

Finite Element Modeling of Bridge Approach and Transition Slabs

FINAL REPORT
September, 2002

Submitted by
HANI NASSIF, Ph.D., P.E.
Assistant Professor

Talat Abu-Amra, Graduate Research Assistant
Nirali Shah, Graduate Research Assistant

Dept. of Civil & Environmental Engineering
Center for Advanced Infrastructure & Transportation (CAIT)
Rutgers, The State University
Piscataway, NJ 08854-8014



NJDOT Research Project Manager(s)
Mr. Nick Vittilo
Mr. Robert Sauber

In cooperation with

New Jersey
Department of Transportation
Division of Research and Technology
and
U.S. Department of Transportation
Federal Highway Administration

DISCLAIMER STATEMENT

"The contents of this report reflect the views of the author(s) who is (are) responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the New Jersey Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation."

1. Report No. FHWA-NJ-2002-007	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle Finite Element Modeling of Bridge Approach and Transition Slabs		5. Report Date September 2002	
		6. Performing Organization Code CAIT/Rutgers	
7. Author(s) Hani Nassif, Talat Abu-Amra and Nirali Shah		8. Performing Organization Report No. FHWA-NJ-2002-007	
9. Performing Organization Name and Address New Jersey Department of Transportation P.O. BOX 600 Trenton, NJ 08625		10. Work Unit No.	
		11. Contract or Grant No.	
		13. Type of Report and Period Covered Final Report 06/01/2000 – 08/31/2002	
12. Sponsoring Agency Name and Address Federal Highway Administration U.S. Department of Transportation Washington, D.C.		14. Sponsoring Agency Code	
15. Supplementary Notes			
16. Abstract The objective of this research study is to identify the probable causes of cracking, location of cracks, factors influencing crack development, and to recommend new design alternatives that could reduce or eliminate crack development in approach and transition slabs. The study included the development of a finite element model that is based on soil-structure interaction using commercial computer software, ABAQUS. A field study was also conducted to determine the extent and probable causes of crack development in approach and transition slabs in different bridge sites. The data collected from field observations were compared with those predicted by the finite element model to determine the reliability and the consistency of the results predicted by the finite element model. Subsequently, a parametric study was developed in order to study the sensitivity of the model to several designs as well as soil parameters. Based on the results of the finite element model, several design alternatives were studied and compared to determine their effectiveness in reducing the possibility of crack development and tolerating higher vehicular loads. Two new design alternatives are recommended based on results from the parametric study and field observations.			
17. Key Words Approach Slab, Transition Slab, Cracks, Finite Elements		18. Distribution Statement	
19. Security Classif (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No of Pages 79	22. Price

Acknowledgements

The authors would like to acknowledge the help of Graduate Research Assistant Yasser Khodair as well as the collaboration of Prof. Nenad Gucunski. The authors would also like to acknowledge the efforts of Nick Vittilo and Robert Sauber of NJDOT for their comments and feedback.

TABLE OF CONTENTS

SUMMARY	1
INTRODUCTION.....	1
OBJECTIVE	2
SCOPE.....	2
BRIDGE APPROACH PROBLEMS.....	3
Differential Settlement.....	3
Rotation and/or Lateral Movement of the Abutment.....	3
Slope Failure	4
Seepage.....	4
Thermal Forces	4
Foundation Settlement	5
Poor Design of Structural Components.....	5
CRITICAL ITEMS IN THE DESIGN AND CONSTRUCTION OF BRIDGE	
APPROACHES	5
Embankment Foundation	5
Abutment Type.....	6
Non-Integral Abutment	6
Integral Abutments	8
LITERATURE SURVEY	8
FIELD SURVEY	13
FINITE ELEMENT MODELING OF REINFORCED CONCRETE	
APPROACH AND TRANSITION SLABS	18
Geometry	18
Loading and Boundary Conditions	21
Soil-Structure Interaction.....	22
Finite Element Analysis.....	22
Bridge Entrance	23
Bridge Exit.....	24
Discussion of Results.....	25
Cracking of Approach and Transition Slabs Due To Settlement	25
Verification of FE Model	27
Skewed versus Non-Skewed FE Model	27
PARAMETRIC STUDY.....	29
Design Parameters	29
Effect of Concrete Compressive Strength, f_c	29
Effect of Steel Rebar Yielding Stress, F_y	31
Effect of Steel Rebar Area, A_s	31
Effect of Slab Thickness.....	31
Effect of Soil Settlement.....	32
Bridge Entrance	32
Bridge Exit.....	33
DESIGN ALTERNATIVES.....	35
Design Alternative 1. Constant Thickness Design	35
Design Calculations	35

Design Alternative 2. Embedded Beam Design	40
Design Calculations:	40
Design Alternative 3. Deep Beam Design	44
Design Calculations:	44
Reinforcement Summary for all design options.....	47
Existing	47
Constant thickness.....	47
Embedded Beam	47
Deep Beam	47
Comparison Between Design Alternatives with No Settlement.....	48
Comparison Between Design Alternatives with Settlement	50
SUMMARY	51
CONCLUSIONS	52
RECOMMENDATIONS	52
FUTURE RESEARCH.....	53
APPENDIX A.....	55
APPENDIX B.....	66
REFERENCES.....	68

LIST OF FIGURES

Figure 1. Cross section of typical NJ approach and transition slabs.	2
Figure 2. Rotation of abutment ⁽²⁸⁾	4
Figure 3. Typical vertical sand drain ⁽²⁸⁾	6
Figure 4. Typical closed abutment ⁽²⁸⁾	7
Figure 5. Typical stub abutment on pile ⁽²⁸⁾	7
Figure 6. Typical spill-through abutment ⁽²⁸⁾	8
Figure 7. Typical integral abutment ⁽²⁸⁾	8
Figure 8. Recommended sequence for embankment-abutment construction ⁽²⁷⁾	10
Figure 9. Typical transverse cracks on bridge 4.	15
Figure 10. Photograph of transverse cracks (1) and (2) in the shoulder and right lane on bridge 4.	15
Figure 11. Photograph of transverse cracks (3) in the middle lane on bridge 4.	16
Figure 12. Photograph of transverse cracks (4) and (5) in left lane and outside lane on bridge 4.	16
Figure 13. Photograph of transverse crack (6) in left lane on bridge 4.	17
Figure 14. Schematic diagram of transverse cracks on bridge 3.	17
Figure 15. Transverse cracks on bridge 15.	18
Figure 16. Plan view of approach and transition slab.	19
Figure 17. Reinforcement details for existing design approach and transition slabs.	20
Figure 18. Plan view of the FEM formulation of the approach slab and transition slab.	21
Figure 19. HS-20 design truck load from AASHTO bridge design specification.	21
Figure 20. An isometric view of the FE model of soil structure interaction showing edge springs.	22
Figure 21. The FE model representing soil structure interaction using spring elements.	22
Figure 22. Location of cracked elements at bridge entrance due to an application of 1.66 HS20 load.	23
Figure 23. Cracked Elements.	23
Figure 24. Tensile axial stresses for cracked elements in transition, due to the application of 1.66 HS20 load.	24
Figure 25. Location of cracked elements at bridge exit due to 4.3 HS20 truck load.	24
Figure 26. Position of the truck at bridge entrance and bridge exit.	26
Figure 27. FE model for (a) skewed and (b) straight approach and transition slabs.	28
Figure 28. Variation of tensile axial stress versus front axle distance for skewed and straight approach slabs at element A shown in Figure 27.	28

Figure 29. Variation of tensile axial stress versus front axle distance for skewed and straight approach slabs at element B shown in Figure 27.	29
Figure 30. Variation of tensile axial stress for critical elements under the effect of a moving HS-20 truck load model ($f'_c = 4500$ psi).	30
Figure 31. Variation of tensile axial stress for critical elements under the effect of a moving HS20 truck load model ($f'_c = 5500$ psi).	31
Figure 32. Design of transverse beam.	35
Figure 33. Design of longitudinal slab.	37
Figure 34. Reinforcement details for proposed constant thickness slab.	39
Figure 35. Design of transverse beam.	40
Figure 36. Design of longitudinal slab.	41
Figure 37. Reinforcement details for proposed embedded beam design.	43
Figure 38. Reinforcement details for proposed deep beam design.	46
Figure 39. Truck location for first cracked element.	48
Figure 40. Bottom layer of approach slab under tension at truck's wheel loads.	48
Figure 41. Top layer of approach slab under compression at truck's wheel loads.	48
Figure 42. Load versus stress for all design options as compared to the existing design.	49
Figure 43. Load versus deflection for existing, constant thickness, deep beam and embedded beam designs.	50
Figure 44. Critical areas at approach slab for sensor locations.	54
Figure 45. Transeverse cracks on bridge 5.	55
Figure 46. Photograph of a transverse crack in shoulder, at exit on bridge 5.	56
Figure 47. Photograph of a transverse crack in right lane, at exit on bridge 5.	56
Figure 48. Photograph of a transverse crack in middle lane, at exit on bridge 5.	57
Figure 49. Photograph of a transverse crack in left lane, at exit on bridge 5.	57
Figure 50. Transverse crack development on bridge 6.	58
Figure 51. Photograph of a transverse crack in shoulder, bridge exit on bridge 6.	58
Figure 52. Photograph of a transverse crack in right and middle lanes, at entrance on bridge 6.	59
Figure 53. Photograph of a transverse crack in right lane, at exit on bridge 6.	59
Figure 54. Transverse crack development on bridge 12.	60
Figure 55. Photograph of a transverse crack in shoulder, at exit on bridge 12.	60
Figure 56. Photograph of transverse crack in middle and left lanes, bridge 12.	61
Figure 57. Transverse crack development on bridge 14.	61

Figure 58. Photograph of a transverse crack on bridge 14.....	62
Figure 59. Photograph of a transverse crack in right lane, at entrance, on bridge14.	62
Figure 60. Photograph of a transverse crack in right lane, at exit on bridge 14.	63
Figure 61. Photograph of a transverse cracks in middle and left lanes, at exit, bridge 14.....	63
Figure 62. Photograph of a transverse cracks in right lane on bridge 14.	64
Figure 63. Photograph of transverse cracks in right lane, at entrance on bridge 14.	64
Figure 64. Photograph of a transverse crack in right lane, at exit on bridge # 14.	65
Figure 65. Detail of existing design of approach slabs.	66
Figure 66. Detail of existing design of transition slabs.....	67

LIST OF TABLES

Table 1. List of bridges located on I-295 southbound used in the field survey.....	14
Table 2. Cracking of A/T slab for loading at bridge entrance and different settlement cases.	26
Table 3. Cracking of A/T slab for loading at bridge exit and different settlement cases.	27
Table 4. Comparison of crack location as a percentage of total slab length.	27
Table 5. Change in the cracking load due to an increase in concrete strength, f'_c	31
Table 6. Reduction in tensile axial stress in elements due to increase in slab thickness.....	32
Table 7. Summary of the effect of soil settlement on the maximum tensile stresses induced in the critical elements.	33
Table 8. Effect of soil settlement on the maximum tensile stresses in the critical elements.	34
Table 9. Summary of the effect of the various parameters on the cracking load.	34
Table 10. Summary of the effect of the various parameters on the cracking load.	34
Table 11. No settlement versus 30% settlement of soil underneath approach slab.....	51

List of Abbreviations and Symbols

a	Effective moment arm
A	Approach slab
A_s	Area of Steel
A/T	Approach and Transition Slab
b	Width of concrete section
d	Effective depth of concrete section
f'_c	Compressive strength of Concrete
F	Front Axle (8 kips)
F_y	Yield strength of steel
l_n	Effective span length
L	Span length
m	Distance of cracked element from SIDE D
M	Middle Axle (32 kips)
M_n	Nominal moment
M_u	Ultimate moment
n	Distance of cracked element from SIDE D
P	Wheel load
R	Rear Axle (32 kips)
S	Spacing between beams
S_h	Maximum permissible spacing between horizontal bars
S_v	Maximum permissible spacing between vertical bars
T	Transition slab
w	Dead load
W	Wing wall
x	Distance of cracked element from SIDE B
y	Distance of cracked element from SIDE B
ϕ	Load factor

SUMMARY

The crack development in reinforced concrete bridge approach and transition slabs has been a persistent problem for many years. Transverse and longitudinal cracking has led to distress of concrete approach slabs, which results in a decrease of their life expectancy and increased maintenance costs for the repair and stabilization of the system. Cracks have been attributed to several causes such as differential settlement between the highway pavement and bridge deck as well as void development under the approach slabs.

The objective of this research study is to identify the probable causes of cracking, the location of cracks, and factors influencing crack development and to recommend new design alternatives that reduce or eliminate crack development in approach and transition slabs. The study included the development of a finite element model that is based on soil-structure interaction using commercial computer software, ABAQUS. A field study was also conducted to determine the extent and probable causes of crack development in approach and transition slabs at various bridge sites. The data collected from field observations were compared with those predicted by the finite element model to determine the reliability and the consistency of the results. Subsequently, a parametric study was developed in order to study the sensitivity of the model to several designs as well as soil parameters.

Finally, based on the results of the finite element model, several design alternatives were studied and compared to determine their effectiveness in reducing the possibility of crack development and tolerating higher vehicular loads.

INTRODUCTION

Bridge approach slabs provide a transitional roadway between pavement and the actual structure of the bridge as shown in Figure 1. This transition is crucial in reducing the dynamic effects imposed on the bridge by heavy truckloads. However, due to the effect of heavy impact loads, coupled with unknown or inadequate soil conditions (e.g. settlement, poor fill material, inadequate compaction, poor drainage, etc.), a number of approach slabs in the State of New Jersey have exhibited transverse structural cracking. This type of transverse cracking, as observed by site engineers of NJDOT as well as the Rutgers Team, occurs even on relatively newly constructed slabs. Despite the implementation of various design schemes for the approach and transition slabs, (e.g. alteration of the thickness of the approach slab, adding a number of rebars, increasing concrete strength, etc.), the structural cracking persisted. In general, this transition area, regardless of pavement type, has been the cause of poor riding quality and cracking of concrete.

Despite the widespread occurrence of bridge approach problems, only a small number of research studies have been performed on the subject. Few studies have been developed for evaluating the cracking behavior of bridge approach slabs in particular.

However, this problem is becoming an increasingly important topic in the effort to deal with the deteriorating infrastructure and rehabilitation of roadways. The disposition or repair of these deteriorating slabs involves clear financial and safety implications. To avoid high costs of replacement or repair, the evaluation must accurately reveal the present conditions and predict any further changes (or deterioration) in the applicable time span. Major decisions must be made to allocate the limited funds available for repair, rehabilitation, and/or replacement, on the basis of a detailed evaluation of the structural integrity of bridge approach slabs. Therefore, an efficient analytical model is needed for the prediction of the cracking behavior and the optimum design schemes in order to ensure crack free slabs. Moreover, it is important to understand the three-dimensional behavior of the slab-soil-vehicle interaction system. The effect of various parameters on the current design provisions used by NJDOT is studied in particular.

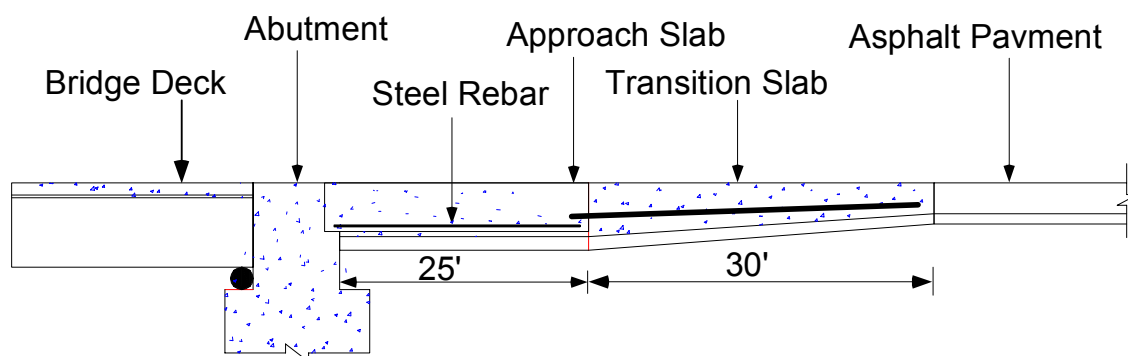


Figure 1. Cross section of typical NJ approach and transition slabs.

OBJECTIVE

The main objective of this study is to evaluate the cracking behavior of approach and transition slabs and the soil-slab-vehicle interaction system. The project identifies the procedure(s) and parameters that most accurately identify the severity of cracking in bridge approach and transition slabs. The end result is to provide a methodology that will enable New Jersey Department of Transportation (NJDOT) to successfully select the appropriate design schemes for bridge approach slabs:

SCOPE

The project identifies the procedure's and parameter's that affect the behavior of approach and transition slabs. The scope of the project is as follows:

1. Develop a detailed 3-D finite element model that will incorporate the nonlinear and cracking behavior of reinforced concrete as well as the inelastic soil properties. The model should simulate actual cracking behavior under various types of truck loading.
2. Compare results from the 3-D model with distress observed on actual structures. The research team will observe a few bridge approach slabs. Then it will

evaluate predictions from the FE Model with actual cracking patterns observed in the field.

3. Perform a comparative parametric study to optimize the slab design.
4. Recommend a new design criterion and practical approaches to design and building approach slabs.

BRIDGE APPROACH PROBLEMS

There are three major problems that can be associated with approach and transition slabs: 1) differential settlement between the highway pavement and bridge deck, 2) lateral movement of embankment, and 3) poor design of structural components. The following sections discuss in detail the contribution of each of these major problems on the adverse performance of approach slabs.

Differential Settlement

Bridge abutments are usually constructed with deep foundations that are considered stable. However, highway pavement is supported by an embankment composed of several fill layers that can settle, if not very well compacted. Differential settlement is affected by various factors such as consolidation of the embankment foundation, volume change of the approach embankment, lateral movement of approach embankment subsurface erosion of the approach fill and embankment, void development under the approach and transition slab, type of abutment, age, abutment skew, heavy traffic flow, and poor compaction quality of the fill material, especially near the abutment area. A major portion of approach slab displacement can be attributed to post-construction consolidation of material within the embankment foundation. Consolidation is a time- dependent process, which starts swiftly and proceeds at a lower rate with time. Consolidation has three phases, as follows:

- Initial compression, which takes place almost instantaneously as a load is applied to a soil mass.
- Primary consolidation, which is the gradual escape of water from the loaded soil.
- Secondary consolidation that occurs as a result of variation in the void ratio of constantly loaded soil due to a plastic readjustment of the soil particles. Secondary consolidation continues to carry on slowly, but, it is significant for organic soil in very soft clay.

Rotation and/or Lateral Movement of the Abutment

Some of the problems in bridge approaches can be associated with longitudinal, rotational, or vertical movements of the bridge abutment. Bridge abutments have been observe moving and rotating in the vertical direction both forward and backward as illustrated in Figure 2. The motion depends on the abutment type, abutment support system, foundation soil type, and construction sequence. Major factors, which may lead

to the rotation and/or movement of the abutment, are slope failure, seepage, thermal forces, and foundation settlement, as discussed by Wolde-Tinsae et al.⁽²⁸⁾.

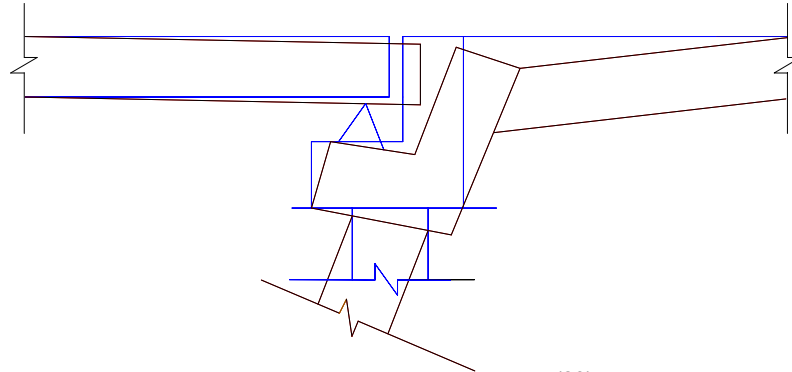


Figure 2. Rotation of abutment⁽²⁸⁾.

Slope Failure

Embankment instability may cause lateral and rotational movement of the abutment. An increase in shear stresses within the embankment sublayers will create lateral pressure on the abutments and supports. This movement may cause bending of the piles, tipping and cracking of walls, and damage to the abutments and expansion joints.

Seepage

Seepage causes reduction in soil resistance and an increase in the lateral pressure of saturated soil near the abutment, which in turn causes movement of the abutment. Cohesionless soils, particularly fine sand and silts, are most susceptible to seepage failure.

Thermal Forces

The performance of approach pavements can be seriously impaired when expansion joints do not function as designed. The primary function of the expansion joint is to relieve stress from expansive, contractive, or frictional forces initiated by temperature, moisture, and slab-base interface properties. Joints are most susceptible to infiltration of incompressible materials during the colder season, when joints tend to open up. Incompressible materials can also infiltrate through poorly sealed joints and/or with base materials pumped into the joint. Joint sealing is influenced by the movement and characteristics of the joint as well as the bonding between the sidewalls of the pavement slabs. During warmer weather, clogged joints inhibit movements of the neighboring slabs and large pressures develop at the abutments. These large pressures displace the pavement and soil adjacent to the abutment, creating a gap when the bridge retracts. Subsequent traffic loading pushes the pavement as well as underlying soil downward and towards the abutment to fill in the gap. This movement results in abrupt differential settlement at the pavement/abutment interface. Maintaining a properly functioning expansion joint is imperative in reducing temperature-induced pressures and consequently, pavement distress in bridge approach areas⁽¹⁶⁾.

Foundation Settlement

Ardani⁽²⁾ inspected several bridge approaches with moderate to severe degrees of settlement. The study concluded that settlement within the foundation is one of the major factors in the development of surface faults in bridge approaches. Post-construction consolidation of the embankment and abutment foundations could result in the lateral movement and rotation of the abutment. Uneven settlement under the abutment due to variations in depth of the compressible foundation layers may cause rotation of the abutment and, consequently, cause cracks in the abutment wall and openings in the joints that connect the wing-walls to the abutment⁽²⁸⁾.

Poor Design of Structural Components

Some of the problems that have resulted from poor design provisions include crack development in the approach slab, the formation of gaps between the back wall and roadway fill, additional stresses on the approach pavement, cracking of back walls, and cracking of wing walls.

CRITICAL ITEMS IN THE DESIGN AND CONSTRUCTION OF BRIDGE APPROACHES

To prevent or minimize bridge problems, careful attention should be given to design and construction techniques⁽²⁸⁾. Critical items in the design and construction of approach slabs are presented below:

Embankment Foundation

One of the major problems with approach slabs is soil settlement. Densification and pre-consolidation of weak and compressible soils (e.g. saturated soft clays), compressible silts, organic clays, and peats by preloading is one of the most prominent methods used in reducing settlement after construction. The effectiveness of preloading prior to construction of the approach pavement depends on the time available for consolidation under the surcharge load and the rate of settlement. Therefore, the sooner the embankment and surcharge are constructed, the more effective they are in attenuating the effect of settlement. In cases where the use of surcharge is uneconomical or if certain time limitations would limit its usage, vertical drains are used to speed the rate of settlement. This procedure is effective in thick homogeneous layers of clay where primary consolidation is the major part of settlement. However, in materials where secondary consolidation prevails, vertical drains are not effective. Two types of vertical drains have been used to accelerate water drainage from soil: 1) sand drains and 2) wick drains. The first type of drain is of 12 to 16 inches in diameter filled with sand, as demonstrated in Figure 3. The second type of drain consists of prefabricated porous plastic or cardboard tubes. The size and spacing for drains are determined for every foundation conditions.

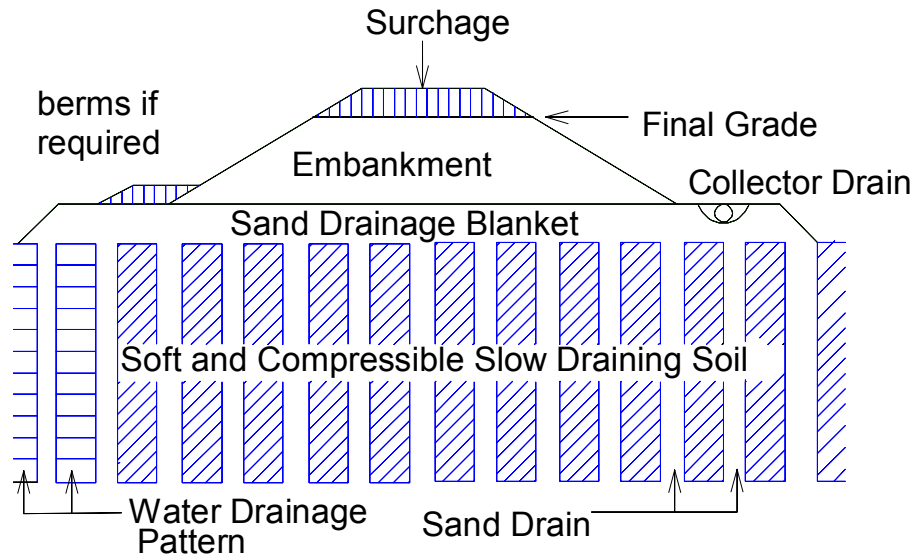


Figure 3. Typical vertical sand drain⁽²⁸⁾.

Abutment Type

The type of abutment affects the magnitude of approach settlement. An abutment is a substructure entity that holds up bridges and approaches. The two primary abutment types are: (1) non-integral abutments and (2) integral abutments. Integral abutments are rigidly connected to the bridge beams and deck with no expansion joint. The non-integral bridge abutment is separated from the bridge beams and deck by a mechanical joint that allows for the thermal expansion and contraction of the bridge.

Non-Integral Abutment

Hopkins and Deen⁽¹²⁾, NCHRP⁽²³⁾, and Timmerman⁽²⁵⁾ have discussed the effect of abutment types on the behavior of both bridges and approach slabs in several research studies. There are three major types for non-integral abutments used for bridges; they are closed abutments, stub or shelf abutments, and spill-through abutments.

A closed abutment has a wall that extends over the entire height of the embankment and must be constructed prior to the embankment as shown in Figure 4. It is difficult to compact the embankment fill near the abutment due to confined space. Closed abutments are also subjected to higher lateral earth pressures than other abutment types.

