

PRECAST PRESTRESSED BRIDGE APPROACH SLAB -COST EFFECTIVE DESIGNS

A Thesis
presented to
the Faculty of the Graduate School
University of Missouri – Columbia

In Partial Fulfillment
of the Requirements for the Degree

Master of Science

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May 2012

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Acknowledgements:

I would like to thank Missouri Department of Transportation (MoDOT), National University Transportation Center (NUTC) at Missouri Science & Technology (MST) and the Civil and Environmental Engineering department at the University of Missouri-Columbia for funding the research project and for providing financial support during my masters program.

I want to convey special thanks to my advisor Dr. V. S. Gopalaratnam for encouraging me throughout my masters program and guiding me through the learning curve. My thesis would not have been possible without his timely feedback and valuable suggestions. I have learnt a lot from him and I owe a lot for instilling critical thinking abilities in me and for transforming me into an engineer.

I want to thank Ma Shuang for her work on a companion master's thesis on Bridge approach slab design incorporating elastic soil support (BAS-ES) and Ravi Shankar Chamarthi for his help in performing life cycle cost simulations of the various design alternatives. I would also like to thank Dr. Ganesh Thiagarajan and Sheetal Ajgaonkar from University of Missouri-Kansas City for their contributions to the collaborative MoDOT project.

My masters would not have been possible without the support of my parents. I cannot thank them enough for encouraging me and for believing in me throughout. I want to thank my friends here and back home for being there for me.

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ABSTRACT

Bridge approach slabs (BAS) are transition slabs used to connect the roadway with the bridge. Among the various problems bridge approach slabs experience, differential settlement is the found to be the major cause leading to approach slab distress. Differential settlement depends on the support conditions, approach slab type, soils conditions underneath it and slab drainage. The approach slab design currently in use by the Missouri Department of Transportation (MoDOT) is a simply supported doubly reinforced concrete slab resting on the abutment on the bridge side and the embankment on the roadway side. The primary objective of this thesis was to suggest an efficient bridge approach slab system which is also cost effective. Two types of precast prestressed slabs, a 12'' deep slab and a 10'' deep slab with a 2'' cast-in-place topping are recommended as the design alternatives. The precast prestressed slabs not only control crack width but also lead to lower user costs especially in urban situations. The suggested alternatives are also effective for rapid replacement/repair operations on bridge approach slabs. Both the precast prestressed design alternatives are designed for service limit states and are successfully verified for ultimate moments. A life cycle analysis (LCCA) was completed to study comparative costs for urban and rural traffic patterns and to investigate the economic effectiveness of the precast prestressed slab designs. The

MoDOT BAS design along with another design alternative called BAS incorporating elastic support (BAS-ES) were included in the LCCA procedure to study the effectiveness of the precast prestressed alternatives. Agency as well as user costs are calculated and compared in urban and rural traffic scenarios for the design alternatives. When present value of total costs are considered, the Fully Precast Prestressed - BAS design is the most cost-effective when AADT counts are high, such as with urban traffic demands. Both agency and user costs decreased with an increase in the discount rate. The decrease in costs was rapid from a discount rate of 4% to 7%. A further increase in discount rate from 7% to 10% did not cause any significant cost drop. In the sensitivity analysis results, higher correlation values were obtained for work zone durations during initial construction. Other significant inputs include value of time for trucks and passenger cars, discount rate and work zone capacity.

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NOMENCLATURE/LIST OF NOTATION

A_c	Area of Concrete
Z_b	Section Modulus with respect to the Top Fiber
Z_t	Section Modulus with respect to the Bottom Fiber
F_i	Initial Prestressing Force
ηF_i	Final Prestressing Force after Losses
σ_{ci}	Allowable Stress in the Bottom Fiber during Initial Prestressing
σ_{ti}	Allowable Stress in the Top Fiber during Initial Prestressing
σ_{cs}	Allowable Stress in the Top Fiber during Service Loading
σ_{ts}	Allowable Stress in the Bottom Fiber during Service Loading
M_{min}	Moment due to the Dead Weight of the Slab
M_{max}	Moment due to the Service Loading
y_b	Clear Cover
f_r	Modulus of Rupture of Concrete
M_{cr}	Cracking Moment in the Slab
M_n	Nominal Moment
v_{ci}	Shear Cracking Stress under Flexure

V_G	Shear due to the Own Weight of the Composite Section
ΔV_u	Factored Shear Force due to Dead Load and Live Load at the Considered Section
ΔM_u	Factored Moment due to the Dead Load and Live Load at the Considered Section
ΔM_{cr}	Moment in Excess of Self Weight Moment
v_{cw}	Web Shear Cracking Resistance
v_{ci}	Shear Resistance of Concrete
Δi_{Fi}	Initial Deflection due to Prestressing Force at the Time of Transfer
f'_{ci}	Initial Compressive Strength of Concrete
E_{ci}	Initial Modulus of Elasticity of Concrete
I_g	Gross Moment of Inertia of the Cross Section
Δi_G	Deflection due to Self Weight at Transfer
Δ_{LL}	Instantaneous Deflection due to Superimposed Dead Load (Lane Load)
$\Delta i_{transfer}$	Total Deflection at the Time of Transfer
Δ_{TL}	Deflection due to Live Load (Tandem Loading)
Δ_{add}	Additional Long Term Deflection

γ_e	Exposure Factor
M_{PPC}	Moment due to the PPC Slab
M_{CIP}	Moment due to the Cast in Place Slab
M_a	Sum of the Moments of PPC Slab and Cast in Place Slab
M_b	Sum of the Moments due to Lane and Tandem Loading
v_{nh}	Nominal Horizontal Shear Strength
V_u	Shear Force at Ultimate
V_{uh}	Shear Stress at the Interface between the Slab and the Topping
A_{vf}	Amount of Reinforcement needed along the Interface
H_u	Horizontal Shear Force along the Interface
L_{inc}	Length of the Inclined Portion of the Shear Key
P_u	Factored Wheel Load Including Dynamic Allowance in Transverse Direction
C	Cohesion Strength of the Grout Material
L_j	Length of the Joint
μ	Friction Coefficient of the Grout Material
A_{st}	Longitudinal Reinforcement along the Shear Interface

f_y	Yield Strength of the Reinforcement
FT	Future Year (FY) AADT
CT	Current Year (CY) AADT

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1. INTRODUCTION

Bridge approach slabs (BAS) are transition slabs which connect the roadway with the bridge. At the bridge end, the approach slab is supported on the abutment which in turn rests on pile foundations. At the roadway end, the slab is supported on the sleeper slab which rests on an embankment. As a result of these different support conditions, under service loading the slab deflects more on the roadway side as the embankment support is relatively weaker than the abutment.

Solving the change in gradient (bump) at the bridge due to the differential settlement of the slab directly was not the focus of the present study. The settlement is a multi faceted problem (structural, geotechnical and hydrological) which cannot be solved by structural engineering techniques alone. Using effective backfill materials and proper backfilling procedures along with designing good drainage is necessary for a comprehensive solution to the bump at the bridge due to BAS settlement.

1.1 Differential settlement of bridge approach slabs:

The differential settlement results in abrupt slope changes causing discomfort to the users of the facility. It also causes cracks in the slab causing the water to penetrate through the slab, further aggravating potential washout of the soil support. Consolidation of foundation soils was found to further accentuate the settlement. Lateral movement of the bridge abutment and embankment settlement are the primary reasons for the faulting of approach slabs. The effects of lateral movement are more severe in integral abutment bridges.

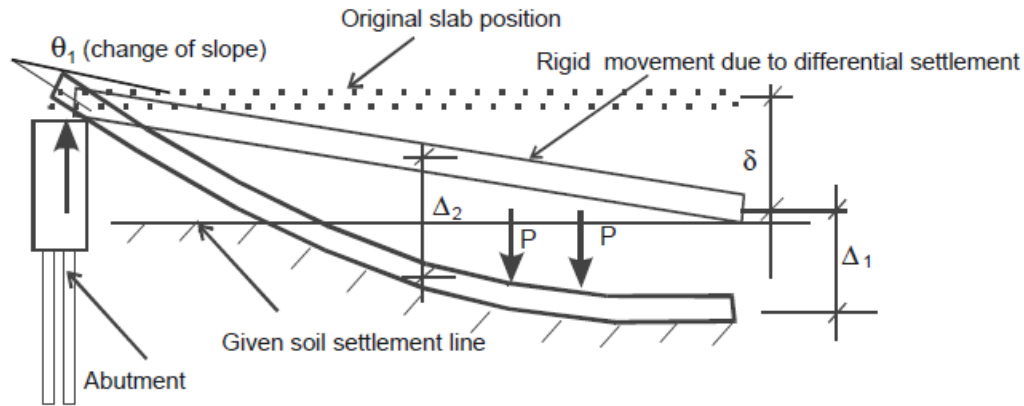


Fig. 1.1. Slab interaction with embankment soil (δ -differential settlement, Δ_1 -settlement due to loading, Δ_2 -slab deflection due to loading, P -loading on the slab, θ -rotation angle) (Cai, Shi, Voyiadjis 2005).

A prestressed slab was expected to perform better under given circumstances as the prestressing force arrests the crack growth preventing any additional loss in the slabs performance.

This thesis is a part of a research study with Missouri Department of Transportation (MoDOT) titled “Bridge approach slabs for Missouri DOT: Looking at alternative and cost efficient approaches”. The primary objective of the thesis was the same as the primary objective of the MoDOT project which is to design a technically viable and cost effective bridge approach slab. The design solutions presented in the thesis include significant additional details beyond that included in the final MoDOT report.

The primary objective of the present thesis was to provide an economic design which effectively reduces crack growth and increases its service life. The secondary objective was to design a rapid replacement alternative to the presently used approach slab. A cost

effective alternative is advantageous as the amount of money saved by the agency can be utilized in improving geotechnical and hydrological aspects of the approach slab thereby mitigating the “bump” at the end of the BAS.

After studying bridge approach slab practices implemented by various state DOT's and considering prior experience on working with precast prestressed slab systems, a prestressed bridge approach slab was suggested as a design alternative to the presently used doubly reinforced concrete slab. As the slab is precast it can be used as a rapid replacement alternative and the prestressing effect in the slab is assumed to improve its service life. Two precast prestressed design alternatives were presented which include a 12'' deep fully precast prestressed slab and a 10'' deep precast prestressed slab with a 2'' cast-in-place topping. The slabs were designed in accordance with AASHTO and MoDOT bridge design manuals and were checked for appropriate serviceability and strength limit states.

A life cycle cost analysis was conducted as part of the study to find out the most economic option of the suggested design alternatives. The presently used Standard MoDOT slab and an approach slab incorporating elastic soil support (BAS-ES) designs a part of the MoDOT project are also included in the life cycle cost calculations. RealCost, an FHWA developed software is used to calculate the life cycle costs of the four design alternatives. Agency as well as user costs are calculated for both urban and rural traffic patterns. A risk analysis is conducted to account for the uncertainty in the inputs and the discount rate effect on the overall life cycle costs is also studied. This is the first known

application of RealCost software for life cycle cost calculations for bridge approach slabs.

1.2 Organization of thesis:

Chapter 1 introduces the differential settlement problem associated with bridge approach slabs and the suggested precast prestressed concrete slab alternatives. It also presents the primary objectives of the thesis. Chapter 2 presents the literature review, a review of bridge approach slab practices by different state DOT's and a summary of different engineering problems leading to bridge approach failure. It also presents basic information relating to life cycle cost analysis (LCCA), software used to complete the LCCA procedure and various rehabilitation procedures used in the current analysis.

Chapter 3 presents the design details of the bridge approach slab used by Missouri Department of Transportation (MoDOT) and a bridge approach slab designed using elastic soil support. Chapter 4 discusses precast prestressed bridge approach slabs, this chapter also outlines the advantages of using precast prestressed slabs and the two different precast options suggested improving the performance and service life of approach slabs.

Chapter 5 discusses the analysis procedures, inputs and results from the life cycle cost analysis for both urban and rural traffic. Both deterministic as well as probabilistic analysis results are presented and compared in this chapter. The results are discussed in detail and are presented in the form of cumulative and mean distributions plots of user and agency costs along with expenditure stream diagrams. Sensitivity analysis results are

presented in this chapter in the form of correlation coefficient plots also known as tornado graphs. Discount rate effect on the total costs is also presented in this chapter.

Chapter 6, 7 and 8 present the conclusions and references.

2. LITERATURE REVIEW

This chapter reviews and summarizes previously conducted research related to approach slab problems and precast prestressed panel systems. It presents an overview of the structural, geotechnical and hydrological issues encountered during the lifetime of a typical bridge approach slab. Studies by different state DOT's were presented with primary emphasis on a project done by the Missouri department of transportation (MoDOT) on bridge approach slabs titled "Bridge approach slabs for MoDOT-Looking at alternative and cost effective approaches". An introduction to life cycle analysis (LCCA) is presented in which basic terms and definitions involved in a typical LCCA procedure were discussed. FHWA developed RealCost software, influence of discount rate on costs and limitations of the cost analysis procedure. Information about various rehabilitation activities that could be carried out during the lifetime of the approach slab was also presented.

2.1. MoDOT PROJECT ON BRIDGE APPROACH SLABS

The primary goal of the MoDOT project titled "Bridge approach slabs for MoDOT-Looking at alternative and cost effective approaches" is to design an effective and economic bridge approach slab systems. The project evaluated current approach slab conditions in Missouri and gathered additional data through surveys from Iowa, New Jersey, Nebraska, Louisiana and other state DOT's. It looked at alternatives to the existing approach slabs like cast in place approach slabs with expansion joint at the abutment (non integral and integral) and precast prestressed approach slabs. A parametric

study is conducted to study the effect of span length, slab thickness, concrete strength and end condition variations. The study also examined possible alternatives to existing approach slabs that needed replacement.

The MoDOT currently uses two approach slab types, a 25' span 12'' thick slab resting on abutment at the bridge end and connected to a sleeper slab at the pavement end (Standard BAS). The other referred to as Modified BAS has approximately half the reinforcing steel but similar geometry as the Standard BAS. A beam on elastic foundation approach and a three dimensional finite element study were used in the design process. The procedure did not consider lane load in combination with the truck or tandem load and therefore cuts down almost 22% of the total cost. For new construction operations two cast in place designs are recommended with different reinforcements both 20' long and 12'' deep were recommended. The study showed that design moments of the slabs can be significantly reduced even if the slab was assumed to be soil supported 50% of its span by poor soil with a subgrade reaction of at least 30 psi/in.

The BAS-ES alternative is designed considering elastic soil support under the slab. The design moment is calculated considering half the span to be supported by poor soil. The elastic support design saved up to 30% of initial construction costs compared to the Standard MoDOT BAS. It has a cost advantage over other alternatives as it has lesser amounts of steel. The design also eliminates the sleeper slabs at the embankment. For approach slabs requiring complete replacements, a precast prestressed slab with

transverse ties has been recommended. A 20' span design for new construction and a 25' span design for replacement operations along with sleeper slabs have been proposed.

RealCost, an FHWA developed software is used to calculate life cycle costs for all the design alternatives. The results showed that when only the present values of agency costs are considered, BAS – Elastic Soil Support design offers the lowest cost option of the four alternates studied. When only present value of user costs are considered, PCPS – BAS offers the lowest cost option of the three alternates studied. When present value of total costs are considered, the BAS – Elastic Soil Support design is the most cost-effective when AADT counts are low. When present value of total costs are considered, the PCPS - BAS design is the most cost-effective when AADT counts are high. The shorter span design recommended in this investigation (BAS – 20' Span Design) falls between BAS ES and PCPS BAS designs. All the three suggested design alternatives are over 20% less expensive than the current MoDOT used approach slab which costs an approximate \$55,500.

The geotechnical side of the problem is solved using controlled low strength materials (CLSM) as an alternative to compacted soils. Preliminary studies showed that the low strength mixtures are capable in solving backfill issues related to approach slabs.

2.2. Projects undertaken by state DOT's using precast prestressed systems:

The IOWA DOT's demonstration project to highlight the importance of precast prestressed panel construction began on Aug 2006 on highway 60 near Sheldon, Iowa.

The two way post tensioned partial width precast panels used are 14 ft x 20 ft x 12 in installed over crushed aggregate base graded to crown. The integral abutment approach slab is 77ft at either end of a skewed bridge. The project showed that precast prestressed concrete pavements can be effectively used for rapid replacement rehabilitations of bridge approach slabs. One of the recommendations of this study is to implement this construction technique into standard practice which will improve the understanding on additional factors that will come into play such as staged construction, panel installations etc...

Caltrans used PPCP systems in a pavement widening project on I-10 to reduce traffic congestion. It was constructed on April 2004 in El Monte California. The 8ft x 37 ft and 10-13 in thick panels are constructed during night time. The project eliminates the construction impact on traffic by carrying out the operations during non peak hours.

“Performance evaluation of precast prestressed concrete pavements” a MoDOT project using precast prestressed slab systems focused on evaluating the performance of innovative precast prestressed concrete pavement (PPCP) system under severe weather and traffic conditions. It concentrated on panel fabrication techniques, hydration and early age performance, pre tensioning transfer and post tensioning operations. The study used an innovative PPCP system to rehabilitate a 1,000 ft. section of interstate highway located on northbound lanes of I-57. Even though similar technology was implemented by other state DOT’s, this study quantified the pavement performance through instrumented pavement panels. The panels were installed with strain gage instrumented rebars, vibrating wire gages, strandmeters and thermocouples. Overall performance of individual panels and the interaction between the panels especially under traffic loads and

seasonal thermal effects was studied. Besides monitoring the pavement performance, the project also demonstrated the effectiveness of remote service monitoring capability of the data acquisition systems. The project made several suggestions on improving the fabrication and construction processes to improve the pavement performance. A time step model was developed which accurately predicted prestress losses due to creep and shrinkage.

2.3. Precast prestressed slab systems:

A prestressed system has most of its concrete area in compression and is effective in resisting the service loads. The prestressed concrete members are subjected to high stresses during initial prestressing and are pretested before subjected to service loading.

Composite construction is very commonly used in bridge as well as residential construction. The precast slab is designed to carry the weight of the cast in place topping before it hardens and starts acting as a composite section. The precast slab therefore has to be checked for additional stresses caused due to the dead weight of the cast in place topping. Providing shear reinforcement between the cast-in-place topping and the precast slab resists the horizontal shear force along the interface. The horizontal shear force criterion should be met during the design as it is important in improving the composite performance of the precast and cast in place assembly.

The interfacial shear force is governed by the depth and width of the stress block.

Depending on the compressive strengths of the cast-in-place slab concrete and the precast

concrete slab, the combination of both the slabs act as a composite section with a transformed width larger than the actual precast slab

In case the reinforcement is not provided at the interface, the time difference in casting between the precast slab and the cast-in place slab creates a moisture content differential between the slabs. The differential moisture content causes varied creep and shrinkage behavior between the slabs which build up additional stresses. These extra stresses induced by creep and shrinkage are not higher in magnitude and are therefore not considered during the design. Providing reinforcement along the interface or roughening the precast slab surface before laying the cast in place topping will reduce these differential movements if any such arise.

2.4. Advantages of using precast prestressed concrete:

In cast in place construction formwork erection and maintaining curing temperatures for the concrete to set. Increase in work zone duration leads to increase in user costs. There are two major advantages in using a precast prestressed slab for the present approach slab issue, the first being the effect of the steel prestressing force present in the slab which increases the service life and performance of the slab. The other advantage is user cost effectiveness of a precast slab. The precast prestressing alternative addresses the performance and cost effectiveness issues the presently used reinforced concrete cast in place approach slab fails to answer.

Factors like Entrapped air, moisture content and curing time play a major role in freeze-thaw and durability performance of a concrete slab. A precast slab is cast in a construction yard where the design mix, air-water content and the surrounding conditions are closely monitored and effectively controlled to optimize the durability and performance of the slab. A disadvantage in precast construction is the casting of precast slab panel joints. It can be solved by improving quality control and maintaining tolerance levels.

The prestressing force provided by steel in a prestressed slab does not prevent the cracking of the slab. It is effective in arresting the crack growth as the pre tensioned steel keeps most of the concrete in the slab in compression. The prestressing force in the slab is provided by 0.5'' dia. 270Ksi stress relieved strands (3 strands per unit feet width of the slab). As a result of the crack arresting capability, the service life of the slab is increased.

The camber formed in the concrete slab due to the initial prestressing force improves the deflection performance of the slab. (The slabs deflection was 85% lesser than the ACI limit under service loading). Previous studies showed that a prestressed slab effectively spans the voids under the slab formed due to backfill runoff compared to a normal reinforced concrete slab.

Another advantage in precast prestressed slab construction is the choice of the placement of expansion joints in the approach slab. Infiltration at the abutment is a critical approach slab issue which causes embankment fill to run off causing void formation under the slab. In a prestressed slab expansion joints can be moved away from the abutment which

lowers the risk of fill runoff. Reduced environmental impacts and work zone safety measures can be achieved through precast construction.

Life cycle cost analysis results showed that user costs constitute a major portion of total costs especially in high traffic zones. Results from the sensitivity analysis concluded that in urban and rural traffic scenarios, user costs depend mainly on work zone duration. The more the work zone duration, the higher the user costs. As prestressed slab panels are precast and are transported to the site for installation, the work zone closures are significantly lesser when compared to cast in place options.

2.5. Problems associated with bridge approach slabs:

National Co-operative Highway Research Program (NCHRP) discovered that the main causes of the differential settlement at the BAS are the settlement of natural soils under the embankment, compression of embankment fills, poor drainage behind the bridge abutment and erosion of the embankment fill.

Creepage of saturated soil causes embankment settlement and plays a major role in aggravating the approach slab distress. (Guiyu, Mingrong, Zhenming et. al.2004). Finite element analysis results indicated that approach slab settlement decreases greatly with an increase in its depth. Reinforcing the foundation surface layer is found to significantly minimize embankment settlement.

