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Field Evaluation of Alternative and Cost-Efficient Bridge Approach Slabs

by

PI: **Ganesh Thiagarajan Ph.D., P.E**
University of Missouri Kansas City

John Myers Ph.D., P.E
Missouri University of Science and Technology

Ceki Halmen Ph.D., P.E
University of Missouri Kansas City

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16. Abstract

Based on a recent study on cost efficient alternative bridge approach slab (BAS) designs (Thiagarajan et al. 2010) has recommended three new BAS designs for possible implementation by MoDOT namely a) 20 feet cast-inplace slab with sleeper slab (CIP20SLP) - for new construction on major roads, b) 25 and 20 feet precast-prestressed slab with sleeper slab (PCPS20SLP)- for replacement and new construction applications on major and minor roads, and c) 25 feet modified BAS without a sleeper slab for new CIP construction on minor roads (CIP25NOSLP). The objective of the project here is to evaluate and compare the field performance of recommended BAS designs, their constructability, and their impact on cost and schedule to the current MoDOT BAS design. Two PCPS20SLP, one CIP20SLP and two CIP25NOSLP implementations have been implemented and studied for this report. One PCPS20SLP panel was also tested in the laboratory for several washout conditions and for its ultimate capacity. Based on field observations, data recorded and analyzed, preconstruction and post construction cost analyses this study has found that all the suggested designs are performing well and are lower in cost compared to current designs used in practice.

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FINAL REPORT

Field Evaluation of Alternative and Cost Efficient Bridge Approach Slabs

Prepared for
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at
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by

PI Ganesh Thiagarajan Ph.D., P.E., (ganesht@umkc.edu) 816-235-1288
Associate Professor, Civil Engineering,
University of Missouri Kansas City, Kansas City, MO-64110
Co-PI John Myers Ph.D., P.E, (myersj@mst.edu) 816-235-1286
Professor, Civil, Arch. & Envir. Engineering
Missouri University of Science and Technology, Rolla, MO - 64110
Co-PI Ceki Halmen Ph.D., P.E, (halmenc@umkc.edu) 816-235-1286
Assistant Professor, Civil Engineering
University of Missouri Kansas City, Kansas City, MO – 64110

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EXECUTIVE SUMMARY

The main purpose of the project "Bridge Approach Slabs for Missouri DOT Field Evaluation of Alternative and Cost Efficient Bridge Approach Slabs" was to study the field implementation of three innovative structural solutions that had been proposed in a previous project. The main objective of the proposed project is to evaluate and compare the field performance of recommended BAS designs, their constructability, and their impact on cost and schedule to the current MoDOT BAS design. Based on field data that was collected, improvements in terms of construction practices, sequencing, design details, and other issues for a successful implementation of the new BAS designs in the future are recommended.

The three solutions presented in the original report were a) 12 inch thick cast-in-place (CIP20SLP) slab of 20 feet span with a sleeper slab support b) 12 inch thick CIP slab of 25 feet span with no sleeper slab support (CIP25NOSLP) and c) 10 inch thick precast, prestressed slabs (PCPS) with transverse ties and a span of 20 feet with sleeper slab for new construction and a span of 25 feet with sleeper slab for replacement slabs. These design solutions were implemented on five bridges across the state. The implementations included two PCPS on a major and a minor road, one 20 feet CIP20SLP BAS with a sleeper slab on a major road and two 25 feet CIP25NOSLP BAS without a sleeper slab on the minor road system.

Evaluation of the performance of the slabs was done using a combination of field testing and laboratory testing. Field evaluation included instrumentation of one of the PCPS slabs and data collection over a period of two years and field testing for deflection and rotation of three BASs using loaded trucks provided by MoDOT. One of the PCPS BAS on MO38 was instrumented with eleven vibrating wire strain gauges and 5 moisture gauges to monitor the internal strains in the slab and the moisture content in the base rock below the slab and data has been collected over a period of two years. The 20 foot CIP20SLP BAS was load tested using trucks once prior to the bridge opening. The PCPS BAS and the CIP25NOSLP BAS on MO38 have been load tested using the trucks both prior and after the bridge was opened for traffic. Furthermore, one 8 foot wide PCPS panel that was constructed as an additional panel when one of the PCPS bridge BAS panels were made was load tested at Missouri University of Science and Technology laboratory under several washout conditions. Its ultimate capacity and deflection response was also determined experimentally in order to compare with the predicted analytical responses. This laboratory testing was not a part of the original proposal and has provided valuable information.

Based on the visual inspections all the bridges are performing very well. No major faults or defects have been observed. The field load testing indicated a deflection of 0.042 in and 0.01 degrees of rotation which are negligible. The field instrumentation has indicated no major water collection in the base rock system and the peak strain from the vibrating wire strain gauge has been 675 microstrains which indicates that the slab is within the elastic range. The laboratory load testing has shown that PCPS panel has a peak load capacity almost 50% more than the designed moment capacity of the slab. Performance at various washout conditions is presented in the report.

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CHAPTER 1 **INTRODUCTION**

Recently Missouri Department of Transportation (MoDOT) in collaboration with Missouri Transportation Institute (MTI) and the National University Transportation Center (NUTC) funded a study to develop cost efficient alternative bridge approach slab (BAS) designs (Thiagarajan et al. 2010). After surveying the cost and performance of different bridge approach slabs being used by Departments of Transportations around the country, performing numerical modeling and simulations, and analyzing cost data, the study recommended three new BAS designs for possible adoption and implementation by MoDOT;

a) 20 feet cast-in-place (CIP) slab with sleeper slab - for new construction on major roads (CIP20SLP)
b) 20 or 25 feet precast-prestressed slab (PCPS) with sleeper slab - for replacement and new construction applications on major and minor roads, (PCPS20SLP) and
c) 25 feet modified BAS without a sleeper slab for new CIP construction on minor roads (CIP25NOSLP).

Additionally, the use of Controlled Low Strength Materials (CLSM), aka flowable fill, as a backfill material behind bridge abutments under the BAS was evaluated as a possibility to solve problematic bridge approach slab issues, such as excessive settlement, loss of support due to erosion, and others. In this first study a preliminary review was performed to evaluate feasibility of producing low cost CLSM mixtures suitable for this application.

In this companion project, researchers collaborated with MoDOT engineers and contractors to evaluate the field performance of the BAS designs, recommended by the earlier study, through their implementation on a series of pilot projects. Field performance of the new BAS designs and their impact on construction duration and cost was evaluated using a variety of methods including visual evaluation, profilograph studies, surveying, field instrumentation, collection of actual cost and duration data from the field engineers, and post construction project interviews with the field engineers.

1.1 Objective

The main objective of this project was to evaluate and compare the field performance of the recommended new BAS designs, their constructability, and their impact on cost and schedule compared with the current MoDOT BAS designs. The research team will also make recommendations via field observations related to construction practices, sequencing, design details, and other issues for successful implementation of the new BAS designs in the future.

1.2 Scope of the Project

This project consisted of eight major tasks performed to accomplish the objective. These tasks are briefly described below and detailed information on each task and results are provided in the following chapters.

1. A literature review on field testing of BASs was performed, including available literature in the Transportation Research Information Database (TRID).
2. Three new BAS designs were constructed on five bridge projects selected by MoDOT as listed in Table 1-1. The 20 feet long cast-in-place design with sleeper slab was tested on a major road and the 25 feet long cast-in-place design without a sleeper slab was tested on two projects on minor roads. The 20 feet precast/prestressed design with a sleeper slab was tested on two projects; one on a major road and the other on a minor road. The precast/prestressed approach slabs consisted of multiple precast/prestressed panels that were connected on site using tie rods. Project plans were developed by MoDOT based on designs and drawings detailed in the earlier study that recommended these new BAS designs (Thiagarajan et al. 2010). The research team reviewed the project plans and specifications and made recommendations for the field instrumentation.

Table 1-1 MoDOT Projects and BASs

No.	Project	Bridge No.	Description	Letting Date	Plans Completion Date	Road System	Approach Slab Type[1]	Slab Thickness (in)/Width (ft)
1	J7B0801K	B0563 (New)	US71 SB over MNA Railroad	11-Apr	1/17/2011	Major	20' CIP with sleeper slab	12 in/ 38 ft
2	J1P1047	A7934 (New)	US136 over Little Tarkio Creek	11-Jun	4/9/2011	Major	20' PCPS with sleeper slab	10 in with 2 in asphalt overlay/ 30 ft
3	J8S2168	A7890 (New)	MO38 over Fork Osage Branch	11-Feb	12/10/2010	Minor	25' CIP w/o sleeper slab	12 in/ 34 ft
4	J4P2203	A7925 (New)	MO45 over Platte River	11-May	3/15/2011	Minor	25' CIP w/o sleeper slab	12 in/ 34 ft
5	J8S2174	A7767 (New)	MO38 over Greer Creek	11-Feb	12/10/2010	Minor	20' PCPS with sleeper slab	10 in with 2 in asphalt overlay 30 ft

[1]CIP Cast-in-place

3. Field performance of CIP BASs was evaluated and compared to the current MoDOT BAS design performance mainly by visual inspection. Visual inspections of BASs included inspection of wing walls, bridge railing, and shoulders for transverse/longitudinal cracks, crack density, and size of cracks. Joints were also evaluated for performance and changes in joint openings over time and were recorded.
4. The precast/prestressed approach slabs and two of the CIP slabs were monitored for settlement and rotations using a high precision Total Station™. Elevation shots were taken from various survey points on the bridges and a stable benchmark position for each bridge. Continued monitoring of these bridges using the established survey points and by visual inspections over the long term will provide valuable data for implementation of the new BAS designs.
5. The precast/prestressed BAS on one side of the bridge on MO38 over Greer Creek was instrumented with embedded vibrating wire strain gauges and moisture gauges for long term monitoring. A Geokon 16 channel field data acquisition system (DAS) and logger connected to the gauges was fixed to the abutment under the bridge deck. The data logger automatically collected and stored data at programmed intervals and collected data was periodically transferred to a laptop computer. The installed system allowed the research team to observe any variations over time due to service loading conditions and/or environmental changes.
6. A static live load test was performed on bridges B0563, A7767, and A7890 shortly after the BASs were installed and approximately six months after the slabs were opened to traffic. Testing was performed in coordination with the local MoDOT engineers using MoDOT supplied standard HS20 trucks. The initial round of testing used empty trucks and loaded trucks were used for the second round of testing. Inclinometers were placed on the approach slabs to measure rotations due to loading during testing. The initial load test provided baseline values to compare the long-term performance and to provide a better understanding of each BAS design when it has full elastic support. Trucks were filled with approximately 10.5 tons of aggregate for the second test which helped show the influence of increased loading on each BAS. The embedded strain gauges in bridge A7767 were also set to the shortest time interval possible to capture strain data during load testing.
7. Samples were collected during the casting of precast/prestressed (PCPS) panels at the precast plant and tested at the laboratory. These laboratory materials tests were performed to verify the initial design strength and to provide additional insight on the long-term performance and behavior of the concrete used for the PCPS BASs. The tests included modulus of elasticity (MOE), compressive strength, creep, shrinkage, and modulus of rupture (MOR). Laboratory testing also included testing of an extra PCPS panel cast at the precast plant following the specifications of the panels cast for the bridge A7767. The PCPS panel was loaded at different support conditions simulating the washout conditions of a slab in the field. Infiltration of water under bridge approach slabs can cause washout of soils consequently changing the support conditions which is a common problem. Rotations, strains, and deflections of the PCPS panel were evaluated under different support conditions using inclinometers, strain gauges, and string pods.
8. Actual cost of recommended new BAS designs, potential cost savings, and duration of major construction activities were documented in collaboration with MoDOT Resident

Engineers and contractors on each project. Lessons learned during field implementation of PCPS slabs are incorporated into this final report.

Accomplishment of these eight tasks are described and presented in this final report to MoDOT. Details of each task, results of laboratory and field testing, actual cost and duration information, lessons learned, and recommendations for future implementation plans are included in the following chapters.

This research is the result of collaboration between University of Missouri-Kansas City (UMKC) and the Missouri University of Science and Technology (Missouri S&T). Dr. Ganesh Thiagarajan (UMKC) served as the principal investigator along with Dr. John J. Myers (Missouri S&T) and Dr. Ceki Halmen (UMKC) as co-principal investigators. The lead graduate research assistants were Jitesh Nalagotla (UMKC), Gunjan Shetye (UMKC) and Nathan Muncy (Missouri S&T). Coreslab Structures, Inc. in Marshall, MO, provided their facilities for the precast-prestressed BAS. Missouri S&T Department of Civil, Architectural, and Environmental Engineering (CArEE) technicians Brian Swift and Gary Abbott provided technical insight and service for the data acquisition system (DAS), total station ™, and inclinometers. Staff members Jason Cox and John Bullock from the Center for Infrastructure Engineering Studies (CIES) at Missouri S&T provided technical assistance when preparing and performing the in-situ load testing. The Missouri Department of Transportation (MoDOT) and National University Transportation Center (NUTC) provided funding for the project and all necessary staff and equipment to perform the load testing. Additional staff was also provided by the Missouri S&T. Missouri S&T efforts primarily focused on the in-situ BAS load testing and the laboratory BAS panel testing in the High-Bay Structural Engineering Research Laboratory (SERL).

CHAPTER 2 **LITERATURE REVIEW**

This chapter provides background information on bridge approach slabs (BASs) and provides insight on previous research studies that have been performed. Common factors that affect BAS performance are discussed along with the MoDOT companion study that led to the development of the new BAS designs implemented and evaluated in this study.

A typical BAS is a reinforced concrete slab that helps to provide a smooth transition between the flexible concrete or asphalt pavement and the rigid bridge superstructure. BASs usually span from 20 ft. to 40 ft. (6.1 m to 12.2 m) and are constructed using either cast-in-place (CIP) concrete or, as new studies show, precast-prestressed concrete (PCPS) slabs. BASs are supported on the abutment on one end and on soil or a sleeper slab on the other [4]. As shown in Figure 2-1, the embankment fill provides a uniform support for the BAS between the abutment and the sleeper slab (not shown in Figure 2-1). The figure also provides a typical BAS layout between the bridge superstructure and the flexible pavement.

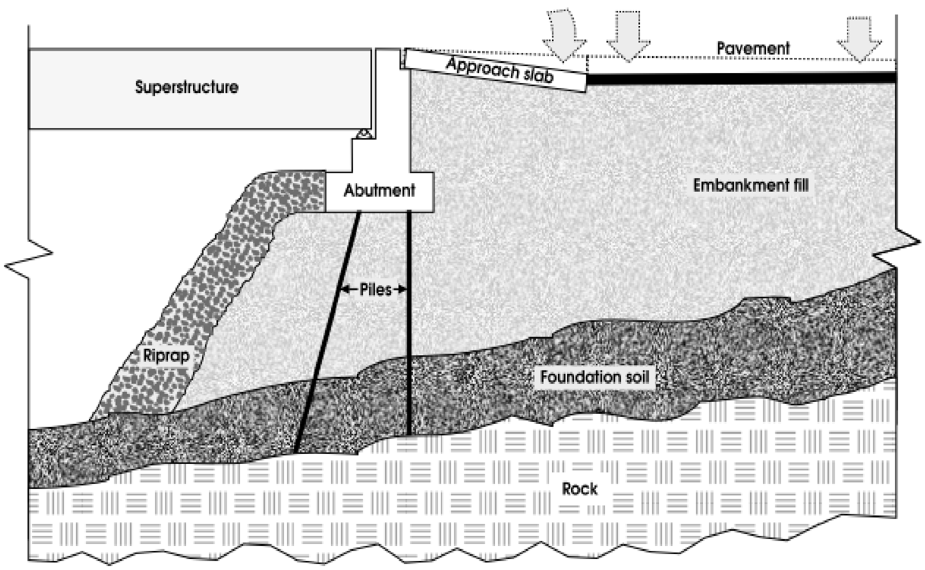


Figure 2-1 Typical BAS Layout [5]

2.1 Factors Affecting BAS

Many factors affect the performance of BASs such as structural, geotechnical, and the erosion of the embankment fill [5]. A survey was performed with the Departments of Transportations (DOT) across 48 states and 25% of the DOTs indicated that they had experienced BAS settlement problems [6]. During the mid-1990's an investigation was performed on 600,000 cast-in-place BASs and the results indicated 150,000 BASs experienced a "bump at the end of the approach slab" [1]. The performance of BAS remains an issue across the United States and is a complicated problem due to the various factors involved.

2.1.1 **Structural Factors**

Many structural factors are a concern when analyzing the performance of a BAS. These factors include the slabs dimensions, reinforcement ratio, the use of a sleeper slab, slab thickness, interaction between the soil and BAS, and how the BAS is connected to the bridge superstructure [1]. There is currently no nationally accepted design standard for BASs. In most cases the use of the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications provide a guideline for an adequate structural design.

2.1.2 **Geotechnical Factors**

Some geotechnical factors that affect the performance of BASs include settlement of the embankment soil and primary and secondary compression of the foundation soil under the abutments [7]. The embankment fill has a significant impact on the performance of a BAS. Three factors that affect the performance of embankment fills is the type of soil, the density at which it is compacted, and if the fill is susceptible to erosion [8]. Figure 2-2 shows a void formation under the BAS near the abutment, due to infiltration of water. The California Department of Transportation (Caltrans) performed a BAS study which indicated erosion and settlement of the soil to be the two main issues with BAS failure. A recommendation to help mitigate water infiltration was to extend the BAS over the wingwalls [9].

Figure 2-2 Erosion of Soil at the BAS and Abutment Joint [3]

2.2 Previous BAS Studies

Due to persistent problems with BASs, many DOTs funded and performed several comprehensive studies to determine the underlying causes of failure. The Washington State Transportation Center evaluated bridge approach slab effectiveness and performed an extensive literature review dating back to a 1959 investigation performed in Los Angeles, California. The literature review included BAS studies in California, Oklahoma, and Colorado [10]. This section will provide more background information on previous studies and include problems that other DOTs are facing.

In Los Angeles, California four systems were investigated to determine if approach settlement had any correlation with soil conditions beneath the BAS. The investigated freeways and soil conditions were as follows

1. Hollywood Freeway predominately clay
2. Harbor Freeway silts and soft clays
3. Santa Ana Freeway granular and cohesive soils
4. San Bernardino Freeway coarse granular material

The study determined differential settlement based on how often the BAS required pavement patching. The Hollywood Freeway BAS experienced 50% of pavement patching while the Harbor Freeway required 70%. The Santa Ana Freeway required 20% of the BAS pavements to be patched in order to maintain safe driving conditions for motorists. The last section of highway, the San Bernardino Freeway, had a coarse granular base and did not have any pavement patching on the BAS [10].

The two key causes identified were compression of the embankment fill and consolidation of the foundation soils. Inadequate compaction during construction was also identified as a contributor to settlement. Based on the observations a few recommendations were made to help reduce BAS settlement. The first recommendation was to replace the soil beneath the BAS with a soil that contains a higher subgrade modulus which causes the BAS to experience less settlement. During the construction phase, it was recommended that the embankment fill have a surcharge load placed on the surface to allow primary consolidation to occur. Last recommendation was to use a 30 ft. long BAS to create a smooth transition between the flexible pavement and the abutment [10].

Another study performed in California investigated different abutment types and various BAS parameters to determine what causes rough transitions. In 1973, 410 bridges and 820 BASs were inspected over an 1,800 mile stretch of highway in California. The investigation focused on three factors visual flaws of the BASs, roughness rating, and any notable flaws with the bridge conditions. After 10 years of data collection, a California Department of Transportation (Caltrans) task force determined that consolidation of the embankment soil was a major contributor in the performance of BAS. Nearly 92% of the BASs had voids formed beneath the slab and embankment fill [10].

Researchers from the University of Oklahoma worked in conjunction with the Oklahoma Department of Transportation (ODOT) to determine the leading causes of BAS settlement. The project involved performing a comprehensive literature review and distributing a questionnaire to state highway offices. The survey indicated that the main causes of settlement were the inadequate compaction of the embankment fill and poor construction practices. Settlement was correlated with the abutment type and stub abutments generally performed better by providing a smooth transition [10].

The Colorado Department of Highways (CDOH) investigated 100 bridges throughout the state of Colorado that had moderate to severe BAS settlement problems. These 100 bridges were visually inspected and the 10 with the most severe settlement were chosen for further investigation. The 10 bridges were located all over the state so the geotechnical conditions for the embankment fill varied between each bridge [10]. Extensive testing was performed on the embankment fill such as tube sampling and a sieve analysis which indicated consolidation was the primary and most serious cause for pavement settlement. This was attributed to poor compaction and use of compressible materials. Erosion was also observed near the inside of the abutment wall caused from infiltration of water under the BAS. Remedial measures such as preloading the embankment fill during construction and installing wick drains were suggested to help reduce these problems [10].

A study performed by Edward Hoppe compared BAS designs and construction practices from DOTs across the United States to the designs used by the Virginia DOT [9]. The goal of this study was to compare different design and construction techniques to help minimize differential settlement. A survey was developed and sent to forty eight state DOTs that included questions related to structural, geotechnical, and construction aspects of states' BASs. Thirty nine DOTs responded to the survey and fifty five percent of respondents indicated that BASs in their state were a significant problem while twenty nine percent viewed it as a moderate problem. Conclusions from the survey indicated that the design and construction practices vary between states. Due to the changing conditions between sites, an engineering analysis was recommended for each site to produce a cost effective BAS. The width of the BAS is usually the full width of the road which reduces erosion, resulting in less differential settlement [9].

A precast-prestressed BAS study was conducted on Highway 60 near Sheldon, Iowa in 2007. This study was intended to evaluate the performance and construction processes of PCPS panels used for BAS applications. Three studies had been performed on precast-prestressed concrete pavements (PPCP) in Texas, California, and Missouri prior to the Highway 60 study. These three studies evaluated various aspects of the construction process and the use of post tensioning after the panels were installed in the field. The Highway 60 report takes this information a step further with the use of prestressing and bi-directional post tensioning [11]. Eight panels were constructed for each PPCP including the BASs. The panels were 14 ft. by 20 ft. and spanned roughly 76 ft. from the bridge deck. The panels were post tensioned in the transverse and longitudinal directions to help each panel function as a single pavement system. The keyways (joints) of the panels were grouted and the surface was textured with a diamond grinder to make the BAS available for traffic [11]. The Highway 60 project demonstrated many different techniques and the use of PPCP for the application of a BAS. The project did include a few problems such as mismatched keyways and minor damage during transportation. These were minor issues but were identified to be addressed in future research. The project was successful and allowed future researchers to use similar panel layouts and construction techniques [11].

2.3 Performance Under Washout Conditions

The performance of BASs after soil erodes near the abutment greatly decreases. If voids form beneath the slab, the BAS relies primarily on its structural capacity. A study was performed that investigated alternative reinforcement materials to use for BASs to maintain satisfactory performance, and reduce cost under various washout conditions [12]. The specimen of interest was the slab with the conventional steel reinforcement as seen in Figure 2-3.

The test setup for this study can be found in Figure 2-4. This includes a load cell for measuring the applied load, rigid steel beams for the frame, and spreader beams to simulate a HS20-44 truck load. This test was designed for the critical loading which occurred at a distance of 4 ft. 6 in. from the abutment. The contact area for the wheel loads, as recommended by AASHTO 2007, was 20 in. by 40 in. [12].

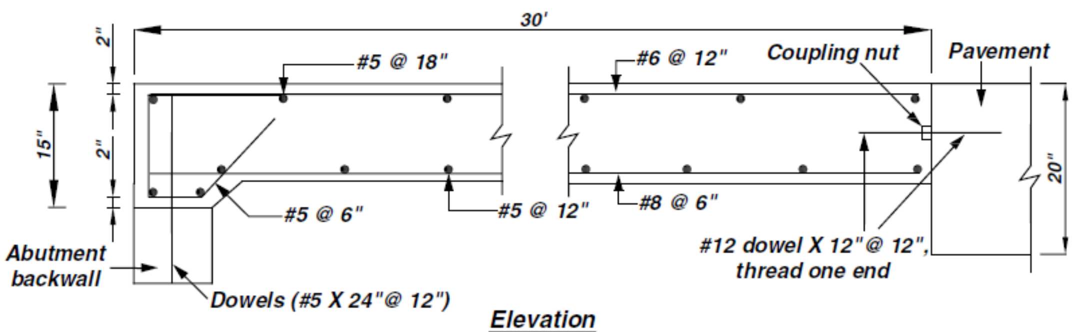


Figure 2-3 Design Details of Conventional BAS [12]

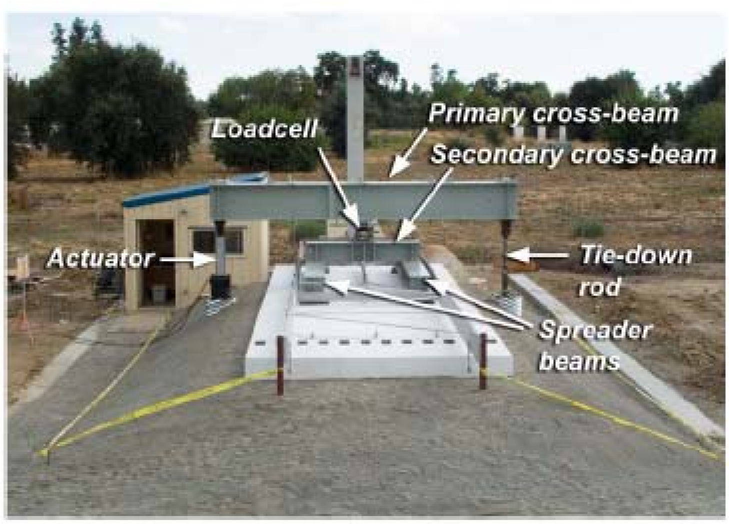


Figure 2-4 BAS Testing Apparatus [12]

The results of these tests were assessed by measured deflection of the slab at the centroid of the wheel loads and the measured rotation of the slab at the abutment. The conventional concrete slab deflection corresponding to the loading is provided in Figure 2-5. This shows that a washout, from the abutment end, condition of up to 50% of the slab length can deflect approximately 4 times more than a slab with 25% voids. This can cause serviceability issues and cause motorists to become uncomfortable. Figure 2-6 shows the abutment end rotation compared to the washout distance for various truck loadings. Two threshold rotation limits (1/200 and 1/100) are also graphed to indicate that the slab needs attention. The rotation limit of 1/200 for a weight of 64 kips and 128 (kips) was exceeded when the washout lengths were 20 ft. and 18 ft. respectively. This is approximately a 60% and 67% reduction for their respective loading conditions [12].

Results indicated that when the slab is fully supported by the soil, the rotation and settlement of the slab were insignificant. It was determined for this panel length that once the washout length reached 6 ft., the slab began losing stiffness. The rotation near the abutment did not exceed a slope of 1/200 until it reached a washout distance of 10 ft. for the loading condition of 256 kips. The deflection was a concern based on a loading of 256 kips. The deflection when the washout was 16 ft. could cause severe motorist discomfort and would need immediate repair based on a differential settlement of 4 in. [12].

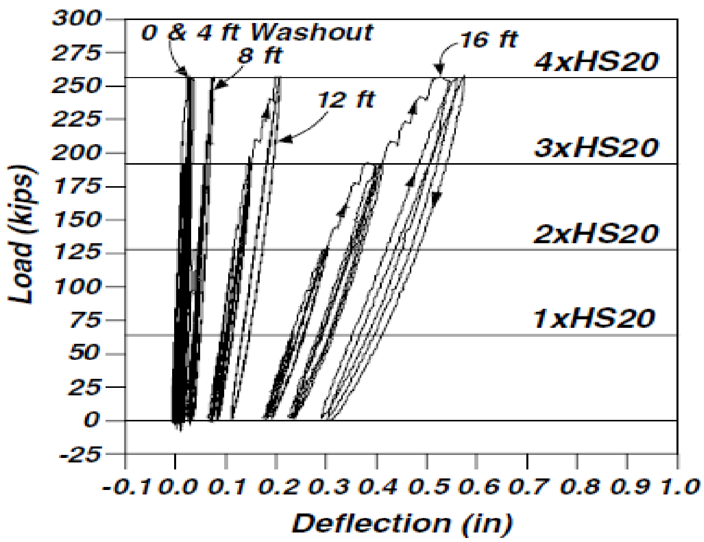


Figure 2-5 Load vs. Deflection at Various Washout Lengths [12]

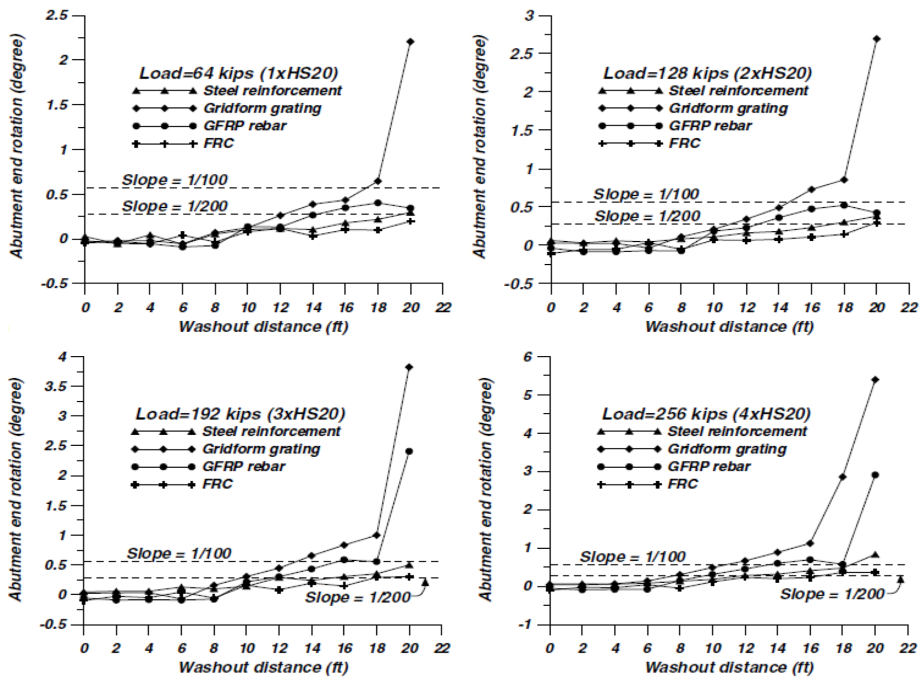


Figure 2-6 Abutment Rotation vs. Washout Distance

2.4 Alternative and Cost Efficient Approaches

This project is an extension to a prior MoDOT study performed by G. Thiagarajan et al. in 2010 entitled "*Bridge Approach Slabs for Missouri DOT Looking at Alternative and Cost Efficient Approaches.*" The objective of the prior work was to investigate alternative BAS designs that could reduce the cost of construction and to develop remedial measures and designs for replacing BASs. As noted earlier, many factors are included in BAS performance but this investigation considered only the structural issues. An extensive literature review was performed pertaining to other DOTs BAS designs. Data from approximately 40 states was collected including the length of the BASs, thickness of the slabs, reinforcement details, and the connection details to the abutment and sleeper slab (where applicable). With this information, the moment capacity of each slab was determined and correlation of moment capacity to span length and thickness was evaluated to determine if there were any trends. It was determined that 37% of the DOTs had BAS with a span of 20 ft. and 33% used a depth of 12in. [1].

2.4.1 Cast-in-place BAS Design

The design of these alternative BASs were based on the AASHTO Standard Specification for Highway Bridges (2004) and the AASHTO Load and Resistance Factor Design (2004) guidelines. Due to the complex nature of the BAS problem, there are no specified design methods or guidelines for BASs provided by AASHTO. The bridge design specifications provides the engineer with tools to design the BASs as a bridge deck that spans over 15 ft. for simply supported boundary conditions. The BAS will have support from the soil immediately after construction but over time the soil can wash out and create voids underneath the approach slab, therefore by designing the BAS as a bridge deck it will provide a simpler solution.

The design loads were determined from the AASHTO standards and design truck loadings. A design tandem was also evaluated which included 25 kip axle loads spaced 4 ft. apart. Based upon calculating the maximum effects due to the various loading conditions, a 20 ft. long and 12 in. thick BAS was recommended for further investigation. In addition to the loading analysis, a cost analysis was performed that helped validate that a 20 ft. long and 12 in. thick slab should be further investigated. Figure 2-7 shows the total cost versus the span length of the approach slab. The cost of a 20 ft. long and 12 in. thick BAS would be approximately $10,000 less than the current 25 ft. long and 12 in. thick MoDOT BAS design. The designs from different DOTs, whose cost were lower than the current MoDOT BAS design, were analyzed in further detail. Based on the span length and slab thickness, the list of acceptable BAS designs were narrowed down to 7. The recommended alternative CIP design is shown in Figure 2-8.

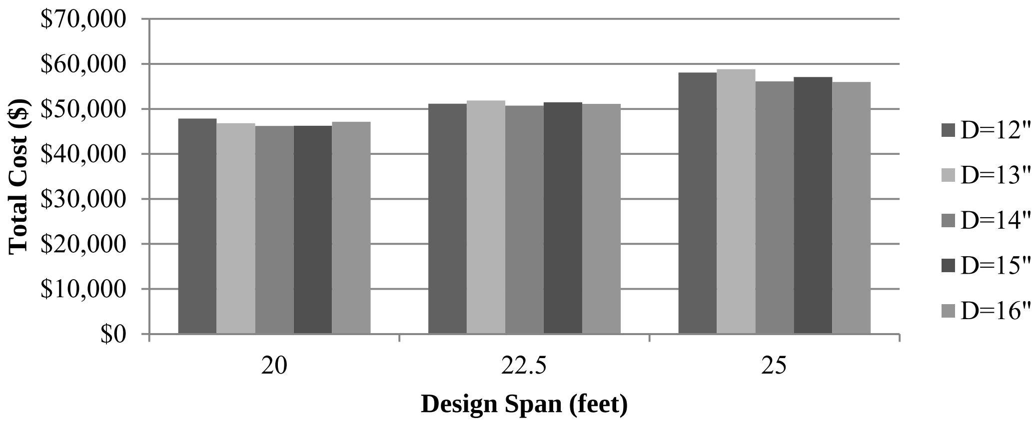


Figure 2-7 Cost of BASs Compared to the BASs Length [1]

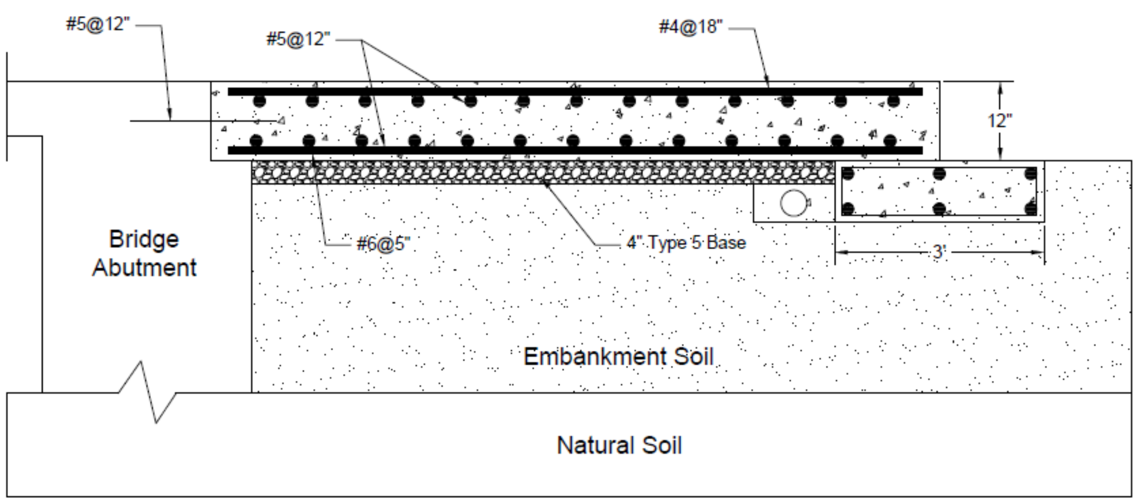


Figure 2-8 Recommended Alternative CIP Design

2.4.2 **Precast-prestressed (PCPS) BAS**

CIP concrete has been the main construction practice nationally for BASs for many years. Although PCPS slabs were used for many pavement projects, they were not used for BASs until the last decade. A joint study of demonstration projects was undertaken in Texas, Missouri, and California using PPCP. The joint study reported the cost of the PPCP panels to be nearly three times higher than CIP, therefore the earlier MoDOT study (the predecessor of this study) aimed to find a cost effective solution for using PCPS panels for BAS applications [1].

Typical precast-prestressed concrete pavements are designed to resist equivalent single axle loads (ESALs) for the life of a pavement. BASs are designed to account for erosion beneath the slab so the designs are similar to a simply supported condition with some soil support rather than ESAL loads. Assuming a simply supported condition is overly conservative, previous studies typically took various soil support conditions into consideration. The Iowa Highway 60 PCPS project study accounted for 50% loss in soil support due to erosion [11].

The BAS designs implemented in this study were also developed using a test matrix that considered various parameters including the slab width to span length ratio, slab boundary conditions, soil conditions, and loading conditions. SAP models were constructed to evaluate performance of recommended CIP and PCPS designs under different soil support conditions. Exclusion and inclusion of the lane loading was also analyzed to see its effects. Result outputs included the moment capacity, slopes, and deflections based on the model of an 8 ft. wide PCPS panel. The results for the 20 ft. and 25 ft. PCPS panels are shown in Table 2-1 and Table 2-2, respectively. Deflections were less than 1.2-1.5 in. for all loading cases and therefore the BAS panels met the serviceability requirements. All slopes were less than 0.287 radians (16 degrees) which indicates that the rotation of the slabs were satisfactorily low.

Table 2-1 Results Based on 50% Washout Conditions for a 20 ft. PCPS BAS [1]

File Name	M_{max} with lane loads (kip-ft/ft)	M_{max} without lane loads (kip-ft/ft)
BAS20-12-ES10-18.4	101.5	62.0
BAS20-12-ES10-30	90.5	55.8
BAS20-12-ES10-50	78.1	48.8
BAS20-12-ES10-100	62.0	39.8
BAS20-12-ES10-175	51.6	33.8
File Name	**δ with lane loads (in)**	**δ without lane loads (in)**
BAS20-12-ES10-18.4	0.330	0.180
BAS20-12-ES10-30	0.281	0.154
BAS20-12-ES10-50	0.226	0.125
BAS20-12-ES10-100	0.156	0.088
BAS20-12-ES10-175	0.114	0.065
File Name	**θ with lane loads (degrees)**	**θ without lane loads (degrees)**
BAS20-12-ES10-18.4	0.258	0.139
BAS20-12-ES10-30	0.224	0.121
BAS20-12-ES10-50	0.186	0.100
BAS20-12-ES10-100	0.138	0.075
BAS20-12-ES10-175	0.107	0.058

Table 2-2 Results Based on 50% Washout Conditions for a 25 ft. PCPS BAS [1]

File Name	M_{max} with lane loads (kip-ft/ft)	M_{max} without lane loads (kip-ft/ft)
BAS25-12-ES10-18.4	128.1	73.5
BAS25-12-ES10-30	111.3	64.7
BAS25-12-ES10-50	94.4	55.8
BAS25-12-ES10-100	75.3	45.8
BAS25-12-ES10-175	63.9	39.6
File Name	δ with lane loads (in)	δ without lane loads (in)
BAS25-12-ES10-18.4	0.480	0.250
BAS25-12-ES10-30	0.390	0.200
BAS25-12-ES10-50	0.300	0.160
BAS25-12-ES10-100	0.204	0.110
BAS25-12-ES10-175	0.150	0.080
File Name	θ with lane loads (degrees)	θ without lane loads (degrees)
BAS25-12-ES10-18.4	0.300	0.150
BAS25-12-ES10-30	0.260	0.132
BAS25-12-ES10-50	0.210	0.110
BAS25-12-ES10-100	0.154	0.070
BAS25-12-ES10-175	0.120	0.062

The recommended moment design capacity based on a soil modulus of 175 pci and 50% soil support, was 40 ft-kips/ft .This design load is comparable to the PCPS panels used in Iowa that were designed for a factored moment of 34.7 ft-kips/ft [11]. The study recommended a 10 in. thick slab that would be overlaid with a 2 in. asphalt layer. Based on the cost analysis this PCPS design was recommended for replacing old slabs as well as for new construction. A 20 ft. long span was recommended for new construction and a 25 ft. long span was recommended for the replacement of an existing BAS. Both PCPS designs included the use of a sleeper slab.

2.5 Summary

An extensive literature review revealed that BASs remain to be a current and widespread problem around the United States. Geotechnical factors are the predominant cause of failure due to poor construction techniques, settlement, and erosion. Once the soil is compromised, the BAS is expected to maintain a satisfactory performance based on its structural capacity. This explains to some degree why a standard design methodology has not been adopted for BASs. Each state has different geotechnical conditions resulting in a different structural design. Previous studies agree that the embankment fills need to meet certain compaction requirements and that a geotechnical engineer should be consulted for each specific project site.

The BAS designs tested in this study are based on a previous MoDOT report that created alternative and cost efficient BASs based on 50% soil support. The new design alternatives meet all serviceability requirements and the new 20 ft. CIP and 25 ft. CIP BASs (without sleeper slabs) have a projected cost savings of 22% to 30%, compared to the current MoDOT BAS designs. The PCPS BAS offers many advantages and is economically feasible when compared to the proposed CIP options.

CHAPTER 3 TEST PROGRAM, INSTRUMENTATION AND PROCEDURES

The three new BAS designs implemented in this study included a 20 foot long cast-in-place (CIP) slab with a sleeper slab, a 25 foot long CIP slab without a sleeper slab, and a 20 foot long precast-prestressed (PCPS) slab with a sleeper slab. The new BAS designs were implemented on five projects determined by MoDOT as shown in Table 3-1. The 25 feet long slab design without the sleeper slab was used on minor roads. All other slabs were used on major roads and were supported with a sleeper slab at the approach pavement connection. All plans were developed based on the designs developed by the research team in an earlier study [1]. Researchers worked closely with selected contractors and MoDOT resident engineers to observe the construction of the new BASs. All slabs were visually inspected at the completion of construction for cracks and they were inspected for a second time at the end of the duration of the project. Visual inspections included inspection of wing walls, rails, and shoulders for cracks and the condition of joints. One of the PCPS BASs was instrumented during production with embedded vibrating wire strain gauges. Before the placement of the instrumented PCPS BAS the embankment fill was also instrumented with moisture gauges to observe water infiltration. A static load testing was performed on three of the bridges that covered one of each type of BAS tested in this project and rotations of BASs were recorded during test using inclinometers. Two of the BASs were also monitored for settlement using a high precision Total Station. Table 3-1 also shows the testing program used on different projects.

Table 3-1 Implementation and Testing of New BAS Designs

Bridge No.	Description	BAS type[1]	Settlement through surveying	Rotation by inclinometers	Static Load Testing	Strain gauges	Moisture gauges	Visual inspection
B0563	US 71 SB over MNA Railroad	20' CIP with sleeper slab		✓	✓			✓
A7934	US 136 over Little Tarkio Creek	20' PCPS with sleeper slab						✓
A7890	MO 38 over Fork Osage Branch	25' CIP w/o sleeper slab	✓	✓	✓			✓
A7925	MO 45 over Platte River	25' CIP w/o sleeper slab						✓
A7767	MO 38 over Greer Creek	20' PCPS with sleeper slab	✓	✓	✓	✓	✓	✓

[1]CIP Cast-in-place, PCPS Precast/Prestressed

3.1 Instrumentation Setup and Installation

3.1.1 **Deflections Measured by Surveying**

One method used to evaluate the performance of the new BAS designs was to monitor the deflections of the slabs over time. Elevations of 6 points on the newly constructed BASs were determined relative to a point on the abutment using a high precision TCA 2003 Total Station manufactured by Leica Geosystems. The abutments were assumed to provide a stable benchmark since they were supported by pile foundations. The initial readings were used as a reference to determine if the BASs were deflecting due to various reasons such as water infiltration and loss of support or due to settlement of embankment soils. The Total Station was placed off the roadway roughly 50 to 100 ft. from the BAS to provide a wide line of sight to the selected points on the BAS. The total station rested on a tripod placed firmly in the soil to eliminate any movement when readings were being taken. Surveying prisms were placed on the selected 6 points of the BAS and on the abutment to perform the surveying. Figure 3-1 shows a surveying prism and the TCA 2003 Total Station. The surveying prisms had to be placed on a smooth and flat surface to collect accurate readings unlike the concrete surface of the BAS. Therefore monitoring plates were installed on the BASs at the selected 6 points. Detailed information on the monitoring plates is provided in the following sections.

Figure 3-1 Surveying prism (left) and the Total Station (right)

Previous load testing indicates that the high precision laser equipment is accurate up to 0.005-in if the total station is used within 200 ft. of the surveying prisms [24]. This system was chosen for the ease of use compared to linear variable displacement transducers (LVDTs) that are typically used for displacement measurements in the laboratory environment. LVDTs require a longer setup time and are difficult to use in field applications. The surveying equipment not only provides accurate readings but is easy to transport, making it the equipment of choice for this field investigation.

3.1.2 **Rotation Measured by Inclinometers**

The rotations of the BASs in the transverse and longitudinal directions during the static load testing were measured with inclinometers. The inclinometers used for field testing were D-series inclinometers produced by Measurement Specialties. The dual axis inclinometer (model NS-5/DMG2-U), shown in Figure 3-2, has high precision for a wide temperature range and the robust steel housing makes it ideal for field studies. This inclinometer provides a measurement range of ±5 degrees and measures the rotations in two directions. Data was collected from inclinometers during static load testing using a computer program developed by the Missouri S&T staff which allowed the inclinometers to capture readings instantaneously. This setup with inclinometers minimized the time traffic had to be stopped and allowed for immediate results.

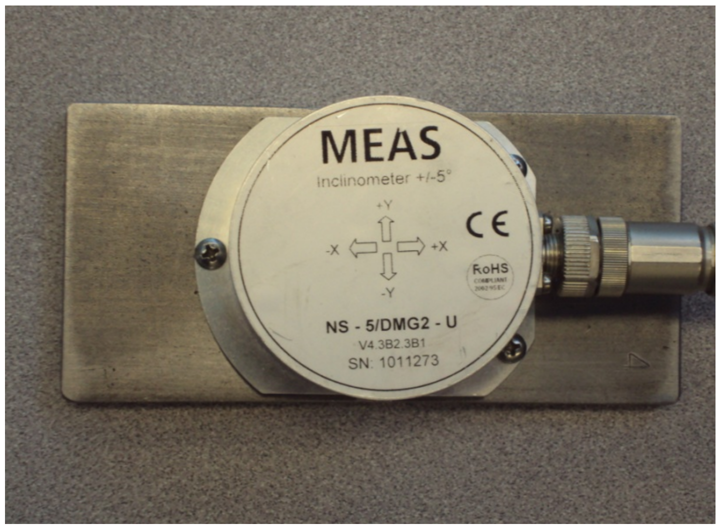


Figure 3-2 Dual Axis D-Series Inclinometer

The contact surface between the inclinometer and the BAS had to be clean and flat to ensure no discrepancies in the results. Therefore inclinometers were also placed on monitoring plates that were installed on the BASs to be used with surveying prisms. The inclinometers were connected to a data acquisition system (DAQ) with 40 ft. cables. The DAQ only functioned as a power supply for this project. All data received from the inclinometers was transferred through the DAQ directly to a laptop computer.

3.1.3 **Monitoring Plates**

The surveying prisms and inclinometers required a smooth and flat surface for testing. Monitoring plates were installed to the BASs to provide a smooth surface for the instrumentation and to provide reference points on the slab to monitor long term performance. The monitoring plates were designed to be embedded into the surface of the concrete so the instrumentation reflects the behavior of the slab. Figure 3-3 shows the details of the monitoring plates. Number three mild steel rebar was welded to the plate to prevent the plate from popping off the non-shrink grout due to temperature changes. The monitoring plates were placed in non-shrink grout approximately ½ in. below the pavement surface. This was done to prevent the plates from being damaged by snow plows and traffic. Prior to installation, the plates were coated with a thin layer of rust prevention spray paint since the plates were exposed to the environment.

The monitoring plates were installed into the 20 ft. PCPS (Bridge A7767) and the 25 ft. CIP BAS (Bridge A7890) after the slabs were constructed and required removing part of the concrete using a jack hammer. There was no interference with either prestressing strands or steel reinforcement during this process. The plates were installed into the holes using a fast setting non-shrink grout as shown in Figure 3-3. The monitoring plates for the 20 ft. CIP BAS (Bridge B0563) were installed during the time of the concrete pour and plates were directly placed into the fresh concrete. Detailed information on the locations of the monitoring plates on each BAS is provided in Chapter 4 that describes the construction of the BASs. Figure 3-4 shows a generic sketch of a BAS and the numbering used for the monitoring plates to place the surveying prisms and the inclinometers.

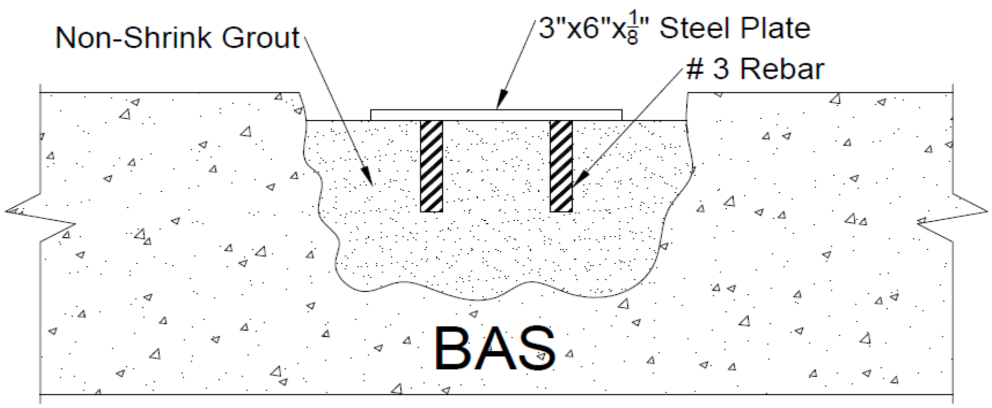


Figure 3-3 Monitoring Plate Details

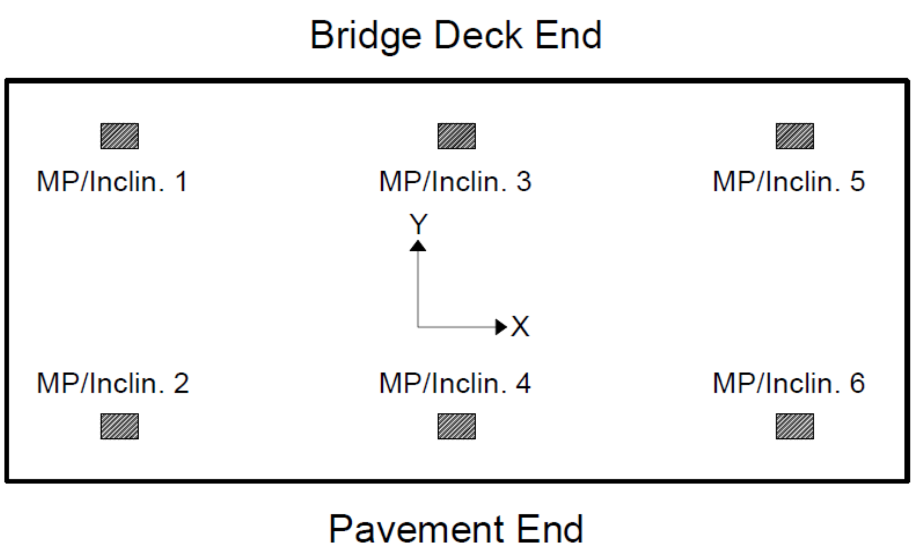


Figure 3-4 Monitoring Plate and Inclinometer Identification Numbers

3.1.4 **Vibrating Wire Strain Gauges (VWSGs)**

The PCPS panels used to construct the BAS on the east side of the bridge A7767 on MO38 were instrumented with embedded VWSGs to monitor strain development over the long term. Geokon VCE-4200 VWSGs were used in this study. These gauges are designed to measure long-term strain in structures such as foundations and piles and are equipped with a thermistor for measuring temperature. The thermistors give a varying resistance output as the temperature in the environment (concrete) changes. Figure 3-5 shows a schematic of a VWSG.

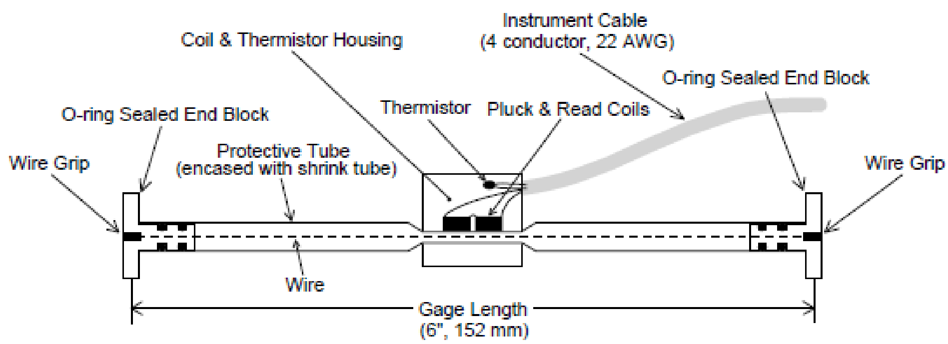


Figure 3-5 Schematic of VWSG Installed in the PCPS Panels

The embedded VWSG measures strains using a steel wire tensioned between two end blocks that are firmly in contact with the mass concrete. Deformations in the concrete cause the two end blocks to move relative to one another altering the tension in the steel wire. The change in tension due to strains in concrete is measured as a change in the resonant frequency of vibration of the wire. Electromagnetic coils that are located close to the wire accomplish excitation and readout of the gauge frequency.

VWSGs can be placed in concrete members in a variety of ways; they can be directly embedded in concrete by pre-attaching the gauges to rebar or tensioning cable, they can be precast into a concrete briquette that is subsequently cast into the concrete member, or they can be placed into the boreholes and grouted. In this study VWSGs were attached to reinforcement prior to casting of the PCPS panels at the precast plant as shown in Figure 3-6. Wooden spacers notched on both sides were used to attach the VWSGs to the reinforcement and to ensure that they will remain parallel to the reinforcement during pouring of concrete. VWSGs were attached to the wooden spacers using electrical tape and these assemblies were attached to the reinforcements at the precast plant using zip lock ties. The wires of VWSGs were also tied to the rebar at every 3 feet using a zip lock tie to prevent damaging and entanglement of wires during concrete pouring. The VWSGs were installed on July 6, 2011 at Coreslab Structures, Inc. prior to the casting of the PCPS panels. A total of 11 VWSGs were installed in the PCPS panels that made up the east side BAS of the bridge A7767. Detailed information on the location and direction of the VWSGs is provided in Chapter 4 that describes the construction of the BASs.

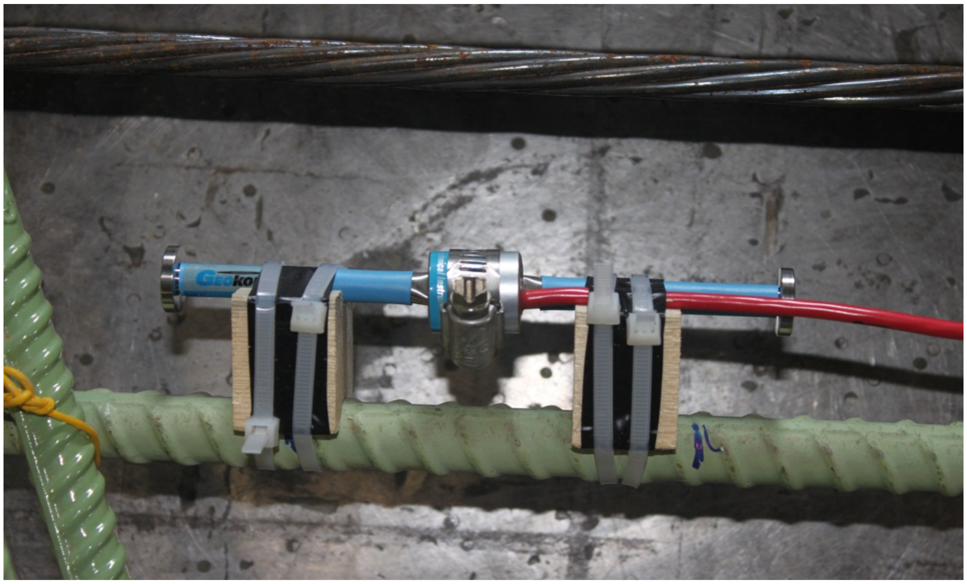


Figure 3-6 VWSG attached to the reinforcement

3.1.5 **Laboratory Testing of VWSGs**

The VWSGs were tested at the University of Missouri-Kansas City (UMKC) laboratory before installation at the precast plant. The gauges were initially tested in zero strain condition and later strained using a clamp and micrometer assembly. The VWSGs have a range of approximately 1500 microstrain in compression or tension. The gauges were tested for two months in the laboratory. The read out values of the VWSGs without a load and the corresponding microstrain values were recorded to be used later for analysis of field data. Table 3-2 shows the recorded values for the unloaded case in the laboratory.

Table 3-2 Laboratory Testing of Vibrating Wire Strain Gauge

	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11
Reading	817	885	857	806	869	838	887	873	843	865	847
Micro-strain	2645	2867	2774	2611	2813	2713	2872	2827	2730	2802	2741

3.1.6 **Data Logger For Strain Gauges**

Geokon LC2x16 sixteen channel data logger was installed at site and used to collect and store VWSG recordings. The LC2x16 sixteen channel data logger is a battery powered measurement instrument that can read up to 16 VWSGs equipped with thermistors and store 3555 data arrays. The data can be downloaded using a USB modem and Log View software installed on a PC or laptop. Each data array contains the strain recorded (in microstrain units) and temperature readings (in degrees Celsius units) for each of the 16 VWSGs. Log View software allows the data to be downloaded in Microsoft Excel format. The microstrain readings can then be converted to actual strain and load values as described in following sections. Figure 3-7 shows the data logger with 16 cable connections for the gauges and the copper grounding cable.

Figure 3-7 LC2x16 data logger

Each VWSG connection cable consisted of five wires that were connected to terminal strips inside the data logger as shown in Figure 3-8. Table 3-3 shows the data transferred in each of the wires of a connection cable. The connections were made before installing the batteries for safe guarding the equipment.

