338.16	6.73	34.45	231.92	0.15	11.39	23.06	82602.00	83486.48
L-6	48	494.90	22.91	11338.16	7.34	33.92	249.11	0.16	10.85	23.07	82602.00	83479.22
L-7	50	494.90	22.91	11338.16	7.65	33.60	257.04	0.16	10.53	23.07	82602.00	83462.90

Fig.5: Change in Y' (tr) with change in area of steel for type C beam

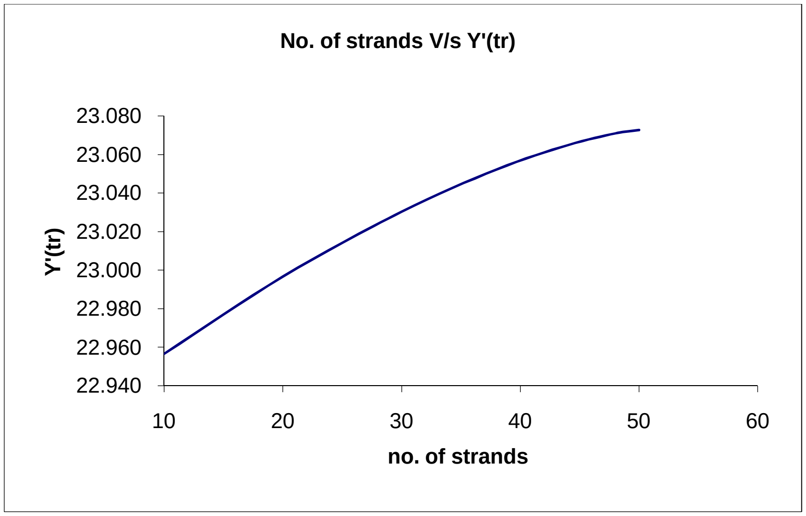
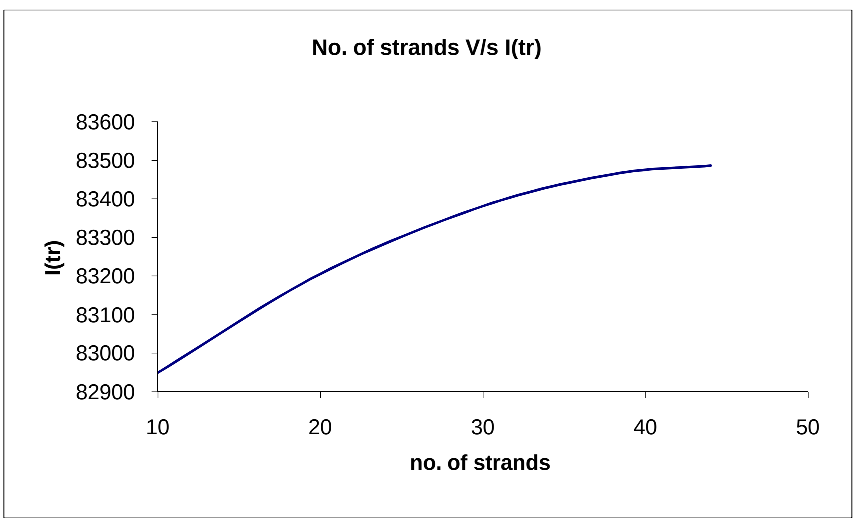


Fig.3: Change in I (tr) with change in area of steel for type C beam



From graph we can observe that after increasing the steel in Type C beam layer wise there is increase in $I_{tr.}$ In the case of Type C BM it is about 0.62%.