Excessive settlements are found to occur in many pile supported bridge approach slabs constructed in Southern Louisiana. The system consists of piles of variable length driven

at uniform spacing along the span. They are designed in a way that the resulting deflection profile of the approach slab would offer a smooth and gradual transition between the bridge and the roadway. Profiler tests, geodectic surveys and International roughness index tests showed that the inconsistent performance of a pile supported bridge approach slab is due to difference in drag load and site conditions. Negative skin friction should be considered during design to mitigate excessive settlement. (Bakeer, Shutt, Zhong et. al. 2005)

Approach slab settlement is caused due to embankment foundation problems especially in areas containing compressible cohesive soils. Settlement problems arise when approach embankments are constructed with soft cohesive soils. When constructed on such soil conditions without adequate amount of reinforcement, the slab cannot resist unsupported length caused due to fill washouts. The slabs failure to support unsupported length leads to cracking or complete failure of the slab. Estimating the amount of unsupported length is a difficult problem in approach slab design. The sleeper slab in a way allows the settlement of approach slab along with the embankment preventing the bump severity at the bridge. In addition to embankment foundation problems, longitudinal pavement growth due to temperature variations also aggravates approach slab distress. (Dupont, Allen, 2002)

Embankment depth is another important factor affecting approach slab settlement. Higher magnitudes of settlement are observed in higher embankments. The study found that differential movement of the slab depends on surrounding soil conditions and a differential settlement of 13 mm is found to require maintenance. (Long, Olson, Stark et. al.2006)

The differential movement is effected by backfilling procedures and the materials used for backfilling. Texas department of transportation (TxDOT) reported backfill losses and consequent cracking in older mechanically stabilized earth retaining structures. The backfill loss was found to be due to water infiltration through joints in the embankment. Ground penetration radar (GPR) and dynamic cone penetrometer (DCP) tests showed that the cracking of the approach slab is due to the use of siliceous gravel aggregate which has a high thermal coefficient. (Chen, Nazarian, Bilyeu, et.al. 2007)

A study by White, Mekkawy, Sriharan, et. al. (2007) focused the effect of paving notch on approach slab settlement. Improperly cleaned pavement notches were found to accentuate approach settlement. A square shaped abutment improves backfill compaction and reduces difficulty in construction. Open graded porous backfill increases drainage performance while clean crushed aggregate improves shear strength. The study suggested under sealing the approach slab by pressure grouting to prevent backfill erosion

Another Texas department of transportation (TxDOT) project investigated on the bump issue and expansion joint problems. The department spends \$7 million dollars annually on approach slab related issues. Settlement at expansion joints was observed in approach slabs constructed using articulation at mid span and wide flange terminal anchorage system. ABAQUS, a finite element analysis software was used to investigate into the problem along with BEST (Bridge to Embankments Simulator of Transition), a bump simulation device. The study concluded the following

- A vertically rigid abutment creates a major difference in settlement between the abutment and the embankment

- One span approaches give smaller bump than two span approaches
- A new backfilling procedure was suggested in which controlled quality backfill is provided within 100ft of the abutment. The fills should be compacted to 95% of modified proctor. In case such a backfill is not possible, the embankment fill within the 100ft zone should be cemented to achieve a smooth transition.

In order to investigate the performance of approach slabs in different soil conditions, a 3D non linear finite element analysis is conducted by Roy, Thiagarajan (2007). The study considered the interaction between the approach slab and embankment soil by incorporating structural and geotechnical factors. Influences of different soil conditions were studied and the approach slab was modeled to be pin connected to the abutment. The results showed that the slab thickness and void development significantly influence the approach slabs performance. Even though a thicker slab improves the slab performance, an effective thickness has to be selected during design considering the project economy and load on the embankment.

In a study by Chai, Chen, Hung et. al. (2009), the effect of washout length on the serviceability of the approach slab is studied for different washout conditions. Approach slabs are typically anchored to the abutment with dowels or threaded rods preventing relative displacement. Steel specimens, double later fiber reinforced polymers and glass fiber polymer rebar were tested for washout conditions ranging from 0-16 ft. The 6ft washout condition showed a significant reduction in stiffness. The stiffness reduction was higher for steel reinforcement and lower for FRP rebars. As stiffness reduction affects serviceability of the slab, a washout length of 6ft can be considered as a threshold limit

for maintenance purposes. The study also found that a 12'' deep approach slab avoided punching shear failure.

Parametric studies by (Cai, Shi, Voyiadjis, et.al. 2005) showed that settlement no longer affects the slabs performance when the slab loses its soil contact. The study found that vertical soil stress under the sleeper slab increases along with differential settlement even after the loss of contact between the slab and the soil. The study concluded that a rigid approach slab decreases the gradient but increases the local soil pressure beneath the slab.

An analytical vehicle-bridge model was developed to model the dynamic behavior of bridges by Shi, Cai, Chen (2008). The dynamic response of bridge approach slabs depends on the type of bridge, vehicle speed and its characteristics. Uneven approach conditions were found to aggravate the dynamic response of the bridge. The study showed that the vehicle speed affects the dynamic performance of a bridge and a critical speed of 55 m/s was found to initiate resonance. Faulting (gradient change) of the approach slab was found to affect the dynamic performance of short span bridges more than long spans.

2.6. Life cycle cost analysis:

The National highway system designation act of 1995 imposed a requirement making LCCA compulsory for National highway system (NHS) projects costing more than \$25 million (Chan, Keoleian, Gabler et al. 2008). Among 80% of states use LCCA in their pavement selection process only 40% of them incorporate user costs. However, user costs constitute a significant part of the total life cycle costs especially in the urban scenario.

Cost estimation is an important phase in a LCCA process, a study by Flyvbjerg (2002) showed that in 90% of the highway projects. The total costs are always underestimated. The report suggests a before and after analysis of the construction as well as maintenance costs will improve the cost estimating process for any future projects. An MDOT study showed that LCCA was able to predict the pavement alternative with lower initial construction costs, but the actual costs of each alternative were over estimated by more than 10% in most cases. While the actual occurrence of activities on the pavements roughly followed the estimated schedules, the actual procedures carried out were different from the estimates.

According to Molenaar et al. 2005 cost estimates will be composed of three types of information, known and quantifiable costs, known but not quantified costs and unrecognized costs. Identifying the risks/uncertainties is often difficult and even harder to quantify. A risk analysis should consist of an initial quantification and a detailed quantification to filter out minor or inconsequential risks. An order of magnitude assessment and likelihood for all events should be accounted during a risk analysis. A Washington state department of transportation (WSDOT) report showed that risks that have the greatest impact on project costs include market conditions, ROW acquisition problems and change in seismic criteria. The risks that had the largest influence on project schedules include national environmental protection act (NEPA)/404 merger processes, changes in permitting, environmental impact statement and ROW value and impact.

In the past, Point estimates are used instead of risk analysis and work zone costs are excluded from total costs simply because they are hard to calculate. Advancement of LCCA can be attributed mostly to the advancement in computing technology which made the most complicated algorithms readily solvable in a short time. Notable consideration should be given to the federal government for encouraging, mandating and guiding the application of LCCA in the evaluation of transportation system (Ozbay, Jawad, Parker, Hussain, .et al. 2004)

The period over which life cycle costs are calculated is termed as analysis period. Even though FHWA suggests a minimum analysis period of 35 years, the Michigan department of transportation (MDOT) in 2005 used an analysis period of 25 years for its pavement life cycle cost analysis.

It should be noted that LCCA is a subset of Benefit-Cost Analysis (BCA). While BCA compares costs and benefits and can address comparison of alternatives with dissimilar benefits, the LCCA compares only costs and assumes equivalent benefits for all options. The LCCA approach is ideally suited for the comparison of various design alternatives of the BAS and their long-term rehabilitation activities.

There are various parameters agencies look at while comparing the design alternatives. Some of the parameters used while conducting a life cycle cost analysis are net present value (NPV), equivalent uniform annual cost (EUAC), useful life and ratio of total life cycle cost (TLCC) to initial cost (IC). In our study the net present value is considered as a parameter for comparing life cycle costs as the analysis period is uniform for all the selected alternatives.

2.6.1. Important terms and definitions:

Analysis Period: It is the period of time during which the initial costs, rehabilitation costs and the maintenance costs are evaluated and compared between various alternatives. This period is common for all the design alternatives. An analysis period of 40 years was chosen in the present study.

Discount rate: Costs cannot be compared if they occur at different times and have to be adjusted to the opportunity value of time. The discount rate is understood as an economic return (interest) on the funds when they are utilized in the next best alternative. As suggested by MoDOT a discount rate of 7% is used in the analysis of all basic cases. Discount rates of 4% and 10% are also used to establish the effect of discount rates assumed on project costs. Real Cost recommends use of a discount rate of 4%.

Net present value (NPV): It is the value of benefits minus costs. NPV is used as a measure in this project to determine the most effective design alternative. As the effectiveness of various alternatives are compared, benefits of the project are almost similar for all the alternatives.

$$NPV = \text{Initial cost} + \sum_{k=1}^N \text{Rehab Cost}_k \left[\frac{1}{(1+i)^{n_k}} \right] \quad 2.1$$

Where 'i' is the discount rate and 'n' is the year of expenditure

The net present value takes time value of money into account but is not usable for alternatives with different service lives. It is a fair indicator of the economic effectiveness of an alternative as the analysis period is equal for all the selected design alternatives.

Deterministic analysis: In this approach, each LCCA input variable like initial construction cost, service life, rehab cost and discount rate are assigned a fixed value. The inputs used here are assumptions based on information provided by MoDOT, FHWA manual and professional judgment.

Probabilistic analysis: In this study, a normal distribution was chosen (with a default standard deviation of $1/6^{\text{th}}$ of the deterministic parameter). The inputs used in the risk analysis are identified by a small ellipsis button on the right of the data field in RealCost. Should one choose to perform probabilistic simulation, these features can be engaged. As the inputs used in the calculation of net present value are uncertain, a probabilistic analysis is conducted in which random input values are generated and the net present values are calculated. Each iteration in the risk analysis is similar to a real time scenario. Monte Carlo simulation is used in RealCost to generate normal probability distributions for the input variables used in risk analysis

Sensitivity analysis: As there are several inputs involved in the calculation of user costs, understanding and optimizing the inputs which significantly influence the final cost is necessary to achieve an economical outcome. A sensitivity analysis is conducted to find out the inputs that significantly affect the final outputs. Correlation coefficient plots also

known as tornado graphs are plotted and the inputs that influence the user costs and agency costs are studied individually. By focusing the engineering efforts on the mitigation of the most sensitive risks, there is a higher probability of completing the projects successfully.

2.6.2. Rehabilitation activities:

URETEK method: The URETEK method was invented in Finland in 1980 and has been used in the US since 1985. High density polymer is injected for lifting the concrete slab which also stabilizes the soil. In this method grout is injected under pressure beneath the slab. Holes of about 5/8'' are drilled in the slabs for every 1.2 m to the base soil and the grout is injected in to the holes. The polymer is injected first to shallower locations (3'-6') and then to deeper locations (7'-30'). URETEK uses expanding polyurethane foam as an injecting material. Polyurethane grout expands 25 times its liquid volume stabilizing and tightening the weak soils. It also increases the load bearing capacity of the soil. The density of the injected polyurethane material depends on the depth of the injection process. URETEK method has an advantage over mud jacking as the injected polyurethane exhibits ductile nature under pavement flexure. The moments of the slab are precisely monitored and controlled by laser level measuring devices on the surface. This method can be used to stabilize low density compressible soils to depths of more than 30' and can lift the slab with an accuracy of 0.1''.

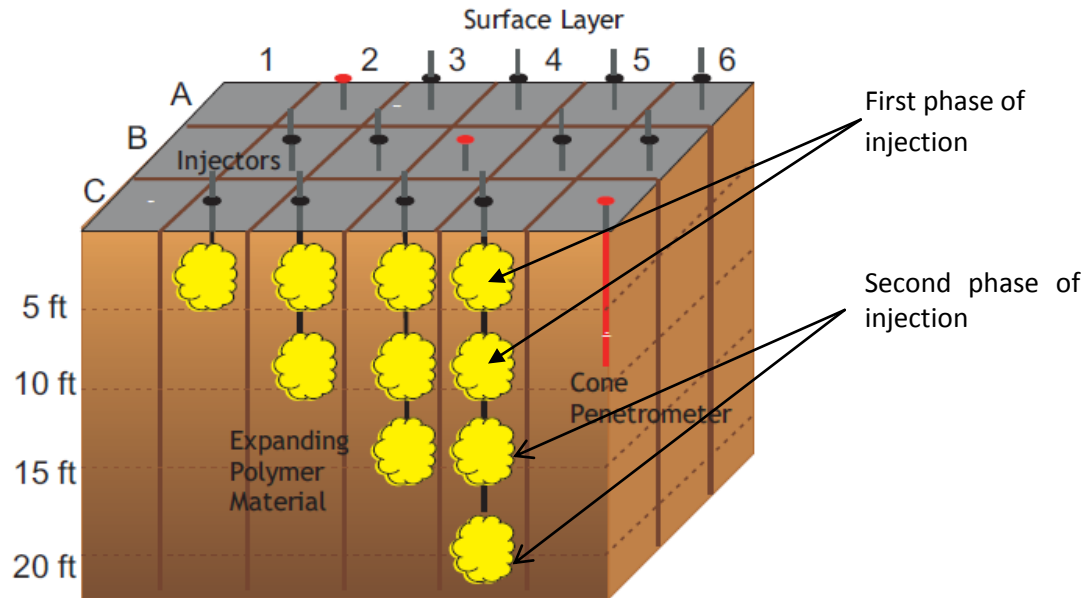


Fig.2.1. Figure showing stages in a typical URETEK process

Mudjacking: Concrete mudjacking is a process in which a concrete grout is injected below sunken concrete slabs in order to raise them back to their original height. The grout fills the voids beneath the slab which pressurizes and hydraulically lifts the slab back to its original position. Holes of 1-5/8" diameter at a center to center distance of 5' are drilled in the concrete slab and an organic or inorganic grout material mixture is pumped under the slab using a two piston pump at a pressure of 500-1,000 psi. The fill holes are then sealed with a water tight material to prevent the swelling of the cement patch. The fill holes are then patched with a 3:1 sand cement mixture and troweled to match the existing surface. The Standard MoDOT BAS is traditionally provided with mudjacking holes during initial construction for later use. This is done to avoid severing of reinforcing steel layers during coring operations.

Table 2.1. Table comparing mud jacking and URETEK processes:

FEATURE	POLYURETHANE	GROUT
Compressive strength	71 psi	80-2400 psi
Shrinkage	1% upon injection and 3% after 10 years	1% during curing and no volume change thereafter
Environmental impact	Chemical, but environmentally neutral	No impact
Effectiveness	Ductile	Not ductile
Cost (Considering that equal amounts of polyurethane and grout are needed)	Higher	Lower

Joint Sealing: Joint sealants are used to seal joints and other openings between two or more substrates. This prevents the entry of water, air and other environmental elements. The sealant is directly pumped from the original drum into the joint by use of an air powered pump. The joint sealant should fill the joint from the top of the backer rod to slightly below the pavement surface (3/8" below the pavement surface). If properly installed the sealant lasts for 5- 10 years. MoDOT uses silicone joint sealants (in preference to polysulfide sealants used by some other state DOTs).

Use of joint sealant as a rehabilitation option in the LCCA study has been studied for exclusive use with Precast Prestressed BAS design. Since this BAS design uses multiple precast slabs joined together with the use of stressed tie rods, the joint sealant can serve functionally to seal joints in the slab. This rehabilitation method has the advantage of

significantly reduced construction times as a result of which user costs are significantly lower. This rehabilitation method can provide the Precast Prestressed BAS design a significant user cost advantage, particularly in an urban setting.

Asphalt Wedging: Use of asphalt wedge is the least expensive rehabilitation option that can address the issue of bump in the BAS at the bridge abutment due to relative settlement of the two ends of the BAS. Even while the life of an asphalt wedge may be relatively small compared to the other rehabilitation methods, the extremely low initial cost offers this approach an advantage. For this LCCA study, a service life of asphalt wedge rehabilitation of 5 years is used.

2.6.3. Realcost software

It is FHWA developed software which can perform deterministic as well as risk analysis for agency costs and user costs. It can compare up to 6 alternatives at a time and uses Monte Carlo simulation to perform risk analysis and monitors the convergence after a prescribed number of iterations. The total iterations and convergence tolerance can be specified by the user. It can generate seven different types of probability distributions like normal, truncated normal, triangular, uniform, beta, geometric and log normal. Normal distribution is used to generate random values for all the inputs used in this study while performing risk analysis. The software uses Monte Carlo simulation for 2,000 iterations and RealCost monitors convergence for 50 iterations. The outputs generated from the software are available in a tabular as well as various graphical formats like tornado graphs, expenditure stream diagrams and median distributions. The software was

designed to compare competing design alternatives for a given pavement project. It however lends itself well to LCCA of bridge approach slabs as well as demonstrated in this investigation.

RealCost uses a marco within MS Excel to perform life cycle cost analysis and hence the Excel application should be executed in a macro-enabled environment. Immediately after the worksheet appears, the “Switchboard” panel opens on top of it (Fig. 2.2). Two primary level inputs are required by RealCost. Project level input includes data on the various primary analysis options, traffic data, value of user time, traffic hourly distribution and added vehicle time and cost. Alternative level input allows input of cost and service life of various rehabilitation options, agency maintenance costs, frequency, user work zone costs, and work zone inputs.

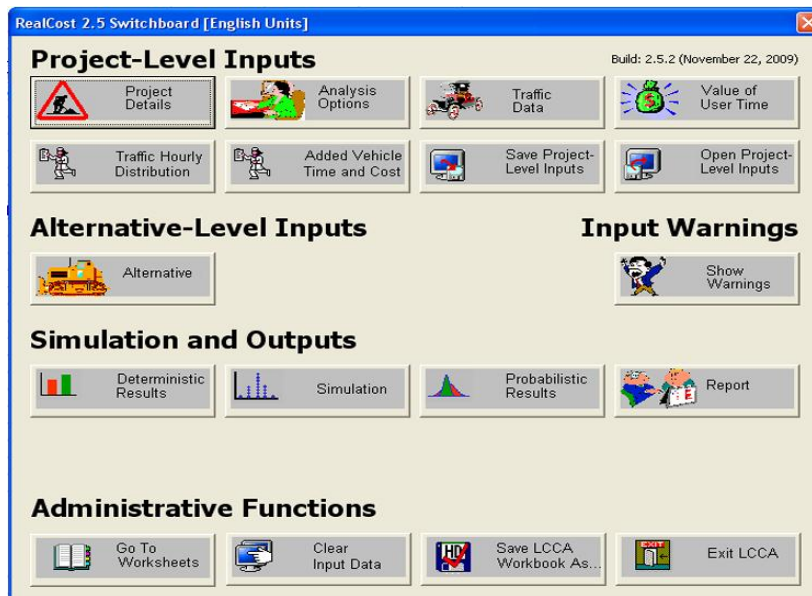


Fig 2.2. RealCost software showing the user-friendly Switchboard

The program allows one to input data either through the “Switchboard” or directly into the Input Worksheet. The next section contains details of the current project and associated inputs entered through the Switchboard. To input values directly into the Input Worksheet, the “Switchboard” interface needs to be closed by clicking the “X” in the upper right-hand corner of the window. To restore it later the drop down menu at the top of the Excel window allows selection of the “RealCost Switchboard.”

2.6.4. Monte Carlo simulation:

It is a computerized mathematical technique that allows people to account for risks in deterministic analysis. It lets you see all the possible outcomes of your decisions and assess the impact of risk, allowing for better decision making under uncertainty by showing the extreme possibilities of occurrence. (Palisade)

Monte Carlo simulation performs risk analysis by building models of probability distributions for any input that has inherent uncertainty. It calculates results each time using a different set of random values from the probability functions or it can reproduce the same set of numerical values. Depending upon the number of uncertainties and the ranges specified for them, it could simulate thousands or tens of thousands of recalculations. The number of iterations used in the present study is 2000, and convergence is monitored for every 50 iterations with a tolerance of 2.5%

During simulation the values are sampled at random from the input probability distributions. Each set of samples is called an iteration, and the resulting outcome from

that sample is recorded. It provides a comprehensive view of what may happen and shows not only what could happen but how likely it is to happen (FHWA)

The normal probability function it uses in the present study is given below

$$f(x) = \frac{1}{\sqrt{2\pi\sigma^2}} e^{-\frac{(x-\mu)^2}{2\sigma^2}} \quad 2.2$$

Where μ is the mean and σ^2 is the variance of the data

The excel based software uses the above normal probability distribution function to generate random variables for all the inputs involved in the calculation of life cycle costs. The user defines the mean and a standard deviation to describe the variation about the mean. In a normal distribution 68% of randomly generated values are within one standard deviation away from the mean. About 95% of the randomly generated values are within two standard deviations away from the mean.

2.6.5. Discount rate:

A key component of most economic analyses is the conversion of expenditures incurred at different times into equivalent constant dollar values. (Embacher, R., Snyder, M. et.al. 2001). Different consumer and industrial cost indices like Means Heavy Construction Historical cost index, discount rate etc. can be used to convert future dollars into present value. Traditional economic analyses often make use of discount rate, which is defined as the difference between the investment (or interest) rate and the inflation rate.

The 4% discount rate suggested by the FHWA is based on the yield on a 10 year treasury note. The yield on the Treasury note is caused due to a decrease in the dollar value due to inflation. After analyzing the yield on a treasury note over a period of time, a discount rate of 4% is fixed by the FHWA.

The two dollar values that can be used in the analysis are current and constant dollars.

Current dollars include the effects of inflation and deflation in the dollar value, while the constant dollars exclude the changes in the dollar value over the analysis period.

Depending on the type of dollars used, a corresponding discount rate is selected. The dollar values used in the present analysis are constant dollars and therefore a nominal discount rate of 4% is used. In a standard LCCA process, the future costs are converted into present dollar values using a quantity called discount rate.