Figure 3-8 Terminal strip

Table 3-3 Data Logger Terminal Strip

Channel Number	Terminal Strip Designator	Terminal Strip Position	Description	Cable Wire Color
1	T1	1H	Vibrating Wire +	RED
1	T1	1L	Vibrating Wire -	BLACK
1	T1	2H	Thermistor +	GREEN
1	T1	2L	Thermistor -	WHITE
1	T1	S1	Analog Ground (shield)	BARE WIRE
2	T2	3H	Vibrating Wire +	RED
2	T2	3L	Vibrating Wire -	BLACK
2	T2	4H	Thermistor +	GREEN
2	T2	4L	Thermistor -	WHITE
2	T2	S2	Analog Ground (shield)	BARE WIRE

Geokon's Log View software provides the versatility of configuring data collection and retrieval options, particularly the option to change time intervals and data collection methodology. The baud rate should be kept at default value of 9600. Making sure that the USB driver is installed and selecting the correct COM port are important troubleshooting activities, if at any point during testing the laptop should be changed.

Table 3-4 shows a partial sample data array. "Sensor Reading1" and "Sensor Temp1" columns in the table represent data for sensor 1. All VWSG readings are recorded in a separate row. The sensor reading value represents the strain in concrete at the VWSG location during the time interval between the readings. Two correction factors need to be applied to convert the sensor reading value to a microstrain (ε_c) reading as shown in Eq. 3-1.

$$\varepsilon_c = d_r \, x \, f_g \, x \, f_v \qquad \text{Eq. 3-1}$$

where; d_r is the sensor reading, f_g is a gauge correction factor, and f_v is a validation error correction factor. The gauge factor for the 4200 model of vibrating wire strain gauge is 3.304 and the validation error correction factor provided by Geokon for the gauges used in this project is 0.98. A value of strain gauge reading of -99999 indicates an error and often occurs when the coil clamp is not properly tightened or the connection of the five different conductors to the terminal strip is not correctly done. Tightening the coil clamp around the coil and gauge bar and checking the terminal strip connections will usually solve the problem unless the gauge itself is damaged. All eleven gauges were tested in the laboratory and after installation at site and were found to give non erroneous and middle range readings (about 2500 microstrain).

Table 3-4 Sample Data File for Strain Gauge Data Logger

Logger Name	Year	Julian Day	Hours and Minutes	Seconds	Battery Voltage	Internal Temp (Degrees Celsius)	Sensor Reading1	Sensor Temp1 (Degrees Celsius)	Array #
LC-2x16	2011	101	1422	12	3.1	23.9	838.28	21.6	8

3.1.7 **Moisture Gauges**

Five Decagon 10HS soil moisture gauges were used to monitor the water content in the aggregate base beneath the PCPS BAS on the east side of Bridge A7767. The 10 HS Decagon moisture gauge (soil moisture sensor), shown in Figure 3-9, measures the volumetric water content (VWC) in one liter volume of soil by measuring the dielectric constant of the soil. Since the dielectric constant of water is much higher than that of air or soil minerals, the dielectric constant of the soil is a sensitive measure of VWC. The sensor measures the average VWC along its length, therefore a vertical installation would integrate VWC over a 10 cm depth while a horizontal orientation will measure average VWC at a constant depth. The VWC of the soil is assumed to be directly related to the possible magnitude of loss of soil support under the slab. In this project moisture sensors were placed 4 inches below the aggregate base that has the purpose of draining the water under the slab.

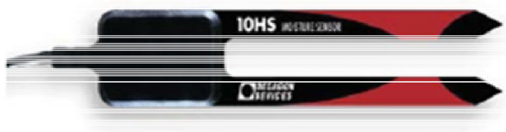


Figure 3-9 10 HS Soil Moisture Sensor

The 10 HS is suitable even when soil heterogeneity is a concern and has an operating temperature from 32 to 122 degrees Fahrenheit (0 to 50 degrees Celsius), which is ideal for conditions at the site of the BAS. The 10HS has a low power requirement and very high resolution. Moisture gauges were tested in the lab prior to the placement in the field. Moisture gauges were embedded in dry sand with a known volume and incremental amounts of water was poured on the sand surface while collecting data from the moisture gauges. A sample test data obtained from one of the moisture gauges is shown in Table 3-5.

Table 3-5 Laboratory Test Results for Moisture Gauge

Moisture Content (m^3/m^3 soil)		
Gauge Recordings		Calculated
(min)	(max)	
0.16	0.16	0.18
0.22	0.23	0.25
0.30	0.30	0.32

3.1.8 **EM 50 Data Logger**

A Decagon EM 50 data logger was used to collect the moisture gauge readings which can read and store data for up to five gauges. Figure 3-10 shows the EM50 data logger. Data was collected at set time intervals and transferred to a laptop using a USB modem at the same time when VWSG data was transferred. The collected VWC data is transferred in Microsoft excel format and application of any correction factors is not necessary since the 10 HS Moisture gauges were pre-calibrated with the EM 50 data logger.

Figure 3-10 Decagon EM50 data logger

3.2 Field Instrumentation of PCPS on MO38

The bridge approach slabs (BAS) for Bridge No A7767 on MO38 in Webster County, Missouri were constructed using precast-prestressed (PCPS) panels. The PCPS panels are 20 feet long, approximately 6 feet in width and 10 inches in depth with a 2 inch asphalt overlay. Performance of the BAS was monitored through visual inspection, field instrumentation and load testing using trucks. The data collected was analyzed and improvements recommended in terms of construction and design. In addition, concrete samples were collected and analyzed to assess the properties of the concrete mixture.

Instrumentation was done only on the bridge approach slab on the west end of the bridge. The instrumentation consisted of Geokon model VCE-4200 vibrating wire strain gauges (VWSGs) and Decagon 10 HS moisture gauges (DMGs). The VWSGs measure the strain and temperature inside concrete. The moisture gauges record the moisture levels in the soil below. The above data was used to check if there was any water leaking into the sub grade soil from between the panels or if there is any retention of water in the sub grade soil below the slab. The VWSGs were embedded inside the BAS panels and DMGs were embedded 4 inches inside the natural soil below a 4 inch sand layer.

Data from the strain gauges and moisture gauges were collected using a Geokon LC2X16 data logger and Decagon's EM 50 data logger. Both the data loggers were anchored to a south end concrete girder under the bridge close to the abutment end. The instrumented panels FS004 to FS008 are shown in Figure 3-11.

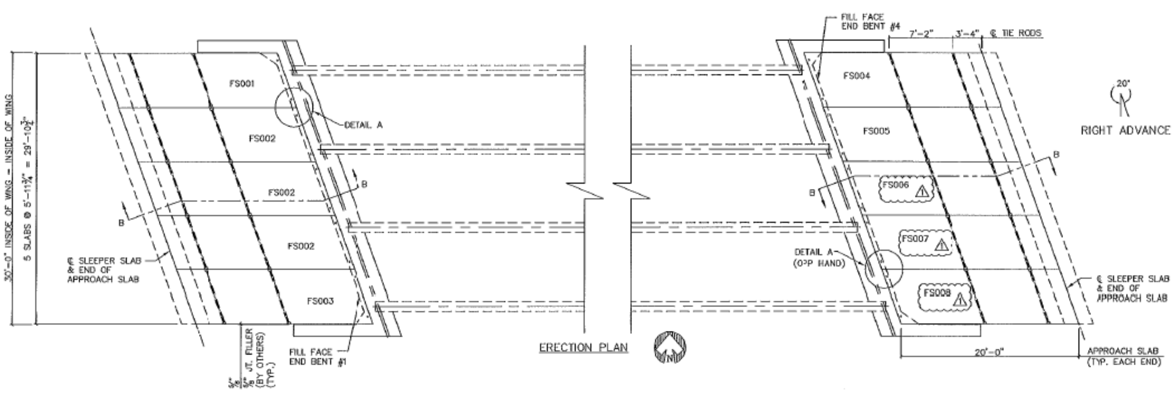
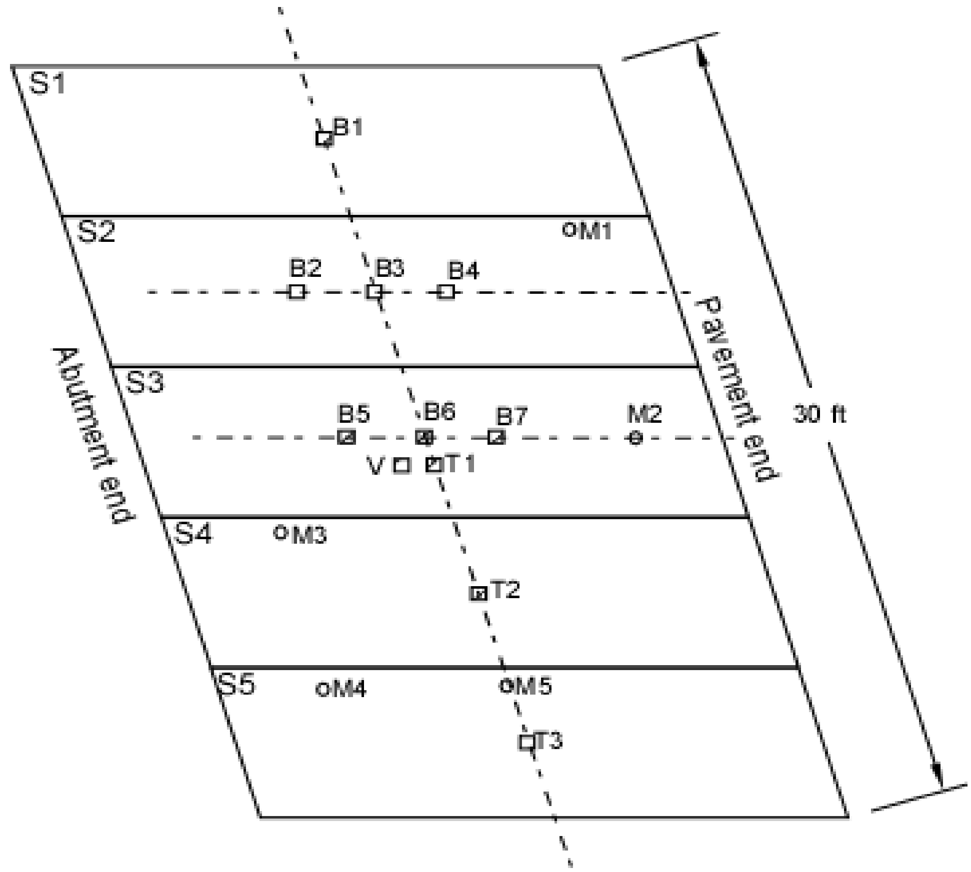


Figure 3-11 Layout of Panels[1]

The instrumentation of the bridge approach slab consisted of eleven Geokon model VCE-4200 vibrating wire strain gauges and five Decagon 10HS moisture gauges as shown in Figure 3-12. Seven strain gauges were anchored to the bottom steel, seven of them being in the longitudinal direction and three in the transverse direction. Most of the gauges placed in the longitudinal direction and at the bottom because the analysis and design showed that tension will be mostly on the bottom face and in the longitudinal direction. Three gauges are anchored to the top steel to measure any tensile stresses at the top face due to swelling of sub-grade soil below the slab panels. The five moisture gauges are placed below each slab panel embedded in the base rock and close to the longitudinal joints, abutment and pavement ends to check if there was any leakage or retention of water. The leakage of water will indicate any disruption in alignment or position of slab panel and the retention of water will indicate failure of drainage system or / and erosion of soil beneath the bridge approach slab.



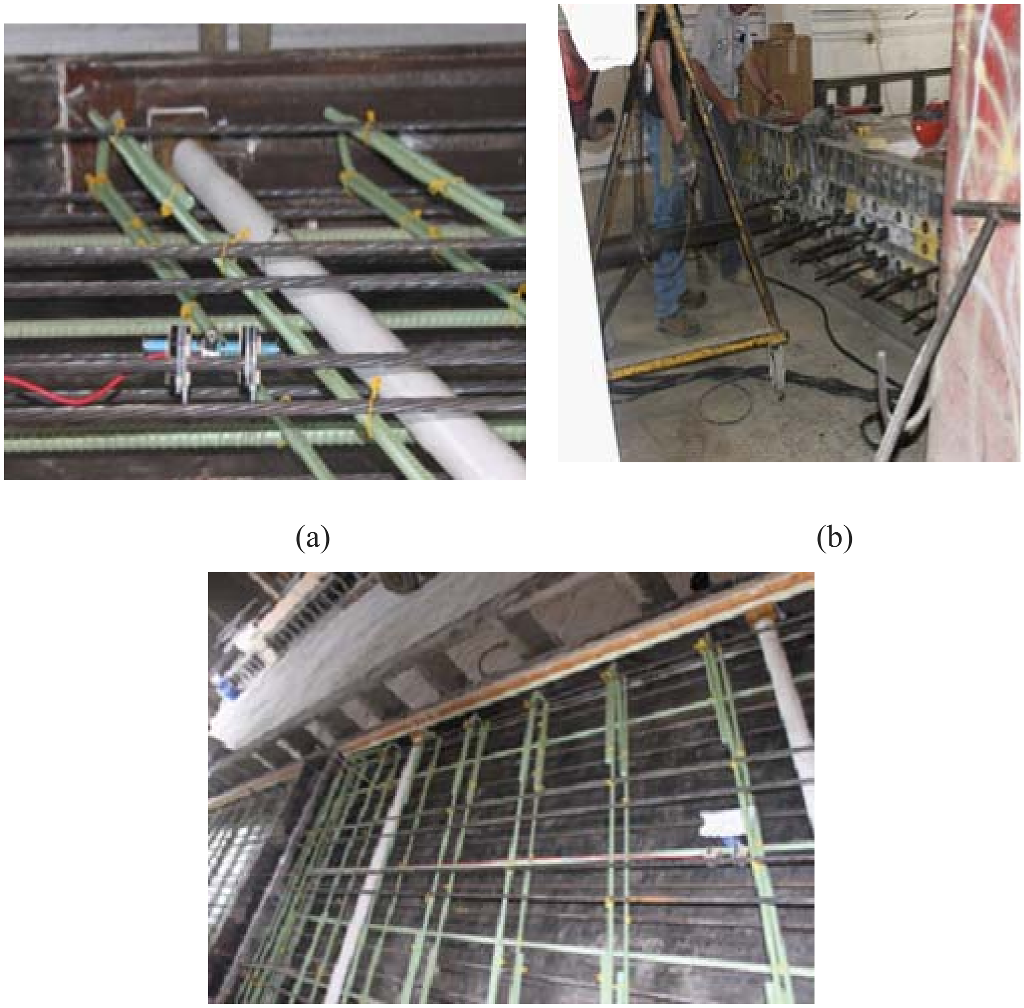
B=VWSG on Longitudinal, T=VWSG on Transverse,

M=Moisture Gauge Location

Figure 3-12 VWSG and Moisture Gauge Locations

3.2.1 **Placement of Strain Gauges**

The gauges were anchored to the steel after the reinforcement was laid out (see Figure 3-13a) and prestress tendons stressed in the formwork (Figure 3-13b). The gauges were anchored as described in Chapter 2. The VWSG-LT-006 gauge was anchored at about 45 degrees angle to allow for proper concreting around the gauge.



(a) (b)

(c)

Figure 3-13 (a) Gauge anchored to steel, (b) Prestressing tendons, (c) Cables tied to steel rebar

The cables were tied to the steel at about every three feet as shown in Figure 3-13c and taken out of the formwork and connected to the data logger for initial readings during concreting. The cables were checked for any damage before concreting of precast panels.

3.2.2 **Concreting of PCPS Slabs**

The concreting was done using a Tuckerbuilt Concrete Transport Vehicle pour truck (see Figure 3-14) and a team of four construction workers spreading and finishing the concrete and one construction worker using an immersion type vibrator (see Figure 3-15). The concrete mix was of a 10 inch slump which made it easy for concreting the instrumented panels. Care was taken by the construction team to pour the concrete away from the gauges first so that the fresh concrete can flow below the gauges and secure them in elevation. The formwork was marked for location of the gauges to avoid damage of gauges during the use of vibrators and other finishing operations (see Figure 3-16). The initial set took approximately 15 minutes and the final set was achieved in approximately 10 hours.

Figure 3-14 Tuckerbuilt Concrete Transport Vehicle

Figure 3-15 Finishing operations on PCPS slab

Figure 3-16 Marking of gauge location on formwork to prevent damage

The first of two concrete pours were for four panels each. The four panels were laid out longitudinally. The prestress tendons were anchored at one end and stressed at the other end of the four panels. The prestress tendons between two panels were cut after the concrete was hardened and gained minimum strength. For the first pour, the data logging of the gauges was stopped before the prestress tendons were cut. For the second pour the data logging was stopped after the prestress tendons were cut to record any changes in concrete strain. The cables were carefully handled during removal of formwork and anchored to the slab panel top surface using zip lock plastic bags and tape. The curing was done using wet burlap. The cables were protected from being wet as they were housed in zip lock plastic bags (see Figure 3-17).

Figure 3-17 Curing of PCPS slabs

3.2.3 **Camber**

The precast panels were measured for camber. Panels FS004 and FS006 showed a maximum of 1/8 inch camber, panels FS005 and FS007 showed a camber of 1/16 inch and panel FS008 had zero camber. The camber was in upward direction and possibly because of less number of prestress tendons in the top surface as compared to the bottom surface of slab panel.

3.2.4 **Placement of Moisture Gauges**

The moisture gauges were embedded into the subgrade soil below the bridge approach slab on August 9 2011. The holes for placing gauges were dug out after placing and compacting (see Figure 3-18) 4 inch layer of type 1 aggregate (type 5 is the typical aggregate base MoDOT requires for roadway but type 1 can be substituted with no effect) on top of natural soil below the bridge approach slab. Rock material was found at most of the gauge locations (see Figure 3-19). The pockets dug out for gauges were filled with natural material. A two inch deep trench was dug out for running the cables out of the subgrade below the bridge approach slab (BAS). All the cables were collected on the north end of the BAS close to the abutment. The trench was filled with type 1 aggregate after running the cables between the probe and abutment (see Figure 3-19).

Figure 3-18 WP 1550 Compactor

Figure 3-19 Embedding Moisture Gauges

Figure 3-20 and Figure 3-21 show the cable and gauge locations of the moisture gauges and Figure 3-22 shows the routing process of all the cables through PVC pipes to the location of the data logger. The data logger for both the strain gauges and the moisture gauges were placed under the bridge adjacent to the outermost girder.

Figure 3-20 Moisture Gauge Cabling

Figure 3-21 Location and Installation of the Moisture Gauges

Figure 3-22 Routing Instrumentation and Data Logger Location

CHAPTER 4 **CONSTRUCTION OF BRIDGE APPROACH SLABS**

This section presents the details of the construction of the five BASs that were implemented on five bridges across the state of Missouri. The implementation included one 20 ft. long prestressed precast BAS with sleeper slab (PCPS20SLP) consisting of 6 feet wide panels, one PCPS20SLP implementation consisting of 8 feet wide panels, one cast-in-place 20 ft. long BAS with sleeper slab (CIP20SLP) and two cast-in-place 25 ft. long BASs without sleeper slab (CIP25NOSLP) implementations. One of the PCPS20SLPs was instrumented with strain gauges and moisture gauges for long term monitoring. The construction details of the slabs and the placement of instrumentation, namely the moisture and strain gauges are described in this chapter.

4.1 BAS Locations

Figure 4-1 shows the locations of the five BAS implemented and studied in this project.

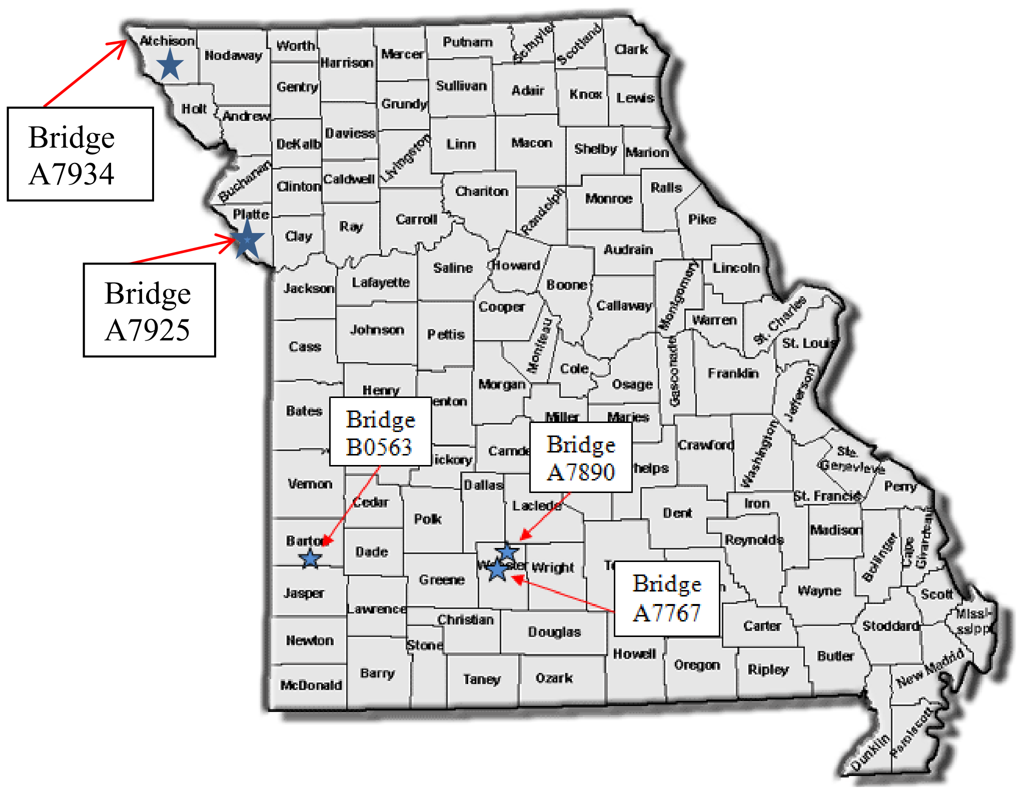


Figure 4-1 BAS Locations in Missouri

4.2 Bridge Approach Slab Construction Drawings

The overall structural details and description of each BAS project can be found in Table 4-1. CIP represents a cast-in-place slab and PCPS represents a precast-prestressed slab, the number 20/25 represents the span of the BAS in feet and SLP/NOSLP represents the presence or the absence of the sleeper slab.

Table 4-1 Reinforcement Summary of Each BAS

BAS Type	Span (ft)	Depth (in)	Cover (in)	Bottom Reinforcement (Main/Distribution)	Top Reinforcement (Longitudinal/Transverse)
CIP20SLP	20	12	3	#6@5" / #5@12"	#4@18" / #5@12"
CIP25NO SLP	25	12	3	#6@8" / #4@12"	#5@12" / #4@12"
PCPS20S LP	20	10	3.5	#6@12" and (12) 1/2" PS Strands / #5@18"	(6) 1/2" PS Strands / #5@18"

The 25 ft. and 20 ft. cast-in-place BAS selections were based on a cost analysis, other DOT design details, and numeric modeling [1]. A 20 ft. by 12 in. thick slab was selected to replace the previous CIP design. The 20 ft. approach slab was designed for a moment demand of 39 kip-ft/ft and rests on a sleeper slab. The CIP slab connects to the abutment with a #5 rebar placed at 12 in. on center. The 25 ft. long BAS design used for this study is based on the MoDOT modified approach slab (MAS). The slab is 12 in. thick and has a design moment capacity of 36 kip-ft/ft. The abutment connection is the same as the new 20 ft. design but is not supported on a sleeper slab. Aside from the abutment connection, the BAS relies only on the support of the soil. A summary of the reinforcement details can be found in Table 4-1.

4.3 MO38 PCPS20SLP Bridge Approach Slab

Twenty feet PCPS panels were selected for the BAS on MO38 spanning Greer Creek. Each panel is 6 ft. wide by 20 ft. long and 10 in. thick. The slab contains a 2.0 in asphalt overlay to provide an adequate riding and wearing surface. The design drawings that were issued to the contractor by MoDOT are provided in Appendix I. Figure 4-2 shows the completed picture of the bridge.

4.3.1 Placement of PCPS Slabs on Subgrade

The PCPS panels were installed at the jobsite on August 9, 2011. Each BAS consisted of 5 panels. The panels were placed on bearing pads on the abutment and sleeper slab to help create a uniform support and to allow for hinging action (see Figure 4-3). The precast panels were placed on the leveled type 1 aggregate using a crane (see Figure 4-4). Two layers of polyethylene sheets were placed between the aggregate (see Figure 4-3) and precast panels on the east end to facilitate sliding without friction between the PCPS slab and the aggregate base. However, the precast panels on the west end were placed directly on top of the aggregate on the west end to allow any leakages of water between panels to seep into the subgrade below. This allowed the moisture gauges to measure any leakages of water from the joints between the panels.

Figure 4-2 Bridge 7767 located on MO38 in Webster County, MO

Figure 4-3 Installation of Plastic Membrane, Bearing Pads, and Joint Filler

Figure 4-4 Placement of PCPS Slab Panel

The instrumented panels were planned to be placed on the east end. However, the moisture gauges were placed on the west end. Because of this and the symmetry between the east and west BAS, this issue was discussed with MoDOT engineers and the instrumented panels were placed on west end so that both the moisture gauge and strain gauge data loggers can be located at one end (west end) of the bridge. During the placement of panel number 1 on east end (see Figure 4-5) it was observed that there were high pockets in the aggregate and because of this the slab panels did not have an even support on the aggregate. The panel and polyethylene sheeting were removed and the aggregate leveled again using a 2” x 6” wooden bar to remove pockets as shown in Figure 4-6. The precast panel was placed on the leveled surface after placing the polyethylene sheet. The lesson learned helped in speeding up the process for the west end.

Figure 4-5 Placement of panel #1

Figure 4-6 Leveling of the aggregate base

The cast in situ construction of the curb on west end did not match the planned dimensions of the set of precast panels Figure 4-7 & Figure 4-8. Hence, the panels were removed after trying to place them initially, and a part of the concrete in the south end wing wall was chipped away to fit the panels between the two curbs. The construction activity was further delayed because the specific washers for connections between transverse tie rods were not delivered by the contractor with the panels. The site contractor's preference was full length tie rods instead of individual tied rods for each panel with connections. This option was discussed with MODOT engineers at the site and it was agreed upon to use the full length option.

Figure 4-7 Alignment of PCPS slab with abutment

Figure 4-8 Placing the last panel

After the installation of the bearing pads and polyethylene sheeting, the PCPS panels were installed and were tied using tie rods in the transverse direction. The tie rod consisted of 1 in. diameter Grade 60 steel (Figure 4-9). After tying the slabs together the keyways were grouted. This helps the BAS function as a single unit without significant movement (deflection and rotation) between each panel. The connection to the abutment was created by drilling holes in the abutment through the preformed holes in the BAS (Figure 4-10), then placing ¾ in. epoxy coated dowel bars through the slab into the abutment. These dowel bars were placed at 24 in. on center and will help to develop a zero moment transfer connection between the BAS and bridge deck. Figure 4-11 shows all the panels placed prior to laying the asphalt layer. Figure 4-2 shows the completed BAS with the asphalt overlay.


Figure 4-9 Picture showing the recesses for transverse tie rods (left) and the tie rod placed in the slabs (right)


Figure 4-10 Drilling Holes in the Abutment to Connect to the BAS


Figure 4-11 Completed Placement of All 5 Panels

The cables from the embedded strain gauges in the precast panels were collected at the sleeper slab end (in the expansion joint) and run under the bridge using PVC conduits. The cables for strain and moisture gauges were run in separate PVC conduits along the curb and back up under the bridge between the 1st and 2nd concrete girders on the north-west end. The level of cables was kept above the planned level of rip rap along the curb to avoid damage to the conduit. The cable conduit connections were sealed with PVC cement. The T pipe joints which collected the cables from individual panels, the open ends of conduit and any other openings were sealed with foam sealant for water tightness (see Figure 4-12 and
Figure 4-13).

Figure 4-12 Sealing PVC Joint Conduits

Figure 4-13 Sealing T Joints and Cable

4.3.2 **Data Collection Equipment**

The data loggers were anchored to the abutment wall below the bridge between the 1^{st} and 2^{nd} girders on the north-west end (see Figure 4-14). Two additional weather proof boxes were also anchored to the abutment wall and the additional spool of cables were housed in the two additional boxes. All the boxes were labeled MoDOT / DANGEROUS / HIGH VOLTAGE to avoid theft during the course of collection of data. The location of data loggers was selected to prevent direct exposure to weather conditions, damage of data loggers and cable conduits during removal of bridge curb formwork and theft. Two one and a half inch holes were made in the storage boxes (one at side and one at bottom) to route the cables from the conduits to the data loggers. The holes were later sealed with foam sealant. The gauge cables were labeled at the end connecting to the data collection equipment (see Figure 4-15).

4.3.3 **Observations During Construction of BAS**

The contractor was advised to cover the east end panels with plastic covers during rainfall to prevent any leakage of water from the panel joints before making the joints water tight with grouting. This would prevent storage of water between the polyethylene sheets and precast panels and thus prevent expansion during winter due to freezing of the water. Gaps were observed between the precast panels and the abutment / curb due to improper alignment with the curbs, particularly on the west end (see Figure 4-16 & Figure 4-17). This gap was planned to be filled with mortar. The inspector suggested that the rip rap may not be placed all the way up to the top level of embankment under BAS. It was also mentioned that the traditional practice is to not place the rip-rap on the downstream end. The strain gauge data was collected at 30 minute intervals and the moisture gauge data at 2 hour intervals for a week and the data was collected and analyzed. The intervals were further set at 45 minutes and 5 minutes for strain and moisture gauges respectively.

Figure 4-14 Data collection equipment

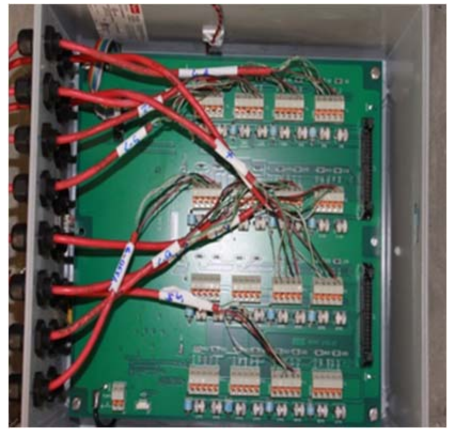


Figure 4-15 Labeling of strain gauge cables

Figure 4-16 Gaps between PCPS & abutment

Figure 4-17 Gaps between PCPS panels

4.3.4 Data Collection Methodology

The strain gauge data was recorded at two minute intervals during the casting of the precast slab panels and up to the development of minimum strength in concrete. This time period includes the hardening of concrete which was approximately ten hours. The strain gauge data were recorded at 30 minute intervals during the placement of the precast slab panels, placement and tightening of tie rods and pouring of the two inch asphalt layer at the site. This data was downloaded and used to estimate the zero reading of each gauge corresponding to zero strain in the gauge. The soil sensor data was recorded during this time to assess if the gauges were functioning properly. The strain gauge and soil sensor data were recorded at fifteen minute intervals during the live load test conducted by MODOT before opening the bridge to traffic. This data was downloaded and used to assess if there was any initial damage to the BAS due to the live traffic loads. The strain gauge and soil sensor data were also recorded at twelve hour intervals over a period of two years. Data was collected and batteries were replaced periodically. The strain gauge and soil sensor data can be recorded at smaller time intervals if required and if any damage is observed in the bridge approach slab either by visual inspection during site visits or by assessment of collected data.

4.4 Monitoring Plate Details

The surveying prisms and inclinometers required a smooth and flat surface for testing. Monitoring plates were constructed to produce this smooth surface for the instrumentation and to provide reference points on the slab to monitor long term performance. The monitoring plates were designed to be embedded in the concrete so the instrumentation reflects the behavior of the slab. Figure 4-18 shows the details of the monitoring plates. Number three mild steel rebar was welded to the plate to prevent the plate from popping off the non-shrink grout due to temperature changes. The plates were then placed in non-shrink grout approximately ½ in. below the pavement surface. This was done to prevent the plates from being destroyed by snow plows and traffic. Prior to installation, the plates were coated with a thin layer of rust prevention spray paint since the plates have direct exposure to water.

The 20 ft. PCPS and 25 ft. CIP BAS monitoring plates were installed after the slabs were constructed so this required a small portion of the concrete to be jack hammered and removed. The plate locations can be found in - for the PCPS slab on bridge A7767. Other drawings showing the location of the monitoring plates for the other two bridges are shown later in this chapter. Each inclinometer had an identification number and was placed in the exact location for each load test to maintain consistency between each test. Figure 4-20 shows the direction of the inclinometers and associated identification number.

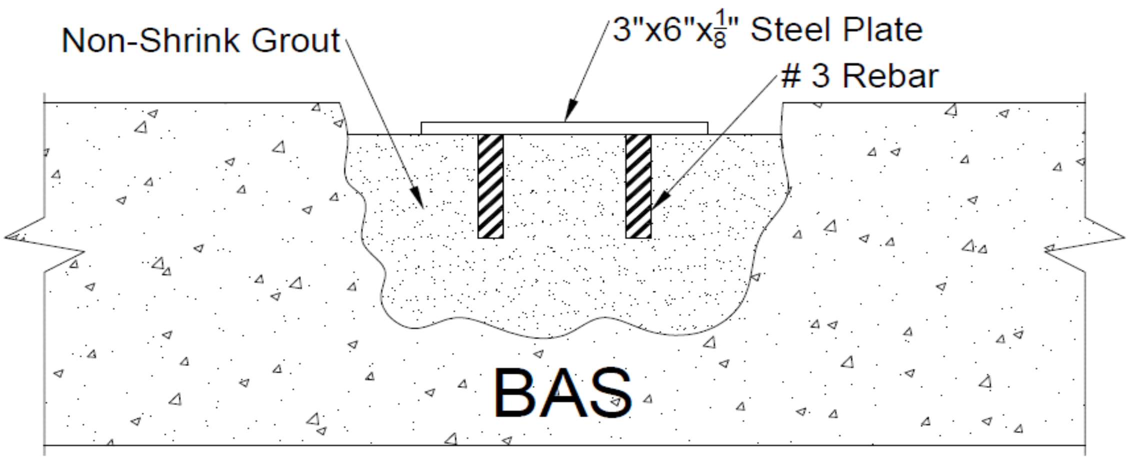


Figure 4-18 Schematic of Monitoring Plates

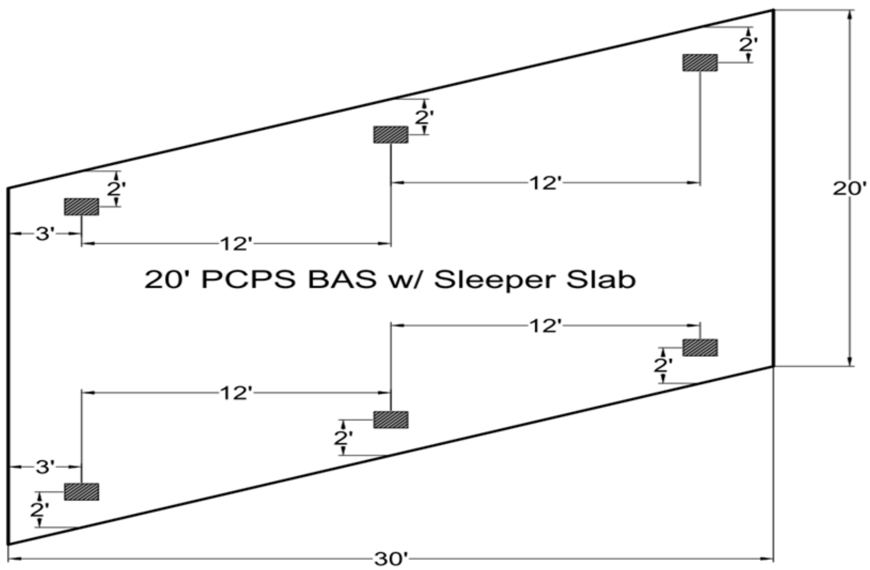


Figure 4-19 Monitoring Plate Locations for the 20 ft. PCPS BAS on Bridge A7767

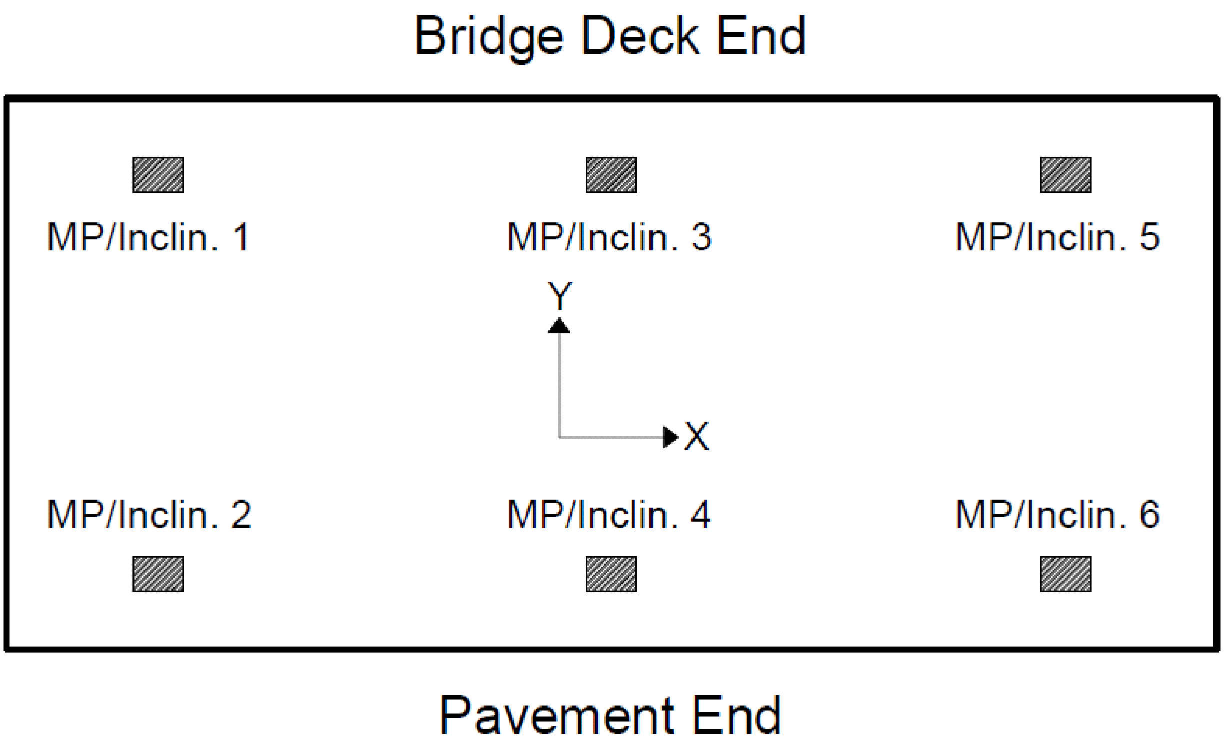


Figure 4-20 Monitoring Plate and Inclinometer Identification Number

4.5 US136 PCPS BAS over Little Tarkio Creek

A 20 ft. PCPS20SLP BAS was selected for the bridge approach slab (BAS) for Bridge No A7934 in Buchanan County, Missouri. Each panel is 8 ft. wide by 20 ft. long and 10 in. thick and has 4 panels on each side of the bridge. This arrangement is in contrast with the MO38 PCPS which had 5 panels that were 6 feet wide on each side of the bridge. This particular arrangement was chosen as the PCPS panel fabricators had indicated that it would be more cost effective to have fewer of the wider panels. The slab contains a 2.0 in asphalt overlay to provide an adequate riding and wearing surface. Design drawings that were issued to the contractor by MoDOT are provided in Appendix II.

After completion of the compaction of the aggregate base the contractors recommended the use of sand or lime to make it easier to level. The large aggregate made it difficult to form a flat surface and the addition of a finer material would help. After the base was level, rubber fabric pads were placed on the sleeper slab and along the bridge deck and wingwall to provide for expansion as seen in Figure 4-21.

Figure 4-21 Placement of First PCPS Panel on US136 on Bearing Pads on Sleeper Slab

Figure 4-21 shows the laying of the first panel at the east end of the bridge. No polythene sheets were specified for this bridge. Figure 4-22 shows placement of the middle panel and Figure 4-23 shows the placement of the last panel. These panels were connected using the shorter transverse tie rods that connect two adjacent panels. It was observed that the construction crew had a difficult time trying to reach in between the panels in order to tighten the nut of the tie rods.

Figure 4-22 Placement of a Central PCPS Panel on Bridge on US136

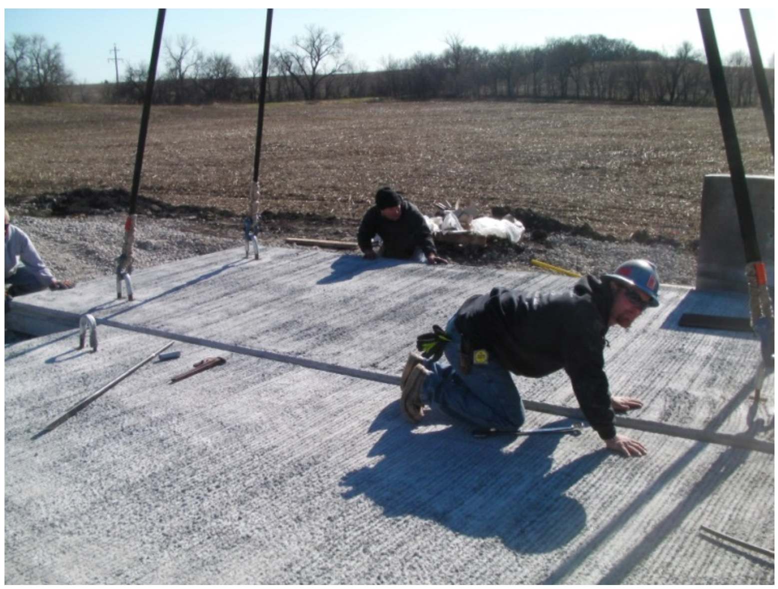


Figure 4-23 Placement of the Last PCPS Panel on Bridge A7934

Placement of the four panels took approximately four hours. Placement of the fourth panel took the longest due to complications aligning the tie rod and access to the key hole for tightening of the hex sleeve shown in the detail drawing. To get the proper alignment the fourth panel was disconnected from the crane and one side of the third panel was connected and lifted slightly so that a construction worker could get his tool between the slabs to tighten the hex sleeve. For this particular job the contractor took three days, the first day to place the panels on the east end, the second day the west end panels, and the third for the overlay. Since it only took a total of eight hours to place and level and connect all eight panels this could be completed in one day if desired.

Based on this observation during construction, the research team felt that using one long tie rod to connect all the panels is a more effective arrangement. The alignment of the holes of all the panels was not an issue due to the QC/QA process in the fabrication facility. The contractor said it took approximately 8 hours to place all eight slabs. Suggestions from the job site include, using one continuous tension rod or using a system similar to precast box girder bridges for the connection between the slabs.

Finally Figure 4-24 shows the laying of the asphalt layer on the east end of the bridge over the PCPS panels as the finished riding surface and Figure 4-25 shows the completed picture of the PCPS system over bridge A7934 on US136.

Figure 4-24 Laying of Asphalt over PCPS Panels on Bridge A7934


Figure 4-25 Completed Picture of the PCPS BAS system over US136

4.6 US71 CIP BAS with Sleeper Slab

The cast-in-place (CIP) design for bridge B0563 is a slight modification from the original MoDOT design. MoDOT's current BAS designs can be found at the following website http//modot.com/business/standard_drawings/approachslab.htm. The as constructed drawings of bridge B0563 are provided in Appendix III. Bridge approach slab (BAS) for Bridge No B0563 in Barton County, Missouri (see Figure 4-26) was constructed using a cast-in-place slab.


Figure 4-26 Bridge B0563 located on US71 in Barton County, MO

The slab is 20 feet long, approximately 38 feet in width and 12 inches in depth. The CIP BASs were constructed in November 2011 for Bridge B0563. The polyethylene sheeting was placed on the base then all metal formwork was constructed. Block outs for mud jacking were placed then the steel rebar cage was constructed. The assembly of the steel cage and formwork prior to pouring is shown in Figure 4-27. The concrete was placed using a hydraulic excavator and concrete bucket as shown in Figure 4-28. As the concrete was poured, a vibratory truss screed was used to finish the concrete surface level with the formwork. Once this was performed, trowels were used to give the BASs its smooth finish. The concrete was then tined using a texture comb to improve skid resistance and traction for motorists upon entering/exiting the bridge. The completed CIP BAS for Bridge B0563 can be found in Figure 4-29.

Six monitoring plates were placed on fresh concrete (see Figure 4-30 and Figure 4-29) for load testing purposes using load trucks and inclinometers. Figure 4-30 shows the schematic of the location of the monitoring plates and Figure 4-31 shows the monitoring plates in this bridge being placed on fresh concrete.

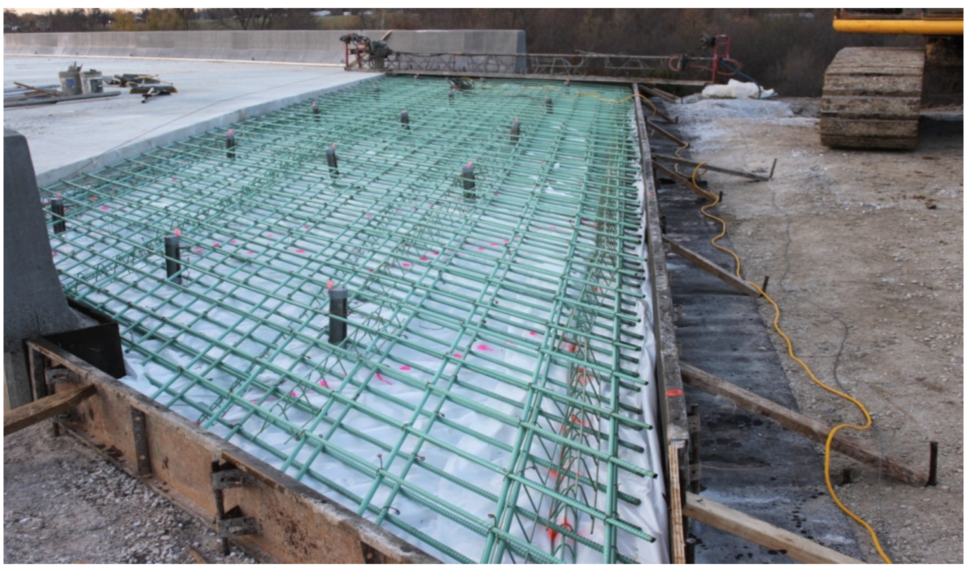


Figure 4-27 Placement of Steel Cage and Formwork Prior to Pouring of Concrete


Figure 4-28 Placement of the Concrete for BAS on Bridge B0563


Figure 4-29 Completed Picture of 20 ft. CIP BAS on Bridge B0563

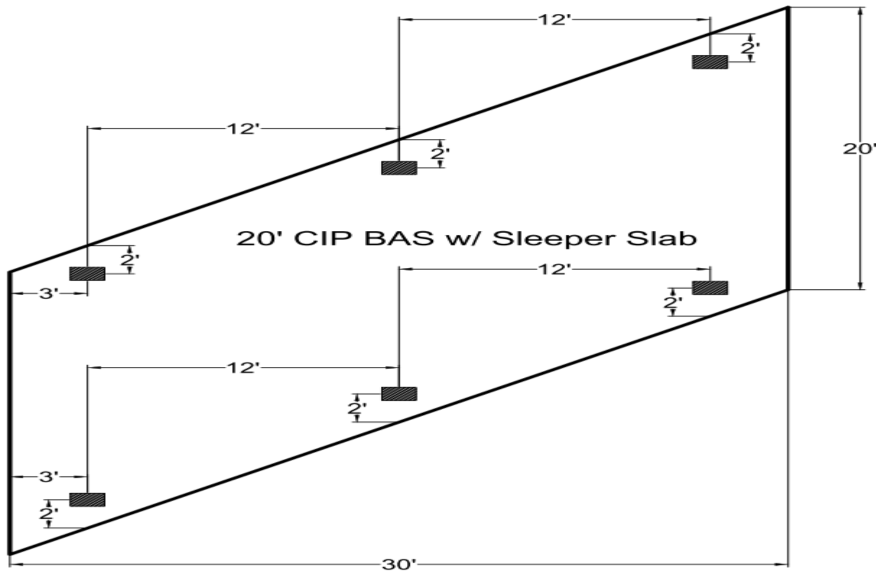


Figure 4-30 Monitoring Plate Locations for the 20 ft. CIP BAS on Bridge B0563


Figure 4-31 Placing Monitoring Plates Over Fresh Concrete on Bridge B0563

4.7 MO38 CIP Bridge over Fork Osage Branch

4.7.1 Slab Details

Bridge approach slab CIP25NOSLP for bridge A7890 in Webster County, Missouri over the Fork Osage Branch was constructed using a cast-in-place slab. The slab is 25 feet long, approximately 42 feet wide and 12 inches deep using reinforced concrete. This BAS rests on the abutment at the bridge end and has no sleeper slab at the pavement end. The bottom longitudinal and transverse reinforcement used are #6 @ 8” c/c and #4 @ 12” c/c respectively. The top longitudinal and transverse reinforcement used are #5 @ 12” c/c and #4 @ 12” c/c respectively. The MoDOT issued drawings of Bridge A7890 are provided in Appendix IV.

4.7.2 **Construction**

The CIP BASs were constructed in November 2011 for Bridge A7890. Due to scheduling issues the research team was not present during the construction of this CIP38NOSLP BAS. However, this construction is a standard BAS CIP construction with no special requirements other than that for the placement of monitoring plates. The locations of the monitoring plates are shown in Figure 4-32. Figure 4-33 shows the constructed photograph of this BAS.

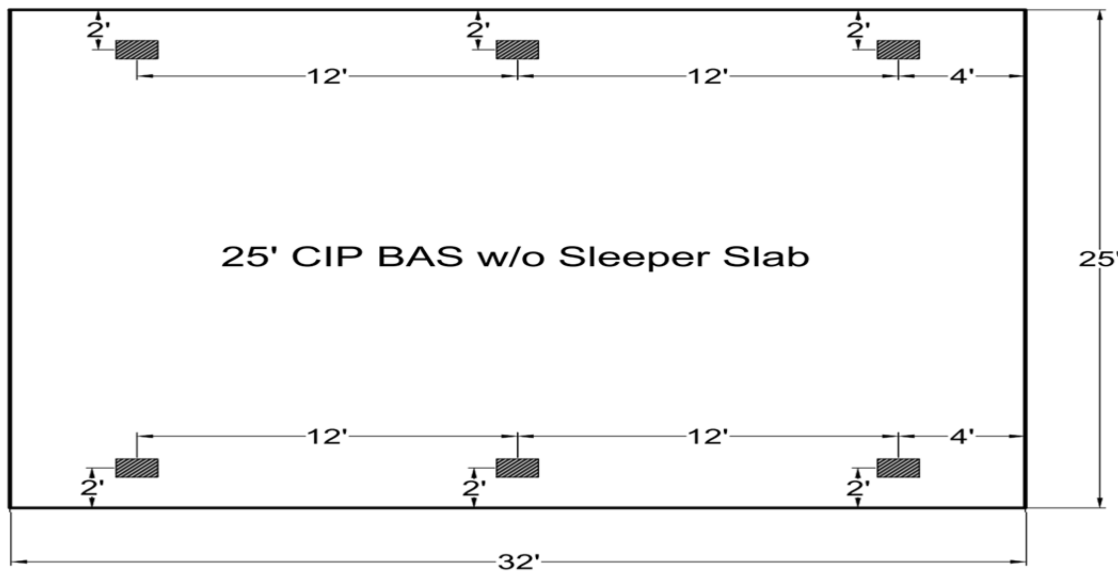


Figure 4-32 Monitoring Plate Locations for the 25 ft. CIP BAS on Bridge A7890

Figure 4-33 Bridge A7890 located on MO38 in Webster County, MO

4.8 MO45 CIP Bridge Over Platte River

4.8.1 **Slab Details**

Bridge approach slab CIP25NOSLP for bridge A7925 in Platte County, Missouri was constructed using a cast-in-place slab (Figure 4-34). It has a 25 foot span, approximately 42 feet wide and 12 inch deep and does not have a sleeper slab. The bottom longitudinal and transverse reinforcement used are #6 @ 8" c/c and #4 @ 12" c/c respectively. The top longitudinal and transverse reinforcement used are #5 @ 12" c/c and #4 @ 12" c/c respectively. At the pavement end of the slab additional #4 bottom transverse rebar were used at 3" c/c. The bridge approach slab was placed on type-1 aggregate base. The schematic view for the slab is provided in Appendix V.

Figure 4-34 shows the constructed photograph of this BAS. One observation that was made on this particular BAS was the loss of some support soil under the BAS after the completion of the electrical work as seen in Figure 4-35.

Figure 4-34 Completed View of the 25 ft. CIP BAS with No Sleeper Slab on MO45

Figure 4-35 Partial Loss of Soil Support at the Edge of the BAS on the West End of MO45

4.9 Base Preparation

The base material beneath the BAS provides a uniform elastic support during service loading and load testing. It is important that all bases and embankment fills are prepared according to specifications to ensure the BASs maintain long term uniform soil support. Soil borings were provided by MoDOT and a soil profile under each BAS was constructed as seen in Figure 4-36, Figure 4-37& Figure 4-38.

4.9.1 **Aggregate Base**

Bridges A7767 and A7890 both have a 4 in. MoDOT Type-1 aggregate base specified in section 1007 of the MoDOT specifications. The current 2011 MoDOT Specifications can be found at the following website http//www.modot.org/business/standards_and_specs/BEGIN.pdf. Bridge B0563 has 4 in. of Type-5 aggregate base beneath the BAS. The Type-5 base has an additional requirement compared to the Type-1 base (type 5 is the standard MoDOT requirement but type 1 has been substituted with no effect). The Type-5 base requires that 0%-15% by weight of the aggregate must pass through the no. 200 sieve. All other gradation requirements are the same for both bases.

The bases under each BAS were compacted using a Wacker WP 1550 vibratory plate compactor as shown in Figure 4-39. The compactor has a compaction depth up to 12 in. and a vibration frequency of 6,000 vibrations per minute. These specifications were used when conducting the lab study on the PCPS panel outlined in a later chapter.

4.9.2 **Embankment Fill**

The embankment fill for the bridges located on MO38 highway was primarily a brown lean clay with cobbles and gravel. The embankment fill for Bridge B0563 located on U.S.71 highway had similar soil characteristics except it contained sand instead of cobbles. The embankment fill was compacted to 95% of the soils maximum density using a Caterpillar CP-433C single drum vibratory compactor. The vibratory compactor compacted the fill in 8 in. lifts as specified by MoDOT specification 203.4.16. Once the embankment fill was complete, 4 in. of aggregate base was placed on the surface.

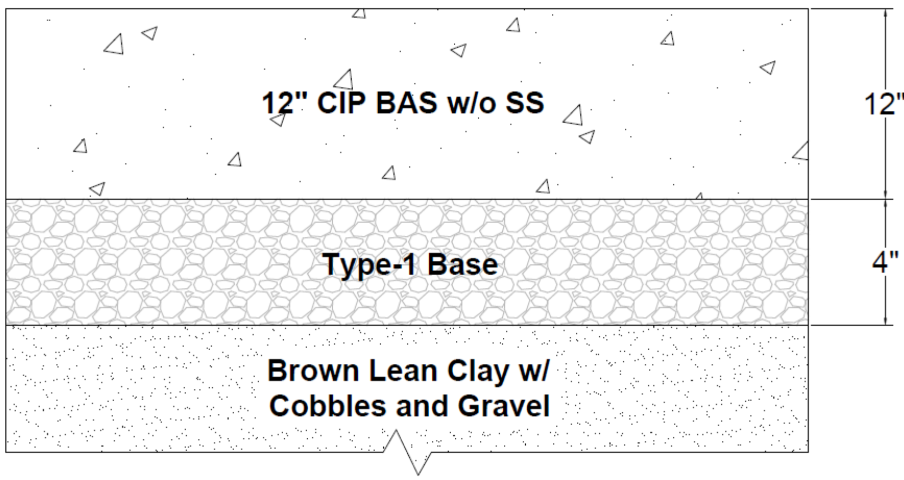


Figure 4-36 Profile of soil under BAS for bridge A7890

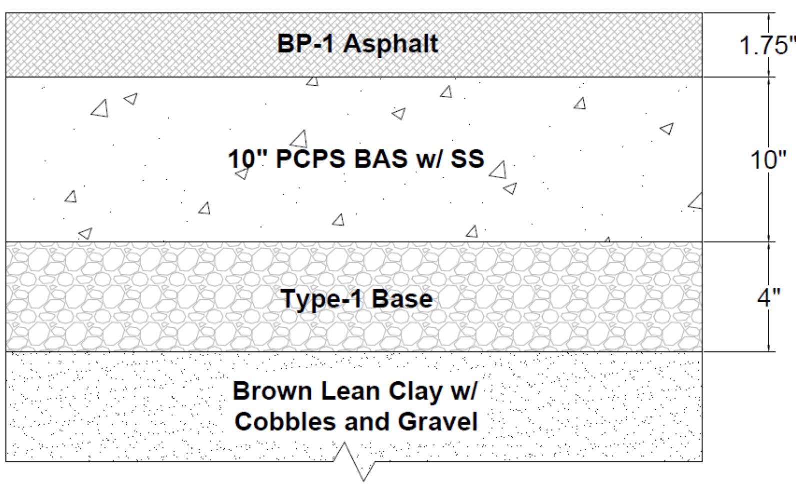


Figure 4-37 Profile of soil under BAS for Bridge A7767

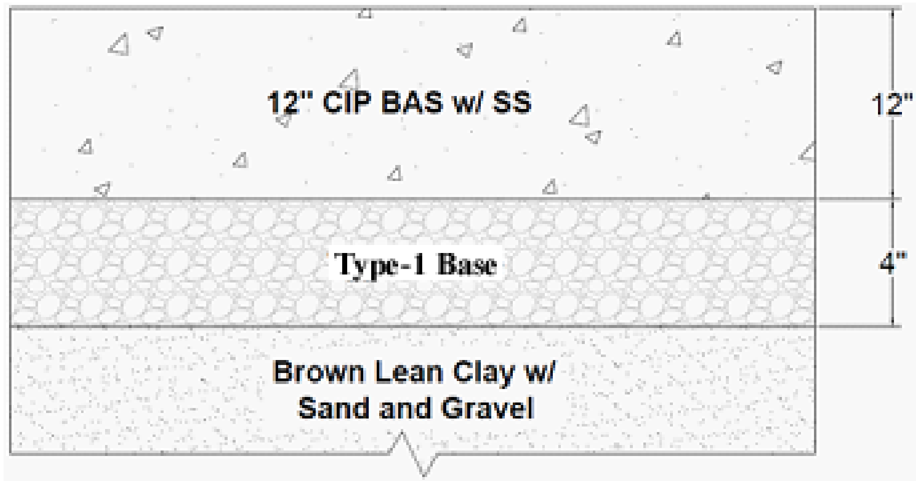


Figure 4-38 Profile of soil under BAS for bridge BO563

Figure 4-39 Compaction of Aggregate Base under PCPS BAS

4.10 Summary

The construction and installation of the new innovative PCPS BAS required additional attention compared to the CIP designs. Proper leveling of the aggregate base to prevent any high or low points was found to be very important to prevent movement of the PCPS panels. Properly leveled aggregate base provided uniform bearing support under the PCPS BAS. The new PCPS BAS system required more inspection during installation to ensure the slab had uniform support from the base. It was also found that using a single tie rod to connect all the PCPS panels in the transverse direction was the most effective connection compared to shorter ties that connected two panels.