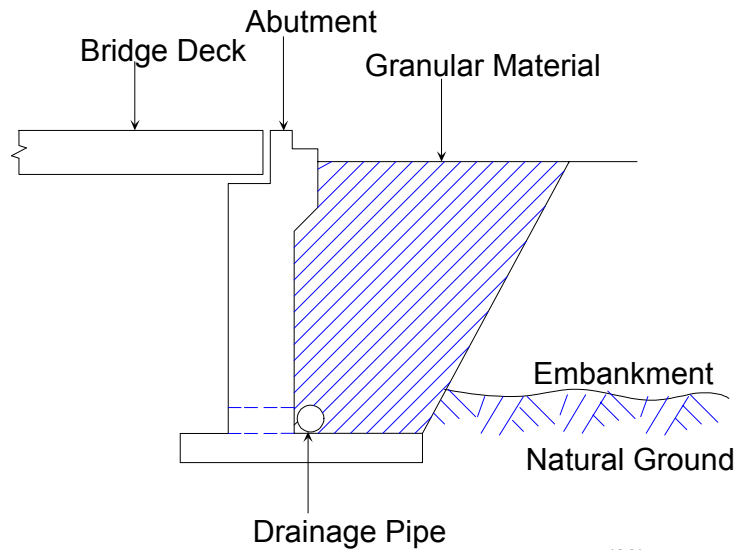


Figure 4. Typical closed abutment⁽²⁸⁾.

Stub or perched abutments are generally constructed after the embankment, whose height corresponds to the base of the abutment. This simplifies the fill compaction process, except in regards to the compaction of backfill behind the abutment. Such abutments may rest on a shallow foundation in the embankment or on piles, as displayed in Figure 5.

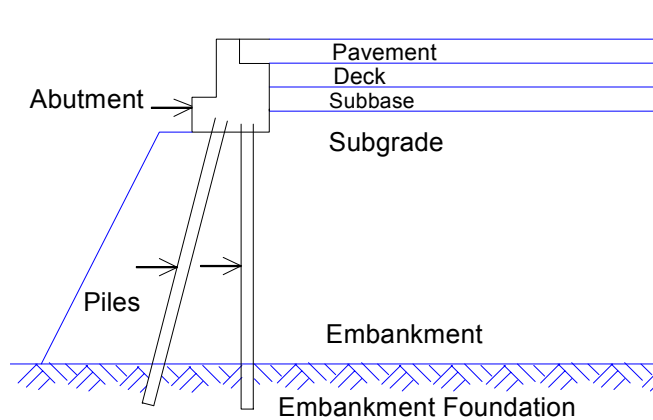


Figure 5. Typical stub abutment on pile⁽²⁸⁾.

Because stub or perched abutments do not extend to the entire height of the embankment, they experience the lowest lateral earth pressures of the three types. Pedestal or spill-through abutments are stub abutments supported on columns. This type of abutment must be constructed before the embankment as illustrated in Figure 6. The embankment fill will be built up on both sides of the supporting columns. It is difficult to compact the fill in the area near the abutment, especially between the columns. However, spill-through abutments experience lower lateral earth pressures than closed abutments, because there is no solid structure preventing the lateral movement of the soil. For the same reason, this lateral movement continues after

construction is complete⁽²³⁾. Spill-through abutments are associated with rougher approaches, as compared to the cantilever type.

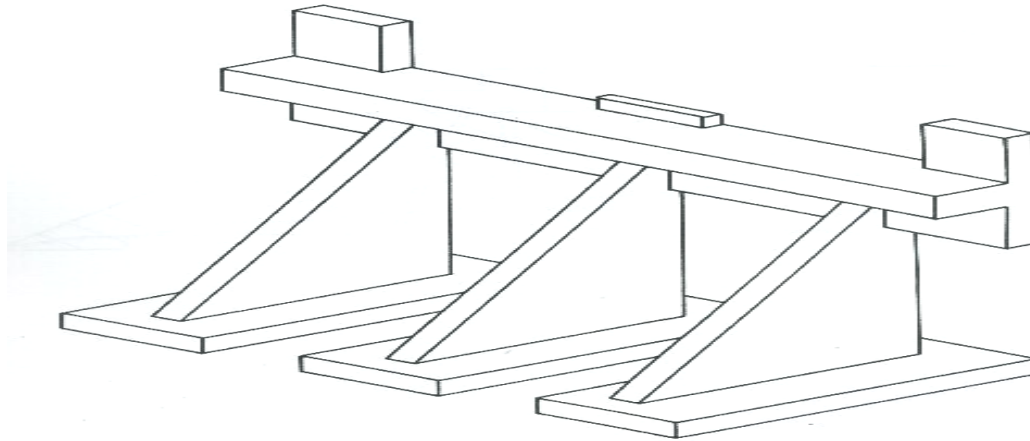


Figure 6. Typical spill-through abutment⁽²⁸⁾.

Integral Abutments

For integral abutments, the bridge and abutments are connected as a single structure with no expansion joint between them (Figure 7). While integral abutments add structural advantages, they also introduce thermal movements in the approach system.

Such thermal movements add to the approach bump problem. Integral abutments also require special attention to the lateral load imposed on the foundation piles by the horizontal movement which is induced by temperature cycles⁽²³⁾.

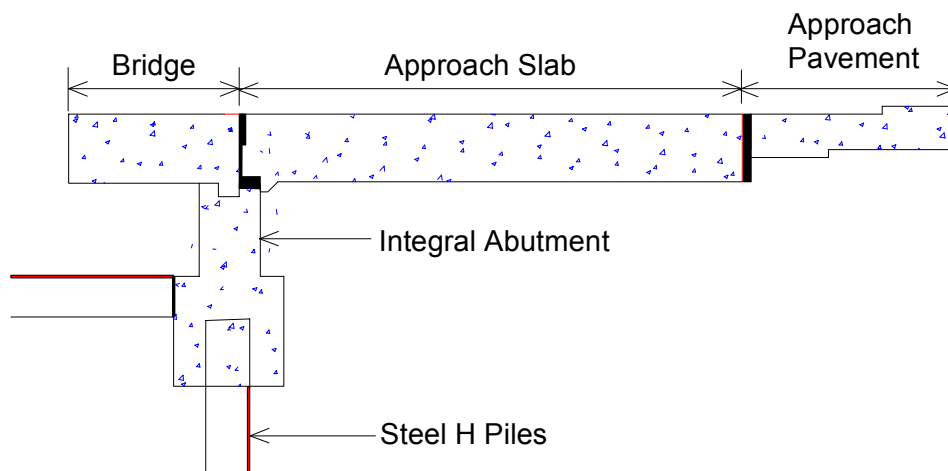


Figure 7. Typical integral abutment⁽²⁸⁾.

LITERATURE SURVEY

Different researchers have performed several comprehensive studies on the performance of approach slabs over the years. Allen⁽¹⁾ conducted a questionnaire that

was sent to all States concerning problems with bridge approaches. The survey included the following seven questions with a summary of the corresponding answers:

- (1) *"Is settlement of bridge approaches a major problem in your State?"* Most respondents mentioned that settlement is a major problem in their states.
- (2) *"Do you use some form of reinforced approach slab?" If so, is it successful?* All participants responded with a "yes" on this question except for Maryland DOT. However, the length and thickness and even the geometrical shape differed from one state to the other.
- (3) *"If reinforced slabs are used, how long are they?"* The length of approach slabs ranged from 20 to 60 feet. However, most of the States use approach slabs having a length of 20 to 30 feet with a thickness of 10 inches.
- (4) *"Are integral end-bents used in your state?" If so, have they performed well?"* The answers to this question varied from one State to the other. However, those States responding with a "yes" reported that integral abutment bridges have generally been successful in most cases.
- (5) *"Are special procedures used when backfilling around the end-bent? What are these procedures?"* While some States responded that there are no special procedures used for backfilling around the end-bent, others have certain requirements for the material used in backfilling and the compaction procedure that follows.
- (6) *"Are abutments on spread footers used in your State? If so, are they successful?"* The answer to this question varied widely from one State to the other. Some States mentioned that abutments on spread footers are not used in their States. Others mentioned that they use this kind of abutment and their success ranged from excellent to unsatisfactory.
- (7) *"Are there any other methods that your state uses to minimize this problem?"* The answers varied widely from one State to the other. Some States mentioned that settlement is a difficult problem because of the wide range of variables involved. Another suggested varying the length of approach slabs in order to minimize settlement. Others mentioned the use of wide expansion joints between approach slabs and concrete pavements, plus massive anchor blocks integral with concrete pavements abutting approach slabs.

The original ground subsidence and fill settlement are major causes of approach maintenance problems. The resulting recommendations included using select fill material for a distance of 45 m (150 ft) from the bridge, waterproofing the approach embankment, and using approach slabs 9 m (30 ft) long. The proposed approach slab should be doweled into the back-wall to ensure a watertight joint. In addition, the slab should be cantilevered over the wing-walls to minimize surface water infiltration.

Holtz⁽¹¹⁾ stated that for highway construction in areas of problem foundation soils, use of berms or flatter slopes, light weight fill materials, pile-supported roadways and embankments, removal of soft or problem materials and replacement by suitable fill, stabilization by consolidation of soft-foundation materials, chemical alteration or stabilization, and physical alteration/stabilization, including densification and reinforcement, becomes helpful.

Research conducted by Wahls⁽²⁷⁾ stated that the performance of a bridge approach is affected by the design and construction of the bridge deck, abutment, and foundation as well as by the roadway pavement system, embankment and embankment foundation. The study called for conducting consolidation tests in order to determine the rate of primary consolidation. This facilitates estimation and minimization of post construction settlement through use of preloading, temporary surcharges, wick drains, and stage construction. The author mentioned several methods for foundation improvement such as removal and replacement as well as an in-situ densification, which includes pre-compression, dynamic compaction, and compaction piles. The researcher also recommended allowing a waiting period after constructing the approach fill for the consolidation of the foundation soils before constructing the abutment and its foundation, except when the approach fill is underlain by rock. Moreover, the study included a special construction sequence for the abutment and embankment construction as demonstrated in Figure 8:

- Construct embankment slopes to ABCFG.
- Excavate to CDEF for end bent.
- Drive piles.
- Place 2-inch mortar bed or class “A” concrete along DE.
- Construct concrete end bent.
- Backfill to FJHG with select granular fill.

Furthermore, Wahls⁽²⁷⁾ recommended stage construction of the embankment in order to take advantage of the gain in shear strength that accompanies consolidation and avoid the overstress that might develop if approach fills are constructed on a very soft foundation.

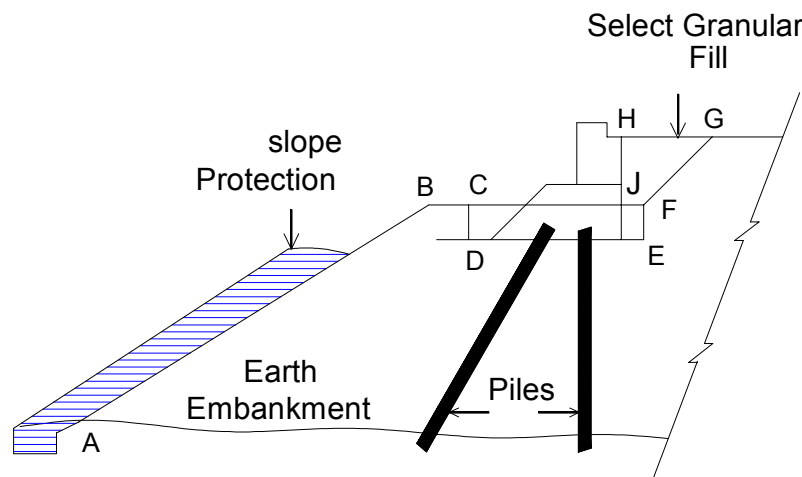


Figure 8. Recommended sequence for embankment-abutment construction⁽²⁷⁾.

Finally, the author mentioned the importance of maintenance for approach pavements and recommended that the minor settlement of reinforced concrete slabs should be repaired using bituminous or epoxy overlays. Likewise, slurry or grout may be pumped through holes drilled in the pavement or horizontal pipes inserted beneath the pavement. Furthermore, providing additional surface or internal drainage to keep water off the slopes is recommended for correcting the surficial erosion of embankments.

Also, the researcher suggested jacking and shimming the superstructure to its original position if an abutment settles excessively.

Kramer et al.⁽¹⁶⁾ summarized findings from various State DOTs in order to identify common problems of approach slabs in these States and their suggested remedies. The study also included several suggestions in order to mitigate the bridge approach settlement problem such as soil replacement, improvement by densification, improvement by grouting, and foundation improvement by surcharging. Additionally, it recommended selection of embankment materials, embankment slopes, surface water control, and embankment reinforcements as techniques for embankment improvement. The study also called for the use of select granular fill and stringent inspections of the placement and compaction of abutment back fill. The study concludes that bridge approach distress is mainly due to the consolidation of foundation soils, the compression of embankment fill, and localized soil settlement near the approach-abutment interface, attributable to inadequate compaction.

Schaefer et al.⁽²⁴⁾ conducted preliminary and extensive field investigations for void development under the approach slab to identify bridges with backfill subsidence problems in South Dakota. Voids under approach slabs in South Dakota varied in size from less than 1 inch to as large as 14 inches. The researchers identified two types of bridge abutments there. The primary type is an integral abutment bridge with a stub abutment supported on a single row of piles. The other type is a non-integral abutment bridge with a stub abutment on piles. According to this research, the primary approach system in South Dakota consists of reinforced concrete approach slabs tied to the abutments of bridges with dowel bars in an integral connection. The model test results linked cyclic movement of integral abutment bridges as the major cause of void development under the approach slabs. The researchers associated void development under approach slabs with integral abutment bridges as well. They also mentioned that there is no way to eliminate the development of void space under the approach slab when an integral abutment bridge approach system is used. Subsequently, they recommended returning to the use of non-integral abutment structures.

A recently completed NCHRP synthesis report on the settlement of approach slabs⁽⁵⁾ concluded that the bump at the end of the bridge affect 25 % of the bridges in the United States, and the estimated maintenance costs are at least \$100 million per year nationwide. The synthesis identifies some factors that tend to minimize the bump. These are abutment and embankment on strong soil, a long enough and strong enough approach slab, well compacted or stabilized fills, appropriate fill material, effective drainage, low embankments, good construction practice and inspection, and an adequate waiting period between fill placement and paving. Furthermore, bridges with abutments on spread footings have fewer bumps than those with abutments on deep foundations. It is mentioned that the combination of the first span and abutment on spread footings plays a role similar to that of the approach slab, and the sleeper slab substantiated this. The study also concluded that the use of approach slabs minimizes or eliminates the problem of the bump. The synthesis of

discussions with DOT engineers and of the survey responses identified the best current practice regarding the bump at the end of the approach slab as follows:

- (1) Treat the bump problem as a stand alone design issue and prevention as a design goal.
- (2) Assign the responsibility of this design issue to an engineer.
- (3) Stress teamwork and open-mindedness among the geotechnical, structural, pavements, construction, and maintenance engineers.
- (4) Carry out proper settlement versus time calculations.
- (5) If differential settlement is excessive, design an approach slab,
- (6) Provide for expansion/contraction between the structure and the approach roadway (fabric reinforcement or flow fill).
- (7) Design a proper drainage and erosion protection system.
- (8) Use and enforce proper specifications.
- (9) Choose knowledgeable inspectors, especially for geotechnical aspects.
- (10) Perform a joint inspection including joints, grade specifications, and drainage.

Hoppe⁽¹⁴⁾ conducted a literature review regarding issues on the use of design and construction of approach slabs. The researcher mentioned that the presence of an approach slab has no effect on the magnitude of differential settlement developed. Moreover, the report identified the primary function of approach slabs as a gradual transition or ramp between the fixed superstructure and the setting embankment. It also included a survey of State DOTs that focused on issues concerning bridge approach slabs. The survey summarized the practices of various DOTs regarding the use, design, and construction of approach slabs and compared them with those used by the Virginia Department of Transportation (VDOT). The results of the survey cited that half of the respondents reported a commonly used slab length of 20 ft with a thickness that ranges from 8 to 17 inches. As for the slab connection with bridge abutments, most of the respondents indicated the use of doweled or tied connections, and 43% use no mechanical attachments for conventional bridges. Dowelled connections are commonly used at integral bridges. Moreover, the survey included a description of fill specifications for bridge approaches where a number of responding DOTs specify select fill material for bridge approach embankments. However, 49% indicated the use of more stringent material specifications for the approach fill as contrasted with a regular highway embankment fill. A typical requirement is limiting the percentage of fine particles to reduce material plasticity and enhance drainage properties. Furthermore, the survey noted that advantages and disadvantages of approach slabs. Of the respondents, 81% of respondents quoted a smooth ride as a primary advantage of the use of approach slabs. A reduced impact on the back-wall was commonly given as a secondary advantage (41%), followed by enhanced drainage control (16%). Two States, Kentucky and Maryland, derive no clearly defined advantages from the use of approach slabs. Among primary disadvantages, the initial high construction cost was quoted by 75% of respondents, and maintenance problems, while settling approach slabs, were quoted by 52% of respondents. Other disadvantages included difficulties with staged construction and increased construction time. The study concluded that every State DOT has unique criteria that govern the use, design, and construction of bridge

approach slabs. It also emphasized that some differential settlement at bridge approaches is inevitable because of differing foundations beneath the bridge and the roadway. Finally, the report recommended the use of approach slabs in the initial construction of all structures on Virginia's highways with heavy traffic volumes. The approach slabs should be constructed to the full width (curb to curb) of roadway to minimize settlement.

FIELD SURVEY

A field survey was performed to gather information on bridge approach slab cracking problems across Southern New Jersey. Moreover, this information was used for the comparison and verification of the finite element model, which was developed based on a typical New Jersey approach and transition slab. The first task associated with this objective was to inspect a representative number of bridges and collect information about the location and extent of cracking and its probable causes. Over 40 bridges were visited, and site investigations were performed on the 18 bridges listed in Table 1. The information collected comprised the bridge location, bridge type, abutment type, location of cracks with respect to abutment, and number of lanes. The sites were inspected for cracking in all lanes of the approach slab and deterioration in the road pavement. More Figures are provided in Appendix A.

The inspections provided valuable information about prominent types of bridges in the State of New Jersey. Most of the bridges have approach and transition slabs on girders with non-integral abutments, and cracks were observed in the majority of these slabs.

Two types of cracks were observed in the approach and transition slabs: (1) transverse cracking perpendicular to the centerline of the roadway, which is the prevailing type, and (2) longitudinal cracking which forms at an angle approximately 30° - 40° to the centerline of the lane. These types of cracks are not as common as transverse cracking.

The first type of crack development in approach and transition slabs can be further classified into two primary categories. The first category corresponds to cracks ubiquitous in all lanes including the shoulders of the bridges. These types of cracks are observed near the abutment of bridges, especially across the skew side of the expansion joint between the bridge and approach slab. The second category corresponds to cracking in heavy traffic lanes, especially in the right and middle lanes. These types of cracks are located in transition slabs towards the asphalt pavement.

Table 1. List of bridges located on I-295 southbound used in the field survey.

BRIDGE No.	BRIDGE LOCATION
1	I-295 S, MILE 66.79, ROUTE 1, EXIT 67.
2	I-295 S, MILE 66.79, BAKERS BASIN ROAD.
3	I-295 S, MILE 66.31, ATMARK TRAIN BRIDGE.
4	I-295 S, MILE 64.75, EAST STATE STREET, EXIT 64.
5	I-295 S, MILE 64.15, NOTTINGHAM, EXIT 63.
6	I-295 S, MILE 61.56, AREANA DRIVE, EXIT 61.
7	I-295 S, MILE 60.36, 1195 ROUTE 29, EXIT 60.
8	I-295 S, OLDEN AVENUE, EXIT 62.
9	I-295 S, MILE 60.02, ANTECEDENT INTERCHANGE BRIDGE, EXIT 60
10	I-295 S, MILE 60.36, JCT 195, ROUTE 29, EXIT 60 A.
11	I-295 S, MILE 59.89, WATSONS CREEK RAILROAD BRIDGE.
12	I-295 S, MILE 57.27, BURLINGTON STREET.
13	I-295 S, MILE 56.83, US 130, EXIT 57.
14	BRIDGE OFF EXIT 43.
15	BRIDGE OFF EXIT 45.
16	BRIDGE OFF EXIT 56.
17	I-295 S, MILE 65.41, SLOAN AVENUE BRIDGE.
18	QUAKER BRIDGE ROAD ADJACENT US1.

Plan view of a bridge showing the bridge entrance, bridge abutment, and bridge exit. The diagram includes dimensions for the bridge deck, shoulders, and approach roads. A north arrow is shown at the top. The bridge entrance side has a 25.5' wide deck, a 19' wide shoulder, and a 17' wide approach. The bridge exit side has a 26.5' wide deck, a 29' wide shoulder, and a 25' wide approach. The total width of the bridge entrance is 58' and the total width of the bridge exit is 89'.

Figure 9. Typical transverse cracks on bridge 4.

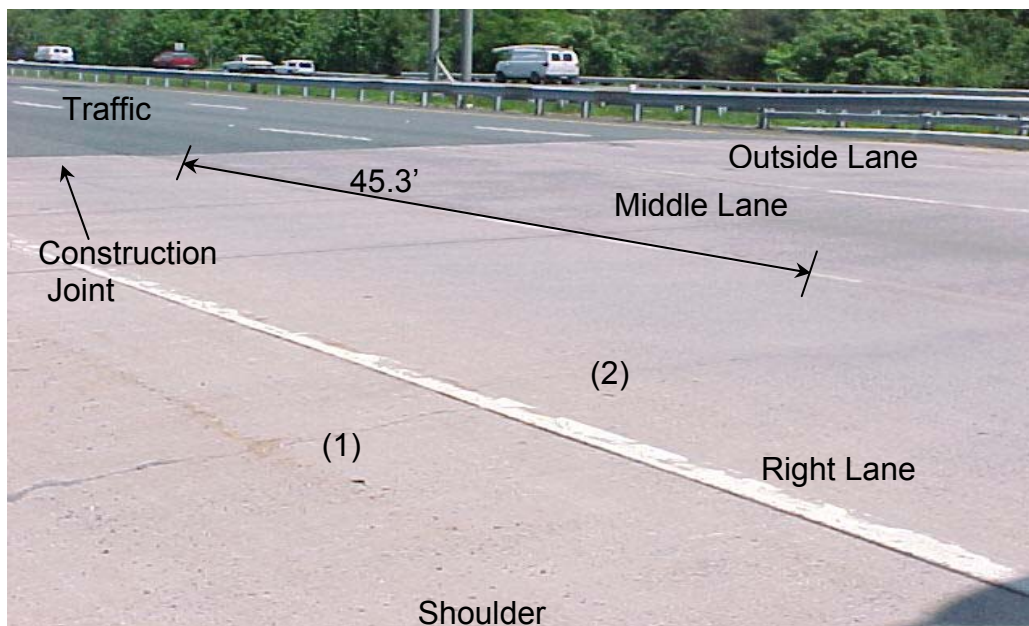


Figure 10. Photograph of transverse cracks (1) and (2) in the shoulder and right lane on bridge 4.

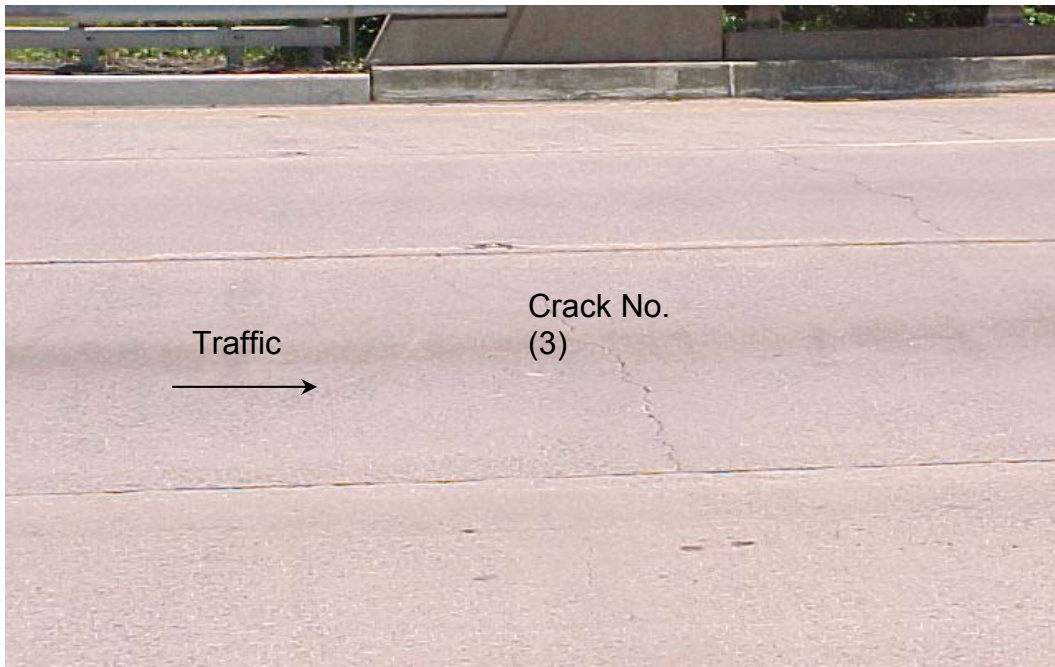


Figure 11. Photograph of transverse cracks (3) in the middle lane on bridge 4.