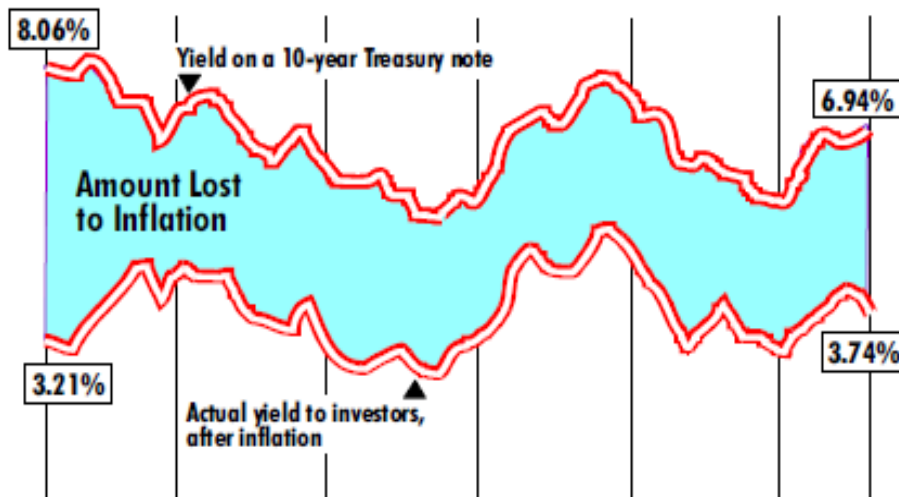


Fig. 2.3. Graph showing yielding trend on a 10-year treasury note

2.6.6. Limitations to LCCA:

According to Gluch and Baumann (2004) there are limitations on current LCCA models in the handling of environmental costs. Environmental costs are often neglected by much software and in highway projects because of the complexity in calculations. The present study did not consider the environmental costs. Highway agencies tend to omit environmental costs as they are hard to quantify and might lead to over estimation of the total costs. Current LCC models may not provide appropriate solutions due to their limitations in handling environmentally related costs. Impacts on the environment like air pollution, noise and water pollution are considered while accounting for the environmental costs. Some methods does not assign a dollar value for environmental costs, instead they compare the impact a project alternative has on the environment. As highway projects take place in different physical, legal and political environments, it is a challenge to develop a universal standard to calculate environmental costs (Goh & Yang et al. 2009)

3. Standard MoDOT BAS and BAS-ES Designs

The present design used by the Missouri Department of Transportation (MoDOT) referred to as Standard MoDOT BAS is presented in this chapter. Basic design details of one of the solutions suggested for new construction of approach slabs, an approach slab considering elastic soil support underneath the slab referred to as Bridge approach slab-Elastic support (BAS-ES) is also presented in this chapter.

3.1 Standard MoDOT BAS:

The standard MoDOT BAS is a 25' long, 12'' deep approach slab designed as a simply supported slab. It rests on the abutment on one end and the sleeper slab on the pavement end. The slab is supported on compacted fill and a layer of 4'' deep Type 5 aggregate base. A perforated pipe is provided adjacent to the sleeper slab for drainage purposes. The approach slab is connected to the end bet using reinforcement to prevent horizontal and vertical displacements.

MoDOT uses two types of approach slabs with different span lengths and different reinforcement conditions depending on the traffic conditions. Both the slab designs are based on the assumption that the slab will eventually lose soil support underneath due to settlement and fill washout and therefore are designed as simply supported slabs.

The loading conditions considered in the design include dead load, lane, truck and tandem loading. As the slab length is 25', tandem loading was proved to be critical than

the truck loading. The slab was designed for an ultimate moment of 957 kip-in and an ultimate shear of 10.66 kips. All serviceability conditions which include shear check, deflection check and AASHTO crack width checks are verified.

3.1.1 Type I: Standard MoDOT BAS:

It is used on all major routes; it is 25' long and 12 inch deep resting on the sleeper slab on the embankment side and on the abutment on the bridge side.

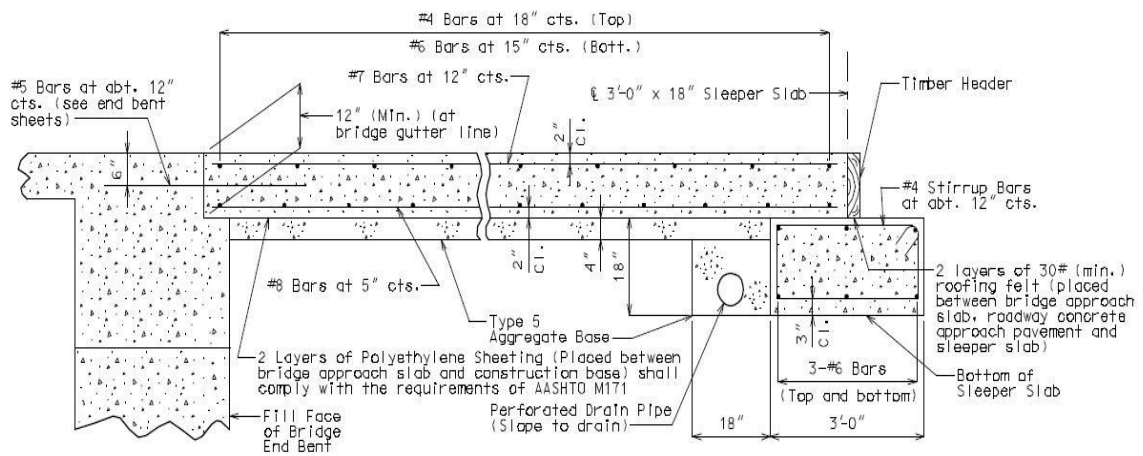


Figure 3.1. Figure showing the design detail of the Standard MoDOT BAS

The bottom longitudinal and transverse reinforcement used is #8 @ 5" c/c and #6 @ 15" c/c. The top longitudinal and transverse reinforcement used is #7 @ 12" c/c and #4 @ 18" c/c as shown in Figure 3.1

3.1.2 Type II: Modified BAS (MBAS):

It is used only on minor routes and has 50% lesser reinforcement than the standard MoDOT BAS. The bottom longitudinal and transverse reinforcement is #6 @ 6" c/c and #4 @ 12" c/c. The top longitudinal and transverse reinforcement used is 5 @ 12" c/c and

#4 @ 18" c/c. The slab span and depth are similar to the standard MoDOT BAS but does not have a sleeper slab at the pavement end. The MBAS design however does not satisfy the design moment requirements, the primary objective of this design was to reduce the construction costs. In the present design lower costs were achieved by reducing the slab length. However, the BAS-ES design approach lowers the cost by meeting the design moment requirements and is also cheaper than the MBAS.

Detailed design drawings and connection details can be found in the link:

http://www.modot.mo.gov/business/standard_drawings2/documents/apn6_sq_n.pdf

3.2 Bridge approach slab-Elastic support (BAS-ES):

In this design approach the approach slab is designed assuming the elastic soil support under the slab. The sleeper slab at the pavement end of the Standard MoDOT BAS is replaced by modified end section reinforcement; it not only reduces the construction cost by 30% but also increases flexural rigidity in transverse direction.

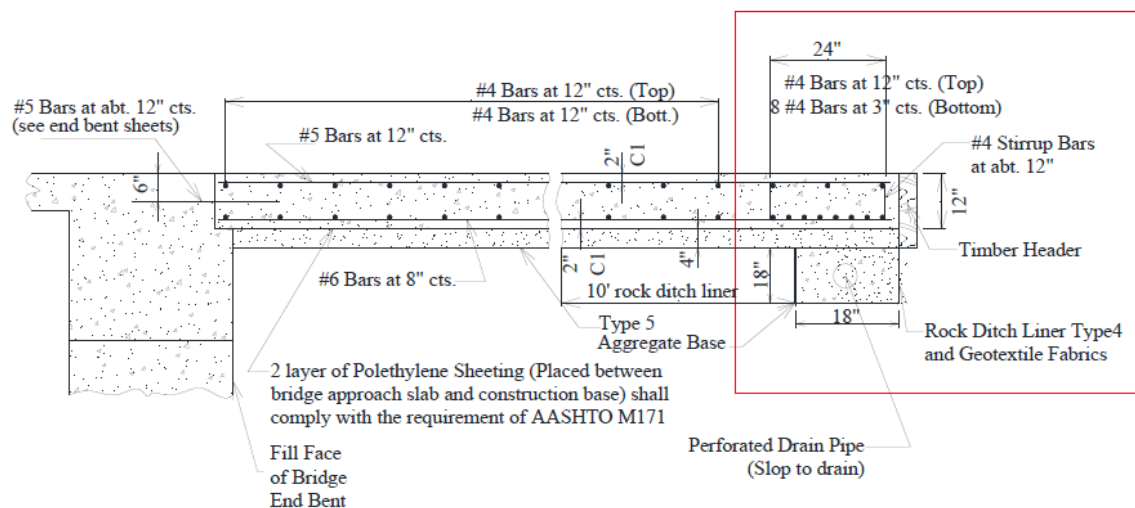


Figure 3.2. Figure showing the design details of BAS-ES

The loads considered in the design include dead load, lane load, truck load and tandem loading. The tandem loading was proved to be critical than the truck loading as the slab span is just 25'. The slab was designed for an ultimate factored moment of 196.88 kip-in and ultimate factored shear of 1.29 kips. The design was then checked for minimum reinforcement requirements, shear capacity and AASHTO crack width check.

The slab is 38' wide, 25' long and 12'' deep similar to the Standard MoDOT design. The top and bottom main steel reinforcements are #5 @ 12'' c/c and #6 @ 8'' c/c. #4 @ 12'' c/c are used as distribution steel reinforcements for both top and bottom layers. As the slab is designed assuming elastic soil support, the usage of sleeper slab is not recommended. Type 4 rock ditch liner is used to contain and confine the Type 5 aggregate ditch which holds the drainage pipe.

The highlighted portion in the above figure shows the end reinforcement detail which contains #4 @ 12'' c/c as stirrup reinforcement and eight #4 @ 3'' c/c as additional transverse reinforcement in the bottom layer. The additional end zone transverse reinforcement improves post crack stiffness and limits longitudinal crack width. The stirrup reinforcement confines the concrete improving the slab performance in transverse bending.

Detailed analysis of the BAS-ES design can be found in the study titled:

4. PRECAST PRESTRESSED BRIDGE APPROACH SLAB

The design and analysis procedure of the two bridge approach slab alternatives, a 12'' deep fully precast prestressed slab 25' long, 38' feet wide and a 10'' deep CIP Topped PC Prestressed BAS similar in length and width with a 2'' thick cast in place topping are presented in this chapter. The approach slab is constructed by installing 25' long panels with varying widths attached together using a Hollow structural steel (HSS) tube-reinforcing steel bar connection system. The slab design, panel reinforcement and panel to panel connections are also presented in detail in this chapter.

4.1 Precast prestressed bridge approach slabs

4.1.1 Fully precast prestressed BAS:

The precast slab effectively reduces the work zone duration and thus incurred additional work zone related user costs. It however has problems in maintaining tolerance levels on site and sub grade preparation issues which can be solved by using surface finishing techniques like diamond grinding.

4.1.2 CIP Topped PC Prestressed BAS:

The cast in place topped precast prestressed approach slab has improved shear transfer capacity over the fully precast prestressed BAS. It however doesn't require any surface finishing operations as the topping is applied on site and any tolerance level differences between slab panels can be leveled.

4.2 Analysis and Design

4.2.1 Loading conditions:

Three different live load combinations which include uniform load, design truck and design tandem loading are applied on the approach slab to get to the critical loading scenario. The uniform loading is a lane load of 640 lb/ft uniformly applied all over the length of the slab and 10 ft in the transverse direction. The design truck loading consists of a truck weighing a total of 72 kips, the rear axle of the truck as well as the rear axle of the tractor truck carry a weight of 32 kips. The spacing between the first two axles is 14 ft while the spacing between the second and third axles can be anywhere between 14-30 ft.

The tandem loading on the other hand is a two axle vehicle each carrying 25 kip load at a spacing of 4 ft. The spacing between the wheels on the axle is 6 ft similar to the design truck. The tandem loading is often proved to be critical than the design truck loading for spans not exceeding 40 ft. As the approach slab is 25 ft in span, the tandem loading along with a lane load of 640 plf was established to be critical over the truck loading.

The slab cross sectional properties depend on the maximum service load moment and the top and bottom fiber stresses it can resist during initial prestressing and service loading, the stresses it can resist depends on the compressive strength of the concrete mix.

Basing on the design moment and a 6 ksi concrete mix, a 12'' x 12'' cross section is considered

Using the equivalent strip method (AASHTO S 4.6.2) to uniformly distribute the lane and the tandem load over the entire slab

$$\text{Equivalent strip method} = E = (84 + 1.44\sqrt{LW}) \leq 12W/N_l \quad 4.1$$

The slab is 25' long and 38' wide

N_l = number of lanes on the approach slab = 2

Live load distribution factor = $1/E = 1/10.7$ lane/ft

Impact factor of 1.33 (33% for prestressed concrete beams) is applied to tandem axle loads to account for the dynamic response of the bridge.

The tandem loading after adding the dynamic bridge response consists of two point loads 3.1 kips each applied on a 25 ft span slab with a 4 ft distance between the point loads.

Along with the tandem loading, a lane load of 640 plf (AASHTO design lane load) is uniformly distributed over the slab.

$$\text{Ultimate moment} = M_u = 1.25(M_{DL}) + 1.75(M_{LL} + M_{TL}) = 957 \text{ kip-in} \quad 4.2$$

4.3 Fully precast prestressed BAS:

4.3.1 Feasibility Domain (Magenl diagram):

Stresses at the top and bottom fibers are calculated for the initial prestressing force and service loading for the given cross section. Magenl diagrams or feasibility domains are plotted with inverse of initial prestressing force before losses ($\frac{1}{F_i}$) on the x-axis and eccentricity of the prestressing force in the precast section on the y-axis. The advantage of this domain being, for a given cross section and a known value of eccentricity, the

minimum required prestressing force can be found, similarly with a known value of prestressing force, a corresponding eccentricity can be found which obeys all the service limit states. Any prestressing force and the corresponding points within the shaded feasibility domain satisfy all the service stress conditions.

The feasibility domain helps in deciding the amount of initial prestressing force and its eccentricity for the slab to remain uncracked under service loading and initial prestressing forces. The first two conditions (I, II) are the stresses in the top and bottom fiber under initial prestressing force. The III and IV service conditions are obtained by calculating the stresses in the top and bottom fibers under service loading (Tandem + lane loading).

I) Initial top fiber stress (tension):

$$e_o = \frac{Z_t}{A_c} + \frac{1}{F_i} (M_{\min} - \sigma_{ti} Z_t) \quad 4.3$$

II) Initial bottom fiber stress (compression):

$$e_o = -\frac{Z_b}{A_c} + \frac{1}{F_i} (M_{\min} + \sigma_{ci} Z_b) \quad 4.4$$

III) Top fiber during service loading (Compression):

$$e_o = \frac{Z_t}{A_c} + \frac{M_{\max} - \sigma_{cs} Z_t}{\eta F_i} \quad 4.5$$

IV) Bottom fiber during service loading (Tension):

$$e_o = \frac{-Z_b}{A_c} + \frac{M_{\max} + \sigma_{ts} Z_b}{\eta F_i} \quad 4.6$$

Z_b , Z_t are the section modulus values w.r.t the top and bottom fibers. A_c is the area of concrete. F_i and ηF_i are the initial and final prestressing forces. $\sigma_{ci}, \sigma_{ti}, \sigma_{cs}, \sigma_{ts}$ are the allowable stresses ('c' stands for compression, 't' stands for tension) during initial and service loading conditions. M_{min} and M_{max} are the dead load and service load moments.

An additional line denoting the maximum practical eccentricity is drawn from the y-axis to intersect the domain formed by the four stress conditions to find out the optimal prestressing force which satisfies the service limit conditions. The maximum practical eccentricity towards the bottom fiber is $y_b - \text{clear cover}$. (y_b is 6'' in our case and a clear cover of 2'' is assumed and therefore a maximum practical eccentricity of 4'' is chosen).

The highlighted region in the Magnel diagram is the feasibility domain, the point where the line (eo-4'', the maximum practical eccentricity) and the $\frac{1}{F_i}$ line intersect gives you the optimal prestressing force which satisfies all the service conditions (actual stresses < allowable stresses).

From the feasibility domain, $1/F_i = 1.14 \times 10^{-5}$ is the optimal prestressing force for an eccentricity of 4'' which satisfies all the four service stress conditions.

Three 0.5'' dia. 270 ksi strands are used per unit feet width of the slab which carry an initial prestressing force of 81 kips without losses.

The applied eccentric prestressing force is divided into symmetric and asymmetric components. The stresses in the top and bottom fiber of the slab are analyzed during initial prestress, after applying the CIP topping and during the service loading. The calculated stress values are checked with the allowable stress limits.

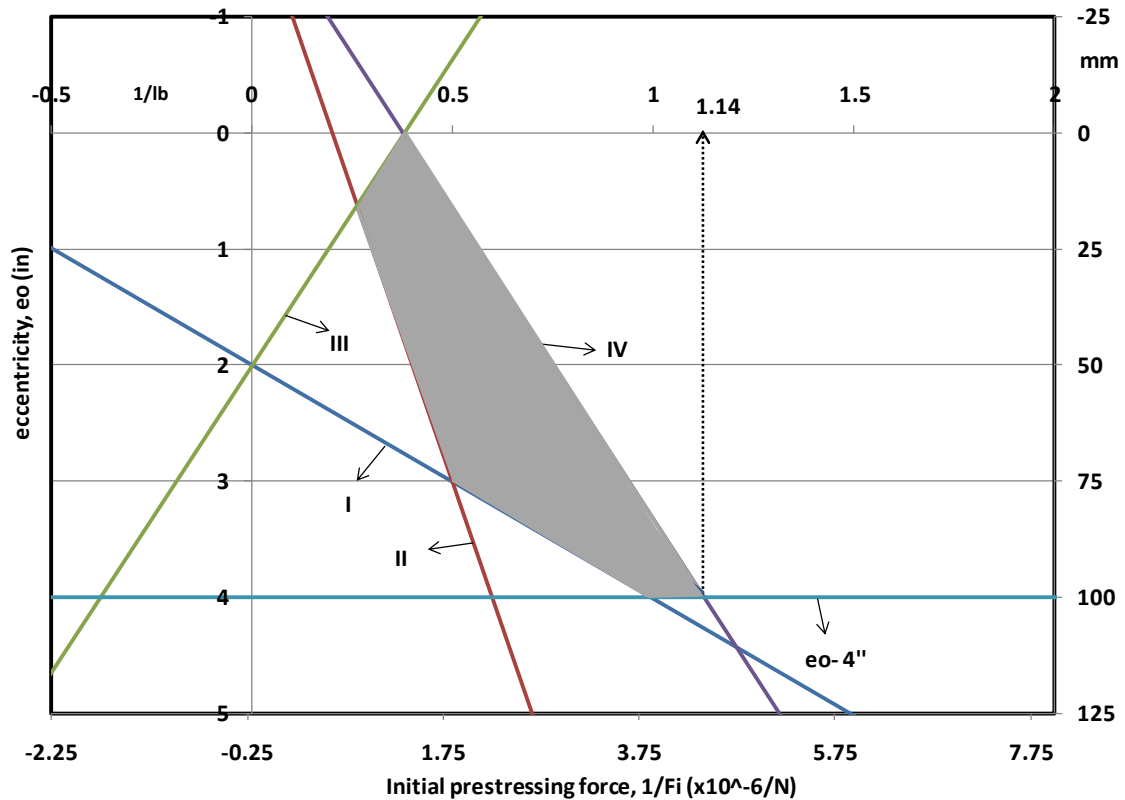


Fig. 4.1. Magenl diagram (Feasibility domain) of the fully precast prestressed slab

The slab was then checked for the stress in the bottom fiber under service loading to calculate the cracking moment. The moment for which the tensile stress in the extreme fiber of the slab reaches the modulus of rupture of concrete is defined as the cracking moment of the slab.

Stress in the bottom fiber under service loading:

$$\frac{F}{A_c} + \frac{Fe_o}{Z_b} - \frac{M_{cr}}{Z_b} = f_r = -6\sqrt{f'_c} \quad 4.7$$

f_r = modulus of rupture of concrete = $-6\sqrt{f'_c}$

M_{cr} , cracking moment which causes the stress in the bottom fiber to reach $-6\sqrt{f'_c}$

The cracking moment was checked for this condition, $\phi M_n > 1.2 M_{cr}$ 4.8

The nominal moment capacity was found to be greater than $1.2 M_{cr}$ denoting that the slab has enough prestressing force and compressive strength in the concrete that can resist the service loading without cracking.

4.3.2 Shear check:

The ACI design approach is based on ultimate strength requirements. ACI shear criterion is verified to see if the 12'' deep slab can resist shear

$$v_u \leq \phi v_n \quad 4.9$$

$v_n = v_c$ = shear strength of concrete, lesser of v_{ci} and v_{cw}

v_{ci} , which is the shear cracking stress under flexure is the sum of shear stress needed for the formation of inclined cracks ($0.6\sqrt{f'_c}$), shear stress due to the self weight of the member and the factored shear stress that causes the cracks to occur in the first place.