APPPENDIX D

DATA FOR BRIDGE OVERHANG DECK TESTING

Data of surface strain for Cast-In-Place bridge deck overhang center testing

	Load														
Demac Points	**0**	**8**	**16**	**24**	**32**	**40**	**48**	**56**	**64**	**72**	**80**	**88**	**96**	**104**	**0**
1	80	76	77	82	80	80	80	72	60	58	66	64	58	51	38
2	87	-294	-292	-290	-289	-286	-275	-235	-28	91	191	277	370	427	-52
3	-144	-151	-148	-148	-150	-146	-146	-145	-144	-145	-153	-160	-149	-149	-166
4	63	69	71	75	78	83	98	199	249	308	377	424	478	516	214
5	8	5	1	5	12	14	12	1	-4	-7	-6	-9	-10	-15	-19
6	-105	-103	-105	-95	-100	-91	-69	44	92	149	217	276	336	374	50
7	62	68	73	74	85	87	105	115	355	461	574	669	756	825	312
8	74	78	76	78	80	82	81	72	70	73	74	69	70	70	72
9	130	-114	-113	-112	-109	-105	-101	-78	145	269	384	465	576	616	116
10	-28	-22	-19	-13	16	13	20	33	24	21	19	18	19	17	-9
11	-60	-56	-56	-49	-52	-37	-20	183	260	345	425	503	618	645	191
12	-4	-4	-1	5	11	21	38	231	307	380	464	532	644	671	228
13	-375	-371	-369	-372	-372	-368	-364	-370	-145	-380	-386	-382	-389	-384	-392
14	32	36	40	49	52	64	79	131	366	477	600	681	804	841	295
15	-27	-17	-14	-9	0	8	28	77	298	413	530	612	732	771	243
16	31	33	34	38	34	43	47	138	188	237	301	346	413	426	161
17	235	239	242	247	250	254	263	347	386	411	443	464	51	708	351
18	-40	-40	-37	-30	-28	-24	-12	75	101	123	150	180	221	239	70

19	-183	-191	-183	-169	-165	-166	-159	-71	-33	22	92	142	200	225	-48
20	0	2	5	7	21	23	34	75	290	401	507	594	696	744	246
21	-166	-182	-174	-160	-164	-165	-164	-164	-167	-466	-163	-162	-164	-162	-168
22	494	493	488	487	487	488	487	489	490	485	486	474	483	472	492
23	124	125	125	128	128	127	125	129	140	161	173	181	200	204	173
24	-93	-93	-96	-92	-92	0	-92	-96	-95	-94	-95	-91	-93	-90	-89
25	-142	-144	-149	-148	-148	-147	-148	-147	-147	-150	-149	-146	-148	-146	-147
26	1035	1016	1016	1016	1016	1015	1033	1014	1033	1015	1026	1044	1021	1037	1045
27	238	233	231	227	227	231	239	250	271	271	278	288	291	265	237
28	-65	-64	-66	-66	-66	-65	-67	-59	-54	-60	-63	-62	-65	-66	-67
29	-440	-401	-436	-441	-441	-448	-456	-466	-467	-524	-514	-520	-505	-524	-460
30	-121	-123	-125	-120	-120	-120	-127	-134	191	-152	-161	-174	-170	-182	-129
31	226	236	209	208	208	199	198	214	218	234	227	212	206	184	203
32	108	122	120	105	105	107	114	122	127	120	130	128	134	136	129
33	-14	-16	53	-2	-2	-5	-4	-10	-2	-2	-5	-8	-4	1	10
34	25	21	21	20	20	18	29	35	43	42	42	42	40	42	37
35	-221	-237	-235	-229	-229	-229	-229	-224	-212	-212	-212	-224	-216	-224	-214
36	-368	-384	-387	-380	-384	-372	-383	-378	-375	-379	-385	-400	-397	-404	-380
37	8	4	8	16	12	11	12	-4	-7	-10	-20	-26	-45	-38	-9
38	-53	-79	-54	-55	-52	-47	-59	-58	-62	-70	-70	-70	-66	-70	-61
39	68	69	68	68	67	68	61	61	60	61	56	51	46	49	90
40	-185	-187	-192	-189	-189	-191	-190	-195	-143	-197	-200	-207	-206	-213	-190
41	-455	-471	-457	-357	-335	-330	-316	-324	-318	-329	-339	-353	-331	-356	-319
42	416	413	410	426	420	421	409	410	399	394	394	402	393	398	400

43	-390	-403	-413	-390	-393	-387	-389	-386	403	-395	-397	-385	-395	-386	-390
44	1593	1579	1606	1569	1571	1570	1570	1578	1612	1611	1609	1572	1602	1560	1611
45	82	90	351	65	63	68	67	68	70	73	77	75	75	70	73
46	-248	-255	-241	-250	-248	-255	-245	-241	-255	-258	-255	-262	-273	-262	-258
47	70	54	79	67	66	67	66	67	63	31	63	58	53	55	61
48	-6	-2	-2	0	4	8	12	8	9	7	9	10	15	17	7
49	339	339	341	338	341	346	362	392	603	715	835	896	-1002	1039	572
50	302	300	300	300	303	305	316	406	449	511	579	618	681	697	437
51	130	131	132	131	135	139	153	224	250	271	301	326	367	380	228

<u>**Data of deflection for Cast-In-Place bridge deck overhang center testing**</u>