CHAPTER 5 **COST ANALYSIS**

This chapter provides a summary of construction costs of new BAS designs implemented in this study on five different projects. The new BAS designs implemented in this study were developed based on an extensive survey review of BAS costs of different state DOTs. The design procedure was completed considering cost as an important factor and initial cost estimates of the new BAS designs were calculated [1]. Although the cost of cast-in-place (CIP) designs could be estimated using the MoDOT unit cost data, the precast-prestressed (PCPS) BAS design cost could not be estimated using historical data. Therefore the research team worked with a precast concrete manufacturer to come up with cost estimates for the proposed PCPS BAS design. Table 5-1 shows the cost of BAS designs currently being used by MoDOT and the estimated costs of the new BAS designs implemented in this study. The costs shown are total labor, material, and equipment costs for the BASs complete in place including the standard base preparation but they don't include the cost of sleeper slabs. The cost of a sleeper slab does not change based on the BAS design and the approximate cost of two, 3 x 38 x 1.5 ft., sleeper slabs is approximately $9,000 complete in place, based on MoDOT unit costs including overhead (5%) and profit (10.5%).

Table 5-1 Estimated Construction Cost of Current and New Designs[1]

BAS	MoDOT	Estimated Cost of Proposed BAS
20' CIP BAS		$43,389[3]
25' CIP BAS	$55,316[2]	$45,375
20' PCPS BAS		$38,584[3]

[1]All costs are for 2 BASs (both sides), 38 feet wide slabs, and include 5% overhead, 10.5% profit, and base preparation
[2]Standard MoDOT design
[3]approximately $2,520 should be added for additional 5 ft of pavement construction on both sides of the bridge ($40-80/yd^2 of extra pavement) to account for length differences between different BASs.

MoDOT also has a 25 ft CIP design with reduced reinforcement that they use on minor road systems without a sleeper slab. The cost of this modified approach slab is $45,336 including same profit and overhead markup. The cost of the standard MoDOT design shown in Table 5-1 and the cost of the modified approach slab both don't include the sleeper slab and the difference is mainly due to reduced reinforcement. The cost estimate shown for the 20 ft PCPS BAS is based on a unit price of $17.25/ft^2 and is for 10 inch thickness. The estimate in Table 5-1 also includes estimated costs of delivery to a site within 100-150 miles radius from the precast plant, installation on site, base preparation, and a 2 inch HMA overlay.

5.1 Bridge B0563 – 20 ft. CIP BAS

The new 20 ft. CIP BAS design developed by the researchers was used on Bridge B0563 on US71 in Jasper County, MO over the M&NA railroad. The bridge and the approach slabs are skewed 45º and the BASs are 38 ft. wide. The BAS was constructed by a local subcontractor. Epoxy coated reinforcement was used for bottom and top mats and plastic cylindrical inclusions were used to allow mud-jacking of the slab later if necessary. The slab was cast using a 3 yard concrete bucket attached to a Komatsu PC220LC hydraulic excavator and a crew of 6 workers including a foreman. The concrete bucket was filled directly from the ready mixed concrete trucks using the chute about 20 ft. away from the formwork and were lifted and transported to the formwork using the hydraulic excavator. A gas electric generator was used to power a concrete vibrator and vibrating screed was used to place the concrete. A broom finish was applied to the surface for a rough surface after placement of the monitoring plates. The monitoring plates were used later to place inclinometers during static load testing. The finished BAS was sprayed with a liquid curing compound and was not covered. Figure 5-1 shows construction equipment used for the BAS on bridge B0563 on US71.

Figure 5-1 Construction of bridge B0563

The cost of construction was \$240/yd^2 which for two BASs came to a total cost of \$41,760. This cost does not include the cost of base preparation. The estimated cost of the 20 ft. CIP BAS was \$43,389, as shown in Table 5-1, including a cost of \$1,467 of base preparation. This indicates that the actual cost of the new 20 ft. CIP design (\$41,760) was very close to the estimate without base preparation (\$41,922). If the estimated base preparation cost of \$1,467 is added to the actual cost of \$41,760 and compared to the current standard MoDOT BAS cost (\$55,316), the new design saved approximately 22% of the initial cost.

5.2 Bridges A7890 and A7925 – 25 ft. CIP BAS

The new 25 ft. long CIP design was implemented on bridges A7890 and A7925. Both bridges are on minor road systems and the BASs were not supported on sleeper slabs. Due to scheduling difficulties the research team was not able to observe the construction of these bridges. Bridge A7890 is over Fork Osage Branch on MO38 and the width of the BAS is 34 ft. The actual cost figures submitted by the contractor for two BASs including the base preparation are shown in Table 5-2. Based on the total cost of $43,292, the actual unit cost of the design was approximately $230/yd^2. Using this unit cost the total BAS cost can be adjusted for a 38 ft. wide bridge to compare it with the estimated cost shown in Table 5-1. The adjusted cost would be $48,385 which is $3,010 (≈ 6.6%) higher than the estimated cost. Even at this slightly higher actual cost the savings compared to the standard MoDOT BAS for a 38 ft. wide bridge is approximately 12.5% of the initial cost.

Table 5-2 Cost of Two 25 ft. CIP BAS without Sleeper Slab for Bridge A7890 on MO38

Item	Cost
Labor	$9,060.00
Permanent Materials	$26,320.00
Consumable Materials	$312.00
Equipment Cost	$2,600.00
Subcontractor Cost	$5,000.00
Total Cost	$43,292.00

The second 25 ft. long BAS was implemented on Bridge A7925 in Platte County, MO on MO45. The width of the BAS was 34 ft. Table 5-3 shows the reported cost figures for two BASs of this bridge, which indicates that the total cost of BASs was $33,822. Based on the area of the BASs this cost would results in a unit cost of approximately $188/yd^2 which is significantly lower compared to the CIP unit costs of the other 25 ft. BAS and the 20 ft. CIP BAS. Adjusting the reported total cost for a 38 ft. width based on the calculated unit cost would result in a total cost of $39,688. Compared to the $55,316 estimated cost of a standard MoDOT BAS for a 38 ft. wide bridge, the new design resulted in a cost savings of approximately 28.3% of the initial cost.

Table 5-3 Cost of Two 25 ft. CIP BAS without Sleeper Slab for Bridge A7925 on MO45

Item	Unit cost (yd^2)	Cost (for both slabs)
Labor	$42.09	$7,577.00
Material	$87.55	$15,759.00
Equipment	$16.40	$2,953.00
Subcontract	$41.86	$7,533.00
Total Cost	$187.90	$33,822.00

5.3 Bridges A7767 and A7934 – 20 ft. PCPS BAS

The new 20 ft. PCPS BAS design was implemented on Bridges A7767 and A7934. BASs on both bridges are supported by a sleeper slabs at the approach pavement end. Bridge A7767 is in Webster County, MO on MO38 over Greer Creek. The PCPS panels used to construct the east side BAS of this bridge were instrumented with VWSGs at the precast plant and also moisture gages were placed in the embankment fill under the BAS during construction.

The BASs of bridge A7767 are skewed 30º and they are 30 ft. wide consisting of five 6 ft. wide PCPS panels on each end of the bridge. The PCPS panels were produced at CORESLAB's Marshall Plant in Missouri and delivered to the site on flatbed trucks. The PCPS panels were placed using a 33 ton rough terrain crane with a 92 ft. boom directly from the truck to the aggregate base covered with polyethylene sheeting. Figure 5-2 shows placement of PCPS panels. Placement of the panels had to be stopped on the east side after the placement of the first panel because of uneven support caused by high spots in the aggregate base. The panel and polyethylene sheeting were removed and the aggregate base was re-leveled using a 2x6 between the abutment and sleeper slab. During the placement of panels on the west end, the south side wing wall had to be chipped to fit the panels between the wing walls.

Figure 5-2 PCPS Panel placement on Bridge A7767

The PCPS panels were anchored to the abutment using dowel bars. The holes for the dowel bars were precast into the PCPS panels, however the abutment was drilled after placement of the panels and non-shrink grout was used to fill the holes. Drilling the abutment after the placement of the panels was preferred to eliminate alignment problems. In the transverse direction the panels were tied using 1 inch continuous tie rods with 6 inch threads at the ends. The outer panels had built in recesses to tighten the rods and to be filled with non-shrink grout. The 30 foot wide BASs were covered with a 2 inch asphalt overlay to match the crown of the pavement. Table 5-4 shows the cost of the two BASs as reported by the contractor.

The total cost of $50,855.48 includes the cost of the sleeper slabs and base preparation including drainage pipe installation which was not considered in the estimated cost of the slabs shown in Table 5-1. Although the cost of two sleeper slabs estimated using unit costs obtained earlier from MoDOT was approximately $9,000, the bid item list obtained from MoDOT for this project showed the cost of two sleeper slabs to be approximately $4,000 including overhead and profit. Subtracting the cost of the sleeper slabs the actual cost of the PCPS BAS ($46,855) is $8,271 higher than the estimated cost of $38,584. As stated earlier this cost includes some more detailed base preparation work including the cost of drain lines whereas the estimated cost included only material and placement cost of the aggregate base. However, the biggest difference comes from the cost of the PCPS panels. The delivered cost of only the PCPS panels in the estimate (for the width of this bridge) was $22,080 for which the actual cost was $35,544. The big difference in the costs can be explained by a risk premium included in the price because of the novelty of PCPS BAS construction. The researchers believe that the bid cost will go down in the future as contractors get used to this construction method of BASs. However, it should be noted that even after this unexpectedly higher cost of the PCPS panels, the actual total cost of the PCPS BASs in this job was lower than the estimated cost of a standard MoDOT BAS of $55,316.

Table 5-4 Cost of Two 20 ft. PCPS BAS with Sleeper Slab for A7767 on MO38

Item	Description	Rate / Hour.	Hours	Total
Labor	Placement of panels, sleeper slab, and base	$40.85	180	$7,353.00
Equipment		$47.84	72	$3,444.48
Materials	6 ft PCPS slab panels from CORESLAB Structures			$35,544.25
Materials	Aggregate base, waterproofing, grout and miscellaneous materials			$4,513.75
Total Cost				$50,855.48

The second bridge, A7934, which implemented the 20 ft. PCPS BAS design, is in Atchison County, MO over Little Tarkio Creek. The BASs were 32 ft. wide. The most important difference between the two implementations of the 20 ft. PCPS design was the width of the PCPS panels used. The A7767 bridge used five 6 ft. wide PCPS panels to construct the BASs but in this project the BASs consisted of four 8 ft. wide panels. The lower number of total panels allowed the precast producer to cast all the panels in two casts instead of three, which had an important effect on the cost. Similar to the A7767 project the PCPS panels were lifted and placed using a crawler crane on compacted and leveled aggregate base. Rubber pads were placed on the abutment and on the sleeper slab prior to the placement of the PCPS panels. Placement of the four panels for one BAS took approximately four hours. Placement of the fourth panel took the longest due to complications aligning the tie rods and accessing the key hole for tightening of the hex sleeve. The fourth panel had to be disconnected from the crane and panel three had to be reconnected to slightly lift the panel. This allowed proper alignment and allowed a construction worker to get his torque wrench between the slabs to tighten the hex sleeve. Figure 5-3 shows the placement of the first PCPS panel and the tie rod holes. The contractor used a crew of five people in addition to a foreman and crew operator to place the slabs and placement of both BASs took approximately 8 hours.


Figure 5-3 Placement of the first PCPS panel on A7934

Table 5-5 shows the contractor provided costs for the construction of BASs on Bridge A7934 on US136. As shown in the table, the researchers were able to get the most detailed cost breakdown from this project. The cost figure includes the cost of sleeper slabs, base preparation, all activities after placement of PCPS panels such as anchoring to abutment, grouting, waterproofing, and placement of 2 inch asphalt overlay. Table 5-5 shows a material cost of \$3,004 for the sleeper slabs and a total labor cost including the labor cost of sleeper slabs. Assuming the in-place cost of sleeper slabs as \$4,000 similar to the sleeper slabs of the A7767, cost of the BASs was approximately \$53,510 which is lower than the standard MoDOT BAS.

5.3.1 **Cost of PCPS Panels**

Comparison of the cost of PCPS panels between the A7767 and the A7934 (Table 5-4 vs. Table 5-5) clearly shows that the cost of the PCPS panels went down as expected for the second project. In addition to lower perception of risk due to repeating a similar project, use of wider PCPS slabs that could be cast in a shorter amount of time affected the price of the PCPS slabs and it went from \$35,544 to \$24,249. It should be noted that the width of BASs on A7767 was 2 ft. shorter compared to the BASs of A7934.

Table 5-5 Cost of Two 20 ft. Precast-prestressed BAS with Sleeper Slab for A7934 on US136

Item	Description	Cost
Materials	8 ft. PCPS panels	$24,248.50
	Water proofing (including hot pour tar)	$1,078.19
	1 in. tie rod assemblies	$1,028.00
	3/4 in. anchor dowels	$72.00
	Nonshrink grout	$208.00
	1/2 in. joint filler	$162.50
	Silicone sealant	$140.00
	4 in. drain pipe	$174.00
	Polyethylene sheeting and tape	$382.00
	1/2 in. continuous rubber pads[1]	$11,400.00
	Aggregate base	$730.00
Sleeper slab	Reinforcement	$1,204.00
	Concrete (15 CY)	$1,800.00
Subcontract	2 in. asphalt overlay	$1,349.00
Equipment	Crane	$3,000.00
Labor	Preparation, pouring of sleeper slabs, placement of slabs, grouting and anchoring of slabs, tar and water proofing	$10,533.81
Total Cost		$57,510.00

[1]This costs were submitted by the contractor who noted that the supplier probably charged them a high price due to the long length of the pads. <u>An alternative solution using plain elastomeric pads will likely provide equal benefits at much lower cost.</u>

5.4 Summary

Table 5-6 Cost[1] of BAS Designs Implemented in this Project shows the recorded cost of BASs on the five bridge projects implemented in this study. The costs are for BASs with different widths and lengths and don't include the cost of sleeper slabs but include the cost of base preparation for the BASs. Column 5 of Table 5-6 shows for each project the percent decrease in cost compared to $55,316 which is the estimated cost for two standard 38 ft. wide BASs. Results clearly show that the new CIP solutions proposed to MoDOT have lower costs compared to the current standard BAS. Assuming the new CIP designs perform as predicted by the numerical analysis and provide a service life equal or better than the service life of the standard BAS design, MoDOT can realize significant cost savings by switching to the new proposed BAS designs. The cost savings for the proposed PCPS BAS design is not as clear as the CIP solutions due to big differences in panel costs and total construction costs between the two implementation projects. However, results clearly indicate that the cost of the new PCPS BAS solution is not higher compared to the standard BAS design. Assuming a similar initial cost, if the PCPS BASs provide a longer service life with lower maintenance costs as expected, they can provide lower life cycle costs to MoDOT. Compared to CIP construction, PCPS panels are produced in a controlled environment with better construction quality control and they are designed to support the traffic loads even in case of 75% base support loss due to erosion. In case of simple settlement without cracking the PCPS BASs can be quickly brought back to level using an asphalt wedge since they are overlaid with a 2 inch asphalt wearing surface. Although there are no significant differences in initial cost for new construction, this project shows that the PCPS BAS solution could be a very advantageous alternative for replacement of existing BASs. Both implementation projects showed that with a stricter dimension tolerance control, straight tie rods, and better base preparation, PCPS slabs can be placed in a 4 hour period using a small capacity crane and a small crew. Considering demolition of existing slab and 2 inch asphalt overlay activities, an existing BAS can be replaced in a day and opened to traffic with the PCPS BAS system. In the case of CIP BAS, even if formwork, steel placement, and pouring is done in one day, MoDOT highway specifications section 502 requires the concrete to reach 3000 psi before it can be opened to traffic. Strength gain depends on site and curing conditions, however considering that conventional 4000 psi concrete gains 60-70% of its strength at 7 days, we can assume curing of at least 5-7 days would be necessary before opening the CIP BAS to traffic.

Table 5-6 Cost[1] of BAS Designs Implemented in this Project

Bridge No.	Description	BAS type	Cost	Decrease (%)
B0563	US71 SB over MNA Railroad	20' CIP with sleeper slab	$41,760	22[2]
A7934	US136 over Little Tarkio Creek	20' PCPS with sleeper slab (8ft panels)	$53,510	3.3
A7890	MO38 over Fork Osage Branch	25' CIP w/o sleeper slab	$43,292	12.5[3]
A7925	MO45 over Platte River	25' CIP w/o sleeper slab	$33,822	28.3[4]
A7767	MO38 over Greer Creek	20' PCPS with sleeper slab (6ft panels)	$46,855	15.3

[1]Costs do not include the cost of sleeper slabs for comparison purposes though they may have been used as part of BAS designs as indicated

[2]with the added estimated base preparation cost of $1,467

[3]when cost is adjusted to $48,385 for 38 ft width instead of actual 34 ft

[4]when cost is adjusted to $39,688 for 38 ft width instead of actual 34 ft

CHAPTER 6 **FIELD LOAD TESTING, STRAIN AND MOISTURE DATA**

Changes in the longitudinal BAS slope, differential deflections, internal concrete strains and moisture conditions under the slab can occur over time due to the settlement of the embankment fill causing the BASs to perform poorly. For this study, visual inspections along with inclinometers, high precision surveying equipment, vibrating wire strain gauges (VWSG) and moisture gauges helped monitor performance of three different BAS. Firstly, a static live load test was performed shortly after the BASs were installed and approximately six (6) months after the slabs were opened to traffic. One bridge B0563 on US71 was load tested prior to its opening while the PCPS bridge A7767 and the 25 feet CIP BAS on a minor road Bridge A7890 on MO38 were field load tested both before and after their opening. This chapter outlines the load test setup and different loading cases used to evaluate the BASs performance.

Furthermore, a total of eleven VSWGs were installed in several PCPS slab panels on A7767 to monitor the long term strains over a period of approximately 18 months. Strains were also recorded during one field load test. In order to evaluate the long term conditions under the approach slabs 5 moisture gauges were installed under the PCPS slabs in the base rock on one side of the bridge A7767. This chapter describes and presents results from the data recorded during this time period. Finally, observations based on visual inspections are also presented at the end.

6.1 Field Load Testing

The initial live load testing of the BASs occurred on November 29, 2011 for Bridge B0563 on US71 and on December 09, 2011 for Bridges A7767 and A7890 on MO38 respectively. The load test provided baseline values to compare the long-term performance and to provide a better understanding of each BAS design when it has full elastic soil support. The second round of live load testing occurred on June 05, 2012 for Bridges A7767 and A7890 on MO38, approximately six (6) months after the first load test. The first load test used unloaded dump trucks and for the second load test the dump trucks were filled with approximately 10.5 tons of aggregate. The two different load tests helped show the influence of increased loading on each BAS. Table 6-1 shows the weights of the HS20 trucks used during both the load tests.

Table 6-1 HS20 Dump Truck Axle Loads

		Load Test 1	Load Test 2
Truck 1 (Kips)	Rear Axle	14.44	31.20
	Front Axle	12.16	11.84
	Total	26.60	43.04
Truck 2 (Kips)	Rear Axle	12.80	30.89
	Front Axle	11.04	11.72
	Total	23.84	42.60

Truck 1 for load test 1 had a higher front axle load compared to truck 2. Truck 1 used in load test 1 had a snow plow installed leading to a higher front axle load. Dimensions of the trucks used for the load test is shown in Figure 6-1.

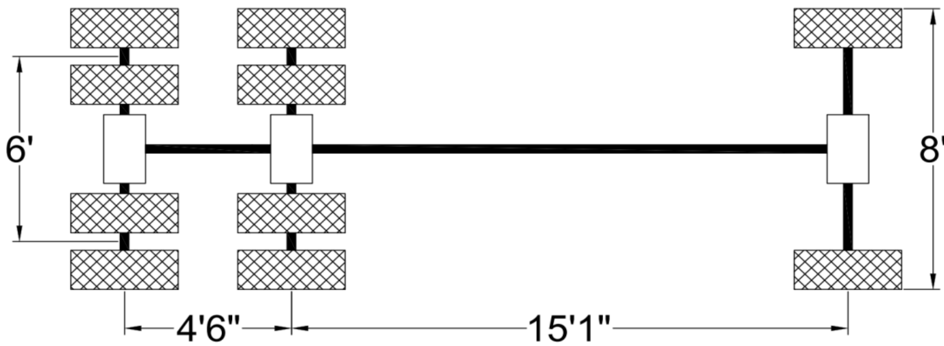


Figure 6-1 Dimensions of HS-20 dump truck

The load test was conducted using two HS20 MoDOT dump trucks as shown in Figure 6-2 and Figure 6-3. Each load test occurred in the lanes where the BASs experience service loading. This provided a uniform test between all three (3) slab designs and represented actual load locations once the bridge was opened to traffic.

Figure 6-2 Overloading left lane for the BAS on Bridge A7890

Each BAS was loaded at four (4) different locations to maximize the movement within the slab. Load case 1 placed the tandem load just on the edge of the BAS near the abutment. Since the tandem carries the majority of the load, this loading case was intended to show rotations in inclinometers 1, 3, and 5 as a truck would enter/exit the approach slab. Load case 2 was performed by placing both tandem loads on the left lane of the BASs. The tandem loads were placed near the longitudinal center of the lane to create an overloaded condition. The rotational behavior of the overloaded lane could be observed in the longitudinal and transverse directions. Load case 3 was designed the same as load case 2 except both tandem loads were placed on the right lane. Similar to load case 1, load case 4 was designed to create rotations in the BAS near the flexible pavement. Load case 4 determined how much rotation the slabs experience based on the support condition (two designs were supported on sleeper slabs while one design relies on the full elastic support of the soil). The diagrams and dimensions of the loading cases mentioned above can be found in Figure 6-4 to Figure 6-6.

Figure 6-3 Tandem loads placed near the flexible pavement on Bridge A7890

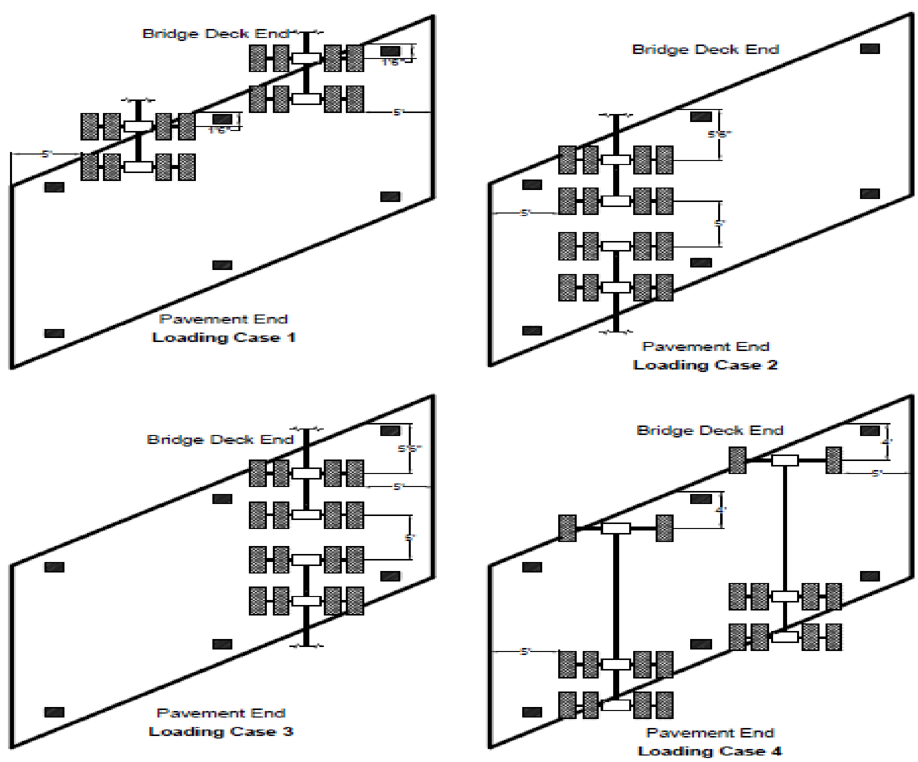


Figure 6-4 Load Cases and Truck Locations for Bridge B0563

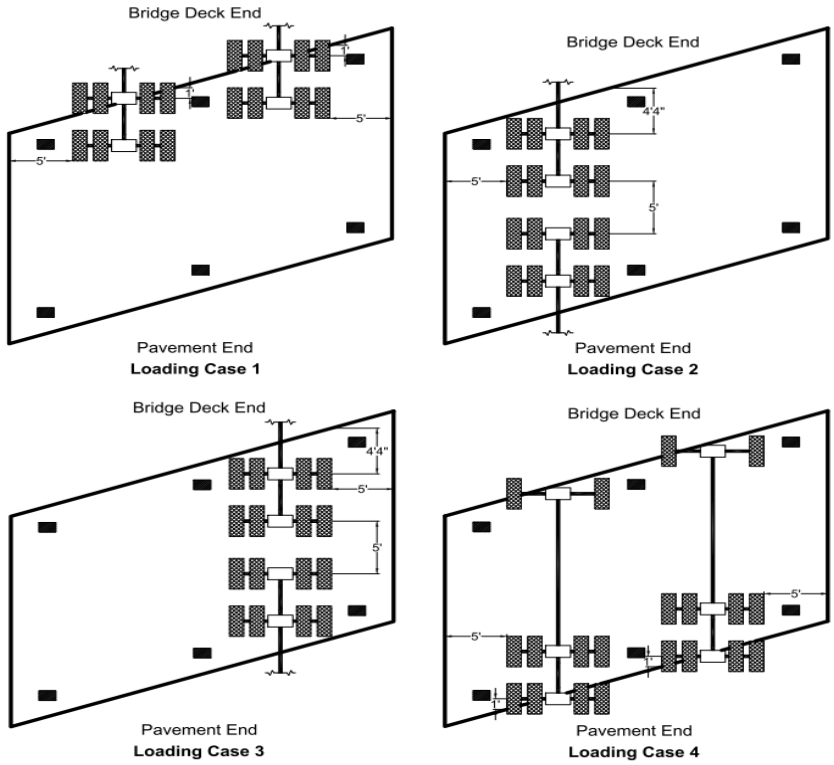


Figure 6-5 Load Cases and Truck Locations for Bridge A7767

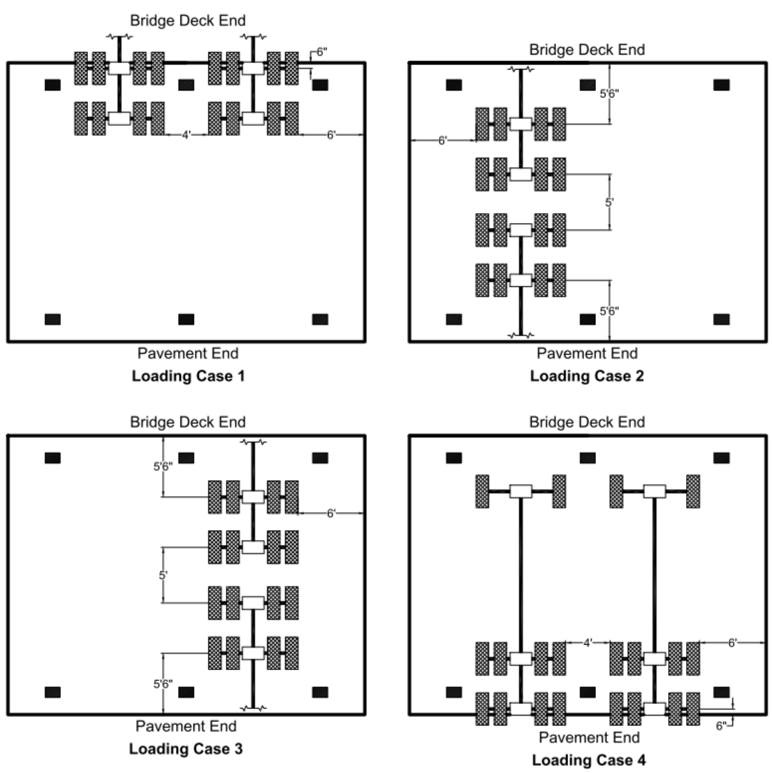


Figure 6-6 Load Cases and Truck Locations for Bridge A7890

Surveying equipment was utilized for monitoring the long term deflection and settlement of the BASs on MO38. All surveying measurements were captured without any external live loads applied to the slab. The Total tation takes minutes to capture the readings of all the surveying prisms so this was not used for each load test. The surveying prisms were placed on monitoring plates 1, 2, 5, and 6 as shown in Figure 6-7 to avoid damage from traffic. During the first load test for all three (3) BASs, the wind caused too much noise in the total station resulting in an error. Since this occurred, baseline surveying measurements were taken on June 05, 2012 before the second load test. Surveying measurements were only captured on Bridges A7767 and A7890 located on MO38. The 20 ft. CIP BAS located on US71 Highway has a high volume of traffic which causes fluctuation in the total station measurements and hence it was neglected from the study after the first load test on November 29, 2011. An additional monitoring plate was installed on the abutment as shown in Figure 6-7, to serve as a reference point for the total station™ surveying equipment. The abutment was chosen as a reference point because of the minimal settlement that will occur compared to the BAS. Field testing results for the rotations and deflections are described next.

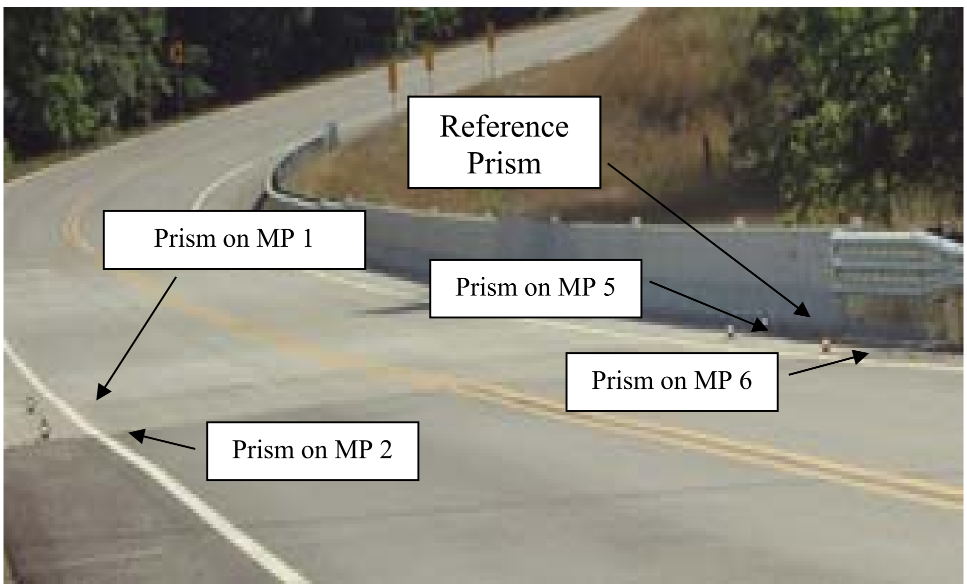


Figure 6-7 Surveying Prism Placement for Monitoring Deflections

6.2 Experimental Results

This section shows the data and analyses from the live load field testing program. Data from the VWSGs and moisture gauges are presented. The rotations and deflections measured with the field instrumentation are also discussed. The results from the live load testing include rotation measurements, baseline deflections at the monitoring plate locations, and differential deflections of the BASs. The inclinometers were placed on each monitoring plate at the locations shown in Figure 6-8 to monitor rotation during the load testing. Surveying prisms were placed on the monitoring plates and deflections were recorded to create a baseline for long term deflection measurements. The differential deflection was measured by placing the prisms on the surface of the BAS near the abutment and flexible pavement.

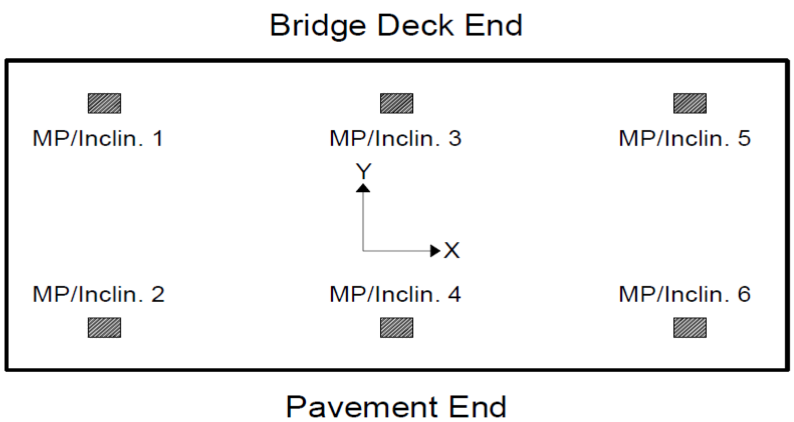


Figure 6-8 Monitoring plate and inclinometer identification numbers

6.2.1 **Rotations**

The rotations were recorded for both live load tests after every load case. Initial and final readings were recorded to ensure the load testing did not cause permanent rotations of the BAS.

Table 6-2 to Table 6-4 show the inclinometer readings for the first load test conducted using unloaded trucks. The inclinometers have the ability to record measurements up to 0.001 degrees but due to the noise from the wind and trucks, the reliability of the results is 0.01 degrees. The inclinometers showed no rotations to the 0.01 degrees for both 20 ft. BASs. The 20 ft. BAS are likely experiencing a slight rotation but due to the precision of the inclinometers no rotation was detected. The 25 ft. CIP BAS experienced a rotation of .01 degrees at monitoring plate 3 for load cases 1 and 4. Inclinometer 1 detected the same rotation for loading case 2 of the 25 ft. CIP BAS.

Table 6-2 Load Test 1 Experimental Rotations for 20 ft. PCPS BAS w/ SS

Load Test 1 Experimental Rotations for 20 ft. PCPS BAS w/ SS (degrees)						
Monitoring Plate	1	2	3	4	5	6
Load Case 1	0.00	0.00	0.00	0.00	0.00	0.00
Load Case 2	0.00	0.00	0.00	0.00	0.00	0.00
Load Case 3	0.00	0.00	0.00	0.00	0.00	0.00
Load Case 4	0.00	0.00	0.00	0.00	0.00	0.00

Table 6-3 Load Test 1 Experimental Rotations for 25 ft. CIP BAS w/o SS

Load Test 1 Experimental Rotations for 25 ft. CIP BAS w/o SS (degrees)						
Monitoring Plate	1	2	3	4	5	6
Load Case 1	0.00	0.00	0.01	0.00	0.00	0.00
Load Case 2	0.01	0.00	0.00	0.00	0.00	0.00
Load Case 3	0.00	0.00	0.00	0.00	0.00	0.00
Load Case 4	0.00	0.00	0.01	0.00	0.00	0.00

Table 6-4 Load Test 1 Experimental Rotations for 20 ft. CIP BAS w/ SS

Load Test 1 Experimental Rotations for 20 ft. CIP BAS w/ SS (degrees)						
Monitoring Plate	1	2	3	4	5	6
Load Case 1	0.00	0.00	0.00	0.00	0.00	0.00
Load Case 2	0.00	0.00	0.00	0.00	0.00	0.00
Load Case 3	0.00	0.00	0.00	0.00	0.00	0.00
Load Case 4	0.00	0.00	0.00	0.00	0.00	0.00

The second load test only included the 20 ft. PCPS and 25 ft. CIP BAS located on MO38. Load testing on the 20 ft. CIP only occurred during the construction of the highway. Once the test was performed the highway opened to a high volume of traffic so future load tests were not possible. The experimental rotation results for the BAS located on MO38 can be found in Table 6-5 and Table 6-6. The rotations for the MO38 BASs performed as expected based on each load case. Despite the 57% increase in load relative to the first load test, the maximum rotation produced by either slab at any given load case is 0.01 degrees. This indicates that the BASs are performing as designed with no significant rotation.

Table 6-5 Load Test 2 Experimental Rotations for 20 ft. PCPS BAS w/ SS

Load Test 2 Experimental Rotations for 20 ft. PCPS w/ SS (degrees)						
Monitoring Plate	1	2	3	4	5	6
Load Case 1	0.01	0.00	0.01	0.01	0.01	0.01
Load Case 2	0.01	0.01	0.01	0.01	0.00	0.00
Load Case 3	0.00	0.00	0.01	0.01	0.01	0.01
Load Case 4	0.00	0.00	0.00	0.01	0.00	0.00

Table 6-6 Load Test 2 Experimental Rotations for 25 ft. CIP BAS w/o SS

Load Test 2 Experimental Rotations for 25 ft. CIP w/o SS (degrees)						
Monitoring Plate	1	2	3	4	5	6
Load Case 1	0.00	0.00	0.01	0.00	0.01	0.00
Load Case 2	0.01	0.00	0.00	0.01	0.00	0.00
Load Case 3	0.00	0.00	0.01	0.01	0.01	0.00
Load Case 4	0.00	0.00	0.00	0.01	0.01	0.00

6.2.2 **Deflections**

Deflections for the 25 ft. CIP and 20 ft. PCPS BAS were measured on 6/5/2012 and 8/30/2012. The first test measurements recorded on 6/5/2012 established baseline readings to monitor the long term deflection of the monitoring plates. The readings taken on 8/30/2012 recorded the deflection of the monitoring plates nearly 3 months after the baseline measurements. Figure 6-9 and Figure 6-10 show the prism points for the 25 ft. CIP and 20 ft. PCPS BAS, respectively. The solid circles are the prisms placed on the monitoring plates (1, 2 ,5, 6) and the hatched circles indicate the prism placement for measuring differential deflection. The results for the deflection of the monitoring plates are shown in Figure 6-11.

The 25 ft. CIP BAS experienced very minimal deflections near the abutment and showed up to 0.032 in. near the pavement end. The 20 ft. PCPS BAS monitoring plates near the abutment experienced less settlement than those near the flexible pavement end. The maximum deflection recorded for the 20 ft. PCPS BAS was 0.042 in. In roughly 3 months the BASs showed minimal deflections at the monitoring plate locations and do not appear to be a problem for motorists. This also indicates that the load is transferring directly from the BAS to the base indicating that no washout or settlement is occurring.

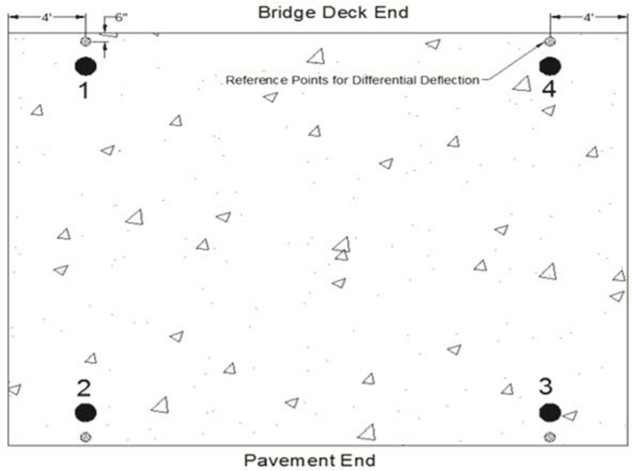


Figure 6-9 25 ft. CIP BAS Prism Locations for Deflection Monitoring

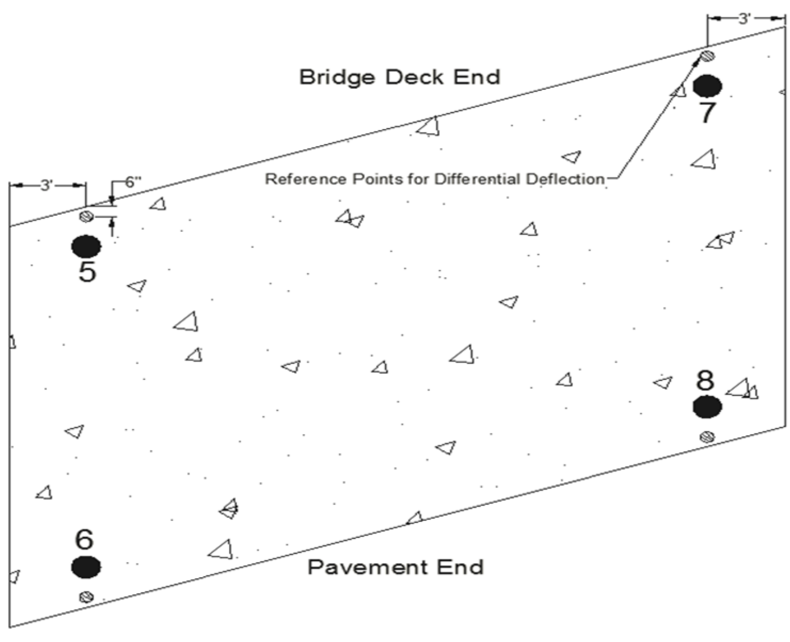


Figure 6-10 20 ft. PCPS Prism Locations for Deflection Monitoring

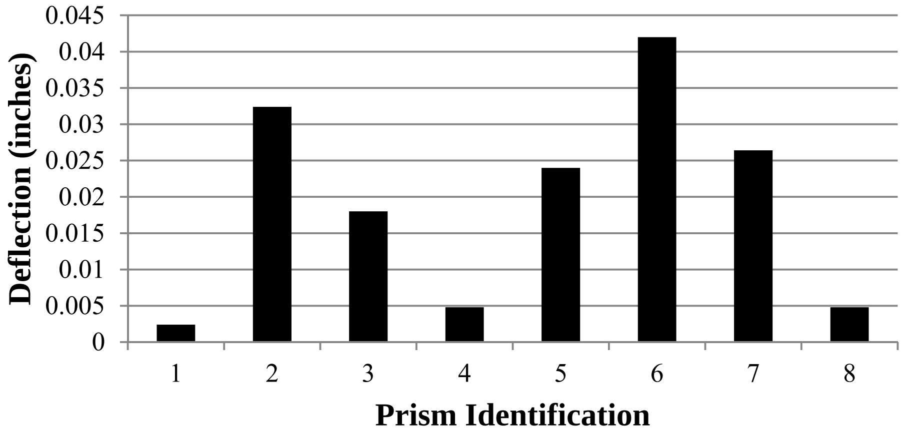


Figure 6-11 3 Month Experimental Deflections of 20 ft. PCPS and 25 ft. CIP BASs No SLP

Longitudinal differential deflection was measured to ensure the BASs were not creating a bump as motorists enter/exit the bridge. The results indicated negligible differential deflections that could be detected by the high precision surveying equipment. The small deflections that were detected are due to slight imperfections in the surface such as texturing (25 ft. CIP BAS No SLP) and the asphalt overlay (20 ft. PCPS BAS SLP). When crossing the bridge in a vehicle, the transition between the bridge and flexible pavement was smooth indicating exceptional performance in both BASs located in Marshfield, MO.

6.2.3 **VWSG Measurements**

VWSGs and moisture gauges were placed in the PCPS BAS panels to measure strains during temperature fluctuations and load testing. The strain gauge and moisture gauge locations are shown in Figure 6-12. Figure 6-13 shows the average monthly concrete strains for each strain gauge in the PCPS panels during a one year period from September 2011 to August 2012. A positive strain increase indicates a compressive strain. As shown in Figure 6-13, the total compressive strain follows changes in temperature. The strain gauges undergo roughly a 50 micro-strain change due to thermal effects throughout a yearly cycle. This provides the baseline behavior of the panels and can be used to determine stresses induced in the panels due to future temperature changes.

In addition to observing the strains during temperature changes, strains during the load testing were monitored. The data logger was setup for 30 minute intervals during the second load test on 6/5/2012 so strain values were not available for this load test. However, data was collected at 1 minute intervals for the first load test using the unloaded trucks. The strain in each gauge was plotted for the 40 minutes it took to perform the load test. During the first load test, the recorded strains did not exceed 1 micro-strain for any of the four load cases indicating that the load is transferring directly through the BAS to the base material without causing distress in the slab. The strain data correlates with the minimal rotations measured by the inclinometers. This also indicates that the base and sub-base have maintained a high subgrade modulus during service loading which provides full elastic support of the BASs.

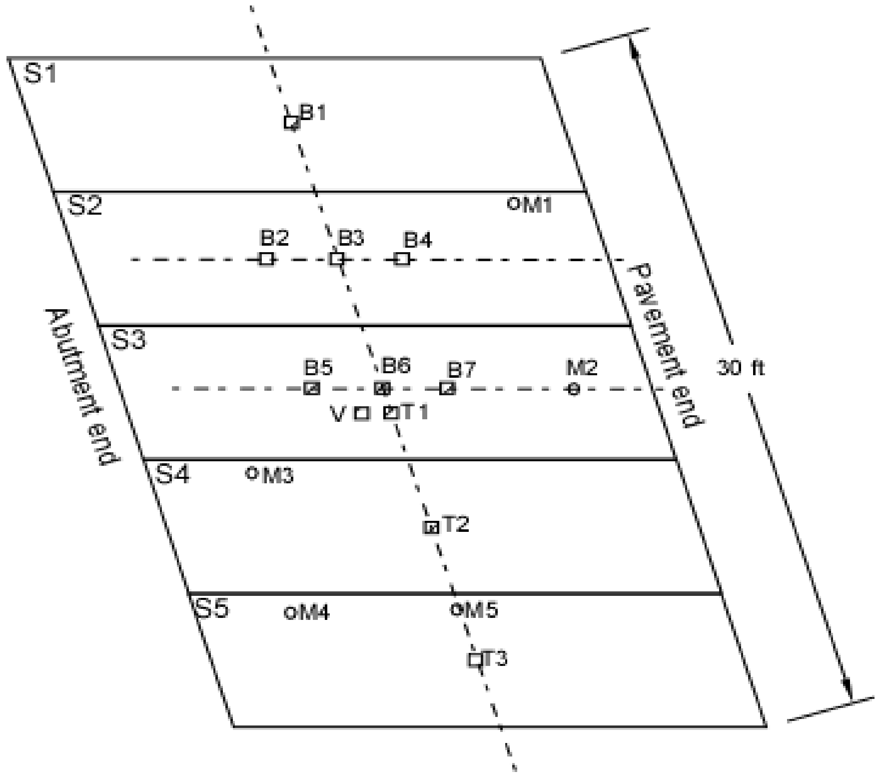


Figure 6-12 Moisture gauge and VWSG Locations

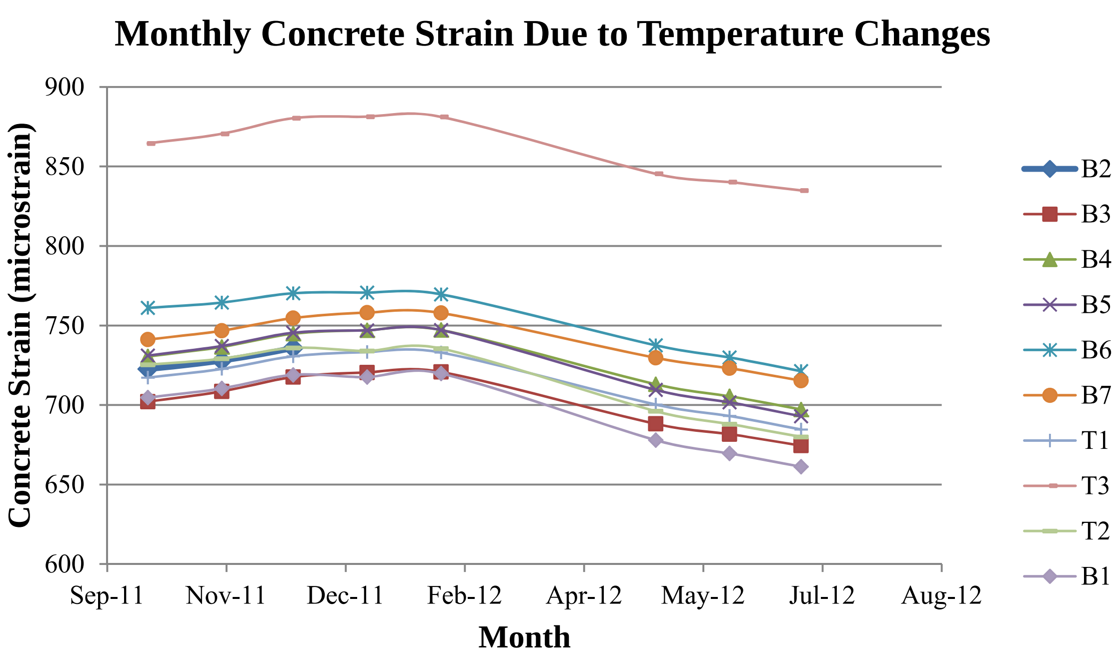


Figure 6-13 PCPS Panel Strains Due to Temperature Fluctuations

Based on the experimental rotations and visual inspection from the second load test, the PCPS BAS has maintained full elastic support from the base material. It can be inferred that the deflection and stress produced from loading the slab is minimal. The strains should continue to be monitored periodically to ensure all gauges are working properly and no abnormalities occur in the data.

6.2.4 **Strain Data Analysis**

This section describes the work undertaken to address the main objective of the proposal, namely to evaluate and compare the field performance of the recommended BAS designs, their constructability, and their impact on cost and schedule to the current MoDOT BAS design. For the purpose of representation of results the strain gauges are classified as shown in Table 6-7.

Table 6-7 Grouping of Strain Gauges for Data Analysis

Set 1	Strain gauges at bottom face of concrete slab placed in longitudinal direction of exterior and intermediate panels. (B1, B2, B3, B4 from Figure 3-12)
Set 2	Strain gauges at bottom face of concrete slab placed in longitudinal direction of middle panels. (B5, B6, B7 from Figure 3-12)
Set 3	Strain gauges at top face of concrete slab placed in longitudinal direction of middle and intermediate panels, and a strain gauge placed in transverse direction at the bottom of concrete slab in middle panel. (T1, T2, T3 from Figure 3-12)

The strain behavior before pouring the concrete and during initial and final hardening of concrete is illustrated in Figure 6-14 to Figure 6-16. The figures illustrate a consistent and predictable concrete strain behavior for all the gauges. Initially, the strain is zero before the concrete pour and there is a temporary tensile strain induced when the concrete is poured. There is a gradual but steep increase in compressive strain during initial hardening of concrete. The compressive strain reduces gradually during the final hardening of concrete. It is also important to note that the pre-stress tendons were anchored at the ends of the formwork during the concreting. To protect the data logging equipment and cables from any possible damage, the tendons were cut after the data logging was stopped and cables spooled up and secured. Hence the effect of pre-stressing force on the concrete strains is not evident in Figure 6-14 to Figure 6-16.

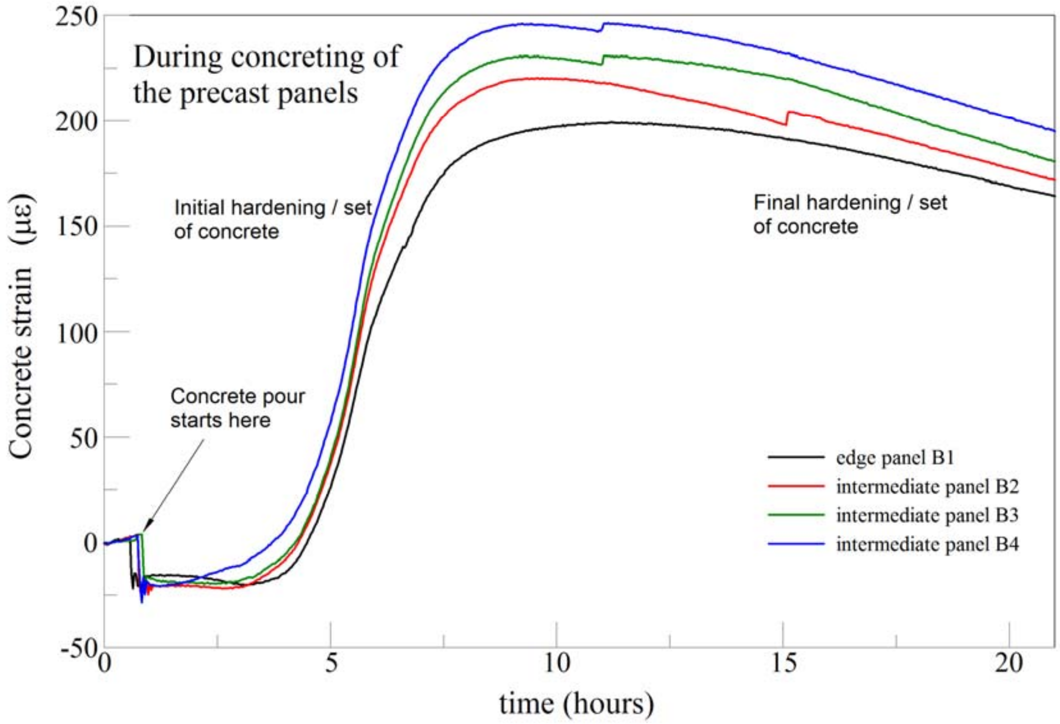


Figure 6-14 Concrete strain during concrete pour and hardening (set 1)

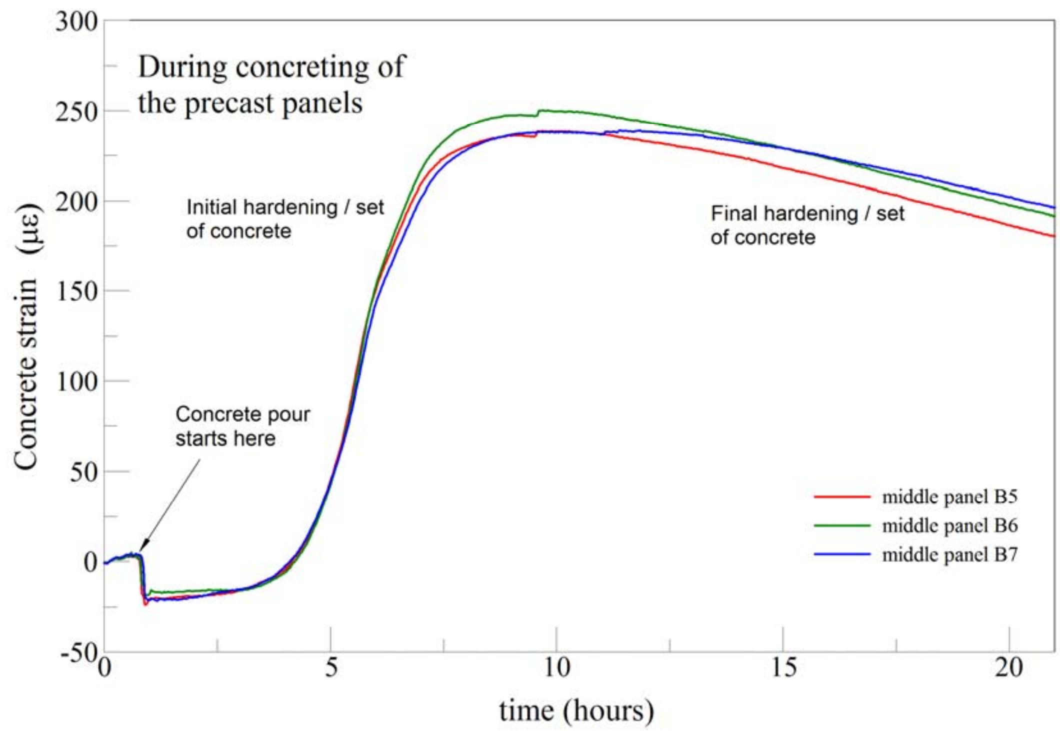


Figure 6-15 Concrete strain during concrete pour and hardening (set 2)

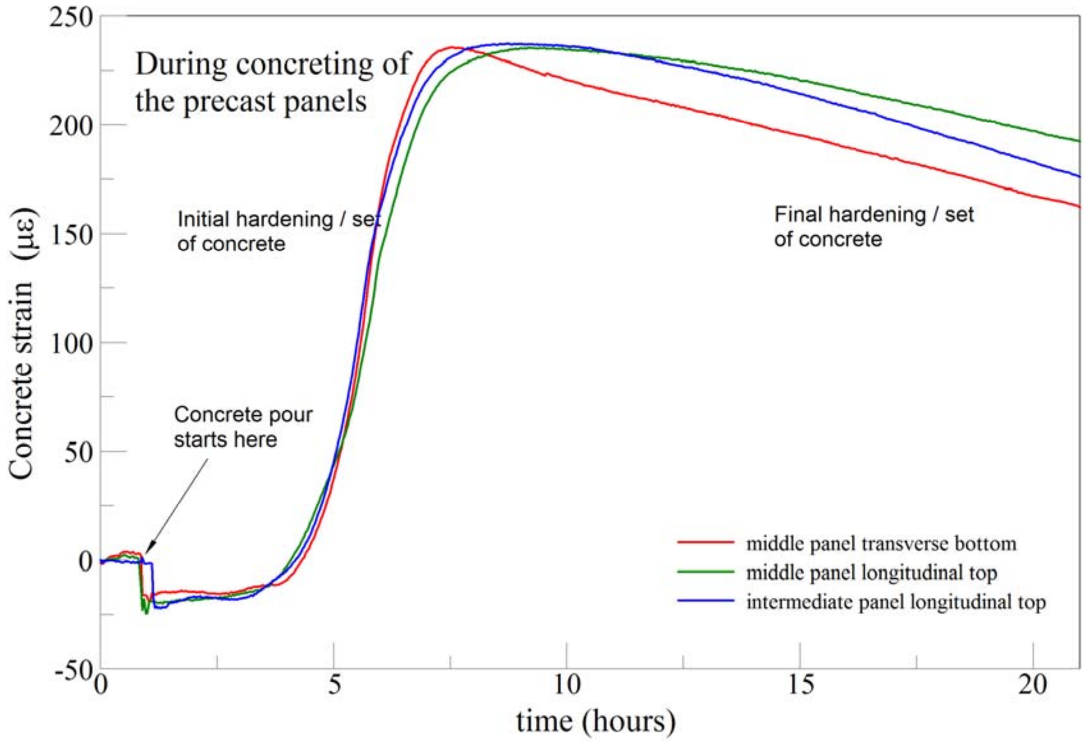


Figure 6-16 Concrete strain during concrete pour and hardening (set 3)

The tensile pre-stressing force in tendons should induce a compressive strain in the concrete even when the slab is not loaded. This compressive strain is illustrated in Figure 6-17 to Figure 6-19. The figures also illustrate the daily variation in strains due to temperature change during the laying of the bridge approach slab panels at site. These figures also show the spike in concrete strain when the bridge approach slab was loaded with a vehicle having 50 tons of axle load. The spike due to load testing can be clearly seen at day 2 readings of Figure 6-17. It was observed that the strain variation due to change in temperature was much larger than that due to the vehicle load. This is because the PCPS BAS is newly constructed and there is no possible loss of support from soil below the BAS at this early stage. However, the magnitude of strain in either case is very small and does not add to any distress in the slabs.