Shear resistance of concrete:

$$v_{ci} = 0.6\sqrt{f'_c} + \frac{V_G}{b_w d_p} + \frac{(\Delta V_u * \Delta M_{cr})}{\Delta M_u b_w d_{pc}} \geq 1.7\sqrt{f'_c} \quad 4.10$$

Width of the section (b_w) is 12'', the depth of prestressing steel (d_p) is 10.5''. Checking the shear at a section 2' from the left of the support

V_G = shear due to the own weight of the composite section = 1375 lbs

ACI factors for dead and live loads are 1.2 and 1.6 respectively

ΔV_u = factored shear force due to dead load and live load at the considered section

$$= 1.2(1.3+3.4) + 1.6(3.1) = 10.66 \text{ kips}$$

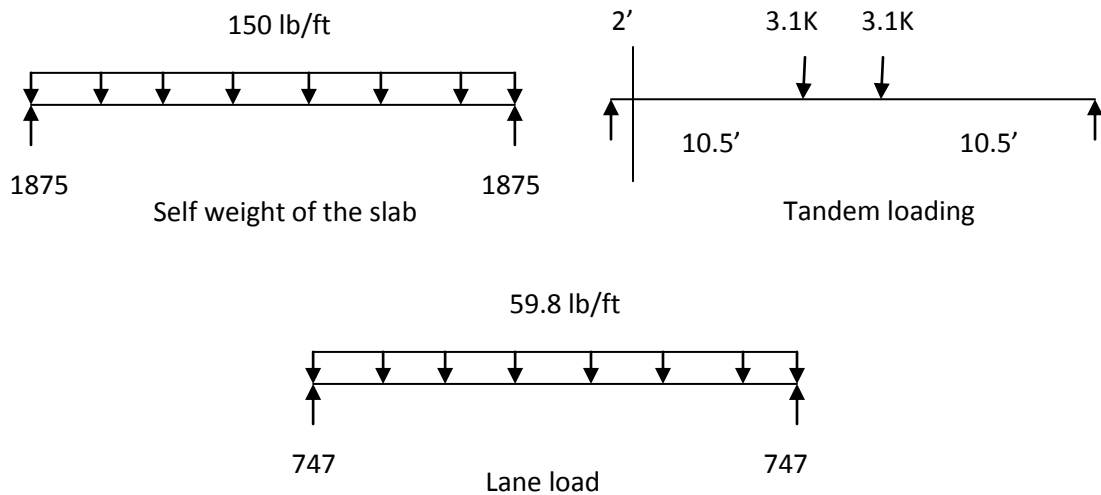


Fig. 4.2. Figure showing different loading conditions on the approach slab

ΔM_u = factored moment due to the dead load and live load at the considered section

$$= 1.2(1.3+3.4) + 1.6(6.2) = 15.56 \text{ kip-ft}$$

$\lambda = 1$ for normal weight concrete

ΔM_{cr} , moment in excess of self weight moment = 36.6 kip-ft

$$v_{ci} = 267 \text{ psi}$$

Web shear cracking resistance:

$$v_{cw} = 3.5\lambda \sqrt{f'_c} + 0.3 \frac{F}{A_c} \quad 4.11$$

F is the prestressing force after losses and A_c is the cross sectional area of the section.

$$v_{cw} = 414 \text{ psi}$$

$$v_{ci} < v_{cw} \quad 4.12$$

\therefore the shear resistance of concrete is more critical than the web shear cracking resistance

$$v_n = v_{ci} \quad 4.13$$

Ultimate shear stress

$$v_u = \frac{V_u}{bd_p} = 65 \text{ psi} \quad 4.14$$

$$\phi v_n = 0.75 * 267 = 200 \text{ psi}$$

$$\phi v_n > v_u \quad 4.15$$

\therefore the slab is safe in vertical shear

As approach slabs are subjected to differential settlement, slab lifting rehabilitation

activities like URETEK and mudjacking are used to lift the slab to its original unsettled

position. Upward forces are exerted on the slab during the slab lifting process which causes to building up of negative moments in the slab. The amount of reinforcement needed to counteract the negative moments for the same stress level (for the same 'a' value) is calculated using the dead load moment of the slab.

#3 @ 12'' is used as compression reinforcement to resist the negative moments on the slab.

4.3.3 Deflection check:

Due to the affect of the prestressed steel tendons in the concrete, a camber or a upward deflection is produced and is designated with a -ve sign, the normal downward deflection due to loading is denoted by a +ve sign.

The slab deflections like initial deflection due to prestressing force, deflection due to self weight at transfer, instantaneous dead load deflection and service loading deflections are calculated and are verified with the deflection limits.

Initial deflection due to prestressing force at the time of transfer, Δ_{iF_i}

f'_{ci} = initial compressive strength of concrete = 5000 psi

E_{ci} = initial modulus of elasticity of the concrete = $57000\sqrt{f'_{ci}}$ = 4030 ksi

F_i = Initial prestressing force = 86.4 kips

L = length of the approach slab= 25'

I_g = gross moment area of the concrete section = $12 \times 12^3 / 12$

$$\Delta_{i_{F_i}} = -0.55 \text{ in}$$

Deflection due to self weight at transfer = Δ_{i_G}

W_G = dead weight of the slab = 150 lb-ft

$$\Delta_{i_G} = 0.08 \text{ in}$$

$$\text{Total deflection at the time of transfer} = \Delta_{i_{transfer}} = \Delta_{i_G} + \Delta_{i_{F_i}} = -0.47 \text{ in} \quad 4.16$$

Instantaneous deflection due to superimposed dead load, i.e. lane load, Δ_{LL}

W = lane load = 59.8 lb/ft

E = modulus of elasticity of concrete = $57000 \sqrt{f'_c} = 5098 \text{ ksi}$

$$\Delta_{LL} = 0.03 \text{ in}$$

Deflection due to live load (Tandem loading) = Δ_{TL}

W = tandem load = 248 lb/ft

$$\Delta_{TL} = 0.13 \text{ in}$$

Heuristic rule to calculate long term deflection

$$\text{Additional long term deflection} = \Delta_{add} = 1.8\Delta_{i_{F_i}} + 2.2\Delta_{i_G} + 2\Delta_{LL} = -0.286 \text{ in} \quad 4.17$$

$$\text{ACI live load deflection limit} = \frac{L}{360} = 0.834 \text{ in}$$

$$\Delta_{TL} = 0.13 < 0.834$$

$$\Delta_{LL} + \Delta_{add} \leq \frac{L}{480} \quad 4.18$$

$$-0.156 \text{ in} < 0.625 \text{ in}$$

The slab is within the deflection limits

4.3.4 AASHTO crack width check:

The slab is also checked for AASHTO crack width, section 5.7.3.4 of the AASHTO bridge manual stated that the spacing of the reinforcement closer to the tension face should be less than

$$\frac{700\gamma_e}{\beta_s f_{ps}} - 2d_c \quad 4.19$$

$$\gamma_e = \text{exposure factor} = 1$$

Clear cover (d_c) for the slab is 2'', height of the slab (H) being 12''.

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = 1.2'' \quad 4.20$$

The calculations showed that the spacing between the strands should be less than 20'' for effective control of the crack width. The actual provided spacing between the strands was lesser than the limit. Therefore, the slab is within limits of AASHTO's crack width checks.

4.3.5 Shrinkage and temperature reinforcement:

Minimum amount of shrinkage and temperature reinforcement:

$$0.11 \frac{A_g}{f_y} = 11 * 144 / 60 = 0.264 \text{ in}^2; \text{ Use \#4 @ 9'' spacing}$$

4.3.6 Final designs:

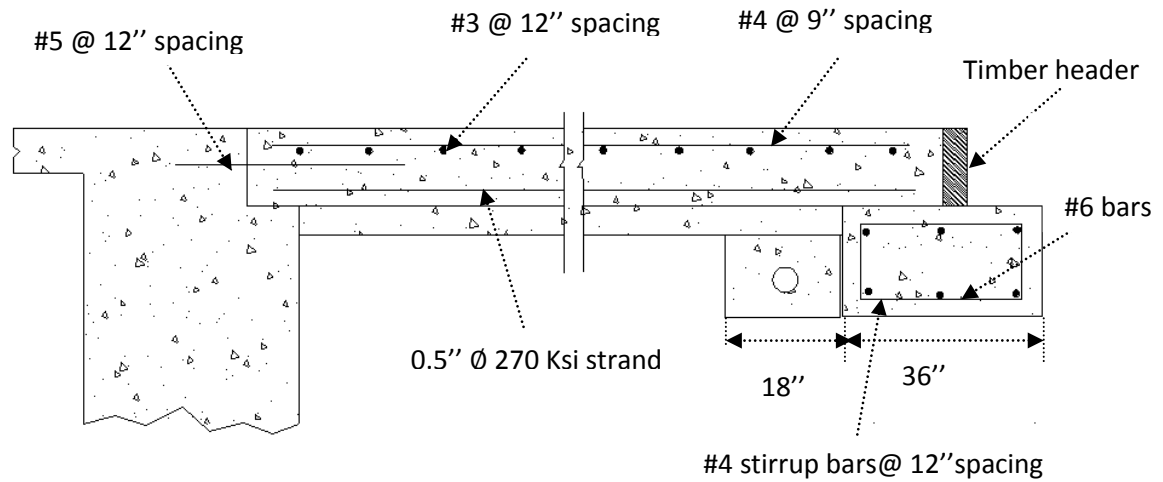


Fig. 4.3 Design details of a fully precast prestressed bridge approach slab

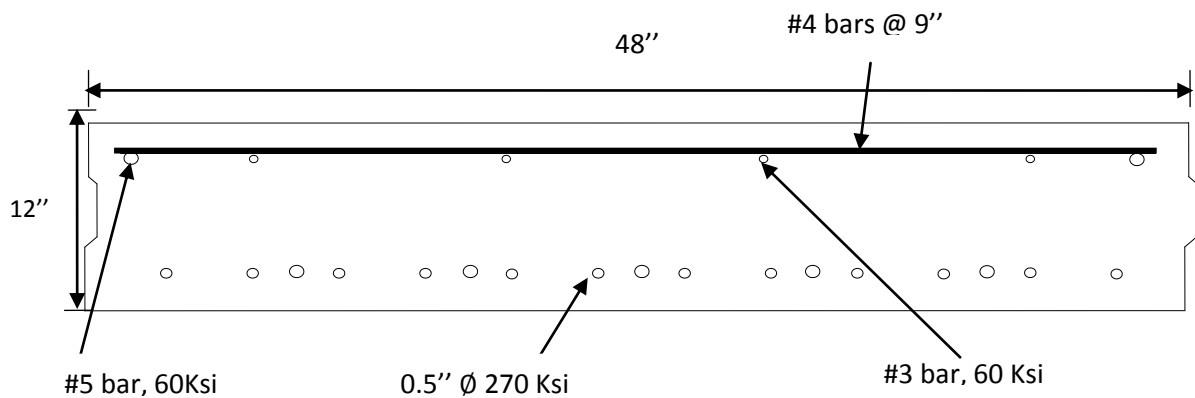


Fig. 4.4 Fully precast prestressed slab panel

4.4 CIP Topped PC Prestressed BAS:

If the concrete used in the precast and the cast-in-place slabs is of different compressive strengths, different stresses are generated in the slabs for the same amount of strain. This difference in the behavior is accounted by assuming the modulus of both the slabs to be the same. A concept called transformed section is applied commonly to the cast in place slab and it is transformed into a fictitious slab having same properties as the precast beam. The concept changes the width of the cast in place slab in order to maintain compression force for the same depth.

4.4.1 Feasibility Domain (Magnet diagram):

The stresses in the top and bottom fibers are calculated for six loading conditions and a feasibility domain is plotted. The first two stress conditions (I, II) calculate the stresses in the top and bottom fibers of the slab after the prestressing force is transferred onto the slab. Once the cast-in-place topping is applied on the slab, it is a dead weight to the precast slab until it hardens. Stress conditions III and IV are the top and bottom fiber stresses in the slab right after the cast in place topping is applied over the precast slab.

I) Initial top fiber stress (tension):

$$e_o = \frac{Z_t}{A_c} + \frac{1}{F_i} (M_{\min} - \sigma_{ti} Z_t) \quad 4.21$$

II) Initial bottom fiber stress (compression)

$$e_o = -\frac{Z_b}{A_c} + \frac{1}{F_i}(M_{\min} + \sigma_{ci}Z_b) \quad 4.22$$

III) Top fiber stress after applying the cast-in-place topping

$$e_o = \frac{Z_t}{A_c} + \frac{1}{\eta F_i}(M_T - \sigma_t Z_t) \quad 4.23$$

IV) Bottom fiber stress after applying the cast-in-place topping

$$e_o = -\frac{Z_b}{A_c} + \frac{1}{\eta F_i}(M_T + \sigma_c Z_b) \quad 4.24$$

Z_b , Z_t are the section modulus values w.r.t the top and bottom fibers. A_c is the area of concrete. F_i and ηF_i are the initial and final prestressing forces. $\sigma_{ci}, \sigma_{ti}, \sigma_{cs}, \sigma_{ts}$ are the allowable stresses ('c' stands for compression, 't' stands for tension) during initial and service loading conditions. M_{\min} and M_{\max} are the dead load and service load moments.

$$M_T = M_{PPC} + M_{CIP}$$

$$M_{PPC} = \text{Moment due to the PPC slab} = 9.76 \text{ kip-ft}$$

$$M_{CIP} = \text{Moment due to the cast-in-place slab (CIP)} = 1.95 \text{ kip-ft}$$

V) Top fiber stress during service loading (Compression):

$$e_o = \frac{Z_t}{A_c} + \frac{Z_t \left[\frac{M_a}{Z_t} + \frac{M_b}{Z_{tc}} \right] - \sigma_{cs} Z_t}{\eta F_i} \quad 4.25$$

VI) Bottom fiber stress during service loading (Tension):

$$e_o = \frac{-Z_b}{A_c} + \frac{Z_b \left[\frac{M_a}{Z_b} + \frac{M_b}{Z_{bc}} \right] + \sigma_{ts} Z_b}{\eta F_i} \quad 4.26$$

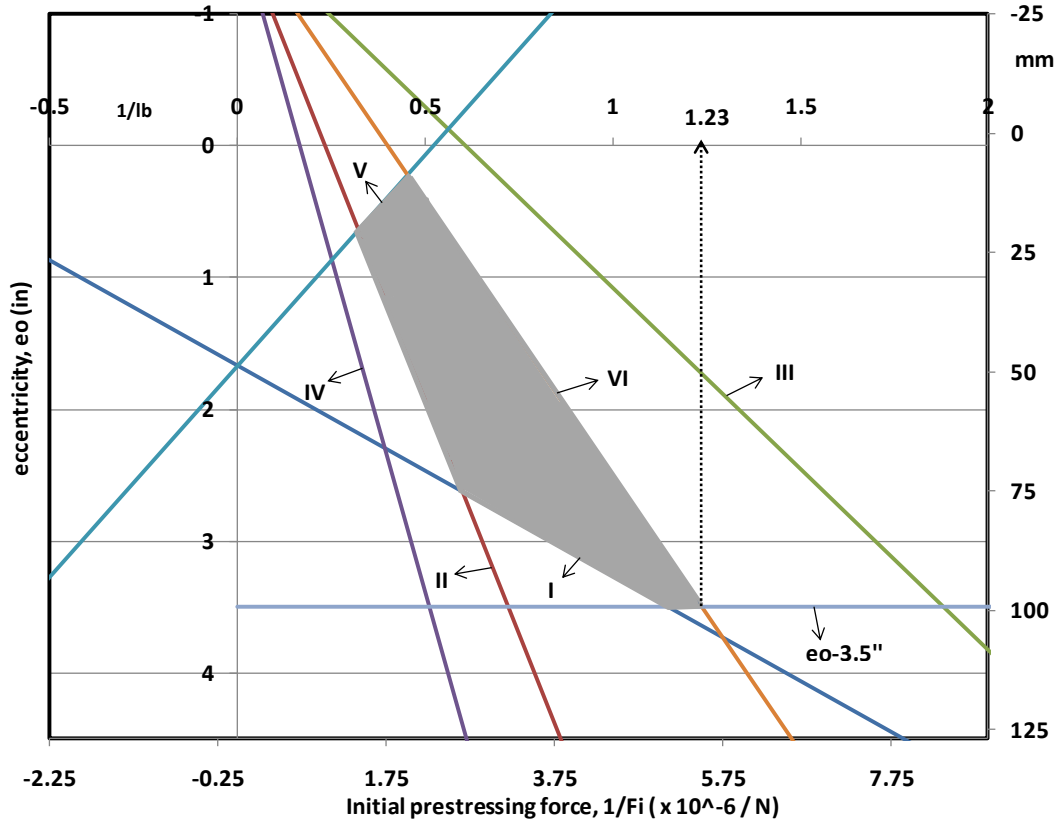


Fig. 4.5. Magnet diagram (Feasibility domain) of the CIP Topped PC Prestressed BAS

$$M_a = M_{PPC} + M_{CIP} = \text{moment due to CIP} + \text{moment due to PPC} = 11.71 \text{ kip-ft}$$

$$M_b = M_{TL} + M_{LL} = \text{moment due to tandem load} + \text{moment due to tandem load}$$

$$= 32.55 + 4.67 = 37.22 \text{ kip-ft}$$

The last two stress conditions V and VI calculate the tensile and compressive stresses under service loading (Tandem loading).

The highlighted region in the above Magnel diagram shows the point where the line (e = 3.5 in., the maximum practical eccentricity) and the $\frac{1}{F_i}$ line intersect gives you the optimal prestressing force which satisfies all the service conditions (actual stresses < allowable stresses).

From the feasibility domain, $\frac{1}{F_i} = 1.23 \times 10^{-5}$ is the optimal prestressing force which can be achieved for an eccentricity of 3.5" without causing any cracks in the slab under the service loading.

Three 0.5" dia. 270 ksi strands are used per unit feet width of the slab which carry an initial prestressing force of 81 kips. The factored nominal moment provided by the prestressing force in the slab was 970 kip-in, which was greater than the design moment. (957 kip-in)

4.4.2 Horizontal shear check:

$$v_{uh} \leq \phi v_{nh} \quad 4.27$$

v_{nh} = nominal horizontal shear strength = 80 psi (ACI value when the contact surfaces are roughened, no ties)

$$\phi = 0.75 \text{ (ACI)}$$

$$\text{Shear force at ultimate} = V_u = 7.8 \text{ kips}$$

Shear stress at the interface between the topping and the slab

$$V_{uh} = \frac{V_u}{bd_p} = \frac{7.8 \times 10^3}{12 \times 10.5} = 62 \text{ psi} \quad 4.28$$

$$\phi v_{nh} = 0.75 \times 80 = 60 \text{ psi}$$

$$v_{uh} \cong \phi v_{nh} \quad 4.29$$

4.4.3 Reinforcement along the interface:

The reinforcement at the interface depends on the horizontal shear at the interface between the precast slab and the cast in place topping

Amount of reinforcement needed along the interface (A_{vf})

$$A_{vf} = \frac{H_u}{\phi f_y \mu} \quad 4.30$$

H_u = Horizontal shear force along the interface (compressive force in the stress block)

$$= 0.85 f'_c \times a \times b \quad 4.31$$

$$= 0.85 \times 6000 \times 1.8 \times 12$$

$$= 11.016 \text{ kips}$$

μ is $1/\lambda$ if the contact surface is intentionally roughened. For normal weight concrete, λ is

1

$$A_{vf} = 0.24 \text{ in}^2$$

4.4.4 Vertical shear check:

$$v_u \leq \phi v_n \quad 4.32$$

$v_n = v_c$ = shear strength of concrete, lesser of v_{ci} and v_{cw}

Shear resistance of concrete:

$$v_{ci} = 0.6\sqrt{f'_c} + \frac{V_G}{b_w d_p} + \frac{(\Delta V_u * \Delta M_{cr})}{\Delta M_u b_w d_{pc}} \geq 1.7\sqrt{f'_c} \quad 4.33$$

Width of the section (b_w) is 12'' and the depth of prestressing steel (d_p) is 10.5''

V_G = shear due to the own weight of the composite section = 1375 lbs

ACI factors for dead and live loads; 1.2 and 1.6 respectively

ΔV_u = factored shear force due to dead load and live load at the considered section

$$= 1.2(1.3+3.4) + 1.6(3.1) = 10.66 \text{ kips}$$

ΔM_u = factored moment due to the dead load and live load at the considered section

$$= 1.2(1.3+3.4) + 1.6(6.2) = 15.56 \text{ kip-ft}$$

λ is 1 for normal weight concrete

ΔM_{cr} , cracking moment in excess of slabs self weight moment = 36.6 kip-ft

$$v_{ci} = 267 \text{ psi}$$

v_{cw} = web shear cracking resistance

$$3.5\lambda \sqrt{f'_c} + 0.3 \frac{F}{A_c} \quad 4.34$$

F is the effective prestressing force after losses and A_c is the cross sectional area of the section

$$v_{cw} = 414 \text{ psi}$$

$$v_{ci} < v_{cw} \quad 4.35$$

\therefore the shear resistance of concrete is more critical than the web shear cracking resistance

$$v_n = v_{ci} \quad 4.36$$

Ultimate shear stress:

$$v_u = \frac{V_u}{bd_p} = \frac{7.8 * 10^3}{12 * 10.5} = 65 \text{ psi} \quad 4.37$$

$$\phi v_n = 0.75 * 267 = 200 \text{ psi}$$

$$\phi v_n > v_u \quad 4.38$$

\therefore the slab is safe in vertical shear

4.4.5 Deflection check:

Similar to the fully precast prestressed slab, #3 bars are used for each 12'' section as compression reinforcement to resist the negative moments on the slab

Initial deflection due to prestressing force at the time of transfer = Δ_{iF_i}

f'_{ci} = initial compressive strength of concrete = 5000 psi

E_{ci} = initial modulus of elasticity of the concrete = $57000\sqrt{f'_{ci}}$ = 4030 ksi

F_i = Initial prestressing force = 86.4 kips

L = length of the approach slab= 25'

I_g = gross moment area of the concrete section = $12 \times 12^3/12$

$\Delta_{i_{F_i}}$ = -0.55 in

Deflection due to self weight at transfer = Δ_{i_G}

W_G = dead weight of the slab = 150 lb-ft

Δ_{i_G} = 0.08 in

Total deflection at the time of transfer = $\Delta_{i_{transfer}} = \Delta_{i_G} + \Delta_{i_{F_i}} = -0.47$ in 4.39