	0	8		16		24		32		40		48		56	
Deflection		**Before**	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**
1	-20	-50	-75	-50	-75	-85	-90	975	975	970	975	975	980	1010	1040
2	0	5	-10	5	-10	-10	-25	-5	-15	-15	-40	-45	-60	-120	-250
3	0	0	0	0	0	0	-5	0	-20	-15	-35	-30	-40	-90	-110
4	0	0	-15	0	-15	-15	-20	0	5	5	-20	-20	-35	-160	-250
5	0	5	-25	5	-25	-35	-45	0	-20	-20	-40	-40	-50	-140	-155
6	0	0	0	0	0	0	0	0	0	0	-15	-15	-25	-65	-130

	64		72		80		88		96		104		0
Deflection	**Before**	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**	
1	1130	1175	1215	1290	1395	1465	1570	1605	1675		1690	1695	1090
2	-250	-290	-290	-405	-375	-405	-425	-440	-490		-480	-540	-110
3	-195	-235	-280	-405	-435	-405	-590	-625	-695		-675	-680	-185
4	-240	-280	-285	-405	-375	-405	-405	-420	-430		-435	-505	-240
5	-250	-310	-310	-410	-370	-410	-480	-515	-560		-530	-530	-190
6	-130	-130	-130	-130	-130	-130	-130	-130	-130		-130	-130	-130

Data of surface strain for Precast bridge deck overhang center testing

	Load							
Demac pts	**0**	**16**	**32**	**48**	**56**	**64**	**72**	**0**
1	-28	-31	-34	-33	-34	-33	-37	-37
2	223	227	225	215	217	218	204	212
3	576	581	568	553	552	552	560	566
4	14	11	10	15	18	19	20	19
5	52	52	44	45	53	53	52	29
6	12	21	14	15	15	21	-9	14
7	-785	-789	-788	-770	-771	-769	-784	-777
8	737	731	727	706	700	701	714	735
9	320	314	307	292	290	299	300	316
10	1434	1421	1405	1376	1373	1365	1392	1366
11	-357	-364	-360	-361	-372	-370	-372	-361
12	13	10	1	-18	-12	-15	-20	51
13	76	71	62	37	27	16	12	14
14	-223	-230	-235	-239	-243	-243	-248	-239
15	707	681	680	691	688	685	340	684
16	-41	-39	-45	-61	-57	-66	-36	-44
17	-5	-6	-8	-6	-8	-3	-7	3
18	-12	-14	-20	-17	-19	-20	-22	-7
19	45	43	39	34	31	25	22	57
20	-35	-36	-34	-43	-61	-49	-46	-23
21	-198	-270	-192	-213	-288	-214	-212	-175

22	61	45	46	50	50	50	48	
23	-938	-944	-947	-965	-959	-974	-957	-948
24	-95	-87	-88	-90	-98	-102	-100	-91
25	-111	-112	-119	-144	-119	-118	-116	-105
26	-48	-43	-46	-41	-42	-40	-38	-34
27	-41	-36	-32	-18	9	39	52	6
28	-70	-67	-68	5	40	73	82	-15
29	297	301	310	346	399	422	430	342
30	-445	-469	-488	-488	-481	-293	-302	-315
31	-13	-6	1	4	11	19	27	1
32	110	126	137	231	267	301	308	190
33	-9	-2	2	39	95	130	143	51
34	-18	-15	-15	-3	-2	-4	-2	-8
35	0	6	15	20	17	17	12	3
36	-86	-74	-64	36	77	119	128	-2
37	131	143	153	195	255	286	294	192
38	1593	1595	1594	1632	1630	1629	1598	1599
39	276	225	209	253	241	295	289	268
40	21	28	48	78	121	148	165	62
41	213	219	234	333	379	428	435	286
42	-208	-191	-203	-160	-141	-117	-111	-148
43	-167	-163	-138	-40	0	54	62	-179
44	-237	-230	-226	-196	-157	-135	-125	-205
45	67	71	74	72	71	71	73	59
46	897	900	914	914	938	958	984	917
47	329	348	375	437	475	516	542	416

48	429	437	442	477	503	525	551	476
49	44	49	47	45	47	45	53	45
50	-347	-364	-358	-246	-338	-321	-319	-352
51	-105	-116	-99	12	70	119	140	-12
52	-248	-240	-230	-185	-178	-162	-152	-208
53	-100	-93	96	-86	-77	-68	-60	-74

Data of deflection for Precast bridge deck overhang center testing

Deflection	0	16		32		48		56		64		72		0
		Before	After	Before	After	Before	After	Before	After	Before	After	Before	After	
1	0	-35	-315	-330	-340	-420	-465	-475	-485	-520		-350	-350	-36
2	0	-10	-35	-15	-75	-195	-415	-520	-605	-740		-1570	-1560	-1880
3	0	0	-10	-10	-95	-125	10	25	10	-5		-15	-115	60
4	0	-20	-15	5	60	140	285	375	445	555		1150	1140	1335
5	0	0	0	0	0	0	0	0	0	0		5	5	5
6	0	-25	-20	-15	-15	-80	-185	-220	260	-335		-555	-545	-580