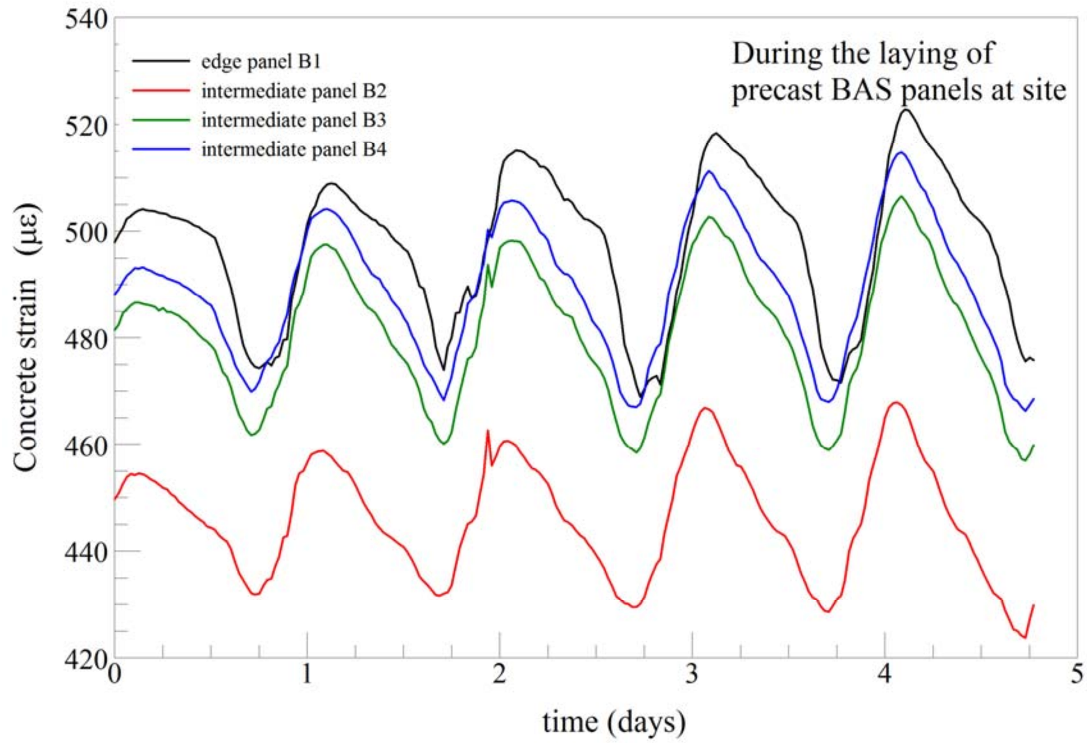


Figure 6-17 Concrete strain variation due to temperature (set 1)

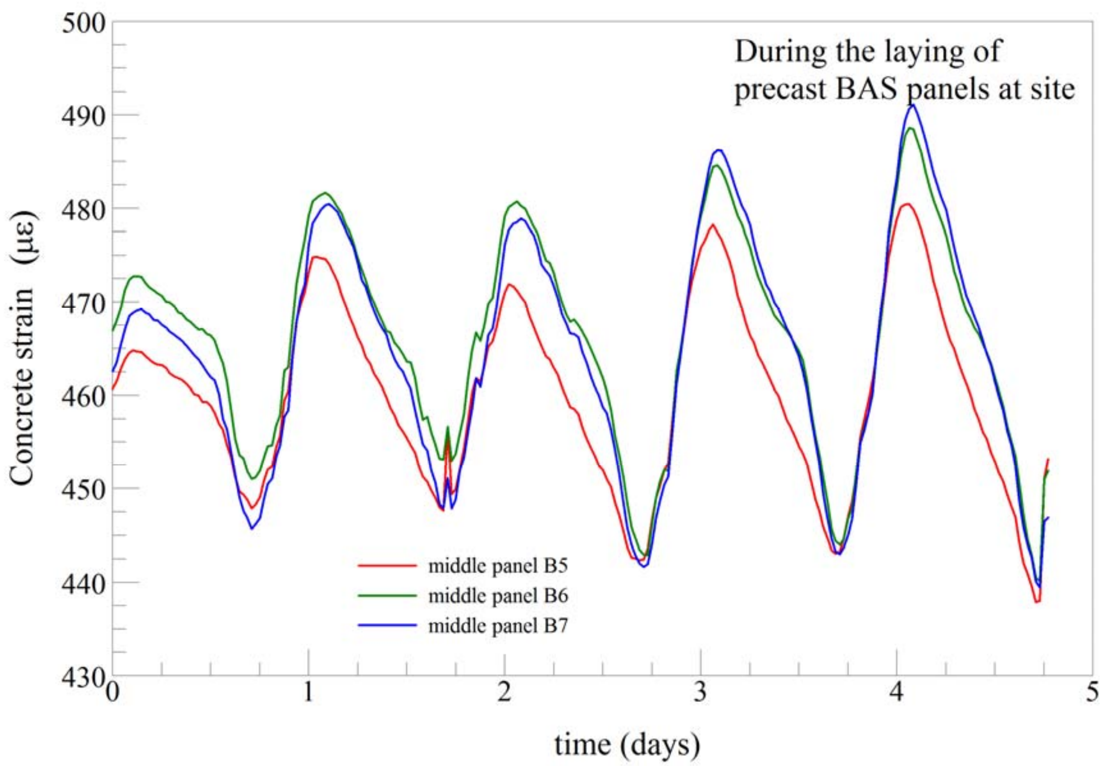


Figure 6-18 Concrete strain variation due to temperature (set 2)

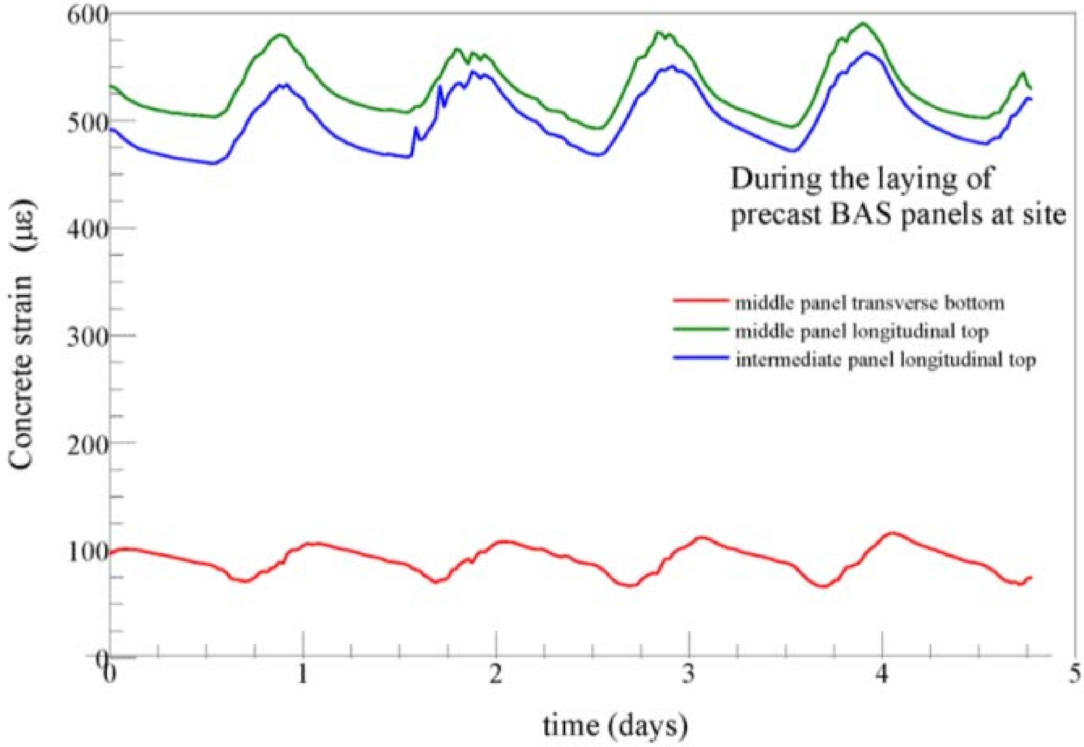


Figure 6-19 Concrete strain variation due to temperature (set 3)

The concrete strains and moisture readings were recorded at about half an hour intervals after the complete construction of the bridge approach slab. Figure 6-20 to Figure 6-25 illustrates the variation in strain due to seasonal change in temperature. It is observed that the strains are decreasing during winter due to the cold climate. The data was collected over a period of 40 days.

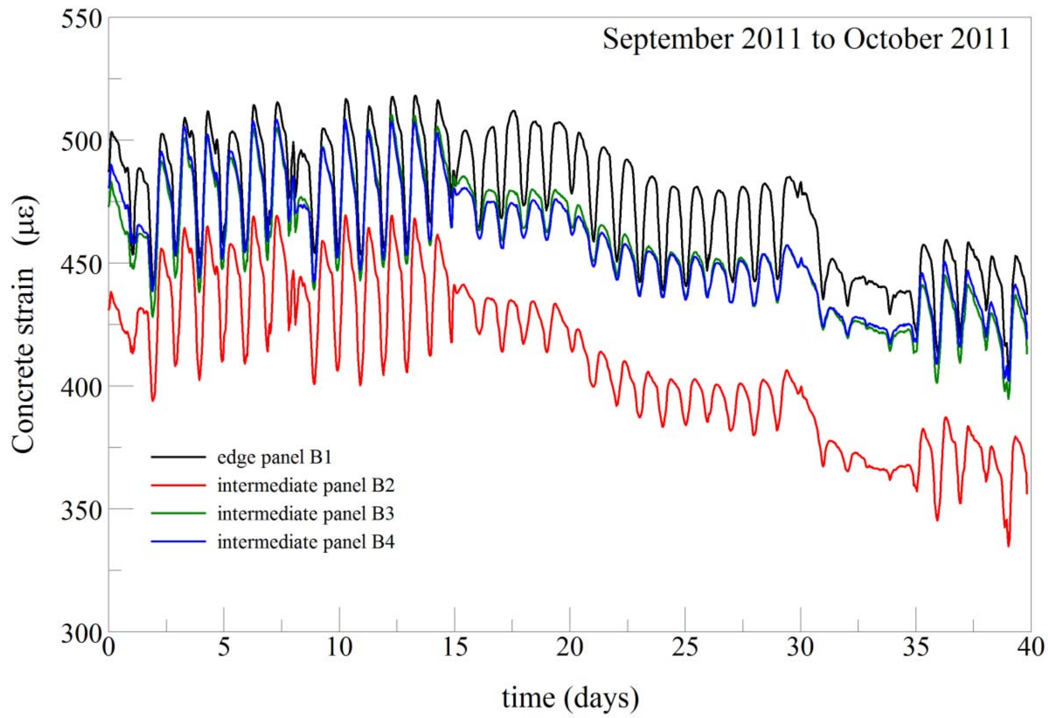


Figure 6-20 Variation in concrete strain (set 1) from September 2011 to October 2011

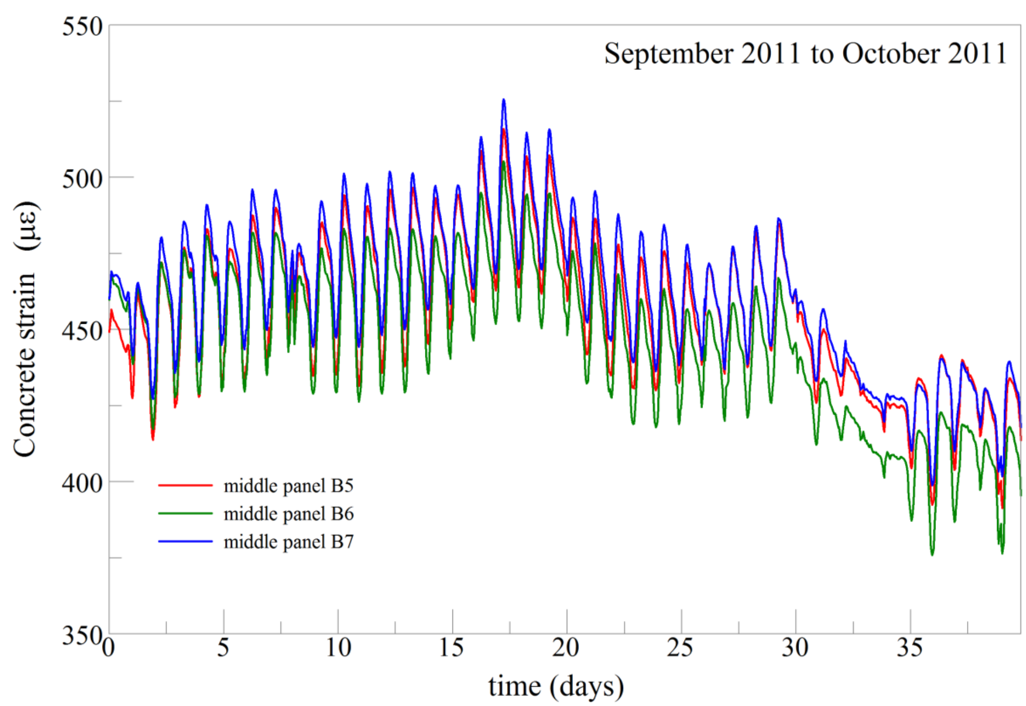


Figure 6-21 Variation in concrete strain (set 2) from September 2011 to October 2011

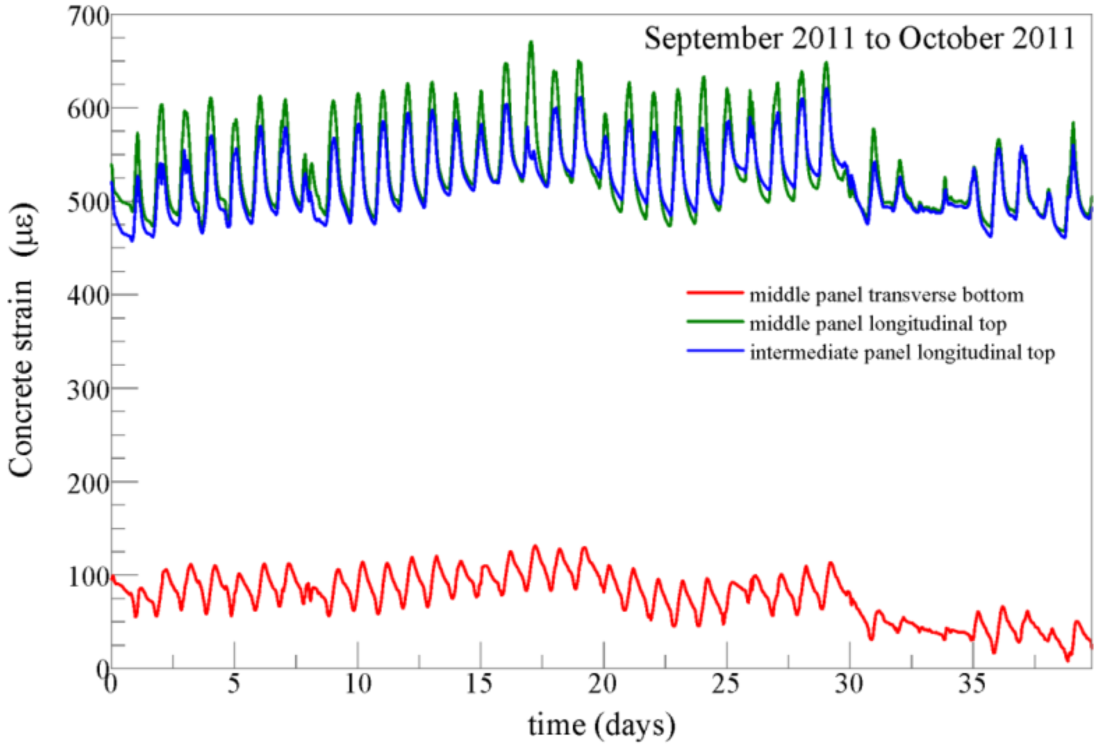


Figure 6-22 Variation in concrete strain (set 3) from September 2011 to October 2011

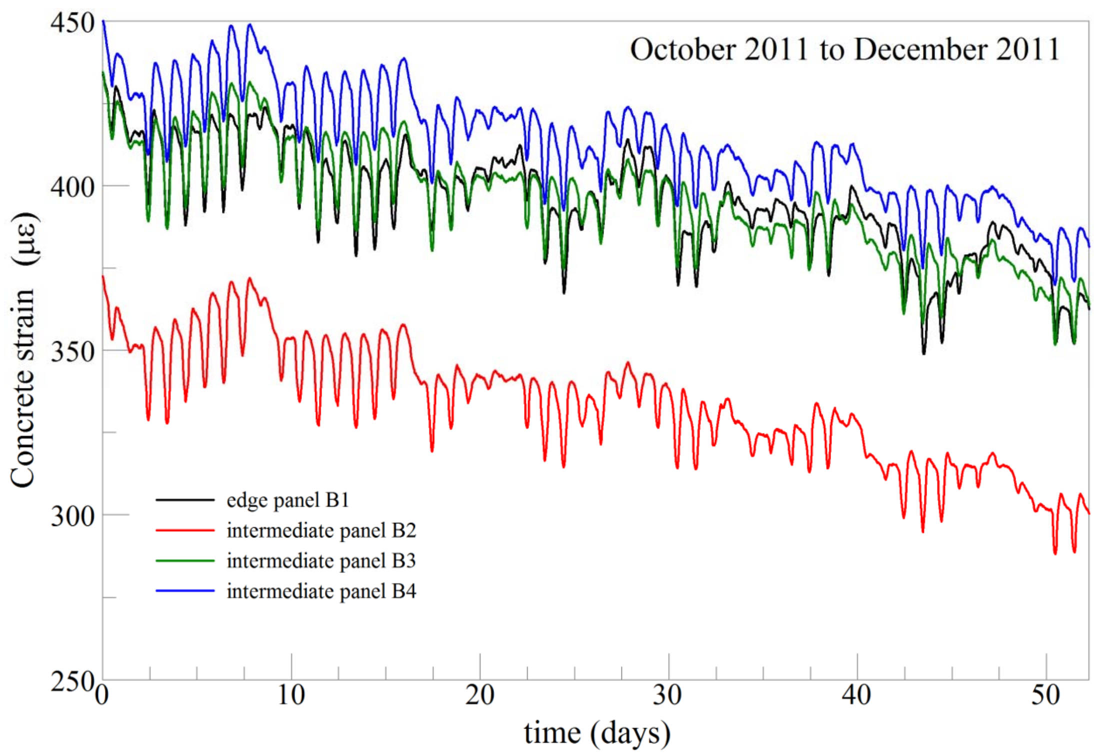


Figure 6-23 Variation in concrete strain (set 1) from October 2011 to December 2011

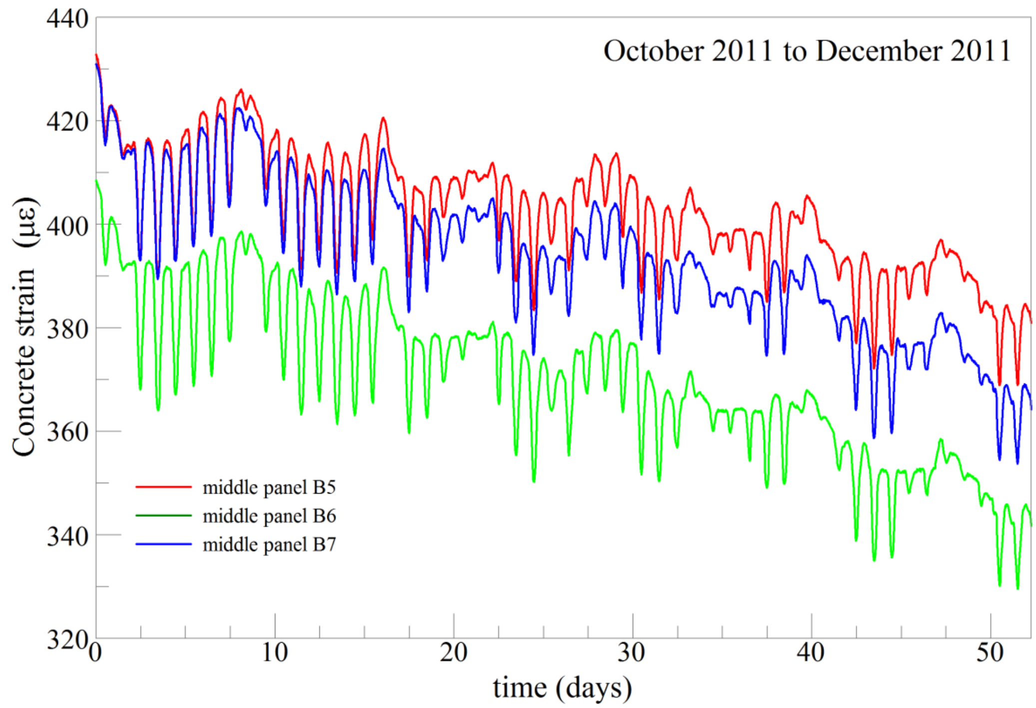


Figure 6-24 Variation in concrete strain (set 2) from October 2011 to December 2011

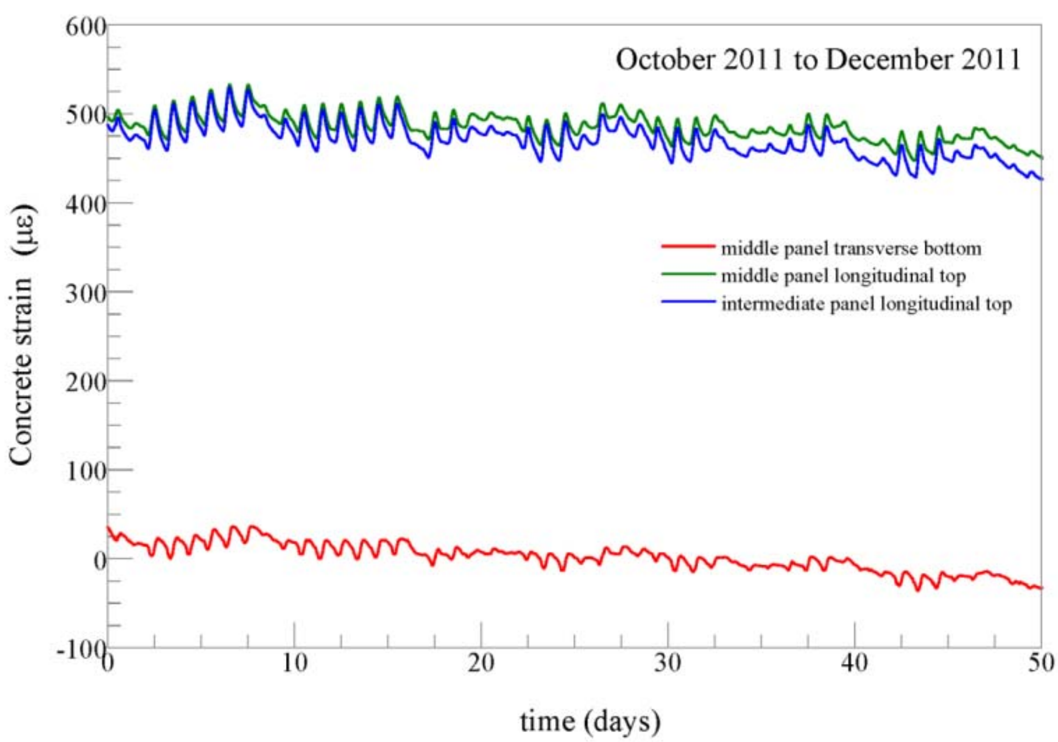


Figure 6-25 Variation in concrete strain (set 3) from October 2011 to December 2011

The concrete strains were recorded at 10 minute intervals from January to February 2012 to check the performance of the data logger and relevant settings. The recorded strains showed that one of the strain gauges was probably damaged due to larger strain than the gauge is capable of withstanding as illustrated in Figure 6-26 below. The other strain gauges showed expected strain values as illustrated in Figure 6-27 to Figure 6-29.

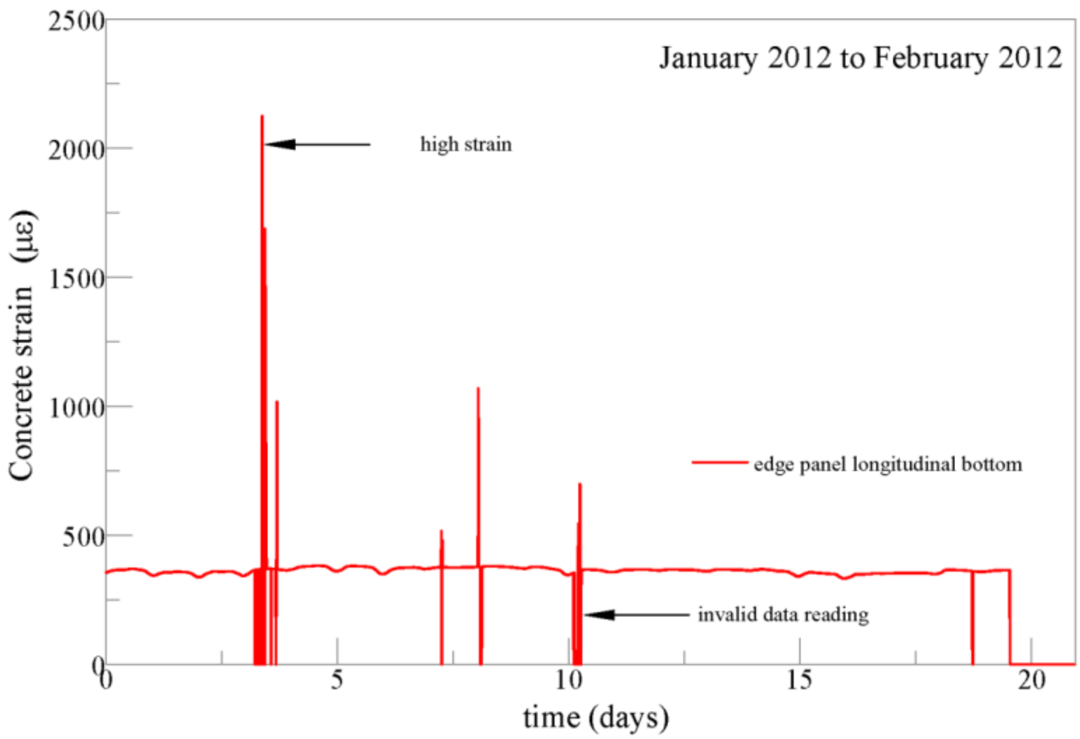


Figure 6-26 Variation in concrete strain in edge panel from January to February 2012

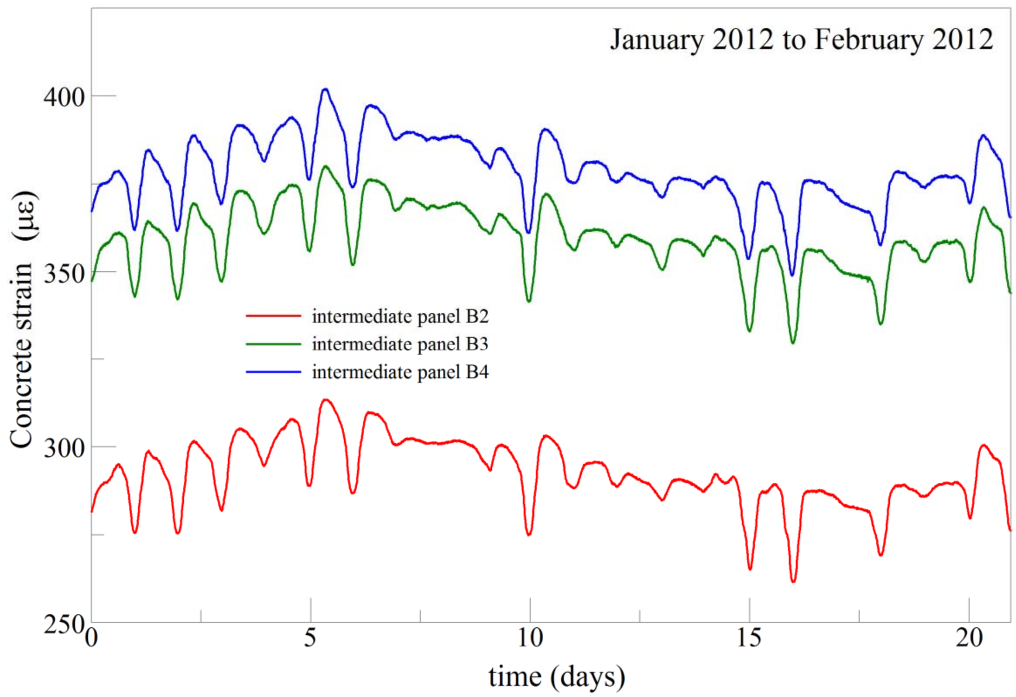


Figure 6-27 Variation in concrete strain in intermediate panels from January to February 2012

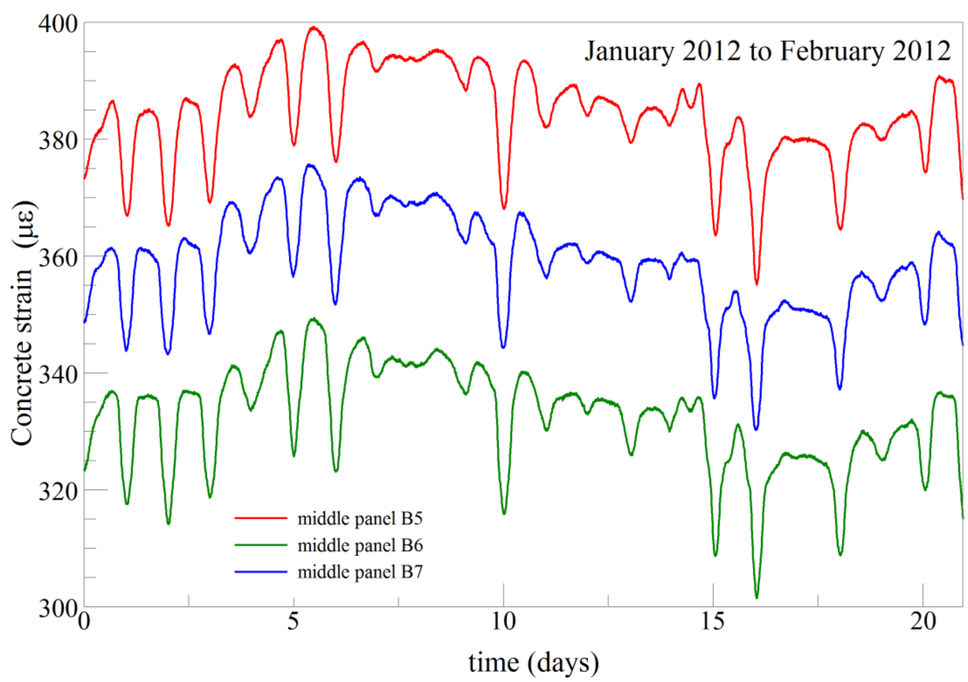


Figure 6-28 Variation in concrete strain in middle panels from January to February 2012

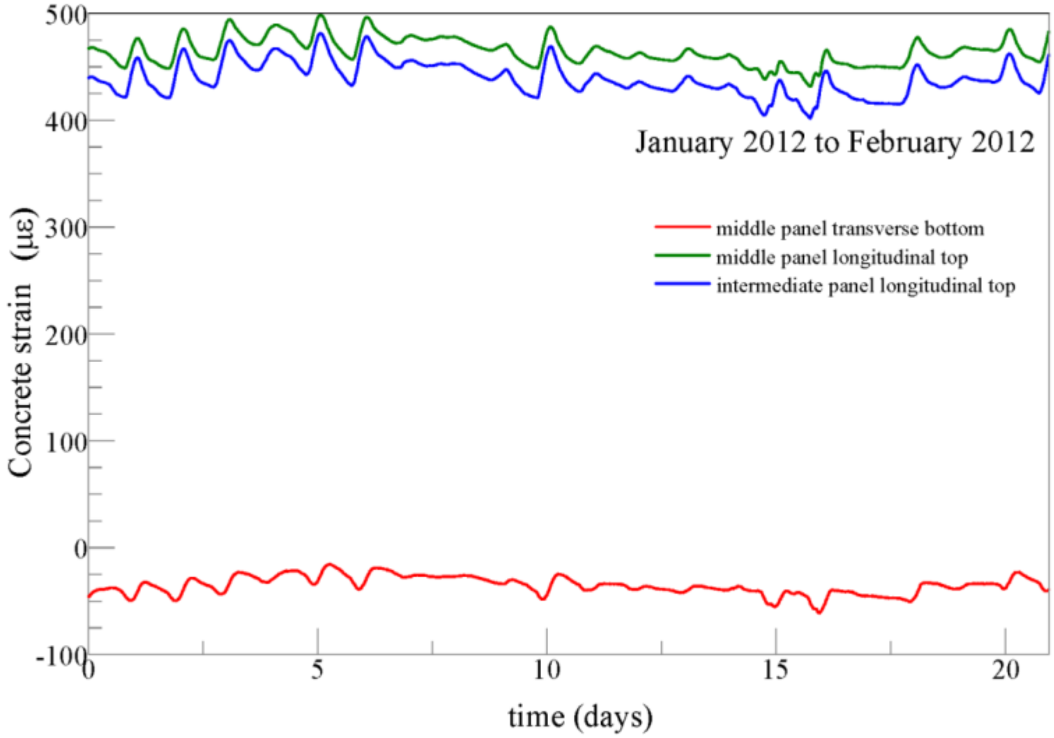


Figure 6-29 Variation in concrete strain in misc. panels from January to February 2012

The concrete strains were recorded at 45 minute intervals from February to May 2012 to collect strain recordings at longer intervals for 2.5 months. The recorded strains from previous data collection had shown that one of the strain gauges was probably damaged due to larger strain than the gauge is capable of withstanding as illustrated in Figure 6-26. Therefore, after collecting the data this time while resetting the data logger for new data this damaged strain gauge channel was disconnected. The other strain gauges showed expected strain values as illustrated in Figure 6-30, Figure 6-31 and Figure 6-32.

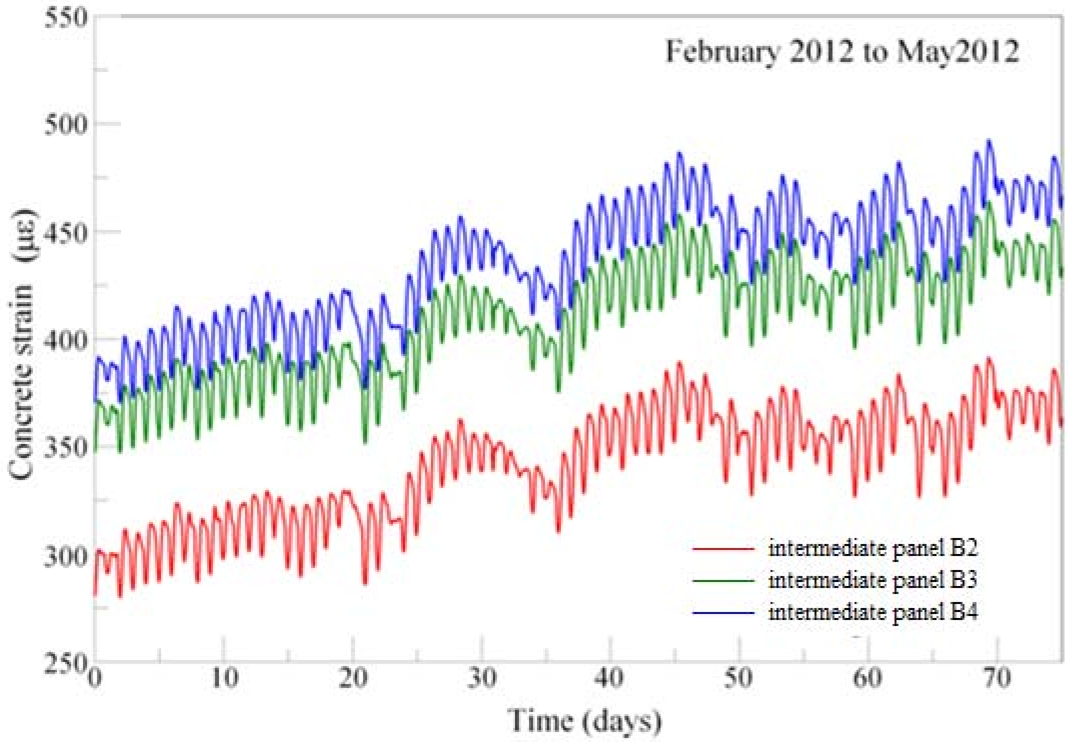


Figure 6-30 Variation in concrete strain in intermediate panels from February to May 2012

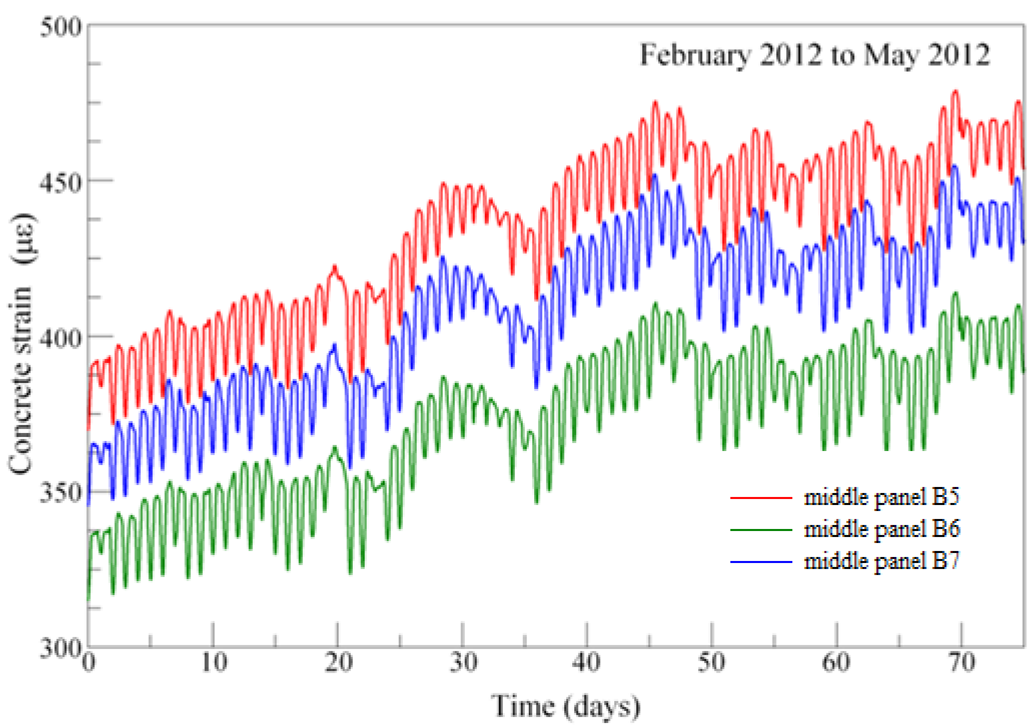


Figure 6-31 Variation in concrete strain in middle panels from February to May 2012

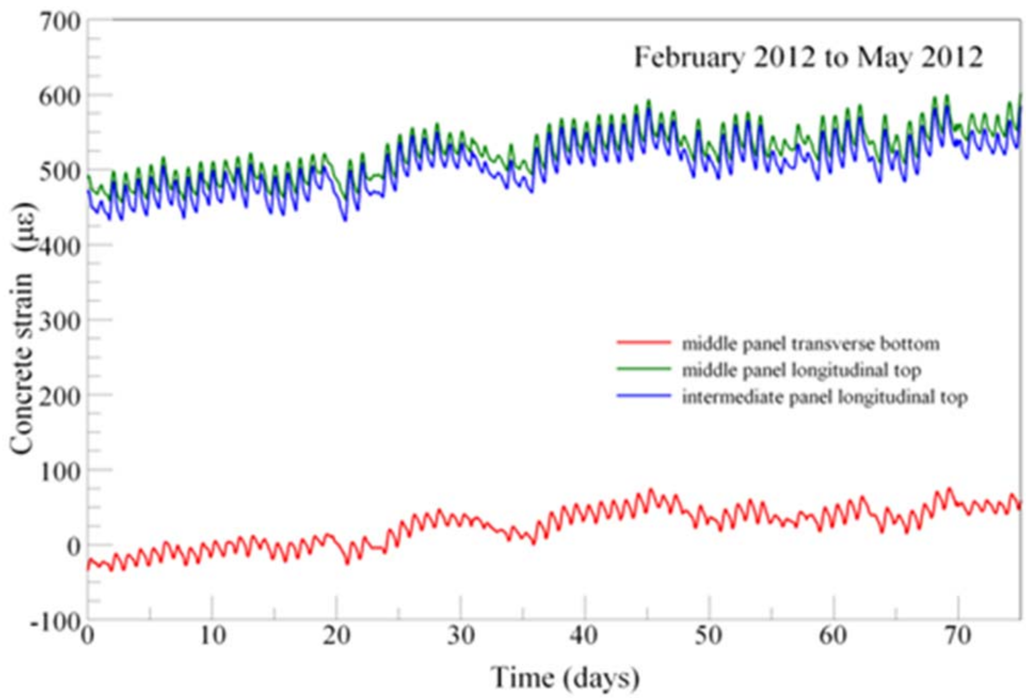


Figure 6-32 Variation in concrete strain in misc. panels from February to May 2012

Figure 6-33 illustrates the concrete strain during the load test conducted on December 9 2011. It is observed that the strain variations are very small and does not add to any distress in slab panels. The strain data collected over a period of 9 months from August 2011 to May 2012 was analyzed for the maximum, minimum and average strain values and has been tabulated in Table 6-8. The variation observed in maximum and minimum strain readings over the same 9 month period is plotted in Figure 6-34 and Figure 6-35 respectively. It is observed from the graph in Figure 6-34 that the peak maximum strain were observed in all gauges in the months of Aug – Oct 2011 and the lowest maximum strains observed in all gauges in the month of Feb 2012. Also, it is observed from the graph in Figure 6-35 the peak minimum strain was observed in all gauges in the months of Aug – Oct 2011 and the lowest strains are observed in all gauges in the month of Feb 2012. Overall, the trend observed in strain variations in each gauge for all gauges is the same.

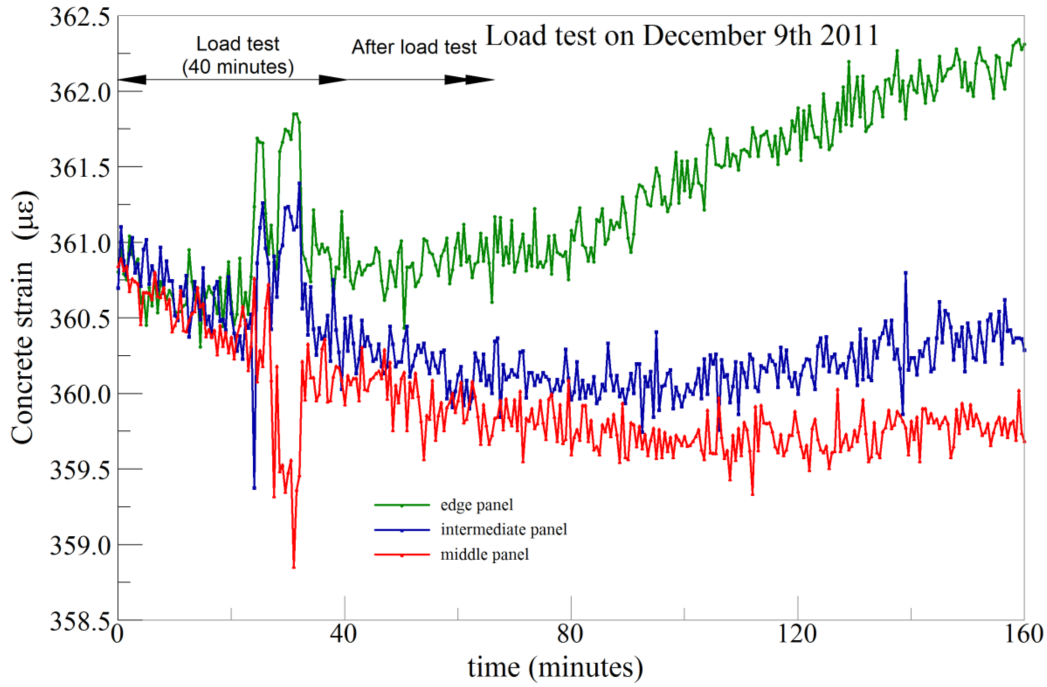


Figure 6-33 Concrete Strain during Load Test for Select Panels

Table 6-8 Strain Variation Observed over a Period of 9 months[1]

Strain Data (Micro-strains)		gauge1	gauge2	gauge3	gauge4	gauge5	gauge6	gauge7	gauge8	gauge9	gauge10
		edge panel long bot	int panel long bot 1	int panel long bot 2	int panel long bot 3	mid panel long bot 1	mid panel long bot 2	mid panel long bot 3	mid panel trans bot	mid panel long top	int panel (north) long top
Aug-Oct 2011	*Max*	518	469	510	508	516	505	526	131	671	621
	Min	405	335	395	402	391	376	399	8	468	457
	Avg	475	411	458	458	456	446	462	77	533	521
Oct-11	*Max*	434	373	435	450	433	408	431	37	533	529
	Min	380	327	387	408	391	361	386	0	471	457
	Avg	411	353	414	431	413	387	410	19	497	486
Nov-11	*Max*	415	357	419	438	421	392	415	22	519	511
	Min	349	295	359	376	372	335	359	-36	447	429
	Avg	394	332	394	411	402	369	393	-1	484	468
12/1/2011 to 12/9/2011	*Max*	389	319	384	400	397	359	383	-14	486	471
	Min	353	289	352	371	369	330	354	-45	446	424
	Avg	373	308	373	390	387	349	372	-26	464	443
1/27/2012 to 1/31/2012	*Max*	2124	305	373	392	393	341	369	-22	494	475
	Min	338	275	342	362	365	314	343	-50	448	421
	Avg	380	293	360	380	382	332	358	-36	469	445
2/1/2012 to 2/17/2012	*Max*	3240668	314	380	402	399	349	376	-16	498	481
	Min	332	262	330	349	355	301	330	-61	431	402
	Avg	306844	292	360	379	385	333	359	-35	463	437
2/17/2012 to 2/29/2012	*Max*	#VALUE!	328	396	420	411	357	389	10	517	505
	Min	#VALUE!	280	347	371	370	315	346	-34	457	433
	Avg	#VALUE!	306	374	396	394	340	370	-13	484	465
Mar-12	*Max*	#VALUE!	377	444	472	464	399	439	60	574	561
	Min	#VALUE!	286	352	377	383	324	357	-26	460	432
	Avg	#VALUE!	335	402	428	427	366	401	18	516	498
Apr-12	*Max*	#VALUE!	392	464	493	479	414	455	76	600	585
	Min	#VALUE!	327	396	426	427	362	401	15	509	483
	Avg	#VALUE!	362	431	459	456	391	429	44	546	527

[1]#VALUE – Indicates gauge failure

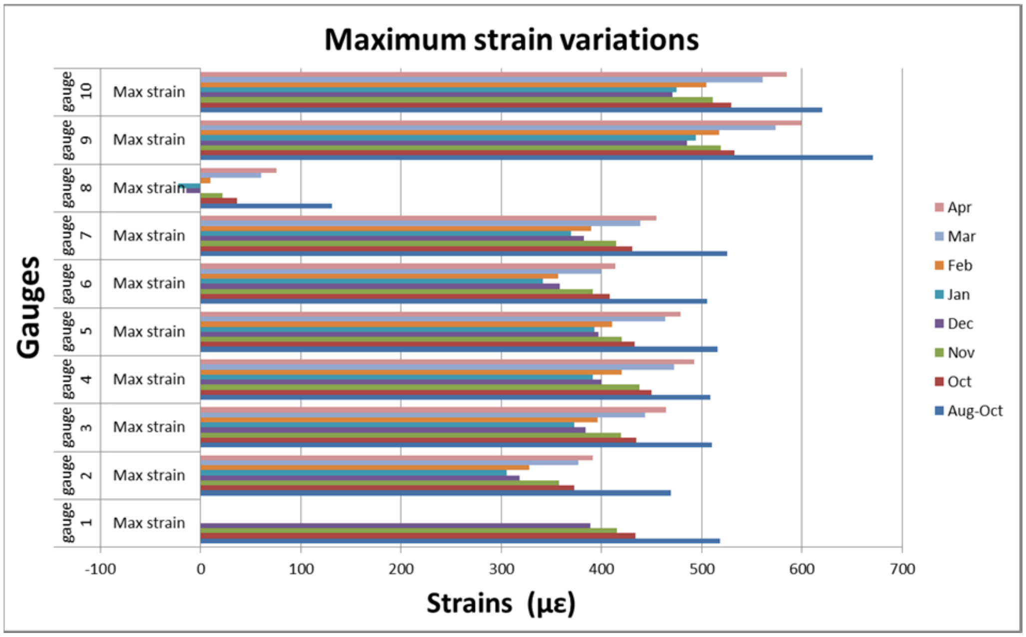


Figure 6-34 Variation in Maximum strain values observed over a period of 9 months

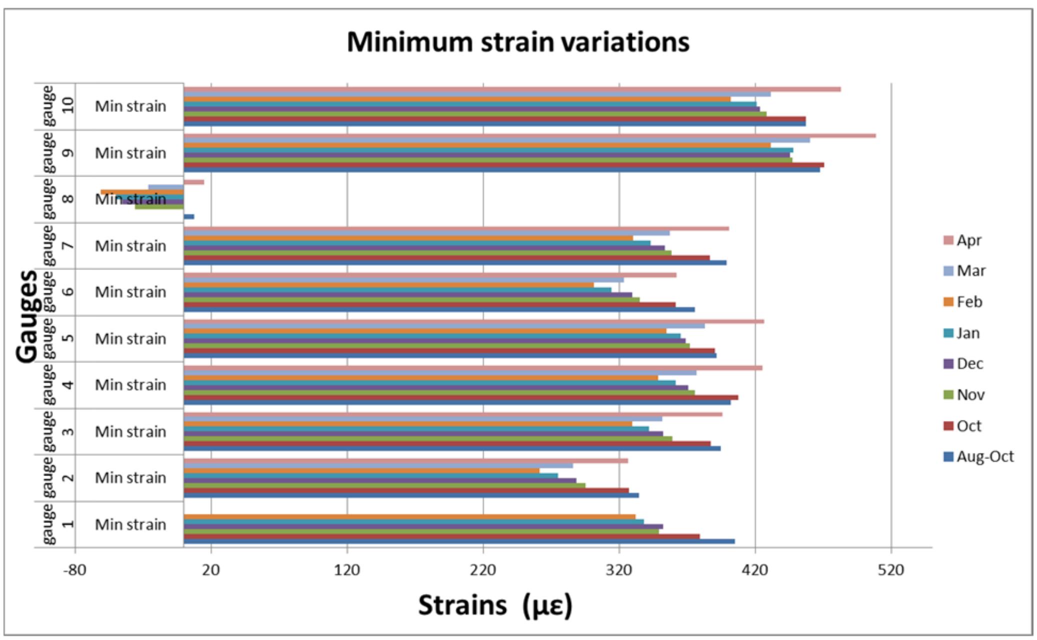


Figure 6-35 Variation in Minimum strain values observed over a period of 9 months

6.2.5 Strain Values over 2 years in PCPS over MO38

Figure 6-36 shows the monthly maximum, average and minimum strain values for all the eleven VSWG strain data recorded during the period of the project. Over the 18 month period it is observed that the peak longitudinal strain recorded was 675 microstrains and around 125 microstrains in the lateral direction. Figure 6-37 shows just the average monthly strain values for the same set of gauges for clarity. It can be seen from both the figures that the strain values tend to peak during the summer months and ebbs during the winter months. The least recorded strain was approximately 250 microstrains in the longitudinal direction.

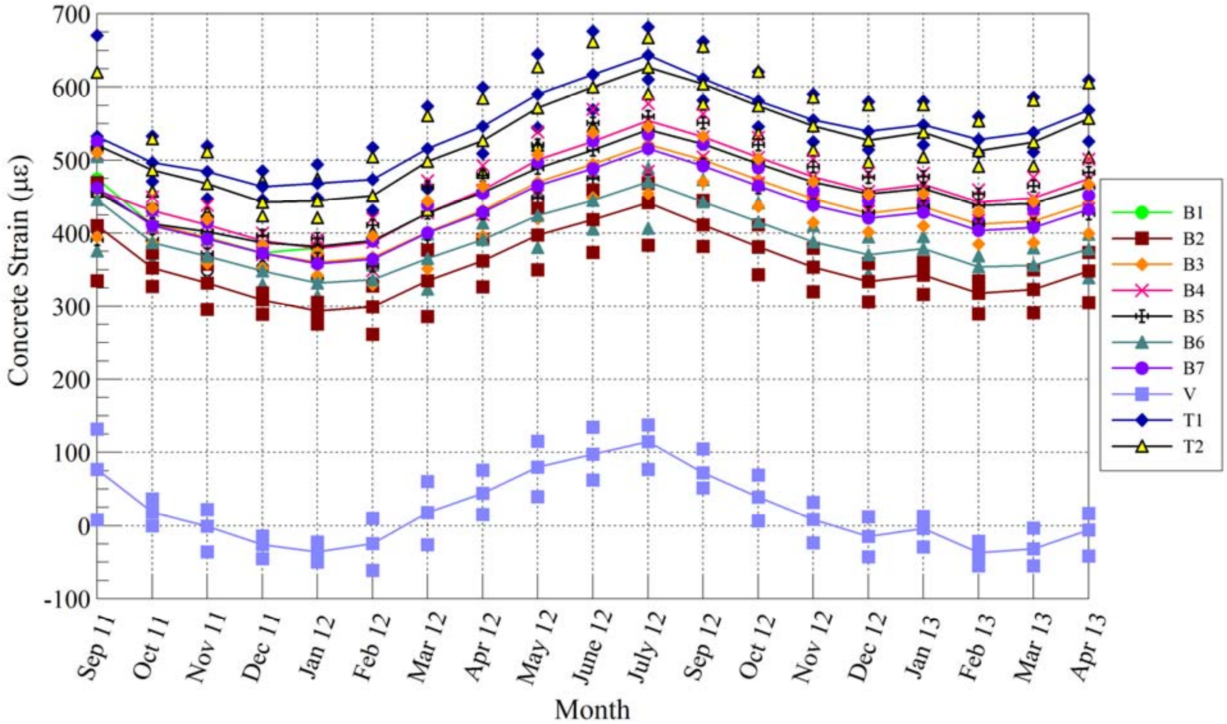


Figure 6-36 Minimum, Average and Maximum Strain Data in Concrete Gauges

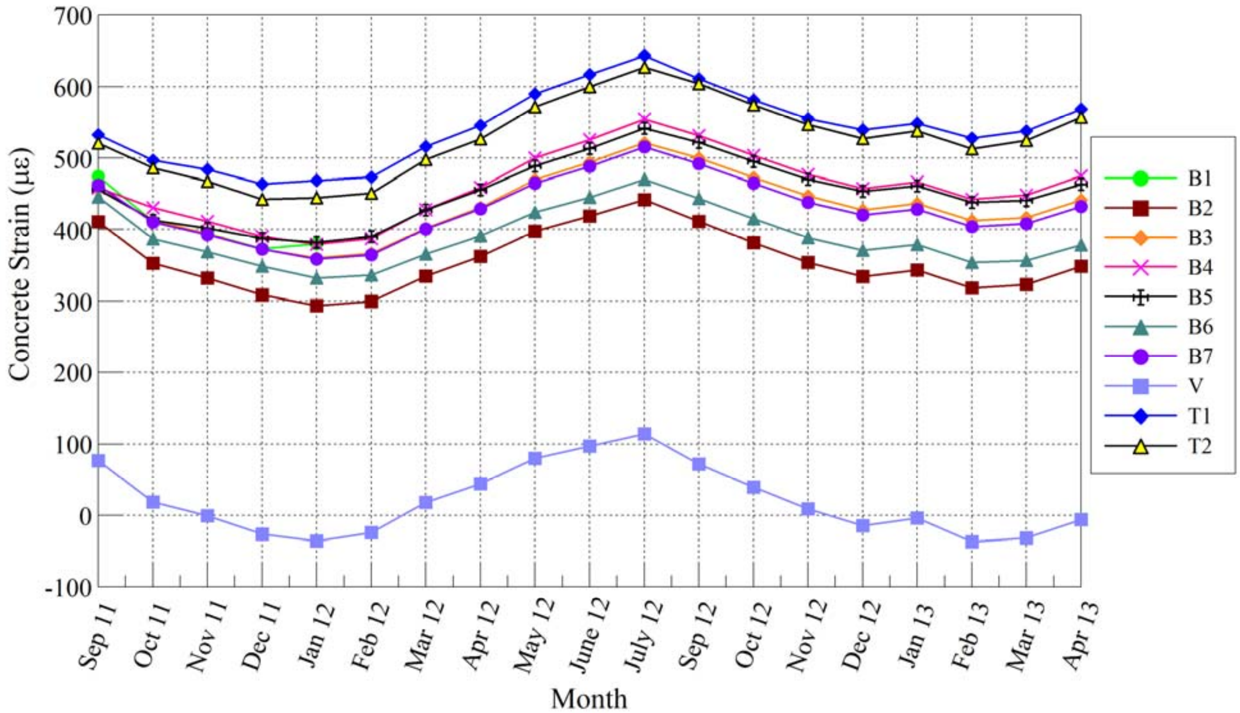


Figure 6-37 Average Strain Value in the Gauges (in microstrains)

The stress in the concrete, determined from finite element analysis using SAP2000v15, due to maximum possible loading condition with and without a sleeper slab is shown in Figure 6-38 and Figure 6-39 respectively. The maximum computed stress value is about 360 psi at about 2.7" from bottom fiber of the concrete slab. Using linear elastic analysis this equates to a microstrain of 100 ($360x10^{-3}/3650x10^{6}$) which is consistent with the observations in observed strain data (about 300-500 microstrain). The difference can be safely assumed to be from the pre-stressing force of strands.