Instantaneous deflection due to superimposed dead load, i.e. lane load, Δ_{LL}

W = lane load = 59.8 lb/ft

E = modulus of elasticity of concrete = $57000\sqrt{f'_c}$ = 5098 ksi

Δ_{LL} = 0.03 in

Deflection due to live load (Tandem loading) = Δ_{TL}

W = tandem load = 248 lb/ft

Δ_{TL} = 0.13 in

Heuristic rule to calculate long term deflection

$$\text{Additional long term deflection} = \Delta_{add} = 1.8\Delta_{i_{F_i}} + 2.2\Delta_{i_G} + 2\Delta_{LL} = -0.286 \text{ in} \quad 4.40$$

$$\text{ACI live load deflection limit} = \frac{L}{360} = 0.834 \text{ in}$$

$$\Delta_{TL} = 0.13 < 0.834$$

$$\Delta_{LL} + \Delta_{add} \leq \frac{L}{480} \quad 4.41$$

$$-0.156 \text{ in} < 0.625 \text{ in}$$

The slab is within the deflection limits

4.4.6 Shrinkage and temperature reinforcement:

$$\text{Minimum amount of shrinkage and temperature reinforcement: } 0.11 \frac{A_g}{f_y}$$

$$= 11 * 144 / 60 = 0.264 \text{ in}^2; \text{ Use \#4 @ 9'' spacing}$$

4.4.7 Final design:

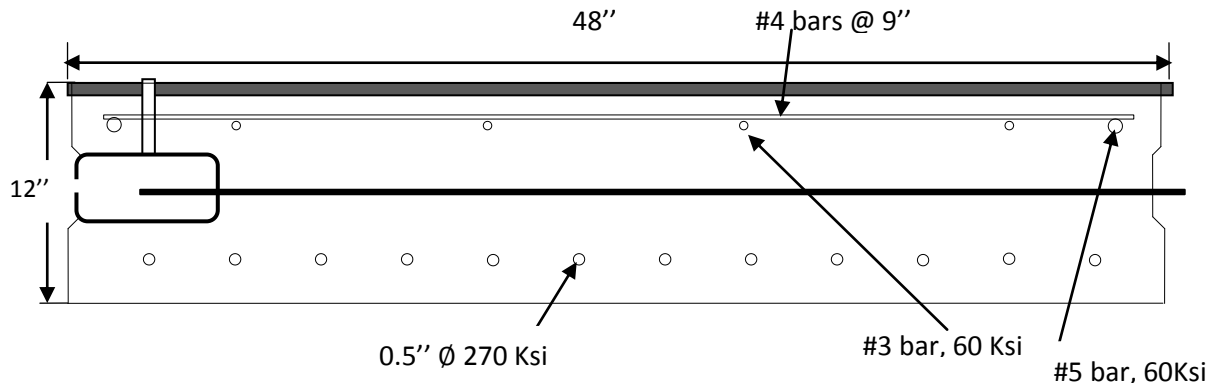


Fig. 4.6. Fully precast prestressed slab panel with the cast in place topping

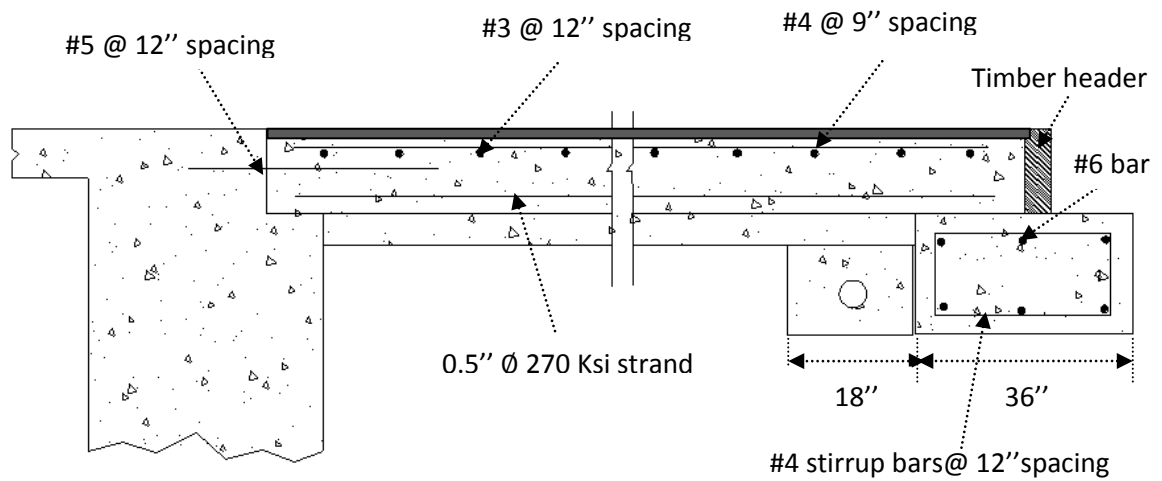


Fig. 4.7. Design details of a CIP Topped PC Prestressed bridge approach slab

4.5 Construction and connections:

4.5.1 Base Preparation:

The current MoDOT practice is to use a 3'' thick graded type V rock aggregate base for cast in place slabs. Another option is a 110 hot mix asphalt base; it is flexible in nature

and eliminates surface roughness. Eliminating surface roughness reduces alignment issues between panels making precast construction easier. Polyethylene sheeting is used over the prepared base to improve the slabs performance during freeze thaw cycles by reducing friction.

4.5.2 Grouting:

Non shrink grout is provided in the slab side of the abutment joint and at the end of the transverse tie pockets. In case significant voids are observed under the slab prior to installation, grouting the voids is necessary to prevent settlement

4.5.3 Panel geometry and alignment:

Eight 4' wide slab panels and a 6' wide slab panel both 25' long and 12" thick are used in the current construction method. The 6' wide slab panel is placed in the center while the 4' wide slabs are aligned on either side of the 6' wide slab. Panel to panel connection details are described in detail in section 4.5.6.

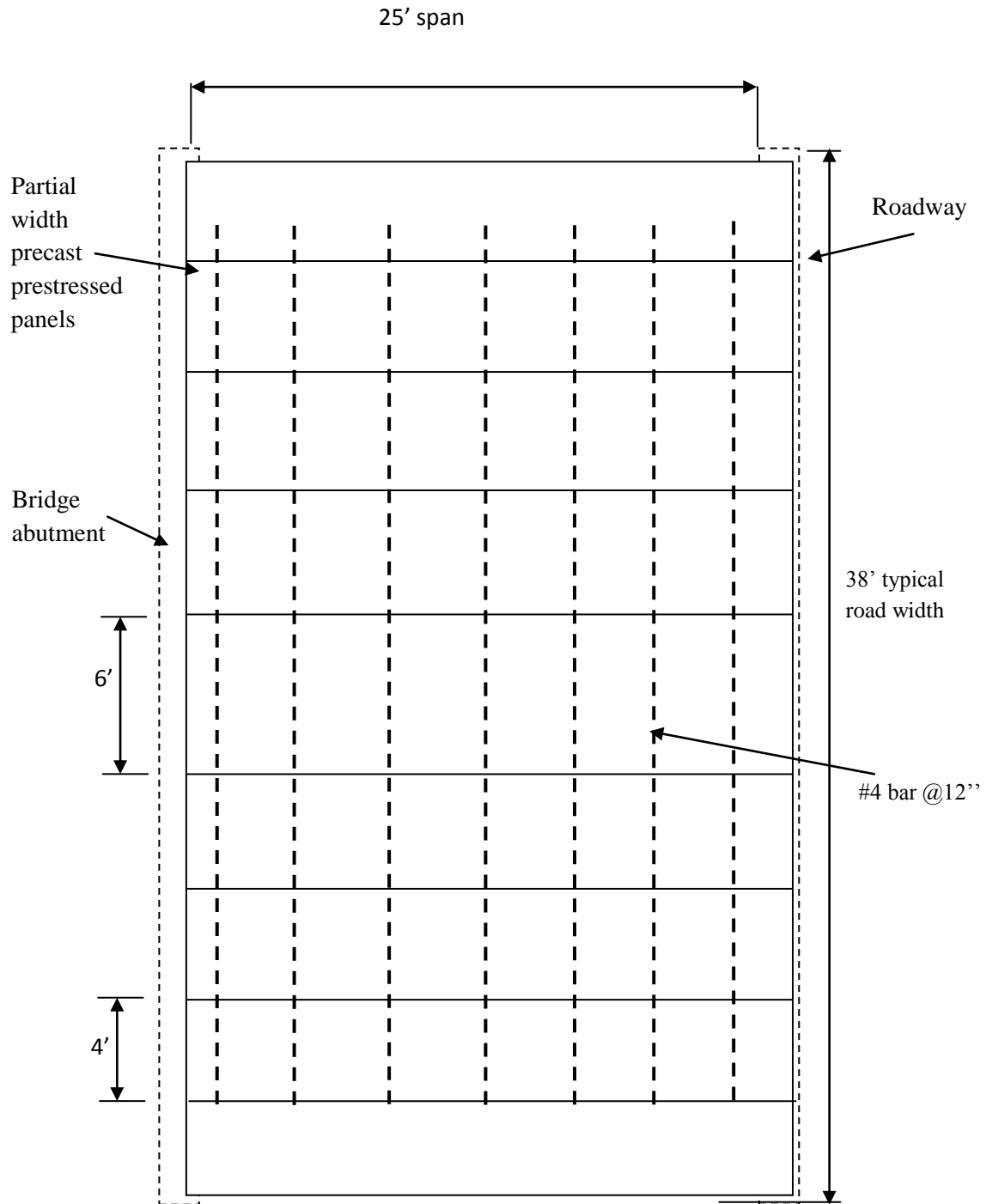


Fig. 4.8. Figure showing slab geometry and panel alignment of a standard BAS

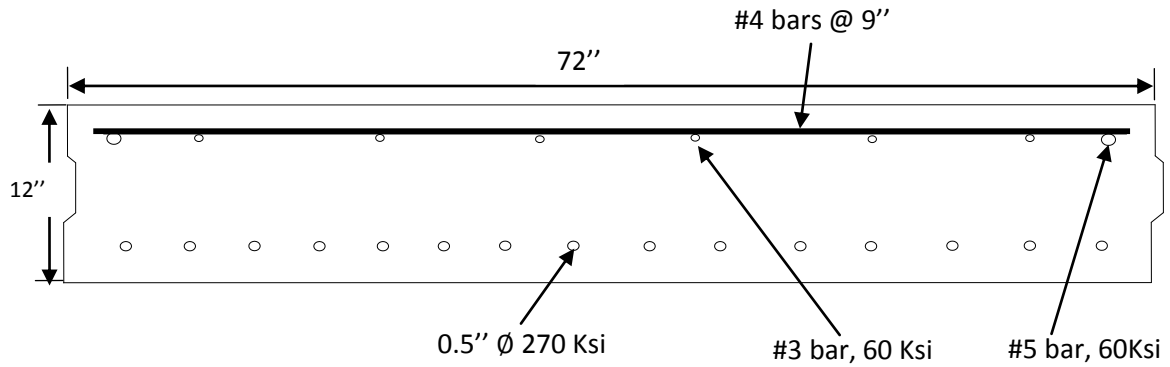


Fig. 4.9. Figure showing the 6' wide central slab panel

4.5.4 Slab to Abutment Connection:

The current MoDOT practice was to use #5 bars @12" c/c anchored both in the approach slab and the abutment. In the present design, the same concept is used to attach the precast slab panels to the abutment as shown in figures 4.3 and 4.7

4.5.5 Shear key design:

The strength of the shear key joint is verified using the modified shear friction theory (Tadros, Badie et. al. 2002). The shear key is checked for bearing and shear failures. The shear key dimensions are important as they enable full transfer of wheel loads from one joint to another.

4.5.5.1 Bearing failure check:

$$P_u \leq \Phi * 0.85f'_c * 12 * L_{inc} \quad 4.42$$

Φ – Strength reduction factor for bearing = 0.7 (AASHTO LRFD 5.5.4.2.1)

f'_c - Concrete strength of the precast panel = 6 ksi

L_{inc} - Length of the inclined portion of the shear key = 0.6''

P_u - Factored wheel load including dynamic allowance in transverse direction (kip/ft) =

(Design wheel load x live load factor x Impact factor)/ width of the contact area

Design wheel load = 16 kips (HL-93)

Live load factor is 1.75, impact factor is 1.33 and width of contact area is 20'' for HL-93 loading.

$P_u = 22.3$ kip/ft

$P_u \leq 0.7 * 0.85 * 6 * 12 * 0.6 = 25.7$ kip/ft > 22.3 kip/ft

4.5.5.2 Shear failure check:

$$P_u \leq \Phi * (12c * L_j + \mu A_{st} f_y) \quad 4.43$$

Φ – Strength reduction factor for shear = 0.9 (AASHTO LRFD 5.5.4.2.1)

C – Cohesion strength of the grout material = 0.15 ksi (AASHTO LRFD 5.8.4.2)

L_j - Length of the joint = 6''

μ - Friction coefficient of grout material = 1.4

A_{st} - Longitudinal reinforcement along the shear interface (#4 @ 12'') = $0.264/12/12 =$

$$0.264 \frac{\text{in}^2}{\text{ft}}$$

f_y - yield strength of the reinforcement = 60 ksi

$$P_u \leq 0.9 * (12 * 0.15 * 6 + 1.4 * 0.264 * 60) = 29.67 \text{ kip/ft} > 22.3 \text{ kip/ft}$$

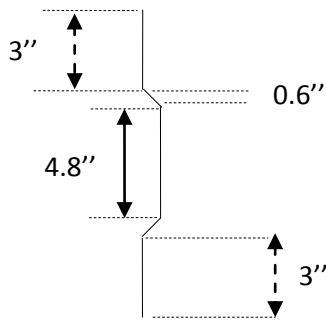


Fig. 4.10. Shear key details

4.5.6 Panel to panel connections:

On one side of the panel, a #4 bar is embedded through the hollow structural steel (HSS) tube which extends 9'' outside the panel to connect with the next panel as shown in fig 4.11. A development length of 9'' is needed for the rebar to develop yield strength. The HSS tube is 4'' long and is provided with a 1'' diameter plastic pipe on its top for grouting after the slabs are connected. Details of the HSS tube are shown in fig.4.12. The HSS tube has a 1.5'' diameter hole on the slab end side through which extended reinforcement bar of the next slab panel is inserted as shown in fig 4.13.

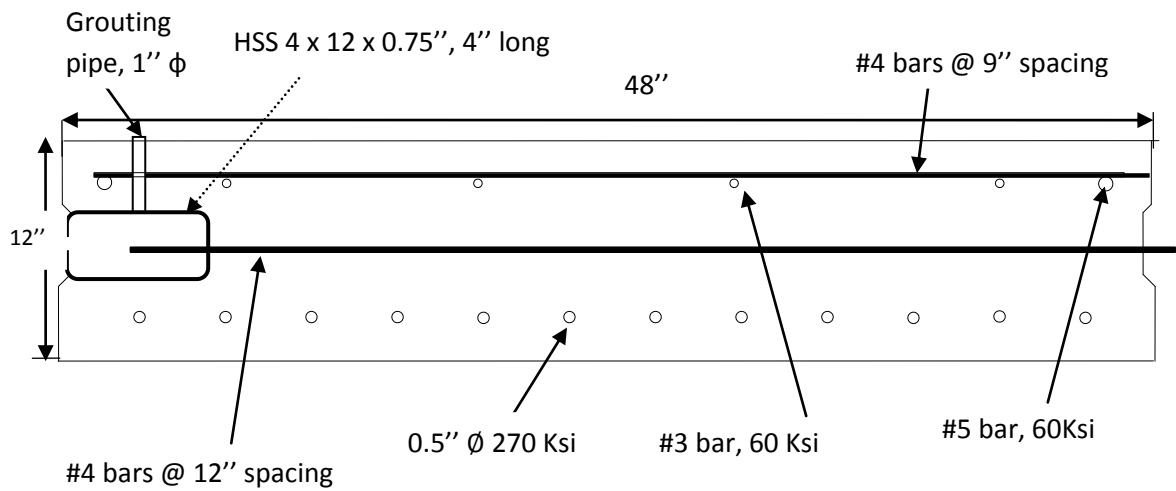


Figure 4.11. Figure showing reinforcement and connection details

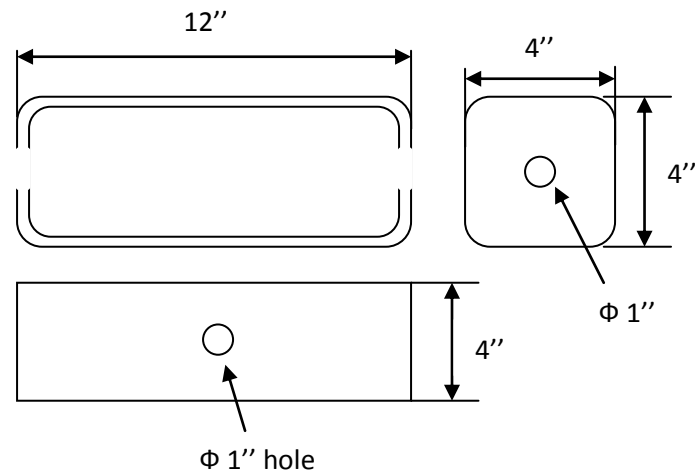


Figure 4.12. Figure showing HSS tube dimensional details

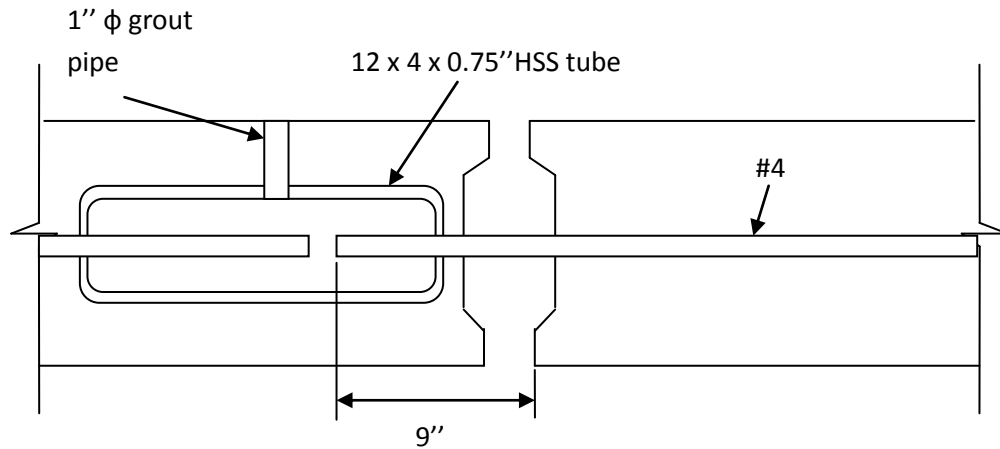


Figure 4.13. Figure showing panel to panel connections

4.6. Summary observations:

Among the loading conditions the slab was subjected to, the tandem loading was proven to be critical than the design truck loading. The optimal prestressing force required to prevent the slab from cracking is determined through the Magnel diagram. Both the fully precast prestressed and the CIP Topped PC Prestressed slabs are checked for deflection, crack width and shear criteria. All the serviceability checks are proved to be within the satisfactory limits. The slabs also passed the ultimate moment check and crack moment checks, the composite slab however has higher design moments than the non-composite slab.

The composite slab was additionally checked for horizontal shear at the cast in place topping and the precast slab interface. Horizontal shear reinforcement is suggested along

the interface to make the two concrete components act as a monolithic slab. Surface finishing might be needed for the fully precast slabs for smooth rideability as the slab panels are installed on site and there could be difference in elevation between individual slab panels, as cast-in place is topping is laid on site for the composite slab, diamond grinding the surface is not necessary. The sub grade base preparation and tolerance issues carefully addressed, the precast prestressed slab alternatives can be effectively used as rapid rehabilitation alternatives.

5. LIFE CYCLE COST ANALYSIS

Life cycle cost analysis is an economic tool employed for engineering agencies to identify the most effective of the design alternatives for a project. The analysis in this chapter is part of a project for the Missouri department of transportation (MoDOT) titled “Bridge approach slabs for MoDOT - Looking at alternative and cost effective approaches”. RealCost, a Microsoft Excel-based software that was developed by the Federal Highway Administration (FHWA) to support the application of LCCA for evaluating various pavement construction and rehabilitation strategies was chosen to compare BAS designs and rehabilitation options. Current dollars are used in the analysis and an appropriate discount rate is selected to account for the future fluctuations in the dollar value. Agency as well as user costs are calculated for both urban and rural traffic scenarios. The analysis also calculated salvage values of remaining service life of the alternatives at the end of the analysis period. A risk analysis was performed to account for the uncertainties involved in the deterministic analysis

5.1. Inputs:

5.1.1. Deterministic analysis inputs:

The following sub-sections describe the input data required for the deterministic analysis of life cycle costs.

Traffic Data:

AADT in Both Directions: Is the total annual average daily traffic (AADT) for both directions in the year of construction of the project. AADT assumed is significantly different for urban and rural traffic histories. For urban traffic an AADT of 18,826 is

assumed while for a rural traffic pattern the AADT value of 2,520 is assumed based on the information provided by MoDOT. The Urban data used here is from a suburban urban area and not from an interstate. The AADT values on an interstate range from 100K-150K vh/day. Using interstate traffic data however increases life cycle costs as increase in traffic volumes lead to higher user costs.

Single and Combo Unit Trucks as % of AADT: It is 40% for urban traffic and 12% for rural traffic. The combined percentage of vehicles is further divided in to single and combo truck percentages based on prior experience.

Annual Growth Rate of Traffic: Represents the increase in the AADT in both directions each year. The AADT increase is calculated by using Eq. 5.1. an equation recommended by CalTrans. The thus obtained values are 2% and 3% for future year and current year, respectively.

$$\text{Increase} = \left[\left(\frac{\text{FT}}{\text{CT}} \right)^{\left(\frac{1}{\text{FY}-\text{CY}} \right)} - 1 \right] \times 100 \quad 5.1$$

where FT =Future Year (FY) AADT, and CT=Current Year (CY) AADT.

Speed Limit under normal operating conditions: In the analysis, speed limits of 70mph and 50mph are used for urban and rural traffics.

Free Flow Capacity: Is the maximum capacity a facility can handle under normal operating conditions. According to Highway capacity manual (1994), the maximum capacity of a 2 lane directional highway is 2,200 passenger cars per hour. This varies according to the percentage of trucks and busses, reduced lateral clearances and restricted lane widths. The RealCost calculated values were 1,833 vhpl in urban where as in a rural scenario it is 2,075 vhpl.

Queue Dissipation Capacity Normal: Represents the capacity of the lane to dissipate a queue. It is assumed to be 200 vph less than the free flow capacity of the lane (FHWA).

Maximum AADT in both Directions: Is calculated for 40 years based on the percentage of increase in the AADT as discussed above.

Maximum Queue Length: Refers to the calculated queue length basing on the number of vehicles queued during the traffic hourly distribution. A queue length of 2 miles is assumed for urban traffic, queuing is assumed not to occur in rural traffic scenarios.