Data of surface strain for Cast-In-Place bridge deck overhang corner testing

Demec	Load						
	0	**16**	**32**	**40**	**48**	**56**	**Breaking**
1	-692	-639	-645	-606	-606	-622	-621
2	298	289	306	278	273	290	298
3	138	139	138	133	135	135	135
4	-72	-85	-71	-63	-69	-79	-69
5	136	135	151	142	150	155	151
6	73	66	78	78	78	87	93
7	18	13	21	17	22	17	13
8	-263	-263	-268	-265	-246	-226	-217
9	-6	-12	44	197	320	385	320
10	-250	-257	-212	-7	-146	-275	-231
11	62	-61	-48	-48	-49	-46	-35
12	81	79	49	73	63	70	76
13	148	147	138	139	142	-349	-4961
14	-3	-4	-7	-2	0	-3	-18
15	10	12	11	7	3	1	-705
16	-56	-59	-61	-69	-66	-502	-4418
17	247	2	2	2	5	2	196
18	168	185	184	116	173	167	-2061
19	60	41	40	31	46	26	80
20	15	21	1	202	445	565	571
21	42	62	56	369	72	136	8

22	9	-18	-36	-17	-15	-415	-2095
23	441	-457	-358	-174	-125	296	179
24	307	-352	-325	-131	-3	60	27
25	300	-308	-284	-294	-261	-289	-288
26	-57	-56	79	386	612	701	489
27	-664	-600	-680	-579	-614	-616	-616
28	-598	-571	-574	-545	-499	-498	-435
29	-428	-429	-430	-428	-419	-404	-391
30	-57	-66	-24	-17	4	40	1092
31	-239	-231	-194	-173	-108	-105	-75
32	-34	-14	-41	-27	77	146	161
33	162	159	170	177	250	304	318
34	924	1111	1054	1039	1061	1115	1161
35	-114	-131	62	350	559	602	255
36	156	165	174	180	237	301	321
37	47	53	52	58	65	36	41
38	225	221	228	231	229	227	227
39	-23	-47	-13	-22	-24	-27	-36
40	-2	6	56	188	340	363	331
46	-76	83	73	85	74	88	15
47	48	-44	-41	-44	-50	-48	-35
48	-258	238	239	213	192	212	229
49	-196	173	178	185	155	161	145

Data of deflection for Cast-In-Place bridge deck overhang corner testing

Deflection	16		32		40		48		56		Breaking
	Before	After	Before	After	Before	After	Before	After	Before	After	
1	45	45	75	80	95	85	85	55	-30	-270	-990
2	0	0	0	0	0	0	0	0	0	0	-420
3	0	0	5	5	0	0	0	0	-95	-260	-895
4	-70	-220	-210	-205	-95	-55	80	145	345	530	645
5	0	-35	-65	-100	-210	-240	-250	-405	-570	-930	3370
6	65	130	110	120	25	0	-90	-125	-300	-435	445
7	-60	-185	-230	-220	-630	-690	-955	-1045	-1285	-2125	9045
8	40	100	155	155	445	480	700	765	990	1600	7420
9	40	90	95	95	45	50	-40	-55	-220	-235	-390

Data of surface strain for Precast bridge deck overhang corner testing

Demec	Load						
	0	16	32	40	48	56	Breaking
1	-692	-639	-645	-606	-606	-622	-621
2	298	289	306	278	273	290	298
3	138	139	138	133	135	135	135
4	-72	-85	-71	-63	-69	-79	-69
5	136	135	151	142	150	155	151
6	73	66	78	78	78	87	93
7	18	13	21	17	22	17	13
8	-263	-263	-268	-265	-246	-226	-217
9	-6	-12	44	197	320	385	320
10	-250	-257	-212	-7	-146	-275	-231
11	62	-61	-48	-48	-49	-46	-35
12	81	79	49	73	63	70	76
13	148	147	138	139	142	-349	-4961
14	-3	-4	-7	-2	0	-3	-18
15	10	12	11	7	3	1	-705
16	-56	-59	-61	-69	-66	-502	-4418
17	247	2	2	2	5	2	196
18	168	185	184	116	173	167	-2061
19	60	41	40	31	46	26	80
20	15	21	1	202	445	565	571
21	42	62	56	369	72	136	8