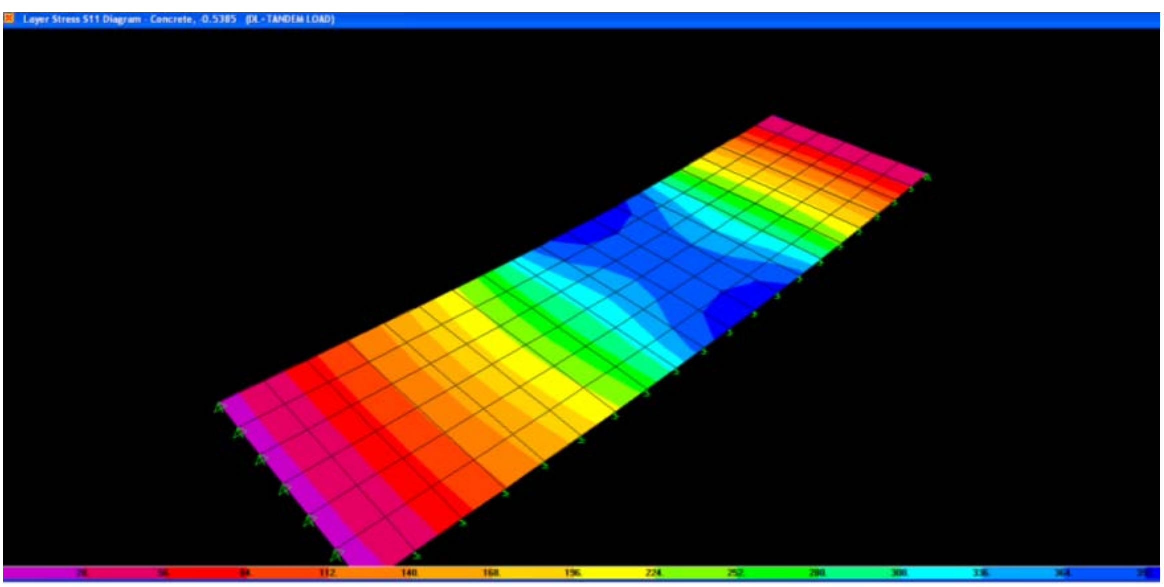


Figure 6-38 Concrete stress at 2.7" from bottom face - SAP model with sleeper slab

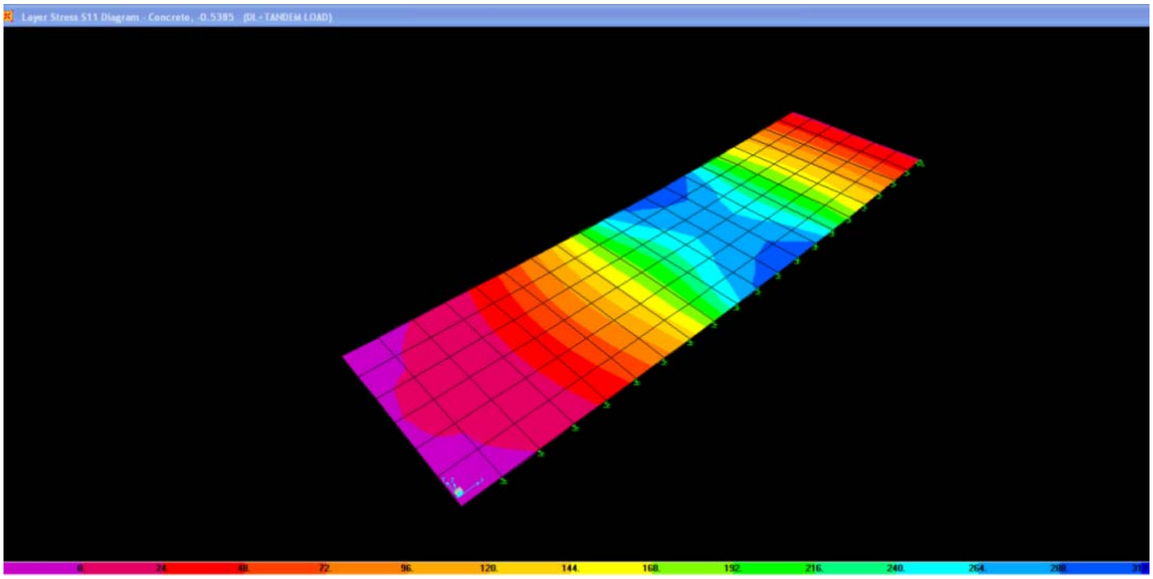


Figure 6-39 Concrete stress at 2.7" from bottom face - SAP model without sleeper slab

6.2.6 **Moisture Gauge Data**

Moisture gauges were placed 4 in. (100 mm) below the soil beneath the PCPS BAS panels. The locations of the 5 moisture gauges are shown in Figure 6-12. The volumetric water content (VWC) was monitored to determine if water was draining properly due to the aggregate base. As shown in Figure 6-12, two moisture sensors were placed near the abutment joint, two near the sleeper slab joint, and one in the middle of the keyway under panel S5. The daily moisture readings were taken shortly after being installed to create a baseline reading and is shown in Figure 6-40. The spike in moisture levels was due to rain. The return of the moisture levels to normal shows the adequacy and proper working of the drainage system at this point of time.

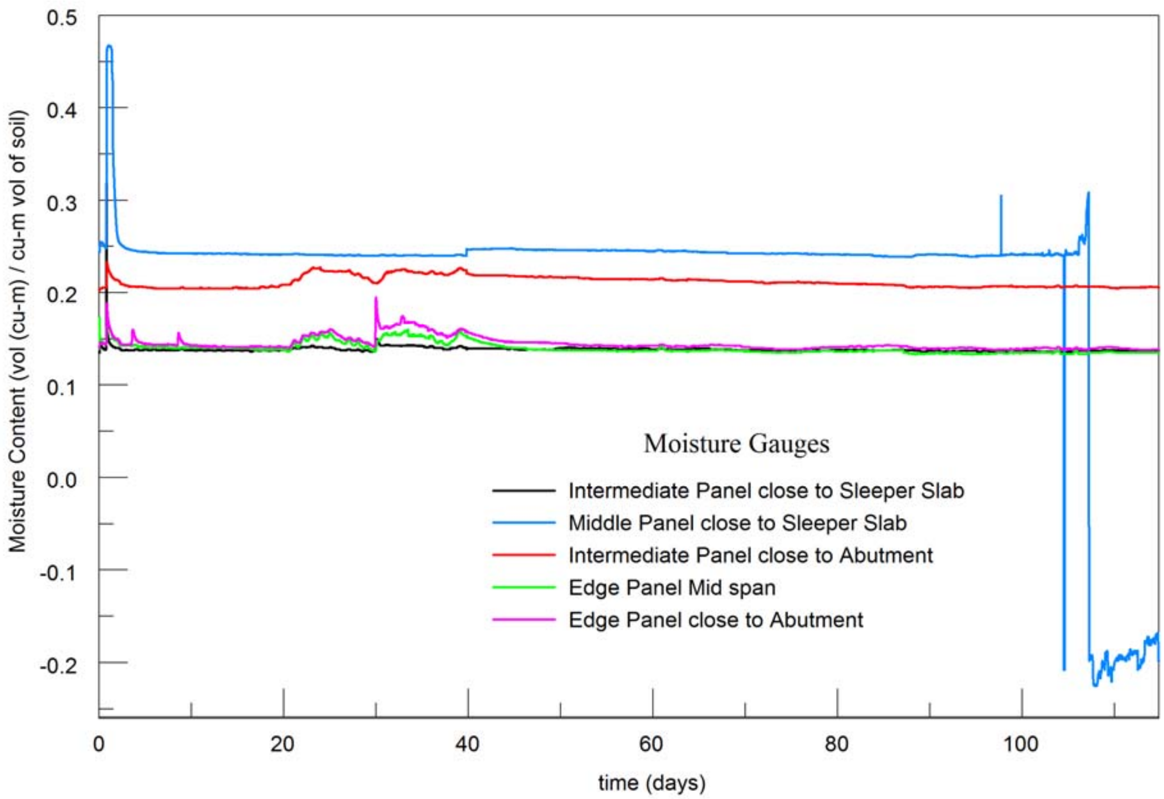


Figure 6-40 Daily variation in moisture levels in BAS subgrade

The average monthly VWC readings are shown in Figure 6-41. After a year in-service and being exposed to environmental conditioning, all gauges maintained a consistent moisture reading. This indicates that water is being properly drained by the base material and not allowing moisture into the soil.

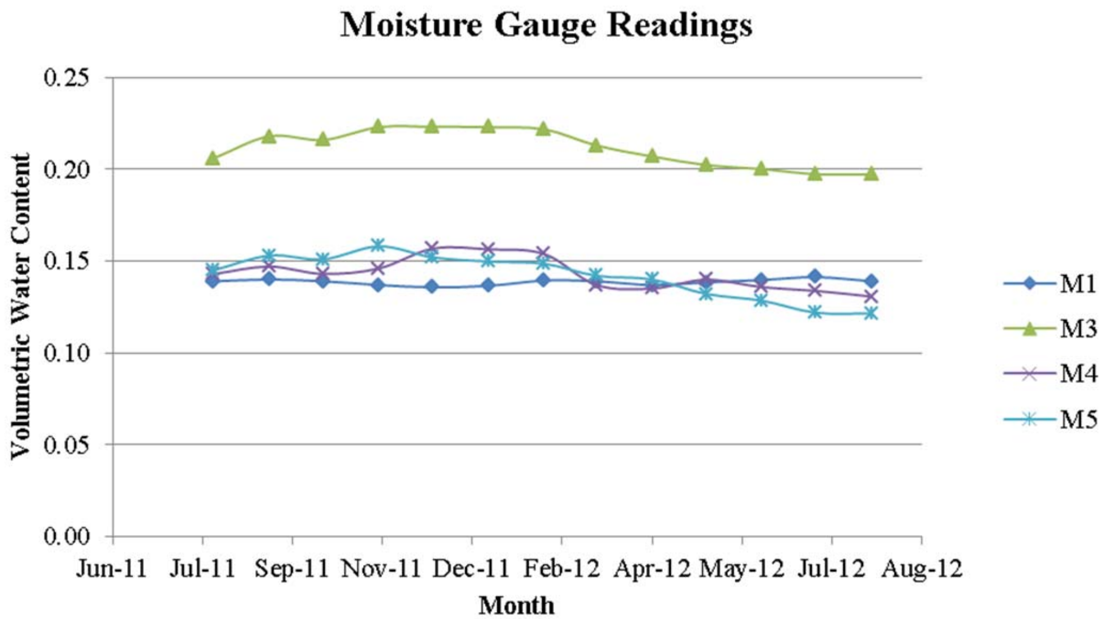


Figure 6-41 Moisture Gauge Readings

Table 6-9 shows the moisture gauge readings taken over a two year period. PJT and PMID descriptions in Table 6-9 indicate moisture gauge location at PCPS panel joints and middle. The LEFT, CENTER and RIGHT indicate the location of the moisture gauges with respect to the span of the PCPS. The LEFT is closer to abutment and RIGHT is closer to sleeper slab and CENTER is in the mid span of the PCPS. Figure 6-42 shows the average moisture content data recorded over the 18 month period in each of the gauges.

It can be seen from the Table 6-9 and Figure 6-42 that one of the gauges towards the sleeper slab (DMG002) and one gauge towards the abutment (DMG003) show higher moisture levels as compared to others until month of March-12. The gauge towards sleeper slab (DMG002) is showing some negative readings at the end. This gauge has probably failed and needs to be checked at next site visit. However, the moisture readings are very stable indicating an effective drainage system working properly.

Table 6-9 Moisture Gauge Readings

Moisture gauge readings in m³/m³ VWC															
	DMG-PJT-RIGHT (DMG001)			DMG-PMID-RIGHT (DMG002)			DMG-PJT-LEFT (DMG003)			DMG-PJT-LEFT (DMG004)			DMG-PJT-CENTER (DMG005)		
	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg
Aug-11	0.135	0.401	0.139	0.242	0.467	0.255	0.203	0.233	0.206	0.140	0.184	0.143	0.141	0.189	0.145
Sep-11	0.136	0.150	0.140	0.239	0.242	0.240	0.205	0.227	0.218	0.136	0.160	0.147	0.141	0.195	0.153
Oct-11	0.136	0.142	0.139	0.238	0.243	0.240	0.210	0.228	0.216	0.136	0.157	0.143	0.142	0.166	0.151
Nov-11	0.135	0.140	0.137	0.241	0.249	0.243	0.217	0.229	0.223	0.136	0.156	0.146	0.152	0.164	0.158
Dec-11	0.134	0.138	0.136	0.245	0.250	0.247	0.219	0.229	0.225	0.143	0.160	0.151	0.150	0.162	0.158
Jan-12	0.134	0.143	0.136	0.246	0.25	0.247	0.219	0.228	0.223	0.142	0.159	0.15	0.152	0.163	0.156
Feb-12	0.138	0.141	0.139	0.246	0.249	0.247	0.216	0.227	0.22	0.138	0.162	0.145	0.143	0.163	0.151
Mar-12	0.137	0.140	0.139	0.241	0.246	0.244	0.209	0.216	0.213	0.135	0.142	0.137	0.139	0.144	0.142
Apr-12	0.136	0.138	0.137	-0.225	0.308	0.153	0.205	0.209	0.207	0.133	0.138	0.135	0.138	0.143	0.140
May-12	0.137	0.140	0.138	-0.228	-0.154	-0.184	0.199	0.207	0.203	0.129	0.136	0.132	0.137	0.143	0.140
June-12	0.138	0.141	0.140	-0.208	-0.134	-0.155	0.196	0.202	0.200	0.124	0.133	0.128	0.132	0.140	0.136
July-12	0.140	0.155	0.141	-0.180	-0.123	-0.139	0.196	0.198	0.197	0.118	0.126	0.122	0.132	0.136	0.134
Aug-12	0.136	0.143	0.139	-0.198	-0.123	-0.147	0.195	0.208	0.197	0.117	0.141	0.121	0.127	0.148	0.131
Sep-12	0.134	0.142	0.136	-0.211	-0.128	-0.170	0.196	0.212	0.201	0.117	0.142	0.123	0.126	0.151	0.134
Oct-12	0.134	0.142	0.136	-0.240	-0.164	-0.209	0.198	0.216	0.206	0.120	0.142	0.130	0.125	0.154	0.140
Nov-12	0.134	0.138	0.136	-0.262	-0.167	-0.227	0.201	0.221	0.212	0.122	0.145	0.135	0.125	0.153	0.143
Dec-12	0.130	0.136	0.133	-0.281	-0.220	-0.256	0.207	0.228	0.218	0.128	0.150	0.141	0.134	0.156	0.147
Jan-13	0.131	0.136	0.133	-0.291	-0.250	-0.277	0.211	0.224	0.217	0.131	0.150	0.142	0.136	0.159	0.150
Feb-13	0.131	0.136	0.133	-0.286	-0.228	-0.259	0.211	0.221	0.214	0.131	0.149	0.137	0.133	0.154	0.142
Mar-13	0.133	0.136	0.134	-0.277	-0.205	-0.249	0.209	0.223	0.214	0.132	0.150	0.138	0.131	0.157	0.141
Apr-13	0.131	0.136	0.134	-0.247	-0.182	-0.210	0.204	0.211	0.206	0.127	0.136	0.130	0.130	0.135	0.132

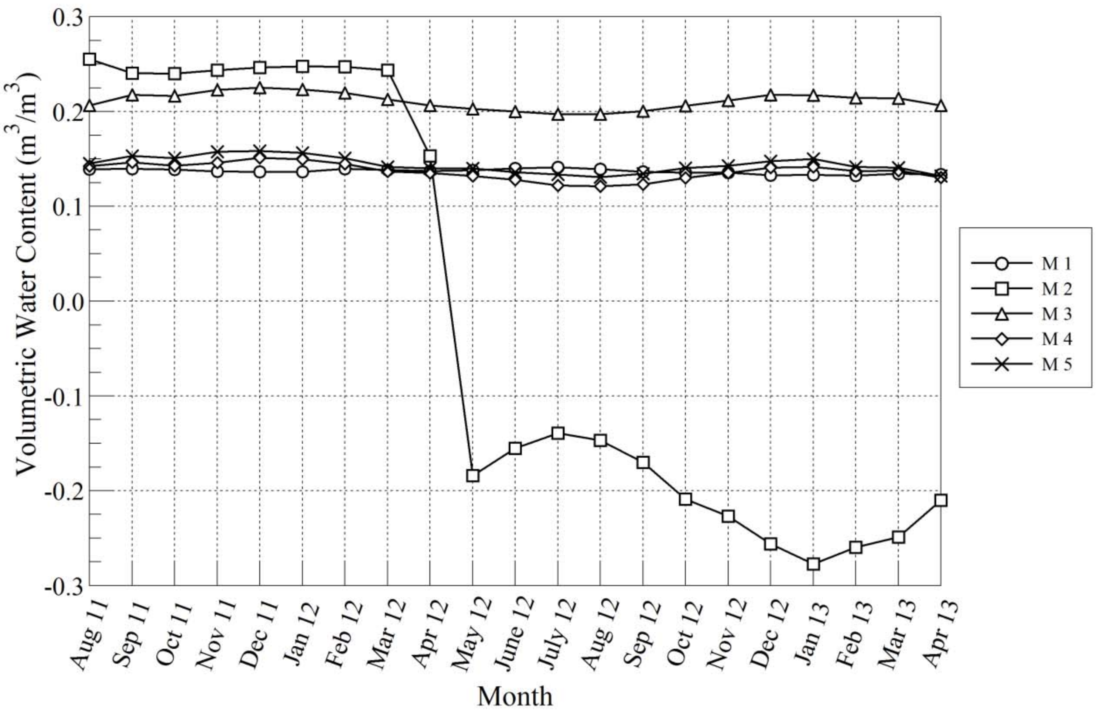


Figure 6-42 Average Moisture Content Data Recorded Over 18 Months

6.3 Visual Inspection

This section presents site observations following construction of the approach slabs. The slabs were monitored through visual inspection periodically over the next 21 months. The BASs were inspected visually for any signs of cracking and deflection. The slabs were also investigated for any signs of washout. There are no current signs of visual washout or deflection which is reflected based on the load testing. However, the 25 ft. CIP BAS on Bridge A7890 showed a slight longitudinal surface crack down the middle of the east bound lane (see Figure 6-43) during the site visit on August 2012. It had a crack width of roughly 0.079 in. (2 mm) and is not considered an issue at this time.

Besides slight surface cracking of the asphalt shoulder there were no significant changes until the final site visit. Between 19 and 21 months a gap of approximately 0.75 inches formed between approach slab and approach pavement of the bridge. This can be seen in Figure 6-44.

a) Measurement of crack width b) Measurement of crack length

Figure 6-43 Surface Crack on 25 ft. CIP BAS on Bridge A7890

Figure 6-44 Gap between the approach pavement and approach slab at north shoulder, center & south shoulder respectively on Bridge A7890 at Final Site Inspection

For the CIP bridge it was observed during the visit on April 22, 2013 that the edge of the BAS had broken at the edge of the asphalt and this is located in the south lane on east end of the bridge. On the West end of the bridge a longitudinal crack can be seen in Figure 6-45. The crack is at about 11 ft. 4 in distance from the wing wall (Figure 6-45 – left) and it is approximately 5 ft 4 in long (Figure 6-45-right).

Figure 6-45 Longitudinal Crack at West End of PCPS on MO38 – 11’4”distance from the wing wall (left) and 5’4” crack (right)

During the last site visit on April 22, 2013 it was observed at the east end of the bridge the asphalt is approximately one inch higher (Figure 6-46) than the slab but is still flush with the bridge. On the west end it can be seen (Figure 6-47) that the approach slab is now about an inch higher than the asphalt but again is still flush with the bridge. At both ends the movement appears to be uniform across the width of the bridge.

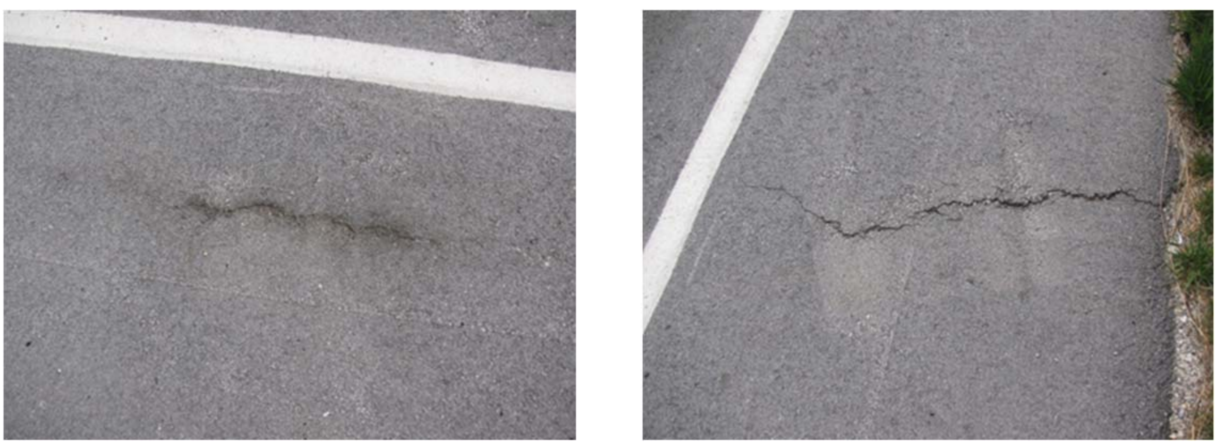
Figure 6-46 Shoulder asphalt level above the CIP BAS on MO38

Transverse view close up view

Figure 6-47 Gap observed between pavement and CIP BAS on MO38

6.3.1 **PCPS Bridge BAS on MO38**

For the PCPS bridge there is longitudinal surface cracking of the asphalt on the north shoulder of the east end on the bridge (Figure 6-48a). On that same shoulder near the end of the slab there is some transverse cracking of the asphalt (Figure 6-48b). During the final site visit in April 2013 a gap between the bridge deck and the asphalt overlay on the west end was observed and is shown in Figure 6-49. The gap appears to be uniform all the way across the bridge.



(a) Minor Longitudinal Crack (b) Transverse Shoulder Crack

Figure 6-48 Asphalt cracks on MO38 PCPS Slab

(a) Transverse view

(b) Close up at widest point

Figure 6-49 Gap Observed Between the Bridge Deck and the Asphalt Overlay on MO38

6.4 Summary

The experimental results indicate that the all three (3) BASs are performing well after approximately 12-18 months of service in the field. Due to the precision of the inclinometers, it is hard to determine field rotations to more than 0.01 degrees. However, the rotations produced by the inclinometers are extremely small indicating that the BASs are fully supported by the base material. The rotations in the transverse (x-direction) were negligible. The measured deflections were small indicating no settlement during the BASs service life of roughly one and a half years.

The VWSGs experienced a change of 1 micro-strain during the first load test. This indicates little flexure in the BAS and that the load is being immediately supported by the base material. The moisture gauges placed below the instrumented PCPS panels on the east side of the bridge did not show significant changes in VWC for the yearlong service life. This indicates that the water that is permeating through the joints is being properly drained by the base material. When crossing the BASs in a vehicle, all three designs provide a smooth transition and show no signs of a "bump" in the road. The new designs show substantial cost savings creating great alternative and cost efficient solutions to the current designs.

CHAPTER 7 **LABORATORY TESTING**

7.1 PCPS Concrete Material Testing

This section of the report includes the casting and mechanical tests performed on one quality assurance/quality control (QA/QC) PCPS slab specimen cast at Coreslab Structures, Inc. These tests were performed to verify initial design strengths and to provide additional insight on the long-term structural performance and behavior of the concrete used for the PCPS BASs. The tests conducted were modulus of elasticity (MOE), compressive strength, creep, shrinkage, and modulus of rupture (MOR). The testing facilities at Missouri S&T were used to perform all of these tests.

The PCPS BASs were cast at Coreslab Structures, Inc. in Marshall, MO. on July 07, 2011. The casting included 10 panels plus the QA/QC specimens. All 10 BAS panels were cast on the same casting bed in 3 pours. All QA/QC specimens were generated on site to accurately represent casting conditions such as temperature and humidity. After the specimens were cast at Coreslab Structures, Inc., they remained with the BAS panels for 24 hours to ensure the curing conditions were the same. Upon removal of the panels, the QA/QC specimens were transported to Missouri S&T's testing laboratories in Rolla, MO. Three 4 in. by 8 in. cylinders and two modulus of rupture (MOR) beams were placed outside to represent the environmental conditioning the panels were experiencing while being stored at Coreslab Structures, Inc. The remainders of the specimens were placed inside the high-bay Structural Engineering Research Laboratory (SERL) at Missouri S&T. The creep and shrinkage specimens were stored and loaded in the SERL basement where the creep frames are located. This location has reduced temperature fluctuations during testing. The information provided in Table 7-1 indicates the test, American Society for Testing and Materials (ASTM) testing method, sizes of specimens, and the dates tested.

7.2 QA/QC Specimen Testing

7.2.1 **Modulus of Elasticity**

The MOE test provides a stress to strain ratio of hardened concrete. The MOE is based upon a stress range of 0 to 40% of the ultimate concrete compressive strength. Some applications of the MOE include computing stress for measured strains [14]. ASTM C469-10 "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression" was used to determine the MOE using two 4 in. diameter by 8 in. long concrete cylinders. The testing was performed in the Butler-Carlton Civil Engineering Materials Load Frame Laboratory using a 600 kip Forney and 400 kip Tinius-Olsen capacity compression testing machines. The MOE specimens were loaded at a constant rate between 35 ± 7 psi/s. During loading, the load and longitudinal strain is recorded when the strain is 50 millionths and the applied load is 40% of the ultimate compressive load. The MOE was then calculated to the nearest 50,000 psi. The test setup is shown in Figure 7-1.

Table 7-1 Specimen Testing Details

Tests Performed	ASTM Test Method	Specimen Size	Testing Dates
Modulus of Elasticity	C469-10	4 in dia. X 8 in cylinder	28 days, 1 year after 28 day
Compressive Strength	C39-11a	4 in dia. X 8 in cylinder	28 days, 1 year after 28 day
Shrinkage	C157-08	4 in dia x 24 in cylinder	24 hours after panels were cast
Creep	C512-10	4 in dia x 24 in cylinder	40 days after panels were cast
Modulus of Rupture	C78-10	6 in x 6 in x 24 in	28 days, 1 year after 28 day

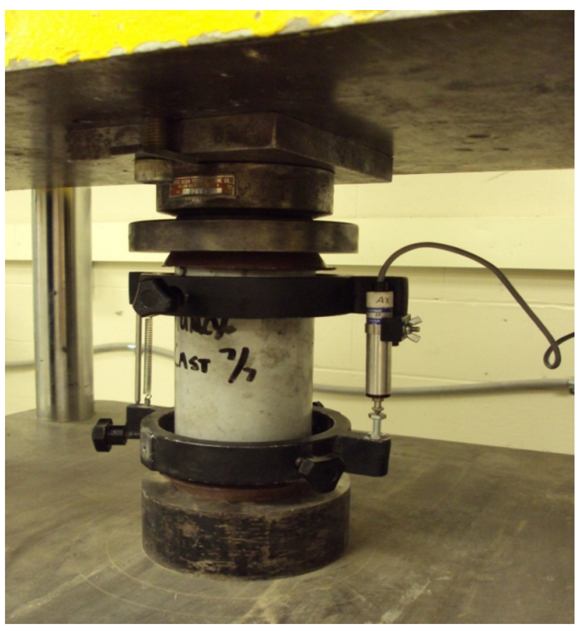


Figure 7-1 Test setup for compressive strength in the Tinius-Olsen

7.2.2 **Compressive Strength**

The compressive strength was determined on 4 in diameter by 8 in. long cylinders following ASTM C39 (2011) "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens." All compressive strength tests were performed in Butler-Carlton Civil Engineering Materials Load Frame Laboratory using the aforementioned Forney and Tinius-Olsen compression machine with an applied loading rate of 35 ± 7 psi/s [15]. The Forney was used for the 28 day compressive strength results while the Tinius-Olsen was used for the 1 year and 28 day results. The hardened concrete cylinders were capped prior to testing with sulfur mortar following ASTM C617 (2011) "Standard Practice for Capping Cylindrical Concrete Specimens" [16].

7.2.3 **Creep and Shrinkage**

Creep and shrinkage are important when designing and accounting for prestress losses in prestressed concrete structures. As a concrete beam or panel shortens due to concrete creep and shrinkage, the prestressing tendons relax and reduce the force applied to the concrete member [17]. Creep is accounted for in the ACI 318-08 Building Code Requirements for Structural Concrete for flexural members under a sustained load. When a flexural member is exposed to a sustained load, time-dependent factors are used to account for the long term deflection from creep [18].

ASTM C512 (2010) [19] and ASTM C157 (2008) [20] were used to determine the creep and shrinkage, respectively. Cylinders that were 4 in. in diameter and 24 in. long were used to perform these tests. These cylinders were constructed by attaching polyvinyl chloride (PVC) pipe to a wooden base using silicone. The cylinders were removed from the molds 28 hours after casting and demountable mechanical (DEMEC) points were installed in 9 different areas using a quick set epoxy. After the epoxy cured, baseline readings were measured and recorded. The total strain of the specimen was determined by averaging all 9 readings per cylinder. Figure 7-2 shows the layout of the DEMEC discs and a cylinder loaded into a creep frame.

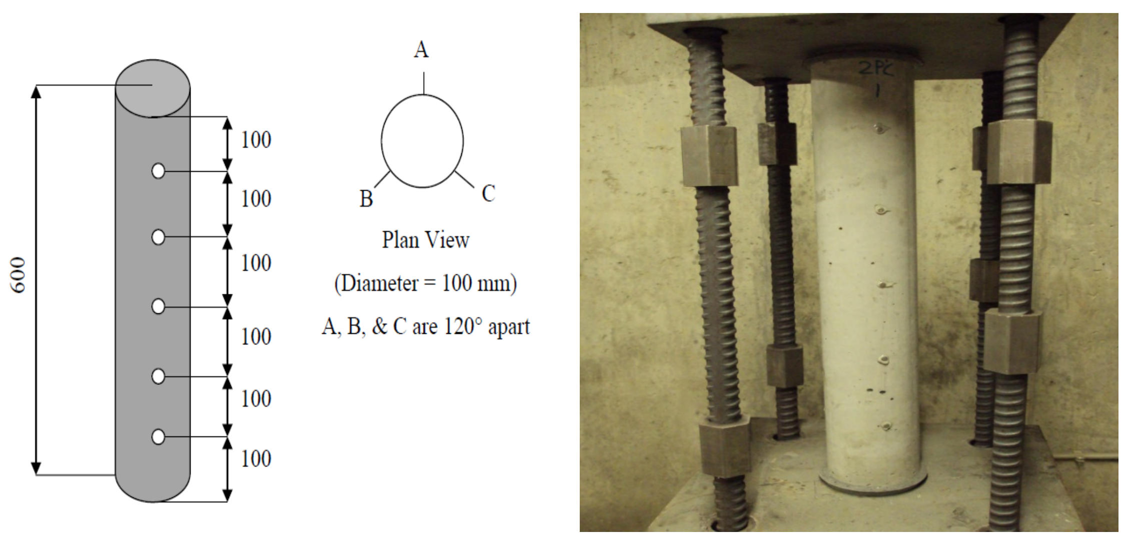


Figure 7-2 DEMEC Point Locations and Cylinder Loaded in Creep Frame [21]

ASTM C157 (2008) “Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete” requires that the room where the cylinders are stored must maintain a temperature and humidity of 73 ± 3 °F and 50 ± 4 %, respectively. The cylinders were stored in the basement of the high-bay SERL in Butler Carlton Hall at Missouri S&T to maintain a consistent temperature and humidity. The humidity varied from 32% to 72% in the laboratory basement. Despite these fluctuations, a majority of the relative humidity readings met the ASTM standards. Measurements were taken more often during the first few weeks to more accurately determine the shrinkage behavior. The cylinders were not moist cured in order to better represent field conditions of the BAS where the in-situ panels are not subjected to continuous moisture; however, the laboratory non-moist curing may be considered slightly more conservative than the panels in the field. After measuring the shrinkage for 40 days one of the three cylinders was loaded into a creep frame. Forty days was selected to approximately correspond to the date when the BAS would be loaded in the field.

It should be noted that two creep cylinders were originally loaded but due to deformation of the plate during loading, failure occurred around the edge of one creep cylinder so it was discarded. The cylinder was loaded at 40% of the 28 day compressive strength and measurements were recorded according to ASTM C 512 (2010) “Standard Test Method for Creep of Concrete in Compression.” The load frame was constructed using a solid steel stand, four springs with an approximate stiffness of 10.3 kip/in, two 2-in. thick metal plates, and four 4-grade 75 bars with couplers. Figure 7-3 details the dimensions and setup of the creep frame.

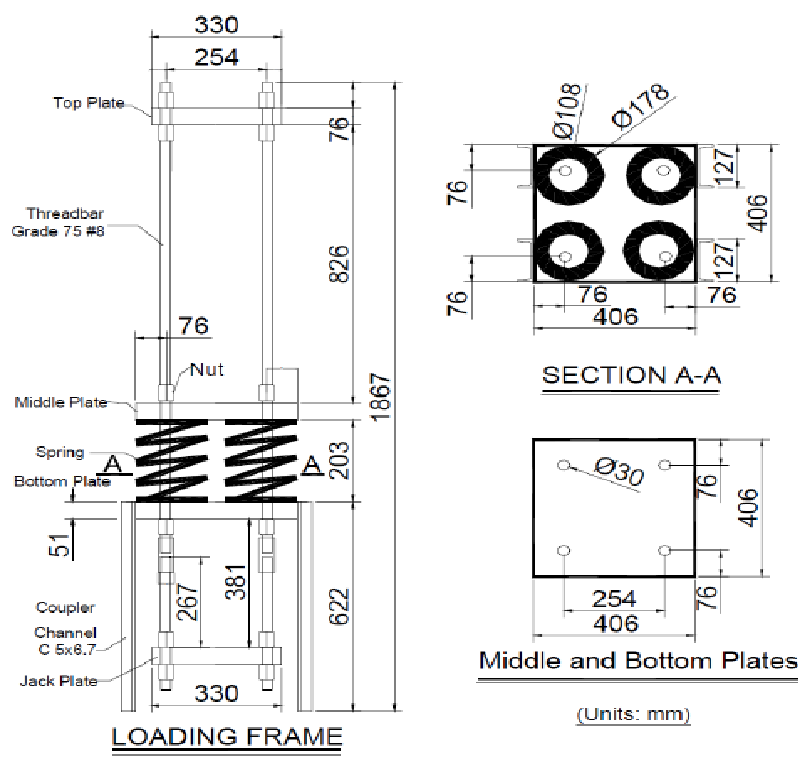


Figure 7-3 Dimensions and Details of the Creep Loading Frames [21]

The creep cylinders were loaded by placing a jack below the creep frame and loading to approximately 40% ± 2% of the 28 day compressive strength. Once the desired axial load was achieved, the connectors were tightened to maintain the load. The jack and load cell was then removed. The lengths of the springs were recorded after the load was applied in order to make adjustments for spring relaxation, should it occur. The length of the springs was measured periodically and all strain measurements were recorded at the same durations as the shrinkage cylinders.

7.2.4 **Modulus of Rupture**

The modulus of rupture (MOR) was performed in the Butler-Carlton Civil Engineering Materials Load Frame Laboratory using the aforementioned Tinius-Olsen testing machine. Concrete specimens with dimensions of 6 in. by 6 in. by 21 in. were tested following ASTM C78 (2010) "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)." The beams were loaded at a rate of 30 lb/s until failure occurred in the tension face of the beam [22]. The MOR load test setup can be seen in Figure 7-4.

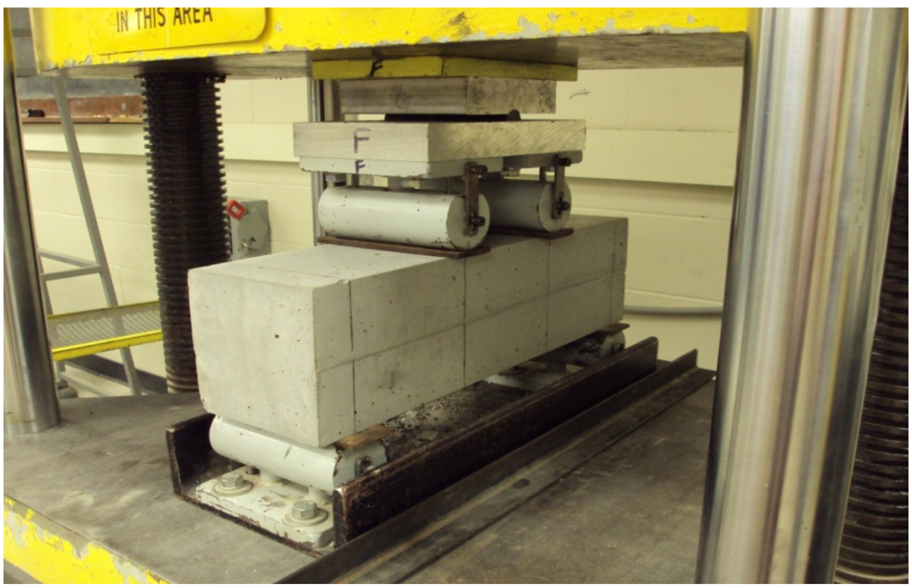


Figure 7-4 MOR Test Setup

7.2.5 **Results**

The results for the MOE, compressive strength and MOR are reported in Table 7-2. Each MOE and MOR test was based on 2 QA/QC specimens while the compressive strength had 3. The PCPS panels reached the design strength of 6000 psi after 28 days of curing. A year later the tested compressive strength was 6910 psi which was slightly higher than the results on 8/04/2011. This is expected due to the long term hydration of the cement. As the strength of the concrete increased, the stiffness of the paste and aggregate matrix increased. The MOE and MOR test results were compared to the ACI empirical equations from the ACI 318-08 Building Code, respectively. These equations are shown in Eq. 7-1 and Eq. 7-2, respectively.

$$E_c = 57{,}000 * \sqrt{f'c} \quad \text{Eq. 7-1}$$

$$f_r = 7.5 * \lambda * \sqrt{f'c} \quad \text{Eq. 7-2}$$

The experimental MOE results were lower than the predicted empirical values based on the ACI 318-08 equations. Eq. 7-1 was designed to predict MOE based on an average of conventional concrete strengths. Previous research indicates that a lower MOE was observed due to a low coarse aggregate content and the result of a softer limestone being produced from the quarry. The experimental MOR results were greater than the values calculated from Eq. 7-2 from ACI 318-08. This indicates a conservative approach when calculating the MOR, which is desired.

Table 7-2 Results of QA/QC Specimens

Test Performed	ASTM Test Method	Testing Date	Results (psi)	ACI 318-08 (psi)
Modulus of Elasticity	C469-10	8/04/2011	3,860,000	4,560,000
		8/04/2012	4,100,000	4,740,000
Compressive Strength	C39-11a	8/04/2011	6,400	N/A
		8/04/2012	6,910	N/A
Modulus of Rupture	C78-10	8/04/2011	613	600
		8/04/2012	773	623

The testing results for the shrinkage and creep specimens are outlined in Figure 7-5 and Figure 7-6, respectively. Both tests were compared to the ACI 209R-92 model which is used to estimate creep and shrinkage for hardened concrete [23]. There are four models presented within the ACI 209 document. The ACI209R-92 was chosen based on the information available from casting the PCPS panels and the model offers a variety of correction factors specific to each test.

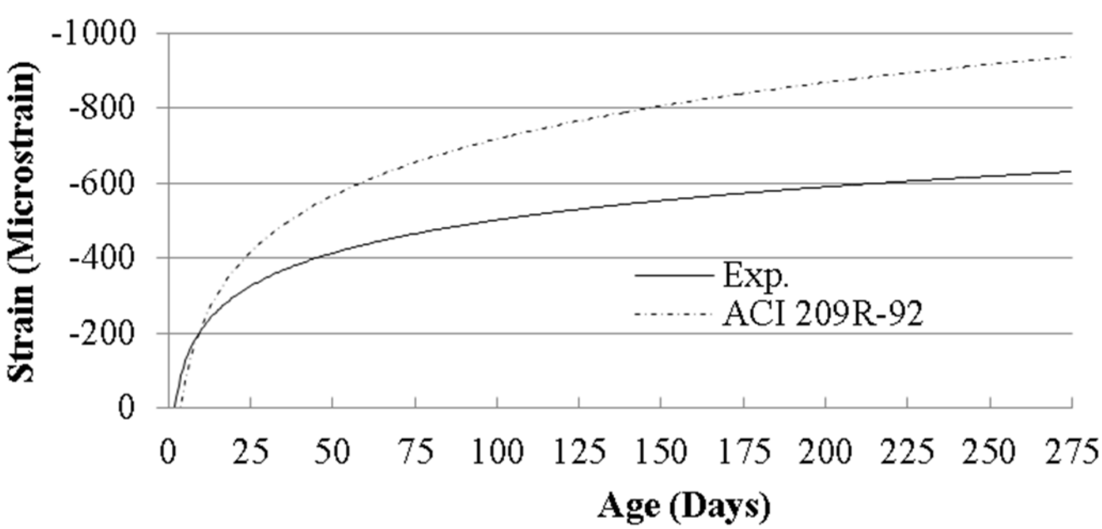


Figure 7-5 Shrinkage Results for the Concrete used in the PCPS Panels

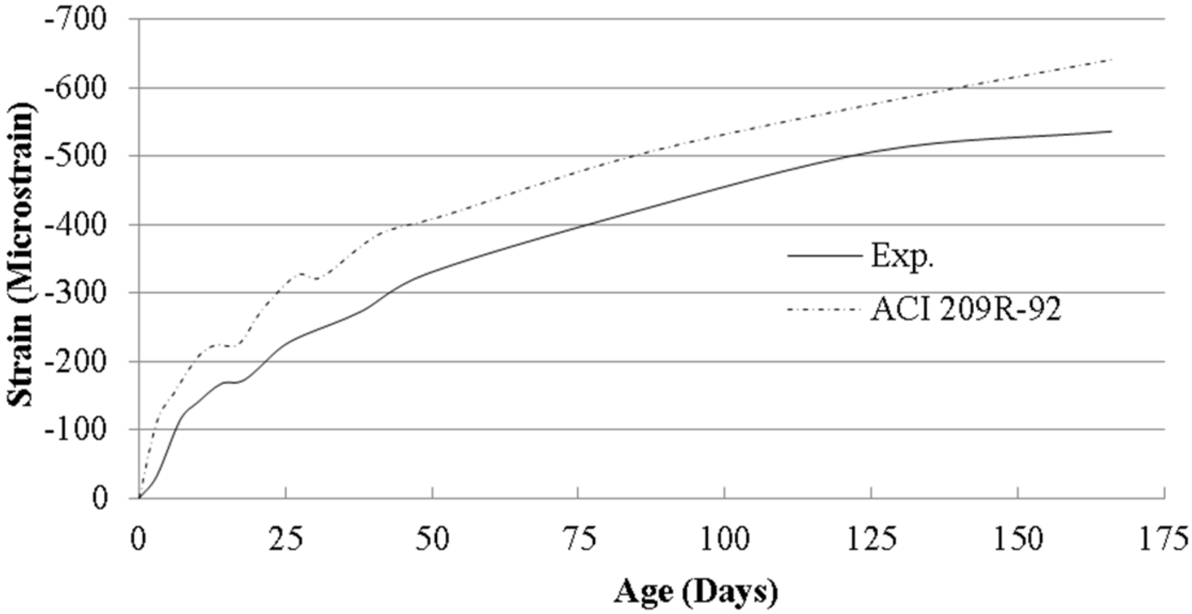


Figure 7-6 Creep Results for Concrete used in the PCPS Panels

Figure 7-5 exhibits the experimental shrinkage data compared to the ACI 209 shrinkage model. As an average empirical equation, the ACI 209 states that the model underestimates the shrinkage for low shrinkage values and overestimates for high shrinkage values. This is true for the mix used for the panels. After 7 days, the model exceeds shrinkage for the panel mix. The difference between the model and experimental data was nearly 300 microstrains at 275 days. This indicates that the model is very conservative at moderate and later-ages. If additional fresh concrete properties were available, a different shrinkage model could be pursued to better estimate shrinkage in the 6000 psi mix used for the PCPS panels. Eq. 7-3 to Eq. 7-13 show the equations used for calculating shrinkage

$$\varepsilon_{sh}(t, t_c) = \frac{(t-t_c)^{\alpha}}{f+(t-t_c)^{\alpha}} \times \varepsilon_{shu}\ (\mu\varepsilon) \qquad \text{Eq. 7-3}$$

$$f = 26.0e^{\{0.36(V/S)\}} \qquad \text{Eq. 7-4}$$

$$\varepsilon_{shu} = 780\gamma_{sh} \times 10^{-6}\ (\mu\varepsilon) \qquad \text{Eq. 7-5}$$

$$\gamma_{sh}=\gamma_{sh,tc}\gamma_{sh,RH}\gamma_{sh,vs}\gamma_{sh,s}\gamma_{sh,\psi}\gamma_{sh,c}\gamma_{sh,\alpha} \qquad \text{Eq. 7-6}$$

$$\gamma_{sh,tc} = 1.202 - .2337\log(t_c) \qquad \text{Eq. 7-7}$$

$$\gamma_{sh,RH} = \begin{matrix} 1.40 - 1.02h & \text{for } 0.40 \le h \le 0.80 \\ 3.00 - 3.0h & \text{for } 0.80 \le h \le 1 \end{matrix} \qquad \text{Eq. 7-8}$$

$$\gamma_{sh,vs} = 1.2e^{\{-0.12(V/S)\}} \qquad \text{Eq. 7-9}$$

$$\gamma_{sh,s} = 0.89 + 0.041s \qquad \text{Eq. 7-10}$$

$$\gamma_{sh,\psi} = \begin{matrix} 0.30 + 0.014\psi & \text{for } \psi \le 50\% \\ 0.90 + 0.002\psi & \text{for } \psi > 50\% \end{matrix} \qquad \text{Eq. 7-11}$$

$$\gamma_{sh,c} = 0.75 + 0.00036c \qquad \text{Eq. 7-12}$$

$$\gamma_{sh,\alpha} = 0.95 + 0.008\alpha \geq 1 \quad \text{Eq. 7-13}$$

Figure 7-6 indicates that the ACI 209 creep empirical model overestimates the creep by roughly 80 microstrains after the concrete has experienced 28 days of sustained loading. The model is conservative compared to the concrete mix used for the PCPS panels. Eq. 7-14 to Eq. 7-23 show the equations used to calculate creep.

$$\Phi(t, t_0) = \frac{(t - t_0)^{\psi}}{d + (t - t_0)^{\psi}} \Phi_u \quad \text{Eq. 7-14}$$

$$\Phi_u = 2.35\gamma_c \quad \text{Eq. 7-15}$$

$$d = 26.0e^{\{0.36(V/S)\}} \quad \text{Eq. 7-16}$$

$$\gamma_c = \gamma_{c,to}\gamma_{c,RH}\gamma_{c,vs}\gamma_{c,s}\gamma_{c,\psi}\gamma_{c,\alpha} \quad \text{Eq. 7-17}$$

$$\gamma_{c,to} = \begin{matrix} 1.25{t_0}^{-0.118} & \text{for moist curing} \\ 1.13{t_0}^{-0.094} & \text{for steam curing} \end{matrix} \quad \text{Eq. 7-18}$$

$$\gamma_{c,RH} = 1.27 - 0.67h \quad \text{Eq. 7-19}$$

$$\gamma_{c,vs} = \frac{2}{3}(1 + 1.13e^{\{-0.54(V/S)\}}) \quad \text{Eq. 7-20}$$

$$\gamma_{c,s} = 0.82 + 0.067s \quad \text{Eq. 7-21}$$

$$\gamma_{c,\psi} = 0.88 + 0.0024\psi \quad \text{Eq. 7-22}$$

$$\gamma_{c,\alpha} = 0.46 + 0.09\alpha \geq 1 \quad \text{Eq. 7-23}$$

7.2.6 **Prestress Losses**

Prestress losses calculations/estimates are essential in the design of prestressed concrete members. Prestress losses must be accounted for to ensure the design does not exceed its serviceability and fiber stress limit states at different design stages. The losses used for this research and discussed in this section include elastic shortening of the concrete, creep and shrinkage of the concrete, and relaxation of the prestressing tendons.

Elastic shortening was calculated when the tendons were released using Eq. 7-24 where n is the modular ratio of the steel and concrete and f_{cs} is the initial prestressing force per area of concrete [21]. Creep prestress losses were calculated using Eq. 7-25 where C_t is the creep coefficient [24]. The creep coefficient was determined from the results. Shrinkage losses were calculated by multiplying the experimental strain from Figure 7-6 by the modulus of elasticity of the prestressing tendons. The prestressing losses due to relaxation of the prestressing tendons were determined by Eq. 7-26 [21] where f'_{pi} is the initial stress in the steel, f_{py} is the yield stress of the steel, and t is in hours. The total losses are shown in Table 7-3 when each field load test was performed.

$$\Delta f_{pES} = nf_{cs} \quad \text{Eq. 7-24}$$

$$\Delta f_{pCR} = C_t n f_{cs}$$ Eq. 7-25

$$\Delta f_{pR} = f'_{pi}(\log(t)/45)((f'_{pi}/f_{py})-.055)$$ Eq. 7-26

Table 7-3 Prestress Losses

Test	Prestress Losses				Total Loss	
	ES (psi)	SH (psi)	CR (psi)	R (psi)	Stress (psi)	% of Jacking
Load Test 1	6208	14445	5311	2984	28948	14.3
Load Test 2	6208	16740	7649	2897	33494	16.6

7.3 Laboratory Testing of the Test PCPS Panel

An additional PCPS BAS panel was constructed by Coreslab Structures, Inc. in Kansas City, MO. The PCPS BAS panel was constructed so researchers could evaluate the BASs performance under various washout conditions and to verify the ultimate strength of an individual panel. The field study used panels that are 20 ft. x 6 ft. x 10 in. but it was later determined that an 8 ft. wide panel is more economical. This laboratory test consists of an 8 ft. wide panel instead of a 6 ft. wide panel used on Bridge A7767. The PCPS panel was cast on 9/11/2012. Prior to casting 2 in. x 4 in. box outs were placed at specified locations on the bottom surface of the panel to later install surface mounted strain gauges. Once the panel was removed from the formwork, it was transported to the Structural Engineering Research Laboratory in Butler Carlton Hall at Missouri S&T on 9/20/2012.

7.3.1 Aggregate Base Test Setup for Washout Testing

The test setup began with installing a containment box lined with plastic polyethylene sheeting for the aggregate base. The containment box was approximately 9 ft. – 6 in. wide by 26 ft. in length. The roller supports for the panel were placed in the box before adding the aggregate base. Once the supports were in place, the type 5 aggregate base was placed in the containment box as shown in Figure 7-7. All aggregate was obtained from Capital Quarries in Rolla, MO.

The depth of the type 5 aggregate base was 4 in. which was used to replicate field conditions and meet MoDOT's current specifications. Once the aggregate was level it was compacted using a Wacker WP 1500 portable vibratory compactor as shown in Figure 7-8. This was the same compactor used for the aggregate base on Bridge A7767. In order to create full bearing support, sand was added near the supports and at low points as needed.

Figure 7-7 Placement of the aggregate base

Figure 7-8 Base Material Compaction

7.3.2 **Testing of Aggregate Base**

Samples of the aggregate base material were taken to determine if the material met MoDOT's gradation requirements and a minimum compaction of 95%. A sieve analysis was performed using ASTM D6913-04 (2009) "Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis." A sample of approximately 11 lbs was used for the sieve analysis. Once the gradation was performed, the weight retained in each sieve was recorded. The sieves and graded test material are shown in Figure 7-9. The percent passing was calculated for each sieve and plotted against the sieve size as shown in Figure 7-10 which indicates that the base material meets MoDOT specifications for a type 5 aggregate base.

Figure 7-9 Sieves and Graded Material

The percent compaction was determined and compared to the MoDOT specifications. There were two tests performed to determine the percent compaction. ASTM D4253-00 “Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table” was used to determine the maximum density of the base material. A mold with known dimensions was filled to the top and the weight was recorded. Then the mold had a weight placed on the surface and was placed on a vibratory table for roughly 10 minutes as shown in Figure 7-11. The new dimensions of the base material in the mold were examined and recorded. The maximum unit weight was calculated to be 120 lbs/ft^3.

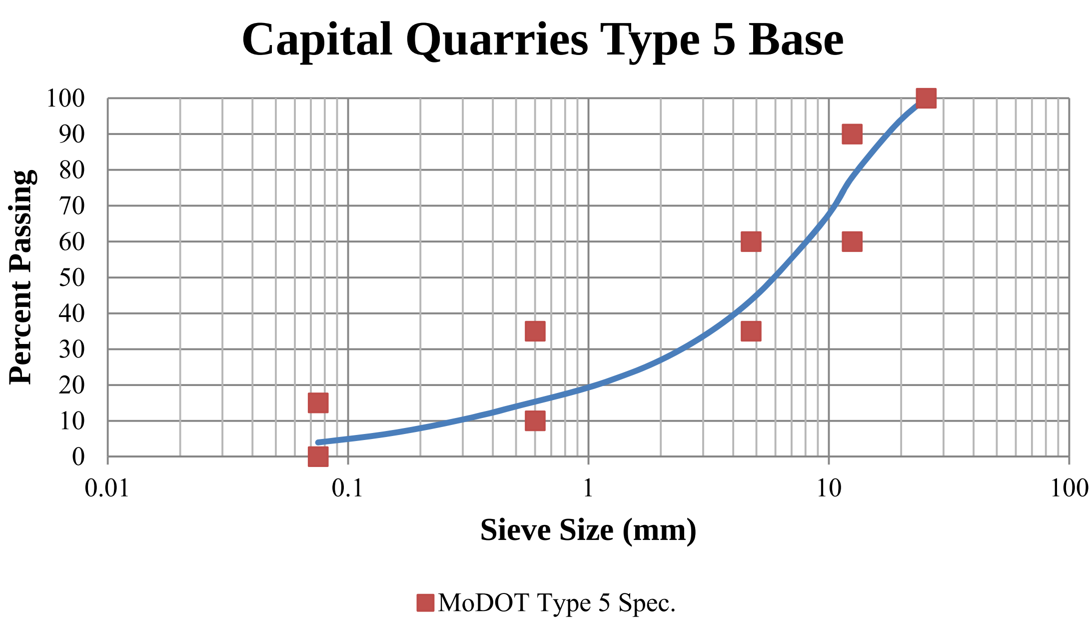


Figure 7-10 Gradation of Type 5 Base

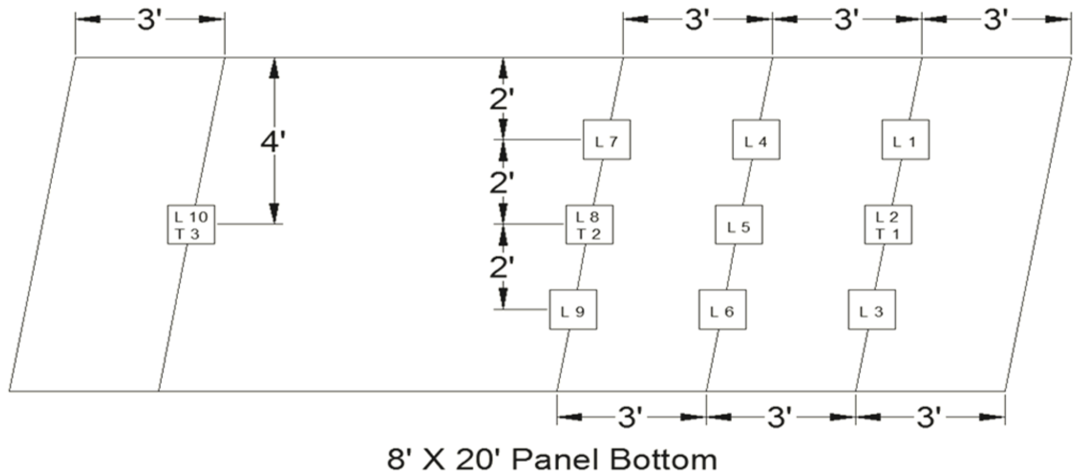
Figure 7-11 Vibratory Table and Compaction Mold

The in place density of the base material was determined using ASTM D1556-07 “Standard Test Method for Density and Unit Weight of Soil in Place by Sand-Cone Method” as shown in Figure 7-12. The weight of the sand and volume of the sand cone was recorded prior to testing. A hole was excavated according to the ASTM and the weight of the removed aggregate was recorded. Based on the weight of the removed aggregate and volume of the sand cone, the in place unit weight of the soil was calculated to be 114 lbs/ft^3. Based on the maximum density it was determined that the compaction of the base material met the MoDOT specification for a minimum compaction of 95%.

Figure 7-12 In Place Density Testing Using the Sand Cone Method

7.3.3 **Strain Gauges**

Surface mounted strain gauges were placed near the bottom of the panel to determine the behavior during the washout testing. The diagram showing the strain gauge locations is shown in Figure 7-13. Ten longitudinal and three transverse strain gauges were placed 1.5 in. from the bottom of the panel. A majority of the gauges were placed where the panel would experience a void/washout condition. Longitudinal gauges were the main interest of this study but transverse gauges were installed to evaluate bending in the slab perpendicular to the "flow of traffic".



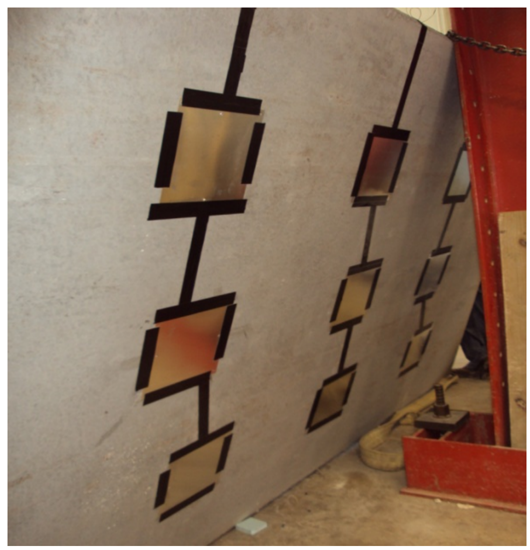
L-Longitudinal, T-Transverse

Figure 7-13 Strain Gauge Locations

When the panel arrived it was placed on its side to allow the strain gauges to be installed. This helped expose all the box outs as shown in Figure 7-14. The inside surface of the box outs were sanded to provide a rough surface profile for the adhesive. AE 10 adhesive from Vishay was placed on the surface to fill in any small voids. After six hours the adhesive was sanded down to the concrete surface and prepared for the installation of concrete strain gauges. Two inch concrete strain gauges were applied using the AE 10 adhesive and left overnight to bond to the concrete. Once the adhesive cured the wires were soldered to the gauges. Prior to load testing, each gauge was tested with a volt meter to ensure it was properly installed. The completed gauge installation is shown in Figure 7-15. The box outs were covered with a thin aluminum plate to protect the gauges and all wires were run in a groove to the edge of the panel. After the installation of the strain gauges was completed, the slab was placed on the base material and metal roller supports.