Value of User Time:

These are the user delay costs and they differ for passenger cars and trucks. The base year values of each vehicle type for the year 1990 are taken from the FHWA manual. The base year values are then adjusted for the current year by using the transportation component in the consumer price index (CPI) for both the years.

$$\text{Escalation factor} = \frac{(\text{CPI of 2009})}{(\text{CPI of 1990})} = \frac{179.2}{130.7} = 1.37$$

5.2

Table 5.1. Value of time in \$/vehicle-hour for 1990 base year and the year 2009

Year	Passenger Cars	Single Unit Trucks	Combination Trucks
1990	9.75	14.96	21.42
2009	13.36	20.49	29.35

Traffic Hourly Distribution:

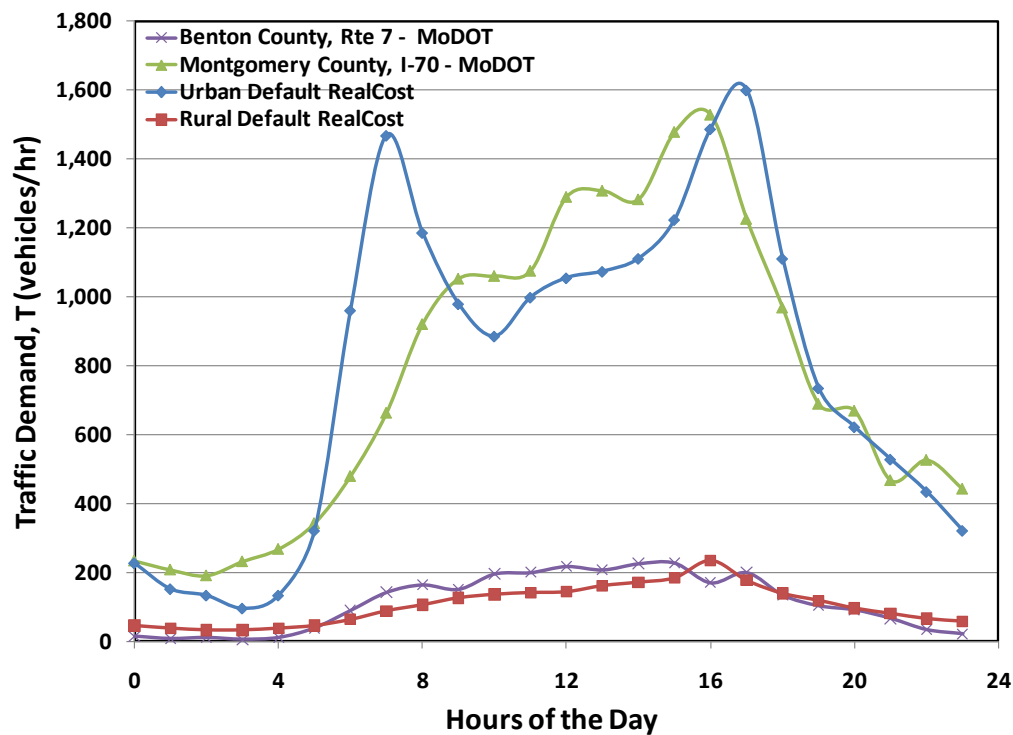


Fig 5.1. Hourly traffic demand history used for a typical urban (Montgomery County, I-70) and rural (Benton County, Rte. 7) used in the research are compared with default histories from RealCost.

Default hourly traffic distribution for urban and rural cases is provided in RealCost. Fig.

5.1 shows the default hourly traffic distribution of the vehicles along with MoDOT

provided data for a typical urban (Montgomery County, I-70) and rural (Benton County, Rte. 7) traffic patterns.

Added Time per 1,000 stops and Vehicle Operating Cost:

“Added Time per 1,000 Stops (hours)” and “Added Cost per 1,000 Stops (\$)” are the quantities used to calculate user delay costs and vehicle costs due to speed changes and stop and go conditions during a work zone. RealCost provides default values based on NCHRP research. These values for the base year 1996 are adjusted (increased) to the present year based on the present year CPI.

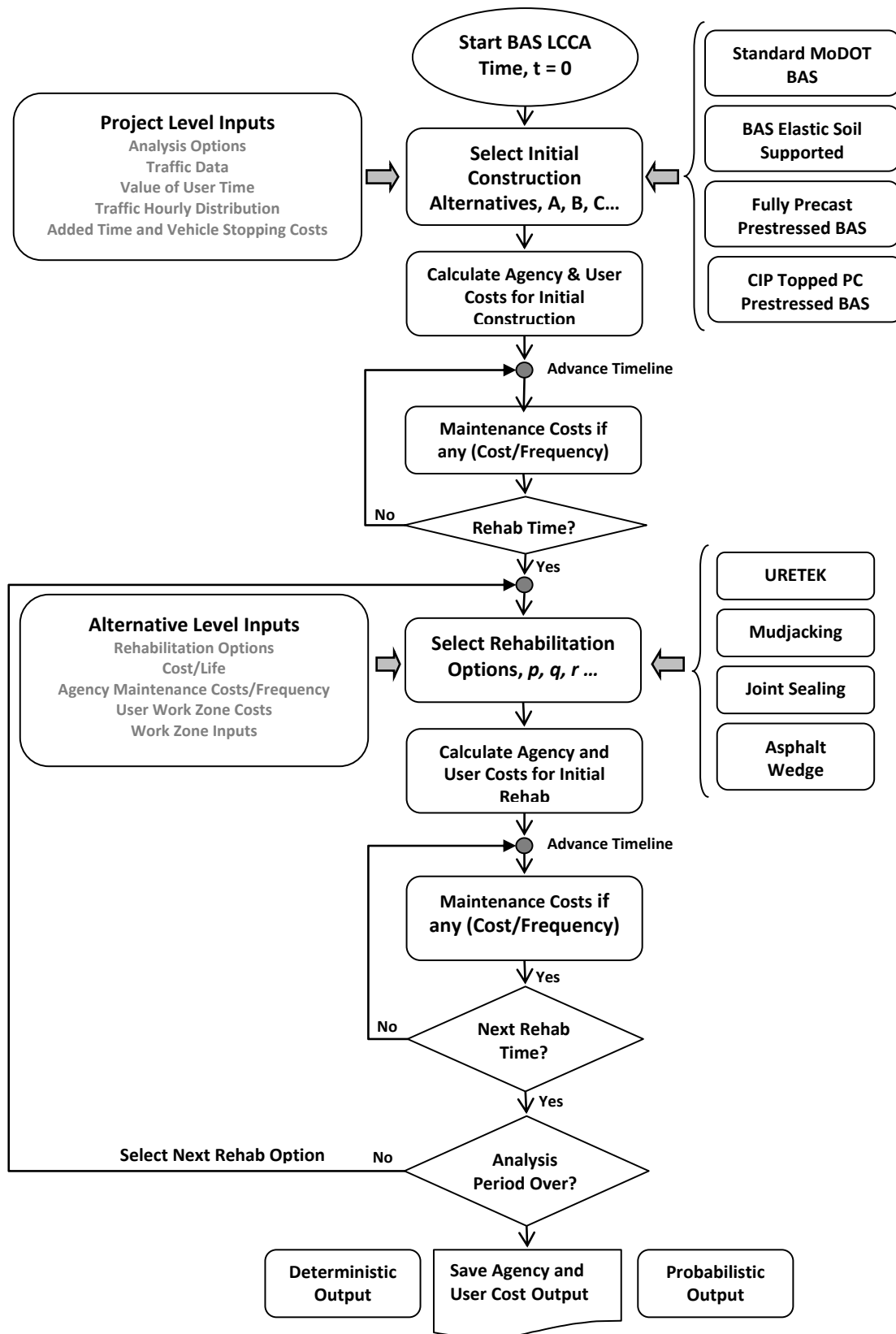


Fig. 5.2. Flowchart highlighting steps in the LCCA process implemented in this study

5.1.2. Alternative level inputs

Alternative Description: The four design alternatives entered are as ‘Standard MoDOT BAS’, ‘BAS-Elastic Soil Support’, ‘Fully Precast Prestressed BAS’ and ‘CIP Topped PC Prestressed BAS’.

Number of activities: The numbers of activities such as initial construction and rehabilitation activities are entered. The precast alternatives have five activities, an initial construction activity and four rehabilitation activities while the cast in place alternatives have three activities, in addition to the initial construction it has two rehabilitation activities during the analysis period.

Activity Description: The description of each alternative is entered based on whether it is initial construction, rehabilitation (such as URETEK, Mudjacking, Joint sealing, Asphalt wedging etc.). Two URETEK rehabs were selected for the cast-in-place slab alternatives and four joint sealing rehabs were selected for the precast prestressed slab alternatives.

Table 5.2. Table showing initial construction and costs and rehabilitation costs of design alternatives

Design alternative	Initial construction costs	Rehabilitation costs (Cost for a single activity)
Standard MoDOT BAS	\$ 61,000	URETEK- \$ 7,140
BAS-ES	\$ 45,000	URETEK- \$ 7,140
Fully Precast Prestressed BAS	\$ 54,000	Joint sealing- \$ 1,400
CIP Topped PC Prestressed BAS	\$ 58,000	Joint sealing- \$ 1,400

Agency Construction Costs: The agency costs involved in each activity such as initial construction and rehabilitation activities listed in table are entered in their respective activity tabs, these values are based on MoDOT provided data.

Activity Service life: The Activity Service Life of each activity is entered based on a combination of the service life estimates on prior experience and information provided by MoDOT. An initial service life of 23 years is assumed for all the design alternatives except for the Fully Precast Prestressed BAS (FPPC), a service life of 20 years is assumed for the FPPC BAS. URETEK method has a 10 year service life and joint sealing has a service life of 5 years.

Work Zone length (miles):

A work zone length of 25' (BAS span) is assumed in this analysis. Even though work zone lengths for conventional pavements during maintenance/rehabilitation activities range from 825 – 2000 ft it is not used in this analysis as a work zone length of 1000ft seems impractical for approach slab rehabilitation.

Work Zone Duration (days): Duration of days the work zone is in operation during the initial construction and the future rehabilitation activities. The work zone duration during initial construction is assumed to be 30 days for the cast in place slabs, 15 days for the fully precast prestressed BAS and 17 days for the CIP Topped PC Prestressed BAS.

URETEK method is assumed to have work zone duration of 3 days and joint sealing is assumed to have 2 day work zone duration.

Work Zone Capacity: Is the vehicular capacity of one lane of the work zone for 1 hour. This was assumed to be 1,240 for a single lane closure based on MoDOT work zone guidelines. The work zone capacity assumed here is only valid for two to one lane freeways in Missouri.

Work Zone Speed Limit: Refers to the speed limit within the work zone and is taken as 50 mph for urban and 60 mph for rural traffic as per MoDOT recommendations.

No. of Lanes Open in Each Direction during Work Zone Operation: This represents the number of lanes open during work zone operations and is assumed as 1 lane.

5.1.3. Probabilistic Analysis Inputs

Depending on how the uncertainty is represented in the analysis two types of probabilistic models are developed. First order models which use the mean of the uncertain inputs where the risk events are modeled as independent and second order models, in which uncertainty is modeled by both the mean and the standard deviation of uncertain variables. Second order models will produce more accurate results at the outermost ranges of the cost distributions. (Molenaar et al. 2005). In the present life cycle cost study, second order models are used even though more time and effort is needed to establish the mean and standard deviation values for all the inputs.

Normal distribution is used to generate random values for all the inputs used in this study while performing risk analysis. A normal distribution is a more appropriate fit than triangular or rectangular distribution as the normal distribution presents mean, minimum and maximum values. FHWA recommends using normal distribution when collected data is available for an input. The RealCost software uses Monte Carlo simulation for 2,000 iterations and monitors convergence for every 50 iterations. Standard deviation values considered in this analysis are in the range of 1/5-1/6 of the mean values.

5.2. Design alternatives:

Four design alternatives are investigated in this study; the standard MoDOT BAS is included as one of the design alternatives in the study to compare the cost effectiveness of the suggested alternatives.

5.2.1. Standard MoDOT BAS:

The standard BAS used by MoDOT is a 12'' deep doubly reinforced 25' (span) approach slab resting on the bridge abutment on one end and a sleeper slab on the embankment on the other end. The approach slab is designed as a simply supported slab neglecting soil support in between the two end supports. The design details are discussed briefly in chapter 3.

5.2.2. BAS-ES:

In this design, the sleeper slab at the pavement end of the conventional MoDOT BAS design is replaced by a modified end-section reinforcement detailing to provide enhanced

local two-way action, providing increased flexural rigidity in the direction transverse to the traffic direction. This alternate BAS–ES design has been shown to result in significantly smaller design moments even when partial washout of soil support is assumed. Results from the LCCA analysis presented in the next section will show that this design alternate would be ideally suited for rural traffic patterns where it has the advantage of low agency cost and relatively small user cost as well. The design details are discussed in chapter 3.

5.2.3. Fully precast prestressed BAS:

The fully precast prestressed BAS is a 12’’ deep slab; it has three 0.5’’ dia. 270 ksi strands per unit width of the slab. It has additional 60 ksi steel reinforcement to resist negative moments, shrinkage and temperature effects. The design has been discussed in detail in chapter 4.

5.2.4. CIP Topped PC Prestressed BAS:

The CIP Topped PC Prestressed BAS is a 10’’ deep precast slab with a 2’’ cast-in-place topping, It has similar reinforcement details as the fully precast prestressed slab, the design has been discussed in detail in chapter 4.

5.3. Results and discussions:

5.3.1. Cumulative distributions:

The agency costs include the initial construction cost and the cost for rehabilitation and maintenance activities carried out during the life time of the approach slab. The costs of

the Standard MoDOT BAS and BAS-ES have been studied using 2 URETEK processes each. The costs of fully precast prestressed BAS and the CIP Topped PC Prestressed BAS are studied using 4 joint sealing rehabilitations each. The cumulative distribution graphs are obtained by implementing several iterations of the inputs using Monte Carlo simulation technique in RealCost. The analysis period over which the life cycle costs are calculated for the design alternatives is 40 years. A discount rate of 4% is used as recommended by FHWA. Fig 5.3 shows the cumulative distribution of the agency costs for all the slab alternatives. The Standard MoDOT BAS was the costliest and the BAS-ES is the most economical of the selected alternatives. The results showed a 90% probability (cumulative) for the BAS incorporating elastic support to yield the lowest costs to the agency.

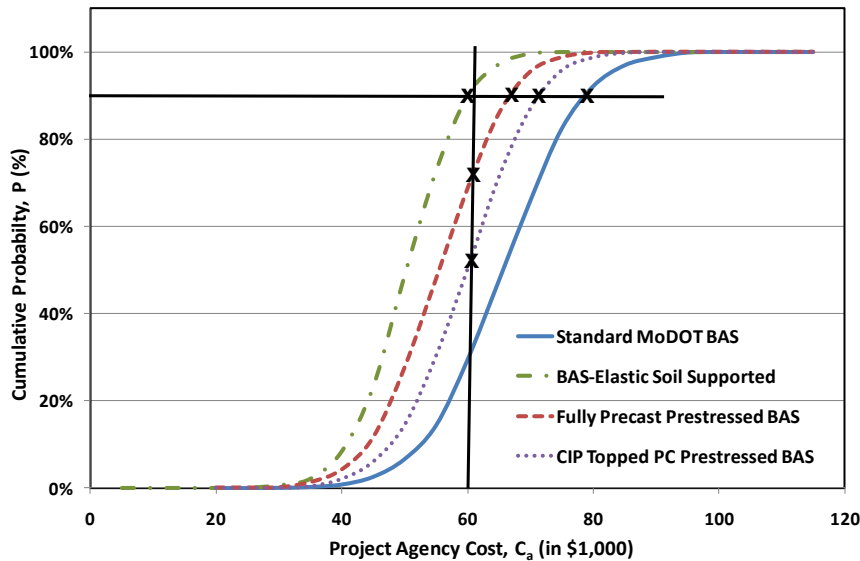
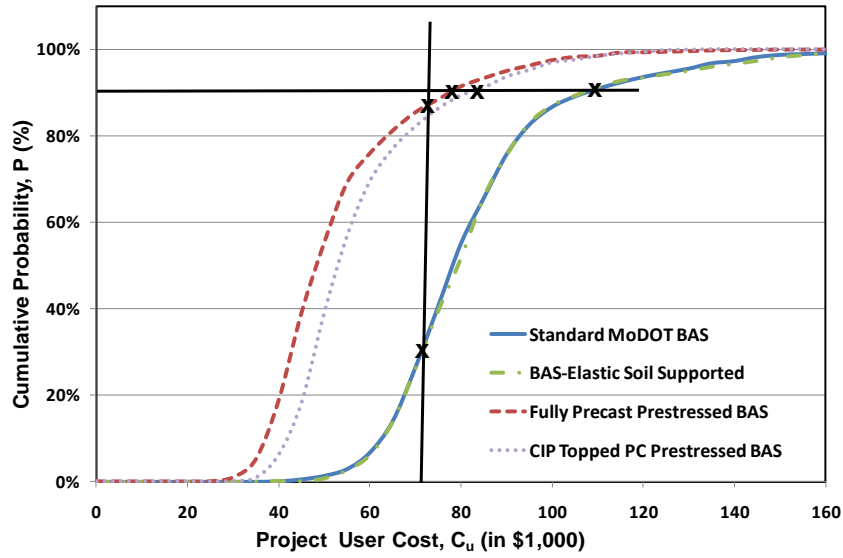


Fig. 5.3. Relative cumulative probability distributions of project costs of typical BAS design alternatives

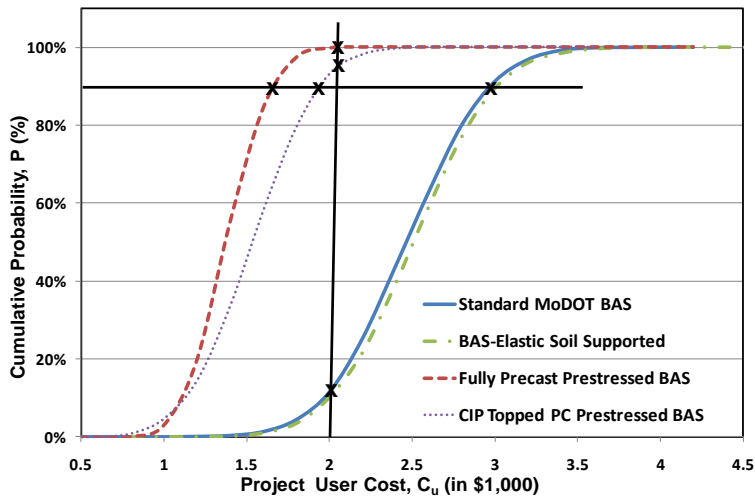
The figure shows that the Standard MoDOT BAS has higher life cycle agency costs than the other design alternatives. Another way to read the plots is that, for a net present value of \$60,000 there is a 30% probability that the Standard MoDOT BAS can be constructed at that cost. There is a 90% probability that the BAS-ES can be constructed for the same cost. The probabilities for the Fully and CIP Topped Precast Prestressed BAS for a cost of \$60,000 are 74% and 52% respectively.

The user costs of BAS-ES and the Standard MoDOT BAS are the same as equal work zone durations are assumed for both the slabs as shown in Fig. 5.4 (a). The difference in the user costs between the Fully Precast Prestressed BAS and the CIP Topped PC Prestressed BAS is due to differences in the work zone durations. The Fully Precast Prestressed BAS is the lowest costing alternative in rural and urban scenarios when only user costs are considered.

Figures 5.4. (a) and 5.4. (b) show the cumulative distribution of user costs for urban and rural traffic scenarios respectively. The urban user costs depend mainly on the work zone duration and its capacity. Rural user costs depend on the value of user costs for the passenger cars as well as combination trucks. It can be understood from the figures that the Fully precast prestressed BAS has 90% lesser user costs than the other design alternatives in both rural as well as urban scenarios. There is an 87% probability that the Fully precast prestressed BAS incurs a user cost of \$70,000 in the urban scenario. The probability for the Standard MoDOT BAS and BAS-ES to incur a user cost of \$70,000 is 30% as shown in Figure 5.4(a).



(a)



(b)

Fig.5.4. Relative cumulative probability distributions of project costs of typical BAS design alternatives for (a) Urban traffic (b) Rural traffic

The rehab activities considered for the slabs, URETEK method and joint sealing have work zone durations of 3 days and 2 days respectively. Even though the work zone durations during rehab activities are not significantly different among the design

alternatives, the main difference in the user costs between the precast slabs (Fully precast prestressed and CIP Topped PC Prestressed BAS) and the cast in place slabs (standard MoDOT BAS, BAS-ES) is the work zone duration during the initial construction period. The precast slabs have a work zone duration ranging from 15-17 days while the cast in place slabs on the have a work zone duration of 30days.

Even in the rural traffic scenario, the Fully Precast Prestressed BAS is the most economic design alternative followed by the CIP Topped PC Prestressed BAS, the high cost alternatives are again the BAS-ES and the Standard MoDOT slab. Even though the economy wise ranking of the design alternatives for the rural scenario is similar to the urban scenario, the difference in the costs between the precast slabs and the cast in place slabs is only in the order of \$ 500 to \$ 1,000; nearly 10 times lesser than the urban scenario.

5.3.2. Mean distributions:

A probabilistic analysis is run to account for the uncertainty involved in the inputs. Table 5.3 shows the mean, standard deviation, minimum and maximum values of the agency costs obtained through running a probabilistic analysis using Realcost software for the four BAS design alternatives. Each and every input involved in the calculation of life cycle costs is assigned a mean value and standard deviation. A normal probabilistic distribution is used to simulate random numbers for the inputs involved in the calculations.

Table.5.3. Mean distributions of costs (Monte Carlo simulation values)

Cost (Present Value) (x\$1,000)		Design Alternatives			
		Standard MoDOT BAS	BAS-Elastic Soil Supported	Fully Precast Prestressed BAS	CIP Topped PC Prestressed BAS
Agency	Mean	\$66	\$50	\$55	\$60
	Standard Deviation	\$10	\$8	\$9	\$9
	Range	\$35 - \$109	\$29 - \$74	\$20 - \$89	\$32 - \$87
Urban User	Mean	\$82	\$83	\$53	\$58
	Standard Deviation	\$23	\$22	\$18	\$17
	Range	\$38 - \$516	\$33 - \$345	\$24 - \$257	\$29 - \$144
Rural User	Mean	\$2	\$3	\$1	\$2
	Standard Deviation	\$0.4	\$0.4	\$0.2	\$0.3
	Range	\$1 - \$4	\$1 - \$4	\$0.7 - \$2	\$0.5 - \$3

The mean distributions highlight the mean value of the normally distributed present values of costs. The difference in mean user costs between urban and rural scenario results due to significant difference in the traffic volumes (AADT: 18,826 (Urban), 2,520 (Rural)).