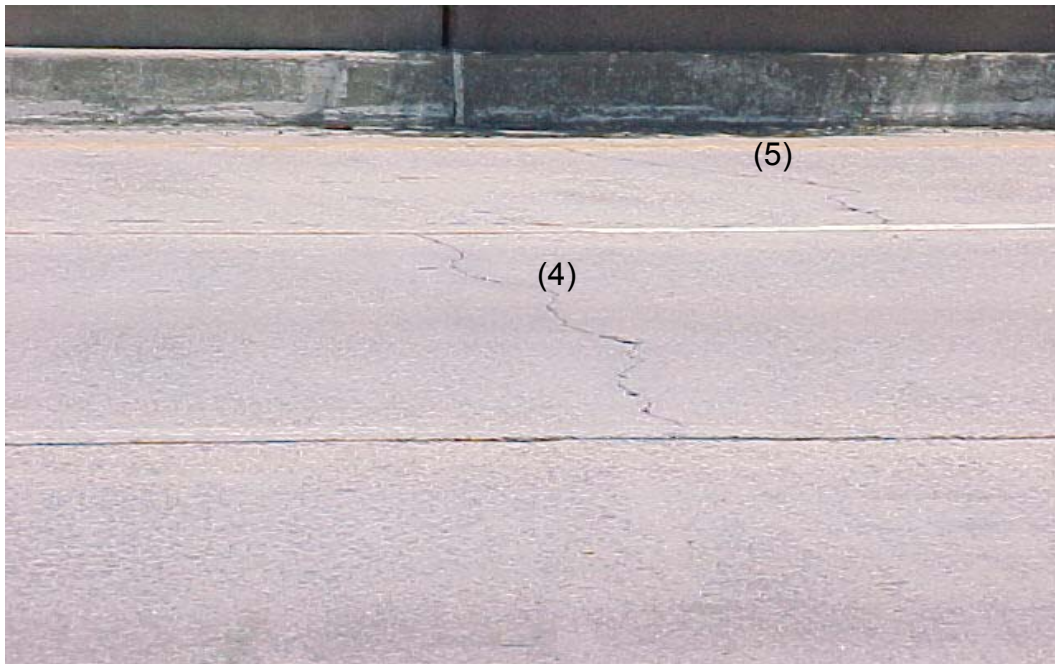


Figure 12. Photograph of transverse cracks (4) and (5) in left lane and outside lane on bridge 4.

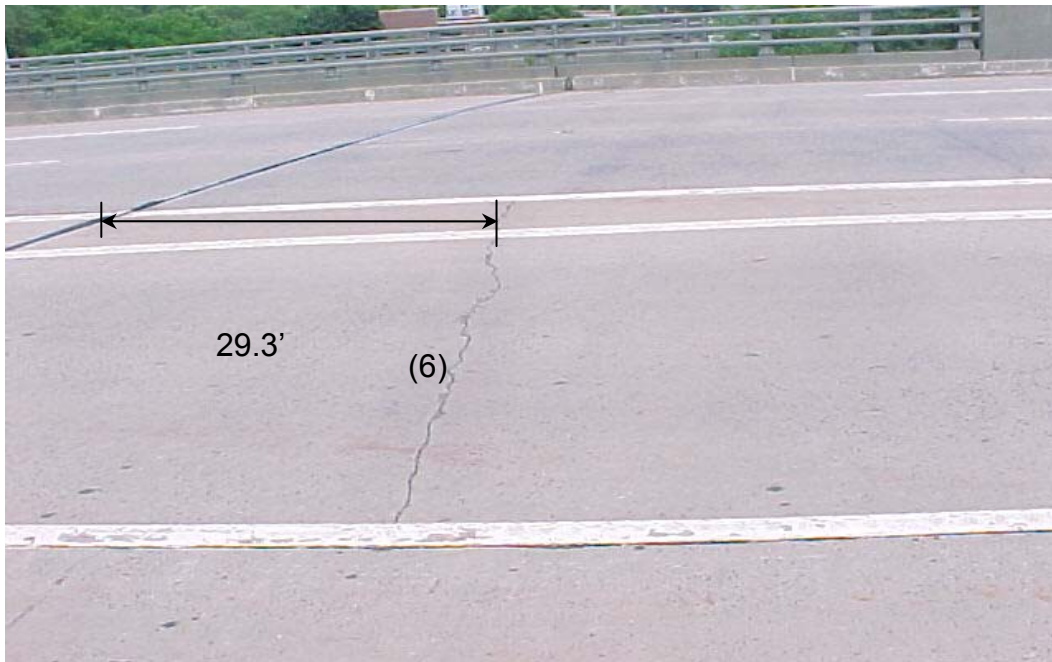


Figure 13. Photograph of transverse crack (6) in left lane on bridge 4.

Example of second category cracking is given in Figure 14 for bridge 3. This figure shows the distance of each crack from the asphalt pavement. Cracks are observed in heavy traffic lanes.

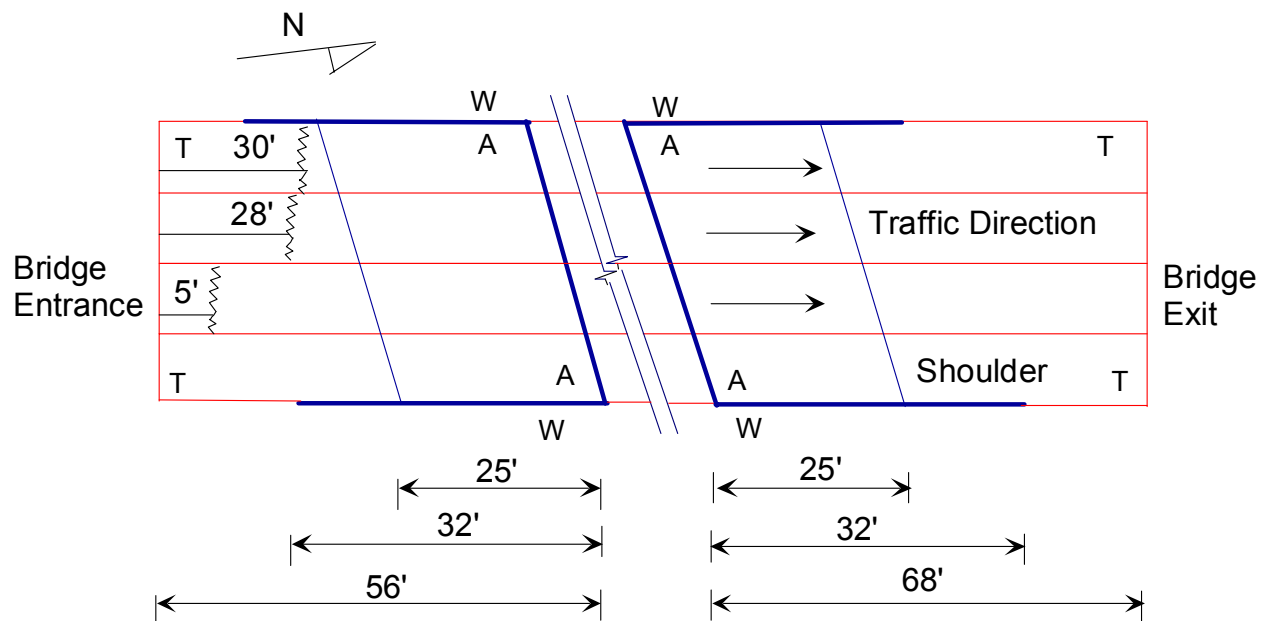


Figure 14. Schematic diagram of transverse cracks on bridge 3.

Another example of a bridge examined in order to verify location of cracking in approach and transition slabs is shown in Figure 15. Bridge 15 is composed of six lanes and two narrow shoulders. The length of the approach and transition slabs is 89 inches. The

bridge is a non-integral abutment bridge with a slab on a girder structural system. The significance of this structure is that it encompasses cracks both near the bridge abutments and towards the asphalt pavements. Moreover, the transition slab is straight across the road pavement and skewed across the bridge abutments. Most of the cracks are located in the first third of the slabs either towards the road pavement or towards the bridge abutments.

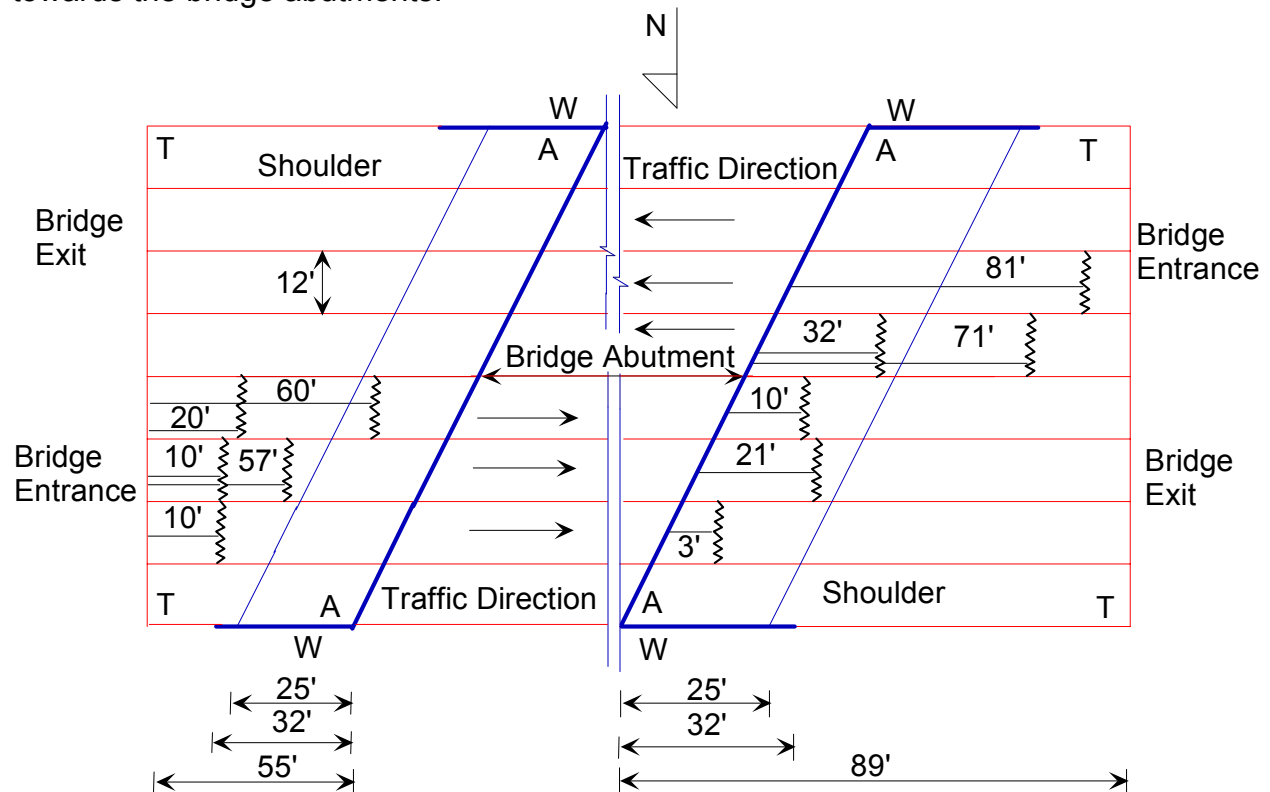


Figure 15. Transverse cracks on bridge 15.

FINITE ELEMENT MODELING OF REINFORCED CONCRETE APPROACH AND TRANSITION SLABS

Five aspects of an element characterize its behavior: family, degrees of freedom, number of nodes and order of interpolation, formulation, and integration. Each of these characteristics is described briefly below.

Geometry

A typical New Jersey approach and transition slab has a total length of 55ft on the shorter side as demonstrated in Figure 16. Moreover, the width of the approach and transition slab was limited to the width of a typical lane, because field observations revealed that most of the slabs were poured with construction joints between the lanes, including shoulders.

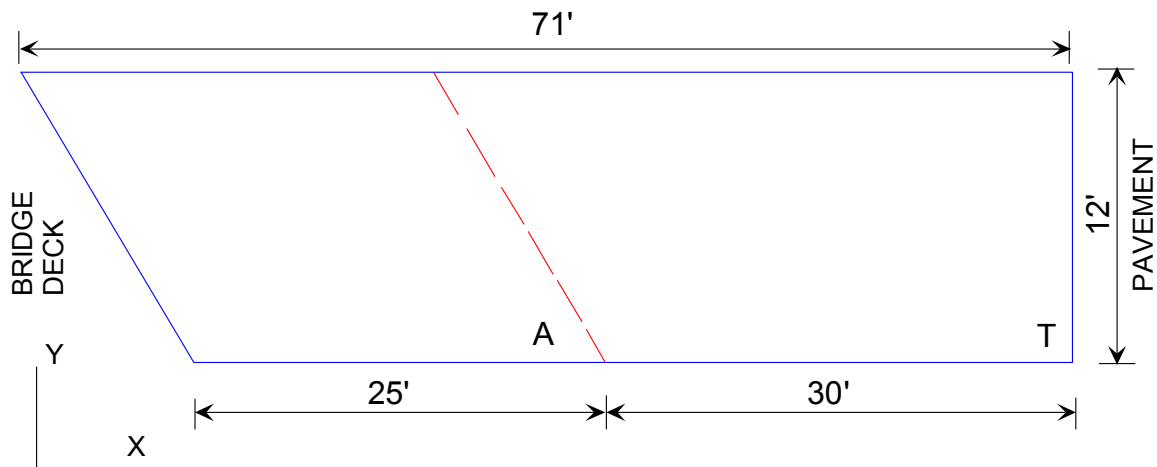


Figure 16. Plan view of approach and transition slab.

Typical drawings of existing approach slabs are shown in Appendix B. Using the drawings mentioned in Appendix B, Figure 17 depicts reinforcement details for the approach and transition slabs using plan, elevation, and cross section diagrams mentioned in Appendix B.

Since size of the mesh is one of the factors contributing to the FE model, arrangement of the elements was given special consideration. The model was divided into elements, each having a length of 10 inches along the x-axis and a length of 7.2 inches along the y-axis. The reinforced concrete slab is modeled using a four noded, reduced integration shell element (S4R). Soil underneath the approach and transition slabs is modeled using spring 1 type elements. The model consists of 1629 nodes and 1632 elements. The approach slab is shaped as a parallelogram with a length of 25 ft and a width of 12 ft. The transition slab has a trapezoidal shape with a smaller base of 30 ft and a width of 12 ft as shown in Figure 18.

For a stress/displacement simulation, there are degrees of freedom for the translations and rotations at each node. Particularly, there are three translation degrees of freedom: (1) translation in the X direction, (2) translation in the Y direction, and (3) translation in the Z direction. There are also three rotational degrees of freedom: (1) rotation about the X-axis, (2) rotation about the Y-axis, and (3) rotation about the Z-axis.

The S4R element type in ABAQUS has true curved shell elements and, therefore, requires accurate calculation of the initial curvature of the surface. The "top" surface of a shell is in the positive normal direction and is referred to as the SPOS face. The "bottom" surface is in the negative direction along the normal and is referred to as the SNEG face. ABAQUS not only defines the top and bottom surfaces of shell elements but also several locations along the cross section of each.

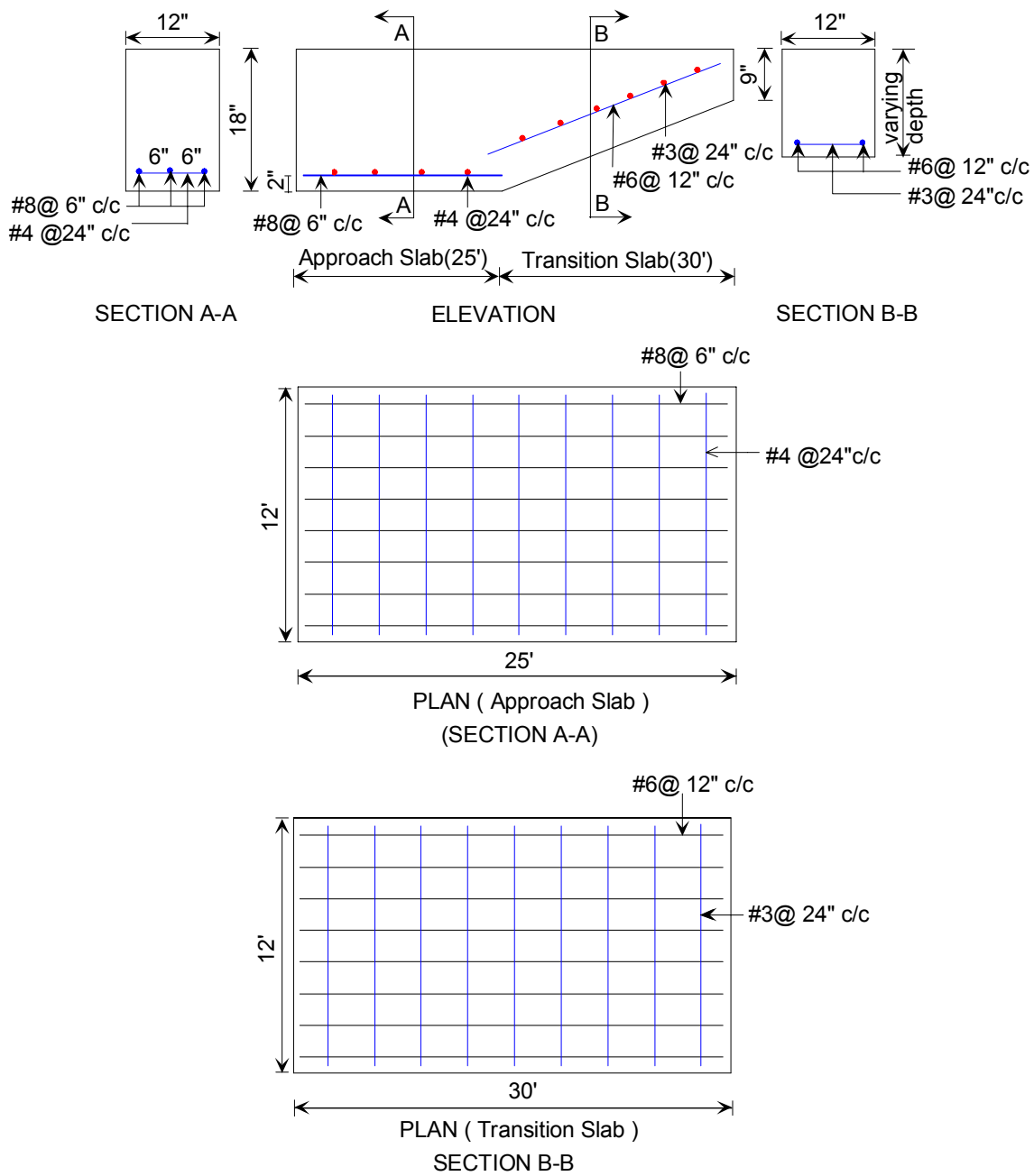


Figure 17. Reinforcement details for existing design approach and transition slabs.

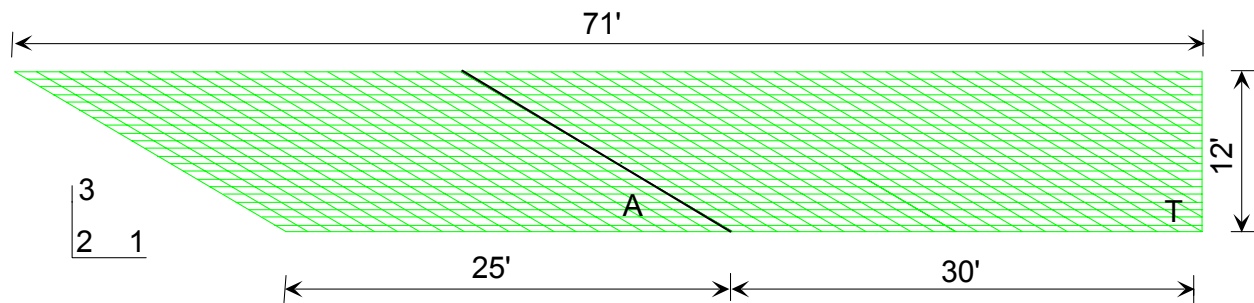


Figure 18. Plan view of the FEM formulation of the approach slab and transition slab.

Loading and Boundary Conditions

The FE model of approach and transition slabs was subjected to multiples of HS-20 bridge design truck loading. The HS-20 truck has three axles and a Gross Vehicle Weight (GVW) of 72 kips as shown in Figure 19. The front axle has two concentrated wheel loads each having a magnitude of 4 kips. Additionally, the middle and rear axles are represented by two concentrated loads having a magnitude of 16 kips each. The truck model is assumed to be 6 ft in width. The distance between the front and rear axle is taken as 28 ft.

The NJDOT approach and transition slab is supported by the bridge abutment from one side and by soil fill from the other side. The connection between the approach slab and bridge abutment will be represented by a pinned connection. Also, soil support at the transition slab end will be represented by linear springs as illustrated in Figure 20. Two boundary conditions will be imposed on the approach slab.

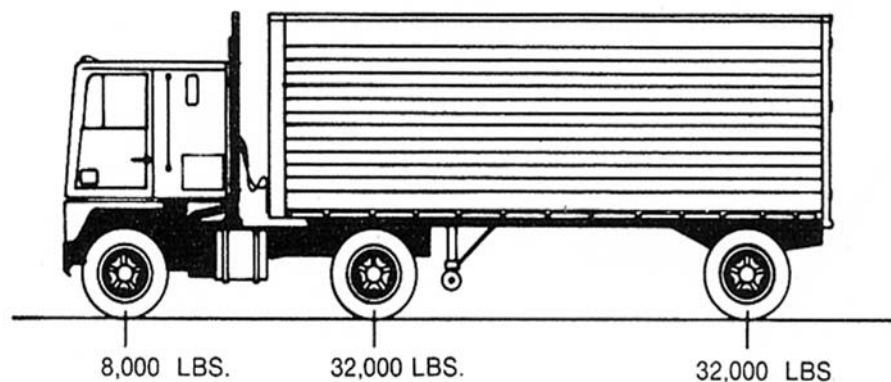


Figure 19. HS-20 design truck load from AASHTO bridge design specification.

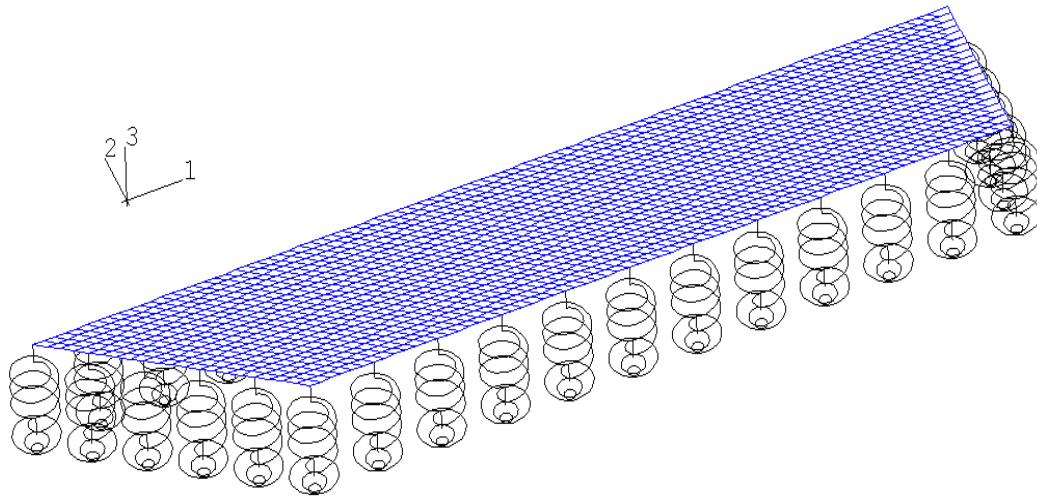


Figure 20. An isometric view of the FE model of soil structure interaction showing edge springs.

Soil-Structure Interaction

The soil strata underneath the slabs are represented by a series of linear elastic springs having constant stiffness as displayed in Figure 21. The springs used in this model are defined by ABAQUS/Standard software as “spring1” type elements. These elements represent a connection between the nodes and the soil support underneath. The soil in this model was assumed to be silty medium dense sand having a stiffness K_s of 25000 KN/cum⁽⁴⁾.

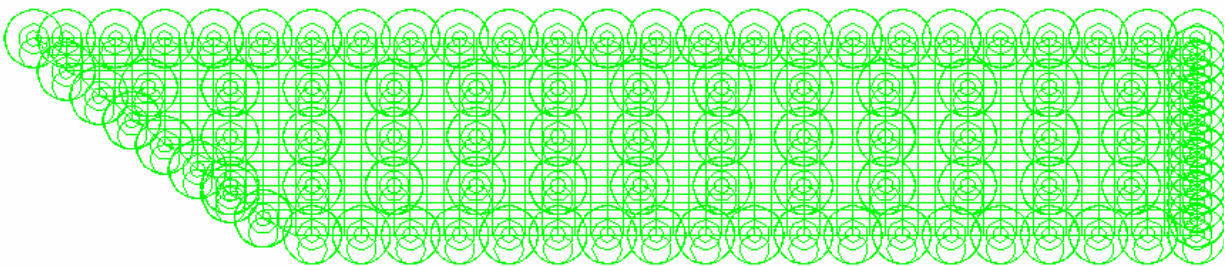


Figure 21. The FE model representing soil structure interaction using spring elements.

Finite Element Analysis

The truck was assumed to be in the middle of the lane. It was placed at consecutive locations either from pavement to bridge (bridge entrance) or bridge to pavement (bridge exit). Ascending step numbers represents each consecutive truck position. The load required for the first element to crack and its position are found. The default output points are at the top and bottom surfaces of the thickness of the shell section (for integration with Simpson's rule) or the points that are closest to the bottom and top

surfaces (for Gauss quadrature). Side D refers to the approach slab towards the abutment and Side B refers to the approach slab towards the pavement, as depicted in Figure 22.

Bridge Entrance

The HS20 truck was placed at consecutive locations from pavement to bridge. The finite element model shows that the slab will crack at the application of a load having a magnitude of 119.5 kips, which is approximately 1.66 times the HS-20 design truck. Elements located at a distance of 9 ft from Side B, as illustrated in Figure 22, crack first when the front axle of the truck is at a distance of 23 ft from the same side. Figure 23 shows cracked elements for 1.66 times the HS20 load for the truck position shown in Figure 22. Figure 24, in turn, demonstrates the stress variation for these cracked elements.

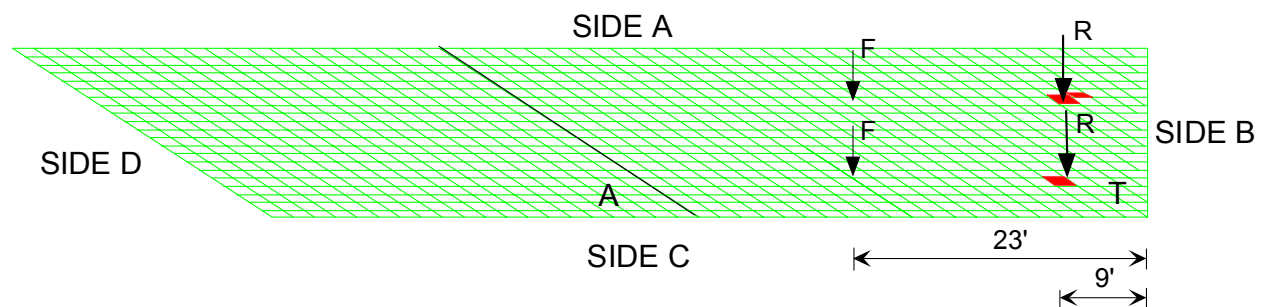


Figure 22. Location of cracked elements at bridge entrance due to an application of 1.66 HS20 load.