22	9	-18	-36	-17	-15	-415	-2095
23	441	-457	-358	-174	-125	296	179
24	307	-352	-325	-131	-3	60	27
25	300	-308	-284	-294	-261	-289	-288
26	-57	-56	79	386	612	701	489
27	-664	-600	-680	-579	-614	-616	-616
28	-598	-571	-574	-545	-499	-498	-435
29	-428	-429	-430	-428	-419	-404	-391
30	-57	-66	-24	-17	4	40	1092
31	-239	-231	-194	-173	-108	-105	-75
32	-34	-14	-41	-27	77	146	161
33	162	159	170	177	250	304	318
34	924	1111	1054	1039	1061	1115	1161
35	-114	-131	62	350	559	602	255
36	156	165	174	180	237	301	321
37	47	53	52	58	65	36	41
38	225	221	228	231	229	227	227
39	-23	-47	-13	-22	-24	-27	-36
40	-2	6	56	188	340	363	331
46	-76	83	73	85	74	88	15
47	48	-44	-41	-44	-50	-48	-35
48	-258	238	239	213	192	212	229
49	-196	173	178	185	155	161	145

Data of deflection for Precast bridge deck overhang corner testing

Deflection	16		32		40		48		56		64		72		Breaking
	1265	1265	2430	2430	3055	3055	3665	3665	4275	4275	4886	4886	5497	5497	6100
1	20	-5	-10	-65	-90	-160	-300	-15	-305	-335	-320	-380	-325	-415	-490
2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-5
3	0	-25	-60	-105	-145	-200	-235	-270	-280	-330	-265	-440	-455	-540	-885
4	0	5	5	5	5	0	0	0	0	0	0	0	0	0	0
5	0	0	-75	-145	-220	-385	-480	-535	-625	-765	-825	-905	-1035	-1140	-1635
6	65	-120	-210	-225	-270	-195	-295	-310	-440	-475	-585	-610	-845	-935	-2865
7	-75	-105	-285	-345	-465	-685	-835	-890	-1100	-1200	-1420	-1490	-1720	-1810	-2050
8	0	0	0	0	0	-35	-105	-140	-270	-315	-540	-590	-965	-1045	-6125
9	130	-460	-350	-350	-235	695	695	695	695	695	680	695	505	455	-4310

VITA

Siddharth Ramchandra Patil

Candidate for the Degree of

Master of Science

Thesis: DEVELOPMENT OF PRECAST BRIDGE DECK OVERHANG SYSTEM

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Kolhapur, Maharashtra, India on 1st May 1984, the son of Ramchandra Ganpatrao Patil and Sumitra Ramchandra Patil.

Education: Received Bachelor of Engineering from Shivaji University in May 2005. Completed the requirements for the Master of Science in Civil Engineering and Structural Engineering as a Specialty at Oklahoma State University, Stillwater, Oklahoma in December2009.

Experience: Employed by the Oklahoma State University as a Graduate Research Assistant during December 2007-August 2008 and as a Graduate Teaching Assistant for the course CIVE 3513 (Structural Steel Design) for Fall 2008 and CIVE 5673 (Concrete Materials and Mix Design) for Spring 2009. Worked as an Entry Level Engineer for Ramchandra G. Patil Builders and Contractors, Gadhinglaj, India during June 2005-June 2007.

Professional Memberships:
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Name: Siddharth Ramchandra Patil Date of Degree: December 2009

Institution: Oklahoma State University Location: Stillwater, Oklahoma

Title of Study: DEVELOPMENT OF PRECAST BRIDGE DECK OVERHANG SYSTEM

Pages in Study: 176 Candidate for the Degree of Master of Science

Major Field: Civil Engineering

Scope and Method of Study:

Almost 24% of the bridges nationwide are structurally deficient or functionally obsolete, this situation demands for new and innovative method for bridge repair and construction that are able to provide cost-effective, long-lasting and rapid construction systems. Precast bridge deck overhang system has shown potential to provide a very effective construction technique which can be implemented for the rehabilitation of existing bridges as well as new bridge construction. For this reason, full-scale testing of precast bridge deck overhang system was done to evaluate the performance for service and strength limit states as per AASHTO LRFD 2007. Testing of adjustable haunch form was also conducted during this research study.

Findings and Conclusions:

The experimental results clearly infers that proposed system shows satisfactory performance for service and strength limits as per AASHTO LRFD 2007 with improved performance over conventionally used CIP bridge deck overhang system. Recommendations for use of adjustable haunch form system are made on the basis the test data.

ADVISER’S APPROVAL: Dr. M. Tyler Ley