As each value represents a possible scenario, considering three standard deviations to the either side of the mean makes sure that each and every possible cost scenario is taken into account during the risk analysis. As shown in fig.5.5 Fully Precast Prestressed BAS has lesser mean costs than the other slab alternatives followed by the CIP Topped PC

Prestressed BAS. The mean user costs for the Standard MoDOT BAS and BAS-ES are the same with a marginal difference in their standard deviations.

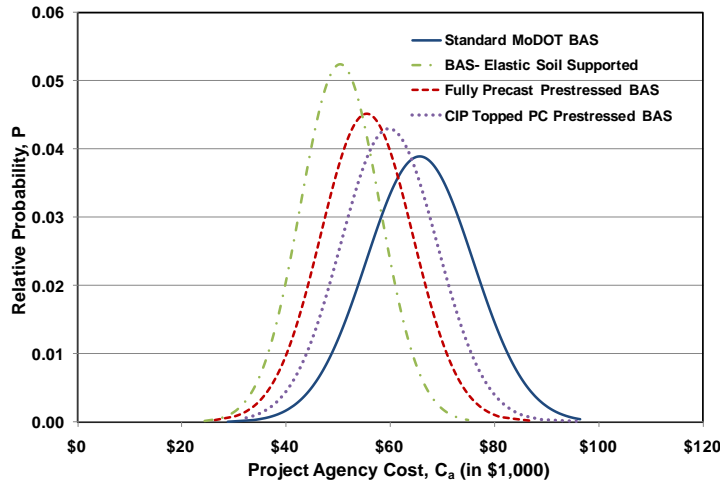
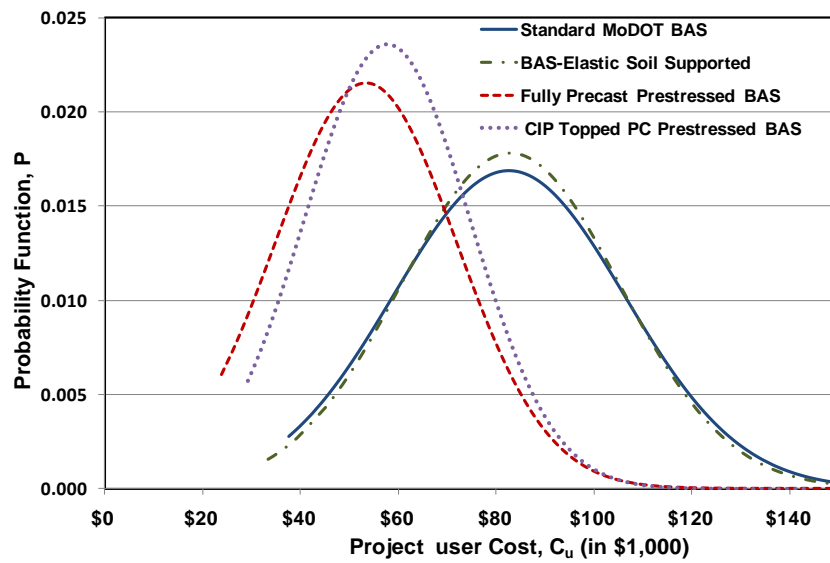


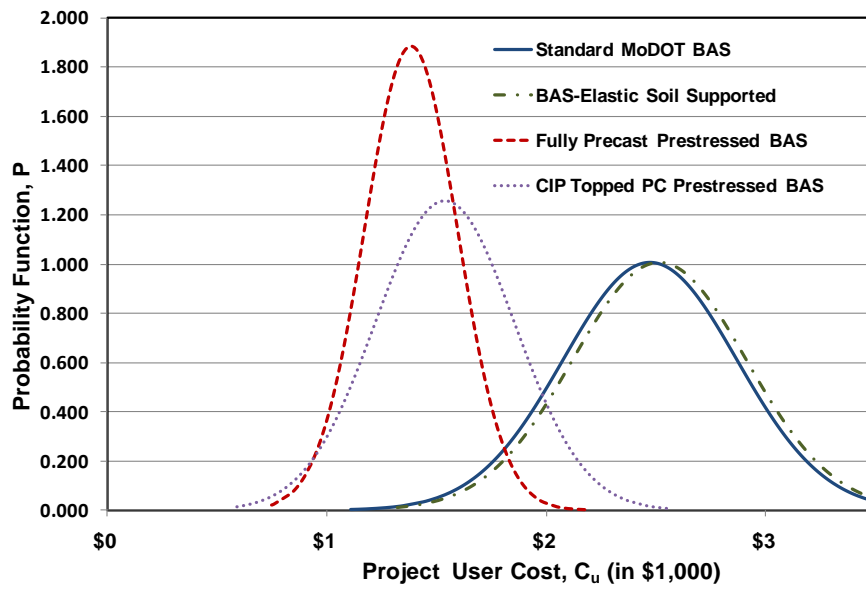
Fig.5.5. Agency cost distributions of typical BAS alternatives

Figure 5.6(a) shows the mean user costs for all the design alternatives for the urban scenario. The Fully Precast Prestressed BAS has lesser mean costs than the other BAS alternatives followed by the CIP Topped PC Prestressed BAS. The mean user costs of the Standard MoDOT BAS and BAS-ES are proportionate as they are assumed to have equal work zone durations.

In the rural scenario as shown in fig. 5.6(b), the Fully Precast Prestressed BAS is the most economic design alternative, followed by the CIP Topped PC Prestressed BAS. The Fully Precast Prestressed BAS has lower work zone closures during the rehabilitation activities for the entire analysis period; therefore its standard deviation is lower than the rest which shows in the form of a raised peak in the rural mean user costs graph



(a)



(b)

Fig.5.6. Relative cumulative probability distributions of project costs of typical BAS design alternatives for (a) Urban traffic (b) Rural traffic

. On the contrary, this behavior is not observed in the urban mean user costs plot, the reason for this could be because of high mean and standard deviation values in the urban scenario (the standard deviations for the urban scenario are 1/3-1/4 of mean, in the rural they are 1/5-1/6 of mean).

5.3.3. Deterministic v/s Probabilistic results:

The agency costs depend on initial construction costs, maintenance costs, rehabilitation costs and year of the rehabilitation activity. The user cost on the other hand depends upon the vehicle operating costs (VOC), crash costs and speed delay costs. High volumes of traffic may lead to additional user costs like vehicle queuing costs especially in urban areas.

Table 5.4. Comparison of deterministic and probabilistic results

Total Cost (Present Value) (x\$1,000)		Standard MoDOT BAS		BAS-Elastic Soil Supported		Fully Precast Prestressed BAS		CIP Topped PC Prestressed BAS	
		Agency	User	Agency	User	Agency	User	Agency	User
Urban	Deterministic Results	\$65	\$73	\$50	\$74	\$56	\$43	\$60	\$47
	Probabilistic Results	\$66	\$83	\$50	\$83	\$55	\$53	\$60	\$58

	Standard Deviation	\$10	\$24	\$8	\$22	\$9	\$19	\$9	\$17
Rural	Deterministic Results	\$65	\$2	\$50	\$2	\$56	\$1	\$59	\$2
	Probabilistic Results	\$65	\$2	\$50	\$2	\$55	\$1	\$60	\$2
	Standard Deviation	\$10	\$0.41	\$8	\$0.40	\$9	\$0.22	\$9	\$0.34

Depending on the traffic demand, work zone costs play a significant part in the overall scheme of life cycle costs. Even though the work zone closure periods are similar in rural and urban traffic scenarios, rural user costs are lower than the urban user costs as the traffic demand is lower. In the probabilistic analysis, as more inputs are involved in the user cost calculations than in the agency costs, the calculated results are not similar to the deterministic results.

The difference in the user costs between the probabilistic and deterministic results in the rural scenario is marginal. In the urban scenario, the cost difference between the deterministic and probabilistic results is significantly large. Looking at the costs in table 5.3, we can see that the difference in user costs between the deterministic and probabilistic results for urban scenario ranges from \$9,000-\$10,000. The difference between deterministic and probabilistic results in the rural scenario is almost negligible (in the order of \$50-\$100). This is due to the fact that there are increased user cost components in the urban traffic scenario, the demand increases in while the traffic capacity of the slab remains the same. The additional user cost components in the urban

scenario include queuing costs of vehicles, speed change costs and work zone delay costs.

It is evident from the results that the work zone costs constitute a significant part in the overall user costs, the lower the work zone duration, lower the user costs. The user costs would therefore be almost similar for the Standard MoDOT BAS and the BAS-ES as both the slabs have equal work zone durations.

5.3.4. Expenditure stream diagrams:

Expenditure stream diagrams are a pictorial representation of money flow for various construction activities carried out during the analysis period. The x-axis shows the pavement life and the y-axis shows the cost incurred by the agency during the service life of the slab for a period of 40 years (analysis period). The major advantage of expenditure stream diagrams being they show the amount of money the agency has to expend in a specific year. Fig. 5.7 shows the expenditure stream diagram for the design alternatives over an analysis period of 40 years. The figure also shows salvage values for BAS alternatives at the end of analysis period.

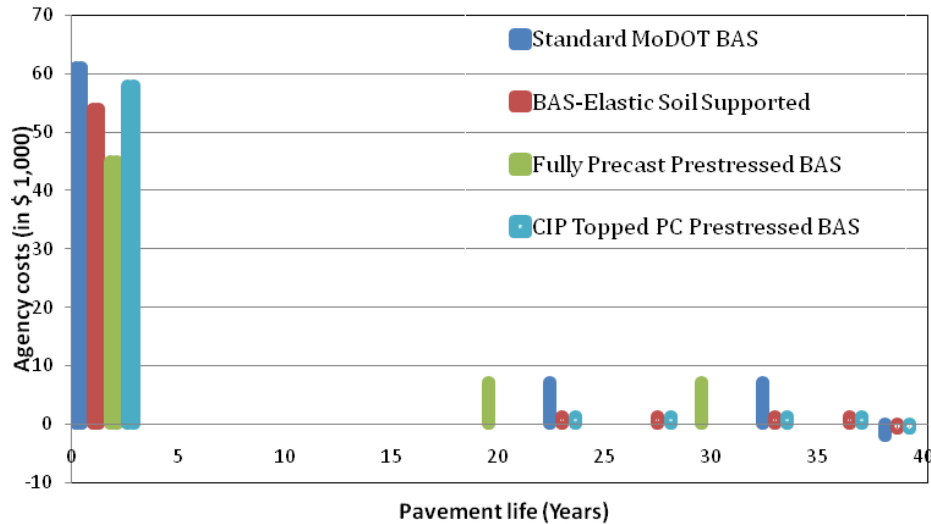


Fig. 5.7. Expenditure stream diagram of the agency costs (Initial construction & rehabs)

The salvage value is the remaining value of the slab at the end of analysis period. The cost occurring during each remaining year after the analysis period is calculated and the thus arrived costs for each year are summed up to obtain the salvage value. Salvage costs are not expenditures but are left over values of the slab. As the last rehabs on the precast BAS are conducted close to the 40 year mark, their salvage values are higher than the Standard MoDOT BAS.

As can be noticed from table 5.4, the agency cost salvage value for the urban and rural scenarios are less for the precast slabs (fully precast prestressed BAS+ CIP Topped PC Prestressed BAS) than for the standard MoDOT BAS. This is because the total costs to the agency during the analysis period are more for the Standard MoDOT BAS than for the precast slabs.

Table. 5.5. Work zone user costs during activities

Alternative		Work Zone User Costs During Activities (x \$1,000)					
		Initial Const.	Rehabilitation				User salvage value (Negative)
			Activity 1	Activity 2	Activity 3	Activity 4	
Urban	Standard MoDOT BAS	\$65	\$12	\$12	-	-	\$4
	BAS- ES	\$65	\$12	\$12	-	-	-
	Fully Precast Prestressed BAS	\$33	\$8	\$8	\$8	\$8	\$5
	CIP Topped PC Prestressed BAS	\$37	\$8	\$8	\$8	\$8	\$5
Rural	Standard MoDOT BAS	\$2	\$0.33	\$0.33	-	-	\$0.12
	BAS- ES	\$2	\$0.31	\$0.31	-	-	-
	Fully Precast Prestressed BAS	\$1	\$0.21	\$0.21	\$0.21	\$0.21	\$0.23
	CIP Topped PC Prestressed BAS	\$1	\$0.22	\$0.22	\$0.22	\$0.22	\$0.12

5.3.4.1. Cumulative expenditure stream diagrams:

The cumulative expenditure stream diagrams show the cumulative cost over an analysis period of 40 years. The figure shows the comparison of cumulative user costs between the urban and rural scenario for the four design alternatives. The two precast prestressed alternatives are stacked as precast slabs and the BAS assuming elastic support and the Standard MoDOT BAS are addressed as cast-in-place slabs.

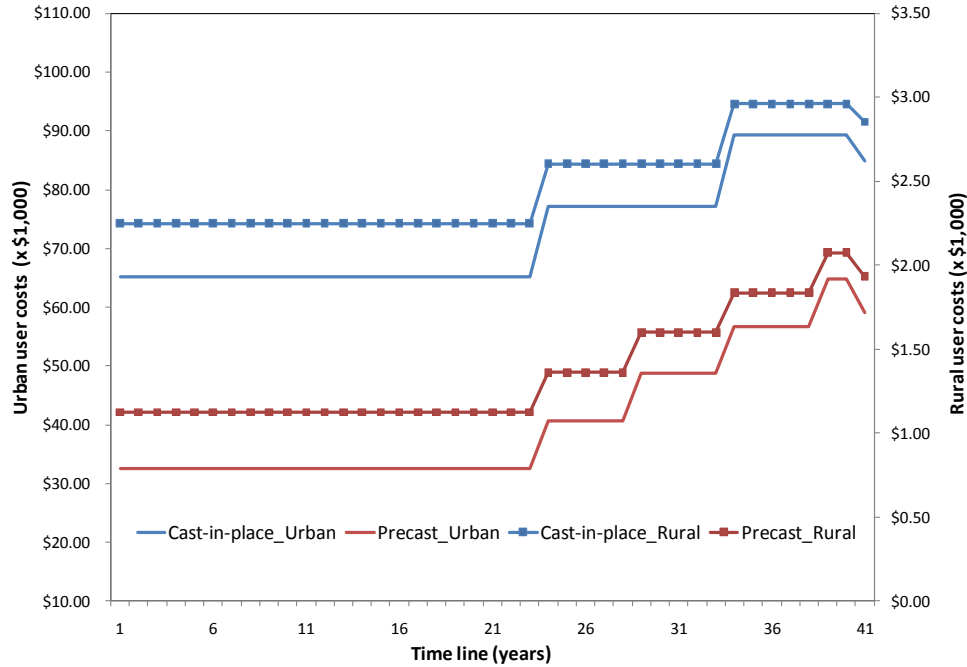


Fig. 5.8. Cumulative expenditure stream diagram

Refer to the primary axis for urban user costs and secondary axis for the rural user costs.

For the precast slabs, average cost is calculated for Standard MoDOT BAS and the BAS-ES. Similarly, for the cast-in-place slabs, average costs are calculated for Fully Precast Prestressed BAS and CIP Topped PC Prestressed BAS. The cumulative distribution curve for the slabs remains linear till an analysis period of 20 years which is the service life of the slab for initial construction. The curve rises at the point of time whenever a rehabilitation activity is conducted.

The downward slope in the curve at the 40 year mark is due to the salvage cost of the slab which is the leftover value of the slab at the end of the analysis period. As could be seen, the downward slope in the curve is more pronounced for precast slabs than in the cast-in-

place slabs as the salvage cost value is higher for precast slabs than the cast in place slabs.

5.3.4.2. Differential user costs:

The cumulative expenditure stream diagrams for the difference in user costs between the urban and rural scenarios for the design alternatives is shown below. The significance of the plot is that it can expedite the process of choosing an economic alternative. In case the user costs are being looked at by the agency, it helps it to choose an alternative which leads to user costs that are similar both in the urban as well as rural scenarios. For example, considering an ideal case in which the agency incurs similar user costs in urban as well as rural traffic scenarios, the difference in user costs between rural and urban traffic would be zero, in other words it would be the best economic alternative.

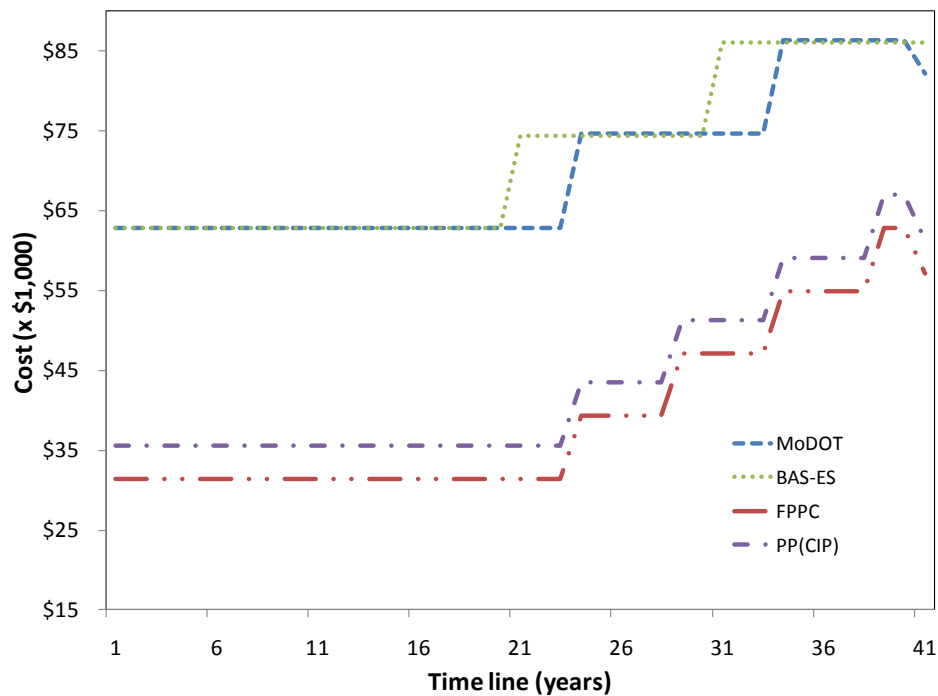


Fig.5.9. Cumulative expenditure stream diagram showing differential user costs

In our case, the Fully Precast Prestressed BAS (FPPC) has lesser differential user costs (the difference in user costs between the urban and rural scenario is just \$30,000) than the other BAS alternatives. Therefore the FPPC BAS has lesser user costs and the standard MoDOT BAS and BAS-ES (difference in user costs is more than \$60,000) have higher user costs in any traffic scenarios.

5.3.5. Correlation coefficient plots (Tornado graphs):

The correlation coefficient plots also known as the tornado plots are part of the sensitivity analysis. As there are several inputs involved in the computation of the total costs, a sensitivity analysis study is conducted to find out the inputs that have a dominant effect on the final output. Any input with a correlation value less than 0.16 is considered ineffective and not to have a significant effect on the final output. A negative correlation value of an input will have a negative effect on the output. A positive correlation value can be understood as having a directly proportional effect on the output, similarly a negative correlation value can be considered as having an inversely proportional effect on the output.

Looking at the urban tornado plot in fig. 5.10 the work zone duration during initial construction has more effect on the total costs than any other input. Another work zone characteristic, the work zone capacity during the rehab operations has a negative correlation to the output meaning that with an increase in the work zone capacity there would be a decrease in the overall costs.

Another manner to understand the plots is that, for example the correlation value for the work zone duration during initial construction is 0.75 which means that if agency cost moves one standard deviation in either direction then the present value of the MoDOT BAS will move 0.86 of standard deviation in the same direction. However in case of a negative correlation value as in the work zone capacity during rehabs in fig. 5.10 , if it moves one standard deviation in either direction, the present value will move 0.25 standard deviation in the opposite direction.

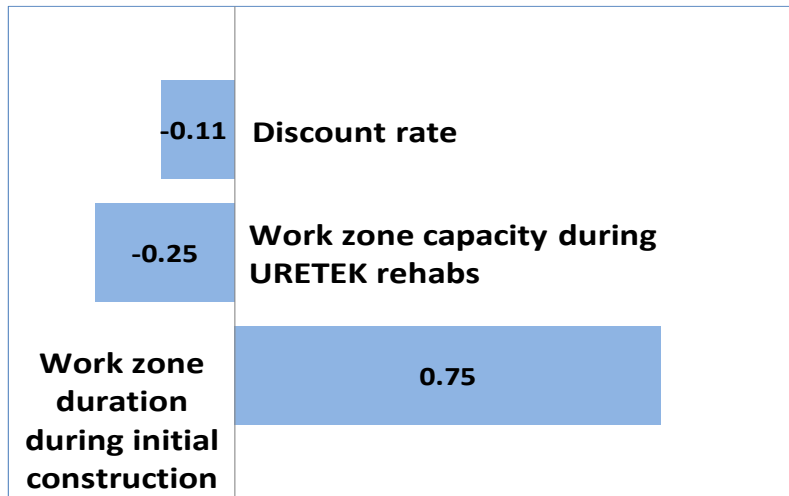


Fig. 5.10. Correlation coefficient plots for Standard MoDOT BAS in Urban

The rural tornado plot fig. 5.11 shows that work zone duration during initial construction and value of time for combo trucks affect the total costs more than the value of time for passenger cars. The urban user cost during the initial construction of the slab is \$65,130, which is higher than the initial construction cost of the slab. The above plots again highlight the effect of work zone characteristics on the overall costs.

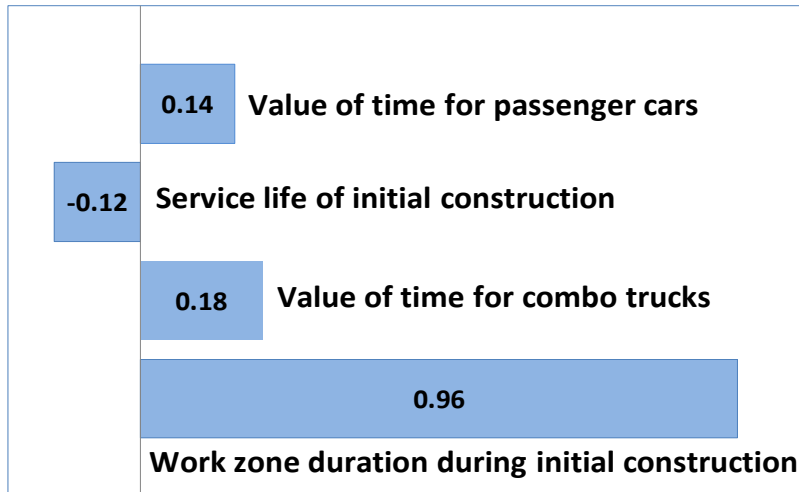


Fig.5.11. Correlation coefficient plot for Standard MoDOT BAS in) Rural

Even though the Fully Precast Prestressed BAS has lower work zone closures than any other alternative, work zone duration has a significant effect on the final output as shown in fig 5.12 and fig 5.13.