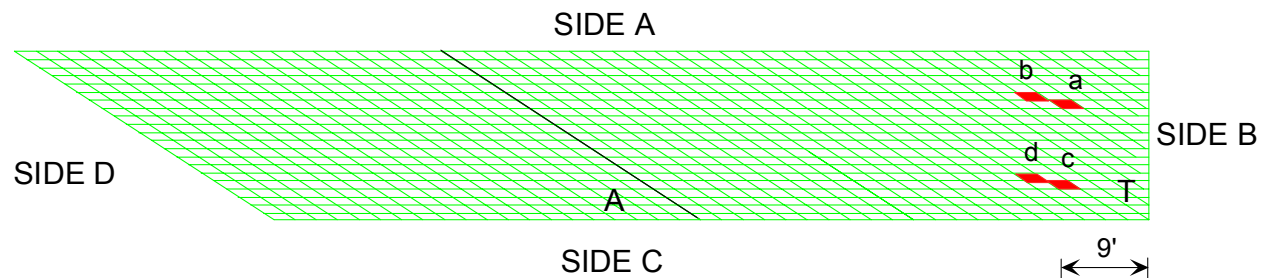


Figure 23. Cracked Elements.

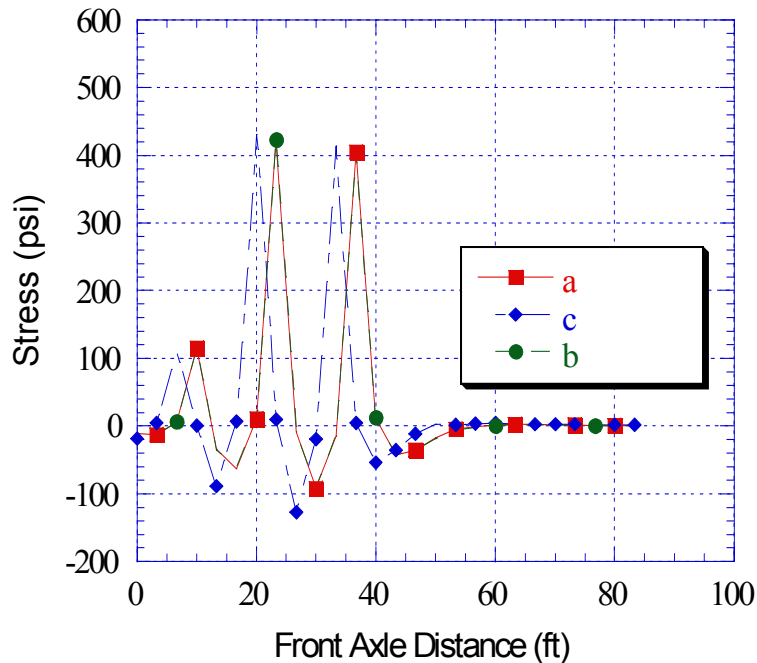


Figure 24. Tensile axial stresses for cracked elements in transition, due to the application of 1.66 HS20 load.

Bridge Exit

HS20 truck was placed at consecutive locations from bridge to pavement. The finite element results show that the load required for the first critical element to crack increases to 4.3 HS-20 (Figure 25). The first element cracked at a distance of 17 ft from SIDE D when the front axle of the truck was at a distance of 31 ft from that side. For a better understanding of the behavior of the model, the load was further increased to 5 HS-20. When the truck moved from the approach to the transition slab, due to excessive cracking in the transition area, the finite element solutions to the model stopped converging. Therefore it is assumed that the middle axle of the truck did not cross the approach slab.

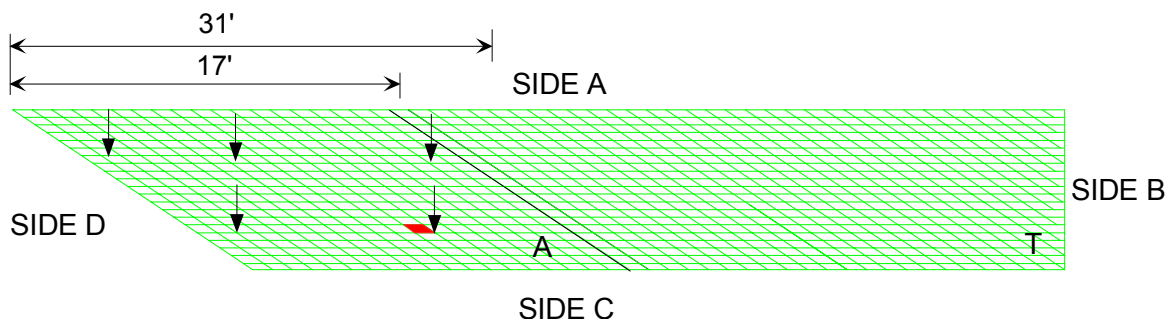


Figure 25. Location of cracked elements at bridge exit due to 4.3 HS20 truck load.

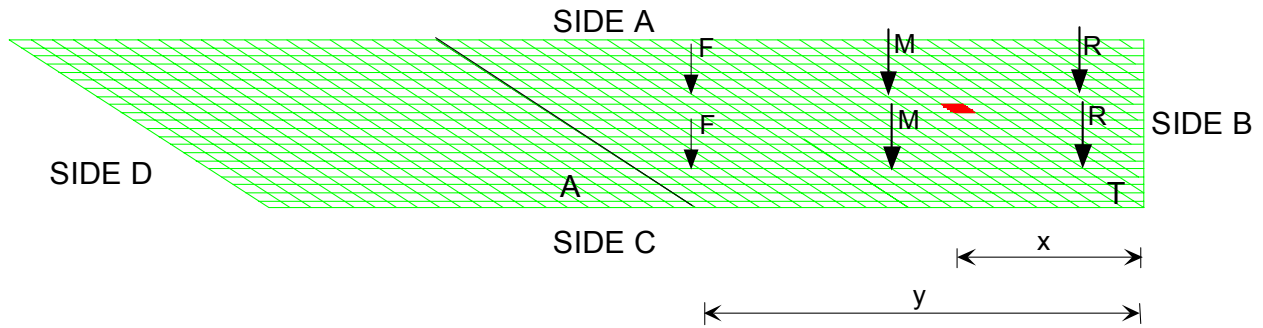
Discussion of Results

It is clear that thickness of approach slab (18 inches) is the most influential factor in the load carrying capacity of approach slabs, which is twice the thickness of transition slab (9 inches). This significant difference in the thickness of the slabs resulted in a higher approach slab versus transition slab strength; hence, a heavier load was required to crack the approach slab. For this reason, no element cracked at a load lower than 4.33 times HS-20. Moreover, the skew angle of the approach slab resulted in an uneven distribution of the axial load, such that only one side of the axles actually had contact with the approach slab. In addition, the pinned connection at the edge of the approach slabs, which links them with the bridge abutment, precludes any displacement taking place along this edge, thus providing more strength to the elements of this region.

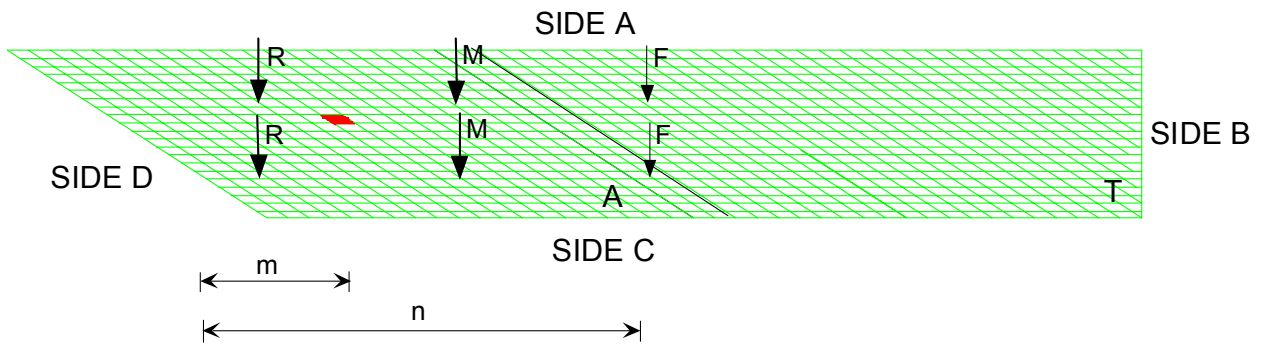
Cracking of Approach and Transition Slabs Due To Settlement

Removing springs from certain locations simulated settlement and/or void development in the model. This study focused on two primary locations: 1) at the bridge entrance and exit with settlement occurring in the middle of the approach and transition slabs under the path of traffic flow and 2) along the edges (or perimeter) of the slabs. Three settlement conditions were considered: 1) mild, 2) significant, and 3) severe settlement. The degrees of settlement correspond to a reduction in the number of middle springs by approximately 10%, 20% and 30%, respectively at the bridge entrance. Additionally, higher rates of settlements of 16%, 30%, and 40% at the bridge exit were considered to be accountable to the effect of the skew angle of the approach slab as well as improper compaction conditions in hard-to-reach soil areas close to the abutments.

The finite element model was formulated, such, that the width of the slabs is equivalent to a typical lane width. This is to account for the usage of construction joints between the lanes in most of the bridges in New Jersey. Therefore, the comparison between field studies and finite element modeling will be based on the location of cracks within each lane. Figure 26(a) and (b) as well as Tables 2 and 3 show the cracking of the approach and transition slabs at the bridge entrance and the bridge exit in different settlement cases.



(a) BRIDGE ENTRANCE



(b) BRIDGE EXIT

Figure 26. Position of the truck at bridge entrance and bridge exit.

Table 2. Cracking of A/T slab for loading at bridge entrance and different settlement cases.

Load case	Traffic flow	Multiple of HS20	Settlement Middle spring	Settlement Edge spring	x ft	y ft	Crack Observed
1	P → B	1.66	0	0	10.0	23.3	T
2	P → B	1.55	20%	0	8.3	20.0	T
3	P → B	1.3	30%	0	10.8	28.0	T
4	P → B	1.0	0	30%	5.0	13.3	T

Table 3. Cracking of A/T slab for loading at bridge exit and different settlement cases.

Load Case	Traffic Flow	Multiple of HS20	Settlement Middle spring	Settlement Edge spring	m (ft)	n (ft)	Crack Observed
1	P → B	4.3	0%	0%	29	35	T
2	B → P	4.2	16%	0%	18	24	A
3	P → B	4.1	30%	0%	9, 12	0	A
4	B → P	3.5	40%	0%	15, 19	25	Junction of A/P
5	B → P	4.0	0%	40%	15, 19	25	Junction of A/P

Verification of FE Model

A comparison of crack locations in the field survey and the FE model is shown in Table 4. Each of the values represent the ratio of the distance of the crack location (measured from the abutment) divided by the total length of the slab, taken as a percentage. This comparison shows that the values obtained from the field survey and the FE model analysis are nearly the same in both cases, i.e., at the bridge entrance and the bridge exit.

Table 4. Comparison of crack location as a percentage of total slab length.

	Bridge Exit	Bridge Entrance
Field survey	22%	83%
FE Model	21%	91%

Skewed versus Non-Skewed FE Model

To observe the effect of the skew angle on bridge approach and transition slab behavior, two finite element models were developed for both straight and skewed approach slabs, respectively, and stress levels in each of the cases were compared. For both cases, the truck was assumed to be traveling in the middle of the lane. The bridge design HS-20 truck loading is used to determine the stresses at two elements, A and B, as shown in Figures 27 (a) and (b), respectively.

Figures 28 and Figure 29 show that for the same loading conditions and at the same location, a skewed approach slab has higher tensile stresses than a straight slab. The difference ranges from 22% to 40% depending on the location of the element considered. Since the skewed approach slab is more critical, the parametric study is performed using the skewed FE model. Moreover, most bridges inspected in this report had a skewed approach slab.

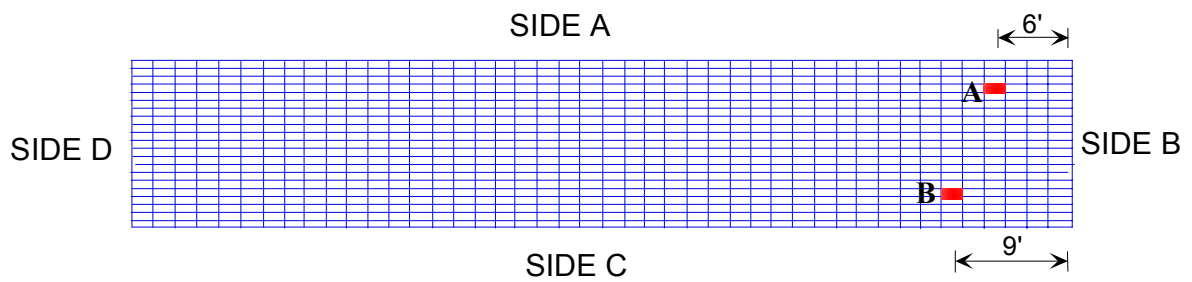
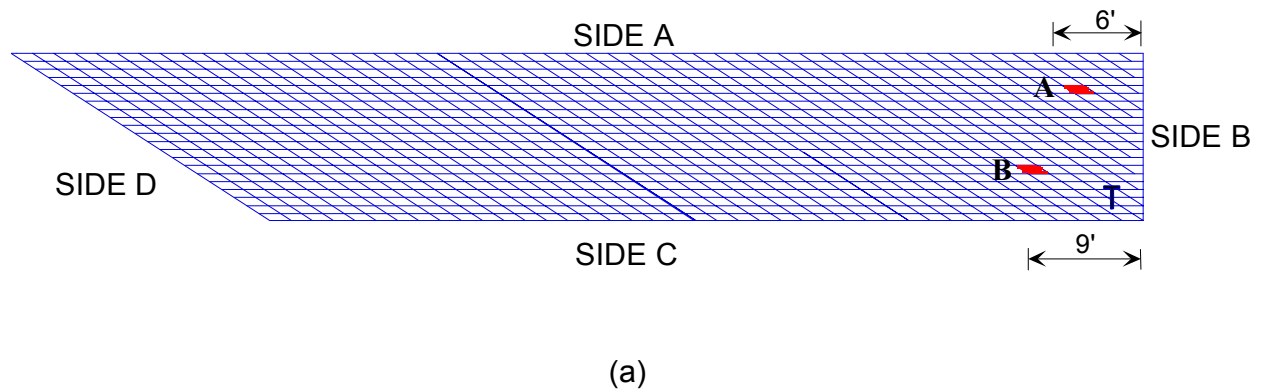


Figure 27. FE model for (a) skewed and (b) straight approach and transition slabs.

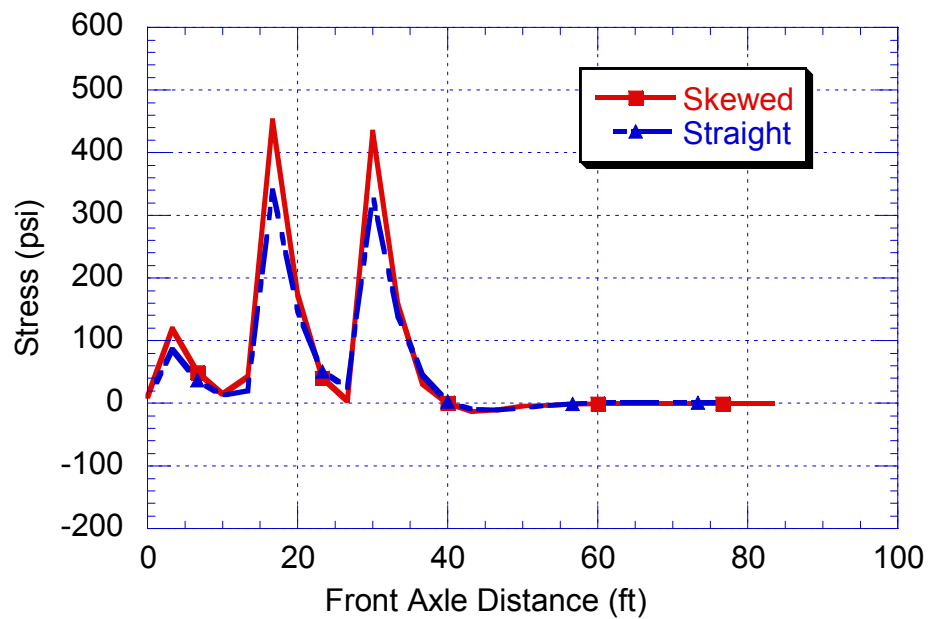


Figure 28. Variation of tensile axial stress versus front axle distance for skewed and straight approach slabs at element A shown in Figure 27.

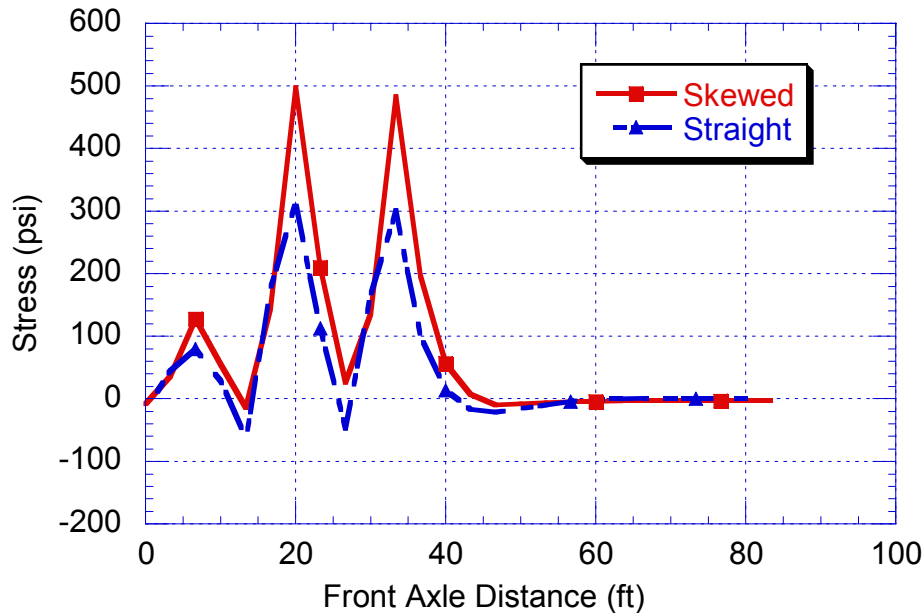


Figure 29. Variation of tensile axial stress versus front axle distance for skewed and straight approach slabs at element B shown in Figure 27.

PARAMETRIC STUDY

The objective of this parametric study is to identify the parameters that affect the cracking as well as the overall behavior of approach and transition slabs. The various parameters describing the interaction between the vehicle-slab-soil system are considered as follows: (1) concrete compressive strength, f'_c , (2) steel rebar yielding stress, F_y , (3) steel rebar nominal cross sectional area, A_s , (4) thickness of the concrete slab, and (5) soil settlement or void development. The results of this parametric study are presented for the critically stressed elements. Based on the results, various design alternatives and recommendations are developed.

Design Parameters

Four important structural parameters out of the five mentioned above are studied: (1) the concrete compressive strength, f'_c , (2) the steel rebar yielding stress, F_y , (3) the steel rebar nominal cross sectional area, A_s , and (4) the concrete slab thickness. To see effects of these important structural parameters during FE analysis of approach slabs, four elements, a, b, c, and d (shown previously in Figure 23) are monitored.

Effect of Concrete Compressive Strength, f'_c

The use of high performance (HP) as well as high strength concrete (HSC) is becoming more frequent in the construction industry. Three different values are assigned to f'_c : 4500, 5500, and 6500 psi, a common practice in design. According to ACI 318-99, the

tensile strength for each of these concrete mixes is 450, 550, and 650 psi, respectively. Figure 30 shows that the tensile stresses for the critical elements reach approximately 330 psi when the front axle of the HS-20 design truck is at a distance of 20 ft from the pavement for elements "a" and "b" and 23 ft for "c" and "d". However, the stresses in the elements drop to nearly 300 psi as the front axle reaches a distance of 33 ft from the road pavement for elements "a" and "b" and 37 ft for "c" and "d". Similarly, the variation of the tensile stresses for a compressive strength of 5500 psi is graphed in Figure 31.

In conclusion, the higher the concrete compressive strength is, the greater the capacity of the element to withstand a rise in tensile stress. Table 5 demonstrates this increase in the cracking load capacity of the slab due to that in the concrete strength, f'_c .

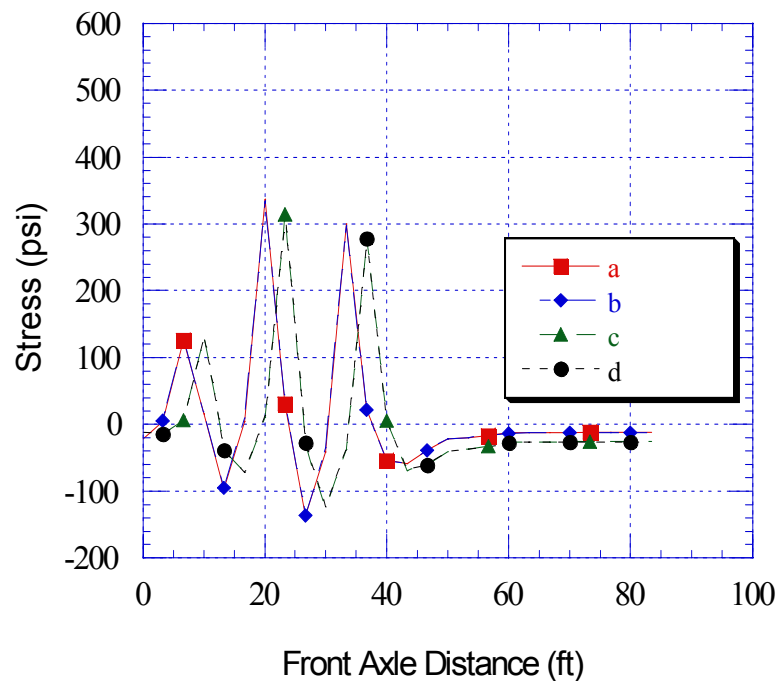


Figure 30. Variation of tensile axial stress for critical elements under the effect of a moving HS-20 truck load model ($f'_c = 4500$ psi).

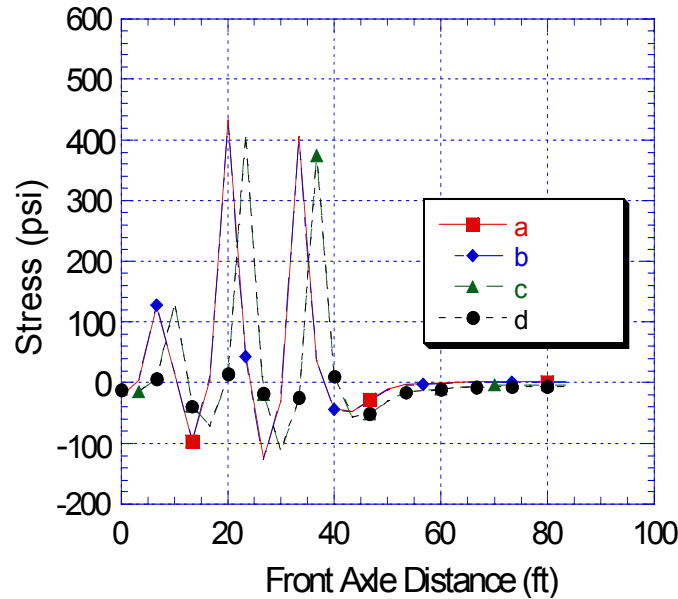


Figure 31. Variation of tensile axial stress for critical elements under the effect of a moving HS20 truck load model ($f'_c = 5500$ psi).

Table 5. Change in the cracking load due to an increase in concrete strength, f'_c .

f'_c (psi)	% Increase in f'_c	Cracking Load (kips)	% Increase in Cracking Load
4500	-	100.8	-
5500	22.2	115.2	14.3
6500	44.4	144	42.9

Effect of Steel Rebar Yielding Stress, F_y

Three cases were considered for this parameter corresponding to 50, 60, and 70 ksi, respectively. No change in the tensile axial stresses of the critical elements was observed. Therefore, increasing the yielding stress of the rebar has no effect on the cracking behavior of the model. The effect of the steel rebar on the cracking behavior of the slabs is limited to the post cracking stage only.

Effect of Steel Rebar Area, A_s

The steel in the existing design, constant thickness design, and deep beam design was used in order to monitor the cracking behavior of the slabs. No remarkable changes were observed in the tensile stresses of the critical elements. Therefore, this parameter is not effective in reducing the number of cracked elements in the model.

Effect of Slab Thickness

The thickness of the transition and approach slab is increased throughout the length by 1, 2, and 3 inches, respectively. A drop in the tensile stresses in the critical elements with an increase in the thickness of the slab is displayed in the Table 6 below.

Table 6. Reduction in tensile axial stress in elements due to increase in slab thickness.

Element No.	Axial Stress (psi) for Various Slab Thicknesses			
	0 inch	1 inch	2 inch	3 inch
a (%Reduction)	413 (0.0%)	358 (13.1%)	316 (23.5%)	281.0 (31.9%)
b (%Reduction)	420 (0.0%)	364 (13.4%)	320 (23.9%)	284.0 (32.4%)
c (%Reduction)	428 (0.0%)	367 (14.3%)	320.0 (25.2%)	283.1 (33.9%)
d (%Reduction)	430 (0.0%)	368 (14.3%)	321.2 (25.3%)	284.0 (33.9%)

Effect of Soil Settlement

The results for the effect of soil settlement are presented for two cases, traffic at the bridge entrance and traffic at bridge exit.

Bridge Entrance

Reduction in the number of springs in the model represents soil settlement. Four different settlement possibilities are studied; they are represented by a reduction in the number of middle springs by 11%, 20%, and 30% respectively. Moreover, the edge springs are also reduced by 30% to determine the effect of soil settlement on the tensile axial stress in the elements having maximum stresses.

Table 7 illustrates that as the number of middle springs is reduced by 10%, the maximum tensile axial stress in element "a" increases by 4.9% from approximately 337 psi to 353 psi.