Figure 7-14 Exposure of Box Outs on the Bottom of the Panel

a) surface mounted strain gauge



b) bottom surface of panel prior to testing

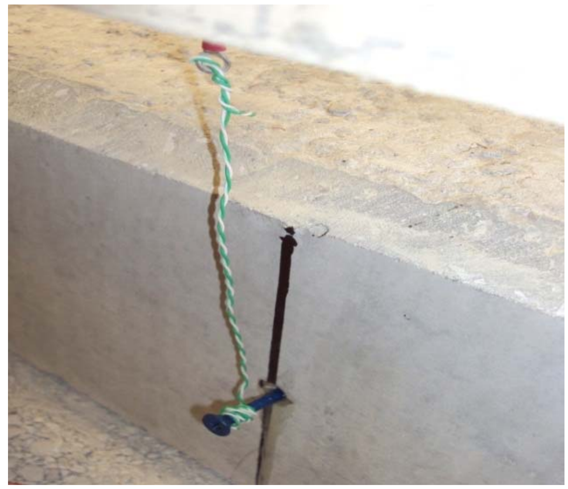
Figure 7-15 Completed Concrete Strain Gauge Installation

7.3.4 **String Potentiometers and Inclinometers**

Eight (8) string potentiometers were used to measure slab deflection as washout conditions were simulated. The string potentiometers measured deflections with a precision of 0.025 in. Each string potentiometer was attached to a wood base that would allow for easy removal during testing. Concrete anchors were installed along the edge of the concrete slab and wire was used to connect the anchor to the string potentiometer as shown in Figure 7-16. The location of each string potentiometer is shown in Figure 7-17. The location of the string potentiometers was designed to monitor changes in deflection at various washout conditions. In addition, inclinometers were placed at the locations shown in Figure 7-17 to monitor rotations. These were the same inclinometers used for the field testing.

The steel framing was constructed as shown in Figure 7-18. A primary beam consisting of a W21X62 wide flange section was used to carry the load applied by the hydraulic jack. The hydraulic jack rested on a 5 ft. long spreader beam to transfer the load to the bearing plates. The 24 in. x 16 in. x 1.5 in. bearing plates represented the area of dual tires on a truck. Prior to placing the plates on the concrete surface, fine sand was placed on the surface to create a uniform bearing pressure when the load was applied. Figure 7-19 shows the spreader beam and the bearing plates. Figure 7-20 indicates the bearing plate locations. The hydraulic jack applied a 16 kip load to the spreader beam which, distributed 8 kips to each bearing plate.

a) String potentiometer sensor



b) Lead measurement point

Figure 7-16 String Potentiometer Installation

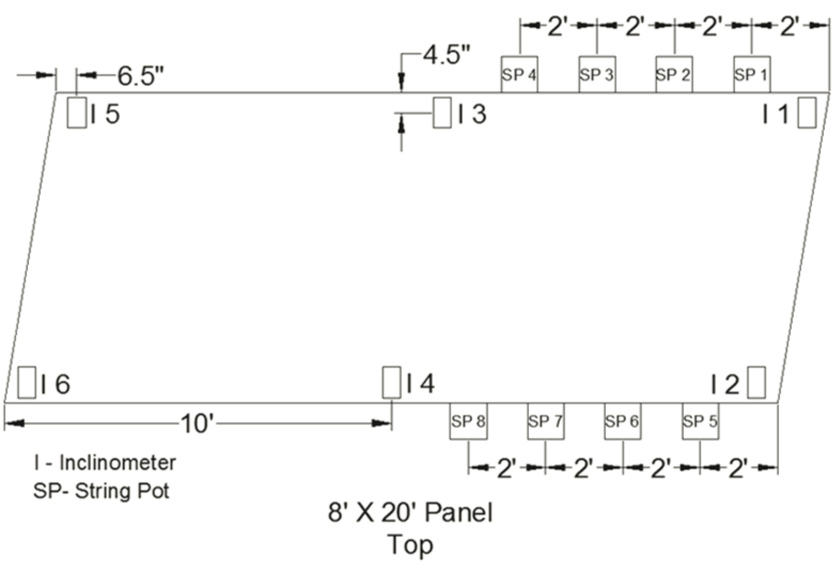


Figure 7-17 String Potentiometer and Inclinometer Locations

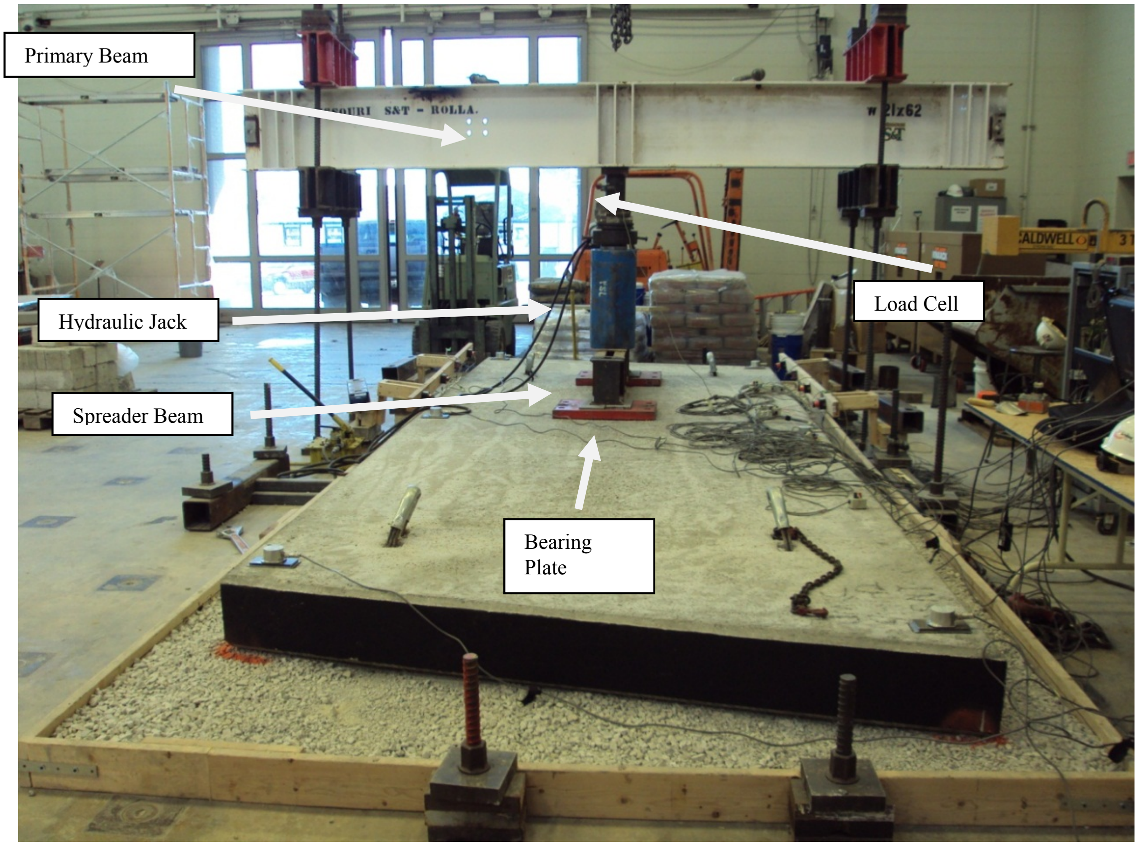


Figure 7-18 PCPS Panel Test Setup

Figure 7-19 Spreader Beam and Bearing Plates

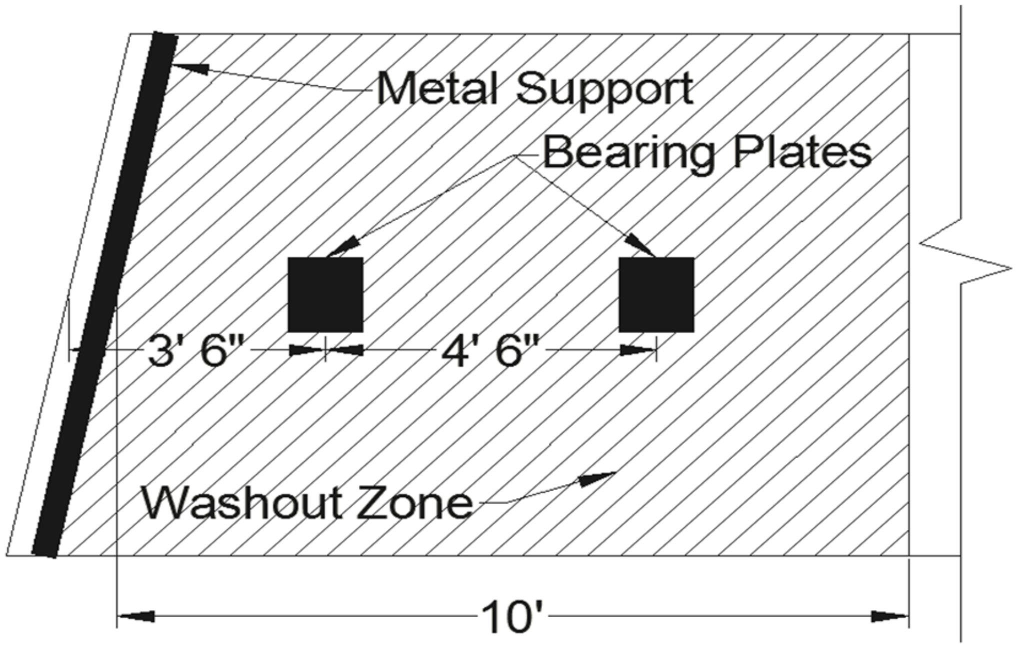


Figure 7-20 Bearing Plate Location

7.3.5 **Test Setup for Loading Panel to Failure**

When testing the panel to flexural failure, a similar steel frame was constructed as outlined earlier in this section. The panel was simply supported on two roller supports as shown in Figure 7-21. The roller supports were placed on a 12 in. structural steel wide flange sections with web stiffeners. The panel was supported at 6 in. from the end producing a longitudinal clear span of 19 ft. The test setup prior to testing is shown in Figure 7-22. This figure shows an LVDT used to monitor deflection. It was later determined that string potentiometers would be used instead of LVDTs.

The same instrumentation was used as outlined earlier in the section. Due to placement of the beam across the width of the panel, inclinometers 3 and 4 were not used for this test. String potentiometers were placed at the center edge of the beam to record maximum deflection produced by the line load. See Figure 7-23 for the instrumentation locations. Note that all strain gauges were used for this test. The layout is the same as shown in Figure 7-13.

Figure 7-21 Panel End Support When Testing to Failure

Figure 7-22 Testing Apparatus for Testing Panel to Failure

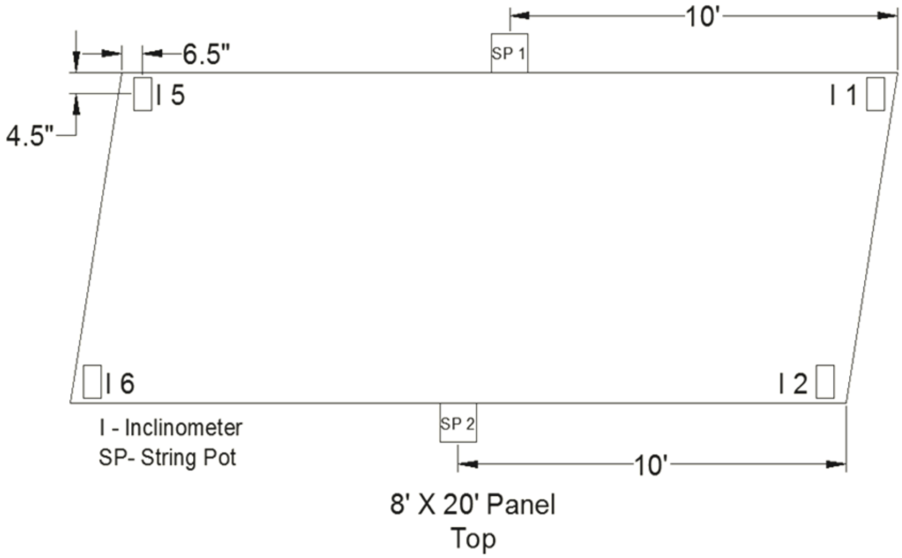


Figure 7-23 Inclinometer and String Potentiometer Layout for Testing Panel to Failure

After the washout tests were completed, a load test to failure was initiated. This test consisted of placing the spreader beam directly on the panel surface. Due to a small localized failure, as a result of a misaligned end plate on the wide flange used that was initially bearing on the edge panel, shown in Figure 7-24, the test was stopped, spalled concrete removed, and a 2 ft. wide by 1 in. thick neoprene pad was placed between the beam and concrete surface to provide uniform bearing support (Figure 7-25). It should be noted that the panel was not loaded above the cracking load when the localized failure occurred. Therefore, the panel remained in the elastic region such that the panel could be unloaded; neoprene pad placed, and load testing re-initiated. The minor edge spalling caused by the end plate was not considered to affect the overall structural behavior of the panel due to the minor spall.

Figure 7-24 Localized Failure Due to Non-Uniform Pressure of Line Load

Figure 7-25 Spreader Beam with Neoprene Pad

7.3.6 **Test Matrix**

The testing of the PCPS BAS panel was designed to test the performance of the panel at various washout lengths should erosion occur in the field. Six loading cases were performed as shown in Figure 7-26. The slab was loaded with full elastic support then with 2 ft. removal of base support up to 10 ft.

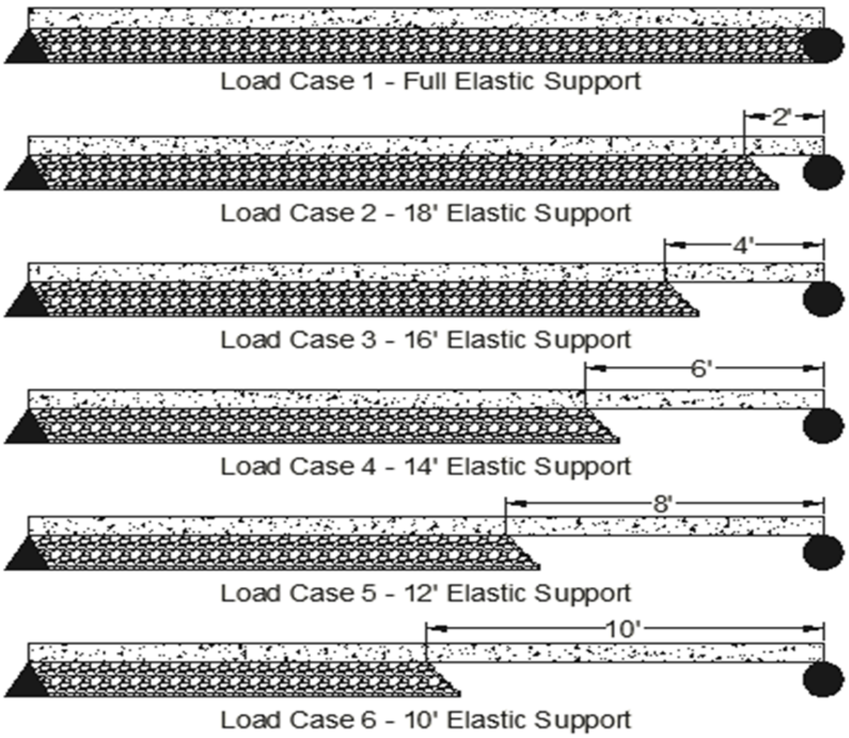


Figure 7-26 20 ft. PCPS BAS Panel washout load cases

Three repetitions were performed for each test. Once all washout conditions were tested, the panel was loaded to failure. For failure, the panel was placed on simple supports and a line load was placed across the center of the slab. Rotation, deflection, and strain values were recorded for each test and the results are discussed in the next section.

7.4 Results

The results discussed in this section include the experimental results for the testing of the PCPS panel at Missouri S&T's SERL. This includes the rotations observed by the inclinometers, strains at 1.5 in. from the bottom surface of the slab, and deflections measured by the string potentiometers. The results from simulating washout conditions and the results from loading the panel to failure are also discussed.

7.4.1 **Testing of Washout Conditions**

The panel was tested to determine its performance based on various washout conditions. This was performed to simulate erosion should water permeate through the PCPS BAS in the field. The test simulated washout conditions in 2 ft. increments up to 10 ft. The design of the PCPS BAS incorporated 50% base support which is reflected in this experiment using 10 ft. of washout.

7.4.2 **Inclinometers**

The rotations measured from the inclinometers are compared to the washout conditions in Figure 7-27. Refer to Figure 7-17 for inclinometer locations. All rotations evaluated are parallel with the length of the slab. There were no noticeable rotations detected by the inclinometers in the transverse direction. As the washout length increased, the rotation in the inclinometers placed at the center (inclinometers 3 & 4) increased after approximately 4 ft. of washout. Inclinometers 1-4 experienced more rotation than 5 & 6. Inclinometers 5 & 6 were on the opposite end of the washout and on the portion of the slab that was fully supported during the entire test. Although there are differences between each inclinometer, the rotation is minimal which reflects the readings taken during the field testing. If the slab reaches a washout condition up to 50%, the rotations are not considered an issue.

7.4.3 **String Potentiometers**

The string plots measured the deflection of the slab where base material was removed. Refer to Figure 7-17 for string potentiometer locations. As the base material was removed to simulate washout, the deflection in the panel was recorded. The results are shown in Figure 7-28. It may be observed that the panel experiences negligible deflections with up to 2 ft. of washout. The results at 4 ft. of washout and greater are not reliable values based on the precision of the string potentiometers. Since the precision is 0.025 in., it is safe to assume the deflections are minimal and are not a concern even at a 50% loss in base material.

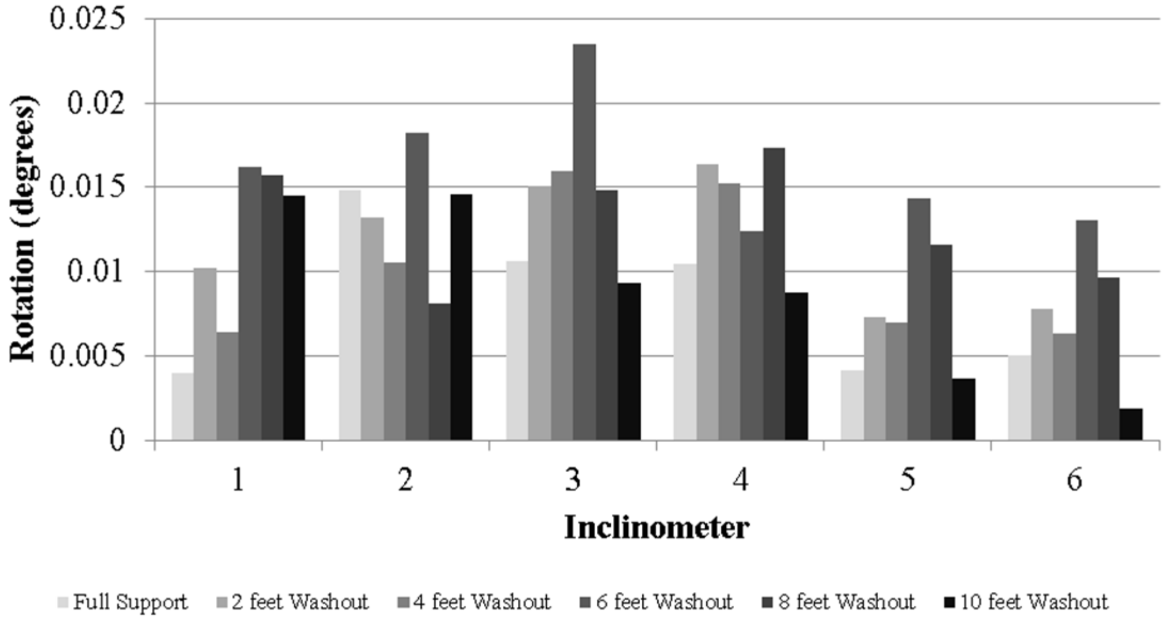


Figure 7-27 PCPS Panel Rotations Due to Washout

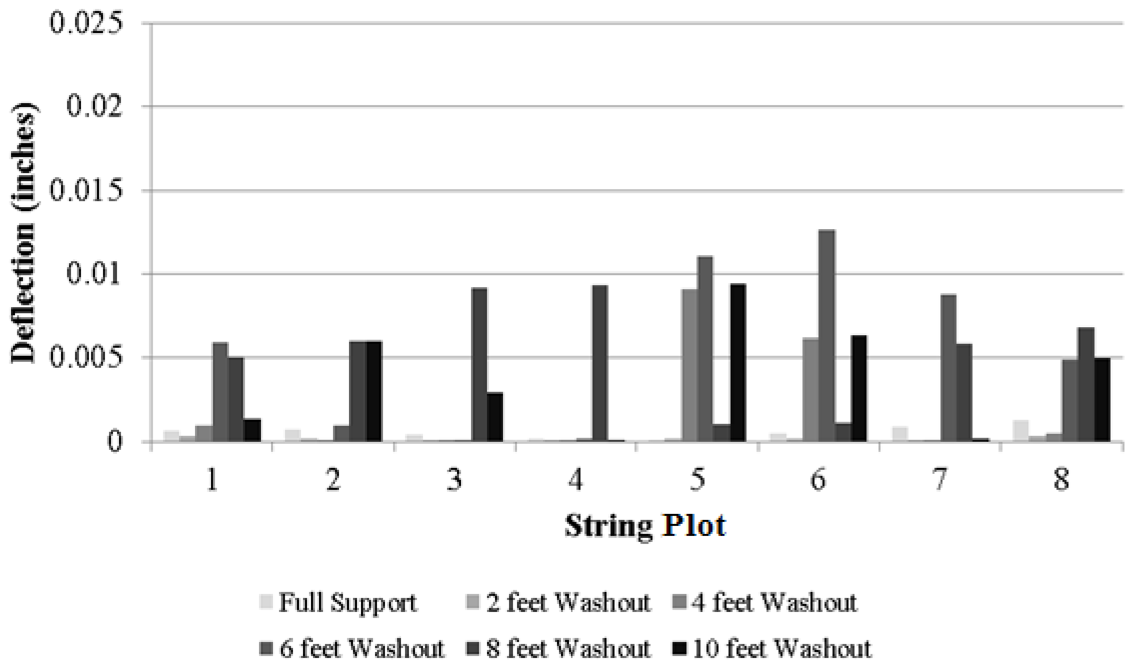


Figure 7-28 PCPS Panel Deflections Based on Washout Lengths

7.4.4 Strain Gauges

Surface mounted strain gauges were used to measure strain at 1.5 in. from the bottom of the slab. Refer to Figure 7-13 for location of the strain gauges. The gauges were analyzed and graphed according to their location on the slab. Each graph contains the strain measurement relative to each washout condition. The graphs for longitudinal strain gauges 1-9 are shown in Figure 7-29 to Figure 7-31.

Strain measurements for gauges 1, 2, and 3 are shown in Figure 7-29. These three gauges were located 3 ft. from the support. This figure indicates that a washout of 6 ft. causes the greatest amount of strain in all three gauges. Gauge 2, which is located in the center of the panel, experiences more strain for each loading case relative to gauges 1 and 3. This is expected due to the location of the bearing plate. These gauges tend to have higher strains than gauges 4-9 because of the amount of bending that occurs at these locations.

Strain gauges 4-6 produced a maximum strain during 6 ft. of washout. The trends are similar to Figure 7-29 but the overall strain values are much lower. This is caused from the location of the gauges and the supporting base material. Figure 7-31 displays gauges 7-9. These gauges are located 9 ft. from the support and show lower strain values than Figure 7-29 and Figure 7-30. As the gauges approach the longitudinal center of the panel, the strains decrease because of the support from the base material. The maximum strain is reached when the panel has a washout of 8 ft. Longitudinal strain gauge 10 was not included in the figures because it showed minimal strain during all washout conditions.

All longitudinal gauges experience similar behavior. As the gauges distance from the support increases, the amount of strain decreases. This is expected because of the increase in support from the base material. It must be noted that the noise produced from the data acquisition system varies between 10-15 microstrain. The higher readings for the full support condition are produced from settling or crushing of the base material.

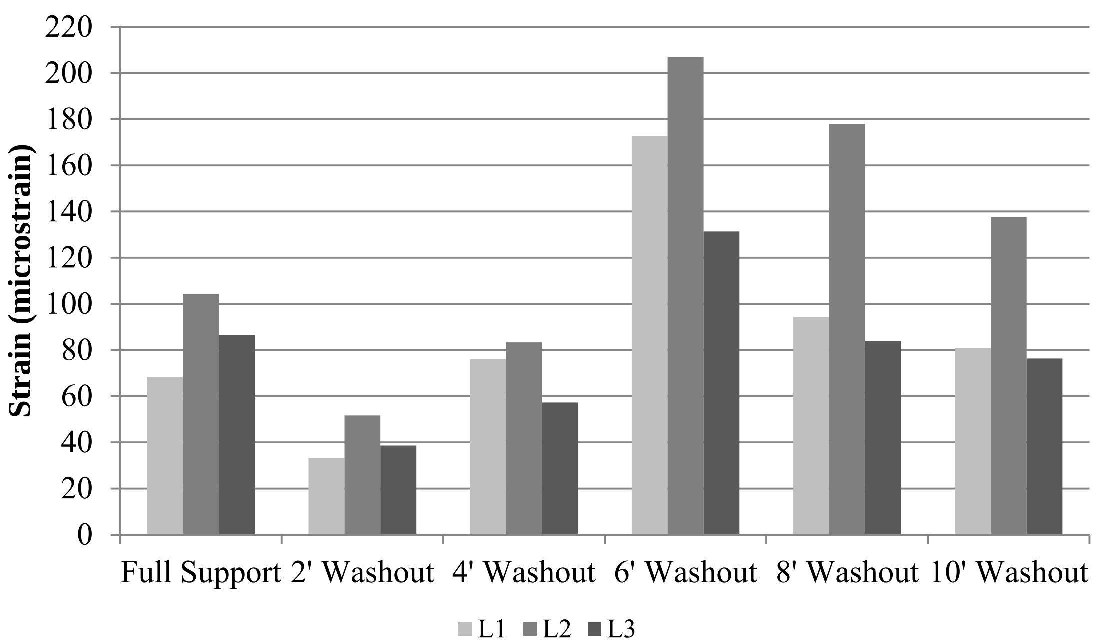


Figure 7-29 Strain in Gauges 1, 2, and 3

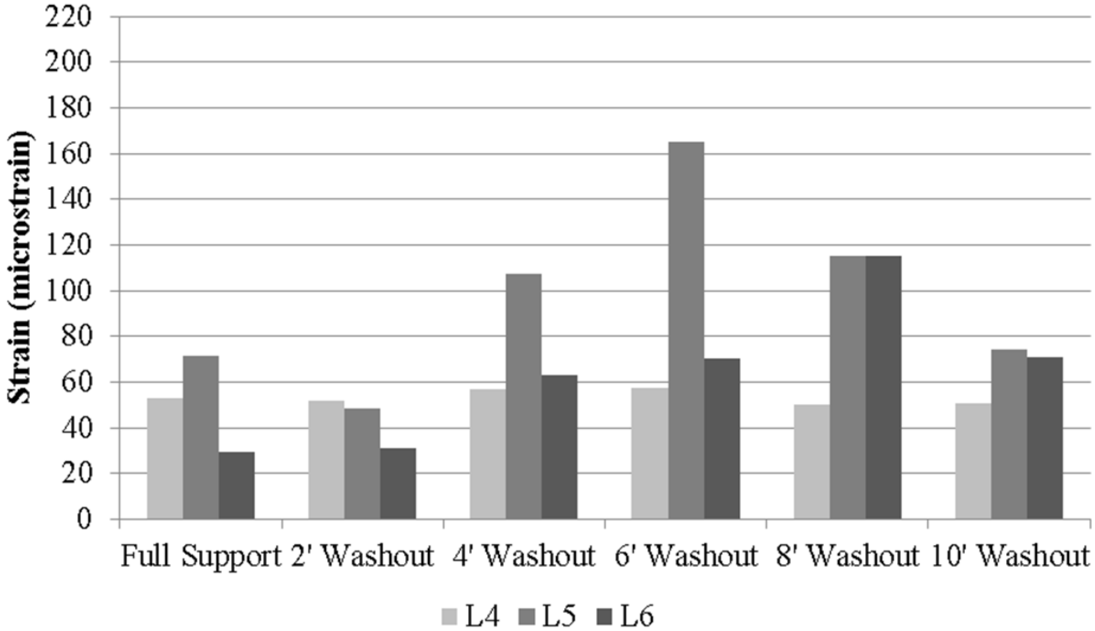


Figure 7-30 Strain in Gauges 4, 5, and 6

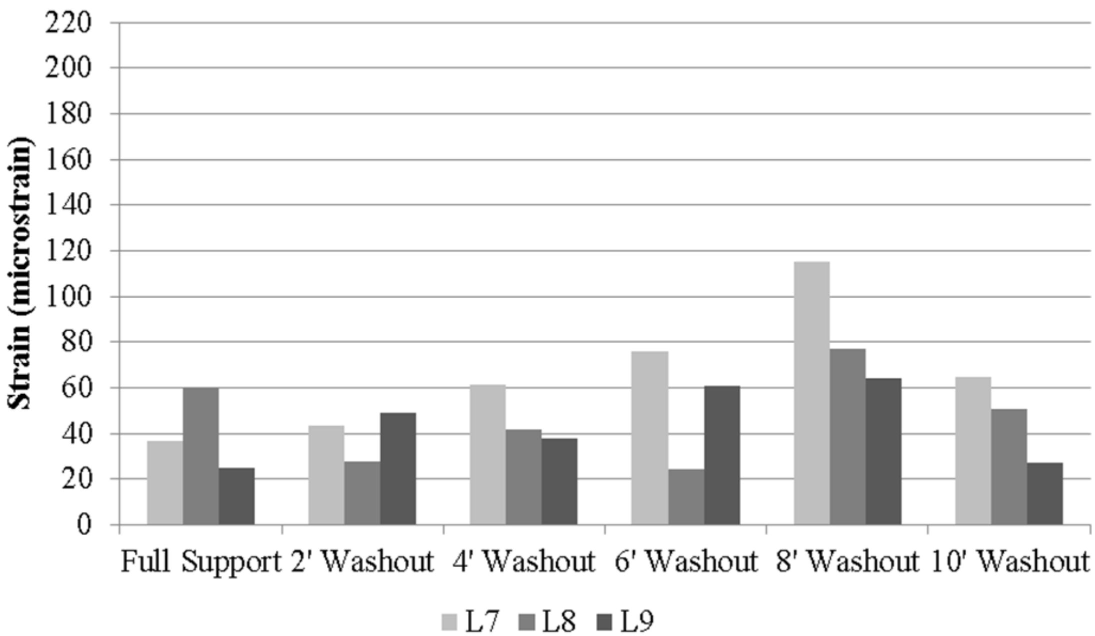


Figure 7-31 Strain in Gauges 7, 8, and 9

7.4.5 **Ultimate Capacity of PCPS Panel**

The ultimate capacity of the panel was determined by applying a uniform line load at the center of the panel. The load applied moment that produced failure in the panel was 57.5 kip-ft/ft width. Once 57.5 kip-ft/ft width was exceeded, the panel began deflecting and no longer carried additional load. The rotation, strains, and mid-span deflection of the panel is discussed in the subsequent sections. The nominal moment capacity of the panel was calculated according to the ACI 318-08 code. Figure 7-32 shows the profile used to calculate the moment capacity. The equations used for calculating the stress in the prestressing tendons and moment are displayed in Eq. 7-27 and Eq. 7-28 respectively. The calculated nominal moment capacity of the section was 58.6 kip-ft/ft width.

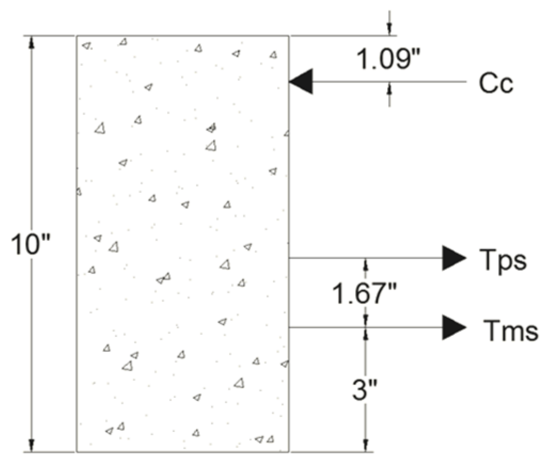


Figure 7-32 Profile of PCPS Panel for calculating the nominal moment capacity

$$f_{ps} = f_{pu}[1 - \frac{\gamma_p}{\beta_1}\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\omega - \omega')\right]] \quad \text{Eq. 7-27}$$

$$M = .85 * f'_c * a * b * \left((10 - g) - \frac{a}{2}\right) + A_{ms} * T_{ms} * (d - \frac{a}{2}) \quad \text{Eq. 7-28}$$

7.4.6 **Inclinometers**

The rotations produced from inclinometers 1, 2, 5, and 6 are shown in Figure 7-33. When the panel reaches its ultimate capacity, the rotation near the supports is roughly 1.64 degrees. All four inclinometer measurements were nearly the same as shown in Figure 7-33. This indicates that the location of the line load was placed symmetrically between the supports. As the loading increases beyond 47.5 kip-ft/ft, the effect of additional loading begins to cause more inclination in the panel. This is expected based on the observed failure mode.

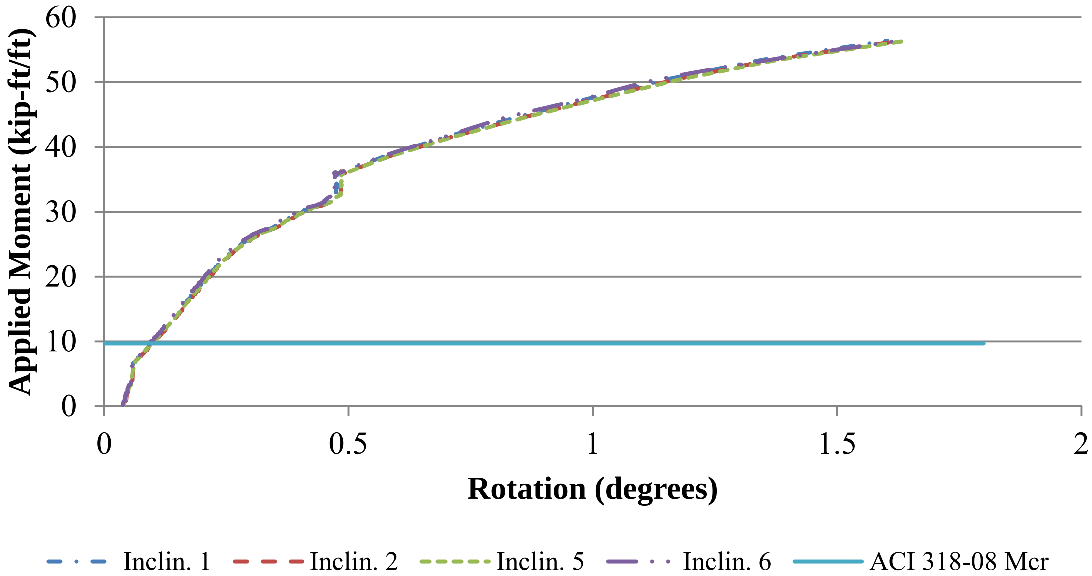


Figure 7-33 Rotation near the Supports of the Panel

7.4.7 **Strains**

The strains in the concrete were observed during the loading of the panel. During loading it was observed that the line load was not uniformly distributed on the surface of the panel. A localized compressive failure was the result when the panel was loaded to 23.75 kip-ft/ft width. When the load was released, gauges L7, L8, L9, and T3 were damaged. Therefore the strain at 9 ft. from the support was not measured. The panel was reloaded without issues.

The applied moment compared to the strain is shown in Figure 7-34 for distances of 3 ft. and 6 ft. from the support. The gauges located 3 ft. from the support experience small strain measurements compared to gauges 6 ft. from the support once the applied moment exceeds 35.6 kip-ft/ft width. As the location of the gauges approaches the center of the slab, the strain readings begin increasing. This is expected since this is where the maximum deflection occurs. Gauges L4, L5, and L6 increase linearly up to 400 microstrain. Once the cracking moment (M_{cr}) is exceeded and larger deflections begin to occur, the strain begins to plateau and increase rapidly with additional loading. Note that there were no changes in strain experienced by the transverse gauges.

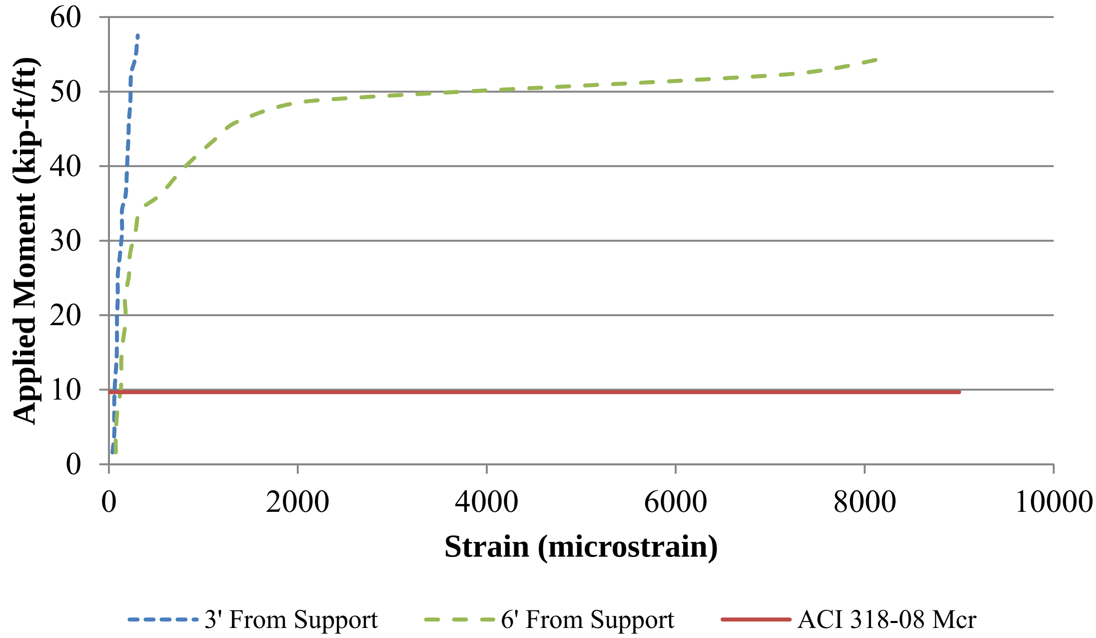


Figure 7-34 Strain compared to the applied moment in the PCPS panel

7.4.8 **Deflection**

String potentiometers were attached at the center edge of the panel during testing to record the vertical deflection relative to the applied moment. Figure 7-35 shows the relationship between the applied moment and midspan deflection. When the applied moment reaches roughly 40 kip-ft/ft width, the deflection is approximately 1 in. Beyond the 1 in deflection, an additional moment of 17 kip-ft/ft width is applied causing a deflection increase of 2.25 in. The stiffness of the panel helps minimize deflection until the cracking moment occurs. Once cracking occurs, the deflection increases at a larger rate with the same applied moment in the non-linear region. Figure 7-36 shows the curve for testing to failure. Additional test data for the laboratory testing of the PCPS panel is provided in Appendix VI.

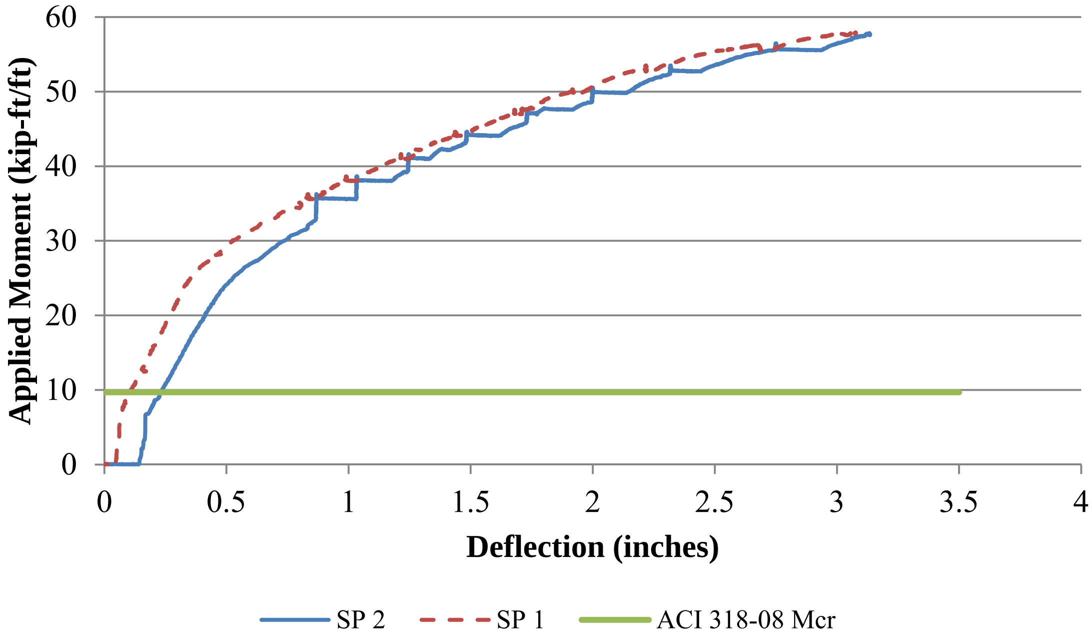


Figure 7-35 Applied Moment vs. Midspan Panel Deflection

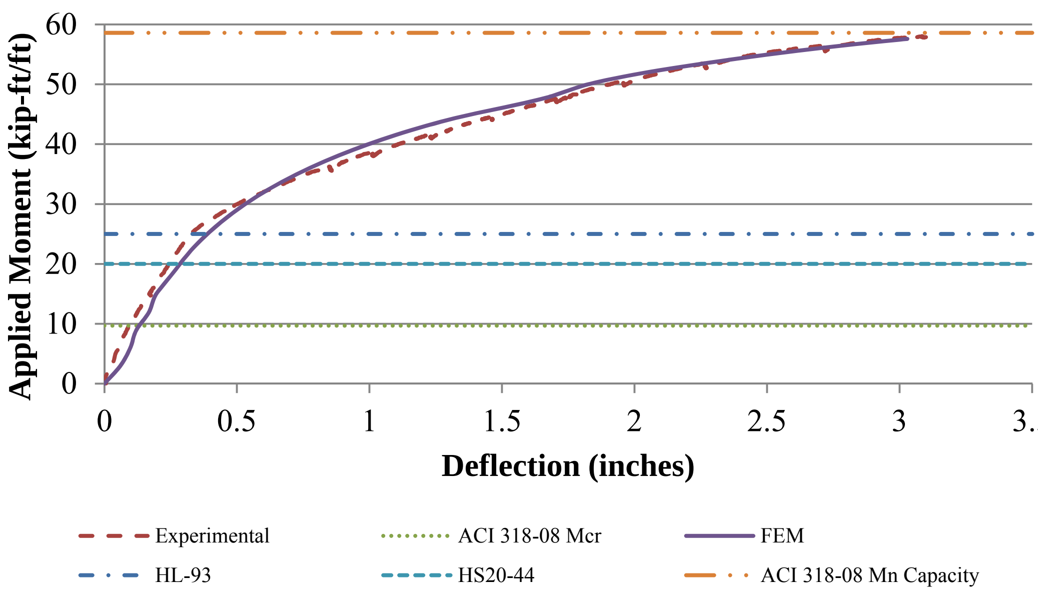


Figure 7-36 Experimental Deflection for Testing the PCPS Panel to Failure

CHAPTER 8 **CONCLUSIONS**

Some of the important findings from this field implementation, field monitoring and load testing, and laboratory testing of the precast-prestressed panels are presented below. Three new cost effective BAS designs were implemented by MoDOT. This included two types of cast-in-place designs on three bridges, namely 20 feet CIP BAS with a sleeper slab and two 25 feet CIP BAS with no sleeper slab and two bridges with precast-prestressed BAS consisting of panels that are 6 feet and 8 feet wide respectively. Comparison of predicted costs versus actual costs of construction has also been performed for this study.

8.1 Cast-in-Place Slabs

1. MoDOT used a mix design with target strength of 4000 psi (27.58 MPa) for the cast-in-place designs. The modulus of rupture estimated by the ACI 318-08 was conservative which indicates the designs using the same mix design proportions and constituent materials will have a higher cracking moment. The empirically predicted modulus of elasticity based on the ACI 318-08 code was larger than the experimental values.
2. The experimental field tests performed on the BASs resulted in small rotations using unloaded and loaded HS20 dump trucks. Bridge B0563 was opened on a major highway system so the only load test included unloaded dump trucks. Initial deflection readings indicate small longitudinal deflections using high precision surveying equipment. Initial differential deflection is caused from surface texturing of the bridge deck. The baseline differences are included and can be used for future reference points.
3. The subsequent field deflection rotations were small and do not suggest conditions hazardous to motorists.
4. Visual inspections of the CIP and PCPS slabs on MO38 done periodically have indicated no damage or distress in the PCPS slabs after eighteen months. However, the CIP BAS on MO38 on bridge A7890 (with no sleeper slab) has shown some differential deflection movement between the slab and the asphalt pavement with the presence of a longitudinal crack. However, the BAS is still flush with the bridge at the bridge end of the slab.

8.2 Precast-prestressed Slabs

1. The PCPS panels installed on Bridge A7767 had additional testing specimens made at Coreslab Structures, Inc. to verify design strengths and concrete properties. The 28 day compressive strength of 6000 psi (41.37 MPa) was satisfied.
2. Creep and shrinkage specimens were monitored for the new PCPS BAS. The maximum shrinkage after 275 days was approximately 625 microstrains. The ACI 209R-92 model is very conservative and overestimates the shrinkage values greatly after 25 days for the mix design utilized. The ACI 209R-92 creep model is conservative at later ages, but was found accurate in predicting the behavior even up to 170 days as performed in this study.

3. The total prestress losses due to creep, shrinkage, relaxation of the strand, and elastic shortening were determined for the days that load testing was performed. The first load test of the PCPS BAS indicated prestress losses of 14.3% of the jacking force and 16.6% for the second load test.
4. When loading the PCPS BAS with unloaded MoDOT HS20 dump trucks, VWSGs in the PCPS panels monitored changes in strain. During the load test small microstrains were detected. This indicates that the panels are fully supported on the base material.
5. During a yearly cycle, the change in strain caused from temperature fluctuations is approximately 50 microstrain. This provides a baseline for future researchers to monitor the slab and determine if additional forces are induced.
6. Moisture gauges placed in the soil beneath the PCPS BAS indicate steady readings of volumetric water content. This indicates that any moisture permeating through the joints is being drained by the base between the soil and approach slab.
7. For alternative solutions where replacement slabs are needed, a precast-prestressed slab with transverse ties is proposed. Detailed cost analyses have been performed for the proposed solution. From the cost observations it is evident that these slabs could be cost effective in new construction as well. Hence, designs for both a 20 foot span (new construction) and 25 foot span (old/replacement construction) have been proposed. Sleeper slabs are recommended for both designs. It has been shown by a cost analysis that the proposed precast solution compares equally with the proposed cast-in-place solution and can be adopted for new construction as well resulting in considerable time and user cost savings.
8. The washout testing performed on the PCPS panel yielded a maximum rotation of 0.023 degrees and a deflection of 0.013 inches. The maximum strain was 205 microstrain which was located in the center of the slab close to the support. All maximum conditions occurred when the washout was 6 feet.
9. Testing panel to failure yielded a failure moment of 57.5 kip-ft/ft width. This is a 44% increase in moment capacity (40 kip-ft/ft width) compared to the design moment. At failure the panel rotated 1.64 degrees at the supports and deflected 3.1 in. at the midspan.

8.3 Cost Analysis

1. The total cost of each approach was monitored throughout the construction process. Results show that all the BASs implemented in this project had an initial cost less than the estimated cost of the current standard MoDOT BAS design.
2. The actual costs of 25 ft. CIP BAS no sleeper slab designs used on bridges A7890 and A7925 were $43,292 and $33,822. The savings compared to the estimated cost of the standard MoDOT BAS design after adjustments for differences in sizes were 12.5 and 28.3 percent, respectively.
3. The cost of 20 ft. CIP BAS with sleeper slab designs used on bridge B0563 was $41,760. Cost savings compared to the estimated cost of the standard MoDOT BAS with sleeper slab design was 22 percent.
4. Assuming the new CIP designs perform as predicted by the numerical analysis and provide a service life equal or better than the service life of the standard BAS design, MoDOT can realize significant cost savings by switching to the new proposed BAS designs.

5. The cost savings due to the use of PCPS BAS design on bridges A7934 and A7767 were 3.3 and 15.3 percent, respectively. Due to difficulties in determining the cost of extra activities necessary for the PCPS slabs such as anchoring to abutments, grouting, and tying of panels there is a large difference between the recorded cost savings. Even if the initial cost savings are negligible, the PCPS BAS design is expected to provide lower life cycle costs due to improved service life of BASs. Laboratory testing of PCPS panels and current field observations show that the PCPS BASs will have a good performance in washout conditions. Due to the asphalt overlay used in this design, simple settlements without cracking of the panels can quickly be repaired using an asphalt wedge.
6. PCPS BAS solution could be a very advantageous alternative for replacement of existing BASs. Both implementation projects showed that with a stricter dimension tolerance control, straight tie rods, and better base preparation, PCPS slabs can be placed in a 4 hour period using a small capacity crane and a small crew. Considering demolition of existing slab and 2 inch asphalt overlay activities, an existing BAS can be replaced in a day and opened to traffic with the PCPS BAS system.

CHAPTER 9 **RECOMMENDATIONS**

Various recommendations have been presented at different sections of the report. The design recommendations presented are in the following sections.

a) Based on the observations during construction of the PCPS panels on two bridge sites (MO38 A7767 and US136 – B0563) the following recommendations are made.
   a. The panels on MO38 were 6 feet wide while the panels on US136 were 8 feet wide. Based on costs to fabricate, 8 feet panels were found to be less expensive and are recommended wherever possible.
   b. The panels on MO38 were tied in the transverse direction with a single transverse tie connecting all five panels. The panels on US136 had smaller tie rods which connected only the two adjacent panels. Based on observations during construction it is recommended to use a single tie rod wherever convenient. This recommendation is based on the observed ease of construction for a single tie.
   c. In order to get the most contact of the PCPS slabs with the base rock layer it is recommended that the drawing notes place special emphasis for the contractor to ensure that the base rock is level to the extent possible. It was observed that if the base layer is not level, the slab sits unevenly.
   d. Cast-in-place abutment and wing walls should have strict tolerance controls on site in order to fit the PCPS slabs.

b) The initial costs of all the implemented slabs are lower than the current MoDOT BAS design. MoDOT should continue efforts to monitor the performance of the BASs implemented in this study to determine the life cycle costs of the slabs based on future maintenance costs.

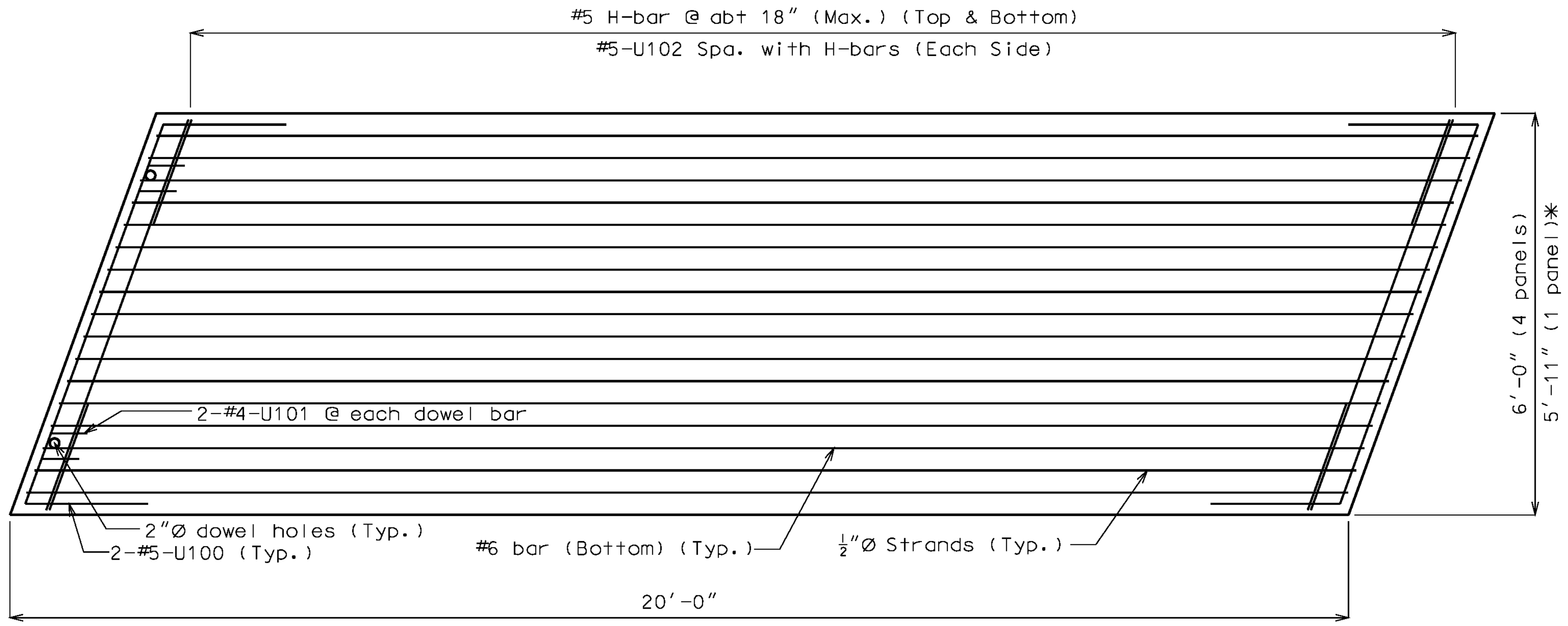
CHAPTER 10 **IMPLEMENTATION PLAN**

The proposed project implemented several solutions developed and proposed in a previous project for new and replacement approach slab applications. The following implementation plan is proposed based on field and laboratory testing of several bridge approach slab solutions.

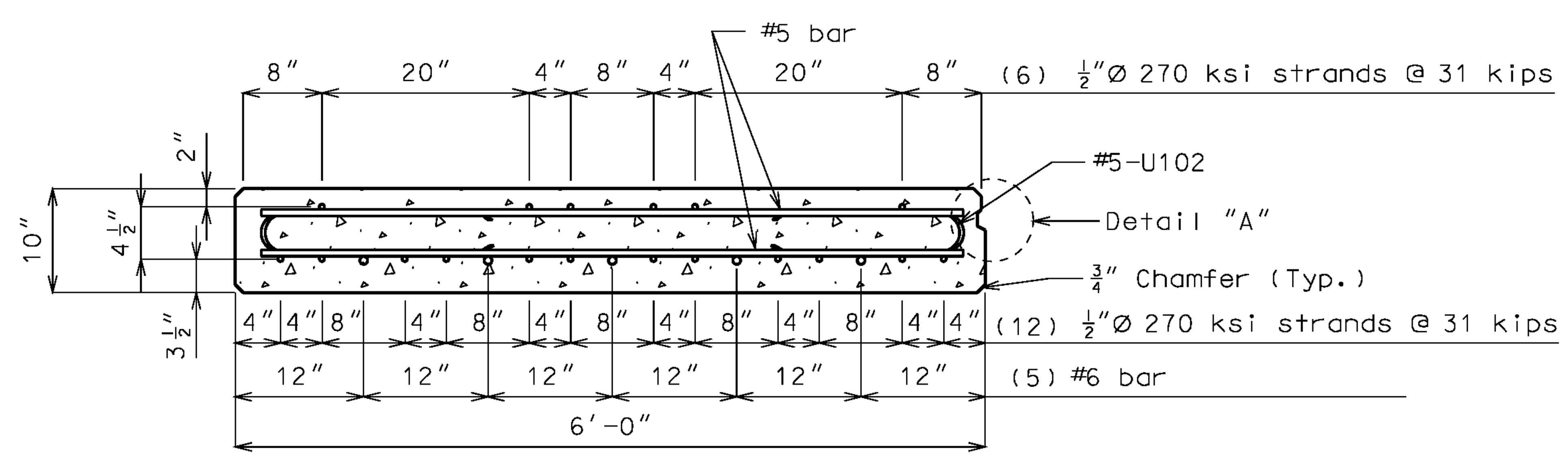
1) For new CIP slabs there are two design recommendations. MoDOT could continue to implement these designs as they have been found to be cost effective. They are,
   a) A 20 foot span and 12 inch thick cast-in-place slab with sleeper slab.
   b) A 25 foot span and 12 inch thick cast-in-place slab with no sleeper slab.
2) Monitoring of long-term differential deflection of the BASs on MO38 should be continued. Strain readings in the PCPS BAS should be taken monthly to determine if additional stresses are applied. The moisture gages should be measured during heavy precipitation to ensure the aggregate base continues to function as a drainage system.
3) Additional live-load testing could be performed on the BASs on MO38. Heavier truck loads could be applied and devices such as string potentiometers could be applied to measure deflection during testing.
4) Implement the PCPS BAS system on a major highway system. Install internal instrumentation such as VWSGs to evaluate structural performance with heavy traffic flow.
5) Implement the PCPS panel system without the use of the sleeper slab on minor roadways. Prior to design, a FE analysis should be performed to determine the effects of elastic support loss on the slab.
6) Continue to monitor the maintenance of each BAS to determine the life cycle costs associated with each design.

APPENDIX I A7767 DESIGN DRAWINGS (PCPS ON MO38)

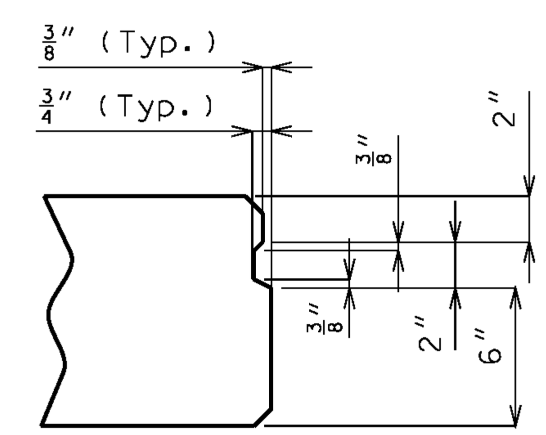
* Final dimension may be adjusted from that shown by the contractor based on field dimensions.