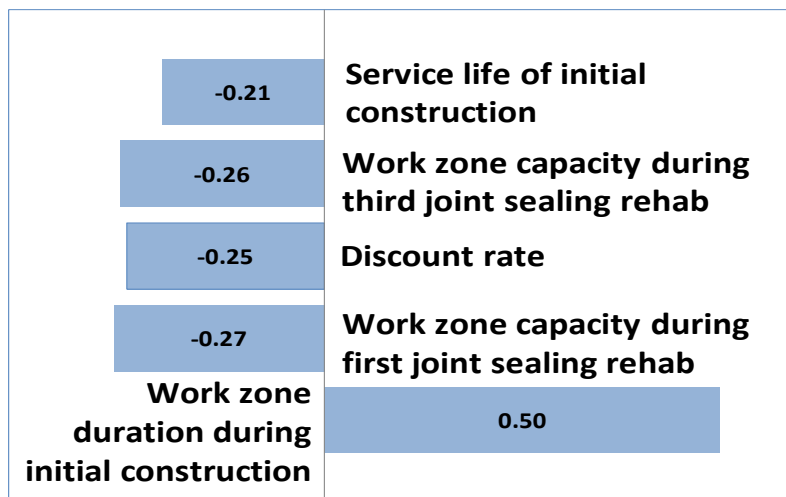


Fig.5.12. Correlation coefficient plot for Fully precast prestressed BAS in Urban

The work zone capacity, another important work zone characteristic also affects the final outcome, increasing the capacity of the work zone would therefore lead to decrease in the

user costs. As the traffic demand is low in rural areas compared to urban areas, the work zone capacity does not affect the rural user costs as can be seen in fig 5.13. The discount rate affects urban as well as rural traffic scenarios. An increase in the discount rate reduces the overall costs.

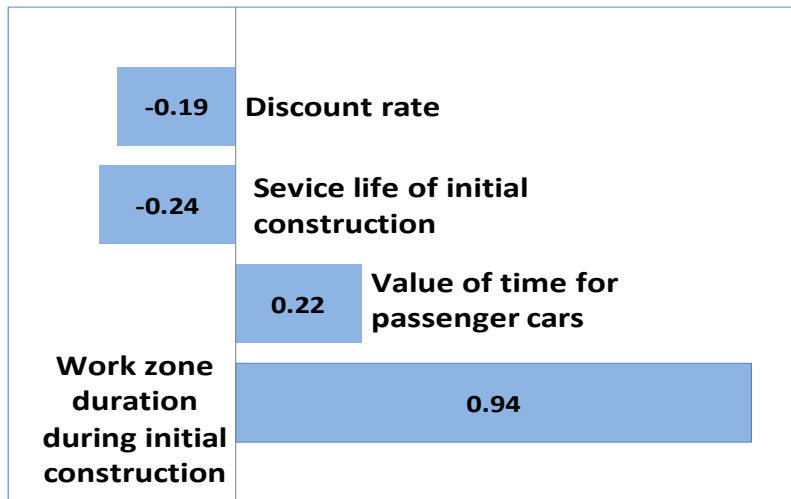


Fig.5.13. Correlation coefficient plot for Fully precast prestressed BAS in Rural

As 60% of the AADT is constituted of passenger cars, the value of time for passenger cars which is taken as \$ 13.37/hr (FHWA recommended) also affects the rural user costs as shown in figures 5.13 and 5.15.

It should not be inferred that the value of time for passenger cars does not affect the urban traffic. Due to the existence of high volumes of traffic in the urban scenario, work zone characteristics highly influence the final user costs than the value of time for passenger cars.

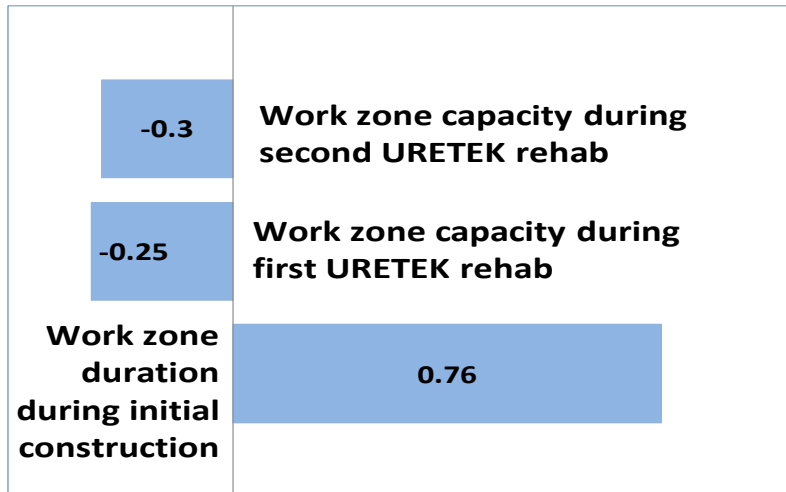


Fig. 5.14. Correlation coefficient plot for BAS-ES in Urban

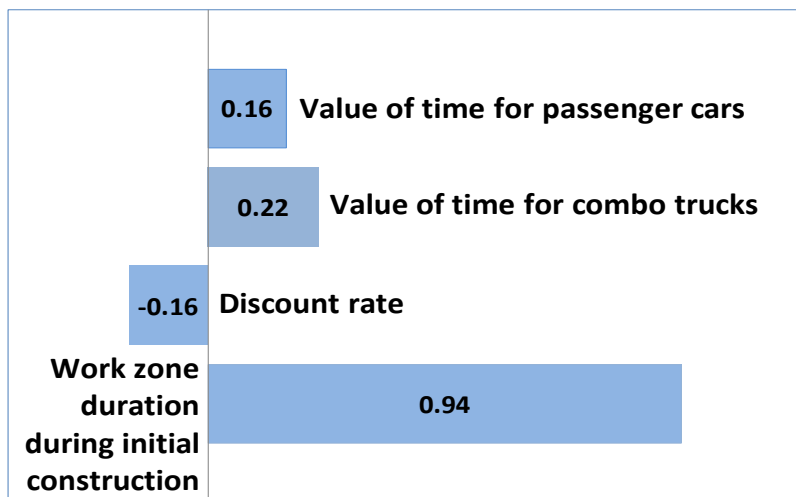


Fig. 5.15. Correlation coefficient plot for BAS-ES in Rural

The sensitivity analysis results for the BAS-ES are shown in figures 5.14 and 5.15. The results are similar to the Standard MoDOT BAS results as similar work zone durations and similar rehabilitation activities are assumed for both the design alternatives. The work zone duration during initial construction (30 days) influences the user costs in both the traffic patterns more than any other user cost parameters.

It can be clearly seen from the results in fig. 5.14 than in areas with high volumes of AADT like the urban scenario; work zone capacity and its duration directly affect the final user costs. While in the rural traffic scenario as AADT counts are comparatively low, the value of user time for both combo trucks and passenger cars affect the final output more than the work zone parameters.

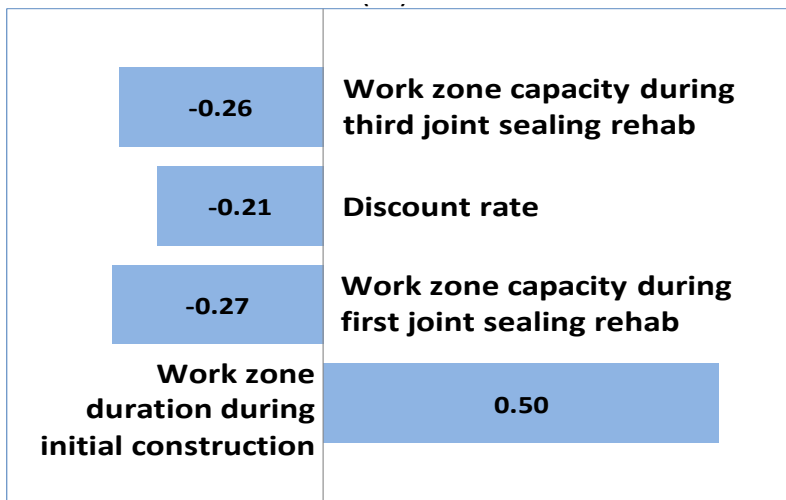


Fig 5.16. Correlation coefficient plot for CIP Topped PC Prestressed BAS in Urban

The CIP Topped PC Prestressed BAS has similar work zone characteristics as the Fully Precast Prestressed BAS except that it has additional 2 day work zone duration during the initial construction period. The work zone duration and capacity again influence the costs in the urban scenario.

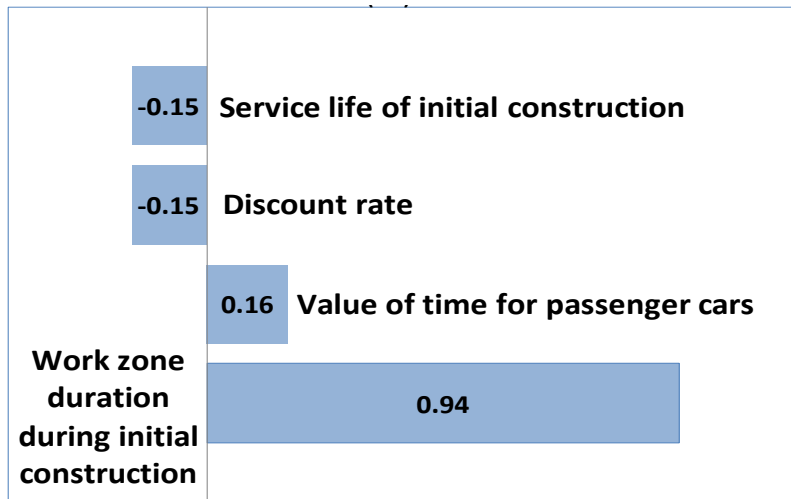


Fig 5.17. Correlation coefficient plot for CIP Topped PC Prestressed BAS in Rural

The work zone capacity during the rehabs influences the urban scenario alone as queue formation takes place only in high volume urban traffic but not in the rural traffic scenario. Queue formation leads to additional user costs like queue dissipation costs, speed change and delay costs. As AADT counts are low in rural areas, the value of time for passenger cars again has a significant effect on the total costs next to the work zone duration.

5.4. Influence of discount rate on costs

In this study different discount rates are considered to study the influence of discount rate on the costs. FHWA recommended a discount rate of 4%; MoDOT suggested a 7% discount rate and an additional 10% discount rate are selected to evaluate the effect of discount rate on dollar value. Both agency and user costs decreased with an increase in the discount rate. As can be seen from table 5.5 the decrease in costs was rapid from a

discount rate of 4% to 7%. A further increase in discount rate from 7% to 10% did not cause any significant cost drop. Looking at the figures in table 5.5, the costs calculated using a 10% discount rate showed that a further increase in the discount rate beyond the 10% mark is not going to have a dominant effect on the costs. As higher costs are recorded in urban traffic the discount rate effect on the costs is clearly visible in the urban scenario than in rural.

The difference in user costs in the urban scenario between deterministic and probabilistic analysis as discussed in previous sections is because of the additional inputs involved in the calculation of user costs; more the inputs involved in calculations, greater the uncertainty. This is the main reason behind the pronounced decrease in the user costs compared to the decrease in the agency costs with increasing discount rates.

Table 5.6. Life cycle agency, user and total costs for urban and rural traffic based on discount rates 4%, 7% and 10% for all BAS alternatives and assumed rehabilitation strategies.

Net Present Value (x \$1,000)			Standard MoDOT BAS			BAS-Elastic Soil Supported			Fully Precast Prestressed BAS			CIP Topped PC Prestressed BAS		
			Agency Cost	User Cost	Total Cost	Agency Cost	User Cost	Total Cost	Agency Cost	User Cost	Total Cost	Agency Cost	User Cost	Total Cost
Urban	Deterministic Results	4%	\$65	\$73	\$139	\$50	\$74	\$125	\$56	\$43	\$98	\$60	\$47	\$106
		7%	\$63	\$69	\$132	\$48	\$70	\$118	\$55	\$37	\$92	\$59	\$41	\$100
		10%	\$62	\$67	\$129	\$46	\$68	\$114	\$54	\$35	\$89	\$58	\$39	\$97
	Probabilistic Results	4%	\$66	\$83	\$149	\$50	\$83	\$133	\$55	\$53	\$109	\$60	\$58	\$117
		7%	\$63	\$72	\$136	\$48	\$75	\$123	\$55	\$43	\$98	\$59	\$47	\$106
		10%	\$62	\$70	\$132	\$47	\$70	\$117	\$54	\$37	\$91	\$58	\$42	\$100
Rural	Deterministic Results	4%	\$65	\$2	\$68	\$50	\$3	\$53	\$56	\$1	\$57	\$59	\$2	\$61
		7%	\$63	\$2	\$65	\$48	\$2	\$50	\$55	\$1	\$56	\$59	\$1	\$60
		10%	\$62	\$2	\$64	\$46	\$2	\$49	\$54	\$1	\$56	\$58	\$1	\$60
	Probabilistic Results	4%	\$65	\$2	\$68	\$50	\$3	\$53	\$55	\$1	\$57	\$60	\$2	\$61
		7%	\$63	\$2	\$66	\$48	\$2	\$50	\$55	\$1	\$56	\$59	\$1	\$61
		10%	\$62	\$2	\$65	\$46	\$2	\$49	\$54	\$1	\$56	\$59	\$1	\$60

5.5. Advantages of precast slabs over cast-in-place slabs:

User cost calculations showed that work zone costs amount for a major portion of the overall life cycle costs. The precast slabs have an inherent advantage over the cast in place slabs in lower work zone durations, reduced work zone durations lead to lower work zone user costs. Over the analysis period of 40 years, the cast in place slabs have an overall work zone duration of 36 days, whereas the precast slabs have a 24 day work zone duration. The deterministic analysis results showed that the work zone cost differences between the cast in place slabs and precast slabs in the urban scenario is \$28,000, whereas in the rural scenario it was \$1,000. The probabilistic analysis results depicted a 90% probability for the work zone cost differences between precast slabs and cast in place slabs in the urban scenario to be \$40,000.

5.6. Summary conclusions:

Results from this analysis demonstrated that it is possible to use LCCA to study the cost to the agency as well as the users for competing BAS design alternatives and rehabilitation strategies. RealCost, originally developed by the FHWA for studying life cycle costs and cost-effective investment strategies in pavement technologies has been very effective for analyzing similar challenges in evaluating bridge approach slabs.

The probabilistic analysis results calculated using RealCost software did not differ from deterministic results when agency costs are concerned, higher discrepancy was found in user costs as more parameters are involved in calculation. The difference between the

deterministic and probabilistic results in the urban scenario is around \$5,000-\$6,000. In the rural scenario, the difference between deterministic and probabilistic analysis is of the order of a few hundred dollars. Results from the sensitivity analysis showed that work zone duration and its capacity influence the user costs both in urban as well as rural traffic patterns. Net present value (NPV) is calculated using three different discount rates to study the effect and extent of discount rate on the present value; the results showed that with an increase in discount rate, the present value decreased.

When only present values of agency costs are considered, BAS – Elastic Soil Support design offers the lowest cost option of the four alternates studied. When only present value of user costs are considered, Fully Precast Prestressed – BAS offers the lowest cost option of the four alternates studied. When present value of total costs are considered, the BAS – Elastic Soil Support design is the most cost-effective when AADT counts are low, such as with rural traffic demand. When present value of total costs are considered, the Fully Precast Prestressed - BAS design is the most cost-effective when AADT counts are high, such as with urban traffic demands.

6. CONCLUSIONS

6.1 Precast Prestressed bridge approach slabs systems:

6.1.1 Fully Precast Prestressed BAS:

The fully precast prestressed slab was designed based on service limit state design and checked for ultimate moment requirements. Tandem loading along with lane loading was proved to be critical than truck loading. Some of the key features of this design are

- The slab panels are 12'' deep, 4' or 6' in width and 25' long. Three 270 ksi strands are used per unit feet width of the slab. The initial prestressing force in the slab is 81 kips/unit feet width.
- The cracking moment (M_{cr}) which causes cracking in the bottom fiber during service loading is calculated and is successfully checked with the condition: $\phi M_n > 1.2 M_{cr}$
- Deflections due to initial prestressing force, self weight of the slab, lane load and tandem load are calculated. Heuristic rule is used to calculate the additional long term deflection. The slabs showed cambers (upward deflections) during initial prestressing and self weight deflection during prestressing transfer checks. All the deflection values are proved to be within the satisfactory limits. The service load deflections were 85% lesser than the ACI limit deflections.
- The shear resistance of the concrete was proved to be critical than the web shear cracking resistance. The factored shear resistance ($\phi v_n = 200$ psi) of the concrete slab is twice larger than the shear at ultimate ($v_u = 65$ psi).

- The factored nominal moment ($\phi M_n = 930$ kip-in) obtained is approximately equal to the moment at ultimate ($M_u = 957$ kip-in)

The precast prestressed slab panels might require diamond grinding after installation in order to achieve smooth rideability. Base preparing operations should be carefully carried out to prevent alignment issues between the slab panels. It has a work zone duration of 15 days, the least among the four BAS alternatives.

6.1.2 CIP Topped PC Prestressed BAS:

The cast in place precast prestressed slab was also designed for service limit state design and checked for ultimate moment requirements. Tandem loading along with lane load is considered in the design. Some of the key features of the design are:

- The slab panels are 10'' deep, 4' or 6' in width and 25' long. A 2'' cast in place topping is applied on site after the panels are installed. Three 270 ksi strands are used per unit feet width of the slab. The initial prestressing force in the slab is 81 kips/unit feet width.
- The cracking moment (M_{cr}) which causes cracking in the bottom fiber during service loading is calculated and is successfully checked with the condition: $\phi M_n > 1.2 M_{cr}$
- Deflections due to initial prestressing force, self weight of the slab, lane load and tandem load are calculated. Heuristic rule is used to calculate the additional long

term deflection. The slabs showed cambers (upward deflections) during initial prestressing and self weight deflection during prestressing transfer checks. All the deflection values are proved to be within the satisfactory limits.

- The shear resistance of the concrete was proved to be critical than the web shear cracking resistance. The factored shear resistance ($\phi v_n = 200$ psi) of the concrete slab is twice larger than the shear at ultimate ($v_u = 65$ psi).
- The slab is also checked for the horizontal shear force criterion. The shear stress along the precast slab and cast in place interface (v_{uh}) is calculated and is compared to the factored nominal horizontal shear strength (ϕv_{nh}). The calculated shear stress is found to be within the satisfactory limits.
- The factored nominal moment ($\phi M_n = 972$ kip-in) obtained is higher than the moment at ultimate ($M_u = 957$ kip-in)

The slab panels do not require diamond grinding as the composite topping applied on site mitigates alignment issues between slab panels. It has a work zone duration of 17 days, slightly higher than the Fully Precast Prestressed BAS.

6.2 Life cycle cost analysis:

RealCost, a FHWA developed software was used to calculate the life cycle costs for the four design alternatives. This is the first known application of this software to bridge approach slabs. The results from the deterministic and probabilistic analysis concluded the following.

- Both the deterministic and probabilistic results showed that the Standard MoDOT BAS (\$65,000) was the costliest and the BAS-ES (\$56,000) was the most economical when the agency costs are concerned.
- The cumulative probability results showed a 90% probability for the BAS incorporating elastic support to yield lowest costs to the agency.
- In both the urban and rural scenarios, the Fully Precast Prestressed BAS has a 90% probability to yield lesser user costs than the other design alternatives.
- The FPPC BAS has a probability of 87% to achieve urban user costs of \$70,000. The probability for the Standard MoDOT BAS and BAS-ES to achieve the same cost is as low as 30%.
- When the deterministic and probabilistic analysis results are compared, the urban user cost difference between the results ranged from \$9,000-\$10,000. The rural user costs difference was almost negligible. (\$50-\$100). The variation is due to the difference in traffic volumes. AADT: 18,826 (Urban). 2,520 (Rural).
- In the sensitivity analysis results, higher correlation values were obtained for work zone durations during initial construction. Other significant inputs include value of time for trucks and passenger cars, discount rate and work zone capacity.
- Both agency and user costs decreased with an increase in the discount rate. The decrease in costs was rapid from a discount rate of 4% to 7%. A further increase in discount rate from 7% to 10% did not cause any significant cost drop.
- The deterministic analysis results showed that the work zone cost differences between the cast in place slabs and precast slabs in the urban scenario is \$28,000, whereas in the rural scenario it was \$1,000.

- The probabilistic analysis results depicted a 90% probability for the work zone cost differences between precast slabs and cast in place slabs in the urban scenario to be \$40,000.
- When only present values of agency costs are considered, BAS – Elastic Soil Support design offers the lowest cost option of the four alternates studied. When only present value of user costs are considered, Fully Precast Prestressed – BAS offers the lowest cost option.
- When present value of total costs are considered, the BAS – Elastic Soil Support design is the most cost-effective when AADT counts are low, such as with rural traffic demand. When present value of total costs are considered, the Fully Precast Prestressed - BAS design is the most cost-effective when AADT counts are high, such as with urban traffic demands.

6.3. Recommendations for future work:

Precast prestressed bridge approach slab systems for approach slabs are not implemented by any state DOT's in the past except in demonstration projects by the Federal Highway Authority (FHWA). Therefore, construction related issues and the extent of prestressing force affect on differential settlement are not studied in the present thesis. In a typical life cycle cost analysis, environmental costs which take into account the project's impact on the environment are also included along with agency and user costs. However, in the present study the environmental aspect of life cycle costs is not studied. The environmental costs are hard to quantify and therefore are omitted during analysis by the highway departments.

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