Furthermore, the stress in this element "a" increases to 370 psi (10%) for a 20% reduction in the middle springs. Also, if the number of springs is further reduced to 30%, representing more soil settlement in the transition slab, the tensile stress in the element increases to 402 psi (19.5%). Similar behavior is observed for elements "b" and "c."

This increase in stress is a consequence of void development under the slab or the settlement of cohesionless soil that was not totally compacted at the time of construction. Development of void and settlement affects the elements mentioned above and elements in adjacent regions. These elements will not have enough resistance to withstand the truckload, resulting in the deflection. Displacement causes tension stresses to increase until finally one or more elements crack. This explanation is valid for both mild and significant settlement cases.

Therefore, the edge settlement of the approach and transition slabs is more critical. In the case of edge soil settlement, the tensile stresses occur at the top surface of the slab, while in the case of middle settlement, they occur in the bottom of the slab. The

edge springs at the entrance of the bridge plays an essential role in the distribution of stresses in the elements and in the displaced shape that the model takes.

However, in the case of middle settlement, there is nothing to resist the rear axle load of the truck, and more concentration of stresses takes place as a result of the stiffness of the springs or the resistance of soil in adjacent areas. Therefore, the stress continues to accumulate in this region until ultimately one or more elements crack. The cracks occur in the bottom surface as a consequence of the concentration of tensile stresses at the (SNEG) surface of the transition slab. The displaced shape in this case is concave with the compression at the top surface (SPOS) and tension at the bottom surface (SNEG), as before. Furthermore, the discrepancy between the results of the edge and middle settlements extends to the values of the stresses induced in the critical elements. The stresses developed as a result of edge settlement are higher than those for middle settlement. The effect of the truckload on the displacement of the model in the case of edge settlement is higher than on its displacement in the case of middle springs removed.

Table 7. Summary of the effect of soil settlement on the maximum tensile stresses induced in the critical elements.

Element No.	Axial Stress (psi) for Various Degrees of Settlement				
	(0 %)	(10 %)	(20 %)	(30 %)	(30 %) Edge
a (%Increase)	337 (0%)	353 (4.9%)	370 (10.0%)	402 (19.5%)	334 (-0.7%)
b (%Increase)	326 (0%)	264 (13.9%)	294 (25.9%)	323 (38.4%)	163 (-3.6%)
c (%Increase)	293.6 (0%)	201 (23.3%)	221 (35.6%)	241 (47.5%)	129 (-4.8%)

Bridge Exit

The effect of soil settlement or void development was also addressed in the case of a truck traffic travelling over the bridge exit. Three different settlement scenarios were considered. These scenarios represent a reduction in the number of middle springs by approximately 1) 16%, 2) 30%, and 3) 40%, respectively. Moreover, a fourth scenario, representing a 40% reduction in the edge springs, is considered to determine the effect of soil settlement on the tensile stress in the critical elements.

Table 8 demonstrates that when the number of middle springs is reduced by 40%, the maximum tensile axial stresses in the critical element "a" is increased by 19% from approximately 350 psi to 471 psi.

Table 8. Effect of soil settlement on the maximum tensile stresses in the critical elements.

Element No.	Axial Stress Due to a Settlement, (psi)				
	(-0%)	(-16%)	(-30%)	(-40%)	(-40%) Edge
a (%Increase)	350 (0%)	350 (0%)	362 (3.4%)	471 (19%)	367 (4.8%)
b (%Increase)	300 (0%)	302 (0.6%)	360 (20.0%)	415 (38%)	309 (2.7%)
c (%Increase)	291 (0%)	300 (3%)	335 (15%)	349 (19.8%)	312.2 (7.2%)
d (%Increase)	199 (0%)	208 (4.4%)	243 (21.8%)	269 (34.8%)	219 (9.8%)

The effects of each parameter, summarized in Tables 9 and 10, are as follows:

- As the amount of soil settlement underneath the middle of the slabs increases, the tensile axial stresses and the number of cracked elements also increase.
- Settlement at the edges of the slab results in higher stresses.
- Settlement at the edges of the slab results in a concentration of tensile stresses at top of the slab.
- Settlement in the middle of the slab results in a concentration of stresses at bottom of the slab.

Table 9. Summary of the effect of the various parameters on the cracking load.

% Change in f_c	% Change in Cracking Load	% Change in F_y	% Change in Cracking Load	% Change in A_s	% Change in Cracking Load
22	14	20	0	115	0
44	43	40	0	144	0

Table 10. Summary of the effect of the various parameters on the cracking load.

% Change in Slab Thickness	%Change in Cracking Load	% Change in Soil Settlement Bridge entrance (A)	% Change in Cracking Load	% Change in Soil Settlement Bridge Exit (B)	% Change in Cracking Load
5	21	10	4	16	2
10	51	20	7	30	5
16	81	30 (Middle)	28	40 (Middle)	19
-	-	30 (Edge)	40	40 (Edge)	7

DESIGN ALTERNATIVES

The parametric study concluded that the slab thickness is the most effective parameter in reducing the tensile stresses in the critical elements. Therefore three new designs are proposed:

- (1) Constant Thickness Design
- (2) Deep Beam Design
- (3) Embedded Beam Design

Design Alternative 1. Constant Thickness Design

According to the analysis and field observations, the location of the crack is in the transition slab. Therefore, the thickness of the transition slab is increased to be the same as approach slab (i.e. 18 inches). Figures 32 and 33 show sketches for design calculations. Figure 34 depicts the detailed dimensions and amount of reinforcement obtained from design calculations for this alternative.

Design Calculations

Simply Supported Beam

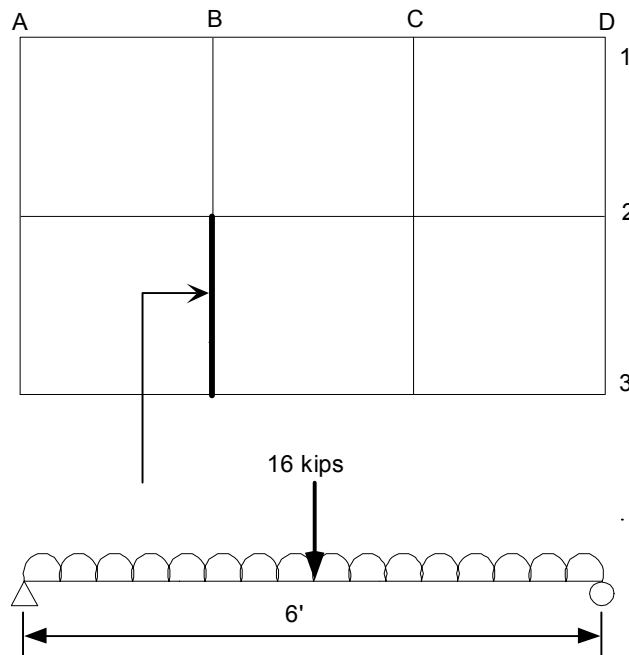


Figure 32. Design of transverse beam.

Dead load, $w = (12 \times 18 / 144) \times 150 = 225 \text{ lb/ft}$

Wheel load, $P = 16 \times 2 = 32 \text{ kips}$

Overloading of truck once in 50 years, $w_{50} = 0.64 \text{ kips/ft}$

Maximum Positive Moment = $w L^2 / 8 + P L / 4 + w_{50} L^2 / 8$
 $= 0.225 \times 6^2 / 8 + 32 \times 6 / 4 + 0.64 \times 6^2 / 8 = 52 \text{ kips-ft}$

This moment is assumed to be distributed over a certain area. The width of the equivalent strip is given by $660+0.55S$ (AASHTO LRFD table 4.6.2.1.3-1). Substituting value of $S=6$ ft gives an equivalent width of 5.5 ft.

Moment per ft = $52 \text{ kipsft} / 5.5 \text{ ft} = 9.45 \text{ kips-ft}$

$M_u = (1.75 \times 9.25) + (1.25 \times 0.18) = 16.45 \text{ kips-ft}$

$M_n = M_u / \phi = 16.45 / 0.9 = 18.3 \text{ kips-ft} = 219,300 \text{ lb-in}$

Nominal moment of resistance can be expressed as,

$$M_n = A_s \times F_y \times \left(d - \frac{a}{2} \right) \quad (1)$$

From the above equation,

$$a = 2d - (2 M_n / (A_s F_y)) \quad (2)$$

We have another equation

$$A_s \times F_y = 0.85 \times f'_c \times b \times a \quad (3)$$

From equations (1) and (3),

$$\left(\frac{A_s^2 \times F_y^2}{2 \times 0.85 \times b \times f'_c} \right) - (d \times F_y \times A_s) + (M_n) = 0 \quad (4)$$

Substituting values as shown below in equation (4),

$F_y = 60,000 \text{ psi}$

$M_n = 219,600 \text{ psi}$

$b = 12 \text{ in}$

$d = 14.5 \text{ in}$ (assuming 16in. cover)

$A_s = 0.25 \text{ in}^2$

Minimum Steel Required (AASHTO LRFD C5.7.3.3.2) =
 $0.03 \times (f'_c / F_y) \times b \times d = 0.03 \times (4.5 / 60) \times 12 \times 14.5 = 0.39 \text{ in}^2$.

Provide 1 # 6 @ 12 in longitudinal direction and #6 @ 12 in c/c in transverse direction.

Slab

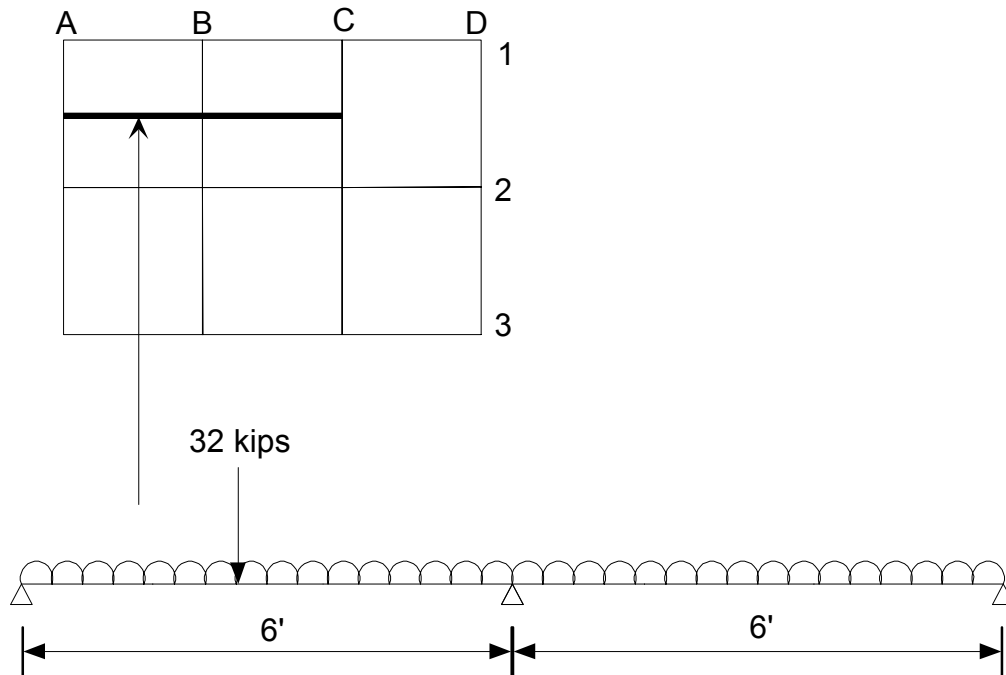


Figure 33. Design of longitudinal slab.

Dead load, $w = 225 \text{ lb/ft}$

Wheel load, $P = 32 \text{ kips}$

Overloading of truck once in 50 years, $w_{50} = 0.64 \text{ kips/ft}$

Maximum positive moment = $[(13/64) \times P \times L] + (0.086 \times w \times L) + (0.086 \times w_{50} \times L)$

$$= \left(\frac{13}{64} \times 32 \times 6 \right) + (0.086 \times 0.225 \times 6) + (0.086 \times 0.64 \times 6) = 40 \text{ kips-ft}$$

This moment is assumed to be distributed over a certain area. The width of the equivalent strip is given by $660 + 0.55S$ (AASHTO LRFD table 4.6.2.1.3-1). Substituting value of $S = 6 \text{ ft}$ gives an equivalent width of 5.5 ft .

Moment per ft = $40 \text{ kips-ft} / 5.5 \text{ ft} = 7.27 \text{ kips-ft}$

$$M_u = (1.75 \times 7.11) + (1.25 \times 0.16) = 12.64 \text{ kips-ft}$$

$$M_n = M_u / \phi = 14 \text{ kips-ft} = 168,000 \text{ lb-in.}$$

$F_y = 60,000 \text{ psi}$

$M_n = 168,000 \text{ lb-in}$

$b = 12 \text{ in}$

$d = 14.5 \text{ in}$ (assuming 3 in. clear cover)

Substituting the above values in Equation (3) gives $A_s = 0.2 \text{ in}^2$.

Minimum Steel Required = $0.03 \times (f'_c/F_y) \times b \times d = 0.03 \times (4.5/60) \times 12 \times 14.5 = 0.39 \text{ in}^2$.

Provide 1 # 6 @ 12 in. both of the ways at the bottom. The same reinforcement is provided at the top in both directions.

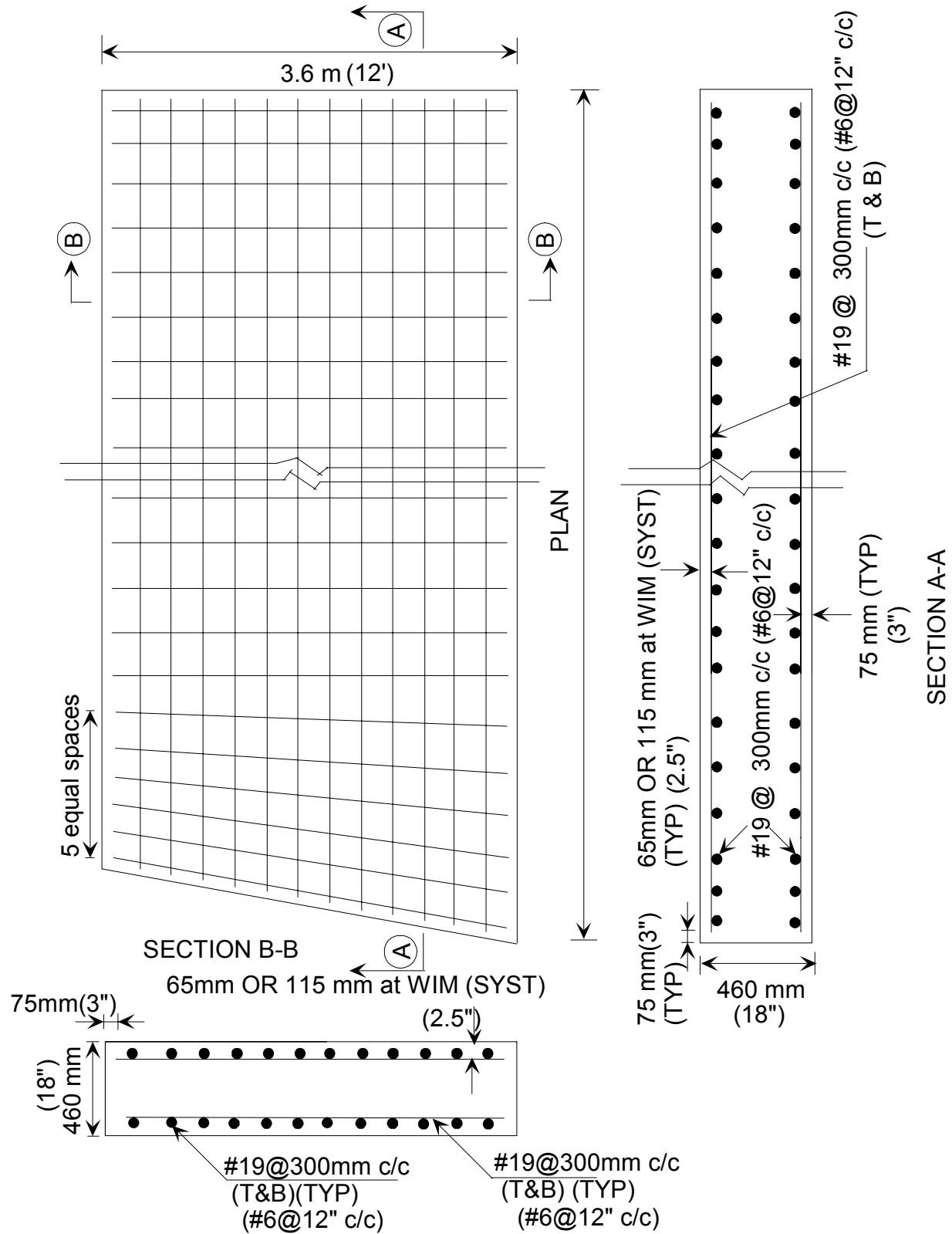


Figure 34. Reinforcement details for proposed constant thickness slab.

Design Alternative 2. Embedded Beam Design

Reinforcement details for Embedded Beam Design are shown in Figure 37. The design calculations are displayed below.

Design Calculations:

Simply Supported Beam (Figure 35)

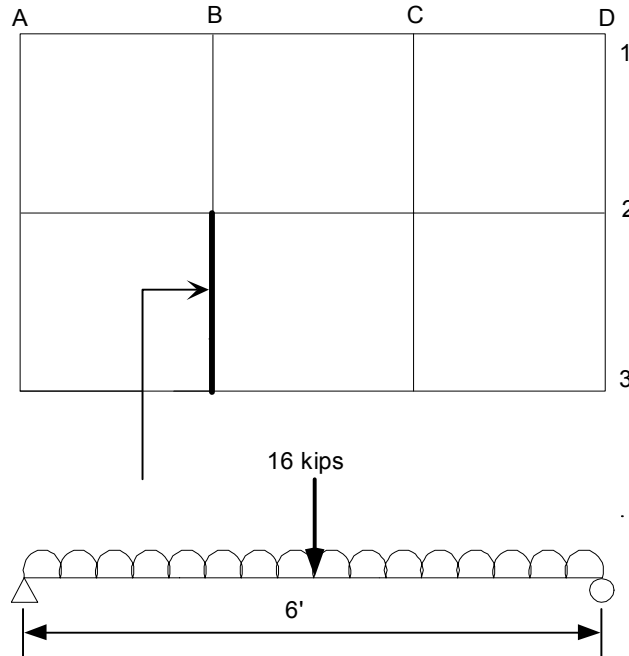


Figure 35. Design of transverse beam.

Dead load, $w = (12 \times 18 / 144) \times 150 = 225 \text{ lb/ft}$

Wheel load, $P = 16 \times 2 = 32 \text{ kips}$

Overloading of truck once in 50 years, $w_{50} = 0.64 \text{ kips/ft}$

Maximum Positive Moment = $w L^2 / 8 + P L / 4 + w_{50} L^2 / 8$
 $= 0.225 \times 6^2 / 8 + 32 \times 6 / 4 + 0.64 \times 6^2 / 8 = 52 \text{ kips-ft}$

This moment is assumed to be distributed over a certain area. The Width of the equivalent strip is given by $660 + 0.55S$ (AASHTO LRFD table 4.6.2.1.3-1). Substituting value of $S = 6 \text{ ft}$ gives an equivalent width of 5.5 ft .

Moment per ft = $52 \text{ kips-ft} / 5.5 \text{ ft} = 9.45 \text{ kips-ft}$

$M_u = (1.75 \times 9.25) + (1.25 \times 0.18) = 16.45 \text{ kips-ft}$

$M_n = M_u / \phi = 16.45 / 0.9 = 18.3 \text{ kips-ft} = 219,300 \text{ lb-in}$

Nominal moment of resistance can be expressed as,

$$M_n = A_s \times F_y \times \left(d - \frac{a}{2} \right) \quad (1)$$

from the above equation,

$$a = 2d - (2 M_n / (A_s F_y)) \quad (2)$$

We have another equation

$$A_s \times F_y = 0.85 \times f'_c \times b \times a \quad (3)$$

From equations (1) and (3),

$$\left(\frac{A_s^2 \times F_y^2}{2 \times 0.85 \times b \times f'_c} \right) - (d \times F_y \times A_s) + (M_n) = 0 \quad (4)$$

Substituting following values in equation (4),

$$F_y = 60,000 \text{ psi}$$

$$M_n = 219,600 \text{ psi}$$

$$b = 12 \text{ in}$$

$$d = 14.5 \text{ in (assuming 16in. cover)}$$

$$A_s = 0.25 \text{ in}^2$$

Minimum Steel Required (AASHTO LRFD C5.7.3.3.2)

$$= 0.03 \times (f'_c / F_y) \times b \times d = 0.03 \times (4.5 / 60) \times 12 \times 14.5 = 0.39 \text{ in}^2.$$

Provide 1 # 6 @ 12 in. longitudinal direction and #6 @ 12 in c/c in transverse. Provide #6 @ 6 ft in longitudinal direction for stiffness.

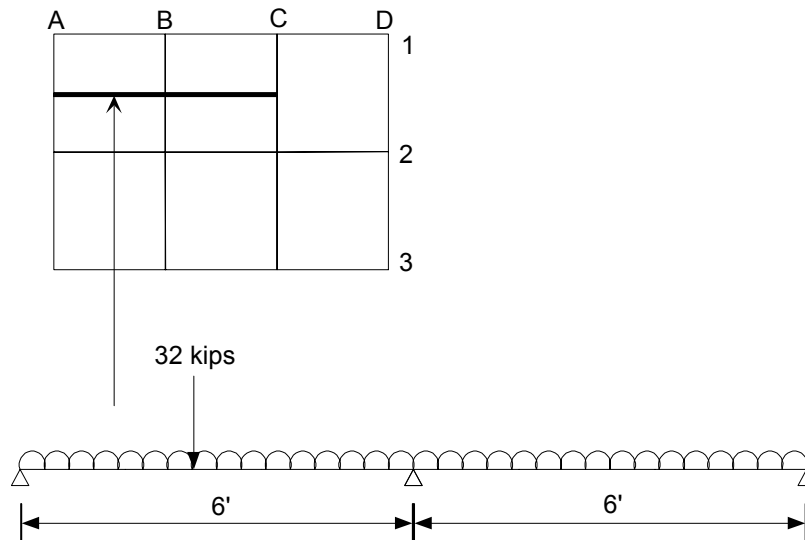


Figure 36. Design of longitudinal slab.

Dead load, $w = 225 \text{ lb/ft}$

Wheel load, $P = 32 \text{ kips}$

Overloading of truck once in 50 years, $w_{50} = 0.64 \text{ kips/ft}$

$$\begin{aligned}\text{Maximum positive moment} &= [(13/64) \times P \times L] + (0.086 \times w \times L) + (0.086 \times w_{50} \times L) \\ &= \left(\frac{13}{64} \times 32 \times 6 \right) + (0.086 \times 0.225 \times 6) + (0.086 \times 0.64 \times 6) = 40 \text{ kips-ft}\end{aligned}$$

This moment is assumed to be distributed over a certain area. The width of the equivalent strip is given by $660 + 0.55S$ (AASHTO LRFD table 4.6.2.1.3-1). The substituting value of $S = 6 \text{ ft}$ gives an equivalent width of 5.5 ft .

Moment per ft = $40 \text{ kips-ft} / 5.5 \text{ ft} = 7.27 \text{ kips-ft}$

$$M_u = (1.75 \times 7.11) + (1.25 \times 0.16) = 12.64 \text{ kips-ft}$$

$$M_n = M_u / \phi = 14 \text{ kips-ft} = 168,000 \text{ lb-in.}$$

$$F_y = 60,000 \text{ psi}$$

$$M_n = 168,000 \text{ lb-in}$$

$$b = 12 \text{ in}$$

$$d = 14.5 \text{ in (assuming 3 in. clear cover)}$$

Substituting the above values in Equation (3) gives $A_s = 0.2 \text{ in}^2$.

$$\text{Minimum Steel Required} = 0.03 \times (f'_c / F_y) \times b \times d = 0.03 \times (4.5 / 60) \times 12 \times 14.5 = 0.39 \text{ in}^2.$$

Provide 1 # 6 @ 12 in both of the ways at the bottom. The same reinforcement is provided at the top in both the directions. Provide #6 @ 6 ft in longitudinal direction for stiffness.