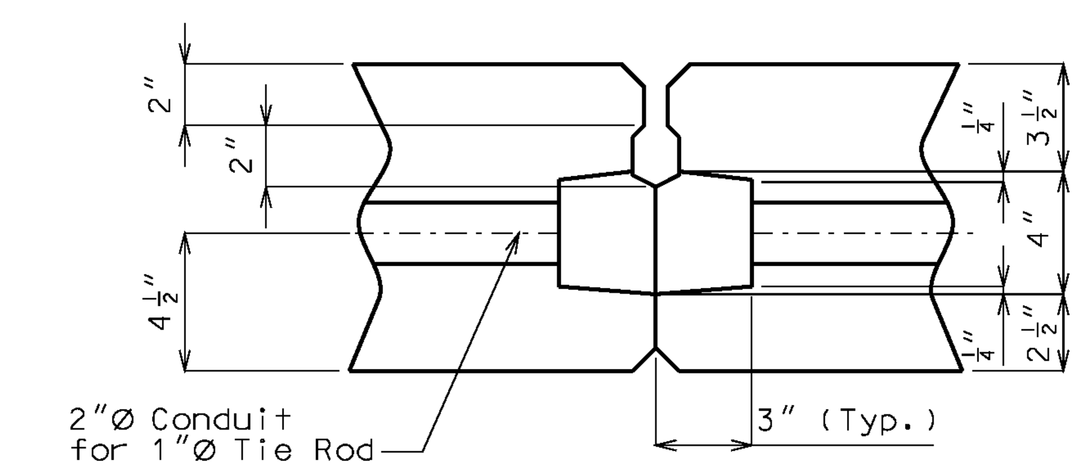
PLAN OF BRIDGE APPROACH SLAB PANEL REINFORCEMENT



SECTION THRU EXTERIOR PANEL

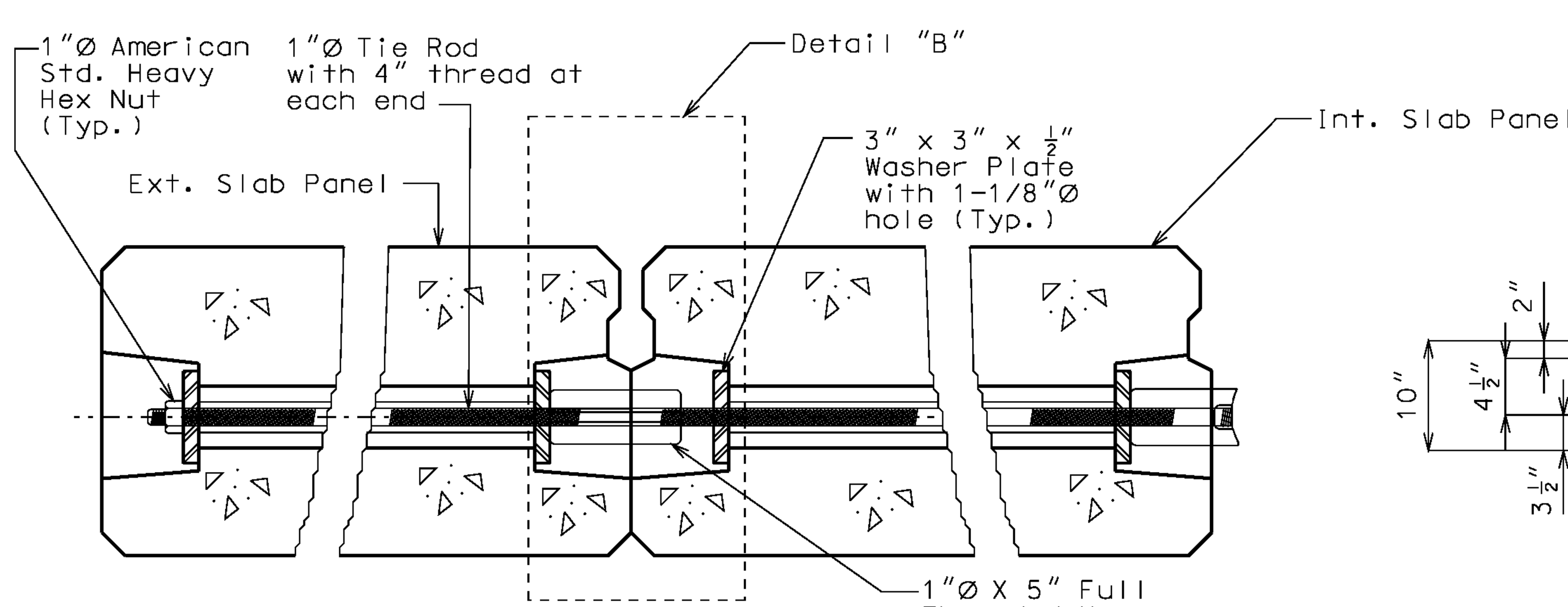


DETAIL "A"

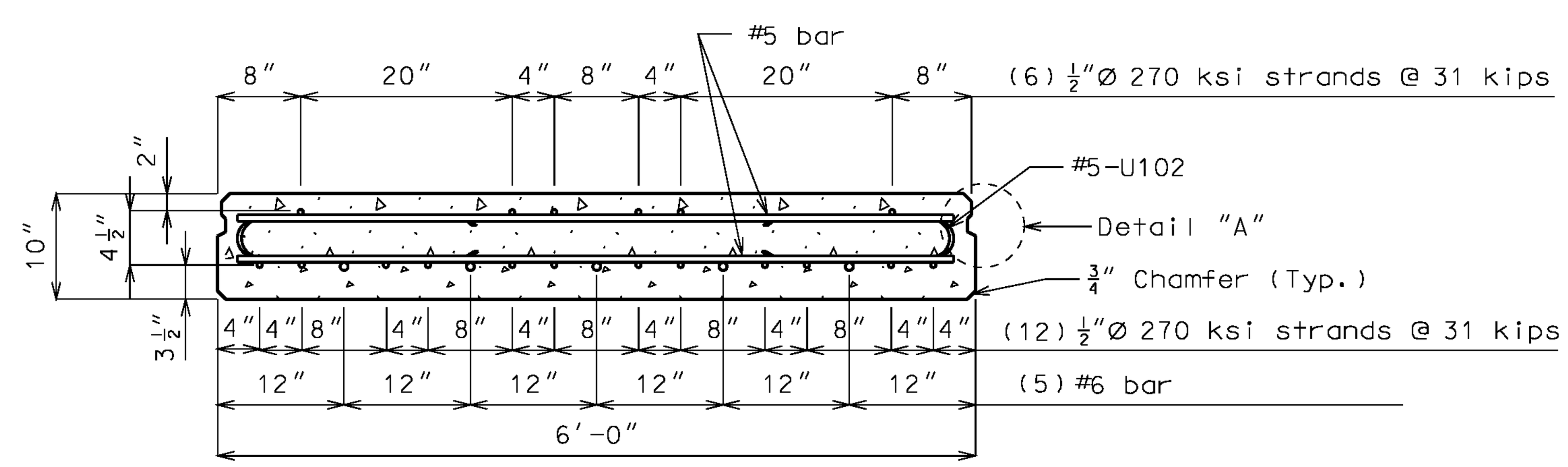


DETAIL "B"

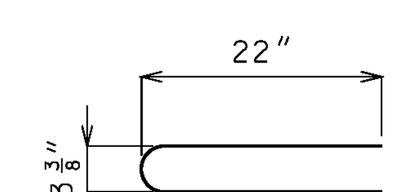
Note: Place 2"Ø conduit on top of bottom strands.



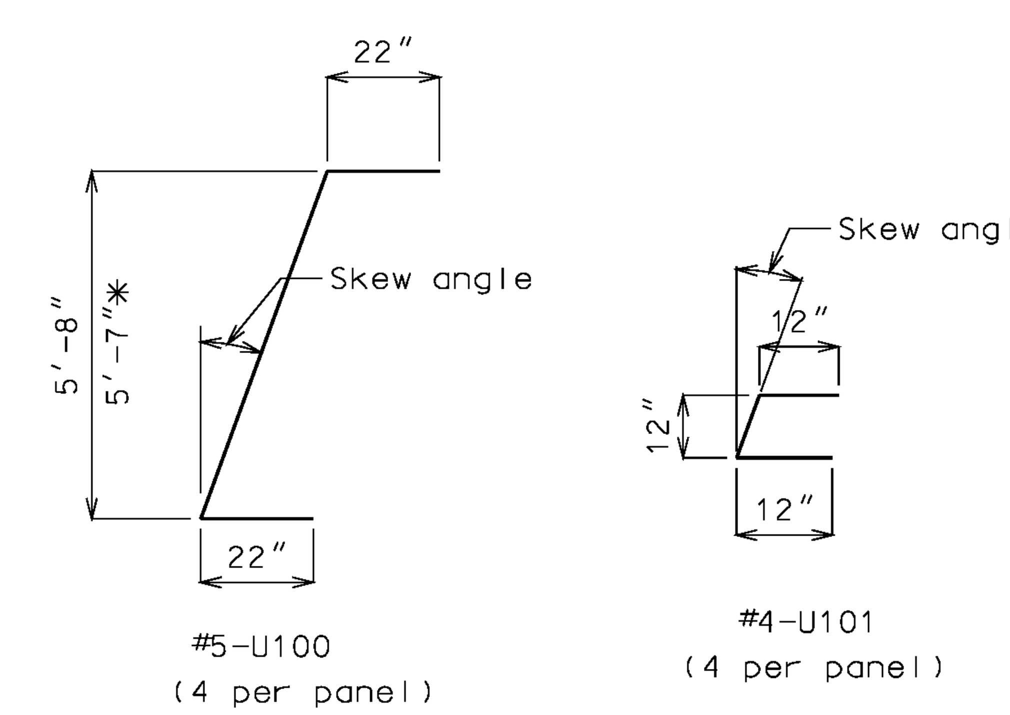
PART SECTION THRU 1"Ø TIE ROD



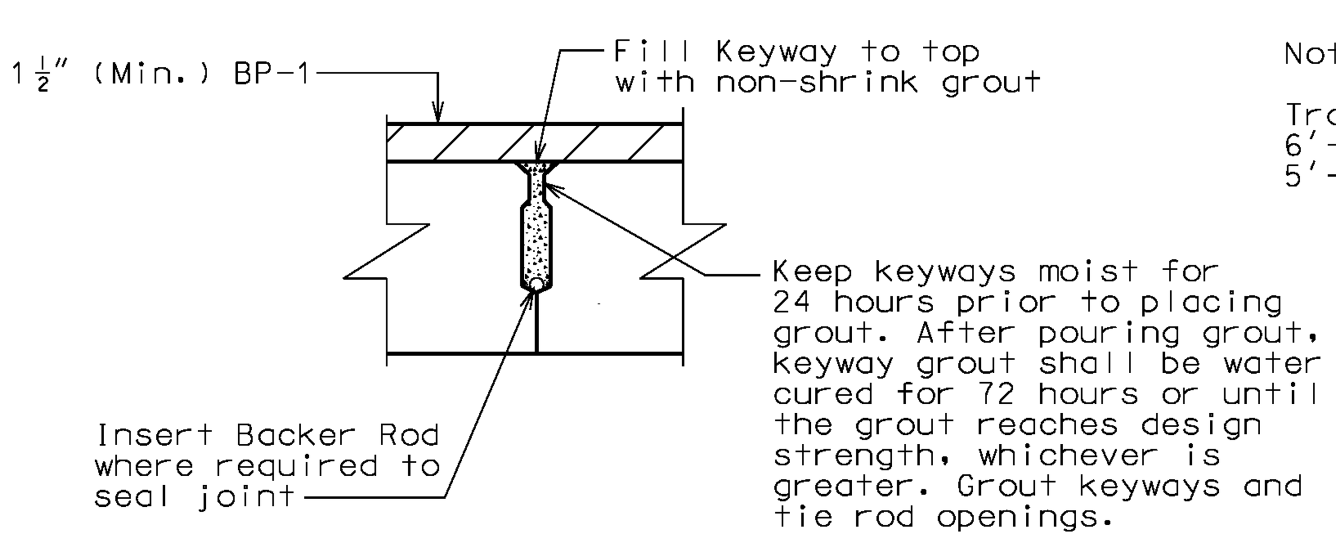
SECTION THRU INTERIOR PANEL



#5-U102



DETAILS OF STIRRUP BARS



Note:

Transverse spacing of longitudinal strands and bars for the 6'-0" panels shown above shall be equally reduced for the 5'-11" panel*.

GENERAL NOTES:

Concrete for bridge approach slab panels shall be Class A-1 with f'c = 6,000 psi, f'ci = 3,500 psi.

The top surface of all panels shall receive a scored finish with a depth of scoring of ⅛" perpendicular to the prestressing strands in the panels.

Prestressing tendons shall be high-tensile strength uncoated seven-wire low-relaxation strands for prestressed concrete in accordance with AASHTO M 203 Grade 270, with nominal diameter of strand = ½" and nominal area = 0.153 sq. in.and minimum ultimate strength = 46.31 kips (270 ksi). Larger strands may be used with the same spacing and initial tension.

Initial prestressing force = 31 kips/strand.

The method and sequence of releasing the strands shall be shown on the shop drawings.

Suitable anchorage devices for lifting panels may be cast in panels, provided the devices are shown on the shop drawings and approved by the engineer. The fabricator shall design and locate the lifting loops for the Precast Concrete Approach Slab Panels.

All dimensions are out to out.

Minimum clearance to reinforcing steel shall be 1½", unless otherwise shown.

All holes in slab panels shall be cast. Drilling is not allowed.

Minimum reinforcement steel length shall be 2'-0".

Prestressing strands at ends of slab panels shall be trimmed to within ⅛" of concrete. Ends of slab panels shall be given 2 coats of an approved asphalt paint.

Work this sheet with Sheet No. 22.

Note:
Nominal lengths are based on out to out dimensions shown in bending diagram and are listed for fabricators use (nearest inch).

DETAILS OF PRECAST CONCRETE APPROACH SLAB PANELS

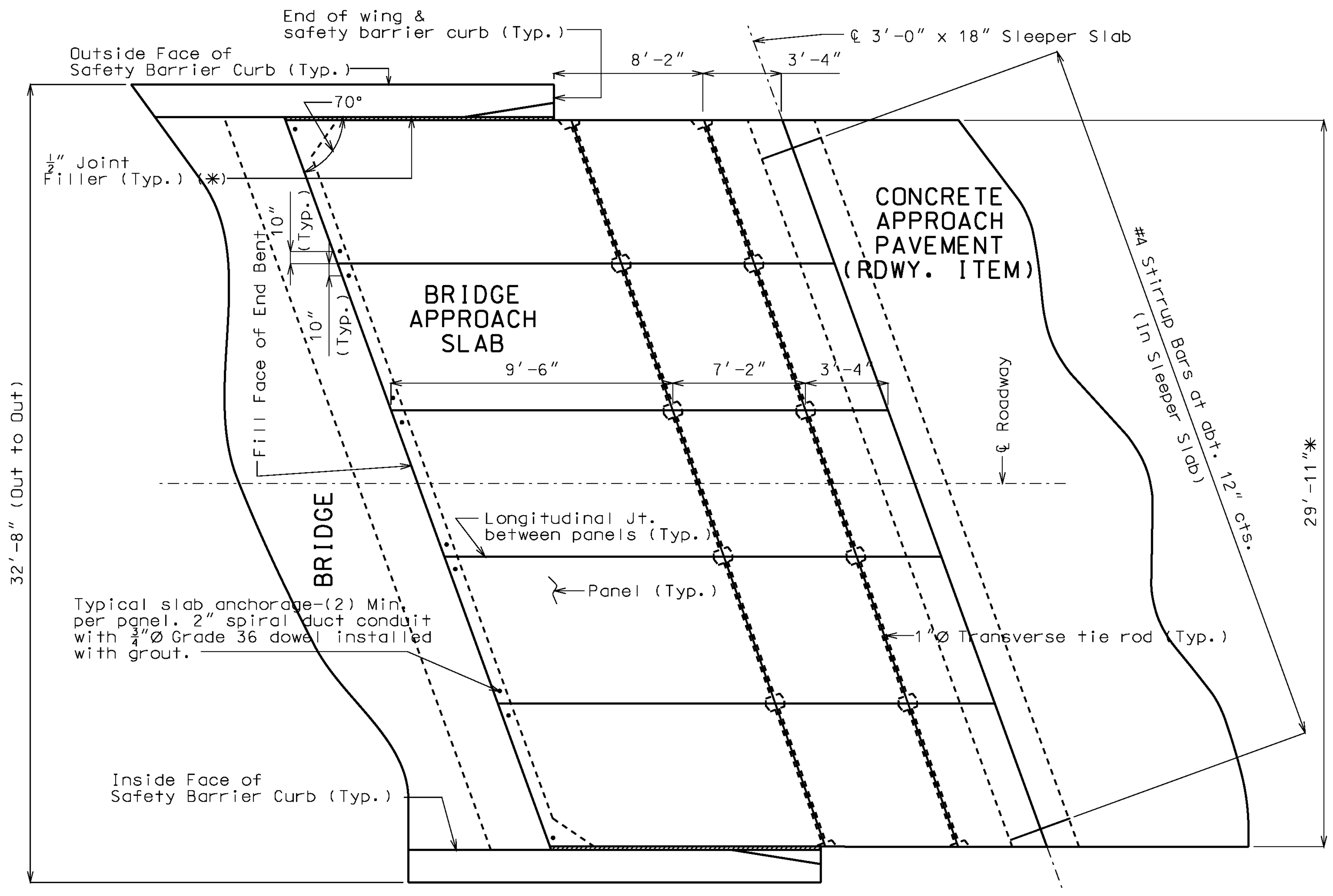
Detailed Nov. 2010
Checked Nov. 2010

Note: This drawing is not to scale. Follow dimensions.

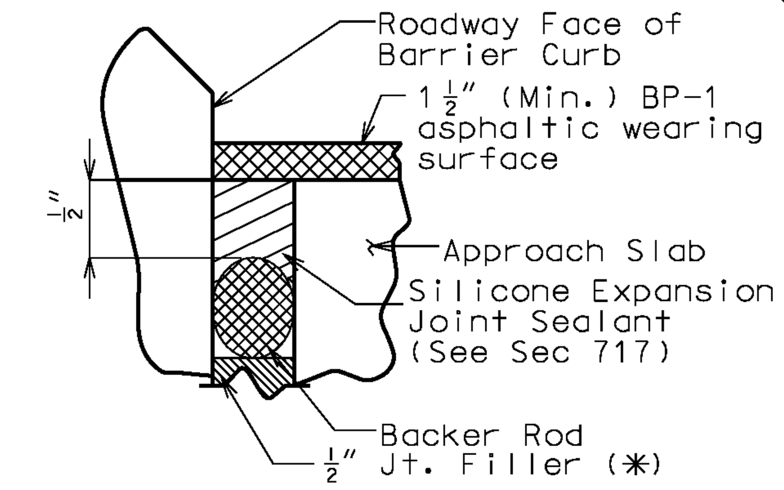
DATE PREPARED	12/15/2010
ROUTE	38
STATE	MO
DISTRICT	BR
SHEET NO.	23
COUNTY	WEBSTER
JOB NO.	J8S2174
CONTRACT ID.	
PROJECT NO.	
BRIDGE NO.	A7767

DESCRIPTION | DATE

MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION
105 WEST CAPITOL
JEFFERSON CITY, MO 65102
1-888-ASK-MODOT (1-888-275-6636)



PART PLAN OF BRIDGE APPROACH SLAB



SECTION THRU JOINT BETWEEN WING AND APPROACH SLAB

GENERAL NOTES:

All joint filler shall be in accordance with Sec 1057 for preformed fiber expansion joint filler, except as noted.

Minimum clearance to reinforcing steel shall be 1½", unless otherwise shown.

✱ Final dimension may be adjusted from that shown by the contractor based on field dimemsions.

Payment for furnishing all materials, labor and excavation necessary to construct the approach slab, including the sleeper slab, underdrain, BP-1 asphaltic wearing surface, joint filler and all other appurtenances and incidental work as shown on this sheet, complete in place, will be considered completely covered by the contract unit price for Precast Concrete Approach Slab (Bridge) per square yard.

For Concrete Approach Pavement details, see roadway plans.

Drain pipe may be either 6" diameter corrugated metallic-coated pipe underdrain, 4" diameter corrugated polyvinyl chloride (PVC) drain pipe, or 4" diameter corrugated polyethylene (PE) drain pipe.

Reinforcement shall conform to the requirements of Section 1036 of the Missouri Standard Specifications.

Keyway surface shall be cleaned to remove form oil or other bond breaking material prior to shipment of the precast concrete approach slab panel. Cleaning shall be done by sandblasting the keyway areas between top of the panel and the bottom edge of the key.

All Tie Rods, Plates, Sleeves, and nuts shall be galvanized in accordance with ASTM A153.

Tighten all tie rods (per precast concrete approach slab) to about one-half of the specified tension before proceeding with the final tensioning.

Tie Rod nuts shall be tightened to provide a tension of one-half that specified for A325 bolts in Section 712.7.3 of the Missouri Standard Specifications.

Use non-shrink grout for filling the keyways.

Keep approach slab panels roadway face up at all times. Support them within 12 inches of the ends only.

Tie Rod Plates shall be ASTM A709 Grade 36.

Sleeves, Nuts and 1" Ø Tie Rods shall be A307.

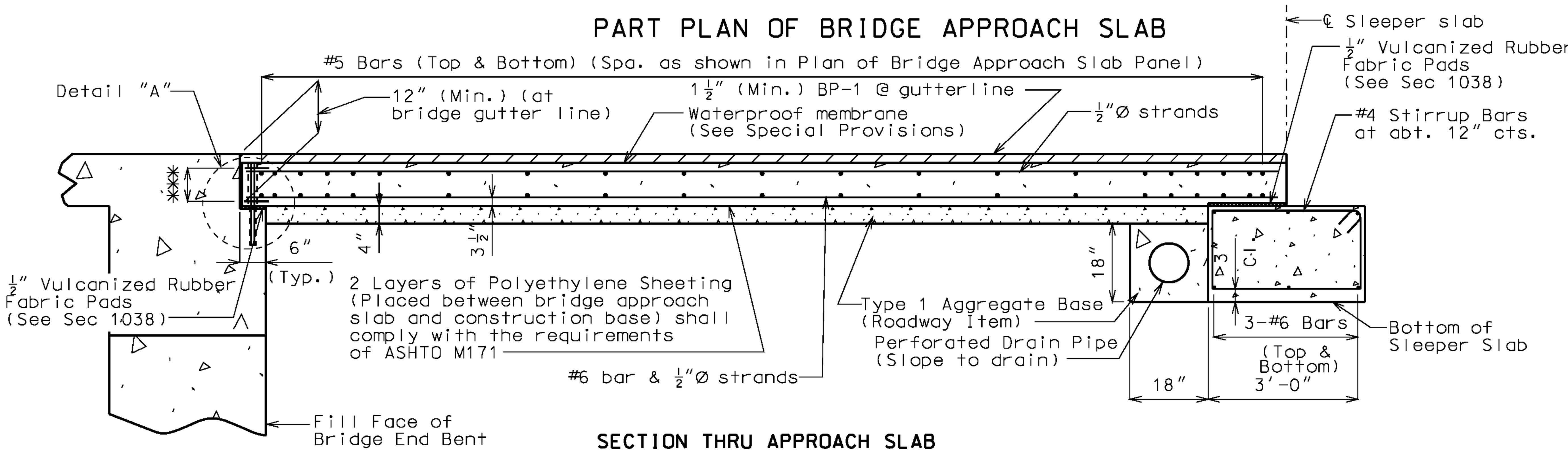
Dowel bars to be grouted in place and allowed to cure a minimum of 72 hours.

Minimum clearance to reinforcing steel shall be 1 1/2", unless otherwise shown.

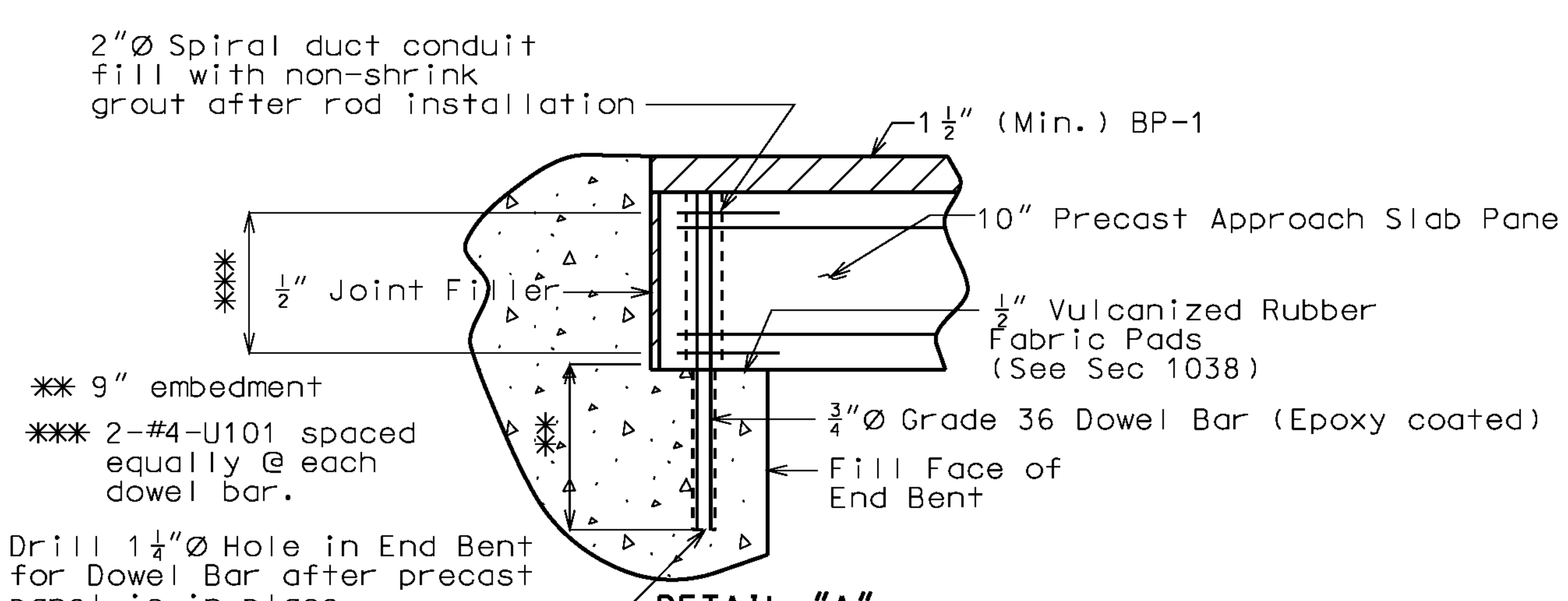
All concrete for the sleeper slab shall be in accordance with Sec 503 (f'c = 4,000 psi).

Hooks and bends shall be in accordance with the CRSI Manual of Standard Practice for Detailing Reinforced Concrete Structures, Stirrup and Tie Dimensions.

Work this sheet with Sheet No. 23.

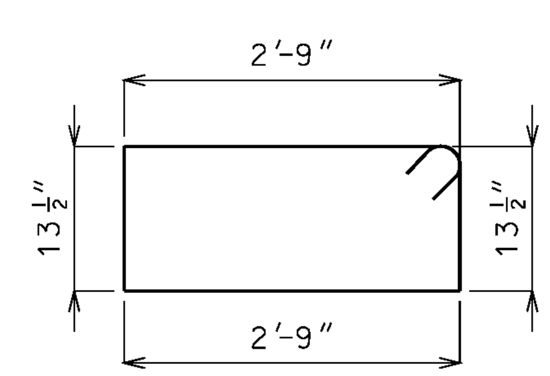


SECTION THRU APPROACH SLAB
Note: Tie Rods not show for clarity.

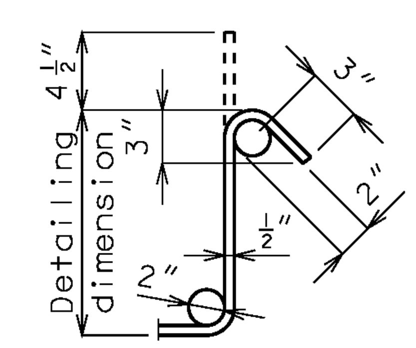


✱✱ 9" embedment
✱✱✱ 2-#4-U101 spaced equally @ each dowel bar.
Drill 1½"Ø Hole in End Bent for Dowel Bar after precast panel is in place

DETAIL "A"

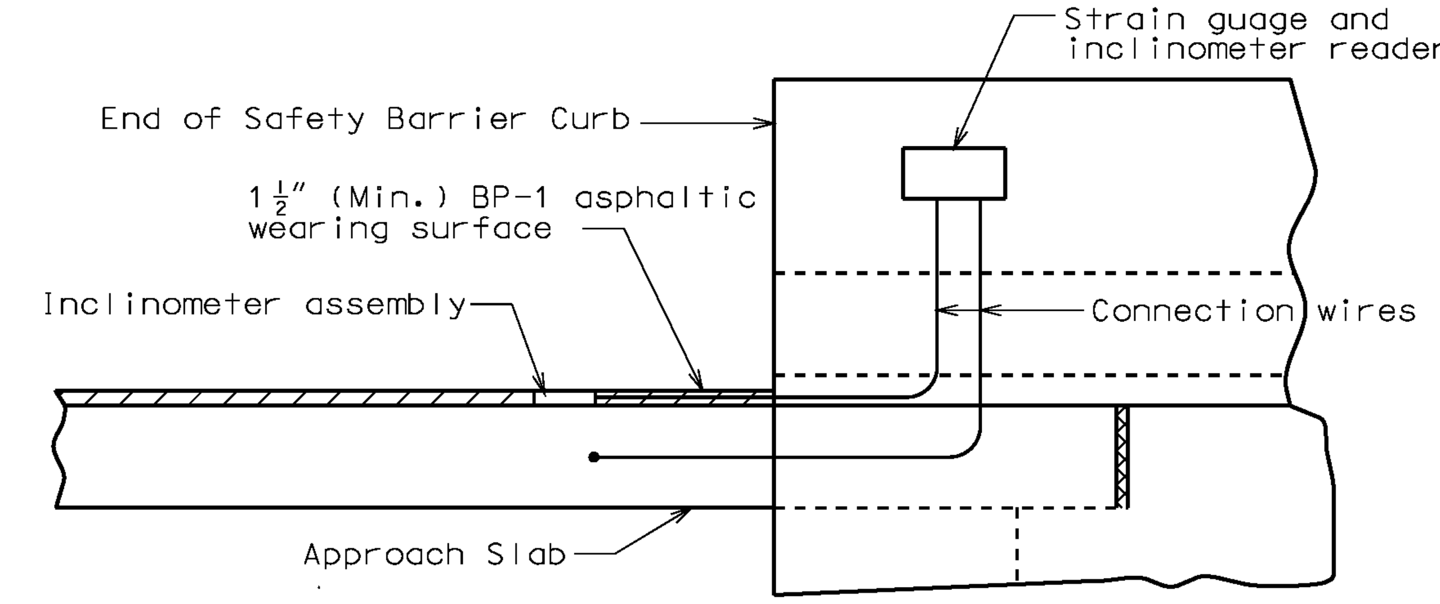


#4 STIRRUP BAR (ACTUAL LENGTH = 8'-3")



TYPICAL 135° STIRRUP HOOK DIMENSIONS BENDING DIAGRAM

Note:
Nominal lengths are based on out to out dimensions shown in bending diagram and are listed for fabricators use (nearest inch).



DETAIL OF FINISHED STRAIN GUAGE AND INCLINOMETER PLACEMENT
Note: Final installation may vary from that shown. (Equipment and installation by Research personnel, see Special Provisions)

DETAILS OF PRECAST CONCRETE APPROACH SLAB

Detailed Nov. 2010
Checked Nov. 2010

Note: This drawing is not to scale. Follow dimensions.

DATE PREPARED	12/15/2010
ROUTE	38
STATE	MO
DISTRICT	BR
SHEET NO.	22
COUNTY	WEBSTER
JOB NO.	J8S2174
CONTRACT ID.	
PROJECT NO.	
BRIDGE NO.	A7767

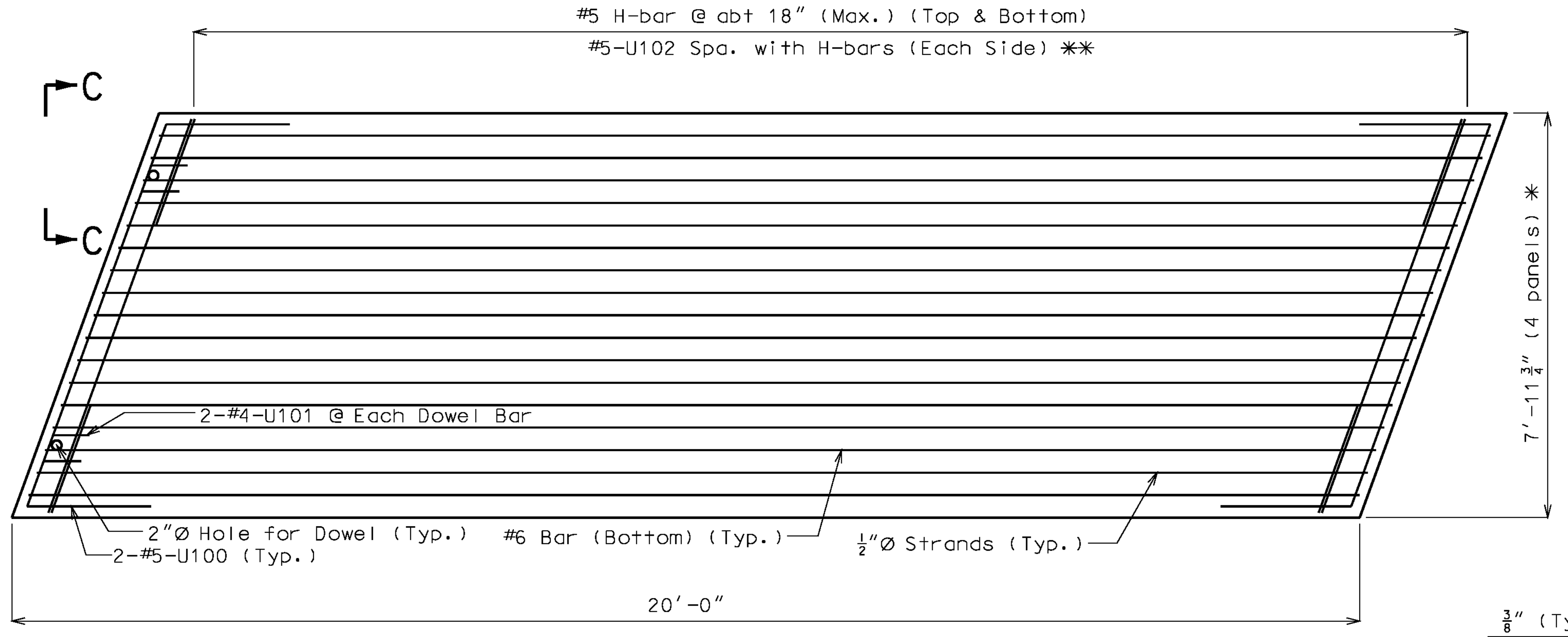
DESCRIPTION

DATE

MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION

105 WEST CAPITOL
JEFFERSON CITY, MO 65102
1-888-ASK-MODOT (1-888-275-6636)

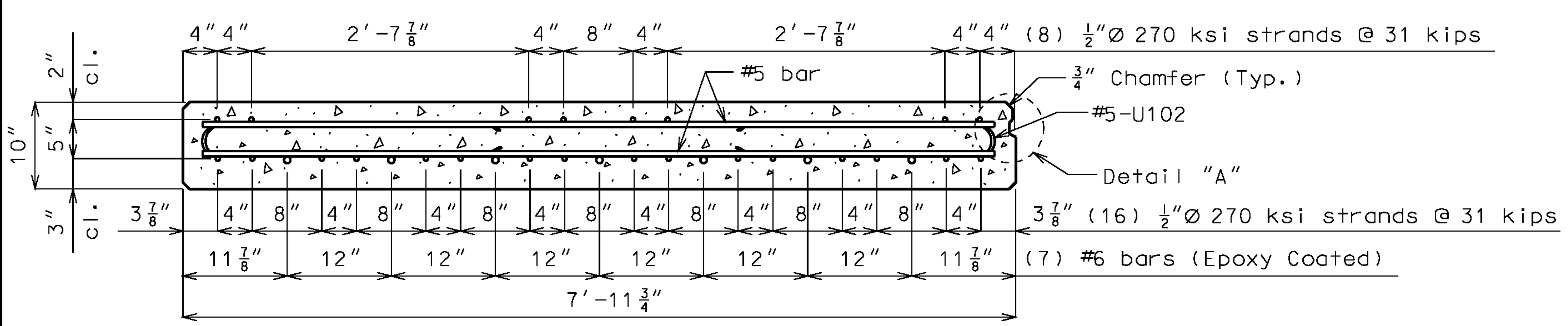
APPENDIX II A7934 DESIGN DRAWINGS



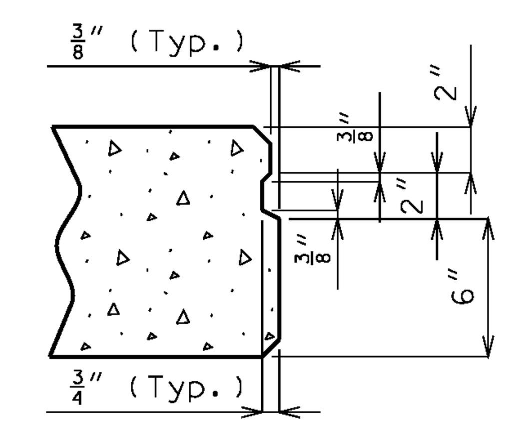
PLAN OF BRIDGE APPROACH SLAB PANEL REINFORCEMENT

✱ Final dimension may be adjusted from that shown by the contractor based on field dimensions.

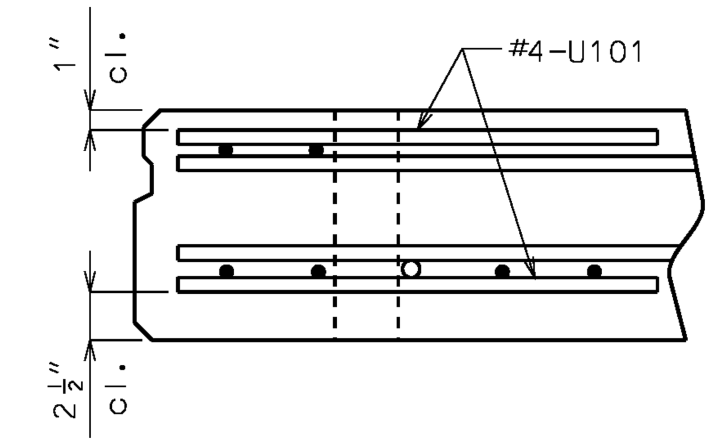
✱✱ Shift minimum extent to clear tie rod pockets.



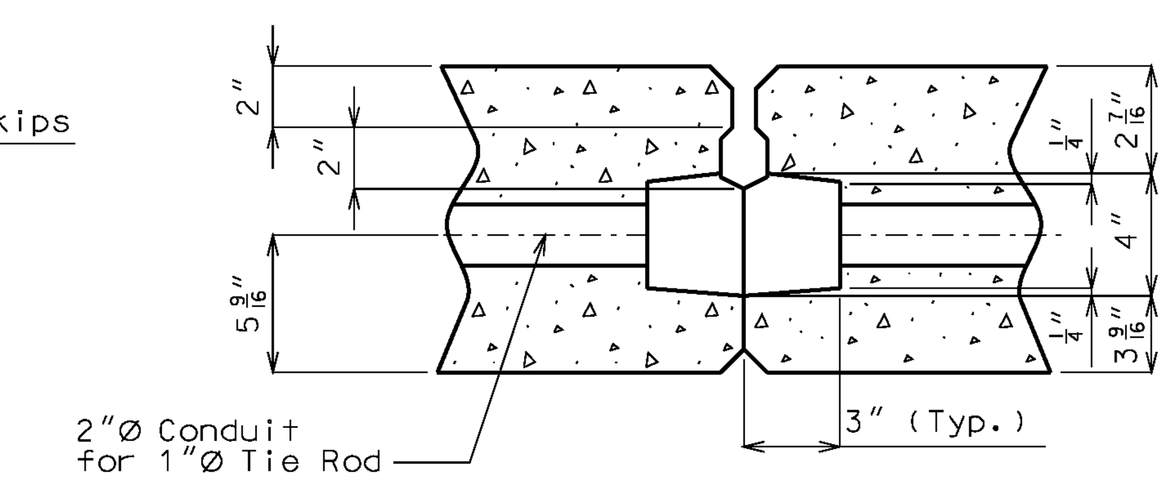
SECTION THRU EXTERIOR PANEL



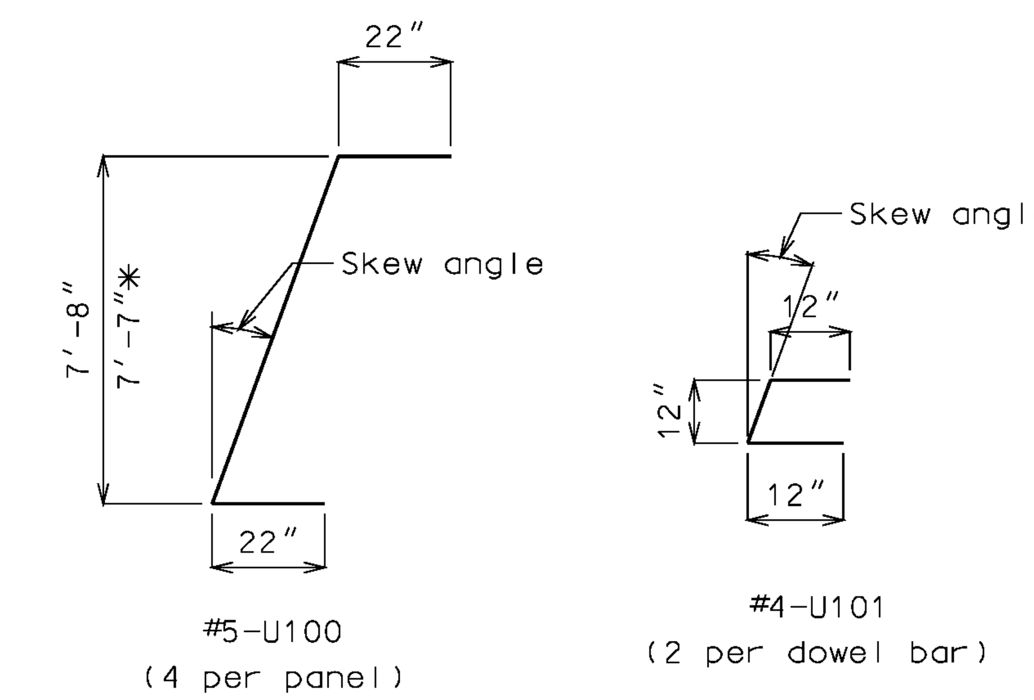
DETAIL "A"



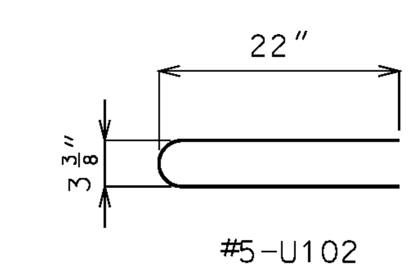
PART SECTION C-C



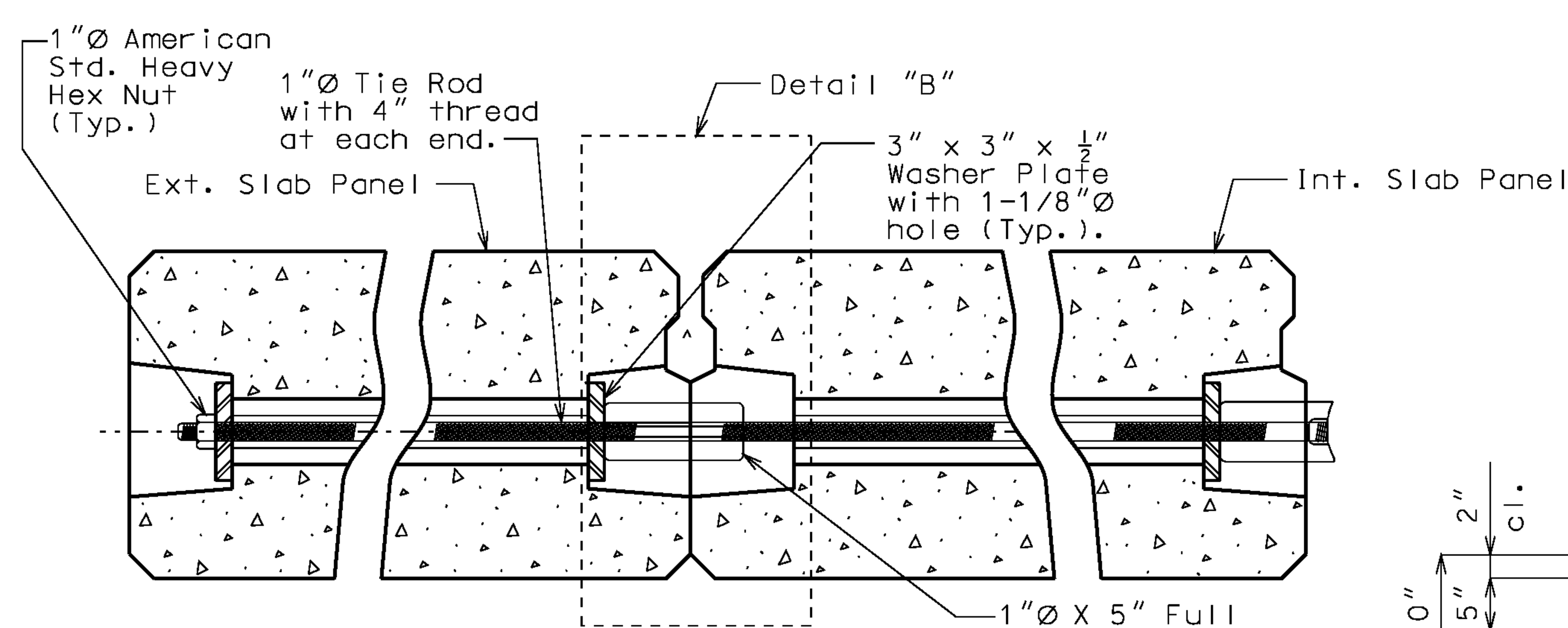
DETAIL "B"



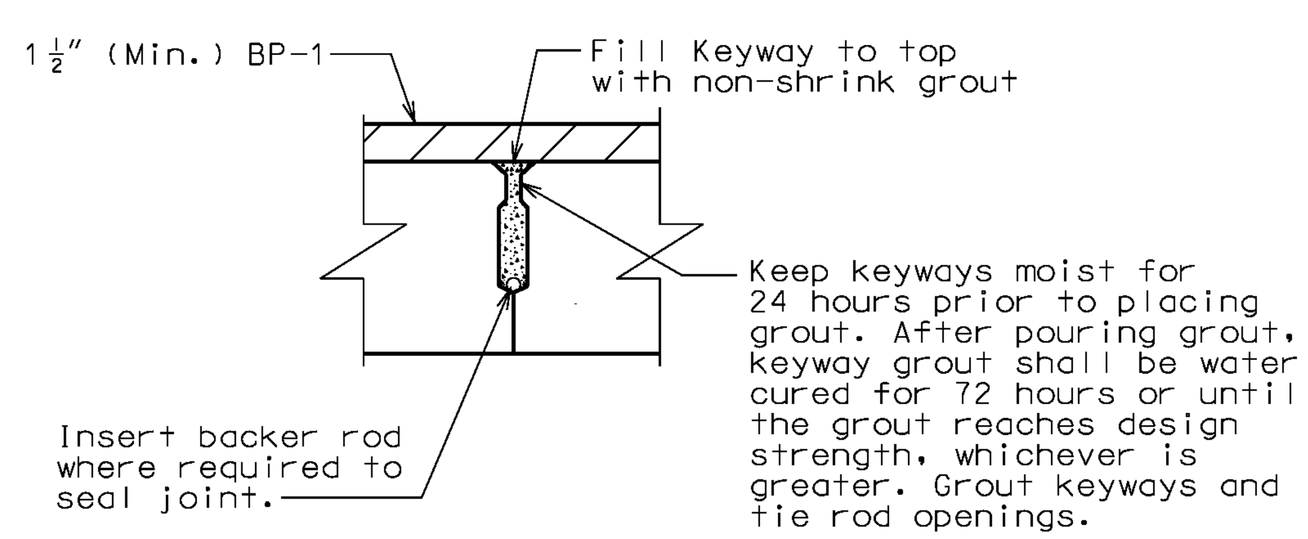
DETAILS OF STIRRUP BARS

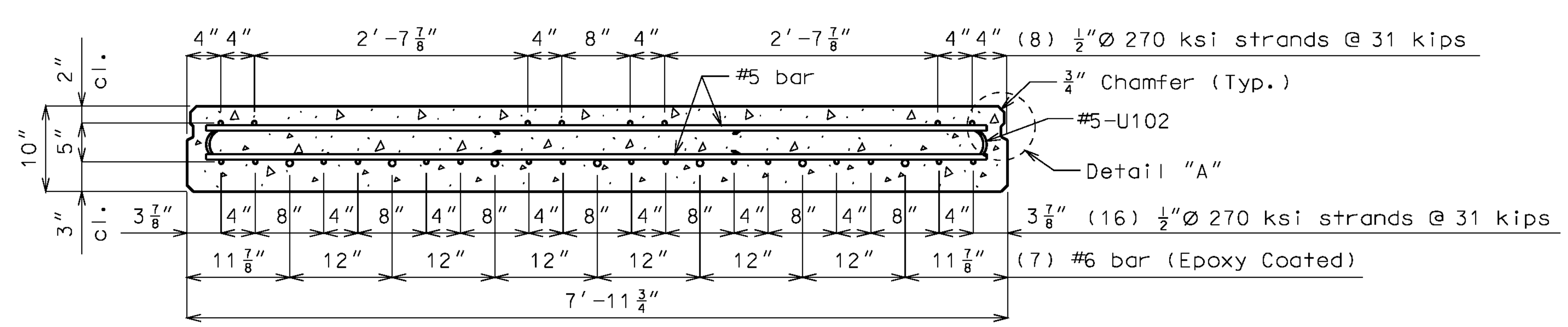


Note:
Nominal lengths are based on out to out dimensions shown in bending diagram and are listed for fabricators use (nearest inch).



PART SECTION THRU 1"Ø TIE ROD





SECTION THRU INTERIOR PANEL

GENERAL NOTES:

Concrete for bridge approach slab panels shall be Class A-1 with f'c = 6,000 psi, f'ci = 3,500 psi.

Top surface of panels shall be finished in accordance with Sec 1029.6.16.

Prestressing tendons shall be high-tensile strength uncoated seven-wire low-relaxation strands for prestressed concrete in accordance with AASHTO M 203 Grade 270, with nominal diameter of strand = 1/2" and nominal area = 0.153 sq. in. and minimum ultimate strength = 46.31 kips (270 ksi). Larger strands may be used with the same spacing and initial tension.

Initial prestressing force = 31 kips/strand.

The method and sequence of releasing the strands shall be shown on the shop drawings.

Suitable anchorage devices for lifting panels may be cast in panels, provided the devices are shown on the shop drawings and approved by the engineer. The fabricator shall design and locate the lifting loops for the Precast Concrete Approach Slab Panels.

Minimum clearance to reinforcing steel shall be 1 1/2", unless otherwise shown.

All holes in slab panels shall be cast. Drilling is not allowed.

Prestressing strands at ends of slab panels shall be trimmed to within 1/8" of concrete. Ends of slab panels shall be given 2 coats of an approved bituminous paint.

Work this sheet with Sheet No. 25.

DETAILS OF PRECAST CONCRETE APPROACH SLAB PANELS

Detailed July 2011
Checked July 2011

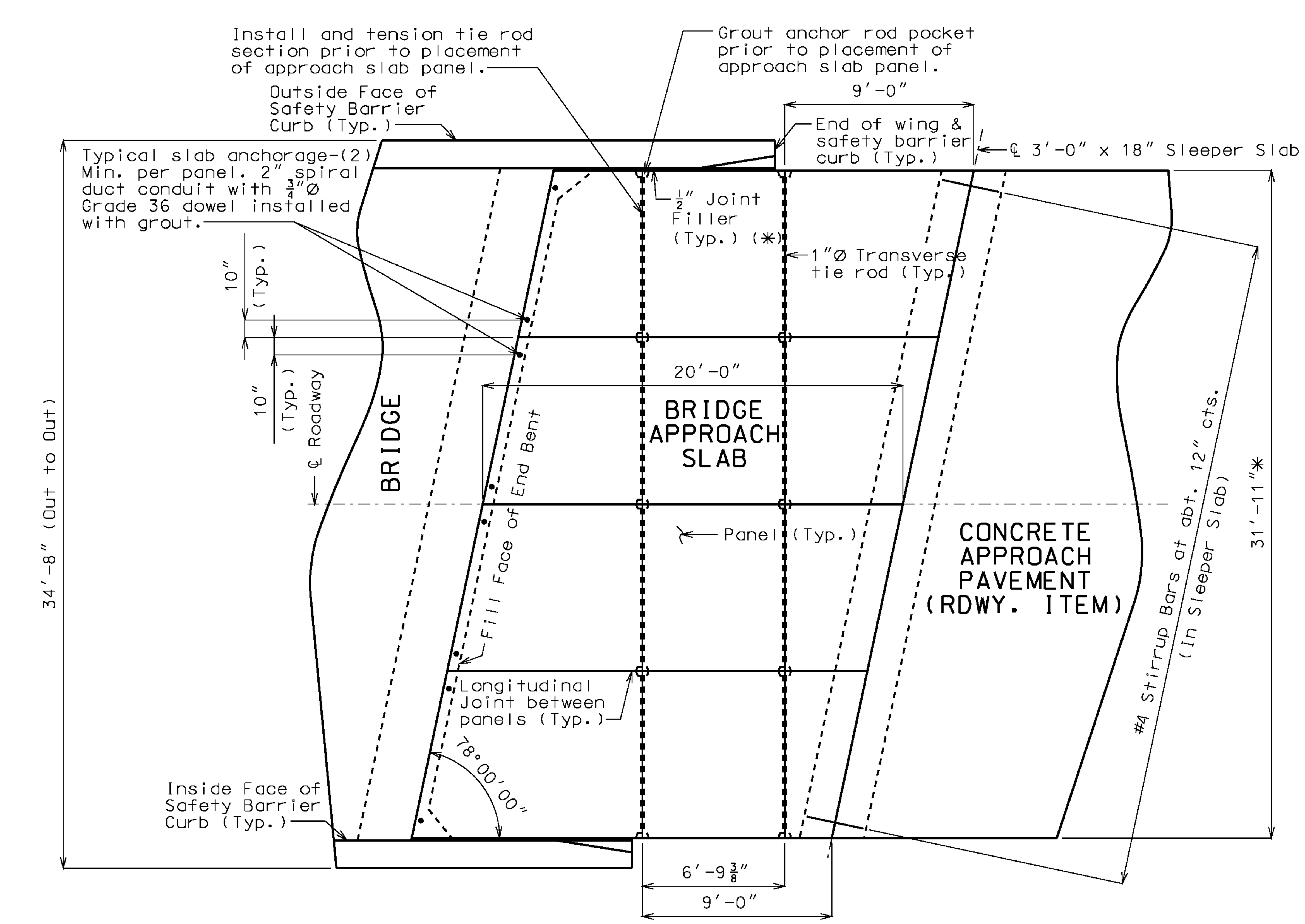
Note: This drawing is not to scale. Follow dimensions.

DATE PREPARED	
8/25/2011	
ROUTE 136	STATE MO
DISTRICT BR	SHEET NO. 26
COUNTY ATCHISON	
JOB NO. J1P1047	
CONTRACT ID.	
PROJECT NO.	
BRIDGE NO. A7934	

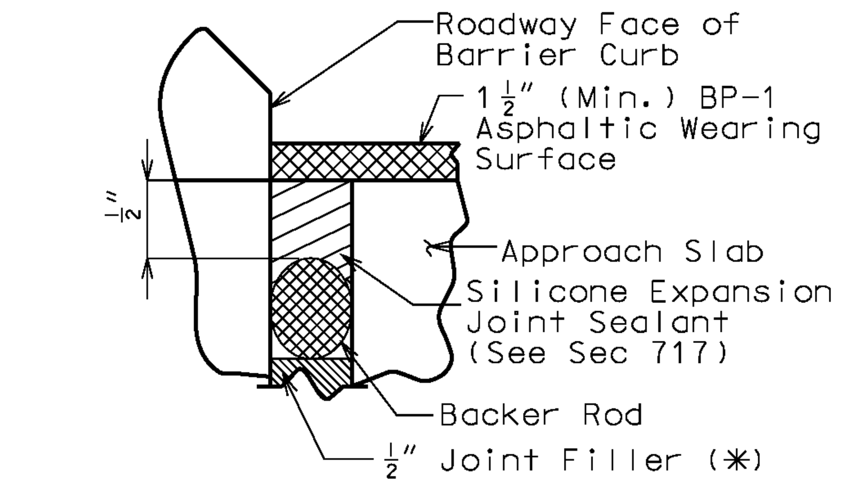
DESCRIPTION

DATE

MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION
105 WEST CAPITOL
JEFFERSON CITY, MO 65102
1-888-ASK-MODOT (1-888-275-6636)
MoDOT



PART PLAN OF BRIDGE APPROACH SLAB



SECTION THRU JOINT BETWEEN WING AND APPROACH SLAB

GENERAL NOTES:

All joint filler shall be in accordance with Sec 1057 for preformed fiber expansion joint filler, except as noted.

Minimum clearance to reinforcing steel shall be 1½", unless otherwise shown.

✱ Final dimension may be adjusted from that shown by the contractor based on field dimensions.

Payment for furnishing all materials, labor and equipment necessary to construct the approach slab, including the sleeper slab, underdrain, BP-1 asphaltic wearing surface, joint filler and all other appurtenances and incidental work as shown on this sheet, complete in place, will be considered completely covered by the contract unit price for Precast Concrete Approach Slab (Bridge) per square yard.

For Concrete Approach Pavement details, see roadway plans.

Drain pipe may be either 6" diameter corrugated metallic-coated pipe underdrain, 4" diameter corrugated polyvinyl chloride (PVC) drain pipe, or 4" diameter corrugated polyethylene (PE) drain pipe.

Reinforcement shall conform to the requirements of Sec 1036.

Keyway surface shall be cleaned to remove form oil or other bond breaking material prior to shipment of the precast concrete approach slab panel. Cleaning shall be done by sandblasting the keyway areas between top of the panel and the bottom edge of the key.

All Tie Rods, Plates, Sleeves, and nuts shall be galvanized in accordance with ASTM A153.

Tighten all tie rods (per precast concrete approach slab panel) to about one-half of the specified tension before proceeding with the final tensioning. Tie rods in the final two approach slab panels to be set may use a continuous tie rod across both panels.