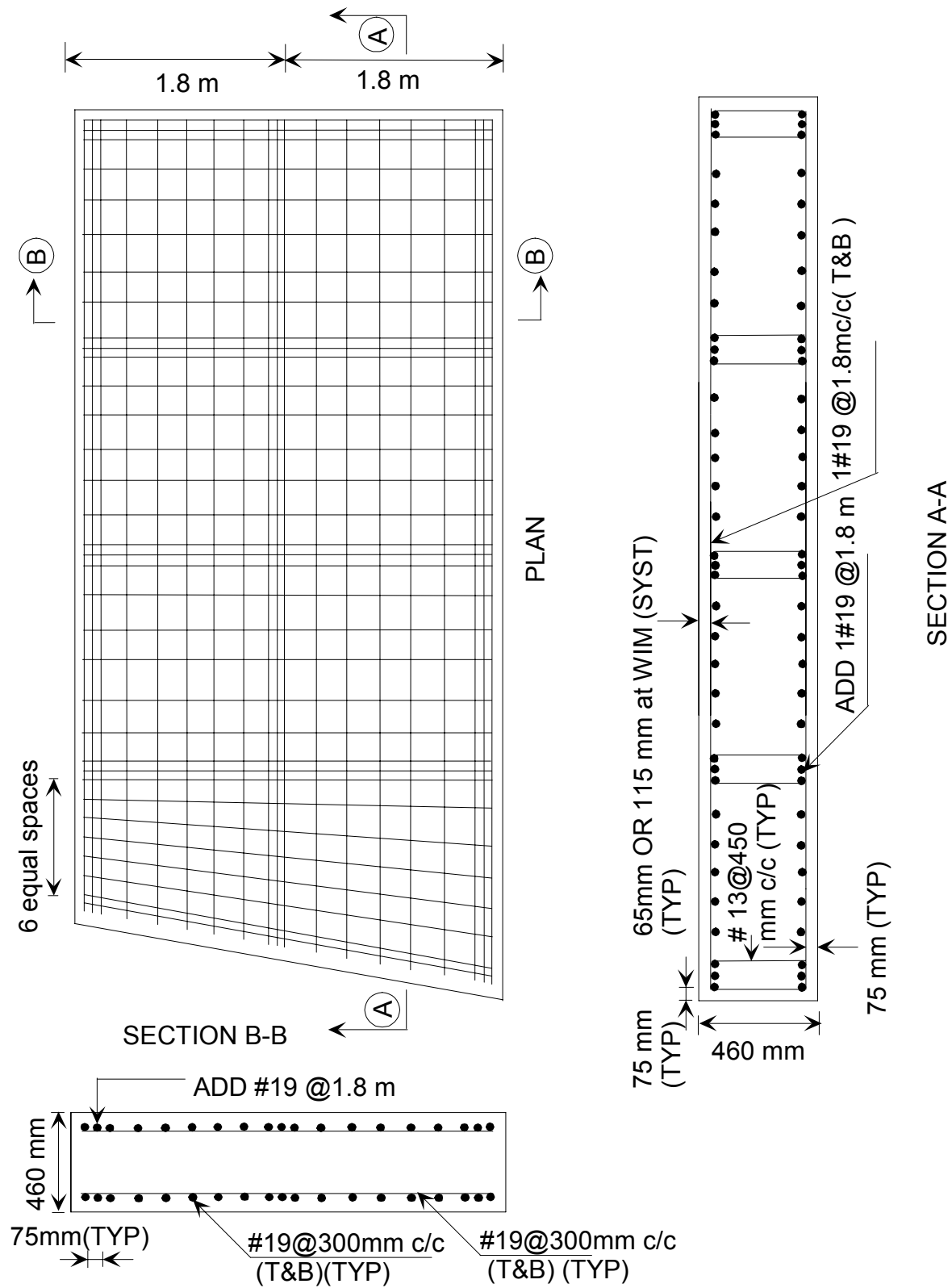


Figure 37. Reinforcement details for proposed embedded beam design

Design Alternative 3. Deep Beam Design

The design objective is to provide an approach and transition slab that can respond to heavy loads coupled with an unavoidable settlement or void development without cracking. The design criteria for such a slab are as follows:

- 1) Should be able to withstand settlement or void development.
- 2) Should not develop any cracking (or at least not in the top concrete fibers).
- 3) Should be designed rationally based on input data and various soil conditions.
- 4) Should address dynamic effect due to road roughness and their vehicle braking forces and impact.
- 5) Should have enough capacity to resist shear as well as flexure.
- 6) Should be cost effective.
- 7) Should not involve major deviation from the current construction practice (i.e., no prestressing, no fiber concrete, etc.)
- 8) Should not exceed 20 inches in thickness.

This alternative is based on assumptions made using the field survey, literature research, and the FE analysis, as follows:

1. Void Development: According to Schaefer et al.⁽²⁴⁾, the length of the voids observed during field studies varied from less than 1 inch to as large as 14 inches. Conservatively, it is assumed that void and/or settlement development is unavoidable and could extend up to 54 inches. (3.75 ft)
2. Loading: Truckloads are site-specific and the load spectra can be amplified by impact and dynamic amplification of the static design loads. According to Nassif et al.⁽¹⁸⁾, who performed WIM on the effect of heavy truckloads on bridges, observed that heavy truckloads exceed legal limits by up to 30%. Moreover, they observed that the dynamic amplification in girder stresses and/or deflections due to the dynamic interaction between vehicle, bridge, and road, varies between 15 to 90% beyond the static loads. Moreover, in the case the of slab on soil, the dynamic amplification is dependent on the type of soil and its stiffness. Therefore, it is assumed that the design loads including impact, braking, and other uncertainties, are 4.3 times HS-20.
3. Thermal Stresses: It is assumed that minimum temperature reinforcement is needed to account for fluctuation in thermal stresses. This amount will be similar to that placed in concrete bridge decks.

Design Calculations:

Check I_r/d to determine if the beam will act as a deep beam.

Assume that $d=0.9h=16$ inches; then $I_r/d = 45/14.5 = 3.1 < 5$; hence, treat as a deep beam.

Taking Live Load as 2 x HS20 (includes 50% impact and 1 time overloading of truck in 75 years).

$$\text{The Distributed Load on the span of the beam} = \frac{\text{The Axle Load of the Design Truck}}{\text{The Distance between the Axles}} \\ = \frac{2 \times 16000}{3.75} + 0.64 \text{ kips/ft} = 8533 \text{ lb/ft} + 640 \text{ lb/ft} = 9170 \text{ lb/ft}$$

$$\text{Dead Load (selfweight)} = \frac{12 \times 18}{144} \times 150 = 225 \text{ lb/ft}$$

$$\text{Total Factored Load} = 1.75 \times 9170 + 1.25 \times 225 = 16,330 \text{ lb/ft}$$

Flexure Design

$$\text{The External Factored Moment, } M_u = w_u l_n^2 / 8 = 16330(3.75)^2 / 8 = 28,705 \text{ lb-ft} = 344,460 \text{ lb-in}$$

$$M_n = M_u / \phi = 344,460 / 0.9 = 382,733 \text{ lb-in}$$

Substituting values as shown below in equation (3),

$$F_y = 60,000 \text{ psi}$$

$$M_n = 382,733 \text{ psi}$$

$$b = 12 \text{ in}$$

From AASHTO-LRFD: Table 5.12.3.1.:

$$d = 14.5 \text{ in}$$

$$A_s = 0.44 \text{ in}^2$$

AASHTO-LRFD: C 5.7.3.3.2:

$$\text{Minimum Steel Required} = 0.03 \times (f'_c / F_y) \times b \times d = 0.03 \times (4.5 / 60) \times 12 \times 14.5 = 0.39 \text{ in}^2$$

Use #6 @ 12" in both directions at bottom layer.

To obtain top reinforcement, AASHTO-LRFD C 5.10.8 denotes the equation:

$$A_s > 0.11 A_g / F_y$$

Therefore, use #3@12" in both directions at top layer.

Shear Reinforcement

The maximum permissible spacing of vertical bars $S_v = \min (d/5) \text{ or } 18 \text{ in}$

$$S_v = 14.4 / 5 = 3 \text{ in}$$

The minimum permissible spacing of horizontal bars $S_h = \min (d/3) \text{ or } 18 \text{ in}$

$$S_v = 14.4 / 3 = 4.8 \text{ in}$$

Therefore, Use $S_v = 3.0 \text{ in}$ and $S_h = 5 \text{ in}$

Provide horizontally 4#3 bars at distance of 2.5 inches in three layers, and provide transversely #3 bars at distance of 12 inches. The details of reinforcement are shown in Figure 38.

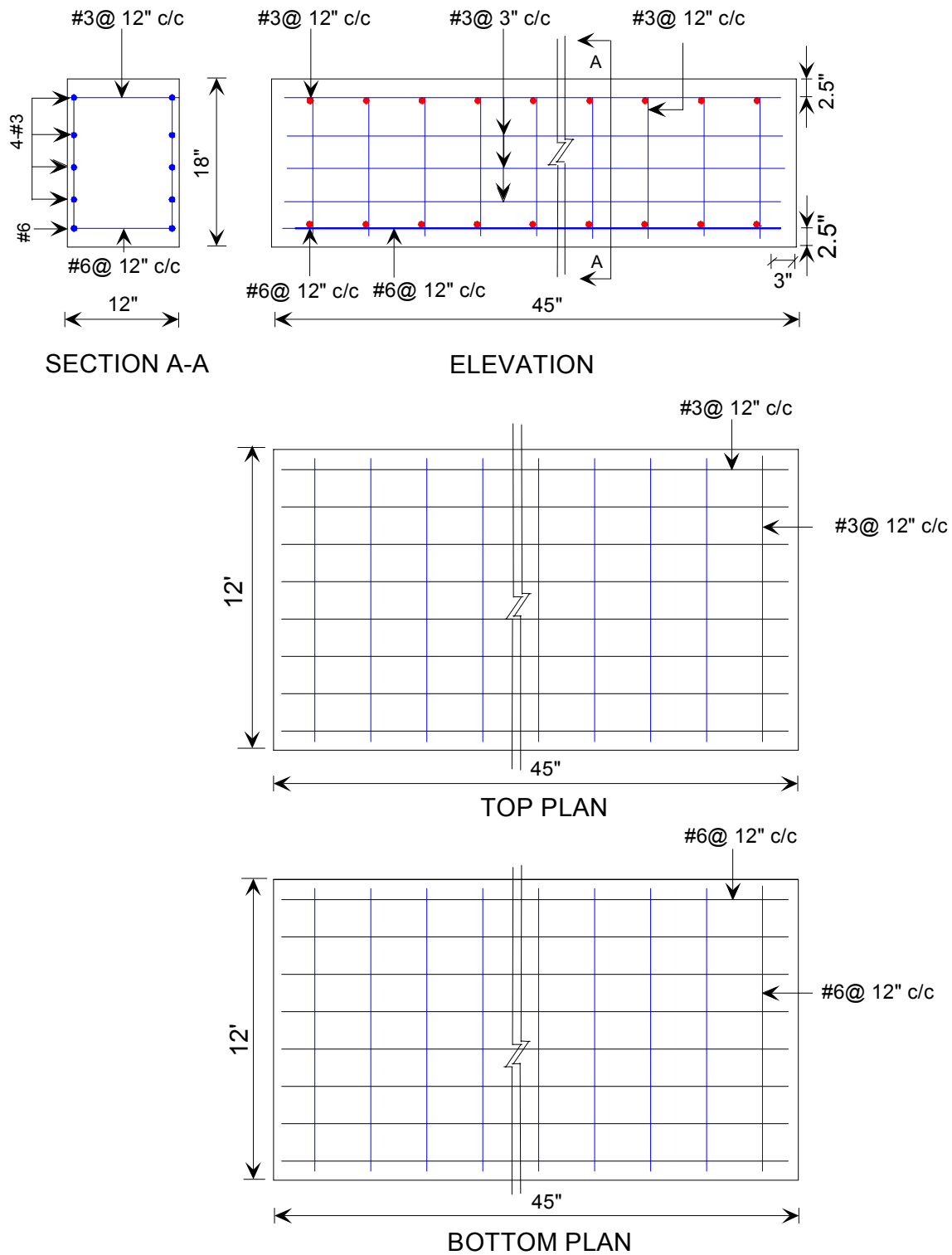


Figure 38. Reinforcement details for proposed deep beam design.

Reinforcement Summary for all design options

Existing

Reinforcement provided:

(A)Approach	
(i)bottom longitudinal	#8@6in.c/c
(ii)bottom transverse	#4@24in. c/c
(B)Transition	
(i)middle longitudinal	#6@12in. c/c
(ii)top transverse	#3@24in. c/c

Constant thickness

Reinforcement provided:

(I)Bottom Longitudinal Rebars	#6@12in.c/c
(II)Bottom Transverse Rebars	#6@12in. c/c
(III)Top Longitudinal Rebars	#6@12in.c/c
(IV)Top Transverse Rebars	#6@12in. c/c

Embedded Beam

Reinforcement provided:

(A)Longitudinal Beams	
(I)Bottom Longitudinal Rebars	3#6
(II)Bottom Transverse Rebars	3#6
(III)Top Longitudinal Rebars	3#6
(IV)Top Transverse Rebars	3#6
(B)Transverse Beams	
(I)Bottom Longitudinal Rebars	3#6
(II)Bottom Transverse Rebars	3#6
(III)Top Longitudinal Rebars	3#6
(IV)Top Transverse Rebars	3#6
(C)Slab	
(I)Bottom Longitudinal Rebars	#6@12in.c/c
(II)Bottom Transverse Rebars	#6@12in. c/c
(III)Top Longitudinal Rebars	#6@12in.c/c
(IV)Top Transverse Rebars	#6@12in. c/c
(D)Stirrups	#4@18in. c/c

Deep Beam

Reinforcement provided:

(level-1)bottom longitudinal	#6@12in.c/c
(level-1)bottom transverse	#6@12in. c/c
(level-2)longitudinal	#3@12in. c/c
(level-3)longitudinal	#3@12in. c/c
(level-4) longitudinal	#3@12in. c/c
(level-5)top longitudinal	#3@12in. c/c
(level-5)top transverse	#3@12in. c/c

Comparison Between Design Alternatives with No Settlement

Three design alternatives are presented in this report: (1) Constant Thickness Design, (2) Deep Beam Design, and (3) Embedded Beam Design.

All alternatives were analyzed using the FE model described earlier. It was observed that the critical elements for all three models, without any settlement conditions, were located at 46 feet from the asphalt pavement side and 2.5 feet from the bottom side as illustrated in Figure 39.

After finding the truck load and the location that creates the first crack in the slab, that truck location will be fixed, and its load will be incrementally increased until failure using the RIKS command in ABAQUS. The elements under the highest stresses, compression, and tension and which deflected the most were monitored and their graphs plotted. Figures 40 and 41 show the contour plot of stresses.

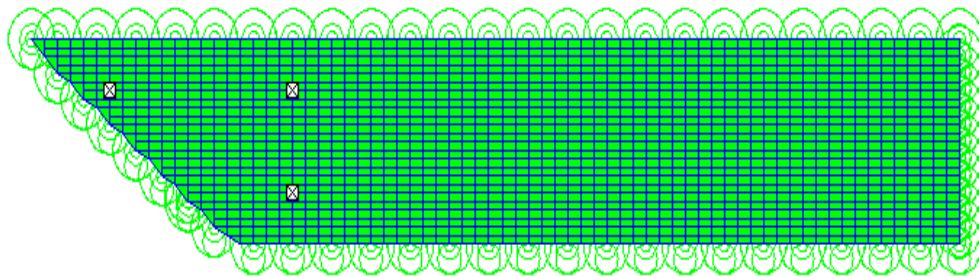


Figure 39. Truck location for first cracked element.

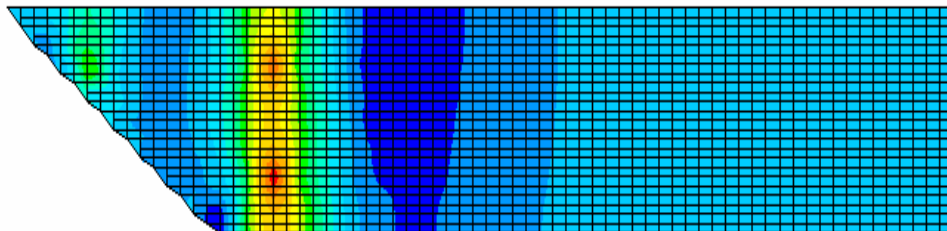


Figure 40. Bottom layer of approach slab under tension at truck's wheel loads

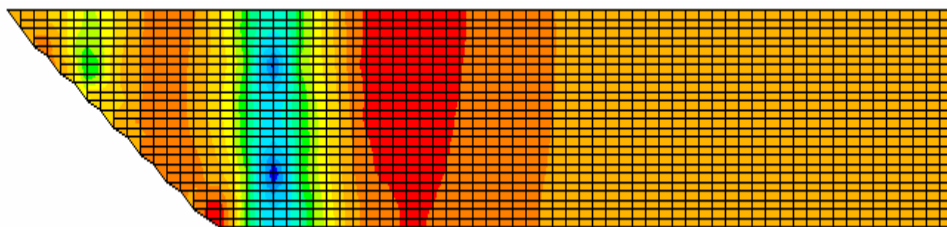


Figure 41. Top layer of approach slab under compression at truck's wheel loads

Figure 42 below graphs the tension and compression stresses variation for several design alternatives. For the existing design, the load required for the first element to crack is 1.66 HS20, whereas for the constant thickness design, deep beam design, and embedded beam design, it is 4.7 HS20. Since the thickness of the slab is the most contributing factor for the load carrying capacity of approach slabs, the existing design slab, which exhibits minimum thickness compared to other design alternatives, a minimum first cracking load. The deep beam design alternative has less amount of steel than the constant thickness and the embedded beam design alternatives, and, therefore, it fails at lower HS20 loads than the constant thickness and the embedded beam alternatives do.

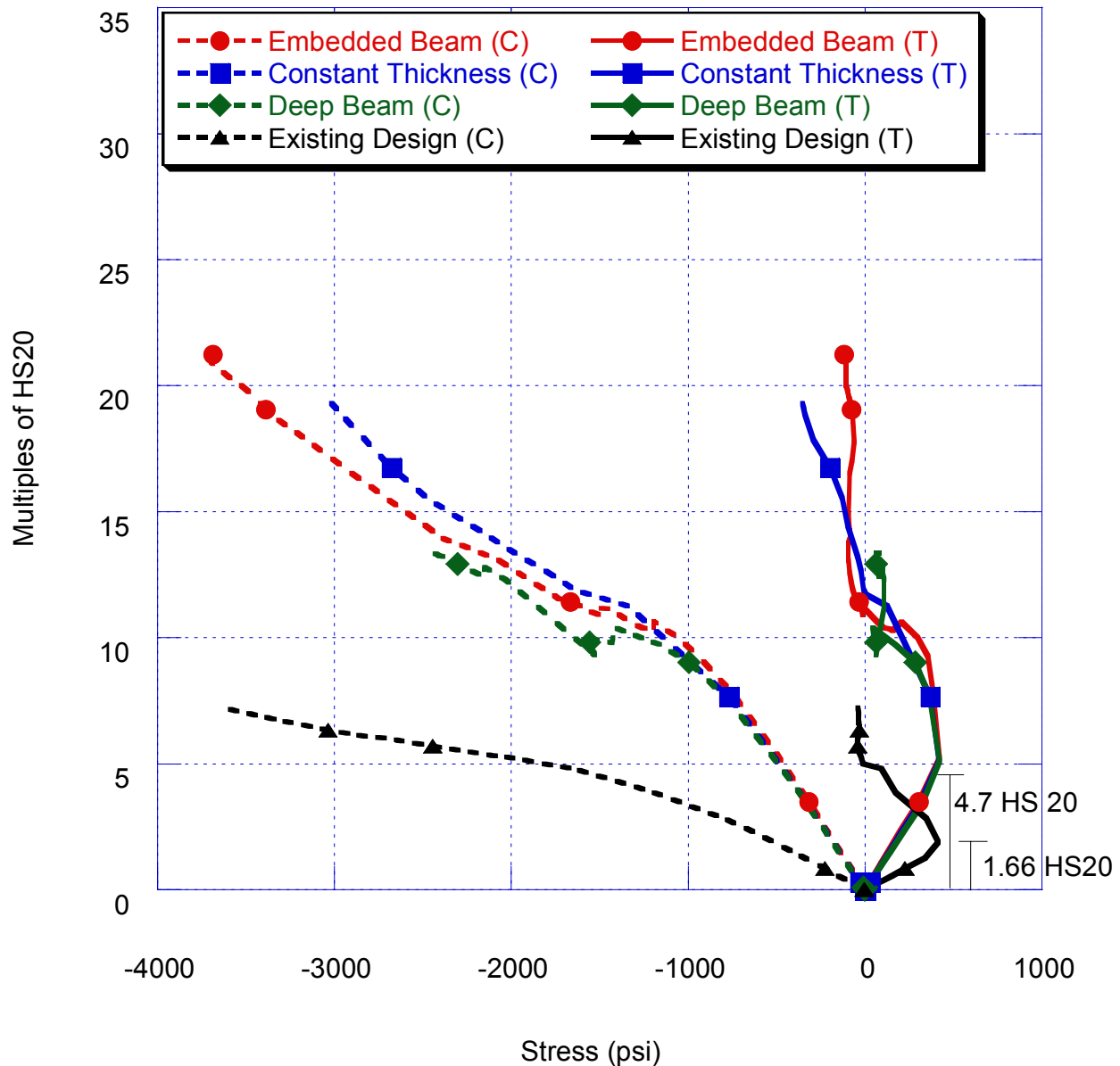


Figure 42. Load versus stress for all design options as compared to the existing design.

Figure 43 below shows a comparison of the load deflection relationship in various design alternatives. It is observed that the slab with Embedded Beam Design reaches a maximum deflection of 0.24 inches at a load of 24 HS20, the slab with Constant thickness Design reaches a maximum deflection of 0.17 inches at a load of 19 HS20, the slab with Deep Beam Design reaches a maximum deflection of 0.13 inches at a load of 13 HS20, and the slab with Existing Design reaches a maximum deflection of 0.16 inches at load of 7 HS20. Thus, the slab with Embedded Beam Design can withstand higher ultimate loads as well as deflections in comparison to slabs with other designs. This capacity to endure higher loads under severe settlement conditions is extremely important in the post-cracking behavior of approach slabs.

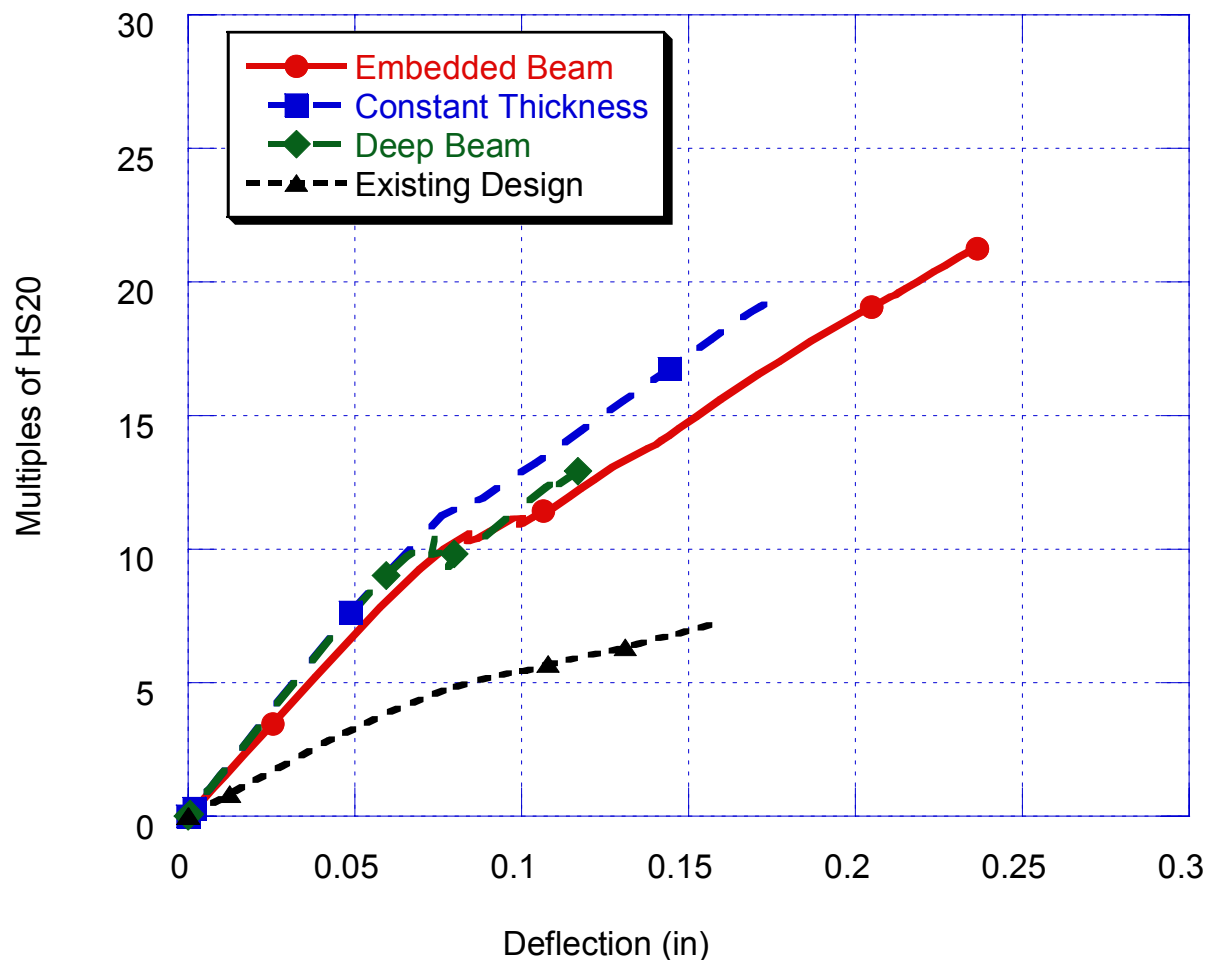


Figure 43. Load versus deflection for existing, constant thickness, deep beam and embedded beam designs.

Comparison Between Design Alternatives with Settlement

Using the FE model developed earlier and removing 30% of the springs in the middle vicinity of the approach slab results in the reduction of the ultimate load carrying

capacity of approach and transition slabs. Table 11 lists a 10% drop in the ultimate load capacity for the existing design and a 17% drop for both constant thickness and embedded beam designs. The 10% drop for the existing slab is due to the area of which the springs were removed, which was at the intersection of the transition slab with the approach slab. Even though the drop is higher for the alternative designs, the ultimate load is 2.5 times greater than that of the existing design. Thus it could be concluded that even with relatively high settlement, the alternative designs can sustain the minimum required load capacity.

Table 11. No settlement versus 30% settlement of soil underneath approach slab.

Design Alternative	No Settlement (Load Multiplier x HS20)	30% Settlement (Load Multiplier x HS20)	Percent Reduction
Existing	1.66	1.5	10 %
Constant Thickness	4.6	3.8	17%
Embedded Beam	4.7	3.9	17%

SUMMARY

Review of design practices revealed that most DOT(s) use concrete approach slabs on their primary highway systems. For the last two decades, NJDOT is facing a persistent problem of the cracking of bridge approach slabs. There is a need to address the problem using a more comprehensive and rational approach.

The field survey conducted disclosed that there are two major categories of cracking: 1) cracks developing in all lanes mainly due to settlement and/or void development underneath the approach slab and (2) cracks in heavily loaded lanes, mainly due to heavy vehicular live load or overloading due to an extreme event.

A Finite Element model of the existing NJDOT approach and transition slab (Appendix B) was developed. The approach and transition slabs were modeled as a trapezoidal shape having the length of the shorter side as 55 ft and the longer side as 71 ft, for a typical traffic lane of a 12 ft width. The finite element model showed that the first crack development in approach and transition slabs occurred at a load of 1.66 HS20. From the parametric study, it was also found that the thickness of the concrete slab plays a major role in the load carrying capacity of the approach and transition slab. However, the model also indicated that other factors such as settlement, void development, and embankment bulging could contribute to intensifying the problem. Moreover the data collected during the field surveys conducted on various bridges in NJ are consistent with those predicted by finite element modeling.

Three Design alternatives were suggested for approach and transition slabs: 1) Constant Thickness Design, 2) Deep Beam Design, and 3) Embedded Beam Design. Finite Element modeling for all three alternatives was done, and it was found from

analysis that the load required for the first element to crack increased from 1.66 HS20 to almost 4.7 HS20.

CONCLUSIONS

From the parametric study, we may conclude the following:

- (1) Increasing the concrete compressive strength, f_c , increases the cracking load capacity. However, it is not very effective in comparison with an equivalent increase in the thickness of the slab.
- (2) Increasing the steel reinforcement yielding stress, F_y , has no effect on neither the cracking load capacity of the approach and transition slab nor the stresses in the critical elements. The effect of the steel rebar is limited to post cracking which is manifested in the bond between the steel rebar and concrete. This bond is represented by tension stiffening in the finite element modeling.
- (3) Increasing the approach and transition slab thickness results in an effective increase in the cracking load capacity.
- (4) Increasing soil settlement decreases the cracking load capacity of the slab.
- (5) Among the three design alternatives suggested, the Deep Beam Design alternative has less of a load carrying capacity, and it takes less deflection compared to the other two design alternatives. Also the laying of reinforcement is difficult in practicality. Hence, going for both the alternatives, Constant Thickness Design and Embedded Beam Design is the best option.

Crack development is not referenced to a single cause or a particular mechanism, but it is a result of the cumulative effects of heavy traffic loading, settlement, void development under the approach slab, erosion of the approach embankment, and embankment bulging. Therefore, settlement and void development coupled with heavy truckloads are among the most probable factors causing crack development.

RECOMMENDATIONS

Based on the results of the analytical modeling and field observations, three design alternatives were developed:

- (1) Constant Thickness Design
- (2) Embedded Beam Design
- (3) Deep Beam Design

Through the parametric study, it was concluded that all three alternatives perform better than the existing design. It was also shown that under the most critical scenario [i.e., heavy truck loads coupled with severe (40%) soil settlement], all three design alternatives could withstand at least 2.5 times the loading carried by the existing design.

Therefore, it is suggested that the following measure be taken for the design of approach slabs:

- (1) Adopt Embedded Beam Design and/or Constant Thickness Design for an approach slab with a total thickness of 18 inches and steel reinforcement as shown in Figure 34 and Figure 38 respectively.
- (2) Use concrete compressive strength f'_c of 6500 psi in order to minimize crack development in the approach and transition slab.
- (3) Proper compaction of the backfill material below the approach slab is extremely important in minimizing settlement.
- (4) Require stage construction of the embankment in order to take advantage of the gain in shear strength that accompanies consolidation.
- (5) Preload the foundation by surcharging in order to mitigate the foundation soil settlement.
- (6) Extend the length of the wing wall beyond the current practice of 25-30 feet to minimize embankment bulging.

FUTURE RESEARCH

1. Construct actual approach slabs using both design alternatives presented earlier, namely: 1) Embedded Beam Design and 2) Constant Thickness Design.
2. Instrument and monitor newly designed/constructed approach slabs and collect data on soil settlement, strains in concrete and rebars, as well as truck loads using a WIM system. For example, the sensitive areas of a two-lane bridge where cracks are expected to first form according to FEM analysis are mapped in Figure 44. Therefore, sensors should be placed mainly in these areas.
3. Perform further detailed analysis of the soil-structure interaction using the appropriate soil model instead of springs.
4. Compare experimental data from field tests and analytical results from the FE model(s).
5. The Doremus Avenue bridge site represents an ideal location for applying steps 1 through 4 described above. The site is also equipped with a fixed WIM site for collecting truckload data on all lanes. The correlation of truckloads and slab response is essential for validating the new design recommendations.
6. Develop a questionnaire for all 50 states about current practice and any in-house design alternatives that are used for approach and transition slabs.

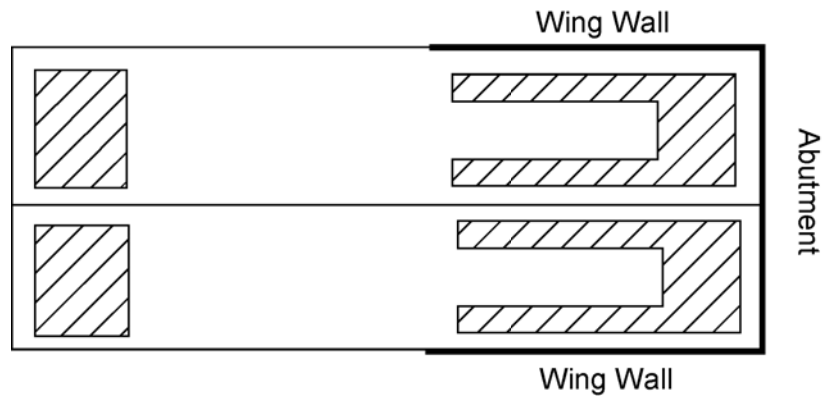


Figure 44. Critical areas at approach slab for sensor locations.

APPENDIX A

Figure A1 shows the schematic location of the cracked approach slab for Bridge 5. Figures A2-A5 show the actual locations of its cracks. Similarly, Figure A6 shows the schematic location of the cracked approach slab for Bridge 12. Figures A7-A9 show the actual locations of these cracks. Furthermore, Figure A10 shows the schematic location of the cracked approach slab for Bridge 6. Figures A11 and A12 show the actual locations of its cracks. Similarly, Figure A13 shows the schematic location of the cracked approach slab for Bridge 19. Figures A14-A16 show the actual locations of its cracks.

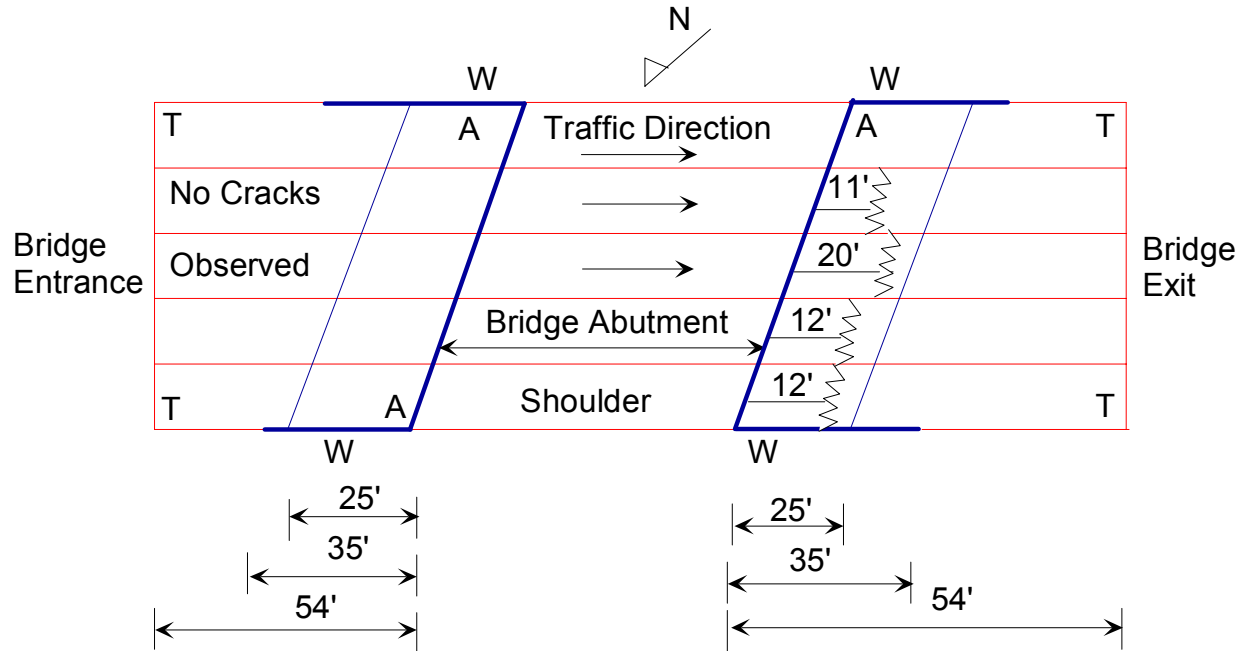


Figure 45. Transverse cracks on bridge 5.

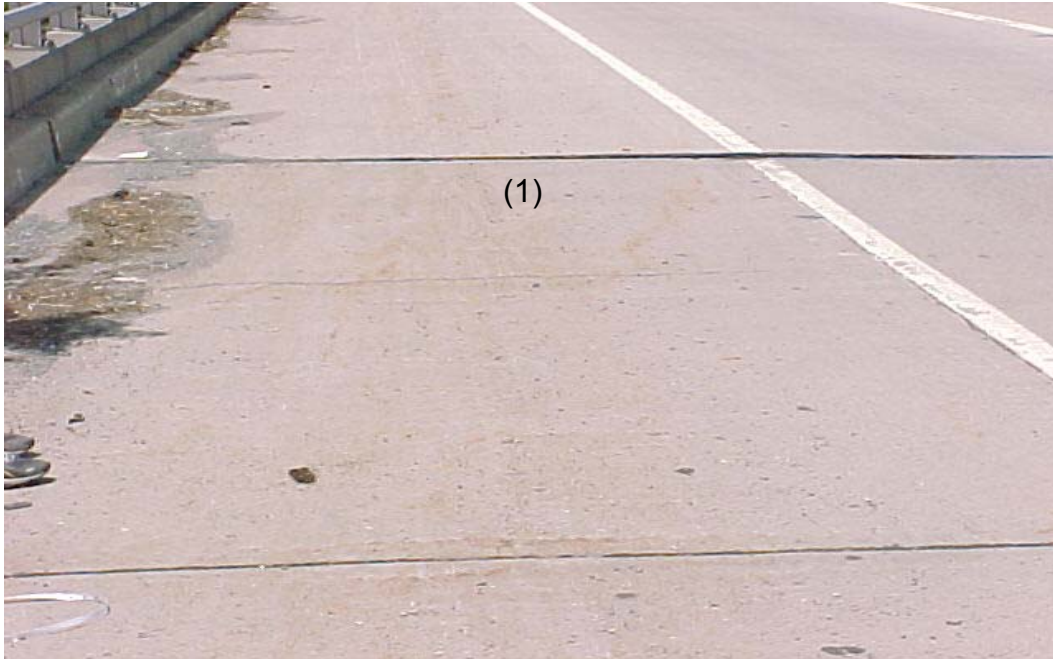


Figure 46. Photograph of a transverse crack in shoulder, at exit on bridge 5.

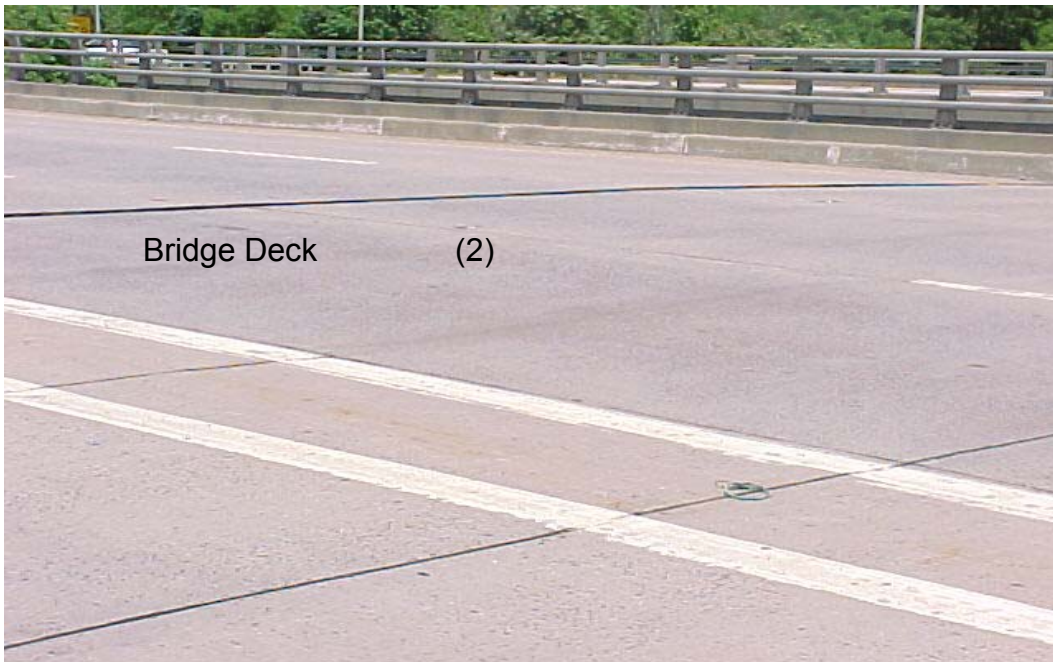


Figure 47. Photograph of a transverse crack in right lane, at exit on bridge 5.

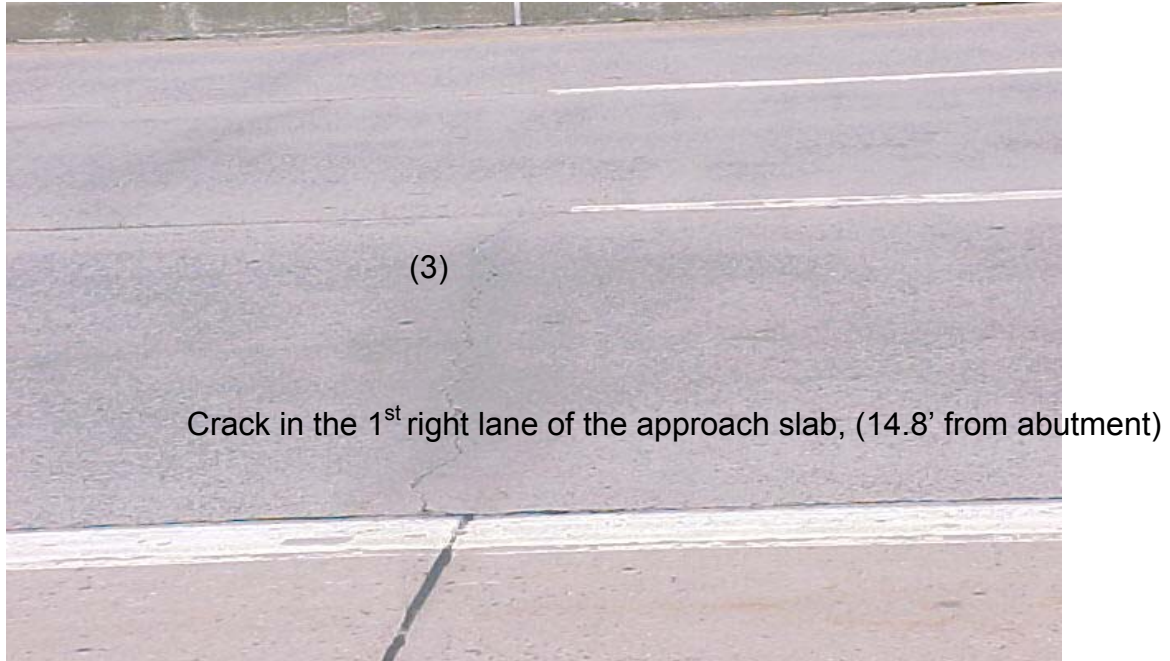


Figure 48. Photograph of a transverse crack in middle lane, at exit on bridge 5.

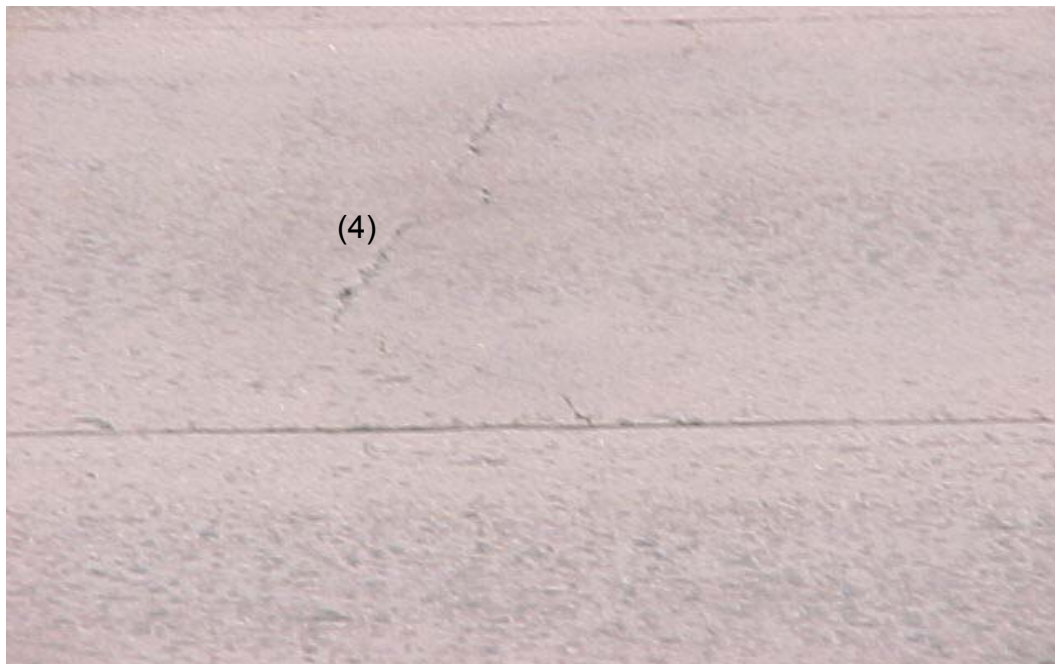


Figure 49. Photograph of a transverse crack in left lane, at exit on bridge 5.

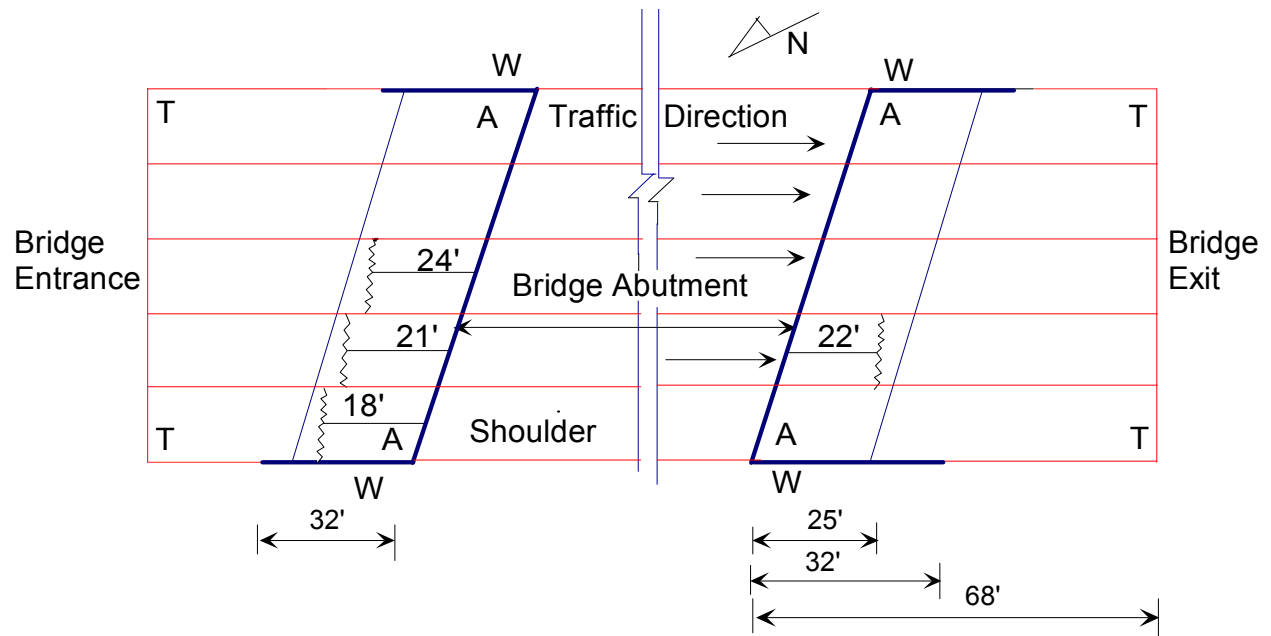


Figure 50. Transverse crack development on bridge 6.

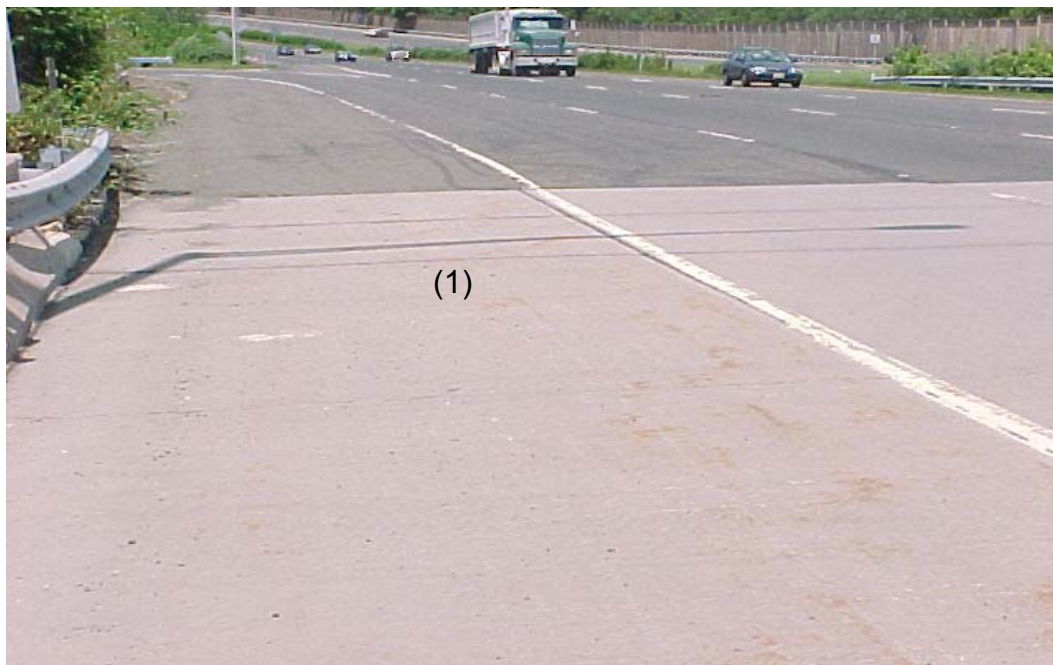


Figure 51. Photograph of a transverse crack in shoulder, bridge exit on bridge 6.

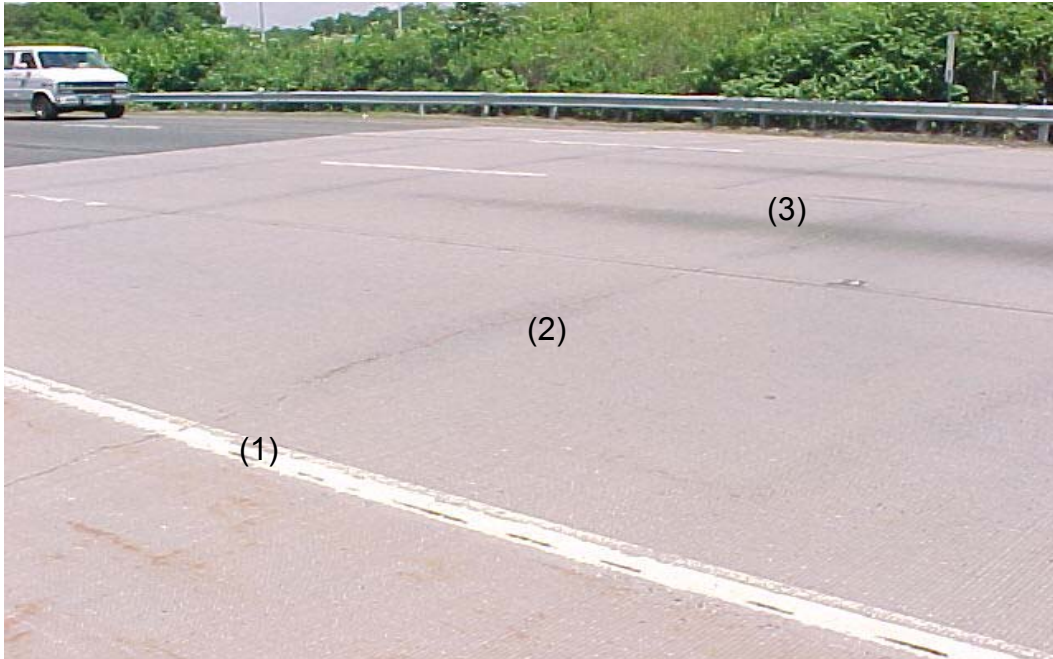


Figure 52. Photograph of a transverse crack in right and middle lanes, at entrance on bridge 6.

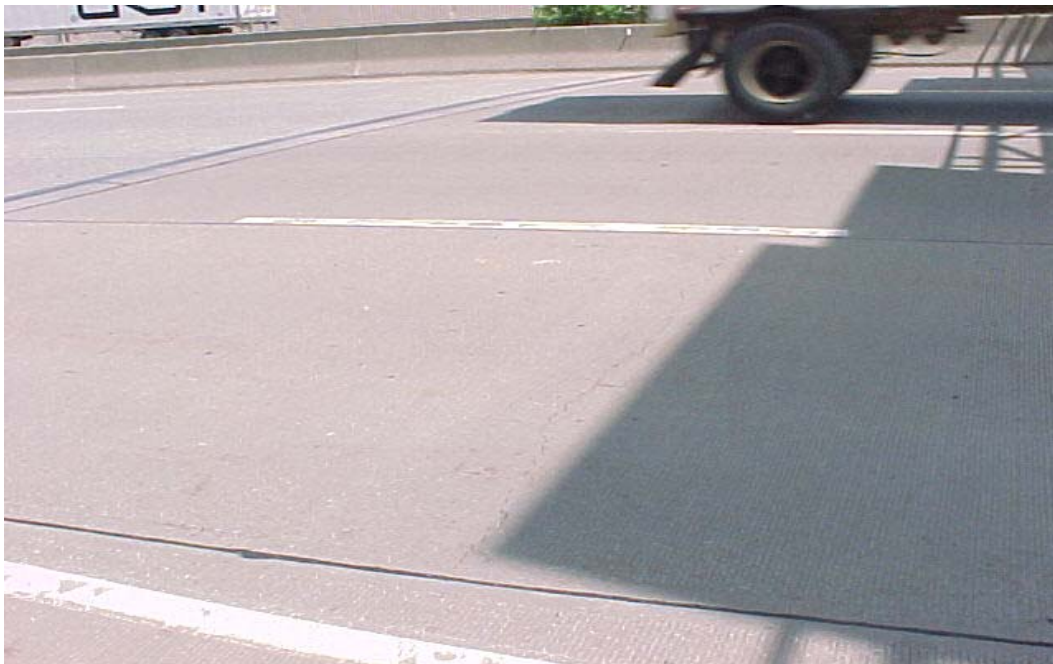
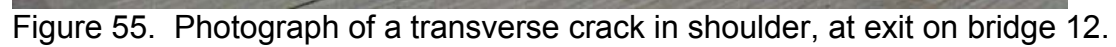


Figure 53. Photograph of a transverse crack in right lane, at exit on bridge 6.