Tie Rod nuts shall be tightened to provide a tension of one-half that specified for A325 bolts in Sec 712.7.3.

Keep approach slab panels roadway face up at all times. Support them within 12 inches of the ends only.

Tie Rod Plates shall be ASTM A709 Grade 36.

Sleeves, Nuts and 1" Ø Tie Rods shall be A307.

Dowel bars to be grouted in place and allowed to cure a minimum of 72 hours.

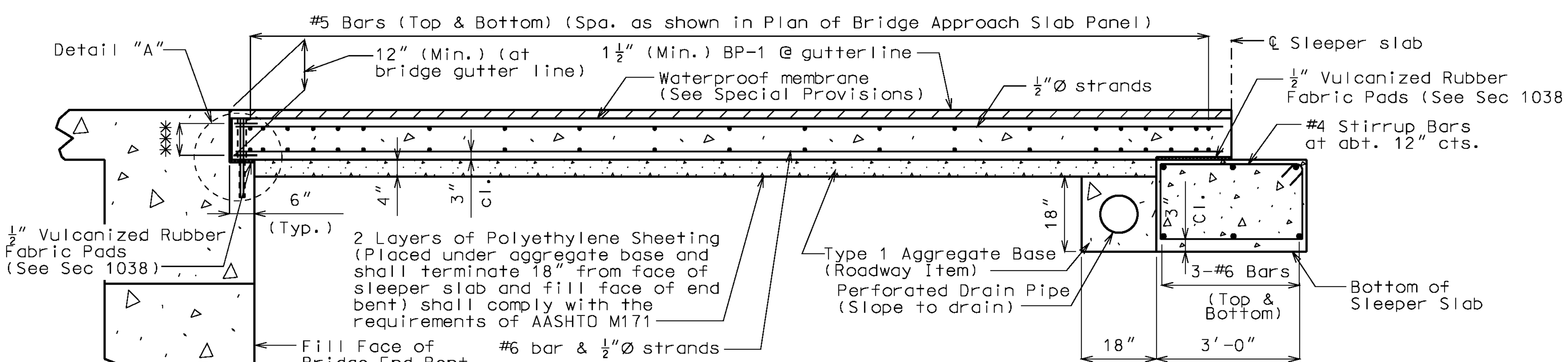
All concrete for the sleeper slab shall be in accordance with Sec 503 (f'c = 4,000 psi).

Hooks and bends shall be in accordance with the CRSI Manual of Standard Practice for Detailing Reinforced Concrete Structures, Stirrup and Tie Dimensions.

Dowel bars may be epoxy coated or galvanized in accordance with ASTM A123.

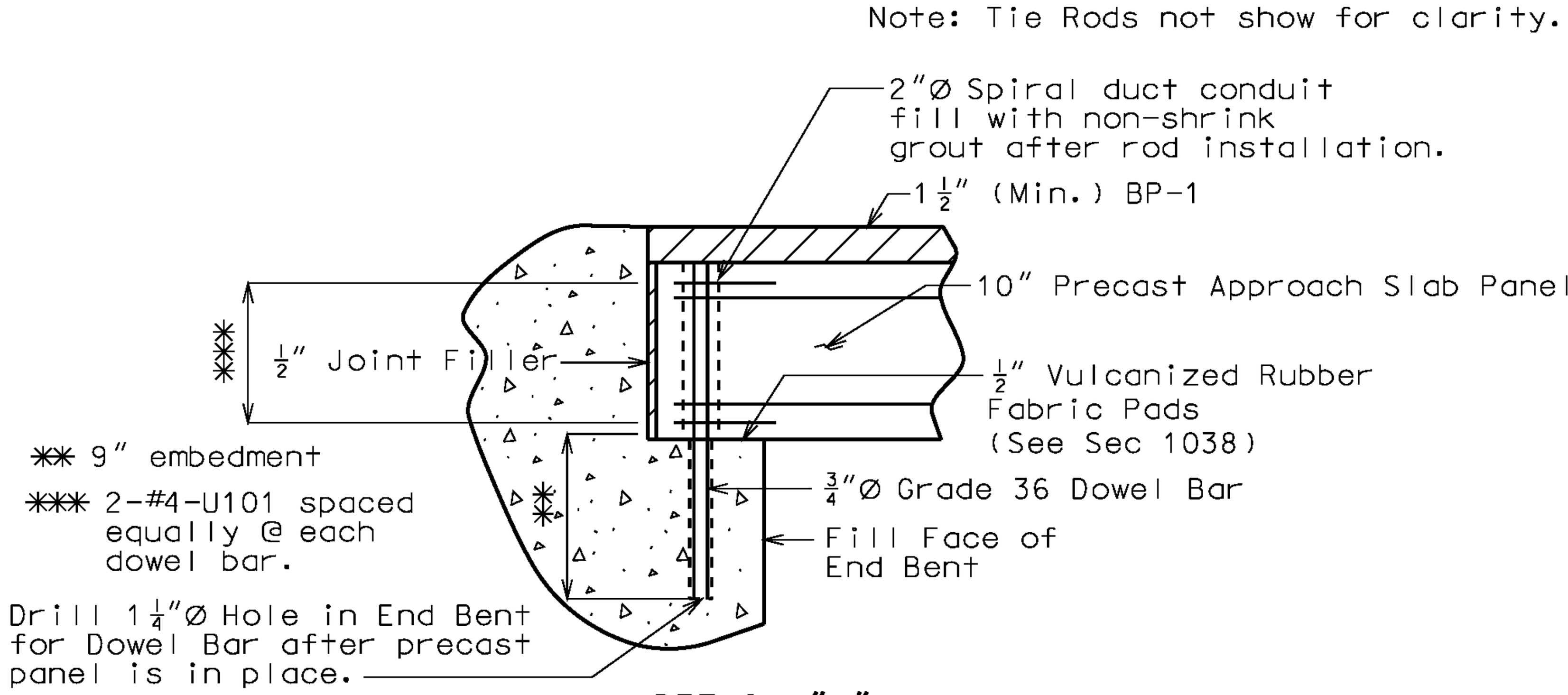
Aggregate base shall be level with the top of bearing pads and graded to be flush with the bottom of approach slab panels.

Work this sheet with Sheet No. 26.



SECTION THRU APPROACH SLAB

Note: Tie Rods not show for clarity.

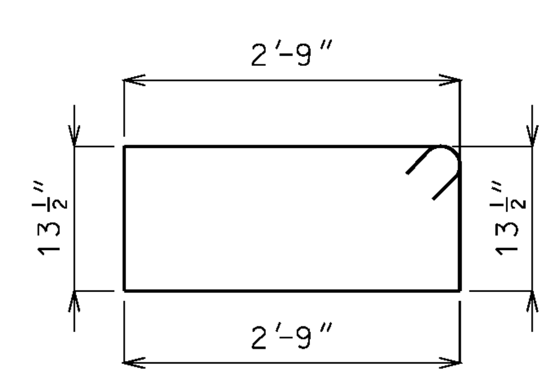


✱✱ 9" embedment

✱✱✱ 2-#4-U101 spaced equally @ each dowel bar.

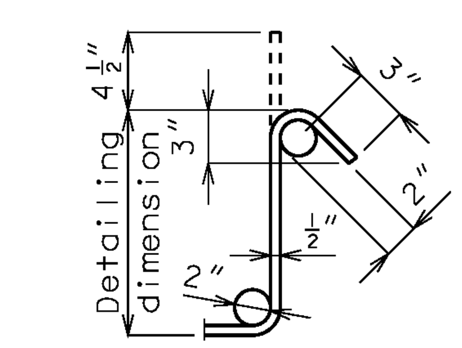
Drill 1¼"Ø Hole in End Bent for Dowel Bar after precast panel is in place.

DETAIL "A"

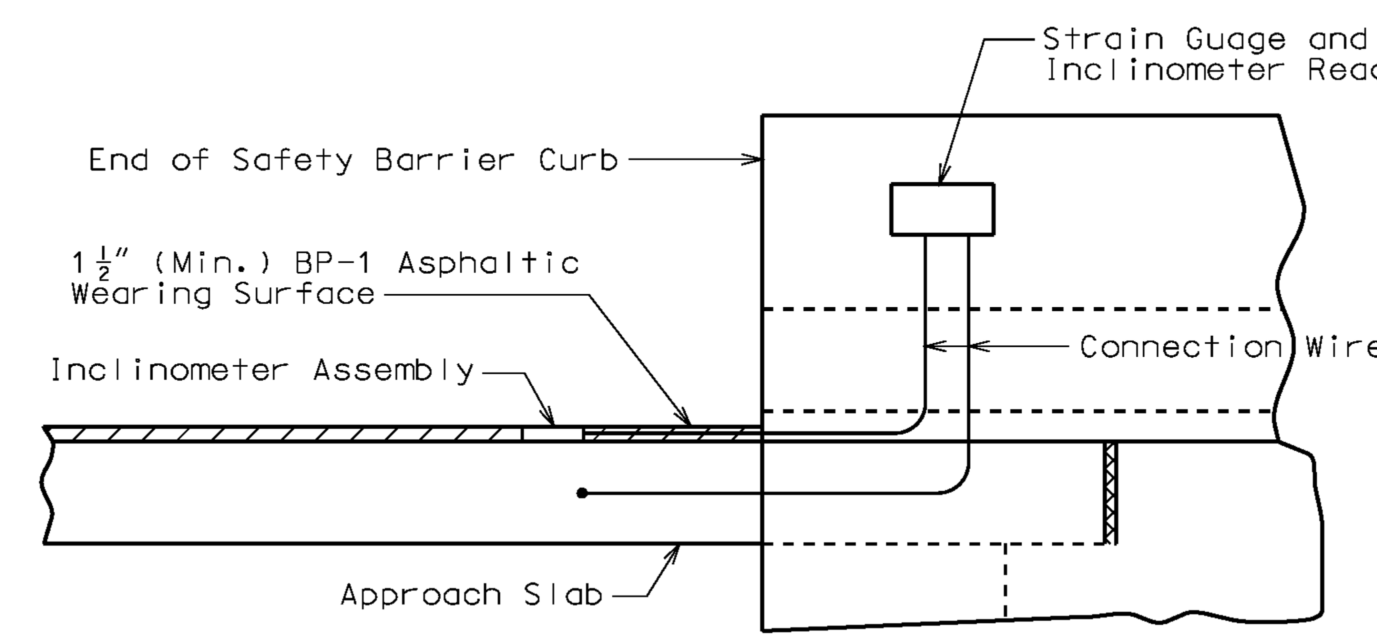


#4 STIRRUP BAR (ACTUAL LENGTH = 8'-3")

Note: Nominal lengths are based on out to out dimensions shown in bending diagram and are listed for fabricators use (nearest inch).



TYPICAL 135° STIRRUP HOOK DIMENSIONS BENDING DIAGRAM



DETAIL OF FINISHED STRAIN GUAGE AND INCLINOMETER PLACEMENT

Note: Final installation may vary from that shown. (Equipment and installation by research personnel, see Special Provisions)

Detailed July 2011
Checked July 2011

DETAILS OF PRECAST CONCRETE APPROACH SLAB

Note: This drawing is not to scale. Follow dimensions.

DATE PREPARED	8/25/2011
ROUTE	136
STATE	MO
DISTRICT	BR
SHEET NO.	25
COUNTY	ATCHISON
JOB NO.	J1P1047
CONTRACT ID.	
PROJECT NO.	
BRIDGE NO.	A7934

DESCRIPTION
DATE

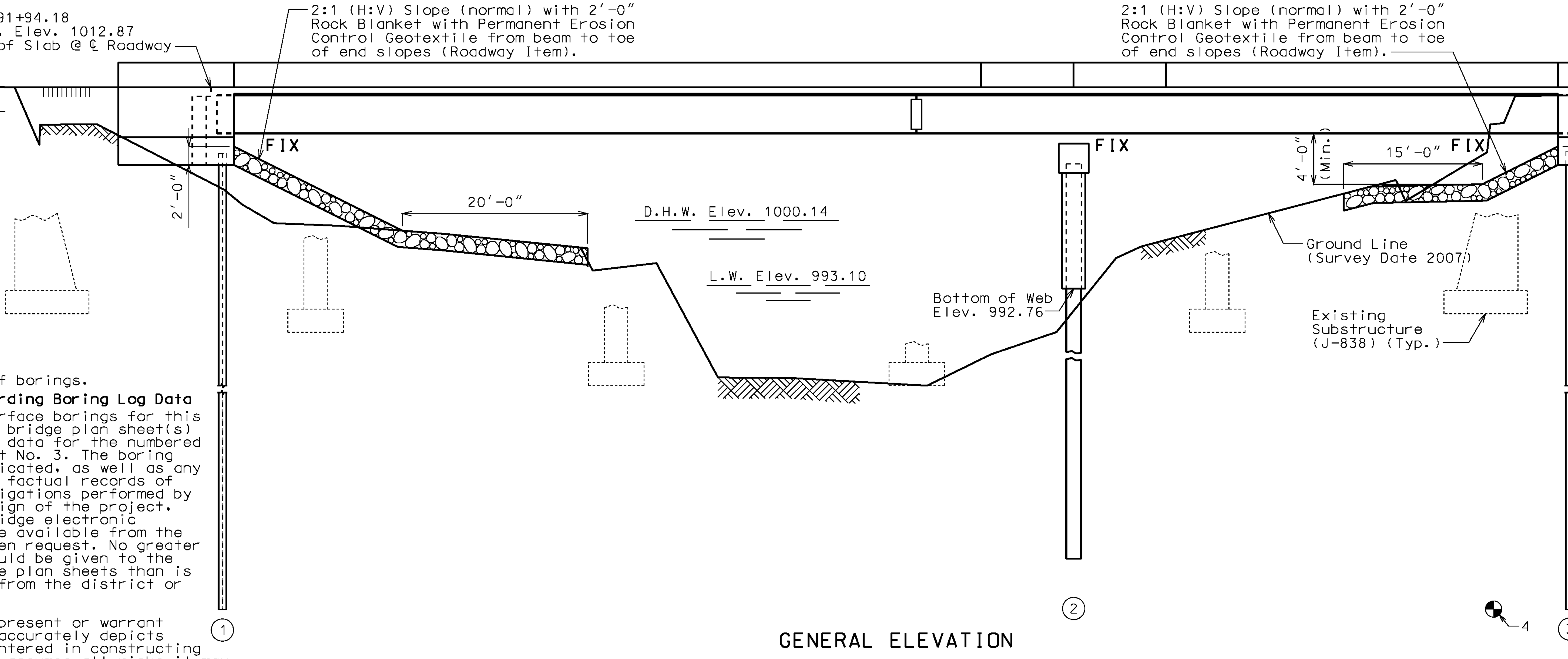
MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION

105 WEST CAPITOL
JEFFERSON CITY, MO 65102
1-888-ASK-MODOT (1-888-275-6636)

SEC/SUR 13/24 TWP 65N RGE 39W

MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION

(92'-55') CONTINUOUS COMPOSITE PLATE GIRDER SPANS



GENERAL ELEVATION

"⯐" Indicates location of borings.

Notice and Disclaimer Regarding Boring Log Data

The locations of all subsurface borings for this structure are shown on the bridge plan sheet(s) for this structure. Boring data for the numbered locations is shown on Sheet No. 3. The boring data for all locations indicated, as well as any other boring logs or other factual records of subsurface data and investigations performed by the department for the design of the project, will be provided in the bridge electronic deliverable file or will be available from the Project Contact upon written request. No greater significance or weight should be given to the boring data depicted on the plan sheets than is subsurface data available from the district or elsewhere.

The Commission does not represent or warrant that any such boring data accurately depicts the conditions to be encountered in constructing this project. A contractor assumes all risks it may encounter in basing its bid prices, time or schedule of performance on the boring data depicted here or those available from the district, or on any other documentation not expressly warranted, which the contractor may obtain from the Commission.

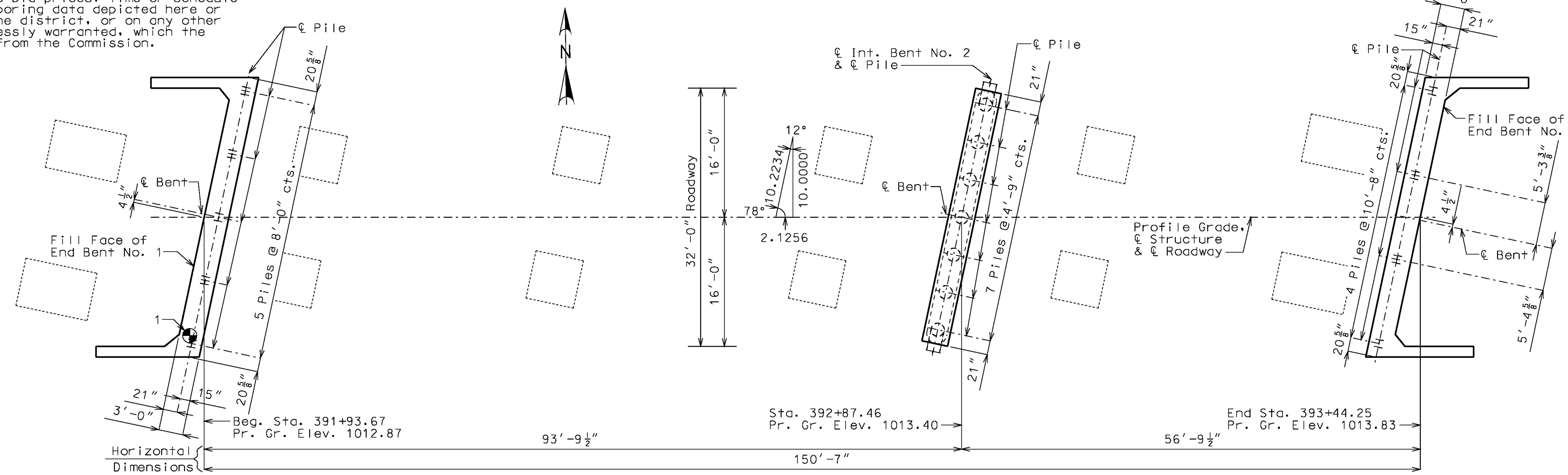
NOTES:

Roadway fill shall be completed to the final roadway section and up to the elevation of the bottom of the concrete beam within the limits of the structure and for not less than 20 feet in back of the fill face of the end bents before any piles are driven for any bents falling within the embankment section.

Old roadway fill under the ends of the bridge shall be removed as shown on plans. Removal of old roadway fill will be considered completely covered by the contract unit price for roadway excavation.

Outline of old work is indicated by light dashed lines. Heavy lines indicate new work.

For General Notes, Foundation Data, Estimated Quantities, Estimated Quantities for Slab on Steel, Location Sketch and Hydrologic Data, see Sheet No. 2.



PLAN

BM-2 CHISELED SQUARE ON S.E. CORNER OF GRAIN BIN. ±111 FT. LT. OF STA. 388+32 ℄ RTE. 136. ELEV. 1019.67 FT.

BRIDGE: RTE. 136 OVER LITTLE TARKIO CREEK

STATE ROAD FROM RTE. M TO NODAWAY CO. LINE

ABOUT 0.4 MILES EAST OF RTE. M

STA. 391+93.67

STD. 617.10

STD. 706.35

Designed Apr. 2011
Detailed Apr. 2011
Checked July 2011

Note: This drawing is not to scale. Follow dimensions.

"THIS MEDIA SHOULD NOT BE CONSIDERED A CERTIFIED DOCUMENT."

DATE PREPARED	
8/25/2011	
ROUTE	STATE
136	MO
DISTRICT	SHEET NO.
BR	1
COUNTY	
ATCHISON	
JOB NO.	
J1P1047	
CONTRACT ID.	
PROJECT NO.	
BRIDGE NO.	
A7934	

DESCRIPTION

DATE

MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION

105 WEST CAPITOL
JEFFERSON CITY, MO 65102
1-888-ASK-MODOT (1-888-275-6636)

APPENDIX III B0563 DESIGN DRAWINGS

Boring 2 (Core) 3+12, 16' RT

Blows Per Foot	
	Elev. 945.8
	Fill, Topsoil Material
	Elev. 945.5
5	Fill, lean clay, yellow brown and gray
5	
7	Elev. 917.8
	Lean Clay, stiff, dark brown, trace roots
	Elev. 916.3
8	Lean Clay, medium to stiff, gray
50/5"	Elev. 903.3
RQD =23%	Gravelly Lean Clay, stiff, yellow brown and gray
	Elev. 900.8
	Auger Refusal @ 45' Slighty Weathered Limestone, hard, thin bedded, gray white
	Elev. 895.8

2
(Core)
3+12
16' RT

Boring 1 (Core) 3+74, 67' RT

Blows Per Foot	
	Elev. 946.9
	Fill, Topsoil Material
	Elev. 946.6
4	Fill, lean clay, yellow brown and gray
5	
6	Elev. 922.4
7	Lean Clay, medium, yellow brown and gray
9	Elev. 898.9
14	Weathered Limestone, moderately hard, thin bedded, gray white
RQD =0%	Elev. 896.9
RQD =98%	Auger Refusal @ 50', Sightly Weathered Limestone, hard, thin bedded, gray white.
RQD =98%	Elev. 894.9
RQD =95%	Limestone, hard, medium bedded, gray white.
RQD =100%	Unconfined Strength @ 58'=6,390 psi
	Unconfined Strength @ 65.3'=10,150 psi
	Elev. 871.9

1
(Core)
3+74
67' RT

Boring 4 (Core) 5+24, 17' RT

Blows Per Foot	
	Elev. 945.2
5	Fill, lean clay, with sand and gravel, brown and dark brown
6	
8	Elev. 915.2
	Lean Clay, very stiff, brown
14	Elev. 902.2
20	Gravelly Lean Clay, very stiff, gray brown
	Elev. 897.7
RQD =12%	Weathered Limestone, thin bedded, gray white
	Elev. 896.7
RQD =100%	Auger Refusal @ 48.5 Slightly weathered Limestone, hard, thin bedded, gray white
RQD =100%	Elev. 892.7
RQD =94%	Limestone, hard, medium to thick bedded, gray white, Unconfined Strength @ 55'=8,070 psi
RQD =96%	Elev. 871.7

4
(Core)
5+24
17' RT

Boring 3 (Core) 5+73, 66' RT

Blows Per Foot	
	Elev. 945.9
	Fill, limestone gravel
11	Elev. 944.4
6	Fill, lean clay, with sand and gravel, dark brown
9	
	Elev. 914.9
12	Lean Clay, with gravel, very stiff, brown and gray
	Elev. 902.4
26	Gravelly Lean Clay, very stiff, brown and gray
	Elev. 898.9
RQD =0%	Weathered Limestone, thin bedded, gray white
RQD =91%	Elev. 897.9
	Auger Refusal @ 48' Slightly Weathered Limestone, hard thin bedded, gray white
	Elev. 894.9
	Unconfined Compression @ 51.4' = 6,940 psi Limestone, hard, medium bedded, gray white
	Elev. 887.9

3
(Core)
5+73
66' RT

SOIL BORING DATA

Detailed December 2010
Checked December 2010

Note: This drawing is not to scale. Follow dimensions.

DATE PREPARED	1/11/2011
ROUTE	71
STATE	MO
DISTRICT	BR
SHEET NO.	3
COUNTY	JASPER
JOB NO.	J7B0801K
CONTRACT ID.	
PROJECT NO.	
BRIDGE NO.	B0563

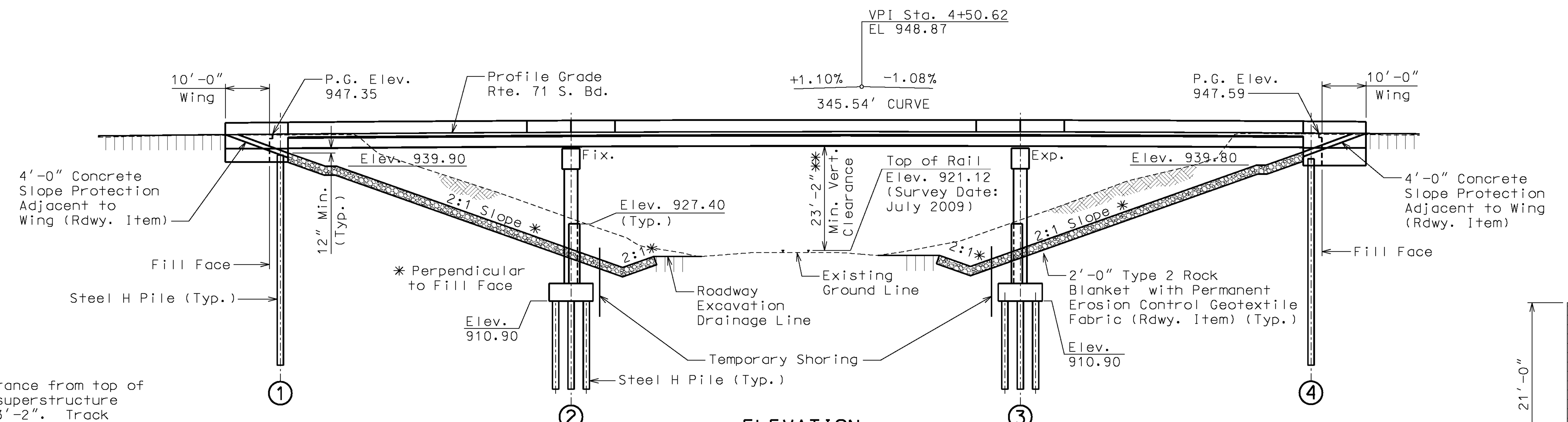
DESCRIPTION | DATE

MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION
105 WEST CAPITOL
JEFFERSON CITY, MO 65102
1-888-ASK-MODOT (1-888-275-6636)

HDR Engineering, Inc.
4435 Main Street
Suite 1000
Kansas City, MO 64111-1856
816-360-2700
Certificate of Authority: 000856

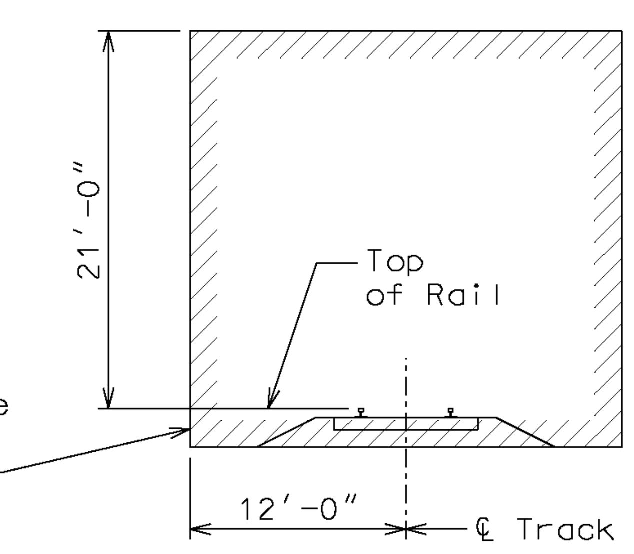
MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION

(66'-102'-66') CONTINUOUS COMPOSITE PLATE GIRDER SPANS

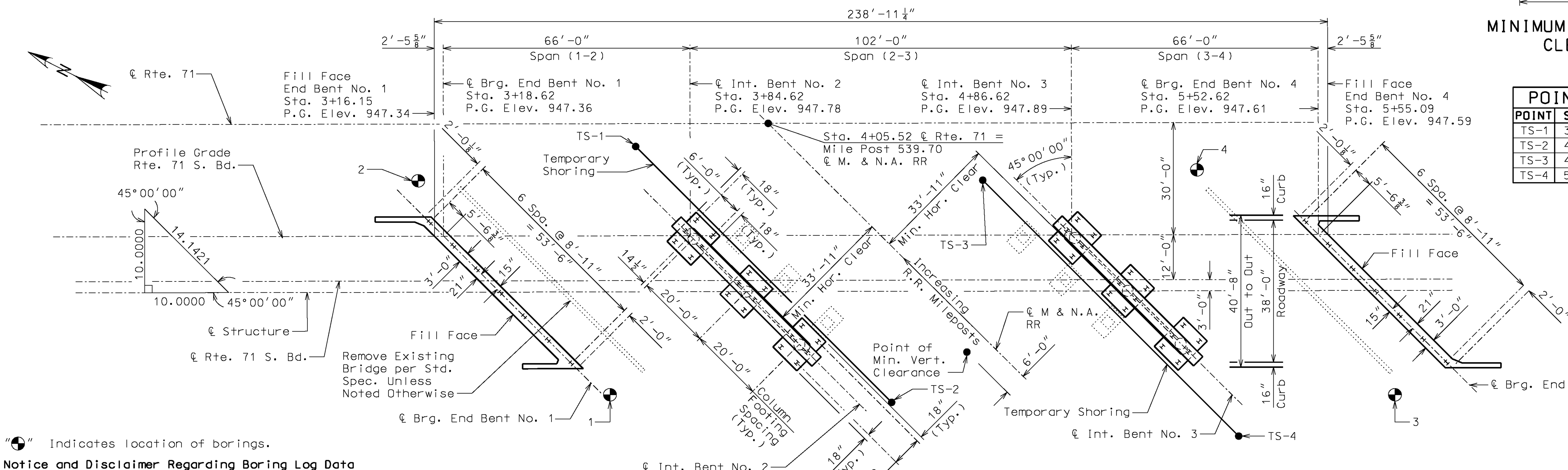


** Final vertical clearance from top of rails to bottom of superstructure shall be at least 23'-2". Track elevations should be verified in the field prior to construction to determine if the final vertical clearance shown will be obtained.

ELEVATION



MINIMUM CONSTRUCTION CLEARANCES



POINT	STATION	OFFSET
TS-1	3+72.04	7'-6¼" RT.
TS-2	4+37.09	72'-6⅞" RT.
TS-3	4+64.15	17'-5⅛" RT.
TS-4	5+29.20	82'-5¾" RT.

Table: POINT DATA TABLE

"⊕" Indicates location of borings.

Notice and Disclaimer Regarding Boring Log Data

The locations of all subsurface borings for this structure are shown on the bridge plan sheet(s) for this structure. Boring data for the numbered locations is shown on Sheet No. 3. The boring data for all locations indicated, as well as any other boring logs or other factual records of subsurface data and investigations performed by the department for the design of the project, will be provided in the bridge electronic deliverable file or will be available from the Project Contact upon written request. No greater significance or weight should be given to the boring data depicted on the plan sheets than is subsurface data available from the district or elsewhere.

The Commission does not represent or warrant that any such boring data accurately depicts the conditions to be encountered in constructing this project. A contractor assumes all risks it may encounter in basing its bid prices, time or schedule of performance on the boring data depicted here or those available from the district, or on any other documentation not expressly warranted, which the contractor may obtain from the Commission.

BM #2
STA. 1069+10.38, 69.34' RT.
ELEV. 929.30
½" IRON BAR WEST SIDE OF RTE. 71
527' +/- NORTH OF THE BARTON/JASPER COUNTY LINE

B.M. #3
STA. 2+67.08, 17.99' LET.
ELEV. 947.00
CHISELED "+" IN CORNER OF CONCRETE DRAINAGE BASIN

ROUTE SR 71 (J7B0801K - SB STRUCTURE)
CONST. STA. 4+05.52
MISSOURI AND NORTHERN ARKANSAS RAILROAD (MNAR)
VAL-STA. 5477+51
AAR/DOT# 443240F (HWY OVERPASS)
RR MP. 539.70
REGION: CENTRAL
SUBDV.: NEVADA

GENERAL ELEVATION AND PLAN

BRIDGE (RTE. 71 S. BD.) OVER M. & N.A. RAILROAD
STATE ROAD FROM BARTON CO. LINE TO JASPER
ABOUT 0.1 MILES SOUTH OF BARTON CO. LINE
STA. 3+84.62
RTE. 71 S. Bd.

SAFE & SOUND

STD. 609.00
STD. 617.10
STD. 706.35

Detailed December 2010
Checked December 2010

Note: This drawing is not to scale. Follow dimensions.

DATE PREPARED	
1/11/2011	
ROUTE 71	STATE MO
DISTRICT BR	SHEET NO. 1
COUNTY JASPER	
JOB NO. J7B0801K	
CONTRACT ID.	
PROJECT NO.	
BRIDGE NO. B0563	

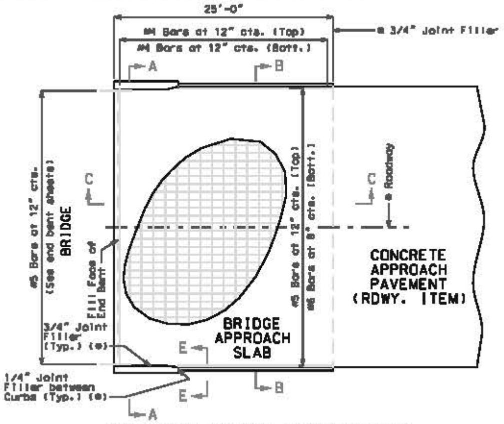
DESCRIPTION
DATE

MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION
105 WEST CAPITOL
JEFFERSON CITY, MO 65102
1-888-ASK-MODOT (1-888-275-6636)
MoDOT

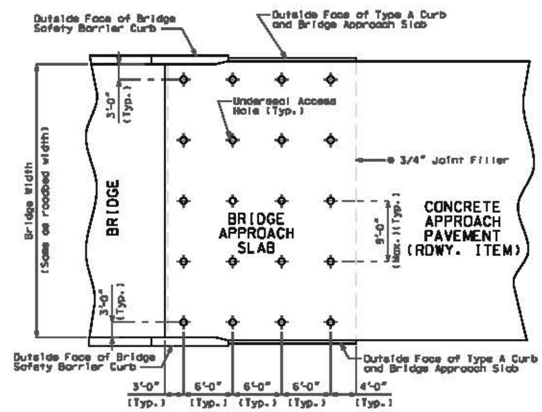
HDR Engineering, Inc.
4435 Main Street
Suite 1000
Kansas City, MO 64111-1856
816-360-2700
Certificate of Authority: 000856

c:\pwworking\oma\d0418703\B_B0563_001_J7B0801K.DGN 1:13:43 AM 1/12/2011

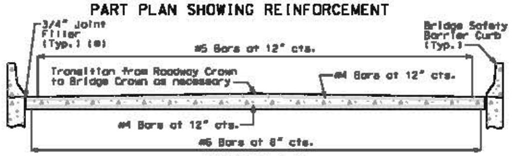
APPENDIX IV A7890 DESIGN DRAWINGS



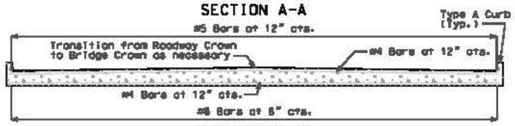
PART PLAN SHOWING REINFORCEMENT



PART PLAN
(SHOWING TYPICAL UNDERSEAL ACCESS HOLE LOCATIONS)

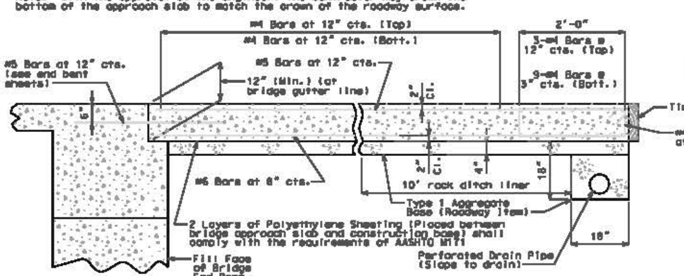


SECTION A-A

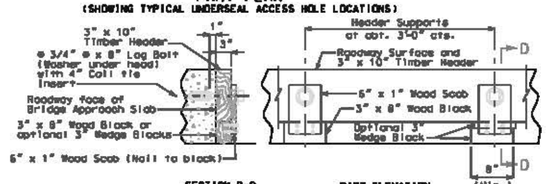


SECTION B-B

Note: With the approval of the engineer, the contractor may crown the bottom of the approach slab to match the crown of the roadway surface.



SECTION C-C

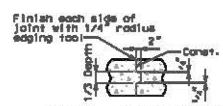


SECTION D-D

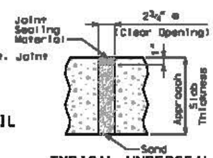
PART ELEVATION

Note: Remove timber header when concrete pavement is placed.

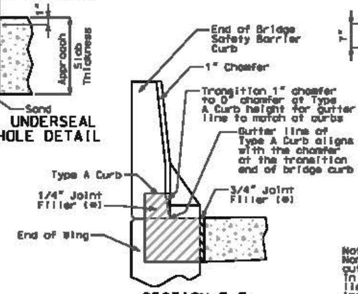
DETAILS OF TIMBER HEADER



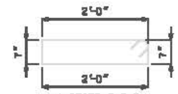
CONST. JOINT DETAIL
(IF REQUIRED)



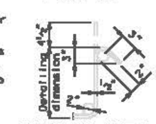
TYPICAL UNDERSEAL ACCESS HOLE DETAIL



SECTION E-E
(BETWEEN CURBS)



#4 STIRRUP BAR



TYPICAL 135° STIRRUP HOOK DIMENSIONS BENDING DIAGRAM

Note:
Nominal lengths are based on out to out dimensions shown in bending diagram and are listed for fabricators use (nearest inch).

GENERAL NOTES:

All concrete for the bridge approach slab shall be in accordance with Sec 503 (f'c =4,000 psi).

All joint filler shall be in accordance with Sec 1057 for preformed fiber expansion joint filler, except as noted.

The reinforcing steel in the bridge approach slab shall be epoxy coated Grade 60 with Fy = 60,000 psi.

Minimum clearance to reinforcing steel shall be 1 1/2", unless otherwise shown.

The reinforcing steel in the bridge approach slab shall be continuous. The transverse reinforcing steel may be made continuous by lap splicing the #4 bars 18".

Mechanical bar splices shall be in accordance with Sec 706.

(*) Seal joint between vertical face of approach slab and wing with "Silicone Joint Sealant for Saw Cut and Formed Joints" in accordance with Sec 717.

Hooks and bends shall be in accordance with the CRSI Manual of Standard Practice for Detailing Reinforced Concrete Structures, Stirrup and Tie Dimensions.

The contractor shall pour and satisfactorily finish the bridge or semi-deep slab before pouring the bridge approach slabs.

Longitudinal construction joints in approach slab shall be aligned with longitudinal construction joints in bridge or semi-deep slab.

Payment for furnishing all materials, labor and excavation necessary to construct the approach slab, including the timber header, underdrain, Type 1 aggregate base, joint filler and all other appurtenances and incidental work as shown on this sheet, complete in place, will be considered completely covered by the contract unit price for Bridge Approach Slab (Bridge) per square yard.

For Concrete Approach Pavement details, see roadway plans.

See Missouri Standard Plans Drawing 609.00 for details of Type A Curb.

At the contractor's option, Grade 40 reinforcement may be substituted for the Grade 60 #5 dowel bars connecting the bridge approach slab to the bridge abutment. No additional payment will be made for this substitution.

When Grade 40 reinforcement is substituted for the Grade 60 #5 dowel bars connecting the bridge approach slab to the bridge abutment, the reinforcement may be bent up to 90 degrees with a 2" minimum radius near the abutment to allow compaction of the backfill material near the abutment. Damage to epoxy coating shall be repaired in accordance with Sec 710.

Drain pipe may be either 6" diameter corrugated metallic-coated pipe underdrain, 4" diameter corrugated polyvinyl chloride (PVC) drain pipe, or 4" diameter corrugated polyethylene (PE) drain pipe.

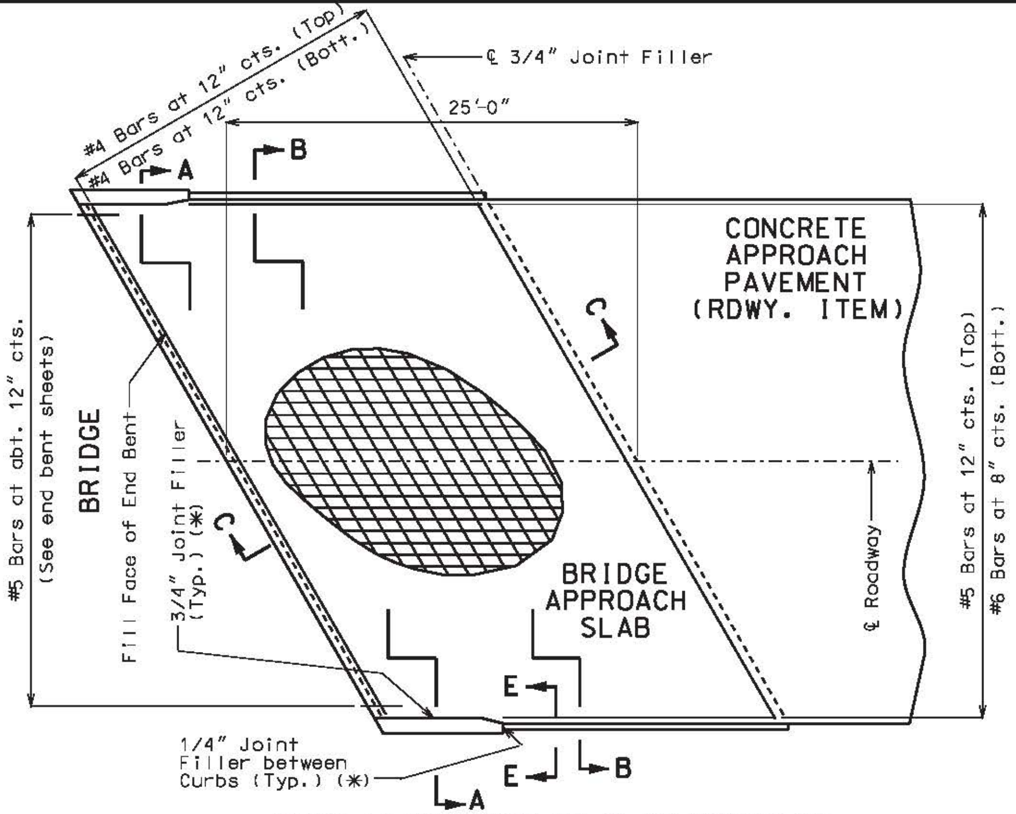
38
BR 28

Detailed Oct. 2010
Checked Oct. 2010

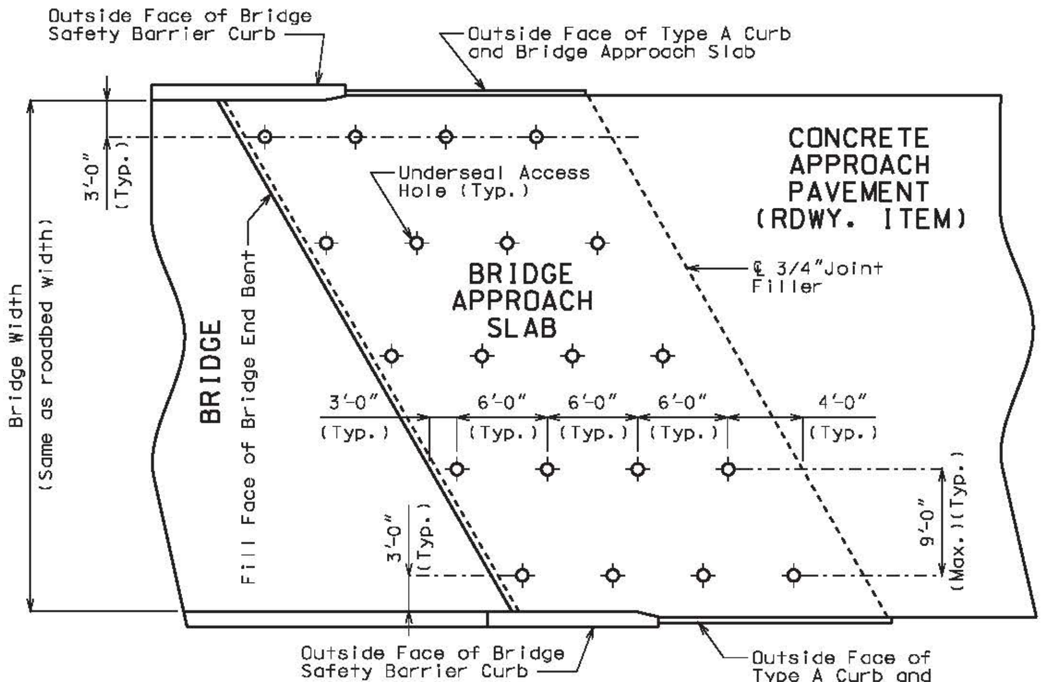
BRIDGE APPROACH SLAB

Note: This drawing is not to scale. Follow dimensions.

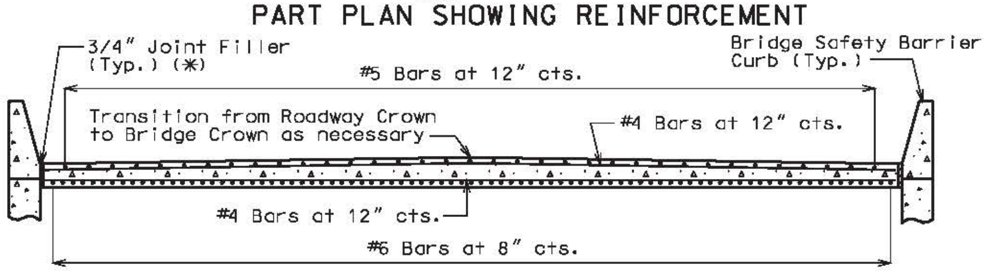
APPENDIX V A7925 DESIGN DRAWINGS



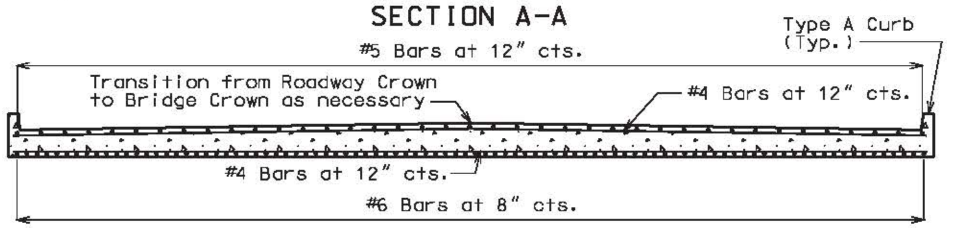
PART PLAN SHOWING REINFORCEMENT



PART PLAN
(SHOWING TYPICAL UNDERSEAL ACCESS HOLE LOCATIONS)

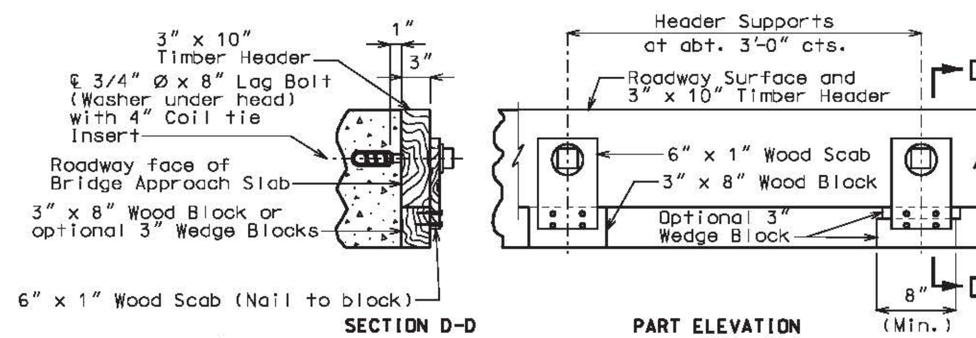


SECTION A-A



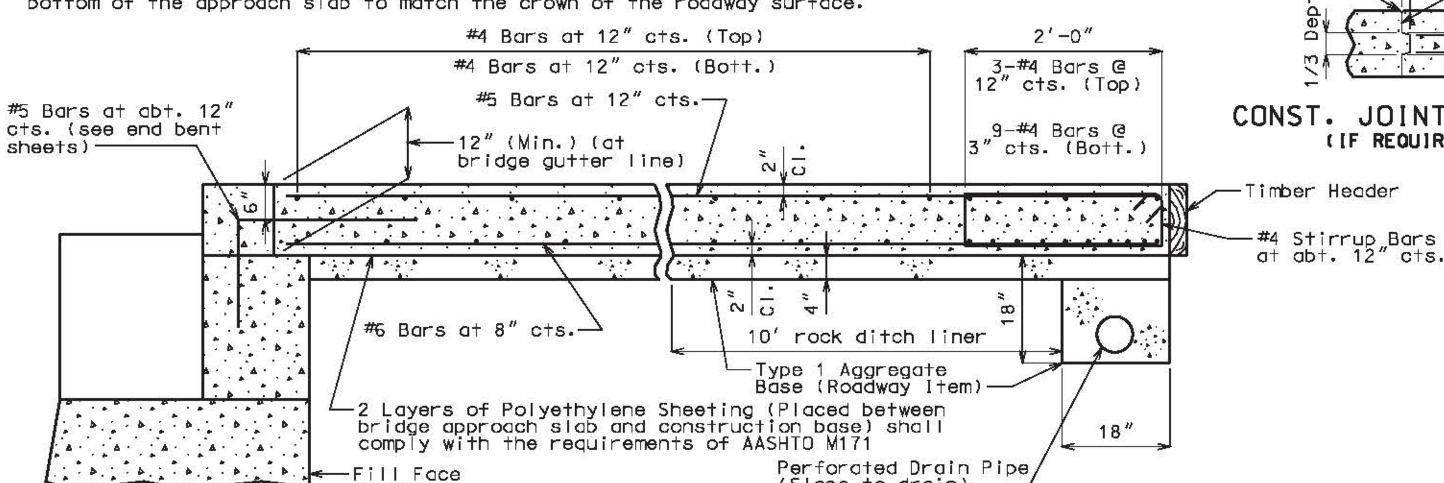
SECTION B-B

Note: With the approval of the engineer, the contractor may crown the bottom of the approach slab to match the crown of the roadway surface.

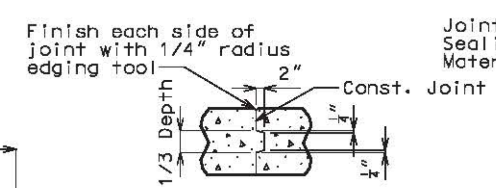


Note: Remove timber header when concrete pavement is placed.

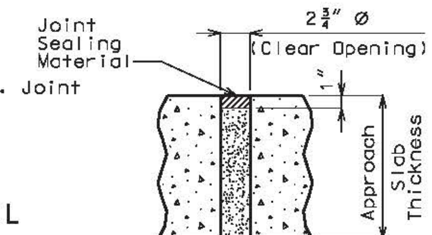
DETAILS OF TIMBER HEADER



SECTION C-C



CONST. JOINT DETAIL
(IF REQUIRED)



TYPICAL UNDERSEAL ACCESS HOLE DETAIL

GENERAL NOTES:

All concrete for the bridge approach slab shall be in accordance with Sec 503 (f'c = 4,000 psi).

All joint filler shall be in accordance with Sec 1057 for preformed fiber expansion joint filler, except as noted.

The reinforcing steel in the bridge approach slab shall be epoxy coated Grade 60 with Fy = 60,000 psi.

Minimum clearance to reinforcing steel shall be 1 1/2", unless otherwise shown.

The reinforcing steel in the bridge approach slab shall be continuous. The transverse reinforcing steel may be made continuous by laps plicing the #4 bars 18".

Mechanical bar splices shall be in accordance with Sec 706.

(*) Seal joint between vertical face of approach slab and wing with "Silicone Joint Sealant for Saw Cut and Formed Joints" in accordance with Sec 717.

Hooks and bends shall be in accordance with the CRSI Manual of Standard Practice for Detailing Reinforced Concrete Structures, Stirrup and Tie Dimensions.

The contractor shall pour and satisfactorily finish the bridge or semi-deep slab before pouring the bridge approach slabs.

Longitudinal construction joints in approach slab shall be aligned with longitudinal construction joints in bridge or semi-deep slab.

Payment for furnishing all materials, labor and excavation necessary to construct the approach slab, including the timber header, underdrain, Type 1 aggregate base, joint filler and all other appurtenances and incidental work as shown on this sheet, complete in place, will be considered completely covered by the contract unit price for Bridge Approach Slab (Bridge) per square yard.

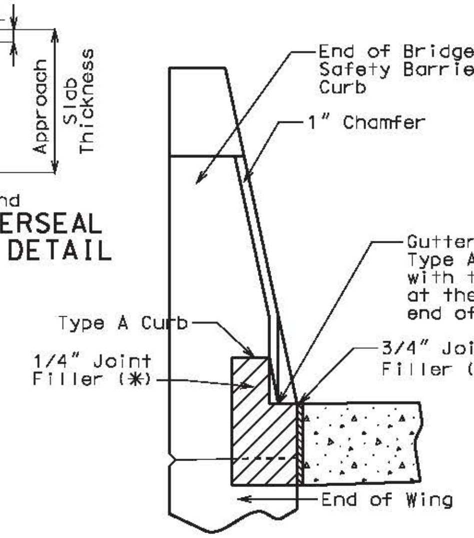
For Concrete Approach Pavement details, see roadway plans.

See Missouri Standard Plans Drawing 609.00 for details of Type A Curb.

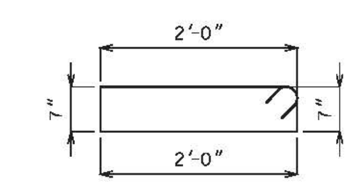
At the contractor's option, Grade 40 reinforcement may be substituted for the Grade 60 #5 dowel bars connecting the bridge approach slab to the bridge abutment. No additional payment will be made for this substitution.

When Grade 40 reinforcement is substituted for the Grade 60 #5 dowel bars connecting the bridge approach slab to the bridge abutment, the reinforcement may be bent up to 90 degrees with a 2" minimum radius near the abutment to allow compaction of the backfill material near the abutment. Damage to epoxy coating shall be repaired in accordance with Sec 710.

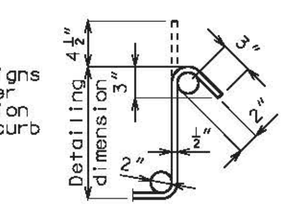
Drain pipe may be either 6" diameter corrugated metallic-coated pipe underdrain, 4" diameter corrugated polyvinyl chloride (PVC) drain pipe, or 4" diameter corrugated polyethylene (PE) drain pipe.



SECTION E-E
(BETWEEN CURBS)



#4 STIRRUP BAR



TYPICAL 135° STIRRUP HOOK DIMENSIONS BENDING DIAGRAM

Note:
Nominal lengths are based on out to out dimensions shown in bending diagram and are listed for fabricators use (nearest inch).

STATE OF MISSOURI
DEAN DAVID FRANKE
NUMBER PE-28132
PROFESSIONAL ENGINEER

THIS SHEET HAS BEEN SIGNED, SEALED AND DATED ELECTRONICALLY.

DATE PREPARED	
3/28/2011	
ROUTE 45	STATE MO
DISTRICT BR	SHEET NO. 33
COUNTY PLATTE	
JOB NO. J4P2203	
CONTRACT ID.	
PROJECT NO.	
BRIDGE NO. A7925	

DESCRIPTION | DATE

MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION
MoDOT
105 WEST CAPITOL
JEFFERSON CITY, MO 65102
1-888-ASK-MODOT (1-888-275-6636)

IF A SEAL IS PRESENT ON THIS SHEET IT HAS BEEN ELECTRONICALLY SEALED AND DATED.

BRIDGE APPROACH SLAB

Detailed Dec. 2010
Checked Jan. 2011

Note: This drawing is not to scale. Follow dimensions.

APPENDIX VI – SUPPLEMENTAL PCPS PANEL RESULTS

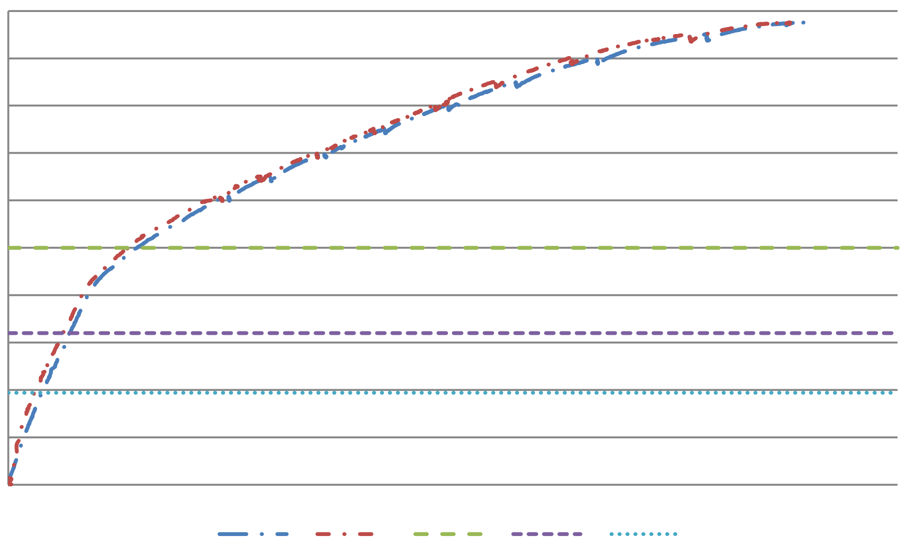


Figure F.1 Applied Load versus Deflection

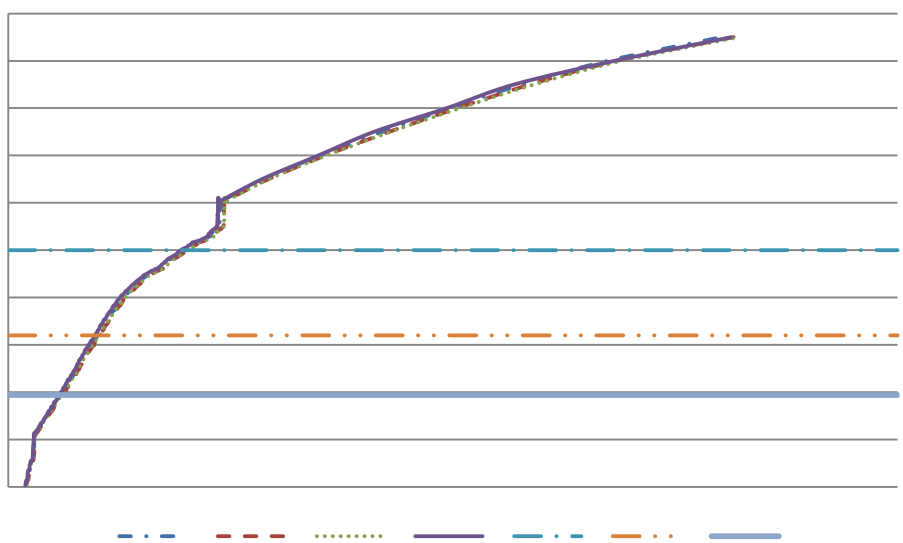


Figure F.2 Applied Load versus Rotation

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