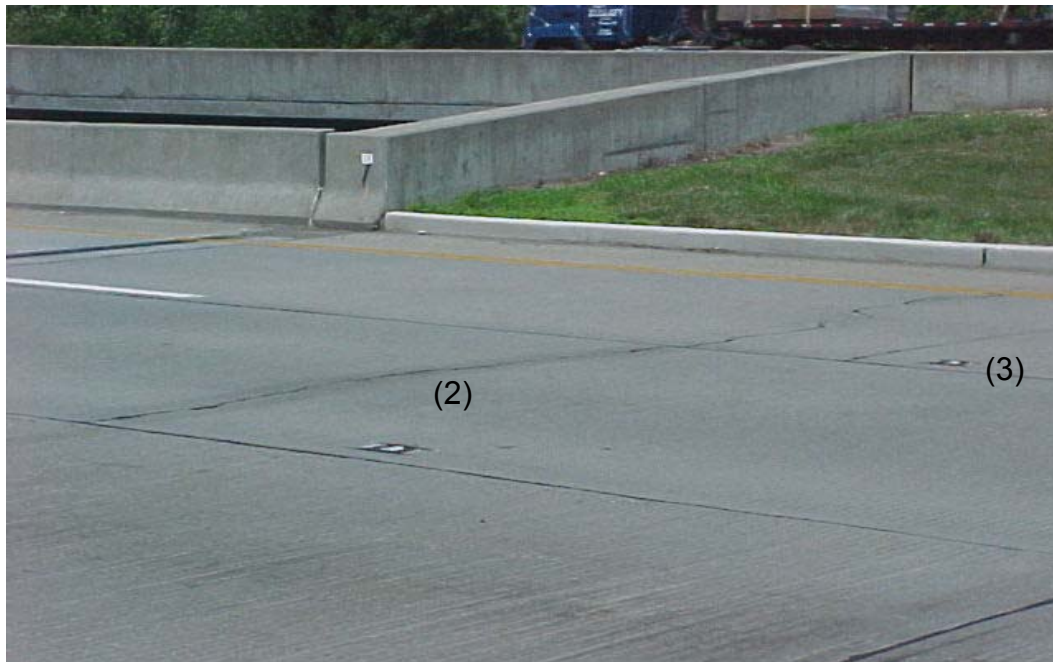


Figure 56. Photograph of transverse crack in middle and left lanes, bridge 12.

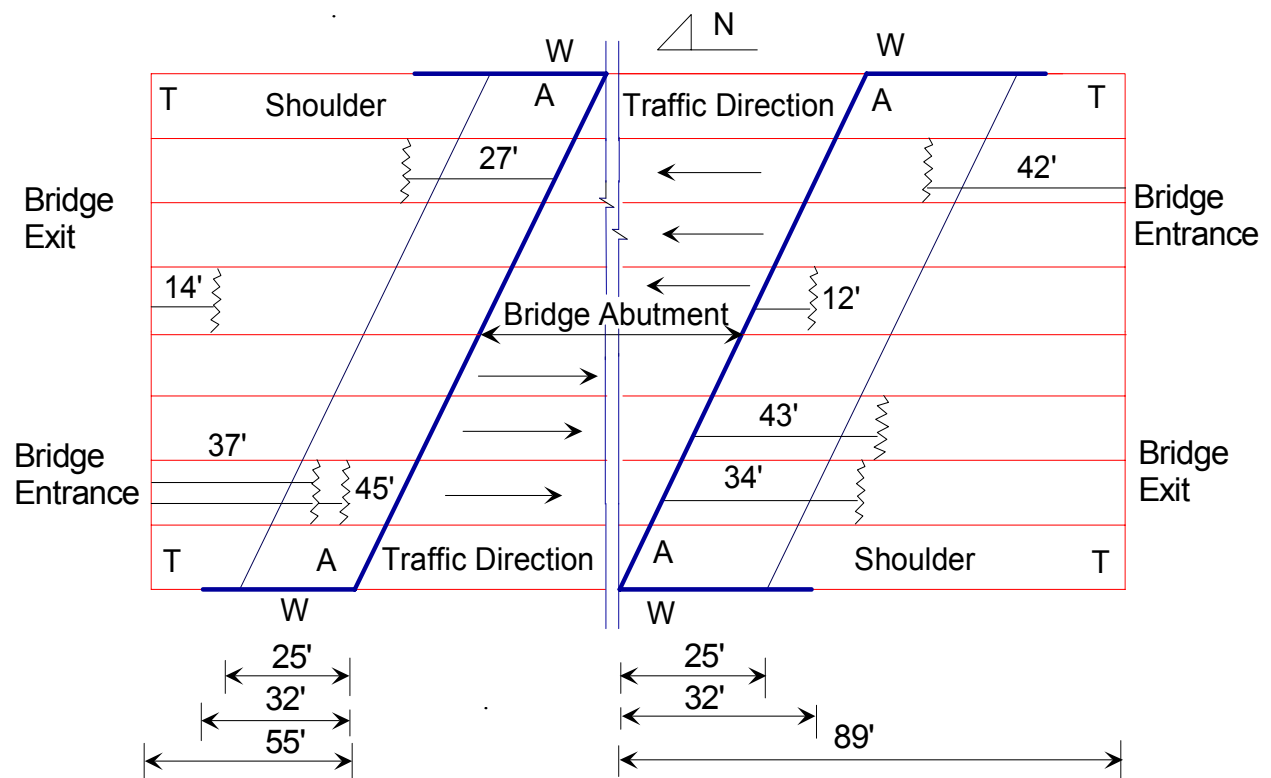


Figure 57. Transverse crack development on bridge 14.

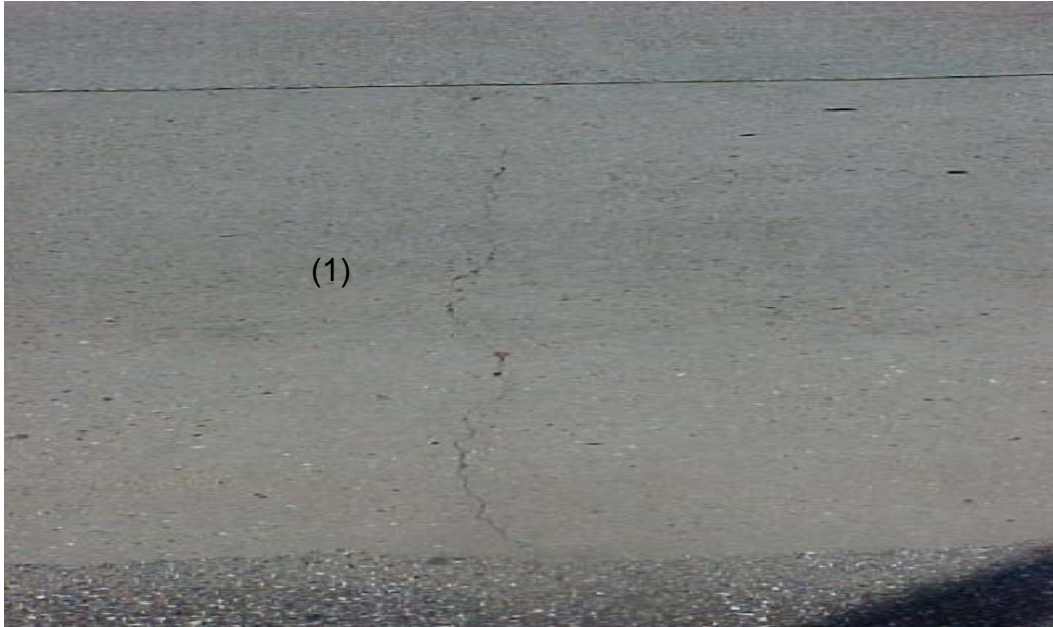


Figure 58. Photograph of a transverse crack on bridge 14.

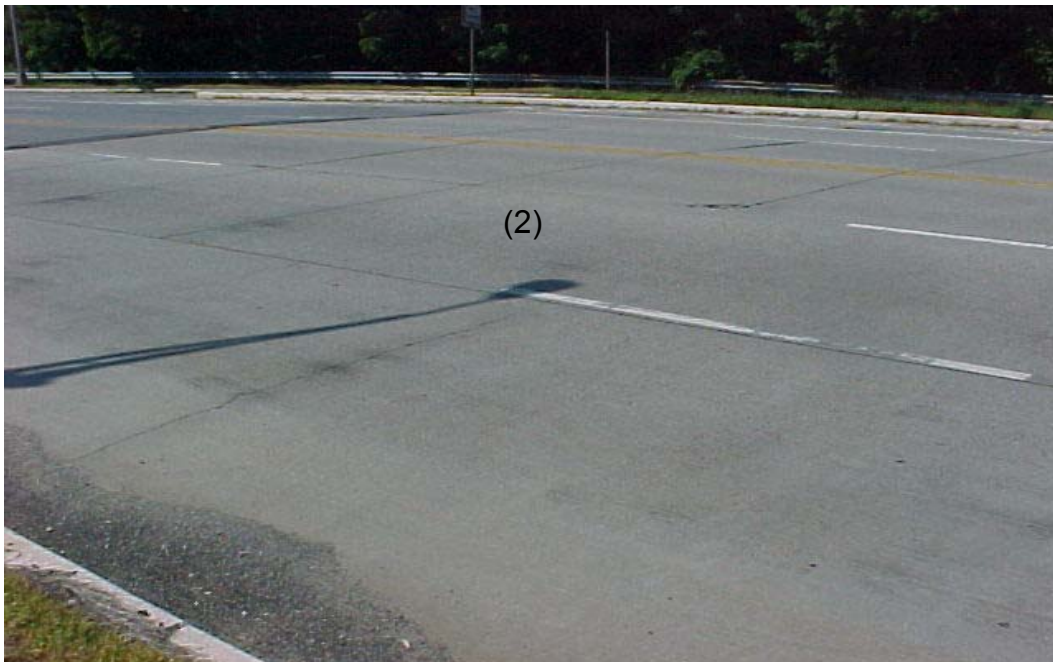


Figure 59. Photograph of a transverse crack in right lane, at entrance, on bridge 14.



Figure 60. Photograph of a transverse crack in right lane, at exit on bridge 14.

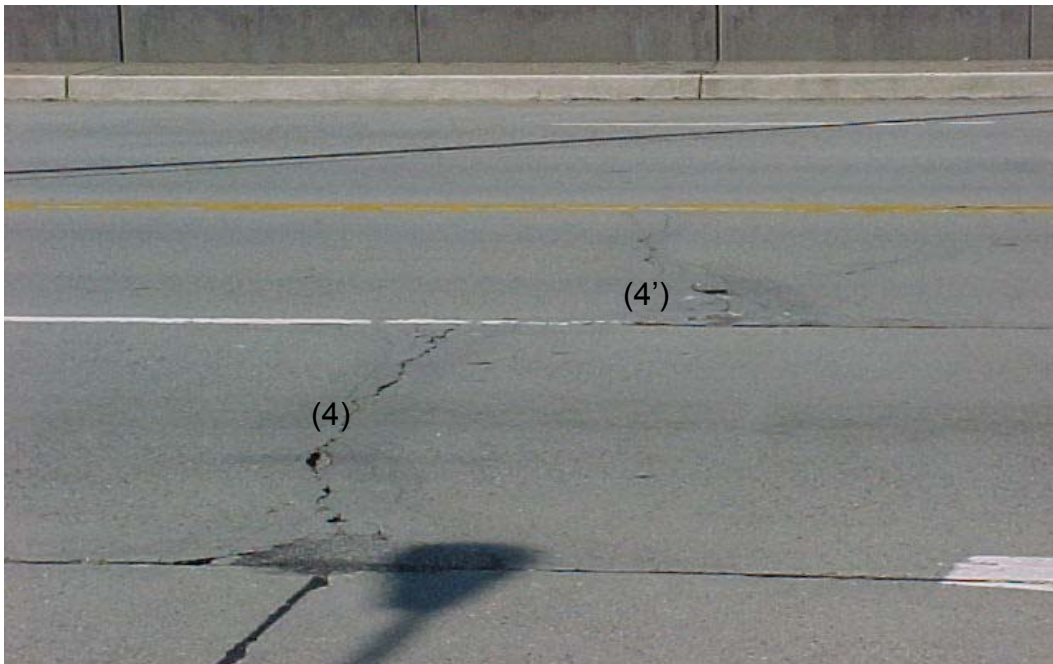


Figure 61. Photograph of a transverse cracks in middle and left lanes, at exit, bridge 14.

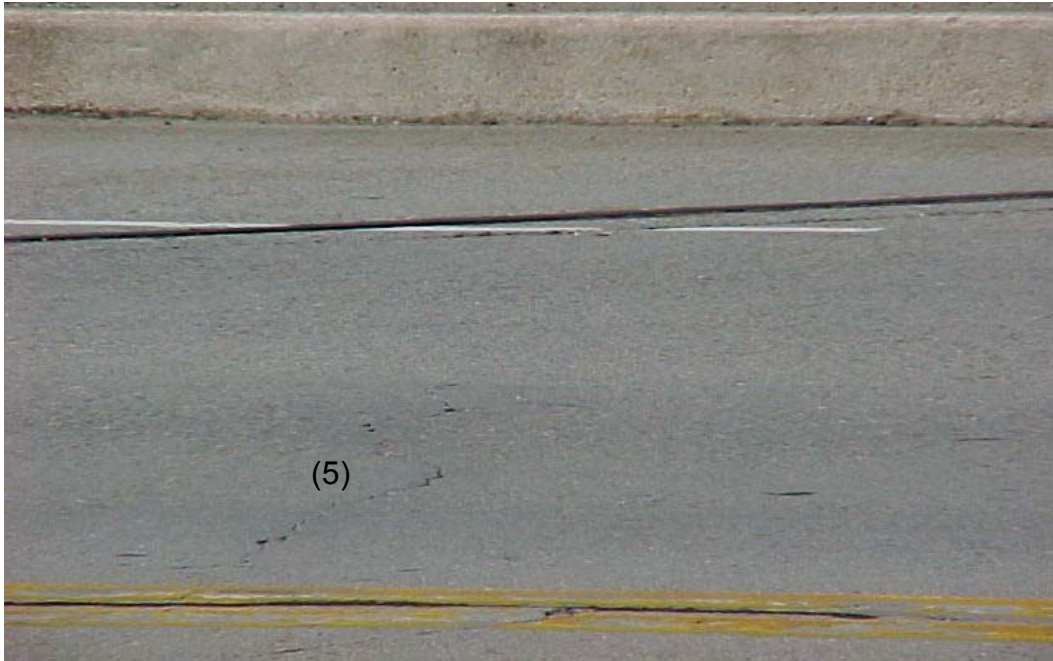


Figure 62. Photograph of a transverse cracks in right lane on bridge 14.

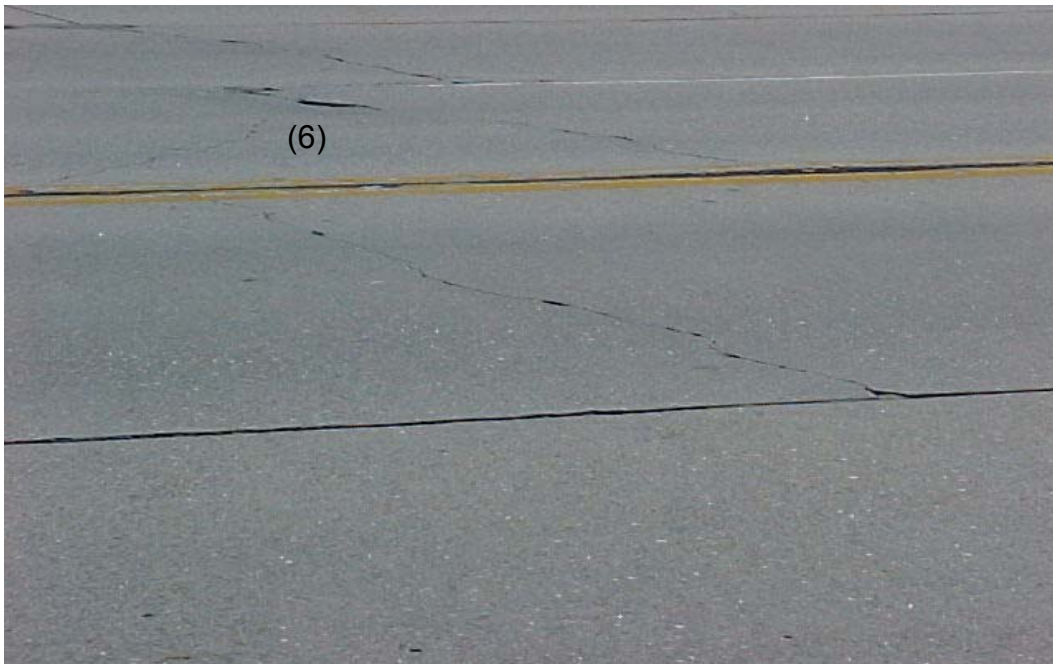


Figure 63. Photograph of transverse cracks in right lane, at entrance on bridge 14.



Figure 64. Photograph of a transverse crack in right lane, at exit on bridge # 14.

APPENDIX B

Details of the existing design of the approach slab, as provided by DOT, are shown in Figure B1 and Figure B2.

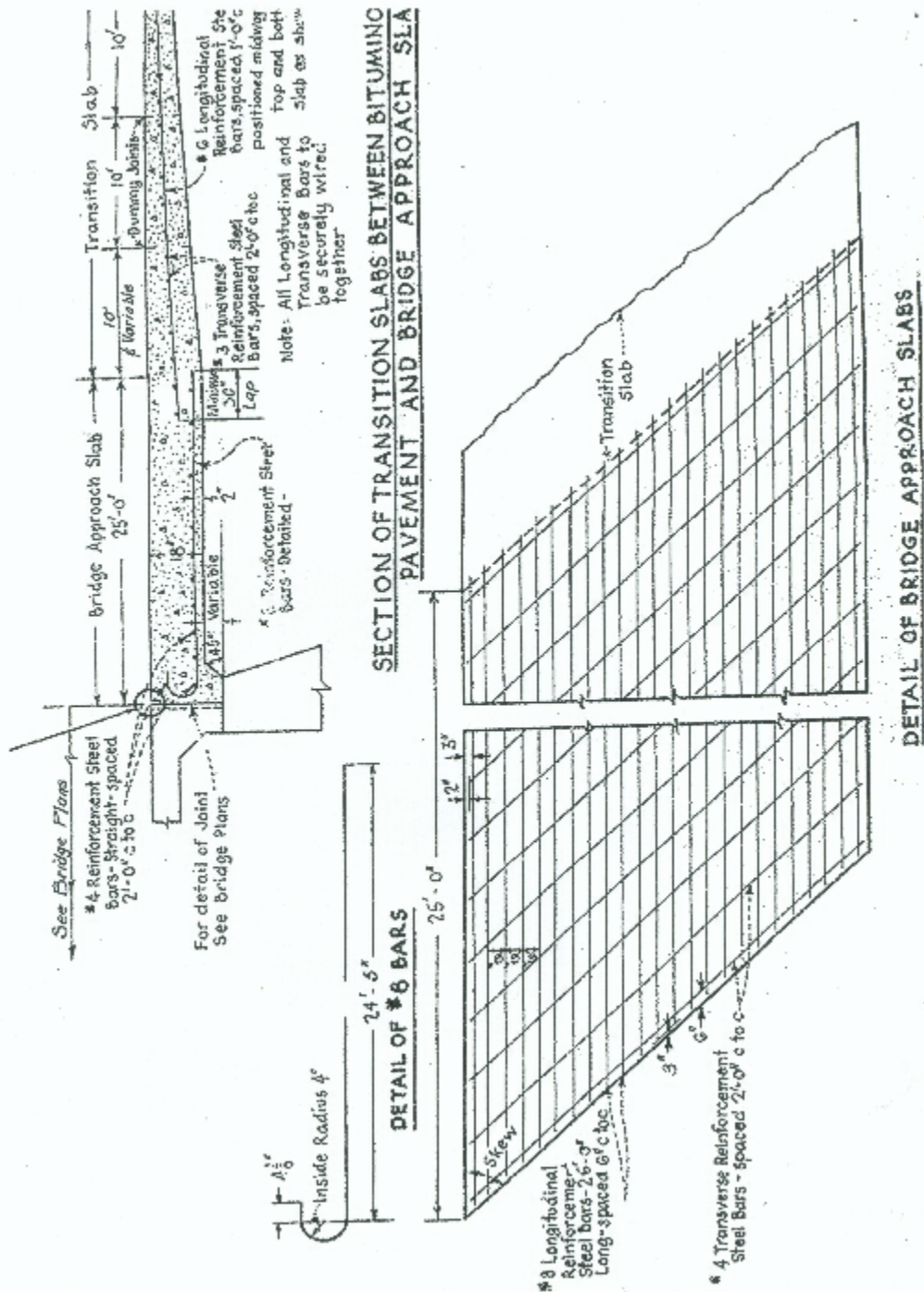


Figure 65. Detail of existing design of approach slabs.

REFERENCES

1. Allen, D.L, *A Survey of the States on Problems Related to Bridge Approaches*: Research Report UKTrP-85-25, Kentucky Transportation Research Program, October 1985.
2. Ardani, A. *Bridge Approach Settlement*: Report No. CDOH-DTP-R-87-6, Colorado Department of Highways, April 1987.
3. Bathe, Klaus J and Wilson, Edward L.: *Numerical Methods in Finite Element Analysis*. Prentice Hall Inc., 1976.
4. Bowels, Joseph E.: *Foundation Analysis and Design. Fifth Edition*, McGraw-Hill, Inc., 1996.
5. Briaud, Jean-Louis, James, Ray W and Hoffman, Stacey B. TRB, NCHRP Synthesis of Highway Practice 234: Settlement of Bridge Approaches "Bump at the End of the Bridge." National Academy Press, Washington, D.C. 1997.
6. Burke M.P. "Integral Bridges". Transportation Research Record 1275, TRB National Research Council, Washington, D.C. (1990).
7. Burnett, David S.: *Finite Element Analysis: From Concepts to Applications*. Addison-Wiley Publishing Company, AT&T Bell Laboratories.
8. Chini, S. A., A.M. Wolde-Tinsae, and M.S. Aggour. "Drainage and Backfill Provisions for Approaches to Bridges". Transportation Research Record 1425, TRB National Research Council, Washington. D.C. (1993).
9. Clough, G.W. and Duncan, J.M. Earth Pressures, Foundation Engineering Handbook, Second Edition, edited by H-Y. Fang, Van Nostrand Reinhold, New York, 1991.
10. Gere, James M and Timoshenko, Stephen P.: *Mechanics of Materials*. PWS Publishing Company, 1997.
11. Holtz, Robert D. Synthesis of Highway Practice: "Treatment of Problem Foundations for Highway Embankments." Transportation Research Board, National Research Council, Washington D.C, July 1989.
12. Hopkins, T.C., and Deen, R.C., *The Bump at the End of the Bridge*: Highway Research Record No. 302, Highway Research Board, National Research Council, Washington, D.C., 1970, PP. 72-75.
13. Hopkins, T.C., *Settlement of Highway Bridge Approaches and Embankment Foundations*: Interim Report KYHPR-64-17, Kentucky Department of Highways, February 1969.
14. Hoppe, E. J.. *Guidelines for the Use, Design, and Construction of Bridge Approach Slabs*. Virginia Department of Transportation and the University of Virginia, November 1999.
15. Huebner, Kenneth H.: *The Finite Element Method for Engineers*. John Wiley & Sons Inc, 1975.
16. Kramer, Steven L and Sajer, Peter. *Bridge Approach Slab Effectiveness*. : Final Report, Washington State Department of Transportation, December 1991
17. MacGregor, James G.: *Reinforced Concrete Mechanisms and Design*. Third Edition, Prentice Hall International, New Jersey 2000.

18. Nassif, H. H. and Nowak, A.S. "Dynamic Load Spectra For Girder Bridges." *Journal of the Transportation Research Board*, No. 1476, TRB, National Research Council, Washington, D.C., 1995, pp 69-83.
19. Nawy, Edward G.: *Reinforced Concrete A Fundamental Approach*. Fourth Edition, Prentice Hall International, New Jersey 2000.
20. NCHRP Synthesis of Highway Practice 2: "Bridge Approach Design and Construction Practices." Transportation Research Board, National Research Council, Washington, D.C, 1969.
21. NCHRP Synthesis of Highway Practice 8: "Construction of Embankments." Transportation Research Board, National Research Council, Washington, D.C, 1971.
22. NCHRP. *Construction and Maintenance of PPC Pavement Joints*: National Cooperative Highway Research Program Synthesis of Highway Practice No. 19, Highway Research Board, National Research Council, Washington, D.C, 1973.
23. NCHRP, Synthesis of Highway Practice 159, "Design and Construction of Bridge Approaches." Transportation Research Board, National Research Council, Washington, D.C, 1990.
24. Schaefer, Vernon R and Koch, Jay C. Void Development under Bridge Approaches: Final Report No SD90-03. South Dakota Department of Transportation, November 1992.
25. Timmerman, D.H. *An Evaluation of Bridge Approach Design and Construction Techniques*: Final Report to Ohio Department of Transportation, Report No. OHIO-DOT-03-77, December 1976.
26. Timoshenko, S.P and Goodier, J.N.: *Theory of Elasticity*. Third Edition, McGraw-Hill, Inc., 1970.
27. Wahls, H.E. *Design and Construction of Bridge Approaches*. NCHRP Synthesis of Highway Practice 159. Transportation Research Board, Washington, DC.. 1990
28. Wolde-Tinsae, A.M, Aggour, S.M, and Chini, S.A. *Structural and Soil Provisions for Approaches to Bridges*: Interim Report AW087-321-046, Maryland Department of Transportation, July 1987.
29. Wolde-Tinsae, A.M., Greiman, L.F. and Johnson B., "Performance of Integral Bridge Abutments," *Journal of the International Association for Bridge and Structural Engineering*, IABSE Periodical 1/1983, February 